

An ACI Handbook The Reinforced Concrete Design Handbook A Companion to ACI 318-14 Volume 1: Member Design SP-17(14) THE REINFORCED CONCRETE DESIGN HANDBOOK A Companion to ACI 318-14 VOLUME 2 VOLUME 1 BUILDING EXAMPLE RETAINING WALLS STRUCTURAL SYSTEMS SERVICEABILITY STRUCTURAL ANALYSIS STRUT-AND-TIE MODEL DURABILITY ANCHORING TO CONCRETE ONE-WAY SLABS TWO-WAY SLAB Volume 1 THE REINFORCED CONCRETE DESIGN HANDBOOK A Companion to ACI 318-14 Editors: Andrew Taylor Trey Hamilton III Antonio Nanni First Printing September 2, 2016 THE REINFORCED CONCRETE DESIGN HANDBOOK Volume 1 ~ Ninth Edition Copyright by the American Concrete Institute, Farmington Hills, MI. All rights reserved. This material may not be reproduced or copied, in whole or part, in any printed, mechanical, electronic, film, or other distribution and storage media, without the written consent of ACI. 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Fergusson Manager, Publishing Services: Barry Bergin Lead Production Editor: Carl Bischof Production Editors: Kelli Slayden, Kaitlyn Hinman, Tiesha Elam Graphic Designers: Ryan Jay, Aimee Kahaian Manufacturing: Marie Fuller www.concrete.org DEDICATION This edition of The Reinforced Concrete Design Handbook, SP-17(14), is dedicated to the memory of Daniel W. Falconer and his many contributions to the concrete industry. He was Managing Director of Engineering for the American Concrete Institute from 1998 until his death in July 2015. Dan was instrumental in the reorganization of Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14) as he served as ACI staff liaison to ACI Committee 318, Structural Concrete Building Code; and ACI Subcommittee 318-SC, Steering Committee. His vision was to simplify the use of the Code for pracWLWLRQHUV DQG WR LOOXVWUDWH WKH EHQH¿WV RI WKH UHRUJDQL]DWLRQ ZLWK WKLV PDMRU UHYLVLRQ RI SP-17. His oversight and review comments were instrumental in the development of this Handbook. An ACI member since 1982, Dan served on ACI Committees 344, Circular Prestressed Concrete Structures, and 373, Circular Concrete Structures Prestressed with Circumferential Tendons. He was also a member of the American Society of Civil Engineers. Prior to MRLQLQJ\$&, DQKHOGVHYHUDOHQJLQHHULQJDQGPDUNHWLQJSRVLWLRQVZLWK96/&RUS%HIRUH WKDWKHZDV3URMHFW(QJLQHHUIRU6NLGPRUH2ZLQJVDQG0HUULOOLQ:DVKLQJWRQ'&+H received his BS in civil engineering from the University of Buffalo, Buffalo, NY and his 06LQFLYLODQGVWUXFWXUDOHQJLQHHULQJIURP/HKLJK8QLYHUVLW\%HWKOHKHP3\$+HZDVD licensed professional engineer in several states. ,QKLVSHUVRQDOOLIH'DQZDVDQDYLGJROIHUHQMR\LQJRXWLQJVZLWKKLVWKUHHEURWKHUVZKHQHYHUSRVVLEOH+HZDVDOVRDQDFWLYH PHPEHURI2XU6DYLRU/XWKHUDQ&KXUFKLQ+DUWODQG0,DQGDGHGLFDWHGVXSSRUWHUDQGIROORZHURIWKH0LFKLJDQ6WDWH6SDUtans basketball and football programs. Above all, Dan was known as a devoted family man dedicated to his wife of 33 years, Barbara, his children Mark, Elizabeth, Kathryn, and Jonathan, and two grandsons Samuel and Jacob. In his memory, the ACI Foundation has established an educational memorial. For more information visit . Dan will be sorely missed for many years to come. FOREWORD The Reinforced Concrete Design Handbook provides assistance to professionals engaged in the design of reinforced concrete EXLOGLQJVDQGUHODWHGVWUXFWXUHV7KLVHGLWLRQVKDWEULQJVLWXSWRGDWHZLWKWKHDSSURDFKDQGSURYLVLRQVKI Building Code Requirements for Structural Concrete (ACI 318-14). The layout and look of the Handbook have also been updated. The Reinforced Concrete Design Handbook now provides dozens of design examples of various reinforced concrete members, such as one- and two-way slabs, beams, columns, walls, diaphragms, footings, and retaining walls. For consistency, many of the QXPHULFDOH[DPSOHVDUHEDVHGRQD¿FWLWLRXVVHYHQVWRU\UHLQIRUFHGFRQFUHWHEXLOGLQJ7KHUHDUHDOVRPDQ\DGGLWLRQDOGHVLJQ examples not related to the design of the members in the seven story building that illustrate various ACI 318-14 requirements. Each example starts with a problem statement, then provides a design solution in a three column format—code provision UHIHUHQFHVKRUWGLVFXVVLRQDQGGHVLJQFDOFXODWLRQV2IROORZHGE\DGUDZLQJRIUHLQIRUFLQJGHWDLOVDQG¿QDOO\DFRQFOXVLRQ elaborating on a certain condition or comparing results of similar problem solutions. In addition to examples, almost all chapters in the Reinforced Concrete Design Handbook contain a general discussion of the related ACI 318-14 chapter. All chapters were developed by ACI staff engineers under the auspices of the ACI Technical Activities Committee (TAC). 7R SURYLGH LPPHGLDWH RYHUVLJKW DQG JXLGDQFH IRU WKLV SURMHFW 7\$& DSSRLQWHG WKUHH FRQWHQW HGLWRUV\$QGUHZ 7D\ORU 7UH\ Hamilton III, and Antonio Nanni. Their reviews and suggestions improved this publication and are appreciated. TAC also appreciated the support of Dirk Bondy and Kenneth Bondy who provided free software to analyze and design the post-tensioned EHDP H[DPSOH LQ DGGLWLRQ WR YDOXDEOH FRPPHQWV DQG VXJJHVWLRQV 7KDQNV DOVR JR WR -R\$QQ %URZQLQJ 'DYLG 'H9DOYH \$QLQG\D'XWWD&KDUOHV'RODQ0DWWKHZ+XVOLJ5RQDOG.OHPHQFLF-DPHV/DL6WHYHQ0F&DEH0LNH0RWD+DQL1DVVLI-RVH Pincheira, David Rogowski, and Siamak Sattar, who reviewed one or more of the chapters. Special thanks go to StructurePoint and Computers and Structures, Inc. (SAP 2000 and Etabs) for providing a free copy of their software to perform analyses of structure and members. Special thanks also go to Stuart Nielsen, who provided the cover art using SketchUp. The Reinforced Concrete Design HandbookLVSXEOLVKHGLQWZRYROXPHV&KDSWHUVWKURXJKDUHSXEOLVKHGLQ9ROXPH DQG&KDSWHUVWKURXJKDUHSXEOLVKHGLQ9ROXPH DQG&KDSWHUVWKURXJKDUHSXEOLVKHGLQ9ROXPH HVLJQDLGVDQGDPRPHQWLQWHUDFWLRQGLDJUDP([FHOVSUHDGVKHHWDUH available for free download from the following ACI webpage links: Keywords: DQFKRULQJWRFRQFUHWHEHDPVFROXPQVFUDFNLQJGHAHFWLRQGLDSKUDJPGXUDELOLW\AH[XUDOVWUHQJWKIRRWLQJV frames; piles; pile caps; post-tensioning; punching shear; retaining wall; shear strength; seismic; slabs; splicing; stiffness; structural analysis; structural analysis; structural systems; structural analysis; structural analysis; structural analysis; structural analysis; structural analysis; structural systems; structural analysis; structural analysis VOLUME 1: CONTENTS CHAPTER 1—BUILDING EXAMPLE 1.1—Introduction, p. 9 1.2—Building plans and elevation, p. 9 2/RDGVS 1.4—Material properties, p. 12 CHAPTER 2—STRUCTURAL SYSTEMS 2.1—Introduction, p. 13 2.2—Materials, p. 13 2.3—Design loads, p. 13 2.4—Structural systems, p. 14 2.5—Floor subassemblies, p. 20 2.6— Foundation design considerations for lateral forces, p. 22 2.7—Structural analysis, p. 23 2.8—Durability, p. 23 2.9—Sustainability, p. 23 2.12—Post-tensioned/prestressed construction, p. 23 2.13—Quality assurance, construction, and inspection, p. 23 CHAPTER 3—STRUCTURAL ANALYSIS 3.1—Introduction, p. 25 <sup>2</sup>2YHUYLHZRIVWUXFWXUDODQDO\VLVS 3.3—Hand calculations, p. 26 3.4—Computer programs, p. 26 3.4 Concrete evaluation, acceptance, and inspection, p. 35 4.5—Examples, p. 35 CHAPTER 5—ONE-WAY SLABS 5.1—Introduction, p. 39 5.2—Analysis, p. 39 5.3—Service limits, p. 39 5.3—Service limits, p. 39 5.4—Required strength, p. 40 5.5—Design strength, p. 40 5.6—Flexure reinforcement detailing, p. 40 5.7—Examples, p. 42 CHAPTER 6—TWO-WAY SLABS 6.1— Introduction, p. 81 6.2—Analysis, p. 81 6.3—Service limits, p. 81 6.4—Shear
strength, p. 82 6.5—Calculation of required shear strength, p. 83 6.6—Calculation of shear reinforcement, p. 84 6.8—Shear reinforcement, p. 84 6.8—Shear reinforcement, p. 84 6.7—Flexural strength, p. 84 6.8—Calculation of shear strength, p. 83 6.6—Calculation of shear reinforcement, p. 84 6.8—Shear reinforcement, p. 84 6.8\\_Shear reinforcement, p. 84 6.8\\_Sh -Introduction, p. 133 7.2-Service limits, p. 133 7.3-Analysis, p. 134 7.4-Design strength, p. 134 7.5-Temperature and shrinkage reinforcement, p. 140 7.7-Examples, p. 143 CHAPTER 8-DIAPHRAGMS 8.1-Introduction, p. 281 8.2-Material, p. 281 8.3-Service limits, p. 281 8.4-Analysis, p. 281 8.5-Design strength, p. 140 7.6-Detailing, p. 140 7.7-Examples, p. 143 CHAPTER 8-DIAPHRAGMS 8.1-Introduction, p. 281 8.2-Material, p. 281 8.3-Service limits, p. 281 8.4-Analysis, p. 281 8.5-Design strength, p. 143 CHAPTER 8-DIAPHRAGMS 8.1-Introduction, p. 281 8.2-Material, p. 281 8.3-Service limits, p. 281 8.4-Analysis, p. 281 8.5-Design strength, p. 143 CHAPTER 8-DIAPHRAGMS 8.1-Introduction, p. 281 8.2-Material, p. 281 8.3-Service limits, p. 281 8.4-Analysis, p. 281 8.5-Design strength, p. 281 8.4-Analysis, p. 281 8.4-Analysis, p. 281 8.4-Analysis, p. 281 8.5-Design strength, p. 281 8.4-Analysis, p. 281 8.4-Analysis, p. 281 8.4-Analysis, p. 281 8.5-Design strength, p. 281 8.4-Analysis, p. 281 8.4-Ana 283 8.6—Reinforcement detailing, p. 284 8.7—Summary steps, p. 286 8.8—Examples, p. 289 CHAPTER 9—COLUMNS 9.1—Introduction, p. 353 9.3—Design limits, p. 353 9.4—Required strength, p. 354 9.5—Design strength, p. 356 9.6—Reinforcement limits, p. 357 9.7—Reinforcement detailing, p. 357 9.8—Design steps, p. 359 9.9—Examples, p. 362 CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 10.1—Introduction, p. 391 10.2—General, p. 394 10.5—Detailing, p. 398 10.6—Summary, p. 399 10.7—Examples, p. 400 CHAPTER 11—FOUNDATIONS 11.1—Introduction, p. 419 11.2—Footing design, p. 419 11.3—Design steps, p. 420 <sup>2</sup>)RRWLQJVVXEMHFWWRHFFHQWULFORDGLQJS 11.5—Combined footing, p. 423 11.6—Examples, p. 425 1.1—Introduction The building according to ACI 318-14. This example building is seven stories above ground and has a one story basement. The building has evenly spaced columns along the grid lines. One column has been removed along Grid C on the second level so that there is open space for the lobby. The building dimensions are: • Width (north/south) = 72 ft (5 bays @ 14 ft) • Length (east/west) = 218 ft (6 bays @ 36 ft) • Height (above ground) = 92 ft • Basement height = 10 ft The basement is used for storage, building services and mechanical equipment. It is ten feet high and has an extra column added in every bay along Grids A through F to support a two-way slab at the second level. There are basement walls at the perimeter. The structural system is an ordinary concrete shear wall in the north/south direction and an ordinary concrete moment frame in east/west direction. These basic systems were chosen as a starting point for the examples. Member examples may be expanded to show how they may be designed in intermediate or special systems but a new structural analysis is not done. The following analysis results provide the moments, shears, and axial loads given in the examples in other chapters in the manual. Those examples may modify this initial GDWDWRGHPRQVWUDWHVRPHVSHFL¿FFRGHUHTXLUHPHQW 1.2—Building plans and elevation The following building plans and elevation provide the illustration of the example building. American Concrete Institute - Copyrighted © Material - www.concrete.org Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING EXAMPLE 10 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Building Example CHAPTER 1-BUILDING Example CHAPTER 1-BUILDI EXAMPLE A-A American Concrete Institute - Copyrighted © Material - www.concrete.org 12 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 1.3—Loads for the example building are generated in accordance with ASCE7-10. The Risk Category is II. Gravity Loads Dead Load, D: • Self weight • Additional D = 15 lb/ft2 • Perimeter walls = 15 lb/ft2 Live Load: • 1st and 2nd Floors: Lobbies, public rooms, and corridors serving them = 65 lb/ft2 • Typical Floor: Private rooms and corridors serving them = 65 lb/ft2 Roof Live Load: • Unoccupied = 20 lb/ft2 • Thermal, Ct = 1.0 • Exposure, Ce = 1.0 • Importance, Is = 1.0 • Flat roof load, Pf = 20 lb/ft2 Lateral Loads Wind Load: • Basic (ultimate) wind speed = 115 mph • Exposure category = C 0 • Wind directional Procedure Seismic Load: • Directional Pr Importance, Ie = 1.0 • Site class = D • SS = 0.15, SDS = 0.16 • S1 = 0.08, SD1 = 0.13 • Seismic design category = B • Equivalent lateral force procedure • Building frame system; ordinary reinforced concrete shear walls in the north-south direction by R=5 by Cs = 0.046 • Moment-resting frame system; ordinary reinforced concrete moment frame in the east-west direction by R=3 by Cs = 0.032 The fcgIRUFROXPQVDQGZDOOVLQPXOWLVWRU\EXLOGLQJVPD\ be different than the fcgXVHGIRUWKHÀRRUV\VWHP&RQFUHWH SODFHPHQWXVXDOO\SURFHHGVLQWZRVWDJHVIRUHDFKVWRU\¿UVW WKHYHUWLFDOPHPEHUVVXFKDVFROXPQVDQGVHFRQGWKHÀRRU members, such as beams and slabs. It is desirable to keep the concrete strengths of the vertical members within a ratio RIRIWKHARRUFRQFUHWHVWUHQJWK6HFWLRQLQ\$&, VWDWHVWKDWLIWKLVUDWLRLVH[FHHGHGWKHARRUFRQFUHWH in the area immediately around the vertical members must be "puddled" with higher strength concrete. Usually this situation only becomes an issue for taller buildings. For this example, the building height is moderate and the loads are typical. The locally available aggregate is a durable dolomitic limestone. Thus, the concrete can readily have a higher fcgWKDQWKHLQLWLDODVVXPSWLRQRISVL\$FKHFNRI the durability requirements of Table 19.3.2.1 in ACI 318-14 shows that 5000 psi will satisfy the minimum fcgIRUDOOH[SRsure classes. For this concrete, a check of Table 19.2.1.1 in ACI 318-14 shows that all the code minimum limits are satisci HG7KHIROORZLQJFRQFUHWHPDWHULDOSURSHUWLHVDUHFKRVHQ • fcg SVL • Normalweight, wc = 150 lb/ft3 • Ec = 4,030 4,030,000 psi • H 5 5 × 10-6/F • eth =5.5 The usee of lightweight concrete can reduce seismic forces tweight co ndat ads. Based on local experience, however, and foundation loads. W H RRI EXLOGLQJ QJ ZRQ W WKLV W\SH ZRQ W UHDWO\ EHQH¿W IURP WKH XVH li h weig The modulus of elastic for concrete, Ec, is off lightweight. alcul ed aaccording g to 19.2. calculated 19.2.2 in ACI 318. For normalweight concr e, Eq 2.2.1.b in ACI 318 is applicable. Software concrete, Eq. 19.2.2.1.b ¿ L ¿QLWHH O SURJUDPVXVLQJ¿QLWHHOHPHQWDQDO\VLVFDQDFFRXQWIRUWKH ect. The P Poisson effect. Poisson ratio can vary due to material erties, but an average value for concrete is 0.2. Recomproperties, PHQGDWLR PHQGDWLRQVIRUWKHWKHUPDOFRHI&FLHQWRIH[SDQVLRQe th, of concrete can be found in ACI 209R. The most common and most available but 20.2.2.4 in ACI 318-14 limits many uses of reinforcing steel to 60 ksi. The modulus of elastic for reinforcement, Es, is given in 20.2.2.2 in ACI 318. Reinforcement Material Properties • fy = 60,000 psi • fyt = 60,000 psi • fy specicHGFRQFUHWHFRPSUHVVLYHVWUHQJWKfcoRISVLXVXDOO SURYLGHV IRU D VDWLVIDFWRU ARRU GHVLJQ, Q WKH 86 UHLQIRUFLQJVWHHOIRUARRUGHVLJQLVXVXDOO SURYLGHV IRU D VDWLVIDFWRU ARRU GHVLJQ, Q WKH 86 UHLQIRUFLQJVWHHOIRUARRUGHVLJQLVXVXDOO American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 2-STRUCTURAL SYSTEMS 2.2-Materials The concrete mixture properties and limits in ACI 318-14, Chapter 19, and the reinforcing steel needs to satisfy the design properties and limits in ACI 318-14). 2.3-Materials The concrete mixture properties and limits in ACI 318-14, Chapter 19, and the reinforcing steel needs to satisfy the design properties and limits in ACI 318-14). Design loads ACI 318-14 assumes that ASCE 7-10 design loads are applied to the building's structural system and to individual PHPEHUV DV DSSOLFDEOH /RDGV DUH DVVXPHG WR EH DSSOLHG vertically and horizontally. Horizontal loads are assumed to Table 2.1-Member chapters Volume no. ACI SP-17(14) I II Chapter name ACI SP-17(14) Building system Structural systems Structural analysis Durability 2QHZD/VODE Two-way slab Beams Diaphragm
Columns Walls Foundations Retaining walls Serviceability Strut-and tie Anchoring to concrete Chapter no. ACI 318-14 ACI SP-17(14) - 1 4 and 5 2 6 3 19 4 7 5 8 6 9 7 12 8 10 9 11 10 13 11 7 and 11 12 24 13 23 14 17 15 ogonal direct ons Two types of lateral loads are act in orthogonal directions. ed in this chapter: discussed nd lloading g (elastic analysis, a 1. Wind ACI 318-14, Chapter 18) Wi quake loading (ACI 318-14, Chapter 18) Wi quake loading (ACI 318-14, Chapter 6) 2. E rthqu ading g (elastic analysis, a 1. Wind ACI 318-14, Chapter 18) Wi quake loading (ACI 318-14, Chapter 18) Wi quake loading (ACI 318-14, Chapter 18) Wi quake loading (ACI 318-14, Chapter 18) Wi quake loading g (elastic analysis, a 1. Wind ACI 318-14, Chapter 18) Wi quake loading (ACI 318-14) loads are induc cture. Wi induced in a structure. Wind loads are externally applied loa d hence,, are related to the structure's exposed loads and, thquake loads and, thquake loads are inertial forces related to the tude and dis magnitude distribution of the mass in the structure. 2.3.1 Win Wind loading—Wind kinetic energy is transformed into terrain, and the location and size of other local structures. The structural response to D WXUEXOHQW ZLQG HQYLURQPHQW LV SUHGRPLQDQWO\ LQ WKH ¿UVW mode of vibration. The when it is resisted by an obstruction. Wind pressure is related to the wind velocity, building height, building surface, the surroundir static approach to wind load design has generDOO\SURYHGVXI¿FLHQW,WPD\QRWEHVDWLVIDFWRU\KRZHYHU for very tall buildings, especially with respect to the comfort of the occupants and the permissible horizontal movement, "or drift," which can cause the distress of partitions and glass. Therefore, to determine design wind loads for very tall buildings, especially with respect to the comfort of the occupants and the permissible horizontal movement, "or drift," which can cause the distress of partitions and glass. tall buildings, wind tunnel testing is not unusual. 2.3.2 Earthquake loading<sup>2</sup>7KHPDLQREMHFWLYHRIVWUXFtural design is life safety; that is, preserving the lives of occupants and passersby. Serviceability and minimizing HFRQRPLFDOORVVKRZHYHUDUHDOVRLPSRUWDQWREMHFWLYHV% studying the results of previous earthquakes on various structural systems, improvements to code provisions and design practices have been achieved. These improvements for PHPEHUVWKDWUHVLVWVLJQL¿FDQWVHLVPLFDFFHOHUDWLRQVDUH American Concrete Institute – Copyrighted © Material – www.concrete.org Structural Systems 2.1—Introduction A chapter on structural systems. A structural systems of reinforced concrete buildings has been introduced into the ACI Code (ACI 318-14). This chapter gives guidance on the relationships among the different chapters and their applicability to structural systems. A structural engineer's primary concern is to design buildings that are structurally safe and serviceable under design vertical and lateral loads. Prior to the 1970s, reinforced concrete buildings that were of moderate height (less than 20 stories), not in seismically active areas, or constructed with nonstructural masonry walls and partitions, designed for lateral forces (ACI Committee 442 1971). Continuing research, advancement in materials science, and improvements in analysis tools have allowed structural engineers to develop economical building designs with more predictable structural performance. VXI¿FLHQW VWDELOLW\ VWUHQJWK DQG VWLIIQHVV ned, desig overall structural integrity is maintained, design loads are resisted, and serviceability limits are m met. The individual ural system are generally members of a building's structural ertic h assumed to be oriented either vertically orr horizontally, with ng structure ure ramps. Chap er 4 the common exception of parking Chapter XF PHPEHUVDQGFRQQHFRI\$&,LGHQWL¿HVWKHVWUXFWXUDOPHPEHUVDQGFRQQHFed concrete bui ing tion types that are common to rei reinforced building nd ddetailing ling code provi ns structural systems with design and provisions (ACI 318-14): D +RUL]RQWDO ÀRRU DQC URRI PHPEHUV RQHZD\ DQG two-way slabs, Chapters 7 and 8) V E +RUL]RQWDO VXSSRUW PHPEHUV EHDPV DQG MRLVWV Chapter 9) F 9HUWLFDOPHPEHUVFROXPQVDQGVWUXFWXUDOZDOOV&KDSters 10 and 11) (d) Diaphragms and collectors (Chapter 12) (e) Foundations—isolated footings, mats, pile caps, and piles (Chapter 13) (f) Plain concrete—unreinforced foundations, walls, and piers (Chapters 15 and 16) In Table 2.1, code chapters 15 and 16) In Table 2.1, code chapters 15 and 16) In Table 2.1, code chapters are correlated with the chapters 15 and 16) In Table 2.1, code chap capacity in regions where yielding is likely, which then protects the overall integrity and stability of the building. Dynamic (modal) analysis is commonly used for larger structures, important structures, or for structures with an irregular vertical or horizontal distribution of stiffness or mass. For very important structures, important structures with an irregular vertical or horizontal distribution of stiffness or mass. example, nuclear power plants—inelastic dynamic analysis may be used (ACI Committee 442 1988). Fig. 2.3.2—Typical distribution of equivalent static lateral . forces "ASCE 7-10." 7KHUHTXLUHPHQWWKDWFROXPQVLQDIUDPHDUHÀH[XUDOO\ DIUDPH ed "st stronger than beams—the so-called "strong column-weak beam" concept se ductility ity and large en gy 2. Improve detailing to increase energy ation in stiffness dissipation capacity (with less de deterioration stiffness and strength) UV WR HQVXUH AH XUDO 'HVLJQLQJ DQG GHWDLOLQJ PH PHPEHUV AH[XUDO trength yielding before reaching nominal sh shear strength 4. Designing and detailing the connections to be stronger than the members framing into them /LPLWLQJVWUXFWXUDOV/VWHPLUUHJXODULWLHV For most structures, the equivalent lateral force procedure given in ASCE 7-10 is used. Based on this procedure, the distribution of design forces along the height of a building's fundamental mode of vibration (Fig. 2.3.2). Applying recorded earthquake motions to a structure through elastic dynamic analyses usually result in greater force demands than from the earthquake design forces specicHGE\PRVWFRGHV7KLVLVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDXVHFRGHV7KLVVEHFDX IDFWRU UHVSRQVH PRGL:FDWLRQ factor), which accounts for the ductility of a building, system over-strength, and energy dissipation through the soil-founGDWLRQV/VWHP3\$6&(`,WVLPSOL:HVWKHVHLVPLFGHVLJQ process such that linear static elastic analysis can be used for building, system over-strength, and energy dissipation through the soil-founGDWLRQV/VWHP3\$6&(`,WVLPSOL:HVWKHVHLVPLFGHVLJQ process such that linear static elastic analysis can be used for building, system over-strength, and energy dissipation through the soil-founGDWLRQV/VWHP3\$6&(`,WVLPSOL:HVWKHVHLVPLFGHVLJQ process such that linear static elastic analysis can be used for building. and assumes a building may be damaged during an earthquake event, but will not collapse. The higher the R-value, the lower the lateral design load on a structures with stiff systems, having low deformation capacity, to 8 for ductile systems, KDYLQJ VLJQL¿FDQW GHIRUPDWLRQ FDSDFLW\, Q D GHVLJQOHYHO earthquake, it is expected that some building members will yield. To promote appropriate inelastic behavior, ACI 318 2.4—Structures must have a continuous load path that can be traced from all load sources or load application to WKH IRXQGDWLRQ 7KH MRLQWV EHWZHHQ WKH YHUWLFDO PHPEHUVFDO PHPUVFDO PHPEHUVFDO PHPUVFDO PHPUVFDO PHPUVFDO PHPUVFDO PHPUVFDO PHPUVFDO P (columns and walls) and the horizontal members (beams, slabs, diaphragms, and foundations) are crucial to this concept. Properly detailed cast-in-place (CIP) reinforced FRQFUHWHMRLQWVWUDQVIHUPRPHQWVDQGVKHDUVIURPWKHARRU into columns and walls, thus creating a continuous load path. 7KH MRLQW GHVLJQ VWUHQJWK \$&, &KDSWHU PXVW of course, adequately resist the factored forces applied to WKH MMRLQW 5HIHU DOVR WR\$&, 5 IRU MRLQW GHVLJQ DQG detailing inform information. ngineers commonly refer to a structure's gravity-loadJ V\VWHP \*/56 UHVLVWLQJ V\VWHP \*/56 DDQ DQG ODWHUDOIRUFHUHVLVWLQJ V\VWHP \$00 PEHUVRID /)56 \$OOPHPEHUVRID&,3UHLQIRUFHGFRQFUHWHVWUXFWXUH FRQ XWHWR \*/56DQGP FRQWULEXWHWRERWKV\VWHPV For lo w-ris structures, ures, the inherent lateral stiffness of the low-rise /56 LV V RI ¿FLHQW WR UHVLVW WKH GHVLJQ ODWHUDO IRUFHV \*/56 RIWHQ VXI¿FLHQW ZLWKRX DQ\ VWRWKHG ZLWKRXWDQ\FKDQJHVWRWKHGHVLJQRUGHWDLOLQJRIWKH\*/56 mem rs. A uilding in members. As the building increases in height, the importance L DQG QG GGHWDLO LOL WKH /5)6 WR UHVLVW ODWHUDO ORDGV RI GHVLJQLQJ GHWDLOLQJ increases. At some po point, stiffness rather than strength will UQ WKH GHVLJQ GHVLJQ RI WKH /)56 ./Q WKH GHVLJQ SURFHVV WKH JRYHUQ W\SH RI / W\SHRI/)56LVXVXDOO\LQAXHQFHGE\DUFKLWHFWXUDOFRQVLGerations and construction requirements. There are several types
of structural systems or a combination thereof to resist gravity, lateral, and other loads, with deformation behavior as follows: 1. Frames<sup>2</sup>/DWHUDO GHIRUPDWLRQV DUH SULPDULO\ GXH WR VWRU\VKHDU7KHUHODWLYHVWRU\GHAHFWLRQVWKHUHIRUHGHSHQG on the horizontal shear applied at each story level. 2. Walls<sup>2</sup>/DWHUDOGHIRUPDWLRQVUHGXHWRERWKVKHDUDQG bending. The behavior predominate mode depends on the wall's height-to-width aspect ratio. 3. Dual systems—Dual systems are a combination of moment-resisting frames and structural walls. The momentresisting frames support gravity loads, and up to 25 percent RIWKHODWHUDOORDG7KHVWUXFWXUDOZDOOVUHVLVWWKHPDMRULW\RI the lateral loading. 4. Frames with closely spaced columns known as cantilevered column system or a tube system<sup>2</sup>/DWHUDO deformations are due to both shear and bending, similar to a wall. Wider openings in a tube, however, can produce a behavior intermediate between that of a frame and a wall. Regardless of the system, a height is reached at which the resistance to lateral sway will govern the design of the American Concrete Institute – Copyrighted © Material – www.concrete.org 15 Structural Systems CHAPTER 2—STRUCTURAL SYSTEMS Fig. 2.4—Structural systems and optimum height limitations (Ali and Moon 2007). ss, not strength, structural systems and optimum height limitations (Ali and Moon 2007). (2010) provides provisions C A through F). As a buildSeismic Design Categories (SDC y increases, in es, from A thr ing's Seismic Design Category through pro sively uct system stem to mainta seismic design and a more ductile maintain an orm e. acceptable level of seismic performance. ries of earthquake arthquake deta g; ACI 318 provides three categories detailing; ordinary, intermediate, and special. T These categories provide an increasing level of system toughness. Building height limits in ASCE 7 (2010) are related to the /)56 For buildings in SDC A and B, wind load will usually FRQWUROWKHGHVLJQRIWKH/)56 For buildings in SDC C, seismic loads are likely to control GHVLJQIRUFHVDQGVHLVPLFGHWDLOLQJLVUHTXLUHG/)56VDUH not limited in height for most systems for this SDC, but interstory drift limits from ASCE 7 (2010) must be met. Again, stiffness, not strength, will likely control the lateralforce-resisting system design. For buildings in SDC D, E, and F, seismic loads almost always control design forces, and increased seismic detailing LV UHTXLUHG /)56 RIWHQ KDYH PD[LPXP KHL]KW OLPLWDWLRQV based on assumed structural systems. Table 2.4 provides ASCE limits for choosing a structural system for a particular building. The ranges of applicability VKRZQ DUH LQAXHQFHG E\ RFFXSDQF\ UHTXLUHPHQWV DUFKLWHFWXUDOFRQVLGHUDWLRQQGDVSHFWUDWLRQQG load intensity and types (live, wind, and earthquake). 2.4.1 Gravity-resisting systems—A gravity-load-resisting V\VWHP \*/56 LV FRPSRVHG RI KRUL]RQWDO ÀRRU PHPEHUV and vertical members. Gravity loads ar are resisted by reinforced concrete members JK D[LDO ÀH[XU WKURXJK ÀH[XUDO VKHDU DQG WRUVLRQDO VWLIIQHVV DQG rma members. strength.. Th Thee related defo deformations are exaggerated and shown in Fig. 2.4.1 2.4.1. 22.4.2 La oad-resist Lateral-load-resisting system—A lateral-forceUHVLVWL JV\ )56 PX UHVLVWLQJV\VWHP/)56 PXVWKDYHDQDGHTXDWHWRXJKQHVV tain integrity y during high wind loading and design to maintain earthq ake accelerations. ations. B earthquake Buildings are basically cantilevered memb esigned ffor strength (axial, shear, torsion, members designed DQGVHUYL DQGPRPHQW DQGVHUYL DQGPRPHQW DQGVHUYL DQGPRPHQW DQGVHUYL FHDELOLW\GHAHFWLRQV IRU HDFK 6'& DV LW DSSOLHV WR D VSHFL¿F VHLVPLF force-resisting frames derive their load resistance from member strengths and connection rigidity. In a moment-resisting frame structure, the lateral displacement (drift) is the sum of three parts: 1) deformation due to bending in columns, EHDPVVODEVDQGMRLQWVDQG GHIRUPDWLRQVGXHWRD[LDO force in columns. Yielding in the frame members or the foundation can VLJQL¿FDQWO\LQFUHDVHWKHODWHUDOGHÀHFWLRQVP"HIIHFWRI secondary moments caused by column axial forces multiSOLHGE\ODWHUDOGHÀHFWLRQVP"HIIHFW ZLOOIXUWKHULQFUHDVH WKHODWHUDOGHÀHFWLRQVP"HIIHFWRI secondary moments caused by column axial forces multiSOLHGE\ODWHUDOGHÀHFWLRQVP"HIIHFWRI secondary moments caused by column axial forces multiSOLHGE\ODWHUDOGHÀHFWLRQVP"HIIHFW structure and WKH IUDPHV DUH LQWHUFRQQHFWHG E\ ARRU GLDSKUDJPV \$&, 318, Chapter 12). Moment-resisting frames usually allow WKH PD[LPXP AH[LELOLW\ LQ VSDFH SODQQLQJ DQG DUH DQ economical solution up to a certain height. American Concrete Institute – Copyrighted © Material – www.concrete.org 16 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Table 2.4—Approximate building height limits for various LFRS Practical limit of system (ASCE 7-10 limit according to SDC) SDC Type of LFRS A and B C D E F Moment-resisting frames (only): 2UGLQDU\PRPHQWIUDPH20) 1/ NP NP NP Intermediate moment frame (IMF) 1/ 1/ NP NF lateral- and gravity-load-resisting system): 26: 1/ 1/ NP NP NP SSW\* 1/ 1/ 160 ft 100 ft 120 VWHPVVWUXFWXUDOZDOOVDUHWKHSULPDU/5)6DQGWKHPRPHQWUHVLVWLQJIUDPHVFDUU/DWOHDVW of the lateral load): 26:ZLWK20) 1/ NP NP NP 26:ZLWK20) 1/ 1/ NP NP NP 26:ZLWK20) 1/ 1/ NP NP NP 06:ZLWK20) 1/ 1/ NP NP 06:ZLWK20) 1/ 1/ NP NP NP 06:ZLWK20) 1/ 1/ NP 06:ZLWK NP SSW with IMF 1/1/160 ft 100 ft 100 ft 100 ft 100 ft SSW with SMF 1/1/1/1/F \* Height limits can be increasedd per ASCE 7 (2010), Section 12.2.5.4. 13 QRWS 1RWHV1/QROLPLW13 QRWSHUPLWWHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLW\ORDG 2.4.2.2 Shear walls are often introduceccle shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLW\ORDG 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLW\ORDG 2.4.2.2 Shear walls are often introduceccle shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLW\ORDG 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWWHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLW\ORDG 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLW\ORDG 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWWHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWYHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWYHWHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWYHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWYHG Fig. 2.4.2.2 Shear walls. )LJ<sup>2</sup>HAHFWLRQVGXHWRJUDYLWYHWHG Fig. 2.4.2.2 Shear walls wal into multistory buildings because of their high in-plane stiffness and strength to resist lateral forces or when the building program is conducive to layout of VWUXFWXUDOZDOOV)RUEXLOGLQJVZLWKRXWDVLJQL¿FDQWPRPHQW frame, shear walls behave as vertical cantilevers. Walls can be designed with or without openings (Fig. 2.4.2.2a) Separate walls can be coupled to act together by beams/slabs or deep beams, depending on design forces and architectural requirements. Coupling of shear walls introduces frame DFWLRQ WR WKH /)56 DQG WKXV UHGXFHV ODWHUDO GHAHFWLRQ RI the system. Reinforced concrete walls are often used around HOHYDWRUDQGVWDLUVKDIWVWRDFKLHYHWKHUHTXLUHG&UHUDWLQJ For shear wall sin orthogonal directions, refer to Table 2.4.2.2. A shear wall building usually consists of a series of parallel shear walls in orthogonal directions, refer to Table 2.4.2.2. A shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear wall building usually consists of a series of parallel shear walls in orthogonal directions. GLVFRQWLnuities in mass, stiffness, and geometry should be avoided. Bearing walls should be located close to the plan perimeter if possible and should preferably be symmetric in plan to reduce torsional effects from lateral loading (refer to Fig. 2.4.2.2c). 2.4.2.3 Staggered wall-beam system—This system uses story-high solid or pierced walls extending across the entire width of the building and supported on two lines of columns placed along exterior faces (Fig. 2.4.2.3). By staggering the ORFDWLRQVRIWKHVHZDOOEHDPVRQDOWHUQDWHARRUVODUJHFOHDU DUHDVDUHFUHDWHGRQHDFKARRU American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 2—STRUCTURAL SYSTEMS 17 Table 2.4.2.2—Shear wall types and functions Behavior Reinforcement Remarks /DWHUDOGHVLJQLVXVXDOO\FRQFHUQHG only with shear strength. Bars evenly distributed horizontally and vertically. Wall foundation must be capable of resisting the actions generated in the wall. Consider sliding resistance provided by foundation. Height-to-length ratio is greater than 2 /DWHUDOGHVLJQPXVWFRQVLGHU both the wall's shear and moment strength. Evenly distributed vertical reinforcement may be concentrated at ZDOOHQGV<sup>2</sup>ERXQGDU\HOHPHQWV9HUWLFDO reinforcement in the web contributes to WKHAH[XUDOVWUHQJWKRIWKHZDOO Wall foundation must be capable of resisting the actions generated in the wall. Consider overturning resistance provided by foundation. Ductile structural wall /DWHUDOGHVLJQLVKHDYLO\LQAXHQFHG E\AH[XUHVWLIIQHVVDQGVWUHQJWK Flexural bar spacing and size should be VPDOOHQRXJKVRWKDWÀH[XUDOFUDFNLQJ LVOLPLWHGLI\LHOGLQJRFFXUV2YHUUHLQIRUFLQJIRUÀH[XUHLVGLVFRXUDJHGEHFDXVH ÀH[XUDO\LHOGLQJLVSUHIHUUHGRYHUVKHDU failure. Acceptable ductility can be obtained with proper attention to axial load level, FRQ¿QHPHQWRIFRQFUHWHVSOLFLQJRI reinforcement treatment of construction MRLQWVDQGSUHYHQWLRQRIRXWRISODQH
buckling. Coupled walls with shallow coupling beams or slabs (Fig. 2.4.2.2b(a)) /LQNVODEÀH[XUDOVWLIIQHVVGHWHriorates quickly during inelastic reversed loading. Place coupling slab bars to limit slab cracking at the stress concentrations at the wall ends. Punching shear stress around the wall ends in the slab needs to be checked. Coupled walls with coupling beams (Fig. 2.4.2.2b(b)) Depending on span-to-depth ratio, link beams may be designed as deep beam's corner to corner QHGE\VSLUDORUFORVHG PD\EHFRQ¿QHGE\VSLUDORUFORVHGWLHV H[XUHDQGVKHDU DQGGHVLJQHGWRUHVLVWÀH[XUHDQGVKHDU directly. Properly detailed coupling beams should maintain their load-carrying capacity under reverse inelastic deformation. ,Q¿OOHGIUDPHV (structural or nonstructural) (Fig. 2.4.2.2b(c)) ehav as braced frames, Frames behave ng tthe lateral al strength and increasing V7 OLQJDFWVDVDVWUXW n ddiagonally ly opposite frame between an creates tes high hig shear corners, and mns. forces in th the columns. HU EH LHQWO' ,Q¿OOZDOOVVKRXOGHLWKHUEHVXI¿FLHQWO\ parated from the th moment men frame separated making them nonstructural), nonst ctura or detailed iled (making urally with the too be connected structural), nonst ctura or detailed iled (making urally with the too be connected structural). QRLQ¿OOVDWDJLYHQVWRU\OHYHOWKDW stor acts as a weak or soft story that is story vul vulnerable to concentrated damage and in instability. )LJE<sup>2</sup>&RXSOHGDQGLQ¿OOZDOOVD VKDOORZFRXSOLQJEHDPVRUVODEVE FRXSOLQJ EHDPVDQGF LQ¿OOZDOOV The staggered wall-beam building is suitable for multistory construction having permanent interior partitions such as apartments, hotels, and student residences. An advantage of the wall-beam building is the large open DUHDWKDWFDQEHFUHDWHGLQWKHORZHUARRUVZKHQQHHGHGIRU parking, commercial use, or even to allow a highway to pass under the building. This system should be considered in low VHLVPLFDUHDVEHFDXVHRIWKHVWLIIQHVVGLVFRQWLQXLW\DWHDFKARRU 2.4.2.4 Tubes—A tube structure consists of closely spaced columns in a moment frame, generally located around the perimeter of the building (Fig. 2.4.2.4(a)). Because tube structures generally consist of girders and columns with low span-to-depth ratios (in the range of 2 to 4), shearing deformations often contribute to lateral drift and should be included in analytical models. Tubes are often thought of as behaving like a perforated diaphragm. Frames parallel to direction of force act like webs to carry the shear from lateral loads, while frames perpendicular to WKH GLUHFWLRQ RI IRUFH DFW DV ADQJHV WR FDUU\ WKH PRPHQW from lateral loads. Gravity loads are resisted by the exterior frames and interior columns. A reinforced concrete braced tube is a system in which a WXEHLVVWLIIHQHGDQGVWUHQJWKHQHGE\LQ¿OOLQJLQDGLDJRQDO pattern over the faces of the building (Fig. 2.4.2.4(b)). This bracing increases the structure's lateral stiffness, reduces the American Concrete Institute - Copyrighted © Material - www.concrete.org Structural Systems Structural walls Short—height-to-length ratio does not exceed 2 18 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 2.4.2.2c—Example of shear wall layouts. Fig. 2.4.2.3—Staggered wall-beam system. moments in the columns and girders, and reduces the effects of shear lag. 2.4.3 Dual systems—Dual systems consist of combining two of the structural systems are discussed in 2.4.3.1 through 2.4.3.6. 2.4.3.1 Wall-frame systems<sup>2</sup>5LJLGMRLQWHG IUDPHV DQG isolated or coupled structural walls can be combined to SURGXFHDQHI¿FLHQWODWHUDOIRUFHUHVLVWLQJV/VWHP%HFDXVH RI WKH GLIIHUHQW VKHDU DQG AH[XUDO ODWHUDO GHAHFWLRQ FKDUacteristics of moment frames and structural walls, careful attention to the interaction between the two systems can improve the structure's lateral response to loads by reducing ODWHUDOGHAHFWLRQV)LJ The wall's overturning moment is greatly reduced by interaction with the frame. Because drift compatibility is forced on both the frame and the wall, and the frame-alone and wall-alone drift modes are different, the building's overall lateral stiffness is increased. Design of the frame columns J IRUJUDYLW\ORDGVLVDOVRVLPSOL¿HGLQVXFKFDVHVDVWKHIUDPH columns are as assumed to be braced against sidesway by the walls. -frame dual sy The walls. -frame dual sy The walls. -frame dual sy The wall sidesway by the structure to gned for a desired yielding sequence under strong be designed grou eams can bbe designed to experience signifground mot motion. Beams icant yielding eldi before ore inelas inelastic action occurs at the bases h walls. wall By creating relative economy be repaired, re ired wall-frame -frame str structures are appropriate for use iin higher seismic mic zone zones. However, note that the variation nd overturning moments over the height of the wall and frame is very different under inelastic versus elastic response conditions. 2.4.3.2 Outrigger system uses orthogonal walls girders, or trusses, one or two stories in height, to connect the perimeter columns to central core walls, thus enhancing the structure lateral forces are placed around the perimeter of the structure at the outrigger levels to help distribute lateral forces. between the perimeter columns and the core walls. These perimeter girders or trusses are called "hat" or "top-hat" bracing if located at the top, and "belt" bracing if located at the top and "between the axial drift and core bending moments can be achieved by increasing the cross section of the columns and, therefore, the axial stiffness, and by adding outriggers at more levels. 2XWULJJHUVDUHHIIHFWLYHLQLQFUHDVLQJRYHUDOOEXLOGLQJVWLIIness and, thus, resist wind loads with less drift. Design of outrigger-type systems for SDC D through F must consider the effect of the high local stiffness of the outriggers on the inelastic response of the entire system. Members framing into the outriggers should be detailed for ductile response. 2.4.3.3 Tube-in-tube—For tall buildings with a reasonably large service core, it is generally advantageous to use American Concrete Institute - Copyrighted © Material - www.concrete.org 19 Structural Systems CHAPTER 2—STRUCTURAL SYSTEMS Fig. 2.4.2.4—Tube systems. Fig. 2.4.3.2—Outrigger system. Fig. 2.4.3.1—Shear wall and moment frame system. The outer tube is formed by the closely-spaced column-spandrel beam frame. A bundled tube system consists of several framed tubes bundled into one larger structure that behaves as a multicell perforated box (Fig. 2.4.3.3). The tube-in-tube system combines the advantages of both the perimeter framed tube systems. perimeter framed tube by reducing the shear deformation of the columns in the framed tube. The tube-in-tube system can EHFRQVLGHUHGDUH¿QHGYHUVLRQRIWKHVKHDUZDOOIUDPHLQWHUaction type structure. American Concrete Institute – Copyrighted © Material – www.concrete.org 20 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 2.4.3.4 Bundled tubes—American Concrete Institute – Copyrighted © Material – www.concrete.org 20 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 2.4.3.4 Bundled tubes—American Concrete.org 20 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 2.4.3.4 Bundled tubes bundled tube system consists of several framed tubes bundled into one larger structure that behaves as a multicell perforated box. Individual tubes can be terminated at different heights. The bundled into one larger structure that behaves as a multicell perforated box. Individual tubes can be terminated at different heights. concrete-steel structures—Mixed concretesteel systems consist of interacting concrete and steel assemblies. The resulting composite structures (large spans and lightweight construction) as well as the favorable characteristics of concrete structures (high lateral stiffness of shear walls and coresteel systems) as well as the favorable characteristics of concrete structures (high lateral stiffness of shear walls and coresteel systems) as well as the favorable characteristics of concrete structures (high lateral stiffness of shear walls and coresteel systems) as well as the favorable characteristics of concrete structures (high lateral stiffness of shear walls and coresteel systems) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favorable characteristics of concrete structures (high lateral stiffness) as well as the favora and high damping). Engineers must address the differential vertical creep and shrinkage between steel and concrete to prevent uneven displacement. Because the erection of steel and concrete structures involves different building trades and equipment, engineers who design mixed construction should consider scheduling issues. 2.4.3.6 Precast structures—Precast concrete members are widely used as components in frame, wall, and wall-frame systems. Mixed construction,
consisting of precast concrete assemblies connected to a cast-in-place concrete core, is also GVRQWKHH[WHQW XVHG7KHHI¿FLHQF\RIVXFKV\VWHPVGHSHQGVRQWKHH[WHQW ure, the si of standardization, the ease of manufacture, simplicity of assembly, and the speed of erection. UJHV 3UHFDVWARRUV\VWHPVLQFOXGHODUJHVWDQGDUGL]HGUHLQIRUFHG te sl (and usually prestressed) concrete slabs, with or without inteore as well as prefabri ted rior cylindrical voids (hollow core), prefabricated D XDOO\DVVHPEOHG URF ULEVODEV5LJLGMRLQWHGIUDPHVDUHXVXDOO\DVVHPEOHGIURP nd coress are assembled from rom H- or T-units, and shear walls and s. Planning ning and desig ng prefabricated single-story panels. designing appropriate connection details for ppanels, fframe member DQGÀRRUDVVHPEOLHVLVWKHVLQJOHPRVWLPSRUWDQWRSHUDWLRQ related to prefabricated structures. s: Three main types of connections are described as follows: 6WHHO UHLQIRUFHPHQW EDUV SURWUXGLQJ IURP DGMDFHQW precast members are made continuous by mechanical FRQQHFWRUV ZHOGLQJ RU ODS VSOLFHV DQG WKH MRLQW EHWZHHQ WKHPHPEHUVLV¿OOHGZLWKFDVWLQSODFHFRQFUHWH, IZHOGLQ] is used, the engineer should specify appropriate welding procedures to avoid brittle connections. 2. Steel inserts (plates and angles) provided in the precast members are bolted or welded together and the gaps are grouted. 3. The individual precast units are post-tensioned together DFURVVWKHMRLQWZLWKRUZLWKRXWDPRUWDUEHG 7KH EHKDYLRU RI D SUHFDVW V\VWHP VXEMHFWHG WR VHLVPLF loading depends to a considerable degree on the characteristics of the connections. Connection details can be developed that ensure satisfactory performance under seismic loadings, provided that the engineer pays particular attention to steel GXFWLOLW\DQGSRVLWLYHFRQ¿QHPHQWRIFRQFUHWHLQWKHMRLQWDUHD 2.5—Floor subassemblies 6HOHFWLRQRIWKHARRUV\VWHPVLJQL¿FDQWO\DIIHFWVDVWUXFture's cost as well as the performance of its lateralforceUHVLVWLQJV\VWHP7KHSULPDU\IXQFWLRQRIDARUV\VWHPLV to resist gravity load. Additional important functions in most buildings are: )LJ<sup>2</sup>7ZRZD\ADWSODWHV\VWHP Fig. 2.5.2—Flat slab with drop panels and capitals. (a) Diaphragm action: The slab's in-plane stiffness maintains the pplan shape of the structure, and distributes horizontal forces to th the lateral-force-resisting system. RPHQWUHVLVWDQ 7K 7 E 0RPHQWUHVLVWDQFH7KHÀH[XUDOVWLIIQHVVRIWKHÀRRUV tegral g and nec may be an integral necessary part of the lateral-forceg sys resisting system. Co rete structures ures are ccommonly analyzed for lateral Concrete RDGV DVVXPLQJ VVXP ORDGV WKH ÀRRU ÀRRU X/V V/VWHP DFWV DV D GLDSKUDJP LOctiff iin its plane. ane. This assumption is not valid for all nitely stiff FRQ2J UDWLR DQG G JHRPHWULHV RI ÀRRU V/VWHPV )DFWRUV affe q di qm stiffne affecting diaphragm stiffness are: span-to-depth ratio of the mensions rrelative to the location of the lateralslab's plan dimensions ting memb load-resisting members, slab thickness, locations of slab RSHQLQJVDQGG RSHQLQJVDQGGLVFRQWLQXLWLHVDQGW\SHRIÀRRUV\VWHPXVHG 7KH ÀRRU V\VWHP ÀH[XUDO VWLIIQHVV FDQ DGG WR WKH ODWHUDO stiffness of the structure. If the slab is assumed to act as part of a frame to resist lateral moments, engineers usually limit the effective slab width (acting as a beam within the frame) to between 25 and 50 percent of the bay width. 2.5.1 Flat plates<sup>2</sup>\$ADWSODWHLVDWZRZD\VODEVXSSRUWHGE\ columns, without column capitals or drop panels (Fig. 2.5.1). 7KH ADW SODWH V\VWHP LV D YHU\ FRVWHIIHFWLYH ARRU IRU commercial and residential buildings. Simple formwork and reinforcing patterns, as well as lower overall building height, are advantages of this system. In designing and detailing plate-column connections, particular attention must be paid to the transfer of shear and unbalanced moment between the slab and the columns (ACI 318, Chapters 8 and 15). This is DFKLHYHGE\XVLQIDVXI¿FLHQWVODEWKLFNQHVVRUVKHDUUHLQforcement (stirrups or headed shear studs) at the slab-column MRLQWDQGE\FRQFHQWUDWLQIVODEAH[XUDOUHLQIRUFHPHQWRYHU the column area. 2.5.2 Flat slabs with drop panels, column capitals, or both<sup>2</sup>7KH VKHDU VWUHQIWK RI ADW VODEV FDQ EH LPSURYHG by thickening the slab around columns with drop panels, column capitals (either constant thickness or tapered), shear FDSVRUDFRPELQDWLRQ)LJ /LNHADWSODWHVADWVODE American Concrete Institute - Copyrighted © Material - www.concrete.org 21 Fig. 2.5.6—One-way slab. )LJ<sup>2</sup>7ZRZD\JULGZDIAH VODE systems normally act as diaphragms transmitting lateral forces to columns and walls. 'URSSDOHOVLOFUHDVHDVODE @WKHFROXPODOGWKXVLPSURYHWKHDELOLW\RIWKHADWVODE GFROXPOFDSLWDOV WRSDUWLFLSDWHLOWKH/)566KHDUFDSVDOGFROXPOFDSLWDOV creasing the slab thickimprove the slab shear strength by increasing ve the slab shear strength ness around the column. To improve ness engineers can provide without increasing the slab thickness, diating out from the he closely spaced stirrups or shear study radiating column. PV VWLQJRQO\RIAD VODE /DWHUDOIRUFHUHVLVWLQJV/VWHPVFRQVLVWLQJRQO\RIADWVODE WLO PHVVWUXFWXUDOZ OOV RUADWSODWHIUDPHVZLWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZLWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKWGXFWLOHIUDPHVZWKRXWGXFWLOHIUDPHVZWKWKYWGXFWXUDOZOOV ble in high se ic or other bracing members, are uns unsuitable seismic areas (SDC D through F). system consisting of a grid of ribs intersecting at a b constant spacing can be used to achieve an appropriate slab depth for the longer span with much less dead load than a solid slab (Fig. 2.5.3). The ribs are formed by standardized dome or pan forms that are closely spaced. The slab thickness between the ULEV LV WKLQ DQG QRUPDOO\ JRYHUQHG E\ ¿UH UDWLQJ UHTXLUHPHQWV 6RPH SDQV DGMDFHQW WR WKH FROXPQV DUH RPLWWHG WR form a solid concrete drop panel, to satisfy requirements for transfer of shear and unbalanced moment between the slab and columns. \$ ZDIÀH VODE SURYLGHV DQ DGHTXDWH VKHDU GLDSKUDJP 7KH VROLG VODE DGMDFHQW WR WKH FROXPQ SURYLGHV VLJQL¿FDQW WZRZD\ VKHDU VWUHQJWK 60DE ÀH[XUDO DQG SXQFKLQ] VKHDU strength can be increased by the addition of closely spaced stirrups radiating out from the column face in two direcWLRQV6WLUUXSVPD\DOVREHXVHGLQWKHULEV%HFDXVHDZDIÀH VODEEHKDYHVVLPLODUO\WRDADWVODE/5)6VFRQVLVWLQJRQO\ RIZDIAHVODEIUDPHVDUHXQVXLWDEOHLQKLJKVHLVPLFGHVLJQ areas (SDC D through F). 2.5.4 One-way slabs on beams and girders<sup>2</sup>2QHZD\ slabs on beams and girders<sup>2</sup>2QHZD\ slabs on beams and girders that span between the girders. 2QHZD\ VODEV VSDQ EHWZHHQ WKH EHDPV 7KLV V/VWHP provides a satisfactory diaphragm, and uses the girder- column frames and beam-column frames to resist lateral ORDGV\$GHTXDWHAH[XUDOGXFWLOLW/FDQEHREWDLQHGE/SURSHU detailing of the beam and girder reinforcement. The beams and slabs can be placed in a composite fashion (with precast elements). If composite, shear connectors are placed at the beam-slab interface to ensure composite action. This system can provide dhat the shear connectors are detailed with VXI¿FLHQWVWUHQ]WKDQGGXFWLOLW\6RPHH[DPSOHVRIWKLVW\SH of slab system include: D 3UHFDVW FRQF VXSSRUWHG RQ ZDOOV RU FDVWLQSODFH oncre e be ming i di dire concrete beams framing directly into columns. E 6WHHOMRLVWVZLWKWKHWRSFKRUGHPEHGGHGLQDFDVWLQ HHOM LWKWKHWR place concrete oncr b. The sl slab. slab formwork is supported from WKH V Z VXSSRUWV WKHMRLVWVZKLFKVXSSRUWVWKHIUHVKVODEFRQFUHWH eams supporting a noncomposite steel deck with n-place concrete slabs. Note that ACI 318 does not govern the st structural design of concrete slabs for composite steel decks. 2.5.5 One-way ribbed slabs (joists)<sup>2</sup>2QHZD\ ULEEHG VODEMRLVW V\VWHPVFRQVLVWRIFRQFUHWHULEVLQRQHGLUHFWLRQ spanning between beams, which span between columns. The size of pan forms available usually determines rib depth and spacing. As with a two-way ribbed system, the thickness of the thin slab between ribs is often determined by the buildLQJ {VUHUDWLQJUHTXLUHPHQWV This} system provides an adequate shear diaphragm and is used in a structure whose lateral resistance comes from a PRPHQWUHVLVWLQJIUDPHRUVKHDUZDOOV2QHURZRISDQVFDQ EHHOLPLQDWHGDWFROXPQOLQHVJLYLQJDZLGHADWEHDPWKDW PD\EHXVHGDVSDUWRIWKH/)56(YHQLIWKHVODEV\VWHP GRHV QRW IRUP SDUW RI WKH GHVLJQDWHG /5)6 WKH HQJLQHHU should investigate the actions induced in the ribs by building drift. 2.5.6 One-way banded slabs—A one-way banded slabs beam can be reinforced with closely spaced stirrups near the support to increase the slab's shear strength. This American Concrete Institute - Copyrighted © Material - www.concrete.org Structural Systems CHAPTER 2-STRUCTURAL SYSTEMS 22 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. 2.5.7-Two-way slab with edge beams around perimeter, system is also sometimes referred to as wide-shallow beams with one-way slabs. \$ VWUXFWXUH XVLOJ WKLV W\SH RI ARU V\VWHP LV OHVV VWLII laterally than a structure using a ductile moment frame with EHDPV RI ORUPDO GHSWK /DWHUDOIRUFHUHVLVWLOJ V\VWHP LV OHVV VWLII laterally than a structure using a ductile moment frame with EHDPV RI ORUPDO GHSWK /DWHUDOIRUFHUHVLVWLOJ V\VWHP LV OHVV VWLII laterally than a structure using a ductile moment frame with EHDPV RI ORUPDO GHSWK /DWHUDOIRUFHUHVLVWLOJ
V\VWHP LV OHVV VWLII laterally than a structure using a ductile moment frame with end-way slabs. ADW SODWH IUDPHV ZLWKRXW ductile frames, structural walls, or other bracing members, are not suitable in SDC D through F. —As shown in F 2.5.7 Two-way slabs with edge beams—As Fig. ms in two tw directions on 2.5.7, the slab is supported by beams the perimeter column lines. This syste system is useful where a I WKH /)56 EHDPFROXPQ IUDPH LV UHTXLUHG DV SDUW RI /)56 7KH ter slab provides high diaphragm sti stiffness,, and the perim perimeter DW WLIIQHVV DQG VWUH QJWK EHDPV FDQ SURYLGH VXI¿FLHQW ODWHUDO VWLIIQHVV VWUHQJWK though frame action for use in SD SDC D throu through F. rid (Section ction 2.5.3) slab may For longer spans, a two-way grid EHXVHGLQVWHDGRIDADWSODWH 2.5.8 Precast slabs may be solid, hollow-core slabs, or cast-in-place beams, or cast-in-place be connections are normally used to transfer in-plane shear forces between precast slabs and their supports. Because precast slabs are individual units interconnected PHFKDQLFDOO\ WKH DELOLW\ RI WKH DVVHPEOHG ARRU V\VWHP WR act as a shear diaphragm must be examined by the engineer. Boundary reinforcement may be required, particularly where the lateral-force-resisting members are far apart. In areas of high seismicity, the connections between the precast slab improves the ability of the slab system to act as a shear diaphragm, and can be used in SDC D through F. 2.6—Foundation design considerations for lateral forces to the ground. A distinction should be drawn between external forces, such as wind, and inertia forces that result from the building's response to ground motions during an earthquake. External lateral forces can include static pressures due to ZDWHUHDUWKRU¿OODQGHTXLYDOHQWVWDWLFIRUFHVUHSUHVHQWLQ] the effects of wind pressures, where a gust factor or impact factor is included to account for their dynamic nature. The soil type and strata usually dictate whether the foundation is deep or shallow. A soils report from a licensed geotechnical engineer provides the detailed information and foundation recommendations that the licensed design profesVLRQDO //3 QHHGV WR GHVLJQ WKH IRXQGDWLRQ )RU VKDOORZ footings, the geotechnical engineer provides an allowable soil bearing pressure for the soil at the foundation elevation. 7KDWSUHVVXUHOLPLWWDUIHWVDFHUWDLODPRXOWRIVRLOGHAHFtion, and includes consideration of the anticipated use of the building. If allowable soil pressure is less than 2500 lb/ft2, the soil is very soft and deep foundation options are usually FROVLGHUHG2WKHUVRLOVLWXDWLROVVXFKDVH[SDOVLYHFOD]RU QRQVWUXFWXUDO¿OOPD\SUHFOXGHWKHXVHRIVKDOORZIRXQGDtions. If the building is below grade, concrete walls can be part of the foundation system. The two types of deep foundations are caissons (also known as piers) and piles. If hard rock is not far below existing grade, caissons can transfer a column load directly to the bedrock. Bearing values for solid rock can be more Ca are large in diameter, usually than 10 kip/ft2. Caissons pproximately 30 in. Piles are generally smaller in starting at anound 12 in., and can be cast-in-place a red hholes or precast piles that are driven into place. in augered signed for lighter loads than caissons are. Piles aaree usu usually designed p Groups of piles mayy be used where bedrock is too deep for cais n. Tops T ca a caisson. of piles or caissons are bridged by pile caps g de bbeams to o distribute column loads as needed. ndations i Shallow foundations are referred to as footings. Types of footings are isolated isolated, combined, and mat. Isolated rectar or square footings are the most common types. Combined footings, if an exterior column is too close to the boundary line, or if columns are transmitting moments to the footings, such as if the column is part of a lateral-forceresisting system. If the column loads are uniformly large, such as in multistory buildings, or if column spacing is small, mat foundation pressures resulting from lateral loads are usually of short duration and constitute a small percentage of the total vertical load effects that govern long-term soil settlements. Allowing a temporary peak in vertical bearing pressures XQGHU WKH LQAXHQFH RI VKRUWWHUP ODWHUDO ORDGV LV XVXDOO\ preferred to making the footing areas with a high groundwater table, or the possibility of sudden consolidaWLRQRIORRVHVRLOVZKHQVXEMHFWHGWRMDUULQJ7KHFDSDFLW\RI friction piles founded in soils susceptible to liquefaction or consolidation should be checked. 2.6.2 Resistance to overturning—The engineer should investigate the safety factor of the foundation against overturning and ensure it is within the limits of the local building code. 2YHUWXUQLQJFDOFXODWLRQVVKRXOGEHPDGHZLWKUHPRYDEOHVRLO American Concrete Institute - Copyrighted © Material - www.concrete.org ¿OORUOLYHORDGFRPSOHWHO\UHPRYHGDQGVKRXOGEHEDVHGRQD safe (low) estimate of the building's actual dead load. 2.7—Structural analysis The analysis of concrete structures "shall satisfy compatibility of deformations and equilibrium of forces," as stated in 6HFWLRQRI\$&,7KH/'3PD\FKRRVHDQ\PHWKRG of analysis as long as these conditions are met. This discussion is intended to be a brief overview of the analysis process as it relates to structural concrete design. For more detailed information on structural analysis, refer to Chapter 3 of this Handbook. 2.8—Durability Reinforced concrete structures are expected to be durable. The design of the concrete mixture proportions should consider exposure to temperature extremes, snow and ice, and ice-removing chemicals. Chapter 19 of ACI 318-14 provides mixture requirements to protect concrete and reinforcement against various exposures and deterioration. Chapter 20 of ACI 318-14 provides concrete cover requirements to protect reinforcement against steel corrosion. For more information, refer to Chapter 4 of this Handbook. 2.9—Sustainability Reinforced concrete structures are eexpected to be as sustainable as practical. ACI 318 allows sustainability sign, but they must st requirements to be incorporated in the design, cea requirements. not override strength and serviceability 2.10—Structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity is to "imp ve The ACI Code concept of structural integrity egrity egr GDPDJH WR D PDMRU VXSSRUWLQJ HOHPHQW RU DQ DEQRUPDO ORDGLQJ HYHQW WKH UHVXOWLQJ GDPDJH PD\ EH FRQ¿QHG WR D er relatively small area and the structure will have a better chance to maintain overall stability" (ACI Committee 442 1971). The Code addresses this concept by providing system continuity through design and detailing rules within the beam and two-way slab chapters. 2.11—Fire resistance 0LQLPXPFRYHUVSHFL¿HGLQ&KDSWHURI\$&,LV LQWHQGHGWRSURWHFWUHLQIRUFHPHQWDJDLQVW¿UHKRZHYHUWKH &RGHGRHVQRWSURYLGHDPHWKRGWRGHWHUPLQHWKH¿UHUDWLQJ of a member. The International Building Code (IBC) 2015

6HFWLRQSHUPLWVFDOFXODWLRQVWKDWGHWHUPLQH¿UHUDWLQJV to be performed in accordance with ACI 216.1 for concrete masonry members. 23 2.12—Post-tensioned/prestressed construction The introduction of post-tensioning/prestressing to FRQFUHWHARRUEHDPVDQGZDOOHOHPHQWVLPSDUWVDQDFWLYH permanent force within the structural system. The engineer should FRQVLGHU KRZ HODVWLF DQG SODVWLF GHIRUPDWLRQV GHAHFWLRQV changes in length, and rotations due to post-tensioning/prestressing affect the entire system. Special attention must be given to the connection of post-tensioned/prestressing force is permanent, the system creep and shrinkage effects require attention. 2.13—Quality assurance, construction, and inspection 7KH ,QWHUQDWLRQDO 6WDQGDUGL]DWLRQ 2UJDQL]DWLRQ 2UJDQL]DWLRQ 2UJDQL]DWLRQ 2UJDQL]DWLRQ 62 GH¿QHV 3TXDOLW\´ DV WKH GHJUHH WR ZKLFK D VHW RI LQKHUHQW FKDUDFWHULVWLFV IXO¿OOV D VHW RI UHTXLUHPHQWV 7KH JRDO RI TXDOLW\DVVXUDQFHLVWRHVWDEOLVKFRQ¿GHQFHWKDWSURMHFWVDUH EXLOW LQ FRPSOLDQFH ZLWK SURMHFW FRQVWUXFWLRQ GRFXPHQWV Chapter 26 of A ACI 318-14 contains requirements to facilitate mplementation of competent construction documents, the implementation nstruction, aand nd d iinspection. material, contains requirements to facilitate mplementation of competent construction documents, the implementation nstruction, aand nd d iinspection. material, contains requirements to facilitate mplementation of competent construction documents, the implementation nstruction and nd d iinspection. construction, REFE REFERENCES Amer an C American Concrete Institute 216 Code Requirements R ACI 216.1-14—Code for Determining Fire Resistance Resist Concrete Assee lies Assemblies 2 Reco ACI 352R-02—Recommendations for Design of Beamnnections in Monolithic Reinforced Concrete Column Connections ures Structures \$&, \$&,5<sup>2</sup>5HVSRQVHRI%XLOGLQJVWR/DWHUDO)RUFHV ACI 442R-88—Response of Concrete Buildings to /DWHUDO)RUFHV American Society of Civil Engineers \$6&(<sup>2</sup>0LQLPXP'HVLJQ/RDGVIRU%XLOGLQJVDQG other Structures International Building Code Authored documents Ali, M. M., and Moon, K. S., 2007, "Structural Development in Tall Buildings: Current Trends and Future Prospects," Architectural Systems CHAPTER 2—STRUCTURAL SYSTEMS 24 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 3—STRUCTURAL ANALYSIS 3.2—Overview of structures "shall um of satisfy compatibility of deformations and equilibrium CI 318-14. The forces," as stated in Section 4.5.1 of AC \FKRRVH OLFHQVHGGHVLJQSURIHVVLRQDO/'3 PD\FKRRVHDQ\PHWKRG ons are met. ACI 318-14, of analysis as long as these conditions evels of analysis: 1) elastic Chapter 6, is divided into three levels GHU LQHODVWLFVHFRQG rm thee use of strut-an -tie order. In additional evels of analysis as long as these conditions evels of analysis: 1) elastic Chapter 6, is divided into three levels GHU LQHODVWLFVHFRQG rm thee use of strut-an -tie order. In additional evels of analysis as long as these conditions evels of analysis evels of analysis as long as these conditions evels of analysis evels ev ACI 318 permits strut-and-tie sco ous regions. modeling for the analysis of discontinuous 8, ACI 318 provisions state tate Except as noted in Chapter 18, etc that the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that reinforced con concrete d It is also members behave elastically under the designer may assume that the designer may assume the designer may assume that the designer may assume the designer may assume that the designer may assume that the desig properties along the member length. ay These assumptions simplify analysis models but they may differ from the actual behavior of the concrete structures are modeled using an elastic analysis. The stability of columns DQGZDOOVPXVWEHFRQVLGHUHGE\ERWK¿UVWRUGHUDQGVHFRQG RUGHU DQDO\VLV )RU ¿UVWRUGHU DQDO\VLV HQG PRPHQWV RI FROXPQVDQGZDOOVDUHFRQVHUYDWLYHO\DPSOL¿HGWRDFFRXQW for second-order effects are calculated directly considering the loads applied on the laterally deformed structure. A series of analyses are made where the secondary moment from each analysis is added to the subsequent analysis until equilibrium is achieved. 3.2.3 Inelastic analysis may take into account material nonlinearity, member curvature and lateral deformation (second-order effects), duration of loads, shrinkage and creep, and interaction with the supporting foundation. The resulting strength must be compatible with results of published tests. This DQDO/VLV LV XVHG IRU VHLVPLF UHWUR¿W RI H[LVWLQ] EXLOGLQ]V design of materials and systems not covered by the code; and evaluation of building performance above code minimum Table 3.2.5—Common analysis types and tools Analysis type First-order /LQHDUHODVWLF Static load Hand calculations Applicable member or assembly Analysis tables\* Continuous one-way slab 6LPSOL¿HGPHWKRGLQ6HFWLRQ 6.5 of ACI 318-14 Two-way slab Direct design method in Section 8.10 of ACI 318-14 Equivalent frame method in Section 8.11 of ACI 318-14 Beam Analysis tables\* Continuous beam 6LPSOL¿HGPHWKRGLQ6HFWLRQ 6.5 of ACI 318-14 Column Interaction diagrams\* der First-order /LQHDUHODVWLF ad Static load Comp er pro Computer programs Second-order /LQHDUHODVWLF Static or dynamic load Computer programs Second-order Inelastic \* Wall Alternative method for out-ofplane slender wall analysis in Section 3.3.3 of this chapter Gravity-only systems Spreadsheet program based on the analysis tools for hand calculations above Program based on matrix PHWKRGVEXWRQO\DQDO\]HARRU assemblies for gravity loads Two d Two-dimensional frames and walls Programs based on matrix methods with iterative capability Three-dimensional structure 3URJUDPVEDVHGRQ¿QLWH element methods with iterative capability Three-dimensional structure Beyond the scope of this Handbook Information can downloaded from the ACI website; refer to Table of Contents. requirements (Deierlein et al. 2010). This handbook does not include discussion of the inelastic analysis approach. 3.2.4 Strut-and-tie-The strut-and-tie method in Chapter 23, ACI 318-14, is another analysis method that is permitted by ACI 318. This method also provides design provisions, it is considered both an analysis and design method. This method is applicable where the sectional strength assumptions in ACI 318, Chapter 22, do not apply for a discontinuity region of a member or a local area. 3.2.5 Analysis types and tools—\$&,LGHQWL¿HVWKUHH JHQHUDO W\SHV RI DQDO\VLV VHH 6HFWLRQ ¿UVWRUGHU American Concrete Institute – Copyrighted © Material – www.concrete.org Structural Analysiss 3.1 -Introduction Structural engineers mathematically model reinforced concrete structures, in part or in whole, to calculate member moments, forces, and displacements under the design loads WKDWDUHVSHFL¿HGE\DVWDQGDUGVXFKDV\$6&(,QDOO conditions, equilibrium of forces, and displacements under the design loads WKDWDUHVSHFL and the design loads WKDWDUH stiffnesses values of individual members for input into the model, under both service loads, are discussed in detail in ACI 318-14, Chapter 6. The factored loads, are discussed in detail in ACI 318-14, Chapter 6. against commonly accepted serviceability limits. 26 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. 3.3.3-Frame analyzed by portal method. linear elastic; 2) second-order linear elastic; and 3) secondorder inelastic. Table 3.2.5 shows some common analysis tools used for different analysis methods, loads, and systems. 3.3-Hand calculations 3.3.1 General-Before computers became widely availDEOHGHVLJQHUVXVHGVLPSOLiHGFRGHHTXDWLRQVWRFDOFXODWH gravity design moments and plications, the shears due to lateral forces. In very limited applications, design of an entire building using hand calculation calculations is still ng cod possible with today's set of building codes. For the large ZHYH D KDQG FDOFXODWLRQ PDMRULW\ RI EXLOGLQJ GHVLJQV KRZHYHU design approach is not practical due to the large number and mbi ns necessary to fully lly complexity of design load combinations meet ASCE 7-10 requirements. fo moment ment and shear—The shear—The 3.3.2 Code design equations for HIX SXUSRVHVRISUH PLVLPSOL¿HGFRGHHTXDWLRQVDUHXVHIXOIRUSXUSRVHVRISUHOLPLg for designing iso nary estimating or member sizing, isolated members or subassemblies, and to complete rough checks of computer program output. Because these equations and d expressions are easy to incorporate into electronic spreadsheets and equation solvers, they continue to be helpful. In the member chapters of this handbook, examples of hand calculations are provided. 3.3.3 Portal method was commonly used before computers were readily available to calculate a frame's moments, shears, and axial forces due to lateral forces (Hibbeler 2015). This method has been virtually abandoned as a design tool with the widespread use of commercial design software programs. The portal method has
limitations as stated in the assumptions and considerations that follow, but is still a useful tool for the designer. With complex, three-dimensional modeling becoming commonplace, there is always a chance of modeling error. The portal method allows the designer to independently and quickly ¿QGDSSUR[LPDWHPRPHQWVDQGVKHDUVLQDIUDPH7KLVFDQ be very useful for spot-checking the program results (Fig. 3.3.3). The basics assumptions to the portal method are: (a) Apply only the lateral loads (c) Shear at each column is based on plan tributary area G ,QAHFWLRQSRLQWVDUHDVVXPHGWREHORFDWHGDWPLGKHLJKW of column and midspan of beams (e) Shear in the beam is the difference between column D[LDOIRUFHVDWDMRLQW (f) Beam axial force is to be zero These assumptions reduce a statically indeterminate problem to a statically determinate problem to a statically indeterminate problem to a statically determinate one. large changes in member VL]HVFDQFDXVH VL]HVFDQFDXVHPHPEHUPRPHQWVWRGLIIHUVLJQL¿FDQWO\ hose calculated by a computer analysis. from those calculated (b) Thee lat lateral deformation will be larger than the lateral deformation is ignored. .4 om program 3.4—Computer programs 3.4—Computin power and structural software 3.4.1 Gen General—Computing KD DQ JJQL¿FDQ KDYHDGYDQFHGVLJQL¿FDQWO\IURPWKHWLPHFRPSXWHUVZHUH o the des introduced to designer. Numerous complex computer ms and specialized spe programs analysis tools have been developed taking takin advantage of increasing computer speeds. &XUUHQWO\GHVLJQHUVFRPPRQO\XVH¿QLWHHOHPHQWDQDO\VLVWR design structures. A multistory building only takes minutes of computer software has also greatly improved: user interfaces have become more intuitive; members can be automatically meshed; and input and output data can be reviewed graphically and tabular in a variety of preproJUDPPHGRUXVHUGH¿QHGPHQXV Although three-dimensional models are becoming commonplace, many engineers still analyze the building as a series of twodimensional frames. Matrix methods mentioned in Table 3.2.5 are programs based on the direct stiffness method. Simpler programs model the structure as GLVFUHWHPHPEHUVFDQEH divided into multiple elements to account for changes in member properties along its length. The two-dimensional frames. stiffness method is relatively easy to program and evaluate. A more sophisticated use of the direct stiffness method is the ¿QLWHHOHPHQWPHWKRG7KHVWUXFWXUHLVPRGHOHGDVGLVFUHWH elements connected at nodes. Each member of the struct- American Concrete.org ture consists of the direct stiffness method is the constant of the struct- American Concrete.org ture consists of the struct- American Concrete.org ture construction Concrete.org ture constructice.org ture constructice.org ture constructice.org t multiple rectangular elements, which more accurately determine the behavior of the members as an assembly of these discrete elements is called "meshing" and can be a time-consuming task. A large amount of data is generated from this type of analysis, which may be tedious for the designer to review and process. )RUVWUDLJKWIRUZDUGGHVLJQVDGHVLJQHUPD\PRUHHI¿FLHQWO\ analyze the structure by dividing it into parts and using less complicated programs to analyze the structures are often symmetrical with regularly spaced columns in both directions. There may be a few isolated areas of the structure where columns are irregularly spaced. These columns can be designed separately for gravity load and checked for deformation compatibility ZKHQVXEMHFWHGWRWKHH[SHFWHGRYHUDOOODWHUDOGHAHFWLRQRI the structure. Buildings designed as moment-resisting frames can often be effectively modeled as a series of parallel planar frames. The complete structure is modeled using orthogonal sets RI FURVVLQJ IUDPHV & RPSDWLELOLW\ RI YHUWLFDO GHAHFWLRQV metry of beams at crossing points is not required. The geometry HP )RU VO FDQ YDU\ GHSHQGLQJ RQ WKH ARRU V\VWHP VODEFROXPQ moment frames, it may be possiblee to m model according to the equivalent frame method in Sect Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, with the limits on geometry given en simpler ler to ignore the slab of ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.12 method in Sect Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.12 method in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is permitted to m el For beam-column moment frames, model om iven in Section 8.11 in ACI 318; however, it is pe often gl For or beams in inte meand model the beams as rectangles. intermesumption of a rectctdiate or special moment frames,, th the assumption rv f tto S angular sections 18.4.2.3, 18.6.5.1, and 18.7.3.2 in ACI 318. 7KHVWLIIQHVVRIWKHEHDPFROXPQMRLQWLVXQGHUHVWLPDWHG mn if the beam spans are assumed to extend between column centerlines and the beam is modeled as prismatic along the entire span. Many computer programs thus allow for the beam to be modeled as spanning between faces of columns. To do this, a program may add a rigid zone that extends from the face of the column to the column centerline. If the program does not provide this option, the designer can increase the beam stiffness in the column region 10 to 20 percent to account for this change of rigidity (ACI 442-71). Walls with aspect ratios of total height to width greater than 2 can sometimes be modeled as column elements. A thin wall may be too slender for a conventional column analysis, and a more detailed evaluation of the boundary elements and panels may be necessary. Where a beam frames into a wall that is modeled as a column element, a rigid link should be provided between the edge of the wall and centerline of the ZDOO\$OO ZDOOV FDQ EH PRGHOHG XVLQJ ¿QLWH HOHPHQW DQDO\VLV:KHUHD¿QLWHEHDPHOHPHQWIUDPHVLQWRD¿QLWHZDOO element, rotational compatibility should be assured. :DOOV ZLWK RSHQLQJV FDQ EH PRUH GLI¿FXOW WR DQDO\]H\$ wall with openings can be modeled as a frame, but the rigidity RIWKHMRLQWVQHHGVWREHFDUHIXOO\FRQVLGHUHG6LPLODUWRWKH EHDPFROXPQ MRLQW PRGHOLQJ GLVFXVVHG SUHYLRXVO\ D ULJLG link should be modeled from the centerline of the wall to the 27 edge of the opening (Fig. 3.4.2a). Finite element analysis and the strut-and-tie method, however, are more commonly used to analyze walls with openings. For lateral load analysis, all of the parallel plane frames in a building are linked into one plane frame to enforce lateral deformation compatibility. Alternately, two identical frames can be modeled as one frame with doubled stiffness, obtained by doubling the modulus of elasticity. Structural walls, if SUHVHQW VKRXOG EH OLQNHG WR WKH IUDPHV DW HDFK ARRU OHYHO (Fig. 3.4.2b). Note that torsional effects need to be considered after the lateral deformation compatibility analysis is run. For seismic loads, rigid diaphragms are required to account for accidental torsion according to Fig. 27.4-8 in ASCE 7-10. 3.4.3 Three-dimensional modeling -A three-dimensional model allows the designer to observe structural behavior that a two-dimensional model would not reveal. The effects of structural irregularities and torsional response can be directly analyzed. Current computer software that provides WKUHHGLPHQVLRQDO PRGHOLQJ RIWHQ XVH ¿QLWH HOHPHQW DQDOyysis with automatic meshing. These high-end programs are capable of run running a modal response spectrum analysis, mic response hist seismic history procedures, and can perform a host e-consuming mathematical tasks. GXFH FRPSXWDWLRQ PSXWDWLRQ PSXWDWLRQ WLPH FRQFUHWH ARRUV DUH VRPH7R UHGXFH time timesconsuming mathematical tasks. mode modeled as rigid dia diaphragms, reducing the number RIG\Q PLF VRIIUHHG RIG\QDPLFGHJUHHVRIIUHHGRPWRRQO\WKUHHSHUÀRRUWZR i tal ttranslations ons and one rotation about a vertical horizontal xis). ASCE SCE 7-10 allows for fo diaphragms to be modeled as axis). rigid if the ffollowing ing condi conditions are met: ((a) For seismic i c loading di loading, no structural irregularities and the -depth ratios are 2 or less (Section 12.3.1.2 in ASCE 7-10) (b) For w wind loading, the span-to-depth ratios are 2 or less (Section 27.5.4 in ASCE 7-10) If a rigid diaphragm is assumed, the stresses in the diaphragm are not calculated and need to be derived from the UHDFWLRQVLQWKHZDOOVDERYHDQGEHORZWKHARRU\$VHPLULJLG diaphragm can also be helpful in analyzing torsion effects. For more information on torsion, refer to the code and commentary in Section 12.8 of ASCE 7. 3.5—Structural analysis in ACI 318 3.5.1 Arrangement of live loads—Section 4.3.3 in ASCE 7 states that the
"full intensity of the appropriately reduced live load applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for if it produces a more unfavorable load effect than the same intensity applied over the full structure or member shall be accounted for intensity applied over structure or member shall be acco member." This is a general requirement that acknowledges greater moments and shears may occur with a pattern load than with a uniform load. There have been a variety of methods used to meet this requirement. Cast-in-place concrete is inherently continuous, and Section 6.4 in ACI 318 provides acceptable arrangements of pattern live load. IRUFRQWLQXRXVRQHZD\DQGWZRZD\ARRUV\VWHPV American Concrete Institute - Copyrighted © Material - www.concrete.org Structural Analysiss CHAPTER 3—STRUCTURAL ANALYSIS 28 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 3.4.2a—Element and frame analogies. Fig. 3.4.2b—Idealization for plane frame analysis. 3.5.2 6LPSOL¿HG PHWKRG RI DQDO\VLV IRU QRQSUHVWUHVVHG continuous beams and one-way slabs—Section 6.5 in ACI 318 provides approximate equations for conservative design PRPHQWVDQGVKHDUVZKLFKJUHDWO\VLPSOL¿HVWKHGHVLJQRI FRQWLQXRXV ÅRRU PHPEHUV 7KLV PHWKRG LV SUREDEO\ XVHG more ofter to estimate initial member sizes for computer LQSXWRUIRULQLWLDOFRVWHVWLPDWHVWKDQIRU¿QDOGHVLJQ 3.5.3 First-order analysis are provided in Section 6.6 in ACI 318. 3.5.3.1 Section properties for elastic analysis are given in Table 6.6.3.1.1(a) of ACI 318. The PRPHQW RI LQHUWLD YDOXHV KDYH D VWLIIQHVV UHGXFWLRQ []k of 0.875 already applied. These properties are acceptable for the analysis, the moment of inertia values in Table 6.6.3.1.1(a) can be multiplied by 1.4. Table 6.6.3.1.1(b) offers a more accurate estimation of stiffness by including the effects of axial load, eccentricity, reinforcement ratio, and concrete compressive strength. These equations can also be used to calculate member stiffness at factored load levels by using the factored axial load and moment, as presented, but the equations can be used to calculate member stiffness for any given axial load and moment. These moment-of- LQHUWLDHTXDWLRQVDOVRKDYHWKHVWLIIQHVVUHGXFWLRQI for all members in a lateral load analysis. This is helpful for hand-calculation methods such as the portal method. It is important to note that the stiffness reduction factor used for moment of inertia discussed previously is for global building behavior. The moment of inertia for second-order effects related to an individual column or wall should have DVWLIIQHVVUHGXFWLRQ[]k of 0.75, as discussed in R6.6.4.5.2 of ACI 318. 3.5.3.2 Slenderness effects<sup>2</sup> ¿UVWRUGHU DQDO/VLV LQ ACI 318 assumes that only primary stresses are calculated. 6HFRQGDU\ VWUHVVHV FDXVHG E\ WKH ODWHUDO GHAHFWLRQ FDXVHG by the design loads are not calculated. First-order analysis is typical when hand-calculated. First-order analysis is typical when hand-calculated. P" HIIHFWV ZKLFK DUH WKH second-order moments caused by vertical loads acting on the EXLOGLQJ (VODWHUDOO\GHIRUPHGFRQ¿JXUDWLRQ)LJ 7R approximately account for these secondary effects, a moment PDJQL¿HULVDSSOLHGWR¿UVWRUGHUFROXPQGHVLJQPRPHQWV American Concrete Institute – Copyrighted © Material – www.concrete.org )LJ<sup>2</sup>3"HIIHFWV 7KHGHVLJQHUPXVWDFFRXQWIRUVOHQGHUQHVVLQD¿UVWRUGHU DQDO\VLV )LJXUH 5 LQ\$&, SURYLGHV D ARZ FKDUW that illustrates the options to account for slenderness. In summary, slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requirement of Section 6.2.5 in ACI 318. If slenderness can be neglected if the column or wall meets the requ cannot be neglected, the next step is to determine if way. If the the building story being analyzed is sway or nonsway, story is nonsway, the column or wall end moments are only PHPEHU PDJQL¿HGIRUPįHIIHFWVDORQJWKHPHPEHU,IWKHVWRU\LV RPHQW DUH PDJQL¿HGIRUPįHIHFWVDORQJWKHPHPEHU,IWKHVWRU\LV RPHQW DUH PDJQLĮHGIRUPįHIHFWVDORQJWKHPHPEHU,IWKHVWRU\LV RPHQW DUH PDJQLĮHGIRUPĮHIHFWVDORQJWKHPHPEHU,IWK P"HIIHFWVDORQJWKHPHPEHUPi DQGDWWKHHQGVGXHWR story drift (P" DU VLVDOORZVIRUV SHU3.5.3.3 Superposition<sup>2</sup>/LQHDUDQDO\VLVDOORZVIRUV SHU3.5.3.3 Superposition<sup>2</sup>/LQHDUDQDO\VLVDOORZVIRUV SHU3.5.3.3 Superposition to be used when combining ations. The des ner helpful when performing hand ccalculations. designer or calculates the member moment, sh shear, and axial loa load for each load. The reactions are then superimposed according to the applicable load combination. Many hand-calculation WRROV VXFK DV WKH PRPHQW PDJQL¿FDWLRQ PHWKRG DVVXPH that the designer is performing a linear analysis with superposition of multiple load effects. 3.5.3.4 Redistribution of moments—ACI 318 allows for WKH GHVLJQHU WR DGMXVW GHVLJQ VODE DQG EHDP PRPHQWV DQG shears by taking advantage of the ductility provided through the code detailing is required IRU FRQWLQXRXV ¿EHUV DW VXSSRUWV DQG PLGVSDQ 0RPHQW redistribution can be very helpful in creating economical GHVLJQV)RUH[DPSOHLQD¿QDOGHVLJQPRPHQWUHGLVWULEXtion may permit the designer to specify uniform beam sizes over multiple beam spacing is not uniform, some beam design moments may be slightly lower than the EHDPUHTXLUHGPRPHQWV2QFHVWHHO\LHOGLQJKDVGHYHORSHG at factored loads, however, the redistribute to regions that have not yet yielded, and the beam design moments will satisfy the beam required moments throughout the multiple beam spans. 3.5.4 Second-order analysis—In Section 6.7 of ACI 318, a second-order analysis assumes that the effect of loads on the laterally deformed structure is included in the computer analysis. The initial P"HIIHFWVROWKHPHPEHUGXHWRVWRU\ drift are computed. A computer algorithm then automatically FDUULHVRXWDVHULHVRILWHUDWLYHDODO\VHVXVLOIWKHVROXWLROFROYHUIHVWRWKH¿ODOVHFROGDU\ 29 moments. Note that linear material properties are used with this method, but the results of a second-order analysis is a nonlinear solution. This is referred to as "geometric nonlinearity." This means that the load cases cannot be computed separately and then combined for the calculation of the secondary moments. Software should be checked to determine how it accounts for P"HIIHFWV6RIWZDUHFDQHDVLO\FDOFXODWHWKHDGGLWLRQDO moment due to building lateral deformation but some software does not calculate the secondary moments along the member is modeled by two elements, the designer must account for the smaller stiffness reduction factor for the moment of inertia (refer to Section 3.5.3.1), because GHÀHFWLRQ DORQJ WKH PHPEHU LV D ORFDO HIIHFW %HFDXVH RI WKH GLI¿FXOW\ RI DSSURSULDWHO\ FDSWXULQJ WKH VHFRQGDU\ moment along the column length, many programs calculate the secondary moments due to lateral deformation
and use 6.6.4.5 of ACI 318 in a post-processing program to account for the second-order analysis—The consideration of the no nonlinearity of the st nonlinearity stress-strain curve for concrete and steel ement, particularly ant in seis become im important seismic analysis. Nonlinearities in stru al re e, whethe structural response, whether arising from material properties as for or co concrete or steel, lo loading conditions (for example, i l load ad effects on bendin axial bending stiffness) le, moments) are best handled by nume cal it numerical iterativee or stepstep-by-step procedures. For inelastic d analysis, nalysis, l i tthe h principle of supe position should second-order Nonlinea analysis is beyond the scope of this not be used., Nonlinear book, Sever Handbook, Several references that provide further information on nonlinear n analysis are ASCE 41-13, FEMA report, and Deierlein et al. (2010). 3.5.6 Finite element analysis<sup>2</sup>7KH¿QLWHHOHPHQWDQDO\VLV of concrete structures is permitted by ACI 318 and can be XVHGWRVDWLVI\HDFKWKH¿UVWRUGHUHODVWLFVHFRQGRUGHUDQG inelastic second-orde analyses as long as the element types are compatible with the response required. Section 6.9 in ACI 318 was added to acknowledge that ¿QLWHHOHPHQWDQDO/VLV 0DQ/ SURJUDPV DUH EDVHG RQ ¿QLWH HOHPHQW analysis and have sophisticated auto-mesh capabilities. Finite element analysis is a tool that may be used for either linear or nonlinear analyses, but care should be exercised in selecting element types, numerical solver methods, and nonlinear element properties. 3.6—Seismic analysis For seismic loads, the structure may go through multiple F\FOHVRIVLJQL¿FDQWLQHODVWLFGHIRUPDWLRQV\$6& (SURYLGHV WKH HTXLYDOHQW ODWHUDO IRUFH (/) DQDO\VLV SURFHGXUH (Section 12.8 of ASCE 7) to allow a linear elastic analysis even though the structure will actually behave inelastically. 7KH(/)SURFHGXUHLVDFRPPRQO\XVHGGHVLJQPHWKRGDQG LVDGHTXDWHIRUPRVWVWUXFWXUHV7KH(/)DQDO\VLVDVVXPHV American Concrete Institute - Copyrighted © Material - www.concrete.org Structural Analysiss CHAPTER 3—STRUCTURAL ANALYSIS 30 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) factor Cd (Fig 3.6) used in analysis. These factors account for the difference between the actual expected design forces and displacements and the estimated behavior. Detailed explanations of these factors and their application can be found in ASCE 7 and FEMA P-750. For reinforced concrete structures, ACI 318 provides structures with the ability to deform inelastically by enforcing special seismic detailing requirements. The seismic detailing requirements in Chapter 18 of ACI 318 are additive to the detailing requirements in the member chapters, or the seismic detailing requirements supersede the member chapter requirements. The detailing requirements IRU D SDUWLFXODU /)56 QHHG WR EH DSSOLHG HYHQ LI VHLVPLF loads do not govern the required strength of the structure. Fig. 3.6-Inelastic force-deformation curve (SEAOC Seismology Committee 2008). an approximately uniform distribution of mass and stiffness along the building height with minor torsional effects. 6WUXFWXUHV DQDO\]HG XVLQJ WKH (/) SURFHGXUH IRU VHLVPLF loads must comply with several limitations. Depending on g the Seismic Design Category (SDC), the building height rities, a Modal and type, and the type of structural irregularities, Response Spectrum analysis (Sectionn 12.9 of ASCE 7), may be required. RegardRFH VDFFHSWDEOHI OO OHVVRILUUHJXODULWLHVWKH(/)SURFHGXUHLVDFFHSWDEOHIRUDOO 1 ft inn height. buildings in SDC A and B up too 160 ov limitations relat d to Section 12.3 of ASCE 7 provides related HV \$6&( &( GHVFULEHV &YH sio 2) reentrant co er; horizontal irregularities: 1) torsion; corner; ut ne offset; an 3) diaphragm discontinuity; 4) out-of-plane and 5) QRQSDUDOOHO V\VWHPV \$6&( DOVR GHVFULEHV ¿YH YHUWLFDO irregularities: 1) stiffness-soft story; 2) weight; 3) vertical ng geometry; 4) in-plane discontinuity in lateral force-resisting V\VWHPV/)56V DQG GLVFRQWLQXLW\LQODWHUDOVWUHQJWK %HFDXVHWKHDFWXDOVWUXFWXUHZLOOXQGHUJRJUHDWHUGHAHFWLRQVDQGVWUHVVHVWKDQSUHGLFWHGE\DQ(/)DQDO\VLV\$6&(7 and ACI 318 have additional requirements to account for WKHDQWLFLSDWHGEHKDYLRU/DWHUDOIRUFHUHVLVWLQJV\VWHPVIRU FRQFUHWH VWUXFWXUHV DUH GH¿QHG LQ\$6&( DQG RI\$&,(DFK/)56KDVDUHVSRQVHPRGL¿FDWLRQFRHI¿FLHQWRRYHUVWUHQJWKIDFWRUDQGGHÀHFWLRQDPSOL¿FDWLRQ REFERENCES American Society of Civil Engineers \$6&(20LQLPXP'HVLJQ/RDGVIRU%XLOGLQJVDQG 2WKHU6WUXFWXU 2WKHU6WUXFWXUHV 6&(26H \$6&(26HLVPLF(YDOXDWLRQDQG5HWURdW5HKDELOListing Buildings Fed cy Manage Federal Em Emergency Management Agency FEM A 44 Improvem of Nonlinear Static Seismic FEMA 440-05—Improvement l s Pr Analysis Proceduress FEM A P-750-NEHRP P NEHRP Recommended Seismic ProviFEMA VLRQV U1H LOGLQJVDQ VLRQVIRU1HZ%XLOGLQJVDQG2WKHU6WUXFWXUHV eference authored references erlein, G. G.; G Reinhorn, A. M.; and Willford, M. R., Deierlein, 2010 "N 2010, "Nonlinear Structural Analysis for Seismic Design," NEHRP Seismic Design, "NEHRP Seism GCR 10-917-5), National Institute of Standards and Technology, Gaithersburg, MD, 36 pp. Hibbeler, R., 2015, Structural Analysis, ninth edition, Prentice Hall, New York, 720 pp. 6(\$2& 6HLVPRORJ\ & RPPLWWHH 3\$ %ULHI \*XLGH to Seismic Design Factors," Structure Magazine, Sept., pp. 30-32. aspx?articleID=756 American Concrete Institute Copyrighted © Material – www.concrete.org 4.1—Introduction Durability of structural concrete is its ability, while in service, to resist possible deterioration due to the surrounding environment, and to maintain its engineering properties. This can be accomplished by proper proportioning and VHOHFWLRQRIPDWHULDOVIRUWKHFRQFUHWHPL[WXUHGHVLJQ2WKHU DVSHFWVLQAXHQFLQJGXUDELOLW\LQFOXGHUHLQIRUFLQJEDUVHOHFtion, detailing, and construction practices. The ACI 318 Code provides minimum requirements to protect the structure against early serviceability deterioration. Depending on Fig. 4.1.1a-Permeability versus capillary porosity for cement paste. Different symbols designate different cement pastes (Powers 1958). exposure conditions, structural concrete may be required to resist chemical or physical attack, or both. The attack mechanisms the Code covers include exposure to freezing and thawing, soil and water sulfates, wetting and drying, and reinforcement corrosion due to chlorides. All these failure mechanisms themselves and how different concrete-making mateULDOVLQFOXGLQJDGPL[WXUHVDQGWKHLUSURSRUWLRQVLQAXHQFH concrete's resistance to these mechanisms. 4.1.1 Permeability<sup>2</sup>3HUPHDELOLW\FDQEHGH¿QHGDV<sup>3</sup>WKH HDVHZLWKZKLFKDÀXLGFDQÀRZWKURXJKDVROLG (liquid, gas, or ions)" (ACI 365.1R; Kosmatka DQG :LOVRQ /RZSHUPHDELOLW\ FRQFUHWHV DUH PRUH resistant to resaturation freezing and thawing, sulfate and chloride ion penetration, and other forms of chemical attack (Kosmatka and Wilson 2011). Concrete permeability is related to poro porosity (volume of voids/pores in concrete) and QQHFWLYLW\RIWKHVH FRQQHFWLYLW\RIWKHVH FRQQHFWLYLW FRQWH FRQWHY FRQWHY FRQWHY FRWHY FR ppores cement pa paste are relevant to concrete durare responsible for the transport properties of FRQ \$& 5.RVP DWNDDQG:LOVRQ 7KHLQAXence of o capi rosity in ccement paste on permeability was capillary porosity t by Powers (Fig. Fig. 4.1.1 reported 4.1.1a) (Powers 1958). Con rete permeability, bility, dif Concrete diffusivity, and electrical conductivity can an bbe reduced ced with llower water-cement ratios (w/c), the use off S d extended t SCMs,, and moist curing (Kosmatka and ). Effects of the w/c and duration of the moist Wilson 2011). When water freezes in concrete, it causes cement paste to dilate destructively by generating hydraulic and osmotic pressure. While hydraulic SUHVVXUH IRUFHV ZDWHU DZD\ IURP WKH IUHH]LQJ ZDWHU¿OOHG capillary cavities, osmotic pressure is produced by water Fig. 4.1.1b—(left) Effect of w/cDQGLQLWLDOFXULQJRQK\GUDXOLFZDWHU SHUPHDELOLW\DQG (right) effect of w/c and curing duration on permeability (leakage) of mortar (Kosmatka and Wilson 2011). American Concrete Institute – Copyrighted © Material – www.concrete.org Durability CHAPTER 4—DURABILITY 32 THE REINFORCED CONCRETE DESIGN HANDBOOK -SP-17(14) Fig. 4.1.3—Effect of w/c ratio on sulfate resistance for different ASTM C150/C150M types of cement, (lower visual rating indicates better resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, and curing/drying on resistance) (Stark 1989). Fig. 4.1.2—Effect of w/c, air-entrainment, ai (Powers 1958). Hydraulic pressures in cement paste are generated by the 9 percent expansion of water when it freezes and changes to ice. For the freezing to take place, a capillary has to reach its FULWLFDOVDWXUDWLRQSHUFHQW¿OOHGZLWKZDWHU.RVPDWND DQG:LOVRQ
2VPRWLFSUHVVXUHVGHYHORSGXHWRYDULRXVPDWND DQG:LOVRQ 2VPRWLFSUHVYDWND DQG:LOVRQ 2VPRWLFSUHVY concentrations of alkali solutions in the paste. When pressure in concrete due to freezing and thawing. The resulting damage occurs, especially if concrete, some damage occurs, especially if concrete due to freezing and thawing. thawing cycles. Deterioration due to freezing and thawing can appear in the form of cracking, scaling, disintegration, or all three of these (Kosmatka and Wilson 2011). /RZ SHUPHDELOLW\ DQG ORZ DEVRUSWLRQ DUH PDLQ FKDUDFteristics needed for concrete to be frost resistant, while airentraining admixtures are used to control the pressure generated in concrete paste during freezing-and-thawing cycles. In other words, high resistance to freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and thawing is associated with entrained air, low w/c, and a drying period freezing and th Sulfates—Sulfates with hydrated DQG XFH VXI&FLHQW LHQW SUHVVX DQG LQGXFH SUHVVXUH WR GLVLQWHJUDWH WKH FRQFUHWH Howe er, formation f n of new crystalline substances due to However, eacti artly responsible for the expansion. If water can an fr fuse out oof capillaries in the cement paste, freely diffuse the volume v ume of growing owing cr crystals cannot exceed the space m. A At th h same time, however, the swelling available to them. the n also arise from the diffusion of the sulfate salts pressure can he gel pores, pores which disturbs the equilibrium between into the the gel an and its surrounding liquid phase, resulting in the movement of more water from the outside into the gel pores (Hewlett 1998). Although ordinary portland cements will not stop the sulfate attack, either (Fig. 4.1.3). Resistance to sulfate attack can be greatly increased by decreasing the permeability of concrete through reduction of the watercementitious material ratio (w/cm) (Stark 1989). 4.1.4 Corrosion—Alkaline nature of concrete (pH greater than 13) will induce formation of a passive, noncorroding layer on reinforcing steel. If, however, chloride ions are present in concrete, they can reach and disrupt that layer and lead to corrosion of steel in the presence of water DQG R[\JHQ 2QFH FRUURVLRQ LQLWLDWHV FRUURVLRQ SURGXFWV form and may cause cracking, spalling, or delamination of concrete. This allows for easier access of aggressive agents to the steel surface and increases the rate of corrosion. Cross-sectional area of the corroding steel will decrease and the load-carrying capacity of the member will be reduced (Neville 2003). Chlorides can be introduced to concrete org CHAPTER 4—DURABILITY 4.2 (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 4—DURABILITY 4.2 (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 4—DURABILITY 4.2 (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or American Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or some admixtures), with deicing chemicals, or american Concrete Institute – Copyrighted © Materials used to produce the mixture (contaminated aggregate or water, or american Concrete Institute), -Background To produce durable structural concrete, concrete materials g and mixture proportions are selected based on design strength ns, and required requirements, anticipated exposure conditions, on of ma service life of the structure. The selection materials and mpani by appropriate mixture proportions has to be accompanied RQWUR WHVWLQJ LQVSHFWLRQ ¿HOG SUDFWLFHV VXFK DV TXDOLW\ FRQWURO DQ QJSUDFWLFHV DQGSURSHUSODFHPHQW¿QLVKLQJDQGFXULQJSUDFWLFHV -14, "The purpo As stated in Section 1.3.1 of AC ACI 318-14, purposee of ic health h and safety by eestababthis Code is to provide for public or strength, gth, stability, serviceser icelishing minimum requirements for ures." Section 44.8 of ability, and integrity of concrete st structures." il requirements i t related ACI 318-14 addresses global durability to material selection for concrete mixtures and corrosion protection of reinforcement. ACI 318-14, Chapters 19 and te 20, provide detailed durability requirements for concrete and reinforcing steel, respectively. ACI 318-14, Chapter 26, GLVFXVVHVZKDWGXUDELOLW\UHTXLUHPHQWVPXVWEHVSHFL¿HGLQ DSURMHFW¶VFRQVWUXFWLRQGRFXPHQWV The Code's durability focus is mainly on concrete resisWDQFHWRÀXLGSHQHWUDWLRQZKLFKLVSULPDULO\DIIHFWHGE\WKH w/cm and the composition of those materials. The use of 6&0V VXFK DV7\SH ) DQG7\SH & A\ DVKHV VODJ FHPHQW silica fume, calcined shale, calcined proportions and composition. In general, SCMs have the following impacts on hardened concrete properties (Kosmatka and Wilson 2011): • Increase long-term strength gain (Type F Å\DVKFDOFLQHGVKDOHVDQGFOD\VORZHUHDUO\VWUHQJWK silica fume and metakaolin increase early strength gain) Reduce permeability and absorption • Improve resistance to corrosion • Increase sulfate resistance (with the exception of Type C À\ DVK ZKLFK PD\ KDYH HLWKHU D SRVLWLYH RU QHJDWLYH effect) • +DYH QR VLJQL¿FDQW LPSDFW RQ DEUDVLRQ UHVLVWDQFH drying creep and shrinkage, and freezing and thawing • May reduce resistance to deicer scaling The Code does not cover all topics related to concrete durability. It does not include recommendations for extreme exposure conditions (that is, acids, high temperature, or H[SRVXUH WR ¿UH DONDOLD]JUHJDWH UHDFWLRQ RU DEUDVLRQ 7KH & RGH FRPPHQWDU\ 5 LGHQWL¿HV WKH LPSRUWDQFH RI preventive maintenance; however, the topic is not explicitly addressed in the Code. Additionally, the Code does not cover ZDWHUSURR¿QJURXWLQHLQVSHFWLRQVFRQGLWLRQDVVHVVPHQWRU service life prediction. Information related to these topics are found in other ACI documents, including: • ACI 201.2R—Guide to Durable Concrete ACI/TMS 216.1—Code Requirements for Determining Fire Resistance of Concrete Against Corrosion • ACI 222.2R—Protection of Metals in Concrete Against Corrosion • ACI 222.2R 222.2R—Report on Corrosion of Prestressing Steels • ACI 222.3R—Causes, Evaluation, E and Repair of Cracks in Concrete Structures Concrete • ACII 224.1R—Causes, 22 —Causes, Evaluation, E and Repair of Cracks in Concrete Structures Concrete • ACII 224.1R—Causes, 22 —Causes, Evaluation, E and Repair of Cracks in Concrete Structures Concrete • ACII 224.1R—Causes, 22 —Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, 22 —Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII
224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Evaluation, E and Repair of Cracks in Concrete • ACII 224.1R—Causes, Evaluation, E and Evaluation Inspection • A ACII 31 311.4R—Guide , 6HUYLFH/ • \$ \$&,5<sup>2</sup>6HUYLFH/LIH3UHGLFWLRQ • A —Guide for C Concrete 562—Cod • ACI 562—Code Requirements for Evaluation, Repair, and Rehabilitation of Concrete Buildings and Com Commentary 4.3—Requirements for concrete in various exposure categories The Code addresses durability by requiring that four exposure categories are: 1. F: concrete member. The four exposure categories are: 1. F: concrete in contact with soil or water containing deleterious amounts of water-soluble sulfate ions; 3. W: concrete in contact with water but not exposed to freezing and thawing, chlorides, or sulfates; 4. C: concrete exposed to conditions that require additional protection against corrosion of reinforcement. Each exposure classes WKDW GH¿QH VHYHULW\ RI WKH H[SRVXUH VWDUWLQ] ZLWK IRU D QHJOLJLEOH H[SRVXUH 2QFH DOO VWUXFWXUDO PHPEHUV DUH assigned exposure classes and the concrete mixtures for these members satisfy those various requirements, the Code's minimum durability requirements are met. 4.3.1 Freezing and thawing (F)—The volume of ice is 9 percent larger than water. As water freezes in satuUDWHG FRQFUHWH FHPHQW SKDVH DQG DJJUHJDWHV DUH VXEMHFW American Concrete Institute - Copyrighted © Material - www.concrete.org Durability through marine exposure (seawater or brackish water). To reduce likelihood of corrosion initiation, total chloride-ion content should not exceed a certain concentration value, referred to as the chloride threshold. A literature review of reported chloride threshold values revealed "that there is no single threshold value, but a range based on the conditions and materials in use" and were found to vary from 0.1 to 1 percent by mass of cement (Taylor et al. 1999). ACI 318-14 limits water-soluble chlorides to 0.15 percent by mass of cement (Taylor et al. 1999). cement for concrete exposed to external chlorides RUVHDZDWHU9DOXHRISHUFHQWWRWDOFKORULGHE\PDVVRI cement is given in British and European Standards (Neville 2003; Whiting et al. 2002). Corrosion of the reinforcing steel in concrete can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum coverate can be reduced or prevented by minimizing the w/cm ratio (permeability), ensuring maximum cover depth of concrete over steel (Stark 2001), and use of corrosion inhibitors or corrosion resistant steel. 33 34 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) to internal pressure, which then causes concrete tensile stresses. If those stresses are greater than the tensile stresses are greater than the tensile stresses are greater than the tensile stresses. expansion after many cycles of freezing and thawing may lead to signifLFDQWFRQFUHWHGDPDJH2QHPHWKRGWRSURWHFWFRQFUHWHIURP freezing-and-thawing damage is to reduce moisture penetration so it does not become critically saturated; however, this is not always possible. The other method is to generate small air bubbles in fresh concrete by addition of an air-entraining admixture, which creates voids for the freezing water to expand into without creating internal stress. The Code requires concrete in structural members exposed to cycles of freezing and thawing to be protected by using airHQWUDLQHG FRQFUHWH\$LU HQWUDLQPHQW VLJQL¿FDQWO\ LPSURYHV resistance of saturated concrete to freezing and thawing. ACI 212.3R provides an in-depth discussion on these materials, their applications, dosage rates, effects on fresh and KDUGHQHGFRQFUHWHDQGRWKHUIDFWRUVWKH\LQAXHQFH 7KHVSHFL¿HGDPRXQWRIDLUHQWUDLQPHQWGHSHQGVSULPDULO\ on frequency of exposure to water (exposure class), but also on nominal maximum aggregate size and concrete compress size and concrete mixtures with smaller nominal maximum aggregate /8 in. aggregate requires size. For example, concrete mixtures with smaller nominal maximum aggregate /8 in. content than concrete with 2 in. aggre.1) The Code requires that at gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO WKHOLFHQVHGGHVLJQSURIHVVLRQDO/'3 VSHFLI\WKHQRP QDO WKHQT A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO WKHQT A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO WKHQT A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO WKHQT A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFLI\WKHQRP QDO A gate (ACI 318-14, Table 19.3.3.1). DO VSHFL as cons ucdepends on locally available aggregates, construction are given in Section 26.4.2.1 of ACI 318-14. Table 19.3.3.1 ng lists target air content for Classes F1, F2 and F3, depending on the nominal maximum aggregate size. Another factor affecting selection of target air content is compressive strength. An air content reduction of 1 percent LVDOORZHGIRUFRQFUHWHZLWKVSHFL¿HGFRPSUHVVLYHVWUHQJWK exceeding 5000 psi (ACI 318-14, Section 19.3.3.3). The reason for air content reduction is that concretes with higher strengths are characterized by lower w/cm and reduced porosity, which improve resistance to freezing-and-thawing cycles. For example, a structural member in Exposure Class F2 with 1/2 in. nominal maximum aggregate size requires concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent (or 6 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent for concrete with a target air content of 7 percent for concrete with a target air concrete with a tar FRQWHQW LV GLI¿FXOW WR DFKLHYH WKH &RGH allows tolerance for air content in as-delivered concrete of ±1.5 percent (or 4.5 to 7.5 percent for concrete of ±1.5 per with compressive strength exceeding 5000 psi). Additional requirements or limitations, such as minimum compressive strength, minimum w/cm, or limits on cementitious materials, depend on the exposure class assigned to a particular member. Interior members, foundations below the frost line, or structures in climates where freezing temperatures are not anticipated are assigned Exposure
Class F0. These conditions, therefore, do not require air entrainment and there is no limit on maximum w/cm or on the use of cementitious materials. The minimum compressive strength for concrete in Exposure Class F0 is the Code minimum: 2500 psi. Freezing-and-thawing cycles have little effect on concrete that is not critically saturated. Structural members exposed to freezing and thawing cycles, but with low likelihood of being saturated, are assigned exposure class F1. Concrete for this exposure must be air entrained (Table 19.3.3.1 of ACI 318) in case there is occasional saturated. maximum w/cm of 0.55 and at least 3500 psi compressive strength. Exposure Classes F2 and F3 are assigned to concrete in structural members with a high likelihood of water saturation during freezing. The distinction between the two classes is that Class F2 anticipates no exposure to deicing chemicals or seawater, while Class F3 does. Concrete in F2 and F3 exposure classes must be air entrained (Table 19.3.3.1 of ACI 318) an and have a maximum w/cm of 0.45 and 5000 psi, respectively. st se The most severee class of eexposure, F3, also has a limit on cem tiou materials erials ious concrete mixtures, given in ACI T ble 226.4.2.2(b). (b). 318, Table Th umm The summary of requirem requirements for concrete in Exposure Category 4.3 S)—All so 4.3.2 Sulf Sulfate (S)—All soluble forms of sulfate, sodium, ium, or m calcium, potassium, magnesium have a detrimental effect on concrete.. Depending on the sulfate form, they react with ated cement phases and result in formation of ettringite), or softens and loses strength (gypsum). The most effective measure to reduce the effects of sulfate reactions, apart from reducing moisture ingress, is to use cements with a low content of tricalcium aluminate (C3A). A more detailed discussion on sulfate's effect on concrete can be found in ACI 201.2R. Exposure Category S applies to structural members that will likely be affected by external source of sulfates, which predominantly come from exposure to soil, groundwater, RU VHDZDWHU 7KH H[SRVXUH FODVVL¿FDWLRQ FODVV LV VHOHFWHG EDVHG RQ WKH FRQFHQWUDWLRQ RI VXOIDWH LRQV 6242-), which should be determined in accordance with ASTM D516 or ASTM D4130 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for soil samples and with ASTM D516 or ASTM D4130 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for soil samples and with ASTM D516 or ASTM D4130 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for soil samples and with ASTM D516 or ASTM D4130 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for soil samples and with ASTM D516 or ASTM D4130 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV WKH /'30 for zDWHU VDPSOHV 7KH & RGH UHTXLUHV YDPSOHV 7KH & RGH UHTXLUHV 7KH & RGH UHTXLUHV 7KH & RGH UHTXLUHV 7KH WKH /'30 for zDWHU YDPSOHV 7KH & RGH UHTXLUHV 7KH WKH /'30 for zDWHU YDPSOHV 7KH RGH YDPSOHV 7KH WKH /'30 for zDWHU YDPSOHV WR VSHFLI\ WKH H[SRVXUHFODVVE\FRPSDULQ]¿HOGWHVWUHVXOWVZLWKFRQFHQtration ranges in Table 19.3.1.1 of ACI 318. Note that VHDZDWHUH[SRVXUHLVFODVVL;HGDV6HYHQWKRX]KWKHVXOIDWH concentration (in seawater) is usually higher than 1500 ppm. The reason for lower class for seawater is the presence of chloride hibit expansive reaction due to sulfate attack. Class S0 is assigned to concrete in members not exposed to sulfates and there is no restriction on w/cm, or type or limit American Concrete Institute – Copyrighted © Material – www.concrete.org on cementitious materials. The only requirement for concrete FODVVL¿HGDV6LVWKHPLQLPXPFRPSUHVVLYHVWUHQJWKEHDW least 2500 psi. Greater minimum compressive strength and maximum w/cm limits are imposed on concrete in Exposure Classes S1 through S3. For these exposure classes, the type RIFHPHQWLVWKHPDMRUUHTXLUHPHQW A summary of all requirements for concrete in Exposure Category S is listed in Table 19.3.2.1 of ACI 318-14. 4.3.3 In contact with water (W)—The durability of structural members in direct contact with water, such as foundation walls below the groundwater table, may be affected by water penetration into or through concrete. Apart from H[WHUQDOV\VWHPVVXFKDVGUDLQDJHV\VWHPVRUZDWHUSURR¿Q] membranes for foundations, the most effective way to reduce concrete permeability is to keep the w/cm low. Concrete for members assigned to Exposure Class W0 has no unique requirements except that it has a minimum compressive strength of 2500 psi. Concrete in structural members assigned to Exposure Class W1 requires low permeability. Table 19.3.2.1 of ACI 318-14 requires w/cm not to exceed 0.50 and compressive strength to be at least 4000 psi. Note that additional requirements are imposed if the member's durability is to be affectedd by reinforcexposure to cycles of ment corrosion, sulfate exposure, or exposure ations for the design and freezing and thawing. Recommendations construction of water tanks and rese reservoirs are provided in ACI 372R. E re Class W are listed ted Requirements for concrete in Exposure 4. in Table 19.3.2.1 of ACI 318-14. sio off reinforcement may 4.3.4 Corrosion (C)—Corrosion QG VWUXFWXUDO FWXUDO FDSDFLW\ RI I D VLJQL¿FDQWO\ DIIHFW GXUDELOLW\ DQG member. Reinforcement corrosion products (rust) are larger in volume re than the original steel and therefore exert internal pressure on the surrounding concrete, causing it to crack or delamiQDWH\$VLJQL¿FDQWORVVRIUHLQIRUFLQJEDUFURVVVHFWLRQOHDGV to increased steel stresses under service load and reduced member nominal strength. Because moisture and oxygen must be present at the steel surface for corrosion to occur, the quality of concrete and the reinforcing bar cover are of great importance. Corrosion can be mitigated by proper mixture design and construction practices; application of sealers, coatings, or membranes that protect concrete from moisture and chloride penetration; use of corrosion resistant reinforcement; or inclusion of corrosion inhibitors in the mixture to elevate the corrosion threshold concentration. Refer to ACI 222.R, ACI 222.3R, and ACI 212.3R for additional information. Each exposure class within the corrosion exposur concrete-making ingredients—that is, FHPHOWLWLRXVPDWHULDOV¿OHDOGFRDUVHDIIUHIDWHZDWHUDOG admixtures, Because chloride limits are imposed even on concrete in Exposure Class C0, all structural concrete must comply with the Code's maximum chloride ion limits. Chlo-35 ride limits for nonprestressed concrete, expressed as percent of cement weight, are 1 percent for Class C0, 0.30 percent for Class C1, and 0.15 percent for Class C2. Chloride limits, Exposure Classes C0 and C1 have no additional requirements, as there is no limit on w/cm and the minimum compressive strength is 2500 psi. Class C2 requires concrete strength of at least 5000 psi, a maximum w/cm RI DQG UHLQIRUFLQJ VWHHO VSHFL¿HG cover to satisfy the Code's minimum concrete cover provisions. The minimum concrete cover depends on exposure to weather, contact with ground, type of member, type of reinforcement, diameter and arrangement (bundling) of reinforcement, method of construction (cast-in-place or precast), and if the member is prestressed. Tables 20.6.1.3.1, 20.6.1.3.2, and 20.6.1.3.3 of ACI 318-14 provide cover provisions for cast-in-place nonpressessed, cast-in-place nonpressessed, cast-in-place nonpressessed or precast. the design requires bundled bars, FKHFN6HFWLRORI\$&, IRUVSHFL¿FUHTXLUHments. Concrete cover requirements in corrosive environments or other severe exposure conditions are more stringent S and are provided in Section 20.6.1.4 of ACI 318-14. rem ments for con re in Exposure Class C are listed ret Requirements concrete 1 of ACI 3318-14. in Tablee 19. 19.3.2.1 4.4— onc valuation acceptance, and 4.4—Concrete evaluation, tion inspection Dur bility requirements rements aare met once concrete proporDurability tions and d pr es satisfy the minimums set by the Code. properties T he ddelivered deliv li To assure that the concrete achieves the desired KH/'3VK GXUDELOLW\WKH/'3VKRXOGVSHFLI\FRQFUHWHHYDOXDWLRQDQG ptance criteria consistent with ACI 318-14, Section DQG DQ ¿HOG LQVSHFWLRQ FRQVLVWHQW ZLWK \$&, Section 26.13. 4.5—Examples The following examples illustrate one approach of implementing minimum durability requirements of the Code. In some cases, durability requirements for material properties may exceed those of the structural design. This is more likely for severe exposure conditions, which require a minimum compressive strength of 5000 psi. In some cases, SCMs may be required, which may extend setting time and HDUO\DIHVWUHOJWKDOGUHVXOWLOPRGL¿FDWLROVWRFROVWUXFtion schedule. For these reasons, cooperation with engineers experienced with concrete suppliers, is recommended. 4.5.1 ([DPSOH,OWHULRUVXVSHOGHGVODEORWH]SRVHGWR moisture or freezing and thawing— Consider the design of D FDVWLQSODFH QRQSUHVWUHVVHG VODE LQ D PXOWLVWRU\ RI¿FH building. It is located in a climate zone with frequent freezing-and-thawing cycles; however, the slab will be constructed during summer and the temperatures at night during constructed during summer and the temperatures at night during constructed during summer and the temperatures at night during constructed during summer and the temperatures at night during constructed during summer and the temperatures at night during constructed during summer and the temperatures at night during constructed during summer and the
temperatures at night during constructed during summer and the temperatures at night during constructed during summer at night during constructed during summer at night during summer a slab to quickly gain strength to meet the construction schedule. For this reason, calcium chloride American Concrete Institute - Copyrighted © Material - www.concrete.org Durability CHAPTER 4—DURABILITY 36 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) was proposed as an accelerating admixture. The required minimum compressive strength, from structural analysis, is 4000 psi. The slab is 7 in. thick with top and bottom mats of No. 5 bars spaced at 8 in. What additional information VKRXOGEHVSHFL¿HGIRUWKHVODEFRQFUHWHWRPHHWGXUDELOLW\ requirements? Answer:7KH¿UVWVWHSLVWRDVVLJQH[SRVXUHFODVVHVZLWKLQ every exposure category to each structural member or group RIPHPEHUV2QFHH[SRVXUHFODVVHVDUHDVVL]QHGWKH&RGH JXLGHV WKH /'3 WR VDWLVI\ WKH GXUDELOLW\ UHTXLUHPHQWV7KH step-by-step instructions are as follows: Step description/ action item Selection and discussion Code reference spaced at 6 in. What additional information is needed for balcony concrete to meet durability requirements? Answer: Durability requirements are met once the most ULJRURXVUHTXLUHPHQWVRIWKH&RGHDUHVDWLV¿HG7KH¿UVWVWHS is to assign exposure classes within every exposure category WRHDFKVWUXFWXUDOPHPEHURUJURXSRIPHPEHUV2QFHH[SRVXUHFODVVHVDUHDVVL]QHGWKHFRGH]XLGHVWKH/'3WRVHWWKH minimum durability requirements. The step-by-step instructions are as follows: Step description/ action item Selection and discussion Code reference Assign exposure classes within each exposure category F2 (concrete exposed to freezing-and-thawing cycles with frequent exposure to water) S0 (soil not in contact with FRQFUHWHORZDQGLQMXULRXV sulfate attack is not a concern) W0WKHUHDUHQRVSHFL¿F requirements for low permeability) C1 (concrete exposed to moisture but not to an external source of chlorides) Table 19.3.1.1 Assign required minimum sive compressive strength 4500 psi (based on F2); because 450 4500 psi iss greater than design streng h of 4000 psi, the 4500 strength psi governs gover Table 19.3.2.1 Assign minimum oncre cove concrete cover .5 in. (exposed (expo 1.5 to weather, No. 5 bar and sm smaller) Table 20.6.1.3.1 Assign nominal nomin Assign i of maximum size aggregate 2 in. ( $1/3 \times 6$ -in. – slab thi k thickness,  $3/4 \times 6$ -in. – in. aggregate with no air content change] Table 19.3.3.1 and Section R26.4.2.1(a)(5) Assign exposure classes within each exposure cl permeability) C0 (concrete dry or protected from moisture) Table 19.3.2.1 Assign maximum w/cm ed on o all expoNot limited (based on F0) Table 19.3.2.1 Assign minimum concrete cover po 0.75 in. (not exposed to ., N weather, slabs..., No. 11 bars and smaller) le Table 1.3.1 20.6.1.3.1 Assign nominal maximum size of aggregate lea bar 2 in. (3/4 x 3 in. clear t spacing - top and bottom mat, or 1/3 x 7 in. - slab thickness); use 1 in. as readily available 26.4.2.1(a)(4) Assign required air content Not air entrained Table 19.3.2.1 Assign limits on cementitious materials No limits Table 19.3.2.1 Assign limits on calcium chloride ion structure No restriction (based on S0) [Note: chloride ions from the DGPL[WXUHZLOOVL]QL¿FDQWO\ affect measured chloride ion content in concrete.] Table 19.3.2.1 1.00 (based on C0, watersoluble chloride-ion content from all concrete ingredients determined by ASTM C1218/ C1218/M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content from all concrete ingredients determined by ASTM C1218/1218M at age between 28 and 42 days) Table 19.3.2.1 0.30 (water-soluble chloride ion content f maximum water-soluble chloride ion (Cl-) content in concrete, percent by weight of cement Provide guidance on cold weather construction Consult ASTM C94/C94M, ACI 306R, and ACI 301 for guidance on temperature limits for concrete delivered in cold weather. Section 26.5.4.1 4.5.2 ([DPSOH %DOFRO] VODE H[SRVHG WR PRLVWXUH DQG freezing and thawing<sup>2</sup>\$Q /'3 GHVLJQV D FDVWLQSODFH QRQSUHVWUHVVHGEDOFRQ\VODELQDPXOWLVWRU\RI¿FHEXLOGLQJ located in a climate zone with frequent freezing-and-thawing cycles. It is anticipated that the balconies will be exposed to moisture, but not deicing salts. The required minimum compressive strength, from structural analysis, is 4000 psi. The balcony slabs are 6 in. thick with top mat of No. 4 bars 4.5.3 ([DPSOH :DOO IRXQGDWLRQ H[SRVHG WR VXOIDWH soil and deicing salts while in service<sup>2</sup>\$Q/'3GHVL]QVD cast-in-place, nonprestressed foundation wall of a partially underground parking structure. The structure is located in a northern climate zone with frequent
freezing-and-thawing F\FOHVKLJKVXOIDWHVRLOFRQWHQWSHUFHQW6242- by mass) American Concrete Institute - Copyrighted © Material - www.concrete.org and exposure to deicing salts are anticipated as a runoff from the nearby streets and a sidewalk. It is desirable for the foundation wall to guickly gain strength to reduce possible frost damage and to meet the construction schedule. The required minimum compressive strength, from structural analysis, is 4000 psi. The foundation wall is 8 in. thick with inside face and outside face mats of No. 4 bars spaced at 12 in. What DGGLWLRQDO LQIRUPDWLRQ VKRXOG EH VSHFL¿HG IRU IRXQGDWLRQ wall concrete to meet durability requirements? Answer:7KH¿UVWVWHSLVWRDVVLIOHISRVXUHFODVVHVZLWKLO every exposure category to each structural member or group RIPHPEHUV2OFHHISRVXUHFODVVHVDUHDVVLIOHGWKHFRGH IXLGHVWKH/3WRVHWWKHPLOLPXPGXUDELOLW\UHTXLUHPHOWV The step-by-step instructions are as follows: Step description/ Action item Selection and discussion Code reference Assign exposure classes within each exposure to deicing chemicals) S3 (structural concrete members in direct contact with soluble sulfates in soil or water) act with W1 (concrete in contact meabil is water and low permeability required) pos to C2 (concrete exposed n ex moisture and an external source m deicing g of chlorides from chemicals) Table 19.3.1.1 Assign minimum compressive strength d C2); 5000 psi (based on F3 and han because 5000 psi is gr greater than design strength of 4000 psi, 5000 psi governs Table 19.3 1 19.3.2.1 Assign maximum w/cm 0.40 (based on F3 and C2) Table 19.3.2.1 Assign minimum concrete cover 2.0 in. - outside face of wall (1.5 in. cover is listed in Table 20.6.1.3.1 for exposure to weather or in contact with ground for No. 5 bar and smaller: cover increased to 2.0 in. based on 20.6.1.4.1) 3/4 in. - inside face of wall (side of the wall not exposed to weather or in contact with ground) Table 20.6.1.3.1 20.6.1.4.1 Assign nominal maximum size of aggregate 1.5 in. ( $1/5 \times 8$  in. – wall thickness,  $3/4 \times 3$ -1/4 in. clear bar spacing – between interior and exterior mats of reinforcing steel); use 1.5 in. Section 26.4.2.1(a)(4) Assign required air content 5.5% ± 1.5% (for 1.5 in. Table 19.3.3.1) aggregate and F3 class) [Notes: 1. Changing to lower nominal maximum aggregate size will require higher air content; \$LUFRQWHQWUHGXFWLRQRI WR" LVDOORZDEOHLI concrete compressive strength exceeds 5000 psi; refer to 19.3.3.3] 37 Assign limits on cementitious materials /LPLWVLQDFFRUGDQFHZLWK7DEOH 26.4.2.2(b) Cement combinations (for Class S3 in Table 19.3.2.1) must be tested in accordance with ASTM C1012/C1012M and meet the maximum expansion UHTXLUHPHOWRI&ODVV S3); check Table 26.4.2.2(c) Table 19.3.2.1 Table 26.4.2.2(c) Assign limits on calcium chloride admixture Not permitted (based on S2 and C2) Table 19.3.2.1 Assign maximum water-soluble chloride-ion (Cl-) content in concrete, percent by weight of cement 0.15 (based on C2, watersoluble chloride ion content from all concrete Institute ACI 201.2R-08—Guide to Durable Concrete ACI 212.3R-10—Report on Chemical Admixtures for Concrete ACI 222R-01—Protection 222R-0 of Metals in Concrete Against osion Corrosion 22.3R-11—Guide Corrosion of Reinforcement in Concrete tices to Mit Mitigatee Corrosion Str res Structures \$& HFL¿FDWLR \$&,^26SHFL¿FDWLRQIRU6WUXFWXUDO&RQFUHWH AC uide to Cold C ACI 306R-10—Guide Weather Concreting AC 334. —Concret Shell Structures-Practice and ACI 334.1R-92—Concrete Com ntar Commentary ease cor ACI 350 350—Please correct document number and add title rrect reference in body of text (Section 4.3.3) CI 372R-13—ACI 372R-13—Guide to Design and Construction of Circular WireW and Strand-Wrapped Prestressed Concrete Structures ASTM International \$670 & 026WDQGDUG 6SHFL¿FDWLRQ IRU Ready-Mixed Concrete ASTM C1012/C1012M-13—Standard Test Method for /HOJWK&KDOJHRI+\GUDXOLF&HPHOW0RUWDUV([SRVHGWRD Sulfate Solution ASTM C1218/C1218M-99—Standard Test Method for Water-Soluble Chloride in Mortar and Concrete \$670 & 026WDOGDUG 6SHFL¿FDWLRO IRU Portland Cement ASTM D516-11—Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL¿FDWLRO IRU Portland Cement ASTM D516-11—Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL¿FDWLRO IRU Portland Cement ASTM D516-11—Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL¿FDWLRO IRU Portland Cement ASTM D516-11—Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL¿FDWLRO IRU Portland Cement ASTM D516-11—Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL¿FDWLRO IRU Portland Cement ASTM D516-11—Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL (Standard Test Method for Sulfate Ion in Water \$670 & 026WDOGDUG 6SHFL 6SHFL¿FDWLRQ IRU &RQFUHWH 0DGH E\ 9ROXPHWULF %DWFKLQJ DQG &RQWLQXRXV Mixing ASTM C1580-09-Standard Test Method for Sulfate in Soil ASTM D4130-15—Standard Test Method for Sulfate Ion in Brackish Water, seawater, and Brines American Concrete Institute – Copyrighted © Material – www.concrete.org Durability CHAPTER 4—DURABILITY 38 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Authored documents Hewlett, P. C., ed., 1998, Lea's Chemistry of Cement and Control of Concrete Mixtures (EB001), 14th edition, fourth printing (rev.), Portland Cement Association, 6NRNLH,/)HESS .RVPDWND 6 + DQG :LOVRQ 0 / Design and Control of Concrete Practice, end of American Concrete Institute, publisher, Farmington Hills, MI, 510 pp. Powers, T. C., 1958, "Structure and Physical Properties of Hardened Portland Cement Paste," Journal of the American Ceramic Society91RSS Stark, D., 1989, "Durability of Concrete in Sulfate-Rich 6RLOV 5' 3RUWODQG & HPHQW\$VVRFLDWLRQ 6NRNLH // 14 pp. 6WDUN <sup>3</sup>/RQJ7HUP 3HUIRUPDQFH RI 3ODLQ DQG Reinforced Concrete in Seawater Environments (RD119)," 3RUWODQG&HPHQW\$VVRFLDWLRQ6NRNLH,/SS Taylor, P. C.; Nagi, M. A.; and Whiting, D. A., 1999, "Threshold Chloride Content for Corrosion of Steel in & RQFUHWH\$/LWHUDWXUH5HYLHZ5' '3RUWODQG&HPHQW \$VVRFLDWLRQ6NRNLH,/SS Whiting, D. A.; Taylor, P. C.; and Nagi, M. A., 2002, 3&KORULGH/LPLWVLQ5HLQIRUFHG&RQFUHWH5' 3RUWODQG&HPHQW\$VVRFLDWLRQ6NRNLH,/SS American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5—ONE-WAY SLABS 5.2—Analysis ACI 318 allows for the designer to use any analysis proce\ GXUHWKDWVDWLV¿HVHTXLOLEULXPDQGJHRPHWULFFRPSDWLELOLW\ as long as design strength and serviceability requirements DUH PHW 7KH & RGH LQFOXGHV D VLPSOL¿HG PHWKRG RI DQDO\VLVIRURQHZD\VODEVWKDWUHOLHVRQFRHI¿FLHQWVWRFDOFXODWH moments and shears. 5.3—Service limits 5.3.1 Minimum thickness—For nonprestressed slabs, the Code allows the designer for slabs not supporting or attached to partitions or other construction likely to be GDPDJHG E\ ODUJH GHÀHFWLRQ WR HLWKHU FDOFXODWH GHÀHFWLRQV or simply satisfy a minimum slab thickness (Section 7.3.1, ACI 318-14). In the case where loads are heavy, nonuniIRUPRUGHÀHFWLRQLVDFRQFHUQFDOFXODWLRQVVKRXOGYHULI\ WKDW VKRUW DQG ORQJWHUP GHÀHFWLRQV DUH ZLWKLQ WKH & RGH limits (Section 24.2.2, ACI 318-14). The Code does not provide a minimum thickness-tospan ratio for PT two-way slabs, but Table 9.3 of The PostTensioning Manual (Post-Tensioning Manual).

Institute (PTI) 2006), lists span-to-depth ratios for different members that have been found from experience to provide satisfactory structural performance. 5.3.2 'HAHFWLRQV—For nonprestressed slabs that are thinner than the ACI 318 minimum, or if the slab resists a Fig. 5.3.3—Load balancing concept. very heavy live load, superimposed dead load or both, and IRU37VODEVDVZHOOWKHGHVLJQHUFDOFXODWHG GHÀHFWLRQV7KH FDOFXODWHG GHÀHFWLRQV7KH FDOFXODWHG (hÀHFWLRQV7KH FDOFXODWHG) any appropriate method, such as classical equations or software results. Note that the spacing of slab reinforcing bar to limit crack width, timing of form removal, concrete quality, timing of construction loads, and other construction variables all FRXOG DIIHFW WKH DFWXDO GHAHFWLRQ 7KHVH YDULDEOHV VKRXOG EH FRQVLGHUHG ZKHQ DVVHVVLQ] WKH DFFXUDF\ RI GHAHFWLRQ calculations. In addition, creep over time will increase the LPPHGLDWHGHAH LPPHGLDWHGHAHFWLRQV LFDOO\ ZLWK D 3 7\SLFDOO\ 37 VODE VODE GHAHFWLRQV DUH XVXDOO\ im m the maximum net concrete small. If th thee designer limits crac tensile stress to below cracking stress under service loads, GHAH RQF LRQVFDQFR GHÀHFWLRQFDOFXODWLRQVFDQFRQVLGHUWKHJURVVVODESURSHUWLHV 5.3. 5.3.3 Co Concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete SVHXG VHUY XUDOVWUHV SVHXGRVHUYLFHAH[XUDOVWUHVVODESURSHUWLHV 5.3. 5.3.3 Co Concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete SVHXG VHUY XUDOVWUHV SVHXGRVHUYLFHAH[XUDOVWUHVVODESURSHUWLHV 5.3. 5.3.3 Co Concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete SVHXG VHUY XUDOVWUHV SVHXGRVHUYLFHAH[XUDOVWUHVVODESURSHUWLHV 5.3. 5.3.3 Co Concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete SVHXG VHUY XUDOVWUHV SVHXGRVHUYLFHAH[XUDOVWUHVVODESURSHUWLHV 5.3. 5.3.3 Co Concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete SVHXG VHUY XUDOVWUHV SVHXGRVHUYLFHAH[XUDOVWUHVVODESURSHUWLHV 5.3. 5.3.3 Co Concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference to a concrete service stress—Nonprestressed slabs igne for strength rength w are designed without reference service stress—Nonprestressed slabs igne for strength rength w are designed stressed slabs igne for strength rength w are designed stressed slabs igne for strength rength w are designed stressed slabs igne for strength rength w are designed stress FRQFUHWH AH[XUDO WHQVLRQ itical part of the design. In Section 8.3.4.1 of stresses is a critical WKHFRQF \$&,WKHFRQFUHWHAH[XUDOWHQVLOHVWUHVVLQQH]DWLYH ent areas aat columns in PT slab is limited to 6 f c'. moment At positive moment sections, Section 8.6.2.3 of ACI 318-14 posi requires slab reinforcing bar if the concrete tensile stress exceeds 2 f c' 7KHVH VHUYLFH WHQVLOH AH[XUDO VWUHVV OLPLWV are below the concrete cracking stress of 7.5 f c', thus KDYLQJWKHHIIHFWRIOLPLWLQJGHAHFWLRQV, QDGGLWLRQ6HFWLRQ 8.6.2.1 of ACI 318-14 requires a PT slab's axial compressive stress in both directions, due to PT, to be at least 125 psi. %HIRUHWKHVODEAH[XUDOVWUHVVHVLQDGHVLJQVWULSFDQEH FDOFXODWHGWKHWHQGRQSUR¿OHVKRXOGEHGH¿QHG%RWKSUR¿OH and tendon force are directly related to slab forces and moments created by PT. A common approach to calculate PT slab moments is the use of the "load balancing" concept 7HQGRQVDUHW\SLFDOO\SODFHGLQDSDUDEROLFSUR¿OHVXFKWKDW the tendon is at the minimum cover requirements at mid-depth at the slab edge (Fig. 5.3.3). The tendon exerts a uniform upward force along its length that counteracts a portion of the gravity loads, usually 60 to SHUFHQWRIWKHVODEVHOIZHLJKWDFFRUGLQJWR/LEE\ hence, the term "load balancing." The load effect from the prestressing force in the tendon is then combined with the load effect of the gravity loads to determine net concrete Institute – Copyrighted © Materialwww.concrete.org One-Way Slabss 5.1—Introduction A one-way slab is generally used in buildings with vertical supports (columns or walls) that are unevenly spaced, creating a long span in one direction and a short span in the SHUSHQGLFXODUGLUHFWLRQ2QHZD/VODEVW/SLFDOO/VSDQLQWKH short direction and are supported by beams in the long direction. During preliminary design, the designer determines the loads and spans, reinforcement type (post-tensioned [PT] or nonprestressed), and slab thickness. The designer determines the concrete strength based on experience and the Code's exposure and durability provisions. This chapter discusses cast-in-place, nonprestressed, and PT slabs. The Code allows for either bonded tendons in a PT slab. Because bonded tendons are not usually used in slabs in the United States, this chapter will address PT slabs with unbonded tendons. At times, the design of a one-way slab will require point rages. Index of a one-way slab will require point rages. local shear forces on the slab,, requiring veriU VWUHQJWK ¿FDWLRQ RI WKH VODE IV SXQFKLQJ VKHDU VWUHQJWK 3XQFKLQJ or two shear is addressed in Chapter 6 for two-way slabs in this Handbook. ning trim m bars can be used d For relatively small slab openings, y geometric g tric stress concentraconce trato limit crack widths caused by cal increase ase in slab thick ess, tions. For larger openings, a local thickness, m may be necessary nd strength. ngth. provide adequate serviceability and 40 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) To achieve Code stress limits, the designer can use an iterative or direct approach. In the iterative approach, the WHQGRQ SUR¿OH LV GH¿QHG DQG WKH WHQGRQ IRUFH LV DVVXPHG 7KH DQDO\VLV LV H[HFXWHG AH[XUDO VWUHVVHV DUH FDOFXODWHG DQG WKH GHVL]QHU WKHQ DGMXVWV WKH SUR¿OH RU IRUFH RU ERWK depending on results and design constraints. In the direct approach, the designer determines the highest tensile stress permitted, then rearranges equations so that the analysis calculates the tendon force required to achieve the stress limit. The Code does not impose a minimum for cast-in-place slabs. For slabs exposed to aggressive environments, engineers usually design of one-way slabs are typically controlled by moment strength. Assuming a uniform load, the designer calculates the unit (usually a 1 ft width) factored slab moments. The required DUHDRIAH[XUDOUHLQIRUFHPHQWRYHUDXQLWVODEZLGWKLVFDOFXlated with the same assumptions as a beam. ment strength—For nonprestressed reinforced slabs, a quick way to calculate a slab's gravity design moments (iff the slab meets the speciRQV WKHPRPHQW iHGJHRPHWULFDQGORDGFRQGLWLRQV LVE\WKHPRPHQWFRHI &KDSWHU RI RI \$&, iFLHQWV LQ 6HFWLRQ RI \$&, ct analysis ysis methods. 318-14 permits other, more exact ion induced duced by prestressing prestre ing For PT slabs, effects of reactions o the factored gr ity (secondary moments) should be add added to gravity l l t Mu moments per Section 5.3.11 of ACII 318-14 tto calculate The slab's secondary moments are a result of the beam's vertical restraint of the slab against the PT "load" at each ed support. Because the PT force and drape are determined during the service stress checks, secondary moments can be quickly calculated by the "load-balancing" analysis concept. A simple method for calculating the secondary moment is to subtract the tendon force multiplied by the tendon eccentricity (distance from the neutral axis) from the total balance moment, expressed mathematically as M2 = Mbal – Pe. The critical locations to calculate Mu along the span are usually at the support and midspan. Section 7.4.2.1 of ACI 318-14 allows Mu to be calculated at the face of support centerline. 5.4.2 Calculation of required shear strength—Assuming a uniform load, the designer calculates the unit (usually a 1 IWZLGWK IDFWRUHGVODEVKHDUIRUFHE\HLWKHUWKHFRHI¿FLHQW method or more exact calculations. 5.5—Design strength 2QHZD/VODEVPXVWKDYHDGHTXDWHRQHZD/VKHDUVWUHQJWK and moment strength in all design strips. 5.5.1 Calculation of design moment strength—The UHTXLUHGDUHDRIAH[XUDOUHLQIRUFHPHQWIRUDQRQSUHVWUHVVHG and PT unit slab width are calculated with the same assumptions as for a beam, Chapter 7, of this Handbook. Table 5.6.1a—As,min for nonprestressed one-way slabs (Table 7.6.1.1, ACI 318-14) Reinforcement type fy, psi As,min, in.2 Deformed bars or welded wire reinforcement • Greater of: 0.0018 × 60, 000 Ag fy 0.0014Ag Table 5.6.1b—Maximum spacing of bonded reinforcement in nonprestressed and Class C prestressed (Table 24.3.2 one-way slabs and beams, ACI 318-14, partial) /HVVHURI: (40,000) 15 | - 2.5cc (fs / 12(40,000/fs) 5.5.2 Calculation of design shear strength is the same as for a beam, Chapter 7, of this Handbook. rei 5.6—Flexure reinforcement detailing RGHUHTXLUHVD PLQL LQ 7KH&RGHUHTXLUHVDPLQLPXPDUHDRIAH[XUDOUHLQIRUFHd iin ttension regions to ensure that the slab ment bee pla placed ation and crack wid deformation widths are limited when the slab's crack g str cracking strength iss exceede exceeded. If more than the minimum area by analysis, that reinforcement area must rea iis required requy be provided. vided Reinforcement forcemen in one way slabs is usually XQLIRUPO\VSDFHGXQOHVVWKHUHLVDVLJQL¿FDQWSRLQWORDGRU XQLIRU O\V XQOHVVWK ope opening.. prestressed reinforced slab - Flexural rein5.6.1 Nonprestressed nt area and placing—For nonprestressed slabs, the forcement PLQLPXPÀH[ PLQLPXPAH[XUDOEDUDUHDAs,min, is given in Table 7.6.1.1 of ACI 318-14 (Table 5.6.1a of this Handbook). 7KHPD[LPXPVSDFLQ]RIAH[XUDOEDUVLV]LYHQLQ7DEOH 24.3.2 of ACI 318-14 (Table 5.6.1b of this Handbook). Because fs is
usually taken as 40,000 psi, the maximum spacing will not exceed 12 in. The bar termination rules in Section 7.7.3 of ACI 318-14 cover general conditions that apply to beams, but because one-way slab bars are usually spaced close to the maximum spacing. This usually results in all bottom bars extending full length into the beams. 5.6.2 Post-tensioned slab – Flexural tendon area and placing. -For PT one-way slabs, the Code does not have a limit for minimum tendon area or a minimum compressive VWUHVVGXHWR377KLVLVFRQVLVWHQWZLWKWKHÀH[LEOHDSSURDFK on service stresses. The Code limits the maximum tendon area and placing—The Code requires the reinforcing bar area, As,min, to be placed close to the slab face at the bottom at midspan and the top at the support. For one-way slab strip is rectangular, Act = 0.5Ag. This minimum is independent of service stress level. American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 5—ONE-WAY SLABS If the designer uses tendons, the minimum slab effective compression force due to temperature and shrinkage tendons is 100 psi. The purpose of this reinforcement is to restrain the size and spacing of slab cracks, which can occur due to volume variations caused by temperature changes and slab shrinkage over time. In addition, if the slab is restrained against movement, the Code requires the designer to provide reinforcement that accounts for the resulting tension stress in the slab. Authored references /LEE\ - 0RGHUQ 3UHVWUHVVHG & RQFUHWH 'HVLJQ Principles and Construction Methods, fourth edition, Springer, 871 pp. Post-Tensioning Institute (PTI), 2006, Post-Tensioning Manual, sixth edition, PTI TAB.1-06, 354 pp. One-Way Slabss The maximum spacing of reinforcing bar in a PT one-way slab is the lesser of 3h and 18 in., I WKH VODE GHVLJQ PRPHQW VWUHQJWK LV IXOO\ VDWLV¿HG E\ the tendons alone, the termination length of As,min bars for bottom bars is (a) and for top bars is (b): (a) At least En/3 in positive moment areas and be centered in those areas (b) At least En/6 on each side of the face of support The termination length for bars that are required for strength are the same as for nonprestressed slabs. 5.6.4 Temperature and shrinkage reinforcement and placing—Shrinkage and temperature (S&T) slab reinforcement is required and could be either reinforcing bar or WHQGRQVSODFHGSHUSHQGLFXODUWRAH[XUDOUHLQIRUFHPHQW If the designer uses reinforcing bar, the minimum area of temperature and shrinkage Grade 60 bar is 0.0018Ag. 41 American Concrete Institute – Copyrighted © Material – www.concrete.org 42 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 5.7—Examples One-way Slab Example 11RQSUHVWUHVVHGRQHZD\VODE<sup>2</sup> 'HVLJQDQGGHWDLOWKHVHFRQGVWRU\EXLOGLQJ7KHRQHZD\VODE<sup>2</sup> 'HVLJQDQGHWDLOWKHVHFRQGVWRU\EXLOGLQJ7KHRQHZD\VODE<sup>2</sup> 'HVLJQDQGHWDLOWKHVHFRQGVWRU\EXLOGLQ</sup> of the slab (Fig. E1.1). Given: Load— Service live load L = 100 psf Concrete— fc' = 5000 psi (normalweight concrete) fy = 60,000 psi Geometry— Span length: 14 ft Beam width: 18 in. Column dimensions: 24 in. x 24 in. )LJ(<sup>2</sup>3ODQRI¿YHVSDQRQHZD\VODE American Concrete Institute – Copyrighted © Material – www.concrete.org ACI 318-14 Discussion Step 1: Geometry Calculation 7.3.1.1 Determine slab thickness using ratios from Table 7.3.1.1.  $h \ge A$  (6 ft)(12 in./ft) = = 7 in. 24 24 Determine cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness:  $h \ge A$  (6 ft)(12 in./ft) = = 7.2 in., say, 7 in. 10 10 43 Because the slab and cantilever thickness the slab and cantilever thickn QRWQHHGWRFKHFNGHAHFWLRQVXQOHVVWKHVODELVVXSporting breakable partitions. Note: Architectural requirements specify a 3/4 in. step at the cantilever. Detail to maintain 7 in. slab thickness. Self-weight Slab: ws = (7 in./12 in./ft)(150 lb/ft3) = 87.5 psf 6WHS/RDGVDQGORDGSDWWHUQV 5.3.1 The service live load is 100 psf in assembly areas and corridors per Table 4-1 in ASCE 7-10. For cantilever use 100 psf. To account for weights from FHLOLQJVSDUWLWLRQV+9\$&VVWHPVHWFDGGSVI ad. as miscellaneous dead load. quired streng h eq The required streng psf 1 (102.55 psf) + 1.6 1. (100 psf) U = 1.2D + 1.6L (5.3.1b) U = 1.2 = 123 psf + 160 psf = 283 psf Controls art oof a The slab resists ggravity only and is not part is ystem, diaphragm. A provide id guidance for add Both ASCE 7 and ACI dressing live load patterns. Either approach is acceptable. wopatACI 318 allows the use of the following two terns, Fig. E1.2: Factored dead load is applied on all spans and fac6.4.2 tored live load on the span and on alternate spans. (b) Maximum negative Mu at a support occurs with IDFWRUHGOLYHORDGRQDGMDFHQWVSDQVRQO\ Fig. E1.2—Live load loading pattern. American Concrete Institute - Copyrighted © Material - www.concrete.org One-Way SLABS 44 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Concrete and steel material requirements 7.2.2.1 The mixture proportion must satisfy the durability requirements of ACI 318-14. The designer determines the durability classes. Please refer to Chapter 19 and structural strength requirements of ACI 318-14. The designer determines the durability classes. ACI 318. ACI encourages referencing \$&,LQWRMREVSHFL¿FDWLRQV 7.2.2.2 By specifying that the concrete mixture must be in accordance with ACI 301-10 and providing the exposure classes, Chapter 19 (ACI 318-14) requirements are VDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW psi. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. The reinforcement must satisfy Chapter 20 of ACI 318-14. The designer determines the grade of bar and if the reinforcing bar should be coated by epoxy or galvanized, or both. Step 4: Slab analysis ldi relies ies on the building's LVW DOORDGVWKHVODE TXDOL¿HV mptions, as discu ed in i the for braced framee assumptions, discussed commentary. 6.6 The analysis should be consistent with the overall assumptions about the role of the slab within the building system. Because the lateral forceresisting-system only relies on the slab's negative design moments are taken at the face of support as is permitted by the Code (Fig. E1.3). By specifying the reinforcing bar grade and any coatings, and that the reinforcing bar must be in accordance with ACI 301-10, Chapter 20 requirements DUHVDWLV¿HG,Q DUHVDWLV¿HG,QWKLVFDVHDVVXPH\*UDGHEDUDQGQR ggs coatings. delin assumptions: Modeling A ume an effective ctive mom Assume moment of inertia for the entire gth oof the slab. ab. length Ig ore ttorsional al stiffness stiffn ZKHQWKHVODELVIXOO\ORDGHGAH[XUDOFUDFNLQ]ZLOO VRIWHQWKHMRLQW Fig. E1.3—Moment envelope. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5—ONE-WAY SLABS 45 The negative moment at the centerline of the end right support is 0.0 ft-kip 7KHPD[LPXPSRVLWLYHPRPHQWLQWKHHQGVSDQ()LVIWNLS7KHLQÀHFWLRQSRLQWVIRUSRVLWLYHPRPHQWV DUHIWIURPWKHH[WHULRUVXSSRUWFHQWHUOLQHHROXPQ OLQH&/ ( 7KHPD[LPXPQHJDWLYHPRPHQWDWWKHIDFHRIWKH¿UVWLQWHULRUVXSSRUWIURPWKHULJKWHQG&/(LVIWNLS 7KHQHJDWLYHPRPHQW¶VULJKWLQÀHFWLRQSRLQWLVIWIURPWKHULJKWHQG&/(LVIWNLS 7KHQHJDWLYHPRPHQW¶VULJKWLQÀHFWLRQSRLQWLVIWIURPWKHULJKWHQG&/(LVIWNLS 7KHQHJDWLYHPRPHQW¶VULJKWLQÀHFWLRQSRLQWLVIWIURPWKHULJKWHQG&/(LVIWNLS 7KHQHJDWLYHPRPHQW¶VULJKWLQÀHFWLRQSRLQWLVIWIURPWKHULJKWHQG&/(LVIWNLS 7KHQHJDWLYHPRPHQW¶VULJKWLQÀHFWLRQSRLQWLVIWIURPWKHULJKWHQG&/(LVIWNLS 7KHQHJDWLYHPRPHQW¶VULJKWLQÀHFWLRQSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQH2QWKHOHIWVLGHDQGIRU WKHIXOOOHQJWKRIWKHVODEWKHUHLVQRLQÀHFWLRQSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQH2QWKHOHIWVLGHDQGIRU WKHIXOOOHQJWKRIWKHVODEWKHULJKWHQAHFWLRQSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQH2QWKHOHIWVLGHDQGIRU WKHIXOOOHQJWKRIWKHVODEWKHUHLVQRLQÀHFWLRQSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQH2QWKHOHIWVLGHDQGIRU WKHIXOOOHQJWKRIWKHVODEWKHUHLVQRLQÀHFWLRQSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQH2QWKHOHIWVLGHDQGIRU WKHIXOOOHQJWKRIWKHVODEWKHVANGAA 7KHPD[LPXPSRVLWLYHPRPHQWLQWKHLQWHULRUVSDQ&/%&LVIWNLS7KHLQÀHFWLRQSRLQWVIRUSRVLWLYH PRPHQWVDUHIWIURPWKHVHFRQGLQWHULRUVXSSRUW centerline. Because of pattern loading, a small negative moment can exist across all spans with the exception of the last span.
7KHPD[LPXPQH]DWLYHPRPHQWDWWKHH[WHULRUOHIWVXSSRUW&/\$LVIWNLSEHFDXVHRIWKHFDQWLOHYHUHG slab. Table 1.1—Maximum moments at supports and midspans Location from left to right along the span Required strength Exterior support First midspan Second support Second midspans Third support Third midspan Mu, ft-kip -6.0 +2.7 -4.7 +3.8 -5.8 +3.6 Required strength Fourth midspan Fifth support Fou Fourth midspan E End support Mu, ft-kip -5.3 +4.4 0 Continue: Step 6: Required strength h 7.4.3.1 The slab's maximum shear support m hear is taken at the support Fou Fourth midspan Fifth support Fou Fourth midspan Fifth midspan E End support Mu, ft-kip -6.3 +4.4 0 Continue: Step 6: Required shear strength h 7.4.3.1 The slab's maximum shear support m hear is taken at the support Fou Fourth midspan Fifth midspan E End support Fou Fourth midspan Fifth mid shear hear under all conditions is 2.4 4 kip (Fig ig E1.4). E1 4) Fig. E1.4—Shear envelope. American Concrete Institute - Copyrighted © Material - www.concrete.org One-Way Slabss Location from left to right along the span 46 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Design moment strength 7.5.1 The two common strength inequalities for oneway slabs, moment and shear, are noted in Section 7.5.1.1. 7.5.2 The one-way slab chapter refers to Section 22.3 for FDOFXODWLRQRIAH[XUDOVWUHQ]WK 7.3.3.1 To ensure a ductile failure usually the Code requires slabs to be designed such that the steel strength exceeds 0.004 in./in. In most reinforced slabs, such as this example, reinforcing bar strain is not a controlling issue. 21.2.1(a) 22.2.2.1 The design assumption is that slabs will be WHQVLRQHGFRQWUROOHG7 [LVDVVXPSWLRQZLOO be checked later. Determine the effective depth assuming No.5 bars and 0.75 in. cover (Fig. E1.5): Fig. E1 F E1.5—Effective ective dep depth American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 5—ONE-WAY SLABS 7.7.1.1 20.6.1.3.1 2QHURZRIUHLQIRUFHPHQW d = t – cover – db/2 22.2.2.1 The concrete compressive strain at nominal moment VWUHQJWKLVFDOFXODWHGDWIcu = 0.003 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and is approximately 10 to 15 percent of the concrete compressive strength. ACI 318 neglects the concrete compressive strength to calculate nominal strength. 22.2.2.3 Determine the equivalent concrete tensile strength to calculate nominal strength. ACI 318 neglects the concrete tensile strength to calculate nominal strength. ACI 318 neglects the concrete compressive strength to calculate nominal strength. 6.18 in., say, 6.0 in. The concrete compressive stress distribution is inelastic at high stress. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the nt rectangular Code allows the use of an equivalent ution of 0.85f 5 cg with a compressive stress distribution depth of: function of concrete compressi Find the equivalent compressive depth, a, om sion force to th ension by equating the compression the tension the tension of concrete compressive stress distribution depth of: function of concrete compressive stress distribution depth of: function of concrete compressive depth, a, om sion force to the equivalent compressive stress distribution depth of: function of concrete compressive stress distribution depth of: function of conc force within the beam cross section:  $\beta 1 = 0.8 \ 0.85 - 0.05(5000 \ \text{psi}) = 0.8100 \ \text{psi} = 0.81000 \ \text{psi} = 0.81000 \ \text{psi} = 1.176 \ \text{As} \ 0.85(5000 \ \text{psi}) = 1.176 \ \text{$ THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 7KHVODELVGHVLJQHGIRUWKHPD[LPXPÀH[XUDO moments obtained from the approximate method above. 7.5.1.1 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two moments. The beam's design strength must be at least the required strength at each section along its length:  $\Box$ Mn • 0u  $\Box$ Vn • 9u Calculate the required reinforcement area: a) ( $\phi$ M n ≥ M u =  $\phi$ As f y | d - | (2) A No. 5 bar has a db = 0.625 in. and an As = 0.31 in.2 Maximum positive moment: 1.176 AS ) (4.9 ft-kip ≤ (0.9)(60 ksi) As | 6.0 in. - (2) A + s'req'd = 0.18 in.2/ft Use No. 4 at 12 in. on center or No. 5 at 18 in. center botto bottom. Try No.5 at 18 in. on center  $(0.31 \text{ in.}2/\text{ft})(12 \text{ in.}/18 \text{ in.}) = 0.21 \text{ in.}2/\text{ft} \text{ As,provd.} = (0.3 \text{ in } 2/\text{ft} \text{ As,provd.} = (0.3 \text{ in } 2/\text{ft} \text{ As,provd.} = 21 \text{ in.} \text{ OK ximu negative gative mom Maximum moment: } 1.176 \text{ As } (9)(60 \text{ ksi}) \text{ s }) \text{ As } = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 31 \text{ in.} \text{ OK ximu negative gative mom Maximum moment: } 1.176 \text{ As } (9)(60 \text{ ksi}) \text{ s }) \text{ As } = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 31 \text{ in.} \text{ OK ximu negative gative mom Maximum moment: } 1.176 \text{ As } (9)(60 \text{ ksi}) \text{ s }) \text{ As } = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 31 \text{ as,provd.} = 31 \text{ as,provd.} = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 31 \text{ as,provd.} = 31 \text{ as,provd.} = 0.24 \text{ 24 iin.}2/\text{ft} \text{ As,provd.} = 0.24 \text{ 24$ in.2/ft > A(s,req'd = 0.24 in.2/ft OK Check if the calculated strain exceeds 0.005 in./in. (tension controlled). Form similar triangles (Fig. E1.6). a = As f y 0.85 f c'b and c = 0.36/0.8 = 0.46 in. c = 0.36/0.8 = 0.46 0.46 in. Fig. E1.6—Strain distribution American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 5—ONE-WAY SLABS 49 Step 8: Design shear strength 7.5.3 Assuming a one-way slab won't contain shear reinforcement, Vn is equal to Vc. Assuming negligible axial force, the Code provides the following expression, 22.5.5.1 21.2.1 Vc = 2 f c'bd Shear strength reduction factor: Vc = 2 5000 psi(12 in.)(6 in.) = 10,180 lb/ft  $\cong$  10 kip/ft [D.75 = Nc = (0.85)(10,000 lb) = 8500 lb > 2400 lb OK This exceeds the maximum Vu (2.4 kip/ft); therefore, no shear reinforcement is required. 6WHS0LQLPXPAH[XUDOUHLQIRUFHPHQW 7.6.1 Check if design reinforcement exceeds the minimum required by the Code. As, min = 0.0018 × 6 × 12 = 0.13 in.2/ft At all critical sections, the minimum 24.4.3.2 area of shrinkage and temperature (S+T) bars is 0.0018Ag. The maximum spacing generative reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature (S+T) bars is 0.0018Ag. The maximum spacing generative reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the minimum 24.4.3.2 area of shrinkage and temperature reinforcement 7.6.4 For one-way with Grade 60 bars, the mini of S+T reinforcing bar is the lesser of 3h and 18 in. S+T steel area = 0.0018 × 12 × 7 = 0.15 in. (a) 1 in. (b) db (b) 0.625 25 in. 3h and 18 in. Controls (a) 12(40,000/40,000) = 12 in. Soft 18 in. Therefore, Section 24.3.2 controls; 12 in. > 18 in. Therefore, Section 24.3.2 controls; 14 in. > 18 in. Therefore,
Section 24.3.2 controls; 14 in. > 18 in. Therefore, Section 24.3.2 controls; 14 in. > 18 in. = 118 in. > 18 in. = 118 in. = No. N 4 at 16 in. placed atop and or No. S SHQG WRWKHERWW SHUSHQGLFXODUWRWKHERWWRPAH[XUHUHLQIRUFHPHQW 50 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 12: Select reinforcing bar size and spacing Based on the above requirement, use No. 5 bars. Spacing on top and bottom bars is 12 in. Note that there is no point of zero negative moment along all spans except the last bay, so continue the top bars across all spans. Also, No. 4 bars), the engineer may desire consistent spacing and reinforcing bar use for easier installation and inspection. Step 13: Top reinforcing bar length at the exterior support, QAHFWLRQSRLQWV The top bars have to satisfy the following provisions: 7KHLQAHFWLRQSRLQWIRUQHJDWLYHPRPHQWDWHQGVSDQ is 5.0 ft from support centerline. 7.7.3.3 Reinforcement shall extend beyond the point at ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWAH[XUHIRUD distance equal to the greater of d and 12db, except at d spans and at free ends supports of simply-supported of cantilevers. Bar cutoffs ([WHQGEDUVEH\RQGWKHLQÀHFWLRQSRLQWDWOHDVW d = 6 in. or (12)(0.625 in.) = 7.5 in. Therefore, use 7.5 in. 7.7.3.8.4 d the negative moment reinforceAt least one-third rt sshall have ave an embedm ment at a support embedmentt length QW HFWLRQDWOHDVWWK JUHDWHVWRI EH\RQGWKHSRLQWRILQÀHFWLRQDWOHDVWWKHJUHDWHVWRI 16 SHUF HEDUVWR d, 12db, and En/16. SHUFHQWRIWKHEDUVWRH[WHQGEH\RQGWKHLQÀHFWLRQ po nt at least point ((14 ft - 1.5 ft)(12)/16 (12)/16 = 9.4 in. > 12db = 7.5 in. > d = 6 in. Because thee 14: Development and splice lengths 7.7.1.2 ACI provides two equations for calculating 25.4.2.3 GHYHORSPHQWOHQJWKVLPSOL¿HGDQGGHWDLOHG,QWKLV The development leng par is already at maximum g, no percentage perce spacing, of bars (as permitted by Section 7.7.3.3 ccheck) can be cut off in the tension zone of a No. 5 black bar in a 7 in. example, the detailed equation is used: slab with 0.75 in. cover is: () | [3 f y  $\psi t \psi e \psi s$  | d Ad = | 40  $\lambda$  f c' (cb + K tr) | b | | (d |) | / (b ( 3 60, 000 psi (1.0)(1.0)(0.8)) Ad = | (0.625 in.) 1.7 in. (40 (1.0) 5000 psi ) where zt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ze = coating factor; uncoated  $z_s = bar$  size factor; No. 7 and larger  $z_t = 1.0$ , because not more than 12 in. of concrete is placed below bars.  $z_e = 1.0$ , because bars are smaller than No. 7 However, the expression: taken greater than 2.5. cb + K tr must not be db = 19 in. 1.06 in. + 0 = 1.7 in. 0.625 in. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5-ONE-WAY SLABS 7.7.1.3 25.5 25.5.1.1 Splice The maximum bar size is No. 5, therefore, splicing is permitted. 25.5.2.1 Tension lap splice length, Est, for deformed bars in tension must be the greater of: 1.3Ed and 12 in. 6WHS%RWWRPUHLQIRUFLQJEDUOHQJWKDORQJ¿UVWVSDQ The bottom bars have to satisfy the following provisions: 7.7.3.3 Reinforcement must extend beyond the point at ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWÅH[XUHIRUD distance equal to the greater of d and 12db, except at supports of simply-supported spans and at free ends o cantilevers. 51 Est = (1.3)(19 in.) = 24.7 in.; use 36 in., QAHFWLRQSRLQWV 7KHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUHIW IURPH[WHULRUVXSSRUWFHQWHUOLQHDW&/)DQGIW IURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQHA((7.7.3.4 & RQWLQXLQ]AH[XUDOWHQVLOHUHLQIRUFHPHQWPXVW RUF LVVDWLV¿HG (2/3)(10,800 llb) = 7200 lb Vu" [Vn at the cutoff point. 2400 lbb < (2/3)(10,800 OK 7.7.3.8.2 ur the maximum posit At least one-fourth positivee mo moment end along the sl b bott reinforcement mu must extend slab bottom ous support port a minimum off 6 in. into the continuous 7.7.3.8.3 \$WSRLQWVRILQÀHFWLRQdb for positive moment tensile reinforcement shall be limited such that Q E EdIRUWKDWUHLQIRUFHPHQWVDWLV¿HVFRQGLWLRQE EHFDXVHHQGUHLQIRUFHPHQWVDWLV¿HVFRQGLWLRQE EHFDXVHHQGUHLQIRUFHPHQWVDWLV¿HVFRQGLWLRQE EHFDXVHHQGUHLQIRUFHPHQWVDWLV and (c) do not Check if bar size is adequate Mn for an 7 in. slab with No. 5 at 12 in., 0.75 in cover is:  $Ed''Mn/Vu + Ea Ea is the greater of d and 12db = 7.5 in. a ( M n = As fy | d - | (2/Mn = (0.31 in.2/ft)(60,000 psi)(6 in. - 18 in.) = 108,252 in.-lb # 108,000 in.-lb + 7.5 in. = 67.5 in. 1800 lb Ad = 19 in. \le 108,000 in.-lb + 7.5 in. = 67.5 in. 1800 lb Ad = 19 in. \le 108,000 in.-lb + 7.5 in. = 67.5 in. = 67$ The bar cut offs that are implicitly permitted from the Code provisions because of reduced required strength along the span do not apply before the LQAHFWLRQSRLQWVIRUWKLVVODEEHFDXVHLIDQ\EDUV were cut off, the maximum reinforcing bar spacing would be violated. does not apply. All bottom bars need to extend at least 7 in. (refer to Section 7.7.3.3) beyond the positive moment LQAHFWLRQSRLQWV OK The Code requires that at least 25 percent of bottom bars be full length, extending 6 in. into the support. Because the cut off locations are close to the supports DQGIRU¿HOGSODFLQJVLPSOLFLW\H[WHQGDOOEDUVLQ into both supports. American Concrete Institute – Copyrighted © Material – www.concrete.org 52 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 17: Slab detailing Fig. E1.7—One-way slab reinforcement. American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 5—ONE-WAY SLABS 53 One-way Slab Example 2: Assembly loading— Design and detail a one-way nonprestressed reinforced concrete slab both for service conditions and factored loads. The one-way slab spans 20 ft-0 in. and is supported by 12 in. thick walls on the exterior, and 12 in. wide beams on the interior. Given Load— /LYHORDGL = 100 psf & RQFUHWHXQLWZHLJKWÛs = 150 lb/ft3 Geometry— Span = 20 ft Slab thickness t = 9 in. One-Way Slabss Material properties— fcq SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi Fig. E2.1—One-way slab framing plan. American Concrete Institute - Copyrighted © Material - www.concrete.org 54 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Calculation Step 1: Geometry 7.3.1.1 7KHVSHFL¿HGVODEWKLFNQHVVLVLQ6LQFHWKHVODE VDWLV¿HVWKH\$&,VSDQWRGHSWKUDWLRV7DEOH A (20 ft)(12 in./ft) = 8.89 in., say, 9 in. WKHGHVLJQHUGRHVQRWQHHGWRFKHFNGHÀHF- h ≥ 27 = 27 tions porting or attached to partitions or This ratio is less than the table value for "both ends other construction likely to be damaged by large FRQWLQXRXV VRGHAHFWLRQVDUHQRWUHTXLUHGWREH GHAHFWLRQVDUHQRWUHTXLUHGWREH GHAHFWLR load is 100 psf per Table 4-1 in ASCE 7-10. A 9 in. slab is a 112 psf dead load. To account for loads due to ceilings, partitions, +9 (5.3.1a) U = 1.4(122) = 171 psf U = 1.2D + 1.6L (5.3.1b) U = 1.2(122) + 1.6(100) = 146 + 160 = 1.40 + 1 306 psf The slab resists gravity only and is not part of a Controls lateral force-resisting system, except to act as a diaphragm. CI provide pro Both ASCE 7 and ACI guidance for addressing live load patt patterns. Either approach is acceptable. followin two patACI 318 allowss the usee of the following terns, Fig. E2.2:: 6.4.2 d iis applied ied on all spa nd Factored lead load spans and factored live load is applied as follows: (a) Maximum positive Mu near midspan occurs with factored live load on the span and on alternate spans. (b) Maximum negative Mu at a support occurs with IDFWRUHGOLYHORDGRQDGMDFHQWVSDQVRQO\ Fig. E2.2—Live load loading pattern. American Concrete Institute -Copyrighted © Material – www.concrete.org CHAPTER 5—ONE-WAY SLABS Step 3: Concrete and steel material requirements of ACI 318-14. The designer determines the durability requirements of Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. 55 By specifying that the concrete mixture must be in accordance with ACI 301-10 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVWSVL \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing \$&,LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if
suggested inforcement must satisfy Chapter 20 of ACI 318-14. The designer determines the grade of bar and if the reinforcing bar should be coated by epoxy or galvanized, or both. Step 4: Slab analysis ding relies on the building's other 6.3 Because the building LVW OORDGVWKHVODE XDOL¿HV PHPEHUVWRUHVLVWODWHUDOORDGVWKHVODETXDOL¿HV me assumptions, mptions, as discu ed in the reinforcing bar must be in accordance with ACI 301-10, Chapter 20 requirements DUHVDWLV¿H DUHVDWLV¿HG,QWKLVFDVHDVVXPH\*UDGHEDUDQGQR coatings. delin assumptions: sumptions: Modeling A ply tthe effective ctive mom Apply moment of inertia for the entire le gth oof the slab. b length Ig ore ttorsional al stiffness stiffness stiffness of beams. Ignore 6.6 VODEDWWKLV2QO\WKHVODEDWWKLVOHYHOLVFRQVLGHUHG The analysis should be consistent with the overall assumptions tions about the role of the slab within the building V/VWHPRQO/UHOLHVR V/VWHPRQO/UHOLHV 7KHFRQQHFWLRQWRWKHZDOOLVPRQROLWKLFKRZHYHUZKHQWKHVODELVIXOO\ORDGHGÅH[XUDOFUDFNLQJZLOO VRIWHQWKHMRLQW5DWKHUWKDQDWWHPSWLQJWRHVWLPDWHDQDSSURSULDWHOHYHORIVRIWHQLQJWKHVODELVVLPSO\PRGeled twice WHHIIHFWRIFUDFNLQJUHGXFHAH[XUDOVWLIIQHVVE\LQFUHDVLQJWKRIVXSSRUW,QWKLVH[DPSOH support lengths are increased to 100 ft long columns. Assume fully connected slab to exterior wall, with uncracked moment of inertia, modelled by a 10 ft, 12 in. x 12. in. columns, and the middle three supports are slender, 100 ft columns. Note: The moments resulting from analysis maximize the moments are taken at the face of support, as is permitted by the Code. American Concrete Institute – Copyrighted © Material – www.concrete.org One-Way Slabss 7.2.2.2 56 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Analysis (a): Note: The moment at the exterior support is near zero (refer to Fig. E2.3): Fig. E2.3—Moment envelope. The negative moment at the centerline of the exterior support is 0.0 ft-kip 7KHPD[LPXPSRVLWLYHPRPHQWLQWKHHQGVSDQLVIWNLS7KHLQÀHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUH IWIURPWKHH[WHULRUVXSSRUWFHQWHUOLQHDQGIWIURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQH WLYHPRPHQWDWWKHIDFHRIWKH¿UVWLQWHULRUVXSSRUWLVIWNLS7KHQHJDWLYHPRPHQW¶V WKHVXSSRUWFHQWHUOLQH2QWKHULJKWVLGHXQGHUWKHSLQQHGDWZDOO ÅHFWLRQSRLQW%HFDXVHRISDWWHUQORDGLQ DVVXPSWLRQWKHUHLVQRLQÅHFWLRQSRLQW%HFDXVHRISDWWHUQORDGLQJDVPDOOQHJDWLYHPRPHQWLQWKHLQWHULRUVSDQLVIWNLS7KHLQÅHFWLRQSRLQWVIRUSRVLWLYHPRPHQWV KH¿SS IW IU KHVHFRQGL DUHIWIURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQHDQGIWIURPWKHVHFRQGLQWHULRUVXSSRUWFHQWHUOLQHGGIWIURPWKHVHFRQGLQWHULRUVXSSRUWFHQWHUGQIWKH DOODVVXPSWLRQWKHUHLV QSR XQGHUWKHSLQQHGDWZDOODVVXPSWLRQWKHUHLVQRLQAHFWLRQSRLQW n lloading, ng, a small neg ss the spa Because of pattern negativee moment can exis exist across span. Table 2.1—maximum moment for hinged end condition (Approach oach (a)) n from left to right along a Location the span strength Exterior support First midspan Second support Second midspan Middle support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Second midspan Middle Support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (end support Mu, ft-kip 0.0 +10.9 -12.4 +7.3 -9.5 Analysis (b) (en PHQWDWWKHIDFHRIWKHH[WHULRUVXSSRUWLVIWNLS7KHQH]DWLYHPRPHQW¶VLQAHFWLRQSRLQW is 3.8 ft from the support centerline. 7KHPD[LPXPSRVLWLYHPRPHQWLQWKHHQGVSDQLVIWNLS7KHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUH IWIURPWKHH[WHULRUVXSSRUWFHQWHUOLQHDQGIWIURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQH 7KHPD[LPXPQH]DWLYHPRPHQWDWWKHIDFHRIWKH¿UVWLQWHULRUVXSSRUWLVIWNLS7KHQH]DWLYHPRPHQW¶V QSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQHDQGWKHULJKWLQAHFWLRQSRLQWLVIWIURPWKHVXSport centerline. 7KHPD[LPXPSRVLWLYHPRPHQWLQWKHLQWHULRUVSDQLVIWNLS7KHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQW DUHIWIURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQHDQGIWIURPWKHVHFRQGLQWHULRUVXSSRUWFHQWHUOLQH The maximum negative moment at the face of the second interior support is 9.9 ft-kip. The negative moPHQW¶VOHIWLQÀHFWLRQSRLQWLVIWIURPWKHVXSSRUWFHQWHUOLQHDQGWKHULJKWLQÀHFWLRQSRLQWLVIWIURPWKH support centerline. Following are the maximum moments from a combination of Analysis (a) and (b) (conservative approach). Table 2.2—Maximum moment for continuity condition between slab on wall (Approach). (b)) Exterior support First midspan Second support Second midspan Middle support Mu, ft-kip -8.7 +6.8 -9.9 +7.2 -9.9 Step 6: Required shear strength 7.4.3.1 The slab's maximum mum shear is taken at the support centerline for simplicity. mp The maximum shear is taken at the support Second midspan Middle support Second midspan Second support Second midspan Middle support strength gth inequalities forr one7.5.1 The two common nt aand shear, hear, are noted in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to be 0.9, which will be checked later. 7.5.2 The one-way slab chapter refers to Section 22.3 \$&, IRUFDOFXODWLRQRIAH[XUDOVWUHQ]WKUHGXF21.2.1 tion factor in Section 7.5.1.2 is assumed to Section 7.5.1 American Concrete Institute - Copyrighted © Material - www.concrete.org One-Way Slabss Location from left to right along the span Required strength 58 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 22.2.1 22.2.2.2 Chapter 22 (ACI 318-14) provides the design assumptions for reinforced concrete members. To calculate As in terms of the depth of the compression block, a, set the section's concrete compressive strength equal to steel tensile strength: Generate the minimum area of steel for the required moment. T=C As = Asfy = 0.856 (5000 psi)(12 in.)a = 0.85a (60,000 psi)(12 in.)a = 0.85a (60, half the bar diameter (for a single layer of reinforcing bar). Assuming a To calculate the minimum required As, set the design No. 5 bar, therefore, strength moment. d LQ[]LQ = 7.9 in. - = Mu ( 2(0.85) |) Table 2.3 following shows the required area of steel corresponding to the maximum moments from a alysis (a) and (b). conservative combination of Analysis m moment of Approaches (a) and (b) Table 2.3—Summary of maximum Location from left to righ right along the span Mu, ft-kip 2 Req'd As, in. per foot Exterior support uppo First midspan Second support suppo Second midspan Middle support -8.7 8.7 +10.9 -12.4 .4 +7 +7.3 -9.9 0.26 .26 0.32 00.37 7 00.22 2 0.28 To ensure a ductile steel strain at til failure ure mode, the st el stra hm n./in.
For ultimate strength must be at least 0.004 in./in. ab such h as this example, exa bar usual reinforced slabs, strain does not usually control the design. calculate reinforcing To calcu nforcing ba strain, begin with force within eequilibrium ilibr ithin the ssection: T=C Asfy = 0.85f 85f 5fcgba A strain diagram is drawn (Fig. E2.5). where b = 12 in./ft; fy = 60,000 psi; and the slab's maximum reinforcement is As = 0.37 in.2 From above calculations: As = 0.85a or a = As/0.85 22.2.2.1 Maximum strain at the extreme concrete FRPSUHVVLRQ¿EHULVDVVXPHGHTXDOWR Icu = 0.003 in./in. Therefore, a = 0.44 in. where a = u1c and u1 = 0.80 for fcgRISVL so c = 0.55 in. From similar triangles (Fig. E2.5): 0.003(7.9 in. - 0.55 in.)  $\epsilon t = 0.040 \ge 0.004$  (0.55 in.) Therefore, the assumption of 0.9 = []s correct. American Concrete Institute - Copyrighted © Material - Copyrighted © www.concrete.org CHAPTER 5—ONE-WAY SLABS 59 22.5.5.1 Vc = 2 f c'bd Vc = 2 5000 psi(12 in.)(7.9 in.) = 13, 400 lb 21.2.1 Shear strength reduction factor: []0.75 = []0 design reinforcement exceeds the minimum ent by Code. required reinforcement As, minn = 0.0018 x 9 in. n. x 12 in. = 0.20 in.2/ft. cr al sections, the required As is greater than At all critical the min minimum. Step 10: Shrinkage and temperature reinforcement rat inforcement ab withh grade 60 bars, thee minim 7.6.4 For one-way slabs rin and temperature 24.4.3.2 mum area of shrinkage temperature (S+T) bars is 0.0018 Ag. Th The maximum spacin spacing of S+T 0.0 reinforcing bar is the lesser of 3h and 18 in. S+T steel area = 0.0018 x 12 in. x 9 in. = 0.20 in.2 Based on S S+T steel area, solutions are No. 4 at 12 in. or No. 5 at 18 in.; use No. 5 at 18 in. placed atop and SHUSHQGLFXODUWRWKHERWWRPAH[XUHUHLQIRUFHPHQW 6WHS0LQLPXPDQGPD[LPXPVSDFLQ]RIAH[XUDOUHLQIRUFHPHQW 7.7.2.1 The minimum spacing between bars must not be 25.2.1 less than the greatest of: (a) 1 in. (b) db (b) 0.625 in. (c) (4/3)(1 in.) = 1.33 in. (c) 4/3dagg Assume 1 in. maximum aggregate size. American Concrete Institute - Copyrighted © Material - www.concrete.org Controls One-Way Slabss Fig. E2.5—Strain distribution. Step 8: Design shear reinforcement, Vn is equal to Vc. Assuming negligible axial force, the Code provides a simple expression: 60 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 7.7.2.2 24.3.2 For reinforcement closest to the tension face, the spacing between reinforcement is the lesser of (a) and (b): (a) 12(40,000/40,000) = 12 in. Controls (b) 15(40,000/40,000) = 2.5(0.75 in.) = 13.1 in. (a) 12(40,000/40,000) = 2.5(0.75 in.) = 13.1 in. (b) 15(40,000/40,000) = 12 in. Controls (c) 15(40,000/40,000) = 2.5(0.75 in.) = 13.1 in. (c) 12(40,000/40,000) = 2.5(0.75 in.) = 13.1 in. (c) 12(40,000/40,000) = 2.5(0.75 in.) = 13.1 in. (c) 12(40,000/40,000) = 12 in. Controls (c) 15(40,000/40,000) = 2.5(0.75 in.) = 13.1 in. (c) 12(40,000/40,000) = 12 in. (c) 12deformed reinforcement is the lesser of 3h and 18 in. 3(7 in.) = 21 in. > 18 in. Therefore, Section 24.3.2 controls; 12 in. Step 12: Select reinforcing bar size Exterior support First midspan Second support Second midspan Middle support No. 4 at spacing, in. 9 7 6 10 hile this continue the top bars across spans. While htly conservative (10 in. versus ersus 12 solution is slightly eer may desire consistent er installation llation and inspe tion. spacing), thee eengineer consistent er installation llation is slightly eer may desire consistent er installation llation is slightly eer may desire consistent er installation llation and inspe tion. SLABS 61 Step 13: Top reinforcing bar length at the exterior support centerline. Bar cutoffs ([WHQGEDUVEH\RQSRLQWIRUQHJDWLYHPRPHQWLVIWIURP support centerline. Bar cutoffs ([WHQGEDUVEH\RQSRLQWIRUQHJDWLYHPRPHQWLVIWIURP support centerline. Bar cutoffs (] + 7.5 in.; Therefore, use 8 in. ~ 7.9 in. 7.7.3.3 Reinforcement shall extend beyond the point at ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWAH[XUHIRUD distance equal to the greater of d and 12db, except at supports of simply-supported spans and at free ends of cantilevers. 7.7.3.8.4 At least one-third the negative moment reinforcement at a support shall have an embedment length EH\RQGWKHSRLQWRILQAHFWLRQDWOHDVWWKHJUHDWHVWRI d, 12db, and En/16. SHUFHQWRIWKHEDUVWRH[WHQGEH\RQGWKHLQAHFWLRQ point at least (19 ft × 12 in./ft)/16 = 15 in. > d = 8 in. > 12db = 7.5 in. Because the reinforcing bar is already at maximum spacing, no percentage of bars (as permitted by Section 7.7.3.3 of ACI 318-14) can be cut off in the tension zone. Bars at the wall connection It is assumed that the wall is placed several days ODE%H EHIRUHWKHZDOODQGWKH FRQQ VODEZLOOEH¿UPO\FRQQHFWHGWKHZDOOZLOOWHQGWR king as it cu restrain the slab from shrinking cures. Many inforcement alo g the slab designers place ex extra reinforcement along o the wall, all, to limit width edge, parallel to widths of possible his restraint. aint. cracks due to this ce lengths hs Step 14: Development and splice eq f calculating devell 7.7.1.2 ACI provides two equations for 25.4.2.3 opment length; VLPSOL¿HGDQGGHWDLOHG,QWKLVH[DPSOHWKHGHtailed]  $y d Ad = | 40 \lambda f c' (cb + K tr) | b | | (d | / | ) (b where zt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ze = coating factor; No. 7 and larger 7.7.1.3 25.5 25.5.1.1 25.5.2.1 c + K tr must not be taken But the expression: b db greater than 2.5. Splice The maximum bar size is No. 5,$ therefore, splicing is permitted. 3.8 ft + 0.5 ft + 1.25 ft = 5.55 ft, say, 6 ft. The development length of a No. 5 black bar in an 7 pment le in. slab with 0.75 in. cover is:  $(3 60, 000 \text{ psi} (1.0)(1.0)(0.8) \text{ Ad} = | (0.625 \text{ in.}) 1.7 \text{ in.} (40 (1.0) 5000 \text{ psi} / = 19 \text{ in.} zt = 1.0, because not more than 12 in. of concrete is placed below bars. } ze = 1.0, because bars are$ uncoated  $z_s = 0.8$ , because bras are smaller than No. 7 1.06 in. + 0 = 1.7 in. 0.625 in. Tension lap splice length,  $\mathcal{E}st$ , for deformed bars in tension must be the greater of: 1.3 $\mathcal{E}d$  and 12 in.  $\mathcal{E}st = (1.3)(19 \text{ in.}) = 24.7 \text{ in.};$  use 36 in. American Concrete Institute – Copyrighted © Material – www.concrete.org One-Way Slabss The top bars have to satisfy the following provisions: 62 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS%RWWRPUHLQIRUFLQJEDUOHQJWKDORQJ¿UVWVSDQ The bottom bars have to satisfy the following provisions: 7.7.3.3 Reinforcement must extend beyond the point at ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWAH[XUHIRUD distance equal to the greater of d and 12db, except at supports of simply-supported spans and at free ends of cantilevers. 7.7.3.4 & RQWLQXLQJAH[XUDOWHQVLOHUHLQIRUFHPHQWLVQRORQ]HUUHTXLUHGWRUHVLVWAH[XUH 7.7.3.5 Flexural IWIURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQHDQDO\VLVE 7KLVFRQGLWLRQLVVDWLV¿HGDORQJDWDQ\VHFWLRQDORQJ the beam span. Note that (b) and (c) do not apply. At least one-fourth the maximum positive moment reinforcement must extend along the slab bottom nimum of 6 in. into the continuous support a minimum \$WSRLQWVRILQAHFWLRQdb for positive moment tensile reinforcementt must be limited such that Ed PHQW IRUWKDWUHLQIRUFHPHQWLVQRWFRQ¿Q E\D EHFDXVHHQGUHLQIRUFHPHQWLVQRWFRQ¿Q E\D EHFDXVHHQGUHQ E Q BAY for an a 9 in. slab with No. 5 at 12 in., 0.75 in. cover is: a) ( M n = As fy | d - | (2) (60,000 psi)(7.9 in. - 0.4 in.) Mn = (0.31 in.2)(140, = 140,000 lb + 8 in. = 58 in. > 19 in. 2800 lb Therefore, No. 5 bar is OK Step 16: First span bottom bar length The bar cut offs that are implicitly permitted from prior Code provisions because of reduced required strength along the span do not apply before the LQÀHFWLRQSRLQWVIRUWKLVVODEEHFDXVHLIDQ\EDUV were cut off, the maximum reinforcing bar spacing would be violated. Because all bottom bars extend past the tensile zone, Section 7.7.3.5 does not apply. All bottom bars need to extend at least 7 in. (refer to Section 7.7.3.3) beyond the positive PRPHQWLQAHFWLRQSRLQWV Ad < The Code requires that at least 25 percent of bottom bars be full length, extending 6 in. into the support. Because the cut off location is close
to the right supSRUWDQGIRU¿HOGSODFLQJVLPSOLFLW\H[WHQGDOOEDUV in. into both supports. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5-ONE-WAY SLABS Step 17: Interior span bottom reinforcing bar lengths, A HFWLRQSRLQWV 7KHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUH ft from the left support centerline and 4.2 ft from the right support centerline. Bar cutoffs 6LPLODUWRWKH¿UVWLQWHULRUVSDQDOOERWWRPEDUV PXVWH[WHQGDWOHDVWLQSDVWLQÀHFWLRQSRLQWV7KH Code requires at least 25 percent of bottom bars be full length, extending 6 in. into the support. Step 18: Top reinforcing bar length at the middle support. ¿QÀHFWLRQSRLQWV 7KHUHDUHQRLQÀHFWLRQSRLQWVRYHUHLWKHUVSDQWKDW frame into the middle support. 63 Create a partial length is 20 ft – 3.6 ft + (2 ft)(0.5) = 13.8 ft, say, 14 ft 0 in. In a repeating pattern, use 3 No. 5 at 14 ft long and 1 No. 5 at 21 ft long. The required reinforcing bar is No. 5 at 12 in. in the PLGGOHVXSSRUW%HFDXVHWKHWRSEDUIURPWKH¿UVWVXSport is No. 5 at 10 in., extend the No. 5 at 10 in. top over the middle support for simplicity. One-Way Slabss Bar cutoffs ill be con Because the top bars will continuous, no bars are cut off. Step 19: Detailing Fig. E2.6—Slab reinforcement detailing. American Concrete Institute - Copyrighted © Material - www.concrete.org 64 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) One-way Slab Example 3: One-way slab post-tensioned - Hotel loading There are four spans of 20 ft-0 in. each, with a 3 ft-0 in. cantilever balcony at each end. The slab is supported by 12 in. walls on the exterior, and 12 in. wide beams on the interior (Fig. E3.1). This example will illustrate the design and detailing of a one-way post-tensioned (PT) slab, both for service conditions and factored loads. Fig. E3.1—One-way slab. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 5—ONE-WAY SLABS ACI 318-14 Discussion Step 1: Geometry 7.3.2 The ACI 318-14 span-to-depth ratios do not apply to PT slabs. The Post-Tensioning Manual, 2006, sixth edition Chapter 9, Table 9.3, suggests a ratio limit of £/48. 65 Calculation For this example, this ratio gives a slab thickness of (20 ft)(12 in./ft)/48 = 5.0 in. This example uses a 6 in. thick slab. 6WHS/RDGVDQGORDGSDWWHUQV 7.4.1.1 For hotel occupancy, the design live load is 40 psf per Table 4-1 in ASCE 7-10. A 6 in. slab is a 75 psf dead load. D = 75 psf + 10 psf = 85 psf 5.3.1 7.4.1.2 U = 1.4D U = 1.2D + 1.6L Both ASCE 7 and ACI CI provide provi guidance for addressing live load patte patterns. Either approach is acceptable. The required strength equations to be considered are:  $U = 1.4(85) = 119 \text{ psf } 1 U = 1.2(85) + 1.6(40) = 102 + 64 = 166 \text{ psf sign to use the following following two ACI 318 allowss the design 3. patterns (Fig. 2007) and ACI CI provide provid$ E3.2): 6.4.2 i applied plied on all spans spa and facFactored dead loadd is tored live load is applied as follows: (a) Maximum positive Mu+ near midspan occurs ZLWKIDFWRUHG/RQWKHVSDQVRQO American Concrete Institute – Copyrighted © Material – www.concrete.org Controls One-Way Slabss The slab resists gravity only and is not part of a lateral-force-resisting system, except to act as a diaphragm. 66 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E3.2—Live load pattern used in elastic analysis of slab for unbalanced loads. Step 3: Concrete and steel material requirements The mixture proportion must satisfy the durability 7.2.2.1 requirements of Chapter 19 and structural strength requirements. The designer determines the durability 7.2.2.2 requirements of Chapter 19 and structural strength requirements. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLVSHFL¿FDWLRQV VSHF By specifying that th the concrete mixture shall be in accordance ce with ACI 3301-10 01- and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG VHV SWHUUHT There are several within ACI al mixture ure options with 301, such as admixtures pozzolans, whic which the m s and pozzolans designer can require, permit, or review iff sugg by the contractor. r Based B ed oon durability ility and strength requirements, and experience mixtures, the compressive strength enc with local cal mixtu RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW RI RQFU SHFL¿HGD ppsi. The reinforcement must satisfy Chapter 20. In this example, unbonded, 1/2 in. single-strand trand tendons are assumed. specifying the reinforcement shall be in accorBy spe dance with ACI 301-10, the PT type and strength, and reinforcing bar grade (and any coatings), Chapter 20 UHTXLUHPHQWVDUHVDWLV¿HG In this example, assume grade 60 bar and no coatings. The designer determines the grade of bar and if the reinforcement should be coated by epoxy or galvanized, or both. 20.3 The Code requires strand material to be 270 ksi, low relaxation (ASTM A416a). 20.3.2.5.1 7KH86LQGXVWU\XVXDOO\VWUHVVHVRUMDFNVPRQRstrand to impart a force equal to the least of 0.80fpu and 0.9fy. immediate and long-term losses will reduce this force. 20.3.2.5.1 where fy = 0.94fu (Table 20.3.2.5.1) 0.80fpu controls, the maximum allowed by the Code. 7KHMDFNLQJIRUFHSHULQGLYLGXDOVWUDQGLV (270 ksi)(0.8)(0.153 in.2) = 33 kip. This is immediately reduced by seating and friction ORVVHVDQGHODVWLFVKRUWHQLQJRIWKHVODE/RQJWHUP losses will further reduce the force per strand. Refer to Commentary R20.3.2.6 of ACI 318-14. A PT force design value of 26.5 kips per strand is common. American Concrete Institute - Copyrighted © Materialwww.concrete.org CHAPTER 5—ONE-WAY SLABS Step 4: Slab analysis 6.3 Because the building relies on the building's other PHPEHUVWRUHVLVWODWHUDOORDGVWKHVODETXDOL¿HV for braced frame assumptions, as discussed in the commentary. 67 Modeling assumptions: Slab will be designed as Class U. Consequently, use the gross moment of inertia of the slab in the analysis. Assume the supporting beams have no torsional resistance and act as a knife edge support. 2QO\WKHVODEDWWKLVOHYHOLVFRQVLGHUHG 6.6 The analysis performed should be consistent with the overall assumptions about the role of the slab within the building system. Because the lateral forceresisting system only relies on the slab to WUDQVPLWD[LDOIRUFHVD¿UVWRUGHUDQDO\VLVLVDGequate. Although gravity moments are calculated independent of PT moments, the same model is used for both. Analysis approach: 7RDQDO\]HWKHÀH[XUDOHIIHFWVRISRVWWHQVLRQLQ]RQ the concrete slab under service loads, the tendon drape is assumed to be parabolic with a discontinuity at the support centerline shown as follows, which imparts a uniform uplift over each span of a prismatic member is calculated as: wp = 8Fa/E2 20.6.1.3.2 ¿OH RVHQWRSURYLGH PD[LPXP] 7KHVWUDQGVSUR¿OHLVFKRVHQWRSURYLGHPD[LPXP ad load and live load. A resistance to dead At the entricity of 0.25 in. n. is chosen exterior supports and midspans, eccentricity is chosen (1 in. cover) )LJ(23RVWWHQVLRQLQJVWUDQGSUR¿OH American Concrete Institute - Copyrighted © Material - www.concrete.org One-Way Slabss where F is the effective PT force is assumed low point). In this eexample am the PT force is assumed low point). In this eexample am the PT force is assumed low point). In this eexample am the PT force is assumed low point. In this eexample am the PT force is assumed low point. to constant eren tendon don drapes. different 68 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Slab stress limits 7.3.4.1 This example assumes a Class U slab; that is, a slab under full service load with a concrete tension 19.2.3.1 stress not exceeding 7.5 f c'. The slab analysis model for the service condition is the same as for the nominal condition. 19.2.3.1 7.5 5000 psi = 530 psi To verify that the concrete tensile stresses are less than 7.5 5000 psi , the net service moments and tensile stresses at the face of supports are needed. This example assumes two parameters: (a) The PT force provides a F/A slab compressive stress of at least 125 psi (9 kip/ft), and E 7KHFRPELQDWLRQRI37IRUFHDQGVWUDQGSUR¿OH provides an uplift force wp to balance at least 75 percent of the slab weight, or 56 psf (Fig. E3.4). The basic equation for concrete tensile stress is: ft = M/S - F/A, where M is the net service moment. ape is 3.25 in. At the exterior support, the drape d force is calculated (Fig. E3.3). The required from: wp = 8Fa/E2 where wp = 0.056 kip/ft S = (12 in.)(6 in.) = 72 in.3 (section modulus), A = (12 in.)(6 in.) = 72 in.2 (gross slab area per foot). F = (0.056 kip/ft)(2 (0.056 kip/ft)(2 0 0 ft fft) 2 = 10.3 kip/ft 8(3.25 in./ft) 8(3 in.)/(12 in./ Fig. E3.4-Uplift force due to tendon layout. 7.3.4.2 The service gravity load is 125 psf. The PT uplift is subtracted from the gravity load. Location from left to right along the span 7.3.4.2 Service loads Cantilever First span Second midspan Gravity uniform uplift, psf 78 69 56 The slab stresses are determined from an elastic analysis using the "net" load, minus the slab's axial compression. Slab axial compression force: F/A = (10.3 kip/ft)(1000 lb/kip) (6 in.)(12 in./ft) = 143 psi. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5—ONE-WAY SLABS 7.3.4.2 19.2.3.1 Location from left to right along the span Exterior support First midspan Interior support Second midspan Net service unbalanced moments, ft-kip/ft 0.2 1.5 2.2 1.1 Net tensile stress, psi [] [] [] The cantilever moment at the support centerline is: wnet 2/2 = (0.078 psf)(3 ft)/2 = 0.351 kip-ft/ft. ACI 318-14 permits to calculate the design slab moment at the faces of interior supports. The aforementioned results show the maximum slab tensile stress (224 psi) calculated for an average PT force of 10.3 kip/ft is less than 7.5 f c' = 530 psi. Therefore, the slab is uncracked and the aforementioned assumption is correct. One-Way Slabss 7.4.2.1 69 American Concrete Institute – Copyrighted © Material – www.concrete.org 70 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS'HÀHFWLRQV 7.3.2.1 This chapter refers to Section 24.2 of ACI 318-14,
3'HÀHFWLRQVGXHWRVHUYLFHOHYHOJUDYLW\ORDGV for allowable stiffness approximations to calculate LPPHGLDWHDQGWLPHGHSHQGHQWORQJWHUP GHÀHFtions. Section 24.2.2 provides maximum allowed span-toGHAHFWLRQUDWLRV Because slab is a Class U, use Iq: 24.2.3.8 Iq = (12 in.)(6 in.)3 = 216 in.4 12 The balanced portion of the total load is offset by the camber from the prestressing, which results in D]HURQHWGHAHFWLRQ8QEDODQFHGORDGKRZHYHU ZLOOUHVXOWLOVKRUWDOGOROJWHUPGHAHFWLROVWKDW must be checked. The following equation, which can be downloaded from the Reinforced Concrete Design Aid - Analysis Tables: . aspx?ItemID=SP1714DA, will be used to calculate FWLRORIWKHVODE DODSSUR[LPDWHPD[LPXPGHAHFWLRORIWKHVODE alanced load (6 span with the largest unbalanced (69 psf). QVGRQR, QJHQHUDOGHAHFWLRQVGRQRWFRQWUROWKHGHVLJQRI PT slabs. "max = (0.0065)(69 psf)(240 in.) 4 = 0.14 in. (4, 030,000 psi)(216 in.4)/(12 in./ft) 000 psi)(2 Expressed ress as a ratio, atio, WLP SHQGHQWGHAHFWL QLVW)7KHDGGLWLRQDOWLPHGHSHQGHQWGHÄHFWLRQLVWKH & E LQ & LQLQ FWL XHWRVXVWDLQHGO DGPXOWL LPPHGLDWHGHÀHFWLRQGXHWRVXVWDLQHGORDGPXOWLnlon and Sup nt, 2011, plied by two (referr to Scanlon Suprenant, 3(VWLPDWLQJ7ZR:D\60DE'HÀHFWLRQV'Concrete International91R-XO\SS 24.2.2 Assume that no portion of the live load is sustained. &DOFXODWHWKHLPPHGLDWHGHÀHFWLRQEDVHGRODWRWDO (0.14 in.)(85 psf - 56 psf)/(69 psf) = 0.06 in. sustained load of 85 psf reduced by the unbalanced Eå LQ E920 load of 56 psf. 7KHORQJWHUPPXOWLSOLHURQWKHLPPHGLDWHGHÀHFtion is 2, so the ratio is: 7KHGHÀHFWLRQUDWLRVDUHPXFKOHVVWKDQWKHOLPLW of EVRGHÀHFWLRQVDUHVDWLV¿HGZLWKRXWPRUH detailed calculations. Step 7: Required moments, including pattern loading, are shown in Fig. E3.5: Fig. E3.5-Moment envelope due to factored loads. American Concrete Institute -Copyrighted © Material – www.concrete.org CHAPTER 5—ONE-WAY SLABS 7.4.1.3 7.4.2.1 71 The Code requires that moments due to reactions induced by prestressing (secondary moments) be included with a load factor of 1.0. Secondary moments are calculated at each support as: M2 = Mpt – Fe. The secondary moment diagram is linear between supports (Fig. E3.6). The PT secondary moments are: Fig. E3.6—Secondary moments, which are the required moments, which are the required strengths, are shown in Fig. E3.7-Moment envelope including factored load effect and secondary moments. Required strength (Gravity only) Face of ex exterior support po First midsp midspan n Face ace of second cond support rt Second econd midspan mid Face of middle support Factored mom. at face, ft-kip [] 5.5 [] 3.6 [] M2, ft-kip [] 5.5 [] 3.6 [] M2, ft-kip [] 5.7 WHQGRQV If the PT tendons alone provide the design strength, . Im the reinforcing bar to be a reduced length. If the PT tendons alone do not provide the design strength, then the reinforcing bar to be a reduced length. If the PT tendons alone do not provide the design strength, then the reinforcing bar to be a reduced length. If the PT tendons alone do not provide the design strength, then the reinforcing bar to be a reduced length. Aps is the tendon area perfoot of slab. UHIHUVWRIRUWKHFDOFXODWLRQRI[]Mn. Section 22.2 for calculation of Mn. Section 22.2 for calculate fps. The span-to-depth ratio is 240/6 = 40, so the following equation applies: f ps = f se + 10,000 + f c' 300p p The reinforcing bar and tendons are usually at the same height at the support and at midspan. American Concrete Institute - Copyrighted © Material - www.concrete.org One-Way Slabss Location L tion from left tto right ght aalong the span 72 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 20.3.2.4 Each single unbonded tendon is stressed to the value prescribed by the supplier. Friction losses cause a variation of fse along the tension length, but for design purposes, fse is usually taken as the average value. The tendon supplier usually calculates fse, and 175,000 psi is a common value. The tendon supplier usually calculates fse, and 175,000 psi is a common value.  $26.8 \text{ kip}/10.3 \text{ kip} \times 12 \text{ in}/\text{ft} = 31 \text{ in., or } 2 \text{ ft}-7 \text{ in. } \text{Aps} = (0.153 \text{ in.})(12 \text{ in}/\text{ft})/(31 \text{ in.}) = 0.059 \text{ in.}2/\text{ft} / 60 \text{ in} 2 = 0.00098 5000 \text{ psi} + 202,000 \text$ 30,000 psi = 205,000 psi and (b) (0.9)(270,000 psi) = 243,000 psi So the design value of fps = 202,000 psi The compression block depth is 5 in. at critical locaH[WH WLRQVH[FHSWDWWKHH[WHULRUMRLQW7KHUHIRUHWKH Code permits a mini minimum d of midspan S Face of middle support []Mn, only tendons, ft-kip 4.16 4.34 44.34 34 4.34 4.34 4.34 Mu, ft-kip [] 5.9 [] 4.3 [] %HFDXVHDOPRVWDOOWKHGHVLJQPRPHQWVDUHJUHDWHUWKDQ]]Mn when considering the tendons alone, standard reinforcement lengths must be used. 6WHS0LQLPXPAH[XUDOUHLQIRUFHPHQW 7.6.2.3 7KHPLQLPXPDUHDRIAH[XUDOUHLQIRUFLQ]EDUSHU foot is a function of the slab's cross-sectional area. Step 10: Design moment strength 7.5.1 The two common strength reduction factor in Section 7.5.1.2 is assumed to be 0.9. As, min = 0.004 × 12 in. × 3 in. = 0.15 in.2/ft. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5—ONE-WAY SLABS 7.5.2 73 Determine if supplying the minimum area of reinIRUFL0JEDULVVXI¿FLH0WWRDFKLHYHDGHVLJQVWUH0JWK that exceeds the required strength. Comparing this value with the required moment strength Mu indicates that the minimum reinforcement plus the tendons supply enough tensile reinforcement for slab to resist the factored loads at Set the section's concrete compression block depth a: Aps f ps + As f y (0.059 in.2) (202,000 psi) + (0.15 in.2) (60,000 psi) + (0.15 in.2) (60, psi)  $a = a = 0.85 \text{ f c}'(12 \text{ in./ft}) (0.85)(5,000 \text{ psi})(12) a = 0.41 \text{ in. For } 3/4 \text{ in. cover: } a) (Mn = \varphi [ Aps f ps + As f y ] | d - | (2) = 0.9[0.059 \times 202,000 + 0.15 \times 60,000](5 - 0.21) = 90,200 \text{ in.-lb} = 7.5 \text{ ft-kip refore, minimum rei reinforcement provides adequate ngth to o resist the app strength applied moment. OK 21.2.1 edu n$ factor: Shear strength reduction Vc = 2 (5000 ( [ps] si (12 in.)(5 in in.) = 8500 lb ()  $\varphi$ Vc = (0.75)(2) 75)(2) 55000 psi (12 in.)(5 in.) = 6364 lb Note: Shear does not typically control the thickness of one-way post-tensioning slab system. The maximum Vu NLSVDWWKH¿UVWLQWHULRUVXSport Therefore, OK. port. American Concrete Institute – Copyrighted © Material – www.concrete.org One-Way Slabss Step 11: Design shear strength im hear is taken at the support 7.5.3.1 The slab's maximum shear im y. centerline for simplicity. 74 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 12: Shrinkage and temperature reinforcement 7.6.4.2 To control shrinkage and temperature stresses in the direction to the span, it is typical to use tendons rather than mild reinforcement in one-way post tensioning slabs. To calculate the number and spacing of temperature tendons, the Code allows the designer to consider the effect of beam tendons on the slab. Assuming the beam is 12 in. x 30 in., the concrete FURVVVHFWLRQDODUHDLQWKHEHDPLQAXHQFHDUHDLV ALQADUHD = (6 in.)(20 ft)(12 in./ft) + (12 in.)(24 in.) = 1726 in.2 Assume the beam has an effective post-tensioning force of 189 kips, which results in an average compression of: 7KLVDPRXQWLVWKHUHIRUHVXI¿FLHQWWRPHHWWKH Code minimum of 100 psi. The Code also has

chree spacing requirements which apply: 1 OE LQ2) = 109 psi. Provide at least one tendon on each side of the beam. oes not exceed If temperature tendon spacing exceeds 4.5 ft, force supplemental reinforcement is required along the DGMD WHQGRQDQF HGJHRIWKHVODEDGMDFHQWWRWHQGRQDQFKRUV ohibited. Spacing above 6 fft is prohibited. rature tendons, sstarting arting at 4 In this example,, ttemperature PD SHFLiHGDWIWR FHQWHU1R IWIURPWKHEHDPDUHVSHFLiHGDWIWR FHQWHU1R rement is ne supplemental edgee rreinforcement needed 6WHS0D[LPXPVSDFLQ]RIÀH[XUDOUHLQIRUFHPHQW 7.7.2.3 7KHPD[LPXPVSDFLQ]RIÀH[XUDOUHLQIRUFLQ]EDULQ 7KHDUHDRIÀH[XUDOUHLQIRUFLQ]EDULQ 7KHDUHDRIÀH] DOVRVDWLV¿HVWKHPD[LPXPVSDFLQ]UHTXLUHPHQW American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5-ONE-WAY SLABS 7.7.3.8.4 The top bars have to satisfy: Reinforcement must extend beyond the point at ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWAH[XUHIRUD distance equal to the At least one-third the negative moment reinforcement at a support shall have an embedment length EH\RQGWKHSRLQWRILQAHFWLRQDWOHDVWWKHJUHDWHVWRI d, 12db, and En/16. In addition, Section 7.7.3.8.4 requires 33 percent of WKHEDUVWRH[WHQGEH\RQGWKHLQAHFWLRQSRLQWDWOHDVW (19 ft)(12 in./ft)/16 = 15 in. The top bars at the exterior MRLQWZLOOH[WHQGIURPWKHHQGRIWKHFDQWLOHYHUSDVW the support and into the span. Because the reinforcing bar is at wide spacing, no percentage of bars (as permitted by Section 7.7.3.8) can be cut off in the tension zone. Balcony considerations VODEUHFHVVRI 7KHDUFKLWHFWXVXDOO\VSHFL¿HVDEDOFRQ\VORSHRI etail about 0.75 in. at the exterior units to guard against water intrusion. In addition, 0\VS WKHDUFKLWHFWXVXDOO\VSHFL¿HVDEDOFRQ\VORSHRI etail result in a slab ab about 1/4 in./ft. The These two details onsider thickness at the ed edge of 4.5 in. Balcony consideruss in detail by Suprenant Supren nt in, "Unations are discussed lco Drainage," 2004, Concr derstanding Balcony Concrete an. International, Jan. Solution 7KHWRS 7KHWRSEDUOHQJWKLVIWEDOFRQ\ SOXVIWLQAHFtion) plus an ex extension of 15 in. A practical length for ars is 6 ft. top bars m ba he outside eedge Trim bar at th T rs usually require two No. 4 continuous The PT suppliers "b ck-u bars bbehind hi d th "back-up" the anchorages, about 2 to 3 in. m th bar can also limit widths of posfrom the edge. These bars si cra unex siblee cracks duee to unexpected restraint, drying shrinkage, or other local issu issues. At the edge of the balcony, PPHQGHGW LWLVUHFRPPHQGHGWRKRRNWKHWRSAH[XUHEDUVDURXQG the continuous edge bars. American Concrete Institute – Copyrighted © Material – www.concrete.org One-Way Slabss Step 14: Top reinforcing bar length at the exterior support 7.7.3 Reinforcing bar lengt DESIGN HANDBOOK—SP-17(14) 6WHS%RWWRPUHLQIRUFLQJEDUOHQJWKDORQJ¿UVWVSDQ 7.7.3 The bottom bars have to satisfy the following provisions: 7.7.3.3 Reinforcement must extend beyond the point at ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWÀH[XUHIRUD distance equal to the greater of d and 12db, except at supports of || || (b 7.7.3.5 Flexural tensile reinforcement must not be terminated in a tensile zone unle unless (a), (b), or (c) is VDWLV2HG Note: the development length of a No. 4 black bar in an 6 inch slab with 0.75 in cover is: ()| 3 60,000 psi 1.0 × 1.0 × 0.8 | Ad = | 0.5 = 13 in | 40 1.0 5000 psi (1.0 + 0) || (| 0.5 ) || Vu'' || V n att the th cutoff t point. DQG V n = 6364 lb (refer to Step 11) Vu OEDQG V IV n = 4243 lb > Vu = 2000 lb therefore, OK V d (c) do not apply. Notee th that (b) and 7.7.3.8.2 7.7.8.3 th the maximum posit At least one-fourth positive moment l reinforcement must extend along the slab bottom into the continuous support a minimum of 6 in. Extend bottom rei reinforcement minimum 6 in. into the supports. ment \$WSRLQWVRILQAHFWLRQdb for positive moment, tensile reinforcement must be limited such that Ed IRUWKDWUHLQIRUFHPHQWVDUH ft from the exterior support centerline and 3.0 ft IURPWKH¿UVWLQWHULRUVXSSRUWFHQWHUOLQH American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 5—ONE-WAY SLABS 77 Bar cutoffs that are implicitly permitted in the aforementioned code provisions do not apply EHIRUHWKHLQAHFWLRQSRLQWVIRUWKLVVODEEHFDXVHLI any bars were cut off, the maximum reinforcing bar spac ing would be violated. Because all bottom bars need to extend at least 5 in. (Refer to Section 7.7.3.3) beyond the positive PRPHQWLQÀHFWLRQSRLQWV The Code requires that at least 25 percent of bottom bars be full length, extending 6 in. into the support. Check bar size Mn is: a) a) ( ( M n = [ As f y ] | d - | + [ Aps f ps ] | d p - | ( ( 2/2) Mn = (0.15 in.2)(202,000 psi)(5 in. - 0.21 in.) = 97.8 in.-kip The elastic analysis indicates that VuDWLQÀHFWLRQ point is 2.0 kips. The term & iss 5 in. & d "Mn/Vu + & Ad < Mn 9 8 in.-kip 97.8 + 5 in. = + 5 = 54 in. Vu 2.0 kip B ause the cut off locat Because location is within a foot of the left ssupport, port extend nd all bott bottom bars 6 in. into left support h to the h edge d of the cantilever. These bottom and then bars will like the bars will like the cut off locat Because location is within a foot of the left ssupport, port extend nd all bott bottom bars 6 in. into left support h to the h edge d of the cantilever. These bottom and then bars will like the bars will like the bars will like the bars of the left ssupport. support sshrinkage and temperature bars in the balcony. alcony. )RU¿HOGSODFLQJVLPSOLFLW\VSHFLI\DOOERWWRPEDUVLQ )RU ¿ this span also extend 6 in. into the right support. 6WHS7RSUHLQIRUFLQJEDUOHQJWKDWWKH¿UVWLQWHULRUVXSSRUW 7.7.3.3 7.7.3.8.4 Bar cutoffs support centerline. Therefore, 6 in. (19 ft x12 in./ft)/16 = 15 in. +RZHYHUWKHWRSEDUVDWWKHH[WHULRUMRLQWZLOOH[tend from the end of the cantilever, past the support and into the span. Because the reinforcing bar is at wide spacing, no percentage of bars (as permitted 7KHUHIRUHWKHWRSEDUVDWWKHH[WHULRUMRLQWZLOOH]tend from the end of the cantilever, past the support and into the span. Because the reinforcing bar is at wide spacing, no percentage of bars (as permitted 7KHUHIRUHWKHWRSEDUVDWWKHH[WHULRUMRLQWZLOOH]tend from the end of the cantilever, past the support and into the span. Section 7.7.3.8) can be cut off in the tension of 15 in. plus 7.5 ft plus 15 in. A practical zone. length for top bars is 16 ft. American Concrete Institute – Copyrighted © Material – www.concrete.org One-Way Slabss !LQWKHUHIRUH1REDULV2. ! LQ RUH1R 78 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 17: Second span bottom reinforcing bar lengths, QAHFWLRQSRLQWV 7KHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUHIW from the left support centerline. Bar cutoffs The bar cut offs that are implicitly permitted by the &RGHGRQRWDSSO\EHIRUHWKHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUHIW from the left support centerline. Bar cutoffs The bar cut offs that are implicitly permitted by the &RGHGRQRWDSSO\EHIRUHWKHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUHIW from the left support centerline. Bar cutoffs The bar cut offs that are implicitly permitted by the &RGHGRQRWDSSO\EHIRUHWKHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUHIW from the left support centerline. Bar cutoffs The bar cut offs that are implicitly permitted by the &RGHGRQRWDSSO\EHIRUHWKHLQAHFWLRQSRLQWVIRUSRVLWLYHPRPHQWVDUHIW from the left support centerline. Bar cutoffs The bar cut offs that are implicitly permitted by the &RGHGRQRWDSSO\EHIRUHWKHLQAHFWLRQSRLQWVIRUWKLV slab, because if any bars were cut off, the maximum reinforcing bar spacing would be violated. Because all bottom bars extend at least 5 in. (Refer to Section 7.7.3.3) beyond the SRVLWLYHPRPHQWLQAHFWLRQSRLQWV The Code requires that at least 25 percent of bottom bars be full length, extending 6 in. into the support. Solution The minimum bottom bar length is (20 ft minus 3.5 IWLQAHFWLRQ SOXVDQH[WHQVLRQRILQPLQXVIW LQAHFWLRQ SOXVLQ\$SUDFWLFDOOHQJWKIRUERWWRP bars is one at 20 ft and three at 16 ft. DWWKH 6WHS7RSUHLQIRUFLQJEDUOHQJWKDWWKHPLGGOHVXSSRUW&/& 7.7.3 ve to t satisfy sfy the following provisions: 7.7.3.3 xtend beyond the point aat Reinforcement mu must extend QJH TXLUHGWRUHVLVW [XUHIRUD ZKLFKLWLVQRORQJHUUHTXLUHGWRUHVLVWÀH[XUHIRUD he greater off d and 12db, except distance equal to the at supports of simply-supported spans and at free ends of cantilevers. 7.7.3.8.4 At least one-third the negative moment reinforcement at a support must have an embedment length EH\RQGWKHSRLQWRILQAHFWLRQDWOHDVWWKHJUHDWHVWRI d, 12db, and En/16. ,QAHFWLRQSRLQWV LQA SRLQWV IRU 7KHLQAHFWLRQSRLQWVIRUQHJDWLYHPRPHQWVDUHIW fr m th rt centerli from the support centerline on both sides. Bar ccutoffs Section 7.7.3.3 requires all bars to extend beyond WKHLQAHFWLRQSRLQWDWOHDVWd (5 in.) or 12 x 0.625 in; therefore, 8 in. In addition, Section 7.7.3.8.4 requires SHUFHQWRIWKHEDUVWRH[WHQGEH\RQGWKHLQAHFWLRQ point at least (19 × 12)/16 = 15 in. Because the reinforcing bar is already at maximum spacing, no percentage of bars (as permitted by Section 11.7.3.5 or 7.7.3.8 of ACI 318-14) can be cut off in the tension can be cut off in the tension zone. Solution 7KHWRSEDUOHQJWKLVWZRWLPHVIWLQAHFWLRQ SOXV an extension of 15 in. at each end) A practical length for top bars is 15 ft. American Concrete.org CHAPTER 5—ONE-WAY SLABS 79 7.7.4.3.1 Post-tensioned anchorage zones must be designed and detailed in accordance with Section 25.9 of ACI 318-14. The concrete around the anchorage is divided into a local zone and a general zone. For monostrand anchorages, the local zone reinforcement, according to Code, "shall meet the bearing resistance requirements of ACI 423.7." ACI 423.7 limits the bearing stresses an anchorage can impose on the concrete, unless the monostrand anchorage is tested to perform, as well as those meeting those stresses. All U.S. manufacturers' supply tested anchorages. For the general zone, Section 25.9.3.1 (a) of ACI 31814 requires two "back up" bars for monostrand anchorages at the edge of the slab, and Section 25.9.3.2 (b) is not applicable to this example. 7.7.4.3.2 Post-tensioning anchorages and
couplers must be designed and detailed in accordance with 25.7. The information in Section 25.7 of ACI 318-14 provides performance requirements for the design of PT anchorage and detailed in accordance with 25.7. The information in Section 25.7 of ACI 318-14 provides performance requirements for the design of PT anchorage and detailed in accordance with 25.7. The information in Section 25.7 of ACI 318-14 provides performance requirements for the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about Section 25.7. Under the design of PT anchorage design, so the engineer rarely (if ever) is concerned about temperature per tendons evenly spaced aced 7.7.6.3 There are 4 temperature W & %HFDXVHWK VSDFLQJ SHUVSDQDWIWLQ2& %HFDXVHWKHVSDFLQJ edg reinforcereinf is less than 4 ft 6 in., noo additional edge d bby the Code. The com mentar ment is required commentary cin temperature tendo recommends placing tendons so that hi the kern off the slab (middle ddl 2 the resultant is within inches). Anchors are usually attached to the outside forms at mid-height of the slab and longitudinally LQ VXFKD manner as to meet this recommendation. Step 21: Detailing Fig. E3.8—Slab reinforcement. American Concrete Institute – Copyrighted © Material – www.concrete.org One-Way Slabss Step 19: Tendon termination There are requirements both for anchorages themselves. 80 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute – Copyrighted © Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS 6.2-Analysis er tto use any analysis proceACI 318-14 allows the designer DQ PHWULFFRPSDWLE LW\ GXUHWKDWVDWLV¿HVHTXLOLEULXPDQGJHRPHWULFFRPSDWLELOLW\ d sserviceability eability requirem ents as long as design strength and requirements ile provisions visions for the D rect are met. The Code includes detailed Direct alent Frame Me od Design Method (DDM) and the Eq Equivalent Method t Element El (EFM), as well as general provisionss for Finite Analysis (FEA). The commentary notes that while the analysis of a slab system is important, the design results should not HG GHYLDWHIDUIURPFRPPRQSUDFWLFHXQOHVVLWLVMXVWL¿HGEDVHG on the reliability of the calculations used in the analysis. 6.2.1 Direct Design Method—The DDM (ACI 318-14, 6HFWLRQ LV D VLPSOL¿HG PHWKRG RI DQDO/VLV WKDW KDV several geometric and loading limitations. Nonprestressed UHLQIRUFHGADWSODWHVADWVODEVDQGZDIAHVODEVFDQDOOEH designed by this method. The code does not permit PT slabs to be designed by DDM. The results of the DDM are the approximate magnitude and distribute the total static moment in the design panel to the column and middle strips are based on papers by Corley, Jirsa, Sozen, and Siess (Corley et al. 1961; Jirsa et al. 1963, 1969; Corley and Jirsa 1970). The total static moment is determined assuming that the reactions are along the faces RI WKH VXSSRUW SHUSHOGLFXODU WR WKH VSDO FROVLGHUHG 2QFH the total static moment is determined, it is then distributed to negative and positive moment areas of the slab. From there, it is further distributed to the column strip and middle strips. 7KH GHVLJQHU XVHV WKHVH PRPHQWV WR FDOFXODWH WKH AH[XUDO reinforcement area in the direction being designed. The designer needs to perform calculations in both directions to determine two-way slab reinforcement. The DDM also provides the design shear at each column. 6.2.2 Equivalent Frame Method—The EFM (ACI 318-14, Section 8.11) can be used for a broader range of slab geometries than are allowed for DDM use, as well as PT slabs. ODWSODWHVADWVODEV DQGZDIAHVODEV FDQDOOEH GHVLJQHG by this method. The EFM assumptions used to calculate the effective stiffness of the slab, torsional beams, and columns DW HDFK MRLQW DUH EDVHG RQ SDSHUV E\ &RUOH\ -LUVD 6R]HQ and Siess (Corley et al. 1961; Jirsa et al. 1963, 1969; Corley and Jirsa 1970). The EFM models a threedimensional slab system by a series of two-dimensional frames that are then analyzed for loads acting in the plane of the frames. The original analysis will work. The analysis calculates design moments and shears along the length of the model. For nonprestressed slabs, the ()0XVHV''0FRHI¿FLHQWVWRGLVWULEXWHWKHWRWDOPRPHQWV into column strips and middle of the QH[WED\DQGLV QH[WED\DQGLV QH[WED\DQGLVGHVL]QHGLQÅH[XUHDVDZLGHVKDOORZEHDP 3 Finite Element Elem 6.2.3 Method—A finite Element Elem 6.2.3 Method—A finite Element Elem 6.2.3 Method great variety of FEA er ssoftware oftware programs form static, tic, dynam that perform dynamic, elastic analysis. Any two-way o-w slab b geometry can be accommodated. Finite eleme ould have element mo models could have beam-column elements that truc ming me model structura framing members along with plane stress leme s; pl ments; and shell elements, brick elements; plate elements; ruer wkD XVHGWRP RUERWKWKDWDUHXVHGWRPRGHOWKHARRUVODEVPDWIRXQGDi ms, wall tions, diaphragms, walls, and connections. The model mesh d should bbe capable of determining the structural size selected RQVHLQVXI¿F UHVSRQVHLQVXI¿FLHQWGHWDLO\$Q\VHWRIUHDVRQDEOHDVVXPStions for member stiffness is allowed. 6.3—Service limits 6.3.1 Minimum thickness<sup>2</sup>)RUQRQSUHVWUHVVHGADWSODWHV DQGADWVODEVWKH&RGHDOORZVWKHGHVLJQHUWRHLWKHUFDOFXODWHVODEGHAHFWLRQVRUVLPSO\VDWLVI\DPLQLPXPVODEWKLFNQHVV\$&,6HFWLRQ 0RVWADWVODEDQGADWSODWH designs simply conform to the minimum thickness criteria DQGWKHUHIRUHGHVLJQHUVGRQRWXVXDOO\FDOFXODWHGHAHFWLRQV for nonprestressed reinforced two-way slabs. The Code does not provide a minimum thickness-to-span ratio for PT two-way slabs, but the ratio for usual conditions is in the range of 37 to 45. 6.3.2 'HÀHFWLRQV<sup>2</sup>)RUQRQSUHVWUHVVHGWZRZD\ADWSODWH RUADWVODEVWKDWDUHWKLQQHUWKDQWKH\$&, PLQLPXPIRU VODEVWKDWUHVLVWVDKHDY\OLYHORDGIRUZDIÀHVODEVDQGIRU 37VODEVWKHGHVLJQHUFDOFXODWHVGHÀHFWLRQV'HÀHFWLRQVFDQ be calculated by EFM, the slab system is modelled in both directions, and the FDOFXODWHGGHÀHFWLRQDWPLGVSDQHOLVWKHVXPRIWKH FROXPQ VWULS GHAHFWLRQ DQG WKH SHUSHQGLFXODU PLGGOH VWULS GHAHFWLRQUHIHUWRWKHFURVVLQJEHDPPHWKRG\$&,5 American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss 6.1—Introduction A two- way slab is usually used in buildings with columns that are approximately evenly spaced, creating a span length in one direction that is within a factor of 2 to the perpendicular direction. Structural concrete two-way slabs, which have been constructed for over 100 years, have taken many forms. The basic premise for these forms is that the slab system transmits the applied loads directly to the supporting FROXPQVWKURXJKLQWHUQDOAH[XUDODQGVKHDUUHVLVWDQFH This chapter only discusses cast-in-place, nonprestressed, and post-tensioned (PT) slabs. The Code (ACI 318-14) allows for either bonded tendons in a PT slab. Because bonded tendons are not usually placed in two-way slabs in the U.S., this chapter only discusses PT slabs. with unbonded tendons. At the preliminary design level, with spans given by the architect, the designer determines the loads, reinforcement s. The type (prestressed or nonprestressed), and slab thickness. perience and the preliminary concrete strength is based on experience ons. Code's exposure and durability provisions. 82 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 7KH FDOFXODWHG GHÀHFWLRQV PXVW QRW H[FHHG WKH OLPLWV LQ Section 24.2 of ACI 318-14. For most buildings, the limit of EIRUORQJWHUPGHÀHFWLRQVXVXDOO\FRQWUROV Note that the spacing of slab reinforcing bar to limit crack width, timing of form removal, concrete quality, timing of construction loads, and other construction variables all can DIIHFWWKHDFWXDOPHDVXUHGGHAHFWLRQ7KHVHYDULDEOHVVKRXOG EH FRQVLGHUHG ZKHQ DVVHVVLQJ WKH DFFXUDF\ RI GHAHFWLRQ7KHVHYDULDEOHVVKRXOG EH FRQVLGHUHG ZKHQ DVVHVVLQJ WKH A PT slab thickness-to span ratios in WKH UDQJH RI WR VODE GHAHFWLRQV DUH XVXDOO\ ZLWKLQ the Code allowable limits. The Code limits the maximum service concrete tensile stress, so GHAHFWLRQFDOFXODWLRQVXVHWKHJURVVVODESURSHUWLHV 6.3.3 Concrete service stress—Nonprestressed slabs are designed for strength without reference to a pseudo-concrete VHUYLFHAH[XUDOVWUHVVOLPLW )RU 37 VODEV WKH DQDO/VLV RI FRQFUHWH AH[XUDOVWUHVVLQQH]DWLYHPRPHQW areas at columns PT slab is limited to 6 f c'. At positive lab oment sections, Section 8.6.2.3 requires slab This bottom bar if the concrete tensile stress exceeds 2 f c'. T ed. Th reinforcing bar is often not required. These service tensile KH FR AH[XUDO VWUHVV OLPLWV DUH EHORZ WKH FRQFUHWH FUDFNLQJ VWUHVV FWR FLQJGHAHFWLRQ Q of 7.5 f c' WKXVKDYLQJWKHHIIHFWRIUHGXFLQJGHÀHFWLRQV,Q es a PT slab's axial comprescom
esaddition, Section 8.6.2.1 requires ue to post-tensioning to bee at sive stress in both directions due least 125 psi. VVH LQ D GHVLJQ VWULS DQ %HIRUH WKH VODE ÀH[XUDO VWUHVVHV VWULS FDQ & G 7KH EH FDOFXODWHG WKH WHQGRQ SUR¿OH QH QHHGV WR EEH GGH¿QHG SUR¿OHDQGWKHWHQGRQIRUFHDUHGLUHFWO\UHODWHGWRWKHVODE forces and moments is to use the "load balancing" g FRQFHSWZKHUHWKHSUR¿OHLVXVXDOO\WKHPD[LPXPSUDFWLFDO FRQVLGHULQJFRYHUUHTXLUHPHQWVWKHWHQGRQSUR¿OHLVSDUDbolic, the parabola has an angular "break" at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that the tendon terminates at middepth at the column centerlines, and that terminates at middepth at the column centerlines, and terminates at middepth the support. These loads are then combined with the gravity loads, and the analysis is performed with a QHWORDG)LJVKRZVWKHFRPPRQO\XVHGVLPSOL¿FDWLRQ RIWKHWHQGRQSUR¿OH1VVPRRWKZLWK reverse parabolas over the interior supports rather than cusps. To conform to the Code stress limits, the designer can use an iterative approach or a direct approach. In the iterative DSSURDFKWKHWHQGRQSUR¿OHLVGH¿QHGDQGWKHWHQGRQIRUFH LV DVVXPHG 7KH DQDO\VLV LV H[HFXWHG ÀH[XUDO VWUHVVHV DUH FDOFXODWHGDQGWKHGHVL]QHUWKHQDGMXVWVWKHSUR¿OHRUIRUFH or both, depending on results and design constraints. In the direct approach, the designer determines the highest tensile stress permitted, then rearranges equations so that the analysis calculates the tendon force needed to achieve the stress limit. Fig. 6.4.1—Punching shear failure. Fig. 6.4.2—Critical .2— section ggeometry. 6.4— hea strength ength 6.4— Shear abs must have h Two-way slabs adequate one-way shear strength esign strip (assuming the slab is a wide, shallow in each design beam) and ade adequate two-way shear strength at each column. The discussion for the nominal one-way shear strength are the same as provided in Chapter 7 (Beams) of this Handbook and is not reproduced here. 6.4.1 Punching shear strength — Two-way shear strength, also called punching shear strength hased on the slab's concrete strength and shear reinforcement when provided. The effect of the slab's AH[XUDOUHLQIRUFHPHQWRQSXQFKLQJVKHDUVWUHQJWKLVLJQRUHG The assumed punching shear failure shape (Fig. 6.4.1) is usually a truncated cone or pyramid-shape surface around the column. 6.4.2 Critical section—For geometric simplicity, ACI 318 assumes a critical section. This is a vertical section extended from the column at distance d/2, where d is the slab's effective depth. In Fig. 6.4.2, the critical perimeter is bo = 2[(c1 + d) + (c2 + d)]. The critical section to calculate concrete shear strength limits are given in terms of stress. As shown below, the shear stress limit for a nonprestressed reinforced slab is the least of three expressions: The Code punching shear strength limit for PT slabs are usually slightly higher than those for nonprestressed rein- American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 83 Table 6.4.3—Calculation of vc for two-way shear (ACI 318-14 Table 22.6.5.2) vc  $4\lambda$  f c' /HDVWRI (4) 2 + | $\lambda$  f c' |  $\beta$ / ( $\alpha$ sd) || 2 + b |/ $\lambda$  f c' o Note:  $\ddot{u}$  is the ratio of long side to short side of the columns, and 20 for corner columns, 30 for edge columns, and 20 for corner columns, 30 for edge columns, and 20 for corner columns. () (a) vc = 3.5\lambda f c' + 0.3 f pc + b b ( $\alpha$  ) od where  $[s is the same as in the column and <math>[s is 40 \text{ for interior columns}, 30 \text{ for edge columns}, 30 \text{ f$ Table bl 6.4.3, 3, the value of fpcc is ire s, limited to 500 psi, the average of fpc in the two directions, ective prestress force rce Vp is the vertical section, and the to 70 psi. Because of the shallow depth of most PT slabs, many engineers conservatively ignore the Vp/bod component when calculating vc. The Code also requires the engineer to consider slab openings close to the column. Such openings, which are commonly used for heating, ventilating, and air condiWLRQLQJ+9\$& DQGSOXPELQJFKDVHVZLOOUHGXFHWKHVKHDU strength. The Code requires a portion of bo enclosed by VWUDLJKWOLQHVSURMHFWLQJIURPWKHFHQWURLGRIWKHFROXPQDQG tangent to the boundaries of the opening to be considered ineffective (ACI 318-14, Section 22.6.4.3). 6.5—Calculation of required shear strength The factored punching stren stresses due to moments transferred from the slab to the column. The two stress diagrams are added and the total is the required shear stress vug is calculated by vug = Vu/bod. To FDOFXODWHWKHVKHDUVWUHVVHVGXHWRVODEEHQGLQJ\$&, ¿UVW stipulates that a percentage of the unbalanced slab moment at the column, MscLVUHVLVWHGE\VODEÀH[XUHZLWKLQDOLPLWHG width over the column. The remaining percentage of Msc is transferred to the slab by eccentricity of shear. The follow two sections of Chapter 8 (ACI 318-14) state that: Fig. 6.5—Assumed distribution of shear stress (ACI 318-14, Commentary Section ection in accordance with Section 8.4.4.1 (ACI 318-14), where  $\hat{U}v \pm \hat{U}f$  Under certain circumstances given in Table 8.4.2.3.4 of \$&, WKH YDOXH RI  $\hat{U}f$  can be increased, which then decreases the fraction of Msc required to be transferred by eccentricity of shear. 1RWHWKDWWKHVHPRGLiHGYDOXHVGRORWDSSO\IRU37VODEV The slab shea stresses due to the unbalanced moment transferred to the column by eccentricity of shear is calcuODWHGE\UvMscc/Jc, where c is the distance from b0 to the critical section about its centroidal axis. When vug is added, the total shear stress diagram is shown by Fig. 6.5. If the maximum total factored shear stress, the slab's concrete shear stress, the slab stress, the slab's concrete shear stress, the slab's concrete shear stress, the slab's concrete shear stress, the slab stress, the slab's concrete shear stress, the slab's concrete shear stress, the slab stress, the slab's concrete shear stress, the slab stress, the added. At times, the design of a two-way slabs requires point loads to be considered, such as wheel loads in parking garages. American Concrete.org Two-Way Slabss forced slabs as shown in the following Eq. (a) and (b). For PT two-way slabs, the designer can use Eq. (a) and (b) unless the column is ous edge than four times the slab thickness h. For many edge columns, this requires shear strength to be calculated by Table 6.4.3. For prestressed, two-way members, vc is permitted to be the lesser of (a) and (b) (ACI 318-14, Eq. 22.6.5.5 (a) and (b): 84 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Table 6.6a vc for two-way members with shear reinforcement (ACI 318-14, Table 22.6.6.1) Type of shear reinforcement Maximum vc at critical VHFWLRQVGH¿QHGLQ\$&, 318-14, Section 22.6.4.2 Stirrups 2\lambda f c' 2\ contribution of shear reinforcement is calculated by: vs = Av fyt/bos (ACI 318-14, Eq. (22.6.8.2)). Table 6.6b—Maximum vu at critical sections GH¿QHGLQ Stirrups φ6 f c' Headed shear stud reinforcement φ8 f c' These result in local shear slab stresses, and thee slab slab'ss REHYHUL¿HG SXQFKLQJVKHDUVWUHQJWKLQWKDWDUHDQHHGVWREHYHUL¿HG force 6.6-Calculation of shear reinforcement can be provided umn. Assuming the he nominal shear strength close to a column. mly spaced, ed, shear streng h is shear reinforcement is uniformly strength DO VHFWLRQ RQ DDW d/2 beyond yon the ¿UVW FKHFNHG DW WKH ¿UVW FULWLFDO trib n of shear reinforced at d/ dd/2 2 beyon he ment. The shear strength is then ch checked beyond the outermost peripheral line of shear reinforcement, without the contribution of shear reinforcement. In slabs without shear reinforcement, the concrete contribution to shear strength is limited to the values in Table 6.6a. There is an upper limit to a slab's nominal shear reinforcement, the concrete contribution to shear strength is limited to the values in Table 6.6b. The Code states this limit in terms of the maximum factored two-way shear stress, vu, calculated at a critical section. Note that the use of stirrups as slab shear reinforcement is limited to slabs with an effective depth d that satisfy (a) and (b): (a) d is at least 6 in. (b) d is at least 16 db, where db is the diameter of the stirrups The use of shear studs is not limited by the slab thickness, EXW WKH VWXGV PXVW ¿W ZLWKLO WKH JHRPHWULF HOYHORS 7KH overall height of the shear stud assembly needs to be at least the thickness of the slab minus the sum of (a) through (c): D &
ROFUHWHFRYHUROWKHWRSÀH[XUDOUHLOIRUFHPHOW (b) Concrete cover on the base rail F 20HKDOI WKH EDU GLDPHWHU RI WKH AH[XUDO WHQVLRQ reinforcement 6.7—Flexural strength After the designer calculates the factored slab moments, WKHUHTXLUHGDUHDRIAH[XUDOUHLQIRUFHPHQWRYHUDVODEZLGWK is calculated with the same behavior assumptions as a beam. 6.7.1 Calculation of required moment strength—There are two calculations for required moment strength for two-way VODEV7KH ¿UVW FDOFXODWLRQ LV WR GHWHUPLQH IDFWRUHG PRPHQWV over the entire panel in the positive and negative moment areas. For nonprestressed reinforced slabs, the slab analysis should provide the distribution of panel factored moments to the column strip and middle strip. For PT slabs, effects of reactions induced by prestressing (secondary moments) need to be included. The slab's secondary moments are a result of the column's vertical restraint of the slab against the PT load at each support. Because the PT load at each support. calculated by the load-balancing analysis concept. A simple way to calculate the secondary moment is to subtract the tendon force times slabFROXPQMRLQW7KHYDOXHRIMsc is the difference between the design moments on either side of the column. 6.7.2 Calculation of design moment strength—In a nonprestressed slab, the required reinforcement area As resisting ting the column and middle strip's negative and posiu placed uniformly across each strip. The tive Mu is usually required reinforcement area AsUHVLVWLQJUfMsc must be placed together ogether are wit a wid within width bslabb. T slabs, sl For PT the tendons are banded, which is where all d are placed together ogether in a line that follows the column tendons ines iin one direction on and un lines uniformly distributed in the other. 7KHV EAH VWUHQJWKF 7KHVODEAH[XUDOVWUHQJWKFDOFXODWLRQVIRUWHQGRQVZLWKf ps ddetermined d from mS Section i 20.3.2.4 of ACI 318-14 substituted equation) in the banded direction are the same, regardless of the tendon's uniform horizonta location within the slab. horizontal For PT slabs, th reinforcement area As resisting the panel's negative Mu is usually placed only at the column region. The As plus Apt resisting ÛfMsc must be placed within a width bslab per ACI 318-14, Section 8.4.2.3.3. If the panel reinforcement already within bslabLVQRWVXI¿FLHQWGHVLJQHUV usually add only AsWRLQFUHDVHWKHÀH[XUDOVWUHQJWK For PT slabs, the Apt provided to limit concrete service WHQVLOH VWUHVVHV ZLOO XVXDOO\ EH VXI¿FLHQW WR DOVR UHVLVW WKH panel's positive Mu. 6.8—Shear reinforcement detailing 6.8.1 Stirrups are provided to increase shear strength, ACI 318 provides limits on their location and spacing in Table 6.8.1. The related ACI 318-14 Commentary Fig. R8.7.6d as shown in the following Fig. 6.8.1 of this Handbook, also includes the two critical section locations: 6.8.2 Shear stude are provided to increase stude are provided to increase shear stude are provided to in following Fig. 6.8.2, which also includes the two critical section locations. American Concrete Institute - Copyrighted © Material - www.concrete.org 6.9—Flexure reinforcement area and placing<sup>2</sup>7KH&RGHUHTXLUHVDPLQLPXPDUHDRIÀH[XUDO reinforcement in tension regions, with the area as shown in Fig. 6.8.1—Arrangement of stirrup shear reinforcement, interior column face Parallel to column face Paral in. Distance from column IDFHWR¿UVWVWLUUXS d/2 Spacing between stirrups d/2 Spacing between vertical legs of stirrups 2d 85 Table 6.9.1 (ACI 318-14, Table 8.6.1.1). If more than the minimum area is required by analysis, that reinforcement area must be provided. 7ZRZD\ VODE AH[XUDO UHLQIRUFHPHQW LV SODFHG LQ WRS and bottom layers. For nonprestressed reinforced two-way slabs without beams, Fig. 6.9.1 (ACI 318-14, Commentary Fig. 8.6.1.1) provides the minimum reinforcing bar extensions, lap locations, and the minimum As at various sections. If the panel geometry is rectangular rather than square, the outer layer is usually placed parallel to the longer span. 6.9.2 Corners—Corner restraint, created by walls or stiff beams, induces slab moments in the diagonal direction and perpendicular to the diagonal. These moments are in addiWLRQ WR WKH FDOFXODWHG AH[XUDO PRPHQWV \$GGLWLRQDC UHLQforcement per ACI 318-14, Section 8.7.3, is required for this condition. 6.9.3 Post-tensioned slab – Reinforcing bar area and placing<sup>2</sup>2YHU HDFK FROXPQ UHJLRQ WKH &RGH UHTXLUHV DQ DUHDRIAH[XUDOUHLQIRUFLQ]EDURIDWOHDVWAcf in each direction, placed within 1.5h of the column. The Code also requires reinforcing bar in positive moment VLIWKHFDOFXODWH DUHDVLIWKHFDOFXODWHGVHUYLFHWHQVLOHAH[XUDOVWUHVVLQRWKHU sua ly midspan att tth areas (usually the bottom) exceeds 2 f c' or if d for strength. ngth. Botto required Bottom bar placement is at the discrehe ddesigner. er. tion of the The Code allows for a red reduced top and bottom minimum gths if the design esign stre bar llengths strength, calculated with only the ten ons is at least ast the re PT tendons, required design strength. The top EDUVP VWH[WHQGDWOHDVWERQHDFKVLGHRIWKHFROXPQ7KH E E QHHGHG G G ERWWRPEDUVLIQHHGHG PXVWEHDWOHDVWEDQGEHFHQWHUHG mum mom at the maximum moment. The shorter lengths often control underr typical spa spans and loadings. If the sectional strength XVLQJ RQO\ RQO WKH DUHD RI 37 LV LQVXI¿FLHQW WR VDWLVI\ GHVLJQ XVLQJ strength, then the minimum top and bottom bar lengths are the same as a nonprestressed reinforced slab. 6.9.4 Post-tensioned slab – Tendon area and placing—A minimum of 125 psi axial compression in each direction and in the code allows banding of tendons in one direction and in the code allows banding of tendons in one direction and in the code allows banding of tendons in one direction and in the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the Code allows banding of tendons in one direction and in the code allows banding of tendons in one direction and in the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the code allows banding of tendons in one direction and in the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the code allows banding of tendons in one direction and in the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the code allows banding of tendons are XVXDOO\SODFHGLQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ other direction the tendon spacing is uniform across the design panel, within the spacing limits of 8h and 5 ft. This layout is predominant in the U.S. The Code also requires at least two tendons to be placed within the column Table 6.8.2—Shear stud location and spacing limits (ACI 318-14, Table 8.7.7.1.2) Direction of measurement Perpendicular to column face Parallel to column face Description of measurement 'LVWDQFHIURPFROXPQIDFHWR¿UVW peripheral line of shear studs Constant spacing, in. All d/2 Nonprestressed slab with vu'[f c' 3d/4 Nonprestressed slabs conforming to Section 22.6.5.4 of ACI 318-14. 3d/4 All 2d American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss CHAPTER 6—TWO-WAY SLABS 86 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig 6.8.2—Typical arrangements of headed shear stud reinforcement and critical sections (ACI 318-14, Commentary Fig. R8.7.7) Table 6.9.1—As,min for nonprestressed two-way slabs Reinforcement type Deformed bars or welded wire r reinforcement cage in either direction for overall building integrity. 6.9.5 Slab openings—For relatively small slab openings, trim reinforcing bar usually limits crack widths that can be caused by geometric stress concentrations and provides adequate strength. For larger openings, a local increase in slab thickness as well as additional reinforcement may be necessary to provide adequate serviceability and strength. REFERENCES American Concrete Institute (ACI) \$&,5<sup>2</sup>&RQWURORI'HÅHFWLRQLQ&RQFUHWH6WUXFtures (Reapproved 2000) Authored references Corley, W. G.; Sozen, M. A.; and Siess, C. P., 1961, "Equivalent-Frame Analysis for Reinforced Concrete Slabs," Structural Research Series No. 218, Civil Engineering Studies, University of Illinois, June, 166 pp. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS 87 & RUOH (\*\*DQG-LUVD-23(TXLYDOHQW)UDPH Analysis for Slab Design," ACI Journal Proceedings9 No. 11, Nov., pp 875-884. LUVD-26R]HQ0\$DQG6LHVV&33(IIHFWV RI 3DWWHUQ /RDGLQJV RQ 5HLQIRUFHG &RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG &RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & RQFUHWH )ORRU 60DEV Proceedings\$6& Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HQ0\$DQG6LHVV&33DWWHUQ /RDGLQJV RQ 5HLQIRUFHG & Civil Engineering 6WXGLHV8UEDQD,/-XO\ -LUVD-26R]HV8UEDQD,/-XO\ -LUVD-26R]HV (91R67-XQHSS Two-Way
Slabss Fig. 6.9.1—Arrangement of minimum reinforcement near the top of a two-way slab (ACI 318-14, Commentary Fig. R8.6.1.1). American Concrete Institute – Copyrighted © Material – www.concrete.org 88 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6.10—Examples Two-way Slab Example 1: Two way slab design using direct design method (DDM) - Internal Frame This two-way slab is nonprestressed without interior beams between supports. This example designs the internal strip along JULGOLQH%0DWHULDOSURSHUWLHVDUHVHOHFWHGEDVHGRQWKHFRGHUHTXLUHPHQWVRI&KDSWHUVDQGHQJLQHHULQJMXGJPHQWDQG ORFDOO\DYDLODEOHPDWHULDOV/DWHUDOORDGVDUHUHVLVWHGE\VKHDUZDOOVWKHUHIRUHWKHGHVLJQLVIRUJUDYLW\ORDGVRQO\'LDSKUDJP design is not considered in this example. Given: 8QLIRUPORDGV Self-weight dead load is based on concrete density including reinforcement at 150 lb/ft3 Superimposed dead load D = 0.015 kip/ft2 /LYHORDGL = 0.100 kip/ft2 Material properties: fcg = 5000 psi fy = 60,000 psi fy = 60,000 psi (DDM) in Section 8.10. Method (DDM) in Section 8.10. M 8.10.1.1 8.10.2.1 7KHVODEJHRPHWU\VDWLV¿HVWKHOLPLWVRI6HFWLRQV 8.10.2.2 8.10.2.3 8.10.2.1 through 8.10.2.4, which allows the use of 8.10.2.4 box three continuous three continuous at least three continuous three c spans in each direcWLRQVR6HFWLRQLVVDWLV¿HG The successive spans are the same lengths so Section LVVDWLV¿HG The ratio of the longer to the shorter panel dimension LVVR6HFWLRQLVVDWLV¿HG The ratio of the longer to the shorter panel dimension LVVR6HFWLRQLVVDWLV↓HG The ratio of the shorter panel dimension LVVR6HFWLRQLVVDWLV↓HG Th JUDYLW\RQO\VR6HFWLRQLVVDWLV¿HG Ratio of unfactored live load to unfactored dead load is approximately 100/102.5 = 0.98. This ratio is less WKDQVR6HFWLRQLVVDWLV¿HG re are ar no o supporting beams so Section 8.10.2.7 is e not app applicable. 8.3.1.1 KLF VIRUGHAHFWLRQ QWURO &KHFNWKHVODEWKLFNQHVVIRUGHAHFWLRQFRQWURO Th oes not in Thiss ex example does include drop panels or shear ca capss so Sectionss 8.2.4 an and 8.2.5 are not applicable. U ing T Using Table 8.3.1.1 wi with fy = 60,000 psi, without drop panels, and assumin assuming the wall performs as a stiff edge he minimu beam, the minimum thickness for the external panel is: The minimum thickness for the internal panels is calculated using the same table and the result is the same table and the result is the same as that for the building is using a slab thickness of 7 in.; therefore, use 7 in. The slightly thicker than QHFHVVDU\VODEDLGVZLWKERWKGHAHFWLRQVDQGVKHDU 8.3.1.3 1RFRQFUHWHARRU¿QLVKLVSODFHGPRQROLWKLFDOO\ZLWKWKHARRUVODE 8.3.2 &DOFXODWHGGHAHFWLRQVDUHQRWUHTXLUHGEHFDXVHWKH VODEWKLFNQHVVWRVSDQUDWLRVDWLV¿HV6HFWLRQ of ACI 318-14. 6WHS/RDGDQGORDGSDWWHUQV 8.4.1.1 The load factors are provided in Table 5.3.1 of ACI The load combination that controls is 1.2D + 1.6L. 318-14. %HFDXVH6HFWLRQLVVDWLV¿HGLQ6WHSSDWWHUQ loading need not be checked. American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss A n 192 in. = = 5.8 in. 33 33 90 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Initial two-way shear check Before performing detailed calculations, it is often EHQH¿FLDOWRSHUIRUPDQDSSUR[LPDWHSXQFKLQ] shear check uses the following limits on the ratio of the design shear strength to the effects of shear stress based on direct shear stress alone ([]vn/vug): For interior columns: []vn/vug• For edge columns: []vn/vug• For edge columns: []vn/vug• For corner columns: []vn/vug• For edge columns: []v options for adding twoway shear strength may be considered. For []vn, the calculations here are discussed in Step 10 more fully:  $vn = 4 \text{ f c}' = 4 5000 \text{ ksi} = 0.283 \text{ ksi } 1000 (4) vn = |2 + |\langle \beta / (\alpha d) vn = |2 + s | bo / \langle f c' = 6 5000 \text{ ksi} = 0.275 \text{ ksi} * 1000 * controls \phi vn = 0.75 \times 0.275 \text{ ksi} = 0.206 \text{ ksi } For vug$ , the calculations here are discussed in Step 7 more fully: vug = Vu bo d 29.6 in.  $\times$  2 and corner columns will not need to be checked in this example. Step 4: Analysis - Direct design method moment determination 8.4.1.5 8.4.1.6 8.10.3.1 The design strip is bounded by the panel center line on each side of the column line and consists of a column strip and two half-middle strips. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS 8.10.4 Because all lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOOVWKH bined with the lateral loads are assumed to be resisted by 1 VKHDUZDOVWKH bined with the lateral loads are assumed to be resisted by 1 VKH factored moments. Mo = qu A 2 A 2n 8 /RQJVSDQ En = 16 ft E2 = 14 ft qu = 1.2 \*(DLsuper + DLslab sw) + 1.6 × LL = 283 psf q A A2 M o = u 2 n = 127 ft-kip 8 Distribute Mo in the end span, from A to B. American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss )LJ(2\*HRPHWU\GH¿QLWLRQVRISDQHOVDQGVWULSV VRI HOVDQGVWULSV 8.4.1.9 The lateral loads in this building are assumed to be resisted by shear walls. The slabs only acting as diaphragms between the shear walls. The slab LVDVVXPHGQRWWRFRQWULEXWHÀH[XUDOUHVLVWDQFHWR lateral loads. 8.4.2 moments are calculated using 8.4.2.1, the DDM of Section 8.10. 8.4.2.2 8.10.1, The slab is eligible for design by DDM as shown in 8.10.3.2.1, directions and column dimensions in plan. 8.10.3.2.2, 8.10.3.2.3 8.10.3.2.3 8.10.3.2.7 He DDM calculates a total panel Mo and then uses [QWVWRGHWHUPLQHPD[LPXPSRVLWLYHDQG negative design moments. In this example, all spans have  $\epsilon_n = 16$  ft. If spans vary, Mo must be calculated for each span length. 91 92 8.10.4.1 8.10.4.2 8.10.4.3 8.10.4.2 gives the MoGLVWULEXWLRQFRHI¿FLHQWV for the slab panel. In Table 8.10.4.2, this example uses the fully restrained column of the table. The reason for this is that the combined member of the wall and column is much stiffer than the slab and little rotation is expected at the slab-to-wall connection. Section 8.10.4.3 gives the option of modifying the factored moments by up to 10 percent, but that allowance is not used in this example. Section 8.10.4.4 indicates the negative moments are at the face of the supporting columns. Section 8.10.4.5 requires that the greater value of the support of th total panel factored moments from 8.10.4 to the column and middle strips for the end span, from A to B. After distributing the total panel Mu as described n 8. able 8.10.5.2 then earlier in Section 8.10.4, Table proportions the ex exterior negative Mu as assumed med to be resisted by the co column strip strip. In Table 8.10.4.2, for the exterior edge being fully restrained: Negative Mu at face of exterior column = 0.65Mo = 83 ft-kip Maximum positive Mu = 0.35Mo = 45 ft-kip Negative MuDWIDFHRI¿UVWLQWHULRUFROXPQ Mo = 83 ft-kip In Table 8.10.5.2, 0.5.2, 2/E1 DQG[f1 = 0. umi thee wall beha Assuming behaves as a beam, C is calculated WRGHWHUPLQHù WR HWHU Eq (8.10.5.2, 0.5.2, E/E1 DQG[f1 = 0. umi thee wall beha Assuming behaves as a beam, C is calculated WRGHWHUPLQHù WR HWHU Eq (8.10.5.2(a) and (b)) t using Eq. Ecb C  $\beta t = 2 \text{ Ecs I s} (x) x^3 y C = |1 - 0.63| (y) 3 x = 10 \text{ in. } y = 120 \text{ in. } C = 37,900 \text{ in.4 Ecb} = \text{Ecs bh3 168 in. } x (7 \text{ in.}) 3 = 4802 \text{ in.4 12 12 37,900} \beta t = 3.92 \times 4802 \text{ Is} = 8.10.5.5 8.10.6 \text{ After distributing the total panel Mu as described earlier in Section 8.10.4, Table 8.10.5.5 then proportions the positive Mu assumed to be resisted$ by the column strip. The total panel Mu from 8.10.4 is distributed into column strip moments and middle strip moments. The middle strip Mu is the portion of the total panel Mu not resisted by the column strip. In Table 8.10.5.5,  $\mathcal{E}2/\mathcal{E}1$  DQGIf1 = 0. Therefore, the top line of the total panel Mu not resisted by the column strip. In Table 8.10.5.5,  $\mathcal{E}2/\mathcal{E}1$  DQGIf1 = 0. Therefore, the top line of the total panel Mu not resisted by the column strip. amounts distributed to the middle strips. Subtract the amounts distributed to the column strips in Section 8.10.5 from the panel Mu calculated in Section 8.10.4. Mu, int. neg, ms = 83 ft-kip - 63 ft-kip = 20 ft-kip Mu, pos., ms = 45 ft-kip - 27 ft-kip = 18 ft-kip - 63 ft-kip = 20 ft-kip Mu, pos.,
ms = 45 ft-kip - 63 ft-kip = 20 ft-kip Mu, ext. neg, ms = 83 ft-kip - 63 ft-kip Mu, ext. neg, ms = 83 ft-kip © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 8.10.7.3 The gravity load moment transferred between slab and edge column by eccentricity of shear is 0.3Mo. 8.10 Repeat the twoway shear is 0.3Mo. 8.10 Repeat the twoway shear is 0.3Mo. 8.10 Repeat the two shear is 0.3Mo. 8.10 in the slab at the exterior column in 8.5. However, because of the wall, two-way shear does not apply to the design at the exterior column. The results are shown for interior panels, with the same negative Mu, neg, ms = 20 ft-kip Mu, pos, ms = 18 ft-kip Fig. E1.3—Final moment distribution but Step 5: Required strength – Factored slab moment resisted by the column 8.4.2.3 Slab negative moments at a column can be unThee column. This difference in slab moments, Msc, must  $\gamma$  f = 2 b1 8.4.2.3.3 be transferred into the column, usually by a combi1+ 3 b2 8.4.2.3.4 QDWLRQRIAH[XUHRUVKHDU(T FDOFXODWHV 8.4.2.3.5 a factor that determines the fraction of Msc trans8.4.2.3.6 IHUUHGE\AH[XUHRUVKHDU(T FDOFXODWHV 8.4.2.3.7) a factor that determines the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fraction of Msc trans8.4.2.3.7 me effective slab width to resist UfMsc is the fractive slab width to resist UfMsc i width This concentration of reinforcement within the effective slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI¿FLHQW WKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ6HFWLRQUHTXLUHVVXI to slab width is considered during the detailing of of the column plus 1.5h of the slab on either slab on either slab width is considered during the detailing of of the column plus 1.5h of the slab on either slab on either slab width is considered during the detailing of of the column plus 1.5h of the slab on either panel moments. resist ÜfMsc. The moment diagram is symmetric about the axis of the column in the center of the building (108 ft). Note that using this moment diagram will result in a net zero Msc7KH"0XVHVDQDUWL¿FLDOXQEDODQFHGORDG condition in Section 8.10.7 to avoid an unconservative design for two-way shear. American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss 5HIHUWR)LJ(IRU¿QDOGLVWULEXWLRQDORQJWKLV column line. The middle strips, one on either side of the column strip. 94 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E1.4—Total panel moments 8.10.7 Msc to satisfy the DDM provisions at an interior 8.10.7.1 column is calculated by Eq. (8.10.7.2): 8.10.7.2 Msc = 0.07[(qDu + 0.5qLu)E2En2 - qDugE2gEng 2] ZKHUHWKHVDPHDVLQRXUH[DPSOHWKHgVLPSO\ indicates the next span. 8.10.7.3 Msc to satisfy the DDM provisions at an exterior column is calculated by Section 8.10.7.3: Msc = 0.3Mo 8.4.2.3.2, Msc is required to be transfe transferred through both 8.4.4.2.2 ZD/VK AH[XUHDQGWZRZD/VKHDULQWRWKHFROXPQ7KH alcu s are discuss two-way shear calculations discussed in Step 7. uired to transfe The amount of ste steel required transfer UfMsc into A LVGHWHUPLQHG WKHFROXPQYLDAH[XUHLVGHWHUPLQHG QODWH 7KHAH[XUDOUHLQIRUFHPHQWGHWHUPLQHG QODWH 7KHAH[XUDOUHLQIRUFHPHQWGHWHUPLQHG As in this is allowed to be us used to meet the require reinforcentorcestep. Therefore, att step ment from this step will be checked. At an interior column: Msc = 0.07[(qDu + 0.5qLu)E2En2 - qDugE2gEng 2] Msc = 0.07[(0.123 kip/ft2 + 0.5(0.160 kip/ft2)) 14 ft(16 ft)2 - 0.123 kip/ft2(14 ft)(16 ft)2] = 20.1 ft-kip At interior columns: erior columns: erior columns: erior columns: erior columns: Msc = 0.3(127 ft-kip) = 38.1 ft-kip At interior columns: erior columns: Msc = 0.3(127 ft-kip) = 38.1 ft-kip At interior columns: erior colum = 12.1 ft-kip U ng th hod descr Using the method described in Step 8, the amount of XUDO UHTXLUHG AH[XUDOVWHHOUHTXLUHGZLWKLQh of the columns: At exterior  $\hat{U}f = 0.6$  Msc = 38.1 ft-kip  $\hat{U}fMsc = (0.6)(38.1 \text{ ft-kip})$   $\hat{U}fMsc = 22.9$  ft-kip Using the method described in Step 8, the amount of XUDO UHTXLUHGZLWKLQh of the columns: At exterior  $\hat{U}f = 0.6$  Msc = 38.1 ft-kip  $\hat{U}fMsc = (0.6)(38.1 \text{ ft-kip})$ AH[XUDOVWHHOUHTXLUHGZLWKLQh of the column is: As = 0.94 in.2/45 in. or 0.25 in.2/ft Step 6: Required strength — Factored one-way shear 8.4.3 2QHZD/VKHDUVODEUDUHO/FRQWUROVRYHUWZRZD/ 8.4.3.1 shear in the design of a two-way slab, but it must 8.4.3.2 be checked. In this section, Vu is determined. In LWLVYHUL¿HGWKDWWKHVODEVKHDUVWUHQJWK[]Vn, is VXI¿FLHQWWRUHVLVWVu. Figure E1.5 shows one-way shears. The shear diagram is symmetric about the axis of the column at the center of the building (108 ft). Vu = 32 kip American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 95 Fig. E1.5—Shear diagram Step 7: Required strength — Factored two-way shear 8.3.1.4 Stirrups are not used as shear reinforcement in this example. 8.4.4, 8.4.4.1, 22.6.4, Determine the critical section for two-way shear 20.6.1.3.1 without shear reinforcement. Calculate bo at an interior column: d bo =  $2 \times (c1 + d) + 2 \times (c2 + d)$  bo =  $2 \times (24 \text{ in.} + 5.6)$ 5 in. in in.) + 2 × (24 in. + 5.6 in.) 118 4 in. in bo = 118.4 verage effective depth (Fig. E1.6)) where d is the average le assumes a es No. 5 bars w n deterand this example when mining d. ed to bee 0.75 in. per Table Tab e 20 Cover is assumed 20.6.1.3.1 Figure E1.7 shows two-way critical sections, bo, at an interior column. Fig. E1.7—Two-way shear critical section locations American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss Fig. g. E1.6—Av E1.6—Av E1.6—Av E1.6—Av E1.6—Average slab effective depth 96 8.4.4.2 8.4.4.2.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Determine vug due to direct slab shear stress. Calculate the direct shear stress at the interior column with full factored load on all spans: V vug = u bod 29.6 in.  $\times$  29.6 in.  $\times$  29.6 in. 283 kip (Vu = |14 ft  $\times$  18 ft - |)  $\times$  144 1000 ft 2 Vu = 70 kip vug = 8.4.4.2.1 8.4.4.2.2 Determine the slab shear stress due to moment. 70 kip = 0.106 ksi 118.4 in.  $\times$  5.6 in. Calculate the shear stress due to moments at an interior column:  $\gamma v = 0.4$  M sc = 20.1 ft-kip c AB = 14.8 in. J c = 97688 in.4 y v M sc c AB = 0.015 ksi Jc 8.4.4.2.3 Calculate vu by combining the two-way o-way direct shear stress and the stress and the stress at an interior column: 10 ksii + 0.015 ksi k = 0.121 ksi vu = 00.106 N e th Note that these calculati calculations are conservative. Msc as ume that some me live lo assumes load is not present to produce alan ments, but vug assumes that full live lo d and dead load are ppresent. load vu = vuug + American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 97 Step 8: Design
strength—Reinforcement required to resist factored moments 8.5.1 There are many methods available to determine [VDVVXPHGWREHIRUÀH]XUDOUHLQIRUFHPHQWUHTXLUHGDWDOOVHFWLRQV reinforced. Using the moments shown in Fig. E1.8 8.5.1.2 within the span in each direction. and E1.9 for the column strip and middle strip, respects.2.2 tively, to determine the reinforcement required at each 8.5.2.1 7RGHWHUPLQHWKHDPRXQWRIAH[XUDOVWHHOUHTXLUHG location. 8.4.2.3.5 this example solves the following quadratic equation: Reinforcement in an exterior panel (a) ( $\phi$ M n =  $\phi$  | As f y | d - | | ( 2// Column strip at the columns: Mu = 63 ft-kip (As fy ())  $\phi M n = \phi$  (bd c'  $\omega$  (d - 0.59 $\omega$ ))  $\omega = 0.0664$  .:  $\omega bdf$  c' As = fy As = (0.0664)(84 in.)(5.6 in.)(5000 psi) 60,000 psi As = 2.61 in.2 6HW [] Mn = \phi MuDQGVROYHIRUA Column sstrip at midspan: Mu = 27 ft-kip ving the quadratic equation gives: Solving 00.0278 0  $\omega$  = 0.0  $\therefore$   $\omega$ bbdf c 'As = fy As = (0.0278)(84 in.)(5.6 in.)(5000 psi) (0 (84 in.)(5.6 i in.2 Middle strip at midspan: As = 0.73 in.2 American Concrete Institute - Copyrighted @ Material - www.concrete.org Two-Way Slabss .09 in.2 As = 1.09 Using the following can be found: 98 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Interior Panels: Column strip at column lines: As = 2.61 in.2 Middle strip at column lines: As = 0.81 in.2 Column strip at midspan: As = 1.09 in.2 Middle strip at midspan: As = 0.73 in.2 Fig. E1.8—Column strip moment diagram 6WHS'HVLJQVWUHQJWK±2QHZD\VKHDUVWUHQJW 0.75 Vn = 2 f c bd  $\varphi \text{Vn} = (0.75)(2)$  () (1 kip) 5000 psi (168 in.)(5.6 in.) (1000 lb) (0.75 Vn) = 100 kip This is greater than the required strength of 32 kip from Step 6; therefore, one-way shear is okay. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 99 Step 10: Design strength – Two-way shear 8.5.3.1.2 Two-way design shear strength is calculated in ac- Interior column: 22.6 cordance with Section 22.6. Shear reinforcement is vn = the least of the three equations from Table assumed to not be required. 22.6.5.2 of ACI 318-14 4 5000 ksi = 0.283 ksi 1000 (4) 6 5000 ksi = 0.424 ksi vn =  $|2 + |fc'| = (\alpha d) vn = |2 + |fc'| = |2$ 2 + s bo // fc ' = 3.89 5000 ksi = 0.275 ksi \* 1000 \* controls  $\varphi vn = 0.75 \times 0.275$  ksi = 0.206 ksi This is greater than the required strength for interior columns is okay. The assumption that two-way shear reinforcement is ORWUHTXLUHG QRWUHTXLUHGDWWKHVHORFDWLRQVLVFRQ¿UPHG QLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 \$WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 WOHDVWDPLQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWDPLQIPXPÀH] tension is are provided at locations where 8.6.1 8.6.1.1 calculated in thee sslab. fy = 60,000 60, psi As,min = 0.0018 × Ag in As,min 14 ft × (12 in./ft.) in = 0.0018 × 7 in. × 1 2 As,min in = 2.12 in. 0LQLPXPAH[XUDOUHLQIRUFHPHQWFRQWUROVDWWKH middle strip of all panels. Step 12: Reinforcement detailing – General requirements 8.7.1 Concrete cover, development lengths, and splice 8.7.1.1 lengths are determined in these sections. 20.6.1 Concrete cover requirements are provided in Table 20.6.1.3.1. The slab is not exposed to weather or in contact with the ground. Assuming No. 5 bars for UHLQIRUFHPHQWWKHVSHFLiHGFRYHULVLQ American Concrete Institute – Copyrighted © Material - www.concrete.org Two-Way Slabss This minimum mum area of reinforcement is split evenly een the colu between column and middle strips; therefore 1.06 in.2 per strip. 100 8.7.1.2 25.4 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Development length is needed to determine splice length. Development length is calculated by Eq. (25.4.2.3(a)). Using normalweight concrete with No. 6 and smaller uncoated bars and the casting position with less than 12 in. of fresh concrete placed below the horizontal reLQIRUFHPHQWPRGL¿FDWLRQIDFWRUVIURP7DEOH are as follows: zs = 0.8 ze = 1.0 zt = 1.0 3 = 1.0 The bar spacing is larger than the distance from the center of the bottom bar to the concrete surface. cb = 0.75 in. + (0.625 in./2) = 0.94 in. db = 0.625 in. Ktr is assumed 0 as permitted by 25.4.2.3. () | f  $\psi$  t  $\psi$  e  $\psi$  s || 3 y d Ad = | 40  $\lambda$  f c' ( cb + K tr ) | b | |/ |/ | d \ b 8.7.1.3, 25.5 It is likely that sp splices will be required during uring construction. All Allowable le locations for ssplices are shown in ACI 318-14 18 Fig. 8.7.4.1.3. Ed = 21. 21.2 in.; use 22 in. /DSVSOLFHOHQJWKVDUHGHWHUPLQHGLQDFFRUGDQFHZLWK / VSO WKVDUHGH Table provided As does not exceed the le 225.5.2.1. The prov re uired As by a substantive amount. Therefore, class required B splices plice are required. in = 27.5 in. Est = 1.3 × 21.2 in. use 21. 21.2 in.; use 22 in. /DSVSOLFHOHQJWKVDUHGHWHUPLQHGLQDFFRUGDQFHZLWK / VSO WKVDUHGHWHUPLQHGLQDFFRUGDQFHZLWK / VSO WKVDUHGHWHUPLQHGLQDFFRUGDQFHZ Est = 28 in. use American Concrete Institute - Copyrighted @ Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS Step 13: Reinforcement detailing - Spacing requirements 8.7.2 Minimum and maximum spacing is determined in accordance with Section 25.2.1. Minimum spacing is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., than minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing is 1.33 in. With a No. 5 bar, this equates to a minimum space of the spa spacing is the lesser of 2h (2 × 7 in.) and 18 in., so 14 in. controls. All other sections, the critical spacing of 14 in. for all sections. Assuming No. 5 bars are used, the spacing for the different areas of the slab are as follows: All spans: Column strip at column line: 2.61 in.2/0.31 i/0.31in.2 = nine No. 5 bars over 7 ft - spacing 0.81 is 14 4 in p in. (maximum spacing controls over minimum dle strip) areaa in the middle Middle strip at midspan: 0.73 in.2/0.31in.2 = six No. 5 bars over 7 ft – spacing is 14 in. (maximum spacing controls over minimum area in the middle strip) 8.4.2.3.2 This is a amounts required to transfer the fraction of factored 14 in. spacing of No. 5 bars. This equates to 0.26 in.2/ VODEPRPHQWYLDÀH[XUHDUHVDWLV¿HGXVLQ]WKH ft. This
meets or exceeds the 0.13in.2/ft and 0.26 in.2/ design slab reinforcement. ft required from Step 5, therefore, Section 8.4.2.3.2 is VDWLV¿HG1RWHWKDWLIWKLVKDGQRWEHHQPHWDGGLWLRQDO steel would have been required to be placed within the HIIHFWLYHVODEZLGWKDVGH¿OHGLO6HFWLRO Step 14: Reinforcement detailing – Reinforcement termination 8.7.4 Reinforcement lengths and extensions are at least Use ACI 318-14, Fig. 8.7.4.1.3 to determine reinforce8.7.4.1 that required by Fig. 8.7.4.1.3 of ACI 318-14, Fig. 8.7.4.1.3 to determine reinforce8.7.4.1 that required by Fig. 8.7.4.1.3 to determine reinforce8.7.4.1 that required by Fig. 8.7.4.1.3 to determine reinforce8.7.4.1 that required by Fig. 8.7.4.1.3 of ACI 318-14, Fig. 8.7.4.1.3 to determine reinforce8.7.4.1 that required by Fig. 8.7.4.1.3 to determine reinforce8.7.4.1.1 that required by Fig. 8. ment in these panels shows the design lengths. 8.7.4.1.2 8.7.4.1.3 American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss umn strip at midsp Column midspan: .31 in.2 = six No. 5 bars over 7 ft - spacing 1.09 in.2/0.31 4 in. (maxim is 14 (maximum spacing controls over strength requirements at this location) 102 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 15: Reinforcement detailing - Structural integrity 8.7.4.2.1 satisfying ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.1 is met when reinforcement is detailed in accordance with Fig. 8.7.4.1.3 (ACI 318-14 detailing provisions. 8.7.4.2.2 Section 8.7.4.2.2 Section 8.7.4.2.2 Section 8.7.4.2.2 Section 8.7.4.2.3 14). Section 8.7.4.2.2 requires at least two of the column strip bottom bars pass through the column reinforcement cage. 6WHS60DEFROXPQMRLQWV 8.2.7 Joints are designed to satisfy Chapter 15 of ACI 15.2.1 318-14. 15.2.2 15.2.3 15.2.5 15.3.1 15.4.2 7KHVSHFL¿HGFRQFUHWHVWUHQJWKRIWKHVODEDQG columns are identical and therefore, 15.2.1 and 15.3 are met. Section 15.2.2 is met in Steps 7 and 10 of this example. Section 15.2.3 is met by satisfying the detailing Sections in 15.4. Section 15.2.5 states that interior columns are restrained because they are laterally supported on four sides by the slab. ection 15.4.2 ap Section applies to columns along the exterior of ding. the buil building. umi thatt No. 4 bar Assuming bars are used as column ties, Eq. E DUHVD D DQGE DUHVDWLV¿HGLIWKHVSDFLQ]RIWKH XPQ KHVODEFR FROXPQWLHVLQWKHVODEFR FROXPQWLHVQH FROXPQWLHVLQWKHVODEFR FROXPQWLHVLQWHY FROXPQWLHVQH FROXPQWLHVLQWHY FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWHY FROXPQWLHVQH FROXPQWLHV FROXPQWLQUH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLHVQH FROXPQWLH 60,000 psi 50 × 24 in. s = 20.0 in. %HFDXVHWKHVSDFLQJLVODUJHUWKDQWKHMRLQWGHSWKRI 7 in., only one tie is required within the slab-column MRLQWLQHDFKH[WHULRUDQGFRUQHUFROXPQ American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6—TWO-WAY SLABS 103 Step 17: Summary tables of required As As required, column strip, in.2 External bays Internal bays Column lines Midspan Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Strength 2.61 1.09 As required, middle strip, in.2 External bays Internal Bays 18: Summary sketches of required bars, middle strip American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS 105 Two-way Slab Example 2: Equivalent Frame Method (EFM) – Internal column line design moments, but any method for analyzing a statically indeterminate structure can be used. This example uses the Hardy column analogy to determine the structural stiffness for the members analyzed. Given: 80LIRUPORDGV Self-weight dead load is based on concrete density including reinforcement at 150 lb/ft3 Superimposed dead load D = 0.015 kip/ft2 /LYHORDGL = 0.100 kip/ft2 Material properties: fco SVL fy = 60,000 psi )LJ(2)LUVWARRUSODQ ACI 318-14 Discussion Step 1: Geometry 8.4.2 The moments in the slab are calculated using the 8.4.2.2 with Section 8.11 of ACI 318-14. Calculation American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss Thickness of slab t = 7 in. 106 8.11.1 8.11.1.2 8.11.1.2 8.11.1.2 8.11.1.2 8.11.2.2 8.11.2.2 8.11.2.2 8.11.2.3 8.11.2.4 8.11.2.5 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. E2.2 shows an isometric of the slab-beam strip and the attached torsional members of the equivalent frame model. A key element of the EFM is that, unlike a beam and column frame, in a slab and column frame some of the unbalanced moments can redistribute around the columns to simulate this effect on slab moments E\LOFRUSRUDWLOJWKHAH[LELOLW\RIWKHVODEWRUVLRODO member in the equivalent column stiffness. Fig. E2. E2.2—Equivalent frame strip Step 2: Analysis – Equivalent colu column stiffness determination iffness XPQVWLIIQHVVPRPHQWFRHI¿FLHQWDQGFDUU\RYHU PR FRHI¿FLHQWDQG DUU\RYHU factor for use in the distribution method. he moment ment distributio (Corley and Jirsa, 1970, "Equivalent Frame Analysis for Slab Design," ACI Journal Proceedings9 67, No. 11, Nov. pp. 875-884). American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 6-TWO-WAY SLABS To determine the equivalent column stiffness, Kec, the stiffness of the torsional member intersection. Kt is determined using an equation given in Commentary Section R8.11.5. The effects of cracking on Kt are neglected in ACI 318-14. This example uses the same concrete strength throughout the structure, so the modulus of elasticLW\LVDOVRFRQVLGHUHGHTXDO7KLVVLPSOL $\dot{c}$ HVWKH calculations. Kt =  $\sum 9 \text{ Ecc } c^2 = 24$  in. Interior column torsional members For the torsional members at the interior columns and the slab portion of the torsional member at the exterior columns, x = 7 in. y = 24 in. and  $(x)x^3y^2 = \sum |1 - 0.63 \times | \sqrt{y}/37$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(7 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(2 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(2 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(2 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(2 \text{ in.})^3 \times 24$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times |
\sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in.  $(C = |1 - 0.63 \times | \sqrt{y}/37)$  in. is monolithic onolithic with the column a torsional member at the mn but the wall rotatio equivalent column, rotation will bbe exerior columns, ns, exterior rsional greater than the col column rotation so the torsional x = 12 iin. ll will bee considered along g with stiffness of the wall y = 113 in. LWVÀH[XUDOVWLIQHVV7KHEDVHPHQWZDOOGLPHQVLRQ And the total C for the exterior column torsional y = 113 in., in the calculations here is the distance mbers is members d to the from the bottom of the slab being designed ( x ) x3 y top of the mat foundation.  $C = \sum |1 - 0.63 \times |\sqrt{y}|$  3 12 in.) (12 in.) 3 × 113 in. (+ 2240 in.4 C = |1 - 0.63 \times |\sqrt{113} in.) 3 C = 60,733 in.4 Kt = 5357 for each side of the column. Therefore, the total torsional member stiffness at an external column is  $Kt = 2 \times 5357 = 10,714$  American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss 8.11.3, 8.11.5 107 108 8.11.4 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) To determine the equivalent column stiffness, Kec, WKHVWLIIQHVVFRHI¿FLHQWVIRUWKHFROXPQVDERYH and below the slab are needed at each intersection. Because the slab thickness, column heights, and foundation thickness geometry is

uniform, Kctop and KcbotDUHFRQVLVWHQWDWHDFKLQWHULRUMRLQWLQWKLV design strip. Kc = kc × Ecc × I c Ac The following values are used in the calculations for Kctop and correspond to Fig. E2.3: ttop = 7 in. hbeam = 2.5 ft Ecol = 15.5 ft h = 18 ft Kctop and correspond to Fig. E2.3: ttop = 7 in. hbeam = 2.5 ft Ecol = 15.5 ft h = 18 ft Kctop and correspond to Fig. E2.3: ttop = 7 in. hbeam = 2.5 ft Ecol = 15.5 ft h = 18 ft Kctop and Kcbot are determined using the Hardy column analogy. (K. Wang, Intermediate Correspond to Fig. E2.3: ttop = 7 in. hbeam = 2.5 ft Ecol = 15.5 ft h = 18 ft Kctop and Kcbot are determined using the Hardy column analogy. Structural tbottom = 7 in. Analysis, McGraw-Hill, New York, 1983). Kctop is determined using the geometry from Fig. E2.3 and E2.4 were combined, (1 Mc) it provides a section cut through the basement + kctop = A c | slab being designed in this example. The bottom Aa I a |/ most slab in Fig. E2.3 and E2.4 were combined, (1 Mc) it provides a section cut through the basement + kctop = A c | slab being designed in this example. The bottom Aa I a |/ most slab in Fig. E2.3 and E2.4 were combined, (1 Mc) it provides a section cut through the basement + kctop = A c | slab being designed in this example. Aa = A col = 15.5 the upper-most beam and slab in Fig. E2.3 is the A 3 15.53 ¿UVWARRUDERYHWKHHQWUDQFHOREE\OHYHORIWKH Ia = a = = 310.3 structure. 12 12 M bot = 1.0 cbottom = 8.04 Please refer to the short discussion at the end of c = cbottom = 8.04 this example regarding an alternate method for (1 8.042) olumns, beams, determining the stiffness for the columns,  $1 \times |$  + = 4.91 kcctop = 18 and slabs. (15.5 310.3 |/ K ctop op = kctop × 1 col c Ac = 4.91 4.9 × 244 = 629 12 × 18(12) Fig. E2.3—Hardy column analogy for the columns above the slab being designed American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 8.11.4 109 The following values are used in the calculations for Kcbot and correspond to Fig. E2.4: ttop = 7 in.  $\mathcal{E}col = 9.42$  ft h = 10 ft tbottom = 3.5 ft (assumed mat foundation thickness) Kcbot is determined using the geometry from Fig. E2.4. (1 Mc) + kcbot = A c | Aa I a | Aa = A col = 9.42 Ia = M bot Aa 3 9.423 = 69.6 12 12 = 1.0 ctop = 5 c  $= \operatorname{ctop} = 5(152) + = 5.33 \text{ kcbot} = 11.46 \times | (9.4269.6) | \text{kcbot} \times I \operatorname{col} 5.33 \times 244 = = 107212 \times 11.46(12) \text{ Ac Fig. E2.4} + Hardy column analogy for the columns below the slab being designed American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss K cbot = 1108.11.4 THE REINFORCED CONCRETE DESIGN$ HANDBOOK—SP-17(14) To determine the equivalent column 1 K ec =  $11 + \Sigma$  Kc Kt 1 1 1 + 629 + 1072 10714 K ec =  $11 + \Sigma$  Kc Kt 1 1 1 + 629 + 1072 382 K ec = 312 K ec = 312Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS Step 3: Analysis – Slab stiffness is determined using the Hardy column analogy. 111 Slab panel (refer to Fig. E2.5) c1 = c2 = 2 ft = 24 in.  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ in}$ .  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ ft} = 218 \text{ in}$ .  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ in}$ .  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ in}$ .  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ in}$ .  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ ft} = 218 \text{ in}$ .  $\mathcal{E}s = 18 \text{ ft} = 218 \text{ ft} = 218$ c) 2) (I col =  $|1 - 2| = |1 - | = 0.7347 (14) (A2) bh3 168 \times 73 = 4802 12 12 c1 + c2 2 A n3 c/2 (A + 2 | n + 1 | Ia = 12 1/I col (22) Ia = 448 c1 c (I col) + 2 (I col) 2 2 2 2 Aa = 16 + (0.7347) 0.73 7) + (0.7347) 2 2 Aa = 17.53 17 Aa = An + (1 Mc) ks = As + (Aa Ia) (1 92) (A + 2 | n + 1 | Ia = 12 1/I col (22) Ia = 448 c1 c (I col) + 2 (I col) 2 2 2 2 Aa = 16 + (0.7347) 0.73 7) + (0.7347) 2 2 Aa = 17.53 17 Aa = An + (1 Mc) ks = As + (Aa Ia) (1 92) (A + 2 | n + 1 | Ia = 12 1/I col (22) Ia = 448 c1 c (I col) + 2 (I col) 2 2 2 2 Aa = 16 + (0.7347) 0.73 7) + (0.7347) 2 2 Aa = 17.53 17 Aa = An + (1 Mc) ks = As + (Aa Ia) (1 92) (A + 2 | n + 1 | Ia = 12 1/I col (22) Ia = 448 c1 c (I col) + 2 (I col) 2 2 2 2 Aa = 16 + (0.7347) 0.73 7) + (0.7347) 2 2 Aa = 17.53 17 Aa = An + (1 Mc) ks = As + (Aa Ia) (1 92) (A + 2 | n + 1 | Ia = 12 1/I col (22) Ia = 448 c1 c (I col) + 2 (I col) 2 2 2 2 Aa = 16 + (0.7347) 0.73 7) + (0.7347) 2 2 Aa = 17.53 17 Aa = An + (1 Mc) ks = As + (1$  $k s = 18 | + 1753448 | / (17.53 Ks = k s \times I s \times Ecs 4.281 \times 4802 = 9518 \times 12 As American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss 42 k s = 4.281 112 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) (1 Mc) As | - (Aa I a | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92) - 18 | - (17.53448 | / C.O.F. = ks (1 92$ 136 Am = 341.33 A2 = Am = 481.6 FEM = Am 48 481.6 = = 0.085 2 17.53 × 182 Aa A s American Concrete Institute – Copyrighted © Material – www.concrete.org Fig. E2.5—Section properties Step 4: Analysis – Moment distribution 8.11.1 This example uses moment distribution with pattern live load in accordance with Section 6.4.3 of \$&,/RDGLQJDOOVSDQVVLPXOWDQHRXVO\ O GRHVQRWQHFHVVDULO\SURGXFHWKHPD[LPXPAH[XUDO stresses in the slab. Therefore, in Section 6.4.3, OLYHORDGSDWWHUQVDUHGH¿QHGIRUXVHZLWKWZRZD\ slab systems. Fig. E2.6 shows examples of the different live load patterns considered in the code. When reduced to face of support, these results are comparable to the DDM analysis in Example 1. 113 PRPHQWGLVW 7KHPRPHQWGLVWULEXWLRQLQ)LJ(VKRZVWKH¿UVW four column lines when full live load is applied to all spans. The structure is symmetrical and repeats from column line 2.5 through column line 5.5. The moment distributions for the different live load patterns are not included here but have been incorporated into the example. The moment diagram (Fig. E2.8) and shear diagram (LJ VKRZWKH2QDOUHVXOWVFRQVLGHULQJWKHOLYH load patterns per Section 6.4.3.3 of ACI 318-14. The shear and moment diagram in Fig. E2.8 and E2.9 are determined using known moments at the end of the slab along with known loads on the slab. The numerical values shown on these diagrams are the maximums determined at each location from the live load patterns discussed in Section 6.4.3.3 of ACI 318-14. Note that the
numerical values shown on these diagrams are the moments and shears reduced to the moments at the face of the columns, not at the midline of the columns as shown in the moment distribution (Fig. E2.7). American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way SLABS 114 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. E2.6-Code live load patterns, example uses 6.4.3.3 (a) and (b). American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS 115 Fig. E2.8-Moment diagram maximum values at face of support and midspan American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss )LJ(<sup>2</sup>0RPHQWGLVWULEXWLRQH[DPSOHSDUWLDOGLVWULEXWLRQZLWKDOOVSDQVZLWKIXOOOLYHORDG H[ DQV IXOOOLYHORDG H[ DQV IXOOOLYHORDG H[ DQV IXOOOLYHORDG H[ DQV IXOOOLYHORDG H] DQV IXOOOLYHORDG H] DQV IXOOOLYHORDG H[ DQV IXOOOLYHORDG H] DQV IXOOOLYHONDG H] DQV IXOOOLYHONDG H] DQV IXOOOLYHONDG H] DQV IXOOOLYHONDG H] DQV the DDM in the Two-way Slab Example 1 of the EFM to be distributed to the column and middle this Handbook for this procedure. strips in accordance with the Direct Design Method (DDM) in Section 8.10 of ACI 318-14. Continuing on, the design solution follows a similar method as the direct design method. American Concrete Institute - Copyrightec © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 117 \$OWHUQDWLYHPHWKRGIRUGHWHUPLQLQJVWLIIQHVVFRHI¿FLHQWVIRUXVHLQWKHPRPHQWGLVWULEXWLRQFDOFXODWLRQV ACI 318-14, Section 6.3.1.1 states that: "Relative stiffnesses of members within structural systems shall be based on reasonable and consistent assumptions." This provision allows the designer to use any set of reasonable assumptions for determining the stiffnesses of the members in a two-way slab system in the EFM. In this example, the Hardy column analogy was used. An alternative method is suggested in the following discussion. Given that Table 6.6.3.1.1(a) of ACI 318-14 will be used to account for the effects of cracking and the approximations in Table 6.6.3.1.1, detailed calculations for kc to include the effects of rigid ends are small compared to the effects of rigid ends are small compared to the effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends are small compared to the effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends on the column stiffness are not warranted (The effects of rigid ends of rigid ends on the column sti = 0.25 I g = 0.25 24(24)3 = 19,353 in.4 12 168(12)3 = 8467 in.4 12 168(7)3 = 1200 in.4 12 Kc = kc Ecc I c Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kc = (4)(1)(19,353) 353 63 = 563 137.5 Upper column: /RZHUFROXPQ Kc = kc Ecc I w Ac Kcs A1 Ks = (4)(1)(1200) = 22 216 Slabs: Combining these values with Kt (Step 2 torsional members from this Example), and using the resulting stiffness values in WKHPRPHQWGLVWULEXWLRQDORQJZLWKWKH¿[HGHQGPRPHQWVZLWKRXWPRGL¿FDWLRQRIWKH¿[HGHQGPRPHQWVZLWKRXWPRGL¿FDWLRQRIWKH¿]), gives results that are approximately 5 percent different from the values shown in the example above. American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss Using these stiffness values to determine Kc to use for moment distribution calculations. Note that because all of the concity is assumed equal to 1 ksi in this example. crete strengths are the same, the modulus of elasticity 118 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Two-way slab is a prestressed solid slab roof without beams between supports. The strength of the slab is checked and two-way slab is a prestressed solid slab roof without beams between supports. Material properties were selected based on the code requirePHQWVRI&KDSWHUVDQGHQJLQHHULQJMXGJPHQWDQGNQRZQDYDLODEOHPDWHULDOV Given: /RDG Superimposed dead load D = 0.015 kip/ft2 Roof live load L = 0.040 kip/ft2 Roof live load L = 0.0 Discussion In the direction taken, there are six spans of 36 ft. The slab is supported by 24 in. square columns. The ACI 318-14 span-to-depth ratios between 40 and 50 are typically reasonable for two-way slab designs (Nawy G., 2006, Prestressed QWDO\$SSURDFK)LIWKHGLWLRQ, Pearson Prentice Hall, New Jersey, 945 pp). Calculation A t A = 432 in. 45 < ... t = 9.6 in. Use a thickness of 10 in. Use a thickness of the slab. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 119 6WHS/RDGDQGORDGSDWWHUQV 8.4.1.2 /RDGLQJDOOVSDQVVLPXOWDQHRXVO\GRHVQRWQHFHVVDULO\SURGXFHWKHPD[LPXPAH[XUDOVWUHVVHVLQWKH slab. Therefore, in Section 6.4.3 of ACI 318-14, OLYHORDGSDWWHUQVDUHGH¿QHGIRUXVHZLWKWZRZD\ slab systems. Fig. E3.2 shows examples of the different live load patterns considered in the code, using Section 6.4.3.2 of ACI 318-14. Two-Way Slabss Section 6.4.3.2 is applicable for this example because the roof live load is less than 75 percent of the combined dead loads. Fig. E3.2—Code live load is less than 75 percent of the combined dead loads. REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Concrete and steel material requirements 8.2.6.1 The mixture proportion must satisfy the durability requirements. The designer determines the durability classes. Please refer to Chapter 4 of this Handbook for an indepth discussion of the categories and classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWFRUUHODWHV with ACI 318. ACI encourages referencing ACI LQWRMREVSHFL¿FDWLRQWKDWFRUUHODWHV with ACI 318. ACI encourages referencing ACI 10 and providing the exposure classes, Chapter 19 (ACI 318-14) requirements are VDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW psi. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. The reinforcement must satisfy Chapter 20 of ACI 318-14. &KDSWHU\$&, UHTXLUHPHQWVDUHVDWLVcHGE\ specifying that the reinforcement shall be in accordance with ACI 301-10. This includes the PT type and In this example, unbonded, 1/2 in. single strand ten- strength, and reinforcing bar grade and any coatings dons are assumed. for the reinforcing bar. The designer determiness the grade of bar and if hould be coated by epoxy or the reinforcing bar should th. In this case, assume grade 60 galvanized, or both. ngs bar and no coatings. res strand d material to be 270 70 ksi, low The code requires TM A416a). 6a). The U.S. industry in ustry usurelaxation (ASTM MD PRQRVWUDQGWRLPSDUWDIRUFH DOO\VWUHVVHVRUMDFNVPRQRVWUDQGWRLPSDUWDIRUFH equal to 0.80fpu, wh which iss the maximum allowed lowed by the
Code. 7KH¿QDOVWUHVVDIWHUDOOORVVHVLVXVXDOO\EHWZHHQ HQJWK RI WRSHUFHQWRIWKHVSHFL¿HGWHQVLOHVWUHQJWKRI low relaxation strands. 7KHMDFNLQJIRUFHSHULQGLYLGXDOVWUDQGLV 7K MDF FHSHULQG 270 ksi × 0.8 × 0.153 in.2 = 33 kip T s is iimmediately diately red This reduced by seating and friction HODVWLFVK ORVVHVDQGHODVWLFVK RUWHQLQJRIWKHVODE/RQJWHUP losses will further reduce the force per strand. Refer to 20.3.2.6 of the Code. R20.3.2.6 An effective PT force design value of around 26.5 k fpu) per strand is common. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6-TWO-WAY SLABS Step 4: Analysis 6.6, 8.11 The analysis performed should be consistent with the overall assumptions about the role of the slab within the building system. Because the lateral force resisting system relies on the slab to transmit D[LDOIRUFHVD¿UVWRUGHUDQDO\VLVLVDGHTXDWH 121 Modeling assumptions: Assume a single moment of inertia for the entire length of the slab. Refer to Section 24.5.2.2 of the Code regarding cracked vs uncracked. Prestressed two-way slabs are required to be designed with service assumptions: Assume a single moment of inertia for the entire length of the slab. Refer to Section 24.5.2.2 of the Code regarding cracked vs uncracked. load limits of: ft < 6 f c' Although gravity moments are calculated independent of PT moments, the same model is used for both. The direct design method is used. In practice, computer analysis software is typical. Analysis approach: 7RDQDO\]HWKHAH[XUDOHIIHFWVRISRVWWHQVLRQLQJRQ the concrete slab under service loads, the tendon drape is assumed to be parabolic with a discontinuity at the support centerline as shown below, which imparts a uniform uplift over each span of a prismatic member is calculated as: wp = 8F 8Fa A2 Two-Way Slabss where w ere F is effective ti PT fforce and a is tendon drape tive ((average erag of the two high points minus the low point). In this his eexamplee the PT fforce is assumed constant for pan but the uplif all spans, uplift force varies due to different UDSHV)LJX WHQGRQGUDSHV)LJXUH(VKRZVWKHWHQGRQSUROH umed in this example. assumed )LJ(<sup>2</sup>7HQGRQSUROH (5 in. - 1 in.) = 6 in. 2 (9 in. - 1 in.) = 6 in. 2 (9 in. - 1 in.) = 6 in. 2 (9 in. - 1 in.) = 8 in. a2 = 2 a1 = American Concrete Institute - Copyrighted © Material - www.concrete.org 122 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Analysis – Slab stress limits 8.3.4.1 The code requires a Class U slab assumption; that is, a slab under full service condition is the same as for the nominal condition. To verify that the concrete tensile stresses are less than 6 5000 psi, the same as for the same as for the nominal condition. net service moments and tensile stresses at the face of supports are needed. This example assumes two parameters: 8.6.2.1 For 5000 psi = 424 psi The basic equation for concrete tensile stress is: ft = M/S - F/A, where M is the net service moment, bt 2 (12 in.)(10 in.) 2 = 200 in.3 6 6 (section modulus), S = and A = bt= 12 in.\*10 in. =120 in.2 (gross slab area (a) the PT force provides a F/A slab compressive per foot). stress of at least 7 percent of the slab weight, or 94 psf. Solve for F: At the exterior support, the drape is a = 6 in. The equation for 94 kip/ft k p/ft = 0.094 8Fa 8F (6 in.) = 2 ((36 ft) 2 (12 in./ft) 2 1 :. F = 30.5 kip/ft Location from le left to rig right along the span Service loads First mid midspan Sixth midspan Sixth midspan Sixth midspan Sixth midspan Sixth midspan Second midspan Second midspan Sixth midspan Sixth midspan Sixth midspan Sixth midspan Second midspan Second midspan Sixth midspan Second midspan Sixth mids 86 54 54 54 56 Using the above information and performing an equivalent frame analysis (refer to Two-way Slab Example 2), the following maximum at the face of the ¿UVWLQWHULRUVXSSRUWDQGLVIWNLSIW Positive moment is maximum at midspan of the UVWDQGVL[WKVSDQVDQGLVIWNLSIW Use these moments to determine the stresses at service load: \$WWKHIDFHRIWKHUVWLQWHULRUVXSSRUW P M + A S ( 30.5 kip 8.1 ft-kip 12 in.) ( 1000 lb ) + × ft = | - | ( 120 in.2 ft ) | ( 1 kip |) 200 in.3 ft = - ft = SVL ≤ SVL : 2. For the positive moment at midspan, it is usually desirable to avoid reinforcement required by Section 8.6.2.3. To avoid this, the tensile stresses in the slab should not exceed 2 5000 psi = 141 psi . P M ft =  $- + A S (30.5 \text{ kip } 12 \text{ in.}) (1000 \text{ lb}) + \times \text{ft} = |-| \times (120 \text{ in.} 2 \text{ ft} / | (1 \text{ kip } | / 200 \text{ in.} 3 \text{ ft} = \text{SVL} \leq \text{SVL} \therefore 2$ . American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 7KHH[DPSOHDVVXPHVWKHGHÀHFWLRQVLQHDFKGLUHFWLRQVDUH FKHFNHGLQWKHORQJGLUHFWLRQVDUH FKHFNHGLQWKHVODE'HÀHFWLRQVDUH FKHFNHGLQWKHVODE'HÀ the uniform live load only is applied LQWKLVGHAHFWLRQFDOFXODWLRQ7KHDQDO\VLVLVDSSUR[Lmate due to several simplifying assumptions, but it provides a reasonable result.  $\Delta$  max = 0.0065wA 4 0.0065(0.040/12)(432) 4 = = 0.19 in. EI 4030(1000) Assuming twice this to account for the two-way action of the slab, "max = 2 × 0.19 in. = 0.38 in. Expressed as a ratio, Eå E/2400. This is much less than the limit of EVRGHAHFWLRQOLPLWV DUHVDWLV¿HG Step 7: Analysis – Balanced, secondary, factored, and design moments Balanced moments are determined using ng the PT The moment curve below the table is the full design Uniform uplift load from thee table of service loads moment curve cu with critical section moments shown ng an equ n the curve. Th g moments shown are at the in Step 5 and performing equivalent frame on The design wo-way face of supports and att the point of maximum positive ents aare determined using balanced moment ment in the he span. Secondary moments rim oments. moments and primary moments. ent are determined usin Factored moments using the om ions required by code and factored load combinations ent frame analysis (refer to xam 2). Two-way Slab Example re determined i d by subtracting th Design moments are the secondary moments from the factored moments. The following table gives the balanced, secondary, factored, and design moments at the face of sup supports across the slab is symmetrical about the third column. Location from left to right along the span (sym about Col 4) Design moment, Mu = Mug - Ms, ft-kip/ft 9.8 23.6 20.7 19.6 19.8 20.1 American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss 6WHS\$QDO\VLV±'HAHFWLRQV 8.3.2 The two-way slab chapter refers the user to Section \$&, WKDWVWDWHV3'HAHFWLRQV 8.3.2 The two-way slab stiffnes approximations to calculate immediate DQGWLPHGHSHQGHQWORQJWHUP GHAHFWLRQV Section 24.2.2 provides maximum allowed span-toGHAHFWLRQVDWLRV6HFWLRQVIRU&ODVV8VODEV Commentary Section R24.2.3.3 of the Code alerts WKHGHVLJQHUWKDWFDOFXODWLRQVIRUGHAHFWLRQVRI two-way slabs is challenging. This example GHWHUPLQHVWKHGHAHFWLRQDQG doubles it for the effect from the other direction. This is not an accurate assumption, but it should give conservative and reasonable results. Note that H[FHVVLYHGHÅHFWLRQVDUHJHQHUDOO\QRWH[SHULHQFHG in PT slabs. 123 124 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E3.4—Moment diagram (negative moment at face of column). Step 8: Required strength – Calculatee requir required As 8.7.5.2 &KHFNÀH[XUHVWUHQJWKFRQVLGHULQJ37WHQGRQV HQJWK J ovide the nec ry If the PT tendonss al alone provide necessary • n,, then the code ppermits mits GHVLJQVWUHQJWK+[]M o be detailed tailed with short reinforcement to shorterr cutcut-off ons alone do not provide rovid the lengths. If the PT tendons design strength, the then the reinforcement is required da lengths. ngths to conform to standard The reinforcing bar and tendons are usually at the same position near the supplier. The value of fse (effective stress in the strand) varies along the tendon length due to friction losses (ACI 423.10R-15), but for design purposes, fse is usually taken as the average value. The dep depth off the equiv equivalent stress block, a, is calcudb by lated a = Aps f ps 0.85 f c'(12 in./ft) 0.8 n./ft) were Aps is the he tendon area per foot of slab. where tion 8.5.2.1 1 refers to Section 22.3 of the Code for Section XODWLRQRI[] n. Section 22.3 refers to Section WKHFDOFXODWLRQRI M 22.2 for calculation of Mn. Section 22.2.4 refers to Section 20.3.2.4 to calculate fps. The span-to-depth ratio is 432/10 = 1/43, so the below equation applies: f c' f ps = f se + 10,000 + 300p p The tendon supplier usually calculates fse, and 175,000 psi is a common value. The force per strand is therefore 175,000 psi x 0.153 in.2 = 26,800 lbs. The required effective force per foot of slab is 30.5 kip/ft, so the spacing of tendons is 26.8 kip/30.5 kip/ft. The value of dp is Aps/(b × dp) = 0.175/108 in.2 = 0.00162. f ps = 175,000 + 10,000 + 5000 = 195,000 psi 0.486 This value has upper limits of fse + 30,000 (= 205,000 psi) and fpy (= 0.9 fpu, or 242,900 psi from commentary), so the design value of fps is 195,000 psi. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 22.2.4.1 Note that the effective depth is 9 in. at critical locaWLRQVH[FHSWDWWKHH[WHULRUMRLQW7KHUHWKH&RGH permits a minimum d of 0.8h, or 8 in. 125 The compression block depth is therefore: a = Aps f ps 0.85 f c'(12 in./ft) =  $0.175 \times 195,000 \times (9 - 0.34) = 266,000$  in.-lb/ft = 22 ft-kip/ft. Location from left to right along the span Face of fourth support First midspan Face of second midspan Face of third support Third midspan Face of fourth support for at the face of fourth support for at the face of third support for at the face of fourth support for at the face of second midspan Face of second midspan Face of second midspan Face of fourth support for at the face of four the second support. The reinforcement required to resist the moments at the face of the second support are required to satisfy the detailing requirements of Section 7.7.3 of the
Code while minimum reinforcing bar lengths can be used at all other locations. Step 9: Required to resist the moments at the face of the second support are required to satisfy the detailing requirements of Section 7.7.3 of the Code while minimum reinforcement 8.6.2.3 7KHPLQLPXPDUHDRIÀH[XUDOUHLQIRUFLQJEDUSHU As, min = 0.00075 × Acf = 0.00075 × 10 in. × 12 in. = foot is a function of the slab's cross sectional area of the slab-beam strips off the two orthogonal equivacting aat a column of a two-way lent frames intersecting slab oment strength of combined pr ress Step 10: Required strength – Des Design moment prestressing steell and bon bonded reinforcement upp g the minimum area a ea of reinSe Set the section's concrete compressive strength equal 8.5.2 Determine if supplying XI¿ WWRDFKLHYHDGH JQVW to steel teel tensile strength, trength, and rearrange for compression IRUFLQJEDULVVXI¿FLHQWWRDFKLHYHDGHVLJQVWUHQJWK d strength. bblock ck ddepth a: that exceeds the re required Aps f ps + As f y 0.85 f c' ((12)  $0.175 \times 195,000 + 0.09 \times 60,000$  a = 0.85 × 5000 × 12 a = 0.78 in. For 3/4 in. cover: a) (Mn =  $\varphi$  [ Aps f ps + As f y ]] d -  $|| 2 | = 0.9 [0.175 \times 195,000 + 0.09 \times 60,000]$  (9 - 0.78) = 292,000 in.-lb = 24.3 ft-kip Comparing this value with the required moment strength Mu indicates that the minimum reinforcement for the slab to resist the factored loads at all locations. American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss a = 126 CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Analysis - Distribute moments to column and middle strips 8.7.2.3 Tests and research have shown that for uniformly 7KHGHAHFWLRQEHKDYLRUDQGFDSDFLW\GLIIHUHQFHVDUH loaded structures variations in tendon distribution not dependent upon the distribution of tendons. It can GRHVQRWDOWHUWKHGHAHFWLRQEHKDYLRURUWKHFDSDF- be extrapolated that distribution of moments to the ity for the same total prestressing steel percentcolumn and middle strips is unnecessary. DJH6HFWLRQSURYLGHVVSHFL¿FJXLGDQFH regarding tendon distribution that allows the use of banded tendon distribution distribution. in one direction. Step 12: Required strength – Factored one-way shear 8.4.3 2QHZD/VKHDUUDUHO\FRQWUROVWKLFNQHVVGHVLJQ Fig. E3.5 shows one-way shear reduced to the face of support. Check 8.4.3.2 section, one-way shear load on the structure is maximum factored shear. determined. Vu = 58 kip/14 ft = 4.1 kip/ft Fig. E3.5—Shear diagram. Step 13: Required strength – Factored two-way shear 8.3.1.4 No stirrups are to be used as shear reinforcement. 8.4.4 8.4.4.1 22.6.4 Determine the location and length of the critical section for two-way shear assuming that shear reinforcement is not required. Figure E3.6 shows this examples critical sections. Note that only the exterior columns: d = 10 in. -1 in. = 9 in. d = 10 in. -1 in. = 9ed © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS Fig. E3.6—Two-way shear critical section locations. 8.4.4.2, Determine factored slab shear stress due to gravity 8.4.4.2.1 loads vug. 127 Direct slab shear stress on slab critical section × (24 in. + 9 in.) + 2 × (24 in. + 9 in.) bo = 132 in. American Concrete Institute – Co exterior columns: V vug = u bo d 33 in. × 28.5 in.) 232 kip ( $f \times 18$  ft f - Vu = | 144 ft  $| / \times (144\ 1000$  ft 2 Vu = 57 ksi vuug = 57 ksi vuug = 57 ksi vuug = 115 kip vug = 115 kip = 0.097 ksi 132 in. × 9 in. American Concrete Institute - Copyrighted © Material www.concrete.org Two-Way Slabss D ect sslab shear Direct ear stress on slab critical section at the i columns: interior V vug = u bo d 128 8.4.4.2.1, 8.4.4.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Determine the slab shear stress due to factored slab moment resisted by column. Shear stress on slab due to moments at exterior columns:  $\gamma v = 0.4 \text{ M} \text{ sc} = 10 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 9.02 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 24 - 21 \text{ ft-kip/ft} \times 14 \text{ ft c} \text{ AB} = 16.5 \text{ in. J c} = 76383 \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ M} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ m} \text{ sc} = 10.079 \text{ ks} \text{ in. 4 } \gamma v \text{ m} \text{ sc} = 10.079 \text{ ks} \text{ sc} = 10.079 \text{ sc} =$ to the way direct shear and the moment y of shea column via eccentricity shear.  $vu = vuug + \gamma v M$  sc c AB Jc Exterior erior columns:: Corner vu = 0.0 0.097 ksi + 0.015 ks ksi = 0.112 ksi 6WHS'HVLJQVWUHQJWK±2QHZD\VKHDU ZD HDU 8.5.3.1.1, Nominal one-way shear strength is calculated in 22.5 accordance with Section 22.5.  $\varphi = 0.75$  Vn = 2 f c'b 'bbd  $\varphi$ Vn = 0.75 × 2 × 5000 psi × 12 in. × 9 in. × 1 kip/ft This is greater than the required strength of 4.1 kip/ft from Step 12, therefore, one-way shear is okay. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS Step 15: Design strength - Two-way shear 8.5.3.1.2, Nominal two-way shear strength is calculated in 22.6 accordance with Section 22.6. Assume shear reinforcement is not required. 129 Exterior column: vn = the minimum of the following three equations from Table 22.6.5.2 (ACI 318-14) 4 5000 ksi = 0.283 ksi Controls 1000 (4) 6 5000 ksi = 0.424 ksi vn =  $2 + fc' = \langle (\beta) | / 1000$  vn =  $4 fc' = (\alpha \times d)$  vn = 2 + s bo  $| / \langle fc' = 4.735000$  ksi = 0.334 ksi 1000  $\varphi$ vn =  $0.75 \times 0.283$  ksi = 0.212 ksi This is greater than the required strength for interior columns of 0.149 ksi from Step 6, therefore, two-way shear at interior columns is okay. Interior column: vn = the minimum of the following three equations from Table 22.6.5.2 (ACI 318-14) 4 5000 ksi = 0.283 ksi Controls 1000 (4) 6 5000 ksi = 0.424 ksi f c' = vn =  $|2 + | \setminus (\beta) / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to
0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s \to 0| / 1000 vn = 4$  f c' = ( $\alpha \times d \setminus vn = |2 + s$ QRWUHTXLUHGDWWKHVHORFDWLRQVLVFRQ¿UPHG Step 16: Reinforcement detailing – General requirements 8.7.1 Concrete cover is determined using Table 20.6.1.3.2 (ACI 318-14). The bottom of this slab is not exposed to weather or in contact with the ground. The specicHGFRYHULVLQ American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss This iis greater ter th than th the required strength for exterior columnss of 0.112 kksi from Step 6, therefore, two-way hear at exteri shear exterior columns is okay. 130 8.7.1.2, 25.4 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Development length is used for splice length determination assuming No. 5 bars. Use Eq. (25.4.2.3(a)) of ACI 318-14 to determine the development length. horizontal reinIRUFHPHQWPRGL¿FDWLRQIDFWRUVIURP7DEOH (ACI 318-14) are: zs = 0.8 ze = 1.0 zt = 1.0 3 Spacing of the bars is larger than the distance from the center of the bottom bar to the nearest concrete surface. cb = 0.75 in. + 0.625 in. = 0.94 in. 2 db = 0.625 in. Ktr is assumed 0 as allowed by 25.4.2.3. () |  $\psi$  t  $\psi$  e  $\psi$  s || 3 fy Ad = | d 440 λ f c' ( cb + K tr ) | b | | / | / | d \ b 8.7.1.3 25.5 21.2 in. Ed = 21. It is likely that splices are required during /DSVSOLFHOHQJWKVDUHGH tion. Allowable loc locations are shown Table 318-14). The provided As is ns for splices are hown in T le 225.5.2.1 (ACI 31) ( ACI 318-14, Fig. 8.7.4.1.3. 7 an two ttimes larger than the required As. not more than ore, class B splices are required. Therefore, Est = 1.3 × 21.2 in. = 27.5 in. use Est = 28 in. Step 17: Reinforcement detailing – Spacing requirements 8.7.2 Minimum and maximum spacing requirements 8.7.2 in. = 27.5 in. use Est = 28 in. Step 17: Reinforcement detailing – Spacing requirements 8.7.2 Minimum and maximum spacing requirements 8.7.2 in. = 27.5 in. use Est = 28 in. Step 17: Reinforcement detailing – Spacing requirements 8.7.2 Minimum and maximum spacing for requirements 8.7.2 Minimum and maximum spacing for requirements 8.7.2 Minimum and maximum spacing requirements 8.7.2 Minimum and maximum space for requirements 8.7.2 25.2.1 reinforcement are also reviewed. 8.7.2.3 Minimum spacing is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., db, and (4/3)dagg. Assuming that the maximum nominal aggregate size is 1 in., db, and (4/3)dagg. 2 in. Maximum spacing is controlled by Section 8.7.2.3. Assuming that this direction is banded, the maximum spacing requirements of Section 8.7.2.3 are not applicable to this direction. The tendons in the orthogonal direction are limited to a maximum spacing of 5 ft. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 6—TWO-WAY SLABS Step 19: Reinforcement detailing – Structural integrity 8.7.5.6 Structural integrity is met using detailing. 6WHS60DEFROXPQMRLQWV Joints are designed to satisfy Chapter 15 of ACI 8.2.7 15.2.1 318-14. 15.2.2 15.2.3 15.2.5 15.3.1 15.4.2 Bonded nonprestressed reinforcement is required for AH[XUHLQRQHORFDWLRQDQG6HFWLRQFRQWUROV termination of the minimum bonded reinforcement in that location. When the termination location is within the minimum lengths of Section 8.7.5.2, it is approximately 6 in. beyond the face of support. This termination location is within the minimum bonded reinforcement in that location. When the termination location is within the minimum lengths of Section 8.7.5.2, it is approximately 6 in. locations indicated in Section 8.7.5.5 satisfy the termination location required by Section 8.7.5.2 for the locations requiring bonded QRQSUHVWUHVVHGUHLQIRUFHPHQWIRUAH[XUDOVWUHQ]WK Requirement Section 8.7.4.2.2 is met when at least two of the PT tendons pass through the column reinforcement cage. In this direction, banding of the post-tensioning tendons makes this a simple requirement to satisfy. The concrete strength of the slab and columns are identical and therefore, Section 15 15.2.2 is met in Steps 6 and 9 of 1 this exa example. uire Section 15 Requirement 15.2.3 is met by satisfying the pprovisions visio in Section ection 15.4.15 R uire ti 15.2.5 15 Requirement Section states that all of the interior colu columns aree restrain restrained because they are laterally su port on four our sides by the slab. that No. 4 bars are used as column ties, Eqs. D DQGE DUHVDWLV¿HGLIWKHVSDFLOJ RIWKHFROXPQWLHVLQWKHVODEFROXPQMRLQWPHHWVWKH following: (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(60,000 psi) = 20 in. (50)(24 in.) = 18.9 in. (15.4.2(a)) s = (0.4 in.2)(15.4.2(a)) s = (0.4 in.2)(15.4.2(a)VSDFLQJLVODUJHUWKDQWKHMRLQWGHSWKRILQRQHWLH LVUHTXLUHGZLWKLQWKHVODEFROXPQMRLQWLQHDFKH[WHULRU and corner column. American Concrete Institute - Copyrighted © Material - www.concrete.org Two-Way Slabss Step 18: Reinforcement termination 8.7.5.2, Reinforcement termination i controlled by Section 8.7.5.5 8.7.5.2. 131 132 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Note: Design post-tensioning is 30.5 kip/ft)/(12 in.) 2QHVWUDQGHYHU\LQZLWKWKLVSURcOH0LQLPXP bonded reinforcement required is 0.09 in.2/ft No.4 bars: 0.2 in.2/(x in.) = 0.09 in.2 LQl )LJXUH(VKRZVWKH¿QDOFRQ¿JXUDWLRQRIWKH No. 4 bar every 2 ft slab. Fig. E3.7—Reinforcement detailing. Note: a minimum of two unbonded PT strands must be placed in both directions through the column cage. American Concrete Institute – Copyrighted © Material – www.concrete.org 7.1—Introduction Structural beams resist gravity and lateral loads, and any combination thereof, and transfer these loads to girders, columns, or walls. They can be nonprestressed, and post-tensioned (PT) beams. The Code allows for either bonded or unbonded tendons in a PT beam. This chapter does not cover precast, composite, or deep beams. Deep beams are also addressed in ACI 318-14 for strength and serviceability. Beams are assumed to be approximately horizontal, with rectangular or teeVKDSHGDVWHPDQGDADQJH FURVVVHFWLRQV7KHADQJHZLGWK of tee-shaped beams are geometrically limited by Section DQGUHVSHFWLYHO\RI\$&,DQGWKHADQJH LVDVVXPHGWRFRQWULEXWHWRWKHEHDP¶VAH[XUDODQGWRUVLRQDO strength. essed, that are Beams, either nonprestressed or prestressed, HFRQVLGH PRQROLWKLFZLWKWKHARRUIUDPLQJFDQEHFRQVLGHUHGODWHUDOO\ ROLWKLF
EUDFHG)RUEHDPVWKDWDUHQRWPRQROLWKLFZLWKWKHARRU\$&, 318-14 provides guidance on the spac spacing of lateral bracing. 7.2—Service limits in etermines the be m's 7.2.1 Beam depth—The engineer determines beam's h, and other material charharconcrete strength, steel strength, n pperformance rmance criteri acteristics to achieve the design criteria for strength and service life. \$IWHU GH¿QLQJ WKH PDWHULDO SURSHUWLHV DQG WKH EHDP¶V design loads, the engineer chooses the beam's dimensions. s These are either provided by architectural constraints, attained from experience, or reached by assuming a depth and ZLGWKDQGWKHQDGMXVWLQJDVUHTXLUHGWKURXJKWULDODQGHUURU Beam depth is addressed in Table 9.3.1.1 of ACI 318-14, which applies if a beam is nonprestressed, not supporting concentrated loads along its span, and not supporting or DWWDFKHGWRSDUWLWLRQVWKDWPD\EHGDPDJHGE\GHAHFWLRQV For prestressed or posttensioned beams, the code does not provide a minimum depth-to-span ratio, but for a superimposed live load in the range of 60 to 80 psf a usual spanto-depth ratio is in the range of 20 to 30. Table 9.3 of The Post-Tensioning Manual (Post-Tensioning Manual) Institute (PTI) 2006) lists span-to-depth ratios for different members that have been found from experience to provide satisfactory structural performance. The slab thickness is considered as part of the overall beam and slab are monolithic or if the slab thickness is considered as part of the slab. 'HÀHFWLRQV—For nonprestressed beams that have depths less than the ACI 318-14, Table 9.3.1.1 minimum, or those that resist a heavy load—usually one- or two-way VODEV VXEMHFWHG WR DERYH SVI<sup>2</sup>DQG IRU 37 EHDPV WKH GHVLJQHU PXVW FDOFXODWH GHÀHFWLRQV 'HÀHFWLRQV IRU FDOFX- ODWLRQ IURP FODVVLFDO DQDO\VLV PHWKRGV RU ¿QLWH HOHPHQW method, can be found in the supplement to this Handbook, ACI Reinforced Concrete Design Handbook Design Aid - \$QDO\VLV7DEOHV\$&,63'\$ 7KHFDOFXODWHGGHÀHFWLRQV should not exceed the limits in Table 24.2.2 of ACI 318-14, DIWHU FRQVLGHUDWLRQ RI WLPHGHSHQGHQW cracked, C, and transition between XQFUDFNHG DQG FUDFNHG 7 EDVHG RQ H[WUHPH ¿EHU WHQVLRQ stress. Class U in Table 24.5.2.1, ACI 318-14 limits the maximum beam tension stress to less than the cracking stress of concrete, 7.5 f c' 'HAHFWLRQFDOFXODWLRQVIRU&ODVV U beams, therefore, can use the gross moment of inertia. 7.2.3 Reinforcement strain limits and concrete service stress 3 1 Strain lim 7.2.3.1 limits—For nonprestressed beams with ha ten percent of gross sectional han design axia axiall force less than 0 cgA g g), the m minimum strain of the tension reinstrength (< 0.1f forc ent iis 0.004. 4. This lim forcement limit is to ensure yielding behavior in cas case of ov overload. restr eams hav Nonprestressed beams, the perm tensioned permissible concrete service stresses n Section 24.5.3 of ACI 318-14. are addressed in tensioned (PT) beams, the analysis of concrete For posttensioned UDO WHQVLRQ VWUHVVHV LV D FULWLFDO SDUW RI WKH GHVLJQ AH[XUDO ,Q 6HFWLR RI \$&, EHDPV DUH FODVVL¿HG ,Q 6HFWLR RI \$&, EHDPV DUH FODVVL¿HG ,Q 6HFWLR RI \$&, EHDPV DUH FODVVL¿HG ,Q 6HFWLR RI \$&, EHDPV DUH FODVVL AB , 20 (uncracked), or T (transition). load that results in moment of at least 1.2 times the section cracking moment. 7.2.3.2 Concrete stresses in PT beams—Before the beam ÀH[XUDOVWUHVVHVFDQEHFDOFXODWHGWKHWHQGRQSUR¿OHDQGWKHWHQGRQSUR¿OHDQGWKHWHQGRQIRUFHDUHGLUHFWO\ related to the beam forces and moments created by the posttensioning. A common approach to calculate PT beam moments is to use the "load balancing" concept, where the SUR¿OHLVXVXDOO\WKHPD[LPXPSUDFWLFDOFRQVLGHULQ]FRYHU UHTXLUHPHQWV WKH WHQGRQ SUR¿OH LV SDUDEROLF WKH SDUDER at middepth at the exterior (refer to Fig. 7.2.3.2). The load balancing concept assumes the tendon exerts a uniform upward "load" in the parabolic length, and a point load down at the support. These loads are then combined with the gravity loads, and the analysis is performed with a net service load. To conform to Code stress limits, the designer can use an iterative approach or a direct approach. In the iterative American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 134 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 7.2.3.2—Load balancing concept. DSSURDFKWKHWHQGRQSUR¿OHLVGH¿QHGDQGWKHWHQGRQSUR LV DVVXPHG 7KH DQDO\VLV LV H[HFXWHG AH[XUDO VWUHVVHV DUH FDOFXODWHGDQGWKHGHVL]QHUWKHQDGMXVWVWKHSUR¿OHIRUFHRU both, depending on results and design constraints. In the direct approach, the designer determines the highest tensile stress, then rearranges equations so that the analysis calculates the tendor force needed to achieve the stress limit. 7.3—Analysis /RDGVDQGORDGFRPELQDWLRQVDUHREWDLQHGIURP&KDSWHU of ACI 318-14. Beams can be analyzed by any method satisfying equilibrium and geometric compatibility, provided GHVLJQVWUHQJWKDQGVHUYLFHDELOLW\UHTXLUHPHQWVDUHVDWLV¿HG Chapter 6 of ACI 318-14. allows for nonprestressed beams satisfying the conditions of Section 6.5.1 to use a simplicHGDSSUR[LPDWHPHWKRGWRFDOFXODWHWKHGHVLJQPRPHQWDQG shear forces in beams at the face of support and at midspan. ulated by th Redistribution of design moments calculated this method is not permitted. AHFW %HDP PRPHQWV VKHDU DQG GHAHFWLRQV DORQJ WKH EHDPV¶ d fr assic elastic sstruclength are commonly calculated from classic intural analysis. The supplement to this Handbook, ACI R Reinbo Design esign Aid – Ana ysis forced Concrete Design Handbook, ACI R Reinbo Design esign Aid – Ana ysis forced Concrete Design Handbook Analysis ab hat provide moment moment Tables (ACI SP-17DA) has tables that rts and midspan for va us and shear upports various oment of in boundary and loading conditions. The moment inertia and modulus of elasticity values used in ACI 318. Redistribution of elastic moments calculated by a classical method is permissible. 7KHHQJLQHHUFDQDOVRXVH¿QLWHHOHPHQWVRIWZDUHWRFDOFXODWHPRPHQWVVKHDUDQGGHÀHFWLRQVDORQJWKHEHDPV¶OHQJWK The moment of inertia and modulus of elasticity values used LQWKH¿QLWHHOHPHQWPRGHOVKRXOGEHFDUHIXOO\FRQVLGHUHGWR REWDLQUHDOLVWLFGHÅHFWLRQVDQGGHVLJQIRUFHV5HGLVWULEXWLRQ RI HODVWLF PRPHQWV FDOFXODWHG E\ DQ HODVWLF ¿QLWH HOHPHQW method is permissible. 7.4—Design strength Beams resist self-weight and applied loads, which can UHVXOW LQ EHDP ÅH[XUH VKHDU WRUVLRQ DQG D[L] along a beam's length, the design strength is at least equal to the factored load effects; mathematically H[SUHVVHGDV]Sn•Su. 7.4.1 Flexure—Reinforced concrete beam design for AH[XUH W\SLFDOO\ LQYROYHV D VHFWLRQDO GHVLJQ WKDW VDWLV¿HV the conditions of static equilibrium and strain compatibility across the depth of the section. Following are the assumptions for strength design method OLVWHGLQ6HFWLRQRI\$&, ¿YHRIWKHVHDUHKLJKlighted as follows: Fig. 7.4.1—Assumed strain and stress at the nominal condition. (a) Strains in reinforcement and concrete are directly proportional to the distance from neutral axis (plane sections remain plane after loading). (b) Maximum concrete compressive strain in the extreme FRPSUHVVLRQ¿EHUVLVLQLQ (c) Stress in reinforcement varies linearly with strain up WRWKHVSHFL¿HG\LHOGVWUHQJWKfy. The stress remains constant y beyond this point as strains constant y beyond this p (d) o c ete compressive compressive stress distribution is assumed to (e) Concrete ngul (Fig. Fig. 7.4.1). be rectangular 77.4.1.1 1 N al ((Mn DQG GHVLJQ ÀH[XUDO VWUHQJWK Nominal (Mn))—A ([] —A section's s s Mn is calculated c from internal forces ([]M QJ WWKH H[WUHPH PH FRQF DVVXPLQJ FRQFUHWH FRPSUHVVLYH ¿EHU VWUDLQ eache 0.0 in. Beam reaches 0.003 in./in. Beams exhibit different behaviors depen ng oon the strain le depending level in the extreme tension reinf ((Section tion 21 21.2 2 ACI 318-14); tension controlled, forcement 21.2.2, FRPSUHVVLR Is •FRPSUHVVLR Is concrete beams behave in a ductile manner by limiting the area of reinforcement such that the tension reinforcement yields before concrete crushes. Tension controlled beam sections produce ductile behavior at the nominal FRQGLWLRQZKLFKDOORZVUHGLVWULEXWLRQRIVWUHVVHVDQGVXIccient steel yielding to warn against an imminent failure. The Code requires a beam's extreme tension reinforcement strain (if factored axial compression is less than 0.1fcgAg) to be at least 0.004 (Section 9.3.3.1, ACI 318-14). This strain corresponds to a reinforcement ratio of about 0.75db. The Code lowers the strength reduction ([])-factor for transition-level strains to account for reduced ductility in these sections. 9DULDWLRQ RI [DFWRUV ZLWK VWHHO WHQVLOH VWUDLQ LV VKRZQ LQ )LJDQGWKHFRUUHVSRQGLQJVWUDLQSUR¿OHVDWQRPLQDO strength (Fig. R21.2.2 (b), ACI 318-14). The basic design inequality is that the factored moment must not exceed the GHVLJQ AH[XUH VWUHQJWK PDWKHPDWLFDOO\ H[SUHVVHG DVMu " []Mn. 7.4.1.2 Rectangular sections with only tension reinforcement is calculated from the internal force couple shown in Fig. 7.4.1. The area of reinforcement is calculated from the equilibrium of forces. It is assumed that tension steel yields American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS T=C (7.4.1.2c) 0.85 f c'b Take moments about the concrete resultant, and Mn is calculated as:  $\beta$  c) a) a ( ( M n = T | di - 1 | = As f y | d - | + Ap f ps | dp - | ( ( 2 / 2 / 2 / ( ) ( ) (7.4.1.2d) where fps is calculated in Section 20.3 of ACI 318-14. Assume that a ù1c and rearrange expressions: Fig. 7.4.1.1—Strain distribution. For reinforced concrete sections with single layer tension reinforcement, d = dt and is = it (7.4.1.3). The stress block JHRPHWULFSDUDPHWHU ù1 is between 0.85 and 0.65. For FRQFUHWH VWUHQJWKV KLJKHU WKDQ SVL WKH YDOXH RI ù1 VKRXOG EH UHYLHZHG 2]EDNNDORJOX DQG 6DDWFLRJOX Ibrahim and MacGregor 1997). For nonprestressed beams, the Apsfps term in Eq. (7.4.1.2c) and (7.4.1.2d) of this Handbook is deleted. 7.4.1.3 Rectangular sections with tension and compression reinforcement—Generally, beams are designed with tension reinforcement
only. To add moment strength, designers can increase the tension reinforcement area or the beam depth. Th The cross-sectional dimensions of some ion, however, aare ree llimited by architectural or funcapplications, onsid tions, and aadditional moment strength can be tional considerations, prov d by adding ng an equa provided equal area of tension and compression The internal force couple adds to the i al m trength without changing the section's uctil y. In such cases, ases, the total moment strength consists ductility. ad ng ttwo components: mponents 1) moment strength from the of adding t i cement compression reinforcement-concrete compression reinforcement Mn = M1 + M2 (7.4.1.3a) M1 is given by Eq. (7.4.1.3a) and M2 is obtained by taking the moment about the tension reinforcement. M2 = Asgfsqd - dq E Assuming fsgLVHTXDOWRfy, Fig. 7.4.1.3—Forces in double reinforced concrete beam. American Concrete Institute – Copyrighted © Material – www.concrete.org F Beams before concrete reaches the assumed compression strain limit of 0.003 in./in. Accordingly, from equilibrium, set steel strength equal to concrete reaches the assumed compression strain limit of 0.003 in./in. concrete force, adding more tension steel does not create an over-reinforced section as long as an equal area is added in the compression zone. The underlining assumption in calculating the steel force couple is that the steel in compression yields at nominal strength, developing a force equal to the tensile yield strength. This assumption is true in most heavily reinforced sections because the compression Grade 60 steel (0.002 in./in. yield strain) is near the extreme compression reinforcement has less strain than 0.002 at nominal strength and, therefore, does not yield. The designer in this case, increases the compression steel strain in compression steel, isgFDQEHFRPSXWHG from Fig. 7.4.1.3 as isg is determined for sections with tension reinforcement to assess if the compression steel yields at nominal strength. 7.4.1.4 T-sections—Cast-in-place and many precast concrete slabs and beams are monolithic, so the slab particiSDWHV LQ EHDP¶V ÀH[XUDO VWLIIQHVV UHVXOWLQ] LQ D 7VHFWLRQ 7VHFWLRQ IIHFWLYHZ 12LGWKRID7VHFWLRQLVWKHHIIHFWLYHZLGWKRIWKH RI\$& DQG WKH VODE DV GH¿QHG LQ 6HFWLRQ RI\$&, rectangular beam forms the web. Pre Precast double T-sections )LJ D<sup>2</sup>(TXLYDOHQW VWUHVV GLVWULEXWLRQ RYHU ADQJH width. DOVR EHQH¿W IURP DQ LQFUHDVH LQ EHDP VWDELOLW\ GXULQJ construction 7KHADQJHZLGWKLQPRVW7VHFWLRQVLVVLJQL¿FDQWO\ZLGHU than the web width (Fig. 7.4.1.4a). For a lightly reinforced section, this often places the neutral axis of the nominal strain GLDJUDPZLWKLQWKHADQJHGHSWK7VHFWLRQVDUHDQDO\]HGWKH same as rectangular sections, with section width equal to the HIIHFWLYHADQJHZLGWK In heavily reinforced T-sections, the area of tension reinforcement in the web (required by the applied moment) EULQJVWKHQHXWUDOD[LVEHORZWKHADQJHFUHDWLQJDFRPSUHVsion zone in the web. In such a case, the total moment strength FRQVLVWVRI WHQVLRQVWHHOIRUFHHTXDOWRWKHADQJHFRQFUHWH compression force; and 2) the remaining tension steel force equal to the web concrete compression force; and 2) the remaining tension steel force equal to the web concrete compression force; and 2) the remaining tension steel force equal to the web concrete compression force. The condition for T-section behavior is expressed mathematically as Mu"\_Mn = [(Mnf + Mnw) (7.4.1.4a) where [  $\phi$ M n f =  $\phi$  | 0.85 f c'bh f [ hf / | \ d - 2 \] ] / ] a a / (  $\phi$ M nw = φAs fy | d - | + φAp f ps | dp - | \ 2 / 2 / (7.4.1.4b) (7.4.1.4c) Many eng Many engineers calculate Mnf¿UVWIURPHTXLOLEULXPWR¿QG WKHD DRI HQVLRQVWH WKHDUHDRIWRWDOWHQVLRQVWH wKHDUHDRIWR 7.4.1.4b. continuou statically indeterminate, PT beams, Forr continuous, o reactions induced by prestressing (secondary effects of moments) need to be included per Section 5.3.11 of ACI 318-14. The beam's secondary effects of moments) need to be included per Section 5.3.11 of ACI 318-14. The beam's secondary effects of moments are a result of the column's vertical restraint of the beam against the PT "load" at each support. Because the post-tensioning (secondary moments) need to be included per Section 5.3.11 of ACI 318-14. The beam's secondary moments are a result of the column's vertical restraint of the column's vertical restraint of the beam against the PT "load" at each support. Because the post-tensioning (secondary moments) need to be included per Section 5.3.11 of ACI 318-14. The beam's secondary moments are a result of the column's vertical restraint of the beam against the PT "load" at each support. force and drape are determined during the service stress checks, secondary moment is to subtract the tendon force times the tendon eccentricity (distance from the neutral axis) from the total balance moment, expressed mathematically as M2 = Mbal - P × e. Fig. 7.4.1.4b—T-section behavior. American Concrete Institute - Copyrighted © Material - www.concrete.org 137 Beams CHAPTER 7—BEAMS Fig. 7.4.1.5—Area of minimum bonded deformed longitudinal reinforcement distribution. 7.4.1.5 0LQLPXPAH[XUDOUHLQIRUFHPHQW—Nonprestressed reinforcement distribution] in a section is effective only after concrete has FUDFNHG,IWKHEHDP¶VUHLQIRUFHPHQWDUHDLVLQVXI¿FLHQWWR provide a nominal strength larger than the cracking moment, the section cannot sustain its loads upon cracking. This level of reinforcement can be calculated under light loads or beams that are, for architectural and other functional reasons, much larger than required for strength. To protect against potentially brittle behavior immediately after cracking, ACI 318-14). As , min = 3 f c' fy bw d (7 (7.4.1.5a) 41 bwdd/fy. but As,min needs to be at least 200b on For statically determina bea beams, where the T-se T-section UF W UHTXLUHG WR SUR YLGH ADQIH LV LQ WHQVLRQ WKH UHLQIRUFHPHQW SURYLGH oxia nominal strength above the cra cracking moment is app approxiular sections. T remately twice that required for rec rectangular Thereed by the smaller of 22bw or fore, bw in Eq. (7.4.1.5a) is replaced WKHADQJHZLGWK6HFWLRQRI\$&, +RZHYHU when the steel area provided in every section of a member LV VXI¿FLHQW WR SURYLGH AH[XUDO VWUHQ]WK DW OHDVW RQHWKLUG greater than required by analysis, the minimum steel area need not apply (Section 9.6.1.3 of ACI 318-14). This exception prevents requiring excessive reinforcement in overlarge beams. For prestressed beams with bonded strands, the minimum reinforcement area is that required to develop a design moment (Section 9.6.2.1 of ACI 318-14): []Mn•Mcr (7.4.1.5b) For prestressed beams with unbonded strands, abrupt AH[XUDO IDLOXUH LPPHGLDWHO] DIWHU FUDFNLQI GRHV QRW RFFXU because there is no strain compatibility between the unbonded strands and the surrounding concrete. Therefore, for unbonded tendons, the code only requires a minimum steel area of 0.004Act. These bars should be uniformly distributed over the precompressed tensile zone, close to the H[WUHPHWHQVLRQ¿EHUV)LJ 7.4.2 Shear—Unreinforced concrete shear failure is brittle. This behavior is prevented by providing adequate shear reinforcement that intercepts the assumed inclined cracks. The beam's shear force is usually maximum at a Fig.7.4.2—Types of cracking in concrete shear failure is brittle. decreases as it moves away from the support, usually at a slope equal to the magnitude of the unit uniform ORDG,QUHJLRQVRIKLJKAH[XUHFUDFNVIRUPSHUSHQGLFXODU to the longitudinal tension reinforcement. Principal tension stresses are approximately horizontal at the longitudinal reinforcement, then change direction gradually to approximately vertical at the location of maximum compression stress, as indicated crackss in Fig. 7.4.2. cated by the cra 7.4.2.1 strength—Concrete beams are designed 1 SShear ear streng h— to resist she shear and torsion and to ensure ductile behavior nominal condition. Shear strength at any location at the th nom ondition. S along a beam bea is calculated located using the simple ACI shear strength 2.5.5.1) Vc = 2 $\lambda$  f c'bw d instead of the more 318-14, Eq. (22.5.5.1) quations iin Table 22.5.5.1 (ACI 318-14). For detailed equations DPV UHVLVWLQJ VLJQL¿FDQW D[LDO IRUFH WKH FRQFUHWH VKHDU t t is calculated per ACI 318-14, Table 22.5.6.1 and strength Eq. (22.5.7.1). Note that for a beam resisting tension, Vc cannot be smaller than zero. The nominal concrete shear strength, Vc, for prestressed EHDPV GH¿QHG DV Apsfps • Apsfpu + Asfy) can be calcuODWHG XVLQJ \$&, VLPSOL¿HG HTXDWLRQV OLVWHG LQ Table 22.5.8.2, but need not be less than Vc calculated by Eq. (22.5.5.1) of ACI 318-14. The more detailed approach to calculate Vc for prestressed beams is to use the lesser of shear diagonal cracking, Vci (Section 22.5.8.3.1 of ACI 318-14) and shear web cracking, Vci (Section 22.5.8.3.2). The factored shear strength), Vu, can be calculated at a distance d from a support face for the usual support condition (Section 9.4.3.2 of ACI 318-14) and Fig. 7.4.2.1a of this Handbook). For other support conditions, or if a concentrated load is applied within the distance d from a support, the required shear strength is taken at the support face (refer to Fig. 7.4.2.1b and ACI 318-14, Fig. R9.4.3.2 (e) and (f)). Beam shear reinforcement are usually U-shaped stirrups (Fig. 7.4.2.1b and ACI 318-14, Fig. R9.4.3.2 (e) and (f)). stirrups. Shear cracking is assumed to occur at 45 degrees from horizontal. \$&,XVHVDWUXVVDODORJIRUVKHDUARZZKHUHWKHVWLUrups are vertical tension ties in the truss with concrete acting American Concrete Institute - Copyrighted © Material - www.concrete.org 138 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig 7.4.2.1a—Free body diagrams of the end of a beam. Fig.7.4.2.1b—Location of critical section for shear in a beam (R9.4.3.2a, ACI 318-14). Fig. 7.4.2.1c—Shear reinforcement is the tension chord and concrete is the compression chord. For design, the tension force in each stirrup leg is assumed to be its yield
strength times the leg area, and beam stirrups usually have two vertical legs. A U-stirrup has an area  $Av = 2 \times (leg area)$ . The beam's nominal steel shear strength is calculated by Vs = Avfytd/s. Designers usually calculate the required Vs and then determine the stirrup size and spacing, so the equation is rearranged as Av/s = Vs/(fyd). 7.4.2.2 Designing shear reinforcement—In general, if a beam's [Vc/2 is less than Vu, shear reinforcement, Av, is OHHGHG7KHVWUHQIWKUHGXFWLRQIDFWRUIRUVKHDULV7KH required area per unit length is: s"[Avfytd]/(Vu±[Vc]) (7.4.2.2) Vu/[] - Vc represents the nominal shear strength provided by shear reinforcement Vs. There are limited exceptions to the above general rules given in Section 9.6.3.1 of ACI 318-14. For example, a beam having width bw more than twice the thickness h does not require minimum shear reinforcement as long as the design concrete shear strength. Note that because the longitudinal beam bars requires support, it is impractical to eliminate beam stirrups, so it is recommended to provide stirrups in all cast-in-place beams, with spacing not exceeding d/2. \$W\SHRIULEEHGARRUVODENORZODVDMRLVWV\VWHPLVRIWHO FROVWUXFWHGZLWKRXWVKHDUUHLOIRUFHPHOWLOWKHMRLVWULEV\$ MRLVWV\VWHP¶VUHODWLYHGLPHQVLRQDOOLPLWVVXFKDVVODEWKLFNness, rib width, and rib spacing, are provided in ACI 318-14, 6HFWLRQ,IWKHULEEHGARRUV\VWHPGRHVQRWFRQIRUPWR DOOWKHFRGHOLPLWVVXFKDVDVNLSMRLVWV\VWHP wKHV\VWHP needs to be designed as a beam and slab system. Section 9.6.3.3 (ACI 318-14) sets lower limits on the Av to ensure that stirrups do not yield upon shear crack formation. The value of Av must exceed the larger of 0.75 f c'bw s /f yt and 50bws/fyt7KH¿UVWTXDQWLW\JRYHUQVLIfcg!SVL\$&, 318-14 has an upper limit of f c' to 100 psi, which corresponds to 10,000 psi concrete strength. Section 22.5.3.2 of ACI 318-14 allows the value of f c' to be greater than 100 psi if the reinforced and prestressed beam has shear reinforcement per Section 9.6.3.3 and 9.6.4.2 of ACI 318-14. Refer to the C Code commentary on this sectional shear stresses the that increase from zero stress at m s sectional nal center to the maximum at the section perimeter. torsion stress occurs Empir al eexpressions ons for to Empirical torsional strength are provided in Secti 9.5.4.1 9.5. of ACI 318Section 318-14. The torsion shear stress adds t the gravity i shear hear stre to stress on one vertical face, but subtracts he opposite vertical face (Fig. 7.4.3a). Refer also from it on the JDIRU WR)LJDIRUWKHGH¿QLWLRQVRIVHFWLRQSURSHUWLHV When designing d for torsion, the engineer needs to distinguish between statically determinate (an uncommon condition) and statically indeterminate (or equilibrium) torsion is the condition). Statically determinate (or equilibrium) torsional resistance, that is the torsional moment cannot be reduced by internal force redistribution to other members. If inadequate torsional reinforcement is provided to resist this type of torsion, the beam cannot resist the applied factored torsion. Statically indeterminate (or compatibility) torsion exists is the condition where, if the beam loses its ability to resist torsion, the moment is able to be redistributed, equilibrium is maintained, and the torsion load is safely resisted by the rest of the structural system. Torsional moments can be redistributed after beam cracking if the member twisting is resisted by compatibility of deformations with the eccentric load (wue) on the ledge to columns through beam torsion. In Fig. 7.4.3b(b), the eccentric load can be resisted by WRUVLRORIWKHEHDPRUE\VODEAH[XUH, ORWKHUZRUGVLIWKH American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS 139 Condition Spacing Provision in ACI 318-14 Vu"  $\varphi Vc < \Psi d \le 24$  in. 2 9.7.6.2.2 3h s \le 24 in. 4 Prestressed  $\varphi Vc + \varphi d f c'bw d d \le 12$  in. 4 Prestressed  $\varphi Vc + \varphi d f c'bw d d \le 12$  in. 4 Prestressed  $\psi Vc + \varphi$ torsional stiffness, the slab can resist the eccenWULFORDGLOJHIIHFWVWKURXJKAH[XUH A beam's cracking torque, Tcr, is calculated without consideration of torsion reinforcement. Tcr = 4λ f c'(Acp) 2 /pcp (7.4.3a) ACI 318-14 assumes that torques less than 1/4 of Tcr ZLOO ORW FDXVH D VWUXFWXUDOO\ VLJOL¿FDOW UHGXFWLRO LO WKH f c' shear strength and thus is ignored. ACI 318 limits to a maximum of 100 psi, which corresponds to 10,000 psi concrete area enclosed on available research. ACI 318-14, Eq. (22.7.7.1a), provides an upper limit to the torque resistance of a concrete beam: Tmax = 17 (Aoh)  $\lambda$  f c' /ph 2 (7.4.3b) where Aoh is concrete area enclosed by centerline of the outermost closed transverse torsional reinforcement (Fig. 7.4.3c). 7.4.3.1 Torsion reinforcement—Concrete beams reinforcement yields. ACI 318-14 VSHFL¿HV EHDP UHLQIRUFHPHQW WKDW UHVLVWV WRUVLRQ EH FORVHG stirrups and longitudinal bars located around the section SHULSKHU\7RUVLRQ FUDFNV DUH DVVXPHG DW DQJOH s IURP WKH member axis, so the torsion strength from closed stirrups is calculated as Tn = 2 Ao At f yt s cot  $\theta$  (7.4.3.1a) where AoLVWKHJURVVDUHDHQFORVHGE\WRUVLRQDOVKHDUARZ path, in.2, At is the area of one leg of a closed stirrup, in.2, and American Concrete Institute - Copyrighted © Material - www.concrete.org Beams Table 7.4.2.2-Shear reinforcement requirements 140 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. 7.4.3b-Determinate and indeterminate torsion. 7.5-Temperature and shrinkage reinforcement 5HIHUWR&KDSWHU2QHZD/VODEVIRULQIRUPDWLRQ Fig. 7.4.3c—Determining Aoh. fyt is the yield strength of transverse reinforcement, einforce psi. ACI EHJU VSHFL¿HVWKDWDQJOH, PXVWEHJUHDWHUWKDQGHJUHHVIRUVLPSOLFLW/LQGHVLJQXVH, o be large enou degrees. Solid concrete sectionss nneed to enough to ion shear ar Tu within the u upper per UHVLVWÀH[XUDOVKHDUVu and torsion 7.1a) limits given by ACI 318-14, Eq. ((22.7.7.1a): 2 2 (Vu) (Tu ph) (Vc) || b d || + || 1.7 A2 ||  $\leq \varphi$  || b d + 8 f c' || w w oh (7.4.3.1b) Where stirrups are required for torsion in addition to shear, Section 9.6.4.2 of ACI 318-14 requires that the area of two legs of a closed stirrup (Av + 2At) must exceed 0.75(bws/fyt) and 50bws/fy. /RQJLWXGLQDO VSDFLQJ RI WKH FORVHG VWLUUXSV PXVW QRW exceed 12 in. ACI 318, Eq. (22.7.7.1b) requires that the longitudinal bar area, AE, be placed around the section periphery. Section 9.6.4.3 of ACI 318-14, requires a minimum area of longitudinal reinforcement AEPLQ be the lesser of (a) and (b): (a) 5 f c'Acp fy f yt (A) - |t| ph \ s) fy (25bw) f yt - |ph | fy fy \ f yt / The torsion strength from longitudinal bars is calculated as (b) 5 f c'Acp Tn = 2 Ao AA f y Ph cot  $\theta$  (7.4.3.1c) 7.6—Detailing The longitudinal bar details includes determining the bar size(s), distribution around the perimeter, lengths, and cutoff points. The stirrup details includes determining size, VSDFLOIDOGFRO2IXUDWLRO 7.6.1 Reinforcement placement—To limit crack widths, it is preferable to use a larger number of small bars, as opposed to fewer large bars, bars 7.6.1.1 Minimum spacing of longitudinal reinforces aci c menW<sup>2</sup>/ROJLWXGLODO VKRXOG EH SODFHG DW /ROJ GL O UHLOIRUFHPHQW I spacing that allows proper placement of concrete. Table p tha ws for prop A-3 oof ACI Reinforced Concrete Design Handbook Design rced Conc Aid – Anal Analysis Tables SP-17DA) shows the 318-14 les (ACI (minimum m sspacing requirements equirements equirement for beam reinforcement. 7.6.1.2 protection for reinforcement—The 7.6. 2 Concrete C te prote reinforcement rein eme should ould be protected against
corrosion and OYLUROPHO E\ D VXI¿FLHOWO\ WKLFN FROFUHWH DIJUHVVLYH HOYLUROPHOWV effer to 20.6.1.3.1 20.6 cover (refer of ACI 318-14), as indicated in Reinforc Concrete Design Handbook Design Aid - ACI Reinforced Analysis Tables (ACI SP-17DA). The engineer should also FRQVLGHU WKH EHDP V UHTXLUHG ¿UH UDWLQJ ZKHQ GHWHUPLQLQJ concrete cover (Section 4.11.2 of ACI 318-14). Considering cover, reinforcement should be placed as close to the concrete surface as practicable to maximize the lever arm for internal moment strength and to restrain crack widths. 7.6.1.3 Reinforcement in a T-section—Where a beam's 7VHFWLRQADQIHVDUHLQWHQVLRQUHLQIRUFHPHQWQHHGVWREHGLVWULEXWHGRYHUWKHADQIHZLGWKRUD width equal to 1/10 the span, whichever is smaller (refer to Section 24.3.4 of ACI 318-14). This requirement is intended to limit slab crack widths that can result from widely spaced reinforcement should be provided LQWKHRXWHUSRUWLRQVRIWKHADQJHWRPLQLPL]HZLGHFUDFNV in these slab regions 7.6.1.4 OD[LPXP VSDFLQ] RI AH[XUDO UHLQIRUFHPHQW— Beams reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between the bars, even when the required tension reinforced with few large bars could experience cracking between tension reinforced with few large bars could experience cracking between tension reinforced with few large bars could experience cracking bars could experien VSHFL¿HV D PD[LPXP VSDFLQ] s, for reinforcement closest American Concrete Institute – Copyrighted © Material – www.concrete.org to the tension face. The spacing limit is the lesser of the two equations that follow: (40, 000) s  $\leq 15$  | -2.5cc (f s |) (40, 000) s  $\leq 12$  | f |) (7.6.1.4) s In the above equation, cc is the least distance from the reinforcement surface to the tension face of concrete, and fs is the service stress in reinforcement. The service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain compatibility analysis under unfactored service stress, fs, can be calculated from strain conditions or designed to be watertight. For such conditions, further investigation is warranted (Section 24.3.5 of ACI 318-14). 141 7.6.1.5 Skin reinforcement—In deep beams, cracks may develop near the beam's mid-depth, between the neutral axis and the tension face. Therefore, the Code requires beams with a depth h > 36 in. to have "skin reinforcement" with a maximum spacing of s DV GH¿QHG LQ (T DQG illustrated in ACI 82.4). For this case, cc is the least distance from the skin reinforcement surface to the side face. ACI 318 does not specify a required steel area as skin reinforcement. Research indicates that No. 3 to No. 5 bar sizes or welded wire reinforcement with a minimum area of 0.1 in.2IWSURYLGHVXI¿FLHQWFUDFNFRQWURO)URVFK 7.6.2 Shear reinforcement. Research indicates that No. 3 to No. 5 bar sizes or welded wire reinforcement with a minimum area of 0.1 in.2IWSURYLGHVXI¿FLHQWFUDFNFRQWURO)URVFK 7.6.2 Shear reinforcement. spacing less than 3 in. can FUHDWHGLI¿FXOWLHVLQSODFLQJFRQFUHWH7KHUHIRUHVRPHHQJLneers increase the stirrups should be distributed across the cross section to engage the full beam width and thereby improve shear resistance (Fig. 7.6.2). 7.6.3 Torsion reinforcement—The detailing requirements resist torsion are listed in ACI 318-14, Sections for beams resisting 9.7.55 and 9.7.6, for longitudinal bars are distributed up perimet around the stirrup perimeter, with at least one longitudinal aced in each ch corner (Section 9.7.5.2). of ACI 318-14). bar is placed To res resist tor torsion, thee stirrup eends are closed with 135-degree k (Fig. Fig. 7.6.2(c) and (e) and 7.6.3). A 135-degree hook where the stirrup end is FRQ¿ GDQG DLQHGDJDL FRQ¿QHGDQGUHVWUDLQHGDJDLQVWVSDOOLQJE\DVODERUADQJHRI i (refer er

to Fi Fig a T-section Fig. 77.6.2(a), (b), and (d)). Splicing stircceptable ffor torsion reinforcement (Fig. 7.6.3). rups is not acceptable Fig. 7.6.1.5—Skin reinforcement for beams and joists with h > 36 in. (R9.7.2.3, ACI 318-14). Fig. 7.6.2a—Longitudinal reinforcement distributed around beam perimeter with closed stirrups. American Concrete Institute Copyrighted © Material - www.concrete.org Beams CHAPTER 7-BEAMS 142 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. 7.6.3-Detailing of closed stirrups for torsion. REFERENCES American Concrete Institute (ACI) ACI SP-17DA-14-Reinforced Concrete Design Handbook Design Aid - Analysis Tables; h t t p s : / / w w w. c o n c r e t e . o r g / s t o r e / p r o d u c t d e t a i l . aspx?ItemID=SP1714DA Authored references Frosch, R. J., 2002, "Modeling and Control of Side Face Beam Cracking," ACI Structural Journal91R0D\ June, pp. 376-385. , EUDKLP+++DQG0DF\*UHJRU-\*30RGL¿FDtion of the ACI Rectangular Stress Block for High-Strength Concrete," ACI Structural Journal91R-DQ)HE pp. 40-48. 2]EDNNDORJOX7DQG6DDWFLRJOX0<sup>3</sup>5HFWDQJXODU Stress Block for High-Strength Concrete," ACI Structural Journal91R-XO\\$XJSS Post-Tensioning Manual, sixth edition, PTI TAB. 1-06, 354 pp. American Concrete Institute – Copyrighted © Material – www.concrete.org 7.7—Examples Beam Example 1: Continuous interior beam Design and detail an interior, continuous, six-bay beam, built integrally with a 7 in. slab. Given: Load— Service additional dead load D = 15 psf Service live load L = 65 psf Beam and slab self-weights are given below. Material properties— fcg SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi 3 QRUPDOZHLJKWFRQFUHWH Span length: 36 ft Beam width: 18 in. Column dimensions: 24 in. x 24 in. Tributary width: 14 ft Fig. E1.1—Plan and partial elevation of 6-span interior beam. American Concrete DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Material requirements 9.2.1.1 The mixture proportion must satisfy the durability requirements of Chapter 19 (ACI 318-14) and structural strength requirements of the Categories and Classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing \$&,LQWRMREVSHFL¿FDWLRQV There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Step 2: Beam geometry 9.3.1.1 Beam depth ,IWKHGHSWKRIDEHDPVDWLV¿HV7DEOH\$&, ned without 318-14 permits a beam to be designed VDVORQJDVWKHEHDP tached tto partitions or other is not supporting or attached WREH FRQVWUXFWLRQOLNHO\WREHGDPDJHGE\ODUJHGHÀHFEHD WLRQV2WKHUZLVHEHDPGHAHFWLRQVPXVWEHFDOFX\WK HFWLRQOLPLWVLQ HFWLRQ ODWHGDQGVDWLVI\WKHGHAHFWLRQOLPLWVLQ6HFWLRQ 89.3.2 of ACI 318-14. Self-weight Beam: Slab: Flange width 9.2.4.2 The beam is placed monolithically with the slab ZLGWK DQGZLOOEHKDYHDVD7EHDP7KHADQJHZLGWK on each side of the slab and slab an the beam is obtained from Table 6.3.2.1. 6.3.2.1 Each side ] 8hslab | of web is } sw /2 the least of | ] A n /8 Flange width: bf = En/8 + bw + En/8 Calculation By specifying that the concrete mixture shall be in accordance with ACI 301-10 and providing the exposure classes, Chapter 19 (ACI 318-14) UHTXLUHPHQWVDUHVDWLVcHG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 5000 psi. The beam has ha four continuous spans, so the controlg condition for beam am depth is one end continuous: ling h = A (36 ft)(12 iin./ft) = 23.35 in. 18. 18.5 18.5 U 30 in. Use wb = [(18 [(1 in.)(30 30 in.)/(14 in.)/(144)](0.150 kip/ft3) = 0.56 kip/ft 8 in in./12)(7 / 12)(in./12)(0.150 kip/ft3) = 1.1 kip/ft ws = (14 ft - 18 8(7 in.) = 56 in. (14 ft)(12)/2 = 84 in. ((36 ft)(12 in./ft) - 24 in.)/8 = 51 in. = 120 in. American Concrete Institute - Copyrighted © Material - www.concrete.org Controls CHAPTER 7—BEAMS145 5.3.1 Beams 6WHS/RDGVDQGORDGSDWWHUQV 7KHVHUYLFHOLYHORDGLVSVILQRI¿FHVDQG psf in corridors per Table 4-1 in ASCE 7-10. This example will use 65 psf as an average as the actual layout is not provided. A 7 in. slab is a 87.5 psf service dead load. To account for the weight of ceilLQJVSDUWLWLRQV+9&V (VWHPVHWFDGGSVIDV miscellaneous dead load. The beam resists gravity only and lateral forces are not considered in this problem. U = 1.4(0.56 kip/ft + 1.1 kip/ft + (15 psf)(14ft)/1000) = 2.6 kip/ft U = 1.2D + 1.6L U = 1.2(2.6 kip/ft)/1.4 + 1.6((65 psf)(14 ft)/1000) = 3.7 kip/ft Controls (14 ft)/1000) = 3.7 kip/ft Controls (14 ft)/1000) = 2.6 kip/ft U = 1.2D + 1.6L U = 1.2(2.6 kip/ft)/1.4 + 1.6((65 psf)(14 ft)/1000) = 3.7 kip/ft Controls (14 ft)/1000) = 2.6 kip/ft U = 1.2D + 1.6L U = 1.2(2.6 kip/ft)/1.4 + 1.6((65 psf)(14 ft)/1000) = 3.7 kip/ft Controls (14 ft)/1000) = 3.7 kip/1RWH/LYHORDGLVQRWUHGXFHGSHU\$6&(LQWKLVH[DPSOH Step 4: Analysis 9.4.3.1 The beams are built integrally with supports: 9.4.1.2 -14 permits perm several analysis Chapter 6 of ACI 318-14 ulate the required strengths) are calculated at the face of the supports. 9.4.1.2 -14 permits perm several analysis Chapter 6 of ACI 318-14 ulate the required strengths) procedures to calculate 6.5.1 uire strengths engths can be ccalculated ulated The beam's required mat o ACI using approximations per Table 6.5.2 of on ns in Section 6.5 318-14, if the conditions 6.5.1 are VDWLV¿HG ic (a) Members are prismatic E /RDGVXQLIRUPO\GLVWULEXWHG (c) L"D (d) Three spans minimum ms are prismatic ismatic www.concrete.org 146 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Using En = 34 ft for all bays results in the following and moment ment diagrams. 6.5.3 Note: lcula using the approximate method cannot anno be redistribute The moments redistributed in accordance with Section 6.6.5.1. m drawn n on the tension side ide of the beam Moment diagram beam. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 147 Beams Step 5: Moment design 9.3.3.1 The code doesn't permit a beam to be designed with steel strain less than 0.004 in./in. at nominal strength. The intent is to ensure ductile behavior. In most reinforced concrete beams, such as this example, bar strain is not a controlled with a moment reduction IDFWRU7 KLVDVVXPSWLRQZLOOEHFKHFNHG later. Determine the effective depth assuming No. 3 stirrups, No. 7 longitudinal bars, and 1.5 in. cover: 20.6.1.3.1 2QHURZRIUHLQIRUFHPHQW d = h - cover - dtie - db/2 22.2.2.1 The concrete compressive strain at nominal moment strength is calculated at: İcu = 0.003 22.2.2.2 QFUHWHLQÀH[XUHLVDYDUL 7KHWHQVLOHVWUHQ]WKRIFRQFUHWHLQÀH[XUHLVDYDULable] property and is approxi approximately 10 to 15 percent mpre of the concrete compressive strength. d = 30 in. -1.5 in. -0.375 in. sstrength: 22.2.2.3 22.2.2.4.1 The concrete compressive stress distribution is inelastic at high stress. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRI 22.2.2.4.3 a ü1cZKHUHü1 is a function of concrete compression force to the tension force within the beam cross section: C=T 0.85fcqba = Asfy 0.85(5000 psi)(b)(a) = As(60,000 psi) = 0.118 As 0.85(5000 psi)(120 in.) For negative moment: b = bf = 120 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(120 in.) American Concrete Institute – Copyrighted © Material www.concrete.org 148 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E1.3— Section compression block and reinforcement locations. 7KHEHDPLVGHVLJQHGIRUWKHPD[LPXPAH[XUDO moments obtained from the approximate method above. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KHEHDPLVGHVLJQHGIRUWKHPD[LPXPAH]XUDO moments obtained from the approximate method above. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KHEHDPLVGHVLJQHGIRUWKHPD[LPXPAH]XUDO moments obtained from the approximate method above. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH²UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH²UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH²UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement locations. 7KH²UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the two section compression block and reinforcement location compression block and reinforcement location compression block and reinforcement location compression block and reinforcement locatio moments. 9.5.1.1 h must be at least equal The beam's design strength h at each section along its to the required strength length: []Mn•Mu []Vn•Vu 9.5.2.1 %HDPLVQRWVXEMHFWHGWRD[LDOIRUFHWKHUHIRUH EMH WRD[LDOIRUFHWK UHIRU assume Pu < 0.1ffcgAg 22.3 )LJ(<sup>2</sup>.H\WRPRPHQWXVHZLWKWDEOHEHORZ )LJ(<sup>2</sup>.H\ Table 1 quired i d rreinforcement 1.1—Required Calculate the required reinforcement area (refer to Fig. E1.2 for design moment values and Fig. E1.4 for moment
location). a) ( $M u \leq \varphi M n = \varphi As f y | d - | \langle 2 \rangle 21.2.1a 2 [A No. 7 bar has a db = 0.875 in. and an As = 0.6 in. a has been calculated above as a function of As 21.2.2 9.3.3.1 Check if the calculated strain$ exceeds 0.005 in./ in. (tension controlled (Fig. E1.5), but not less than 0.004 in./in. As f y a and c =  $a = \beta 1$  0.85 f c'b ZKHUHù1 = 0.8 (calculated above) Note: b = 18 in. for negative moments and 120 in. for positive moments.  $\epsilon t = \epsilon cu$  (d - c) c Number of No. 7 bars Location catio Mu, ft-kip As, req'd, in.2 Req'd Prov. MU1 267 2.23 3.7 4 MU2 428 3.65 6.1 7 MT 389 3.30 5.5 6 ME+ 306 2.49 4.1 5 M I+ 267 2.17 3.6 4 1RWH7KHEHDPDWWKH¿UVWLQWHULRUVXSSRUWLVGHVLJQHG for the larger of MUI 267 2.40 1.88 0.0313 Y MUI 267 2.40 1.88 0.0313 Y MUI 267 2.40 1.88 0.0313 Y MUI 2428 4.20 3.29 0.0179 Y MT 389 3.60 2.82 0.0208 Y ME + 306 3.00 0.35 0.166 Y MI + 267 2.40 0.28 0.208 Y 7KHUHIRUHDVVXPSWLRQRIXVLQJ[LVFRUUHFW American Concrete Institute - Copyrighted © Material - www.concrete.org 149 Beams CHAPTER 7—BEAMS Fig. E1.5—Strain distribution across beam section. 9.6.1.1 9.6.1.2 Minimum reinforcement The provided reinforcement must be at least the minimum required reinforcement at every section along the length of the beam. As = 3 f c' fy bw d 200 bw d fy (9.6.1.2a) As = 3 5000 psi (0.1.2b) b As = 3 5000 psi 60,000 (9.6.1 (9.6.1.2b) b) As = 200 (18 in in.)(2)(2 in.)(27.5 in.) = 1.65 in.2 60 60,000 psi Controls 44 psi, Eq. (9.6.1.2a) As = <math>3 f c' fy bw d 200 bw d fy (9.6.1.2b) b As = 3 f c' fy bw d 200ccontrols. trols. Because fcg > 4444 R uire reinforcement forcement areas exceed the minimum Required uired reinforcement forcement forcement forcement areas exceed the minimum Required uired reinforcement forcement DUHVDWLViHGWKHGHVLJQVKHDUIRUFHLVWDNHQ at critical section at distance d from the face of the support (Fig. E1.6). Fig. E1.6—Shear at the critical section. [email protected] = (72 kip) - (3.7 kip/ft)(27.5 in./12) = 63.5 kip 22.5.5.1 Vc = 2 f c bw d Vc = 2 5000 psi(18 in.)(27.5 in.)/1000 = 70 kip 21.2.1(b) Shear strength reduction factors []shear = 0.75 []Vc = (0.75)(70 kip) = 52.5 kip 9.5.1.1 &KHFNLI[]Vc • Vu []Vc = 52.5 kip 9.5.1.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Prior to calculating shear reinforcement, check if the cross-sectional dimensions satisfy Eq. (22.5.1.2):  $Vu \le \varphi(Vc + 8 \text{ f c'bw d}) 21.2.1(b) \square Vu \le \varphi$ satisfying Eq. (22.5.10.1) is required at each section where Vu  $\exists Vs \ge Vu - Vc \varphi \exists Vs \le Vu - Vc \varphi$   $\exists Vs \le 4 f c'bw d ? 11.0 kip 2(0.11 in.)$  $(60,000 \text{ psi})(27.5 \text{ in.}) = \varphi \text{ s} = 24.8 \text{ in.}$  This is a very large spacing and must be checked against the maximum num stirrup spacing ng is the threshold value, the 1 in. d/2 = 27.5 in./2 = 13.8 lesser of d/2 and 24 in. Use s = 12 in. < d/2 = 13.8 in  $\therefore$  OK 9.6.3.1 In the region where Vu"  $\Box$ Vc/2, shear reinforcement is not required. In this example, shear reinforcement is not required. In this example, shear reinforcement, however, is provided over the full length. 6SHFL $\dot{c}$ HGVKHDUUHLQIRUFHPHQWPXVWEHDWOHDVW 9.6.3.3 Av , min s = 0.75 f c bw f yt Av, min s  $\geq 0.755000$  psi 18 in. = 0.016 in.2 /in. 60,000 psi and Av, min s OK Controls = 50 bw f yt Av, min s  $\geq 5018$  in. = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av, min s  $\geq 5018$  in. = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av, min s  $\geq 5018$  in. = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av, min s  $\geq 5018$  in. = 0.015 in.2 /in. s s 12 in. 60,000 psi and Av, min s  $\geq 5018$  in. = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av, min s  $\geq 5018$  in. = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. 60,000 psi and Av = 0.018 in.2 /in. s s 12 in. s s 12 Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 151 9.7.2.2 24.3.4 Beams Step 7: Reinforcement detailing Minimum bar spacing greater of {db | 4 / 3(d) agg [ 1 in. Assume maximum aggregate size is 0.75 in Therefore, clear spacing between horizontal bars must be at least 1.0 in. 7HQVLRQUHLQIRUFHPHQWLQADQJHVPXVWEHGLVWULEXWHGZLWKLQWKHHIIHFWLYHADQJHZLGWKbf = 120 in. (Step 2), but not wider than: En/10. 0.875 in. 4/3(3/4 in.) = 1 in. En/10 = (34 ft)(12)/10 = 40.8 in. < 120 in., say, 41 in. %HFDXVHHIIHFWLYHADQJHZLGWKH[FHHGVEn/10, additional bonded reinforcement is required in the outer SRUWLRQRIWKHADQJH Table 1.3—Top flange bar distribution Use No. 5 for additional bonded reinforcement. This requirement is to control ntrol cracking in the slab due to wide spacing of bars aacross the full effective ÀDQJHZLGWKDQGWRSURWHFWÀDQJHLIUHLQIRUFHPHQW RSUR is concentrated with within the web width. \* Location Loc Prov. No. 7 in web No. 7 in w )RUWKH¿UVWLQWHULRUVXSSRUWSODFHWHQVLRQUHLQhe higher er design mome forcement. Fo For ns refer to Fig. E1.4. moment locations Exterior span positive tiv moment ment reinforced Concrete Design Handbook Design Aid – Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA. The spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in. + 0.75 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirrup = 0.375 in.) + 4db + 4(1 in.)min,spacing (25.2.1) where dstirru 12.75 in. < 18 in. OK 7KHUHIRUH¿YH1REDUVFDQEHSODFHGLQRQHOD\HU in the 18 in. beam web (Fig. E1.7). Spacing between longitudinal bars: 2.1 in. > 1 in. OK Fig. E1.7—Bottom reinforcement layout. American Concrete Institute - Copyrighted © Material - www.concrete.org 152 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 9.7.2.2 24.3.1 Maximum bar spacing at the tension face must not exceed the lesser of:  $24.3.2 (40, 000 \text{ psi}) s = 15 | | - 2.5cc fs ( 40, 000 \text{ psi}) s = 12 | | fs ( 40, 000 \text{ psi}) s = 12 | | fs ( 40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40,
000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{ in.} (40, 000 \text{ psi}) s = 12 | = 12 \text{$ in. 7KLVOLPLWLVLQWHQGHGWRFRQWUROAH[XUDOFUDFNLQ] width. Note that cc is the cover to the No.7 bar, not to the tie. Bottom reinforcement: If bars are not bundled, 2.3 in. spacing is provided (Fig. E1.7), therefore OK American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 153 Beams %RWWRPEDUOHQJWKDORQJ¿UVWVSDQ &DOFXODWHWKHLQÀHFWLRQSRLQWV)LJ( Fig. E1.8—Moment diagram of exterior span. ,QÀHFWLRQSRLQWV)LJ( Fig. E1.8—Moment diagram of exterior span. ,QÀHFWLRQSRLQWV)LZ( Fig. E1.8 diagram (Fig. E1.9a): Mmaxíwu(x)2/2 = 0 (3 6 ft(306 ft-kip) – (3.7 3.7 kip/ft)(x) kip/ft) 2/2 = 0 12. ft, say, y, 13 ft x = 12.86) (QAHFWLRQSRLQWIRUQRSWHQVLRQ<sup>2</sup> UVWVSDQ Exterior support: &DOFXODWHWKHLQAHFWLRQSRLQWIRUQHJDWLYHPRPHQW diagram (Fig. E1.9b): iMmaxiwu(x)2/2 + Vux = 0 [IWNLS ±NLSIW x)2/2 + (63 kip)x = 0 x = 4.96 ft, say, 5 ft )LJ(E±±,QÀHFWLRQSRLQWRIH[WHULRUQHJDWLYH moment. American Concrete Institute – Copyrighted © Material – www.concrete.org 154 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) , QÀHFWLRQSRLQWIRUWRSWHQVLRQí)LUVWLQWHULRU support &DOFXODWHLQÀHFWLRQSRLQWIRUWKHQHJDWLYHPRPHQW diagram (Fig. E1.9c): iMmaxiwu(x)2/2 + Vux = 0 [IWiNLS ±NLSIWx)2/2 + (72 kip)x = 0 x = 7.32 ft, say, 7 ft 6 in. )LJ(F±±,QÀHFWLRQSRLQWRILQWHULRUQHJDWLYH moment. 9.7.1.2 25.4.2.2 25.4.2.4 25.4.2.4 25.4.10.1 Development length of No.7 bar 7KHVLPSOL $\dot{c}$ HGPHWKRGLVXVHGWRFDOFXODWHWKH development length of No.7 bars. Ad = f y $\psi$ t  $\psi$ e 20 $\lambda$  f c' db where zt = 1.3 for top bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQz d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQZ d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQZ d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQZ d below more than 12 in.. of fresh sh concrete is pl placed ottom bars, because zt EDUORFDWLRQZ d below more than 12 pl ced bbelow more than 12 in. placed them. RU e = 1.0, because bbarss are ze FRDWLQJIDFWRUz uncoated Top bars: (60,000 psi)(1.3)(1.0) Ad = (0.875 in.) = 48.3 in. 5000 psi)(1.0)(1.0) 1.0) psi)(1.0)(1.0) psi)(1.0)(1.0)(1.0)(1.0)(1. iin. The calculated development lengths could be reduced according to the ratio of: Areq'd/Aprov. except as required by Section 25.4.10.2. In this example, development reduction is not applied. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 155 First span top bars Exterior support Bars must be developed at locations of maximum stress and locations along the span where bent or terminated tension bars are no longer required to UHVLVWAH[XUH Beams 9.7.3.2 Four No. 7 bars are required to resist the factored negative moment at the exterior column interior face. Calculate a distance x from the column face where two No. 7 bars can resist the factored moment.  $x^2 + (63 \text{ kip})(x)^2 = -2(0.5 \text{ in.} 2)(0.9)(60 \text{ ksi}) - (267 \text{ ft} - \text{ kip}) - (3.7 \text{ kip/ft}) / (2(0.6 \text{ in.} 2)(60 \text{ ksi}) / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 27.6 \text{ in.} - | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2 \times | 27.6 \text{ in.} - | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2 \times | 27.6 \text{ in.} - | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2 \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) | / (1) \times | 2(0.85)(5 \text{ ksi})(18 \text{ in.}) |$ WKH\DUHQRORQJHUUHTXLUHGWRUHVLVWAH[XUHIRUD eater of d or 12db. distance equal to the greater No bars: rs: For No.7 2 in. Controls Contro 1. d = 27.5 2ddb = 12(0.875 875 in.) in ) = 10.5 in. 2. 12d Th refo extend nd the m Therefore, middle two No. 7 bars the ater of the development length (51 in.) and the sum cal cutoff point and d from column face of theoretical E1. (refer to Fig. E1.10): 9.7.3.8.4 i At least one-third of the bars resisting negative moment at a support (two No. 7 > 1/3 of four No. 7) must have an embedment length beyond the LQÅHFWLRQSRLQWWKHJUHDWHVWRId, 12db, and En/16. 24 in. + 27.5 in. = 51.5 in., say, 54 in. Controls For No.7 bars: d = 27.6 in. Controls 12db = 12(0.875 in.) = 10.5 in.  $\epsilon n/16 = (36 \text{ ft} - 2 \text{ ft})/16 = 2.1 \text{ ft} = 26 \text{ in.} \text{ Extend the remainder outside two No.7 bars, the greater of the development length (51 in.) beyond the LQÅHFWLRQSRLQWIWLQ 60 in. + 27.5 in. = 87.5 in. > 75 in.$ Controls Therefore, extend bars minimum 87.5 in., increase to, say, 90 in. = 7 ft 6 in. from column face and extend the two exterior No. 7 bars the engineer may terminate the two interior No. 7 bars at a distance 4 ft 6 in. from column face and extend the two exterior No. 7 bars the full span length of the beam to support the shear reinforcement stirrups as shown in Fig. E1.11. American Concrete Institute - Copyrighted © Material - www.concrete.org 156 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) First span top bars 9.7.3.2 9.7.3.3 Interior support Following the same steps above, seven No. 7 bars are required to resist the factored moment at the 
$\frac{12}{20.85}(5 \text{ ksi})(18 \text{ in.})$ No. 7 bars the greater of the development length (51 in.) from column face and d from theoretical cutoff point (39 in.) 39 in. + 27.5 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; herefore, extend four No.7 bars 69 in. 9.7.3.2 9.7.3.3 n. > 12ddb = 10 10.5 5 in in. d = 27.5 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 12ddb = 10 10.5 5 in in. d = 27.5 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 12ddb = 10 10.5 5 in in. d = 27.5 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 in.; increase to 69 in. (5 ft 9 in.) 69 in. > 51 in. = 66.5 bars the larger of gth (51 in.) beyond the theoretical the development llength off ppoint int (38 in d d LQEH\RQGWKHLQÅHFcutoff in.)) and tion poi point (7 ft 6 in. = 990 in.). The latter controls (Fig. 10). E1.10). n. + 27.5 in. = 117.5 in. > 39 in. + 51 in. = 90 in. 90 in. OK ulat are performed to present the codee re ments In practice, the engineer may Note: These calculations requirements. eyond the development length from column terminate the four interior No. 7 bars at a distance d (27.5 in.) beyond o No.7 at 10 ft 0 in. from the support for a length of the beam to support the shear reinforcement stirrups as shown in Fig. E1.11. First span bottom bars )ROORZLQJWKHVDPHVWHSVDERYH¿YH1REDUV are required to resist the factored moment. x2 = 2(0.6 in.2)(0.9)(60 ksi) 2 (  $2(0.6 \text{ in.} 2)(60 \text{ ksi}) (1) \times 27.5 \text{ in.} - 27.5 \text{ in.}$ maximum positive moment at midspan (Fig. E1.10). Extend the remaining two No. 7 bars at least the longer of 6 in. into the column or Ed = 39 in. past the theoretical cutoff point (Fig. E1.10). American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS At least one-fourth of the positive tension bars must 7ZR1REDUVRXWRIWRWDO¿YH1RZLOOEHH[WHQGHG into the column: extend into the column at least 6 in. 7ZR1REDUV!¿YH1REDUV OK 9.7.3.8.3 3RLQWRILQÀHFWLRQRFFXUVDWIWIURPWKHFROXPQIDFH \$WWKHSRLQWRILQÀHFWLRQRFFXUVDWIWIURPWKHFROXPQIDFH \$WWKHSRLQWRILQÀHFWLRQWRILQÀHFWLRQHFYY VL]HVDWLV $\dot{c}$ HV Vu = 63.5 kip – (3.7 kip/ft)(4 ft) = 48.2 kip M Ad  $\leq$  n + Aa Vu At that location, assume two No. 7 bars are effective: where Mn is calculated at the section are stressed to fy. Vu is calculated at the section. At support,  $\dot{c}$  a is the embedment length beyond the center of the column. The term  $\dot{c}$  a is the HPEHGPHQWOHQJWKEH\RQGWKHSRLQWRILQÅHFWLRQ limited to the greater of d and 12db. 9.7.3.5 () M n = 2 0.6 in.2 (60 ksi) / × 27.5 in. – (2)(0.85)(5 ksi)(120in.) // (EDUVDUHFXWRIILQUHJLRQVRIÅH[XUDOWHQVLRQWKHQ Mn = 1982 in.-kip a bar stress discontinuity occurs. Therefore, the 1982 in.-kip Ad  $\leq$ 27.5 in. = 68.6 in., say, 69 in. EDUVPXVWQRWEH FRGHUHTXLUHVWKDWAH[XUDOWHQVLOHEDUVPXVWQRWEH 48.2 kip terminated in a tensile zonee unless (a), (b), or (c) is Thiss length exceed sceeds  $\mathcal{E}d$  = 39 in., therefore OK VDWLV¿HG t cutoff point (a) Vu" []Vn at the ovides double th (b) Continuing ba bars provides the area [X WKHFXWRIISRLQWDQGWK UHTXLUHGIRUÀH[XUHDWWKHFXWRIISRLQWDQGWKHDUHD [XU KHFXWRIISRLQWDQGWKHDUHD [XU KHFXWRIISRLQWDQGW ]] Vn . 1 ft and  $\epsilon_{n/2} = 1.17$  ft (a) At 10 3.7 kip/ft) (17 k (27.5 in.) 5 70 kip +  $\varphi$ Vn = 0.75 // 12 in. (c) Stirrup or hoop area in excess of that required for shear and torsion is provided along each terminated i i  $\square$  n = 75.2 kip  $\square$ V bar or wire over a distance 3/4d from the terminated i i  $\square$  n = 75.2 kip  $\square$ V bar or wire over a distance 3/4d from the terminated i i  $\square$  n = 75.2 kip  $\square$ V bar or wire over a distance 3/4d from the termination  $\square$ Vn = 2/3(75.2 kip) = 50 kip point. Excess stirrup or hoop area shall be at least 50 kip > 37.1 kip, therefore, OK. 60 bws/fyt. Spacing s shall not exceed d/(8ub). Because only one of the three conditions needs to be VDWLV¿HGWKHRWKHUWZRDUHQRWFKHFNHG Step 8: Integrity reinforcement 9.7.7.2 2QHRIWKHWZRFRQGLWLRQZDVVDWLV¿HGDERYHE\H[WHQGLQJWZRDUHQRWFKHFNHG Step 8: Integrity reinforcement 9.7.7.2 2QHRIWKHWZRFRQGLWLRQZDVVDWLV¿HG bars, but at least two bars, must be continuous. No. 7 bars into the support. Also, two bars are more than 1/4 of the provided reinforcement. 9.7.7.3 Beam longitudinal bars must be enclosed by closed stirrups along the clear span. 2SHQVWLUUXSVDUHSURYLGHGWKHUHIRUHWKHVHFRQG FRQGLWLRQZLOOQRWEHVDWLV¿HG Beam structural integrity bars shall pass through the region bounded by the longitudinal column bars. At least two No. 7 bars are extended through the column longitudinal reinforcement. Therefore, satisfying this condition. American Concrete Institute – Copyrighted © Material – www.concrete.org Beams 9.7.3.8.2 157 158 9.7.7.5 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Splices are necessary for continuous bars. The bars shall be spliced in accordance with (a) and (b): splice length = (1.3)(development length) (a) Bottom bars (positive moment) shall be spliced at or near the support Est = 1.3(39 in.) = 50.7 in., say, 4 ft 3 in. (b) Top bars (negative moment) shall be spliced at or near an Est = 1.3(51 in.) = 66.3 in., say, 5 ft 9 in. Fig. E1.10—End span bar cutoff locations. Note: Numbers shown in bold control the bar lengths. Step 9: Internal spans Flexural bars were calculated above in Step 5. Six No. 7 top bars are required at supports Four No. 7 bottom bars are required at midspan 9.7.6.2.2 Stirrup size and calculated following Step 6: No. 3 at 12 in. are not required over the full length of the beam, it is; however, good practice to maintain stirrups at 12 in. on center. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 159 Beams Step 10: Detailing Fig. E1.11—Beam bar details. Notes 30DFH¿UVWVWLUUXSDWLQIURPWKHFROXPQIDFH 2. The contractor may prefer to extend two No. 7 top reinforcement over the full beam length rather than adding two No. 5 hanger bars. Bars should be spliced at mid-length. American Concrete Institute – Copyrighted © Material – www.concrete.org 160 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E1.12—Sections. 1RWH5HIHUWR6WHS7DEOHIRUADQJHQHJDWLYHPRPHQWUHLQIRUFHPHQWSODFHPHQW American Concrete Institute - Copyrighted © Material - www.concrete.org 161 Beam Example 2: Single interior beam Design and detail a one-span Beam B1 built integrally with a 7 in. slab of a seven-story rames into girder Beam B2 at each end as shown in Fig. E2.1. Given: Load— Service dead load D = 15 psf Service live load L = 100 psf Material properties— fcq SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi Span length: 36 ft Beam width: 18 in. Fig. E2.1—Plan of one-span interior beam. American Concrete Institute -Copyrighted © Material – www.concrete.org Beams CHAPTER 7—BEAMS 162 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Material requirements of ACI 318-14. The designed determines the durability classes. Please refer to Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing \$&,LQWRMREVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing \$&,LQWRMREVSHFLµk \$&,LQW pozzolans, which the designer can require, permit, or review if suggested by the contractor. Step 2: Beam geometry Beam depth 9.3.1.1 ACI 318-14 permits a beam whose size satiscHV7DEOHWREHGHVLJQHGZLWKRXWKDYLQJ EHDPLVQRW WRFKHFNWKHEHDPGHAHFWLRQLIWKHEHDPLVQRW artitions or other supporting or attached to partitions EHGDPD FRQVWUXFWLRQOLNHO\WREHGDPDJHGE\ODUJHGHAHFHAHFW WLRQV2WKHUZLVHGHAHFWLRQVPXVWEHFDOFXODWHG QOLP DQGWKHGHAHFWLRQVPXVWEHFDOFXODWHG specifying that the concrete mixture shall be in accordance with ACI 301-10 and providing the exposure classes, Chapter 19 (ACI 318-14) requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 5000 psi. upported bbeam the recommended depth For a si simple supported fr m Ta 1 1: from Table 9.3.1.1: h = A (36 ft)(12)(12 in./ft) in./ft = 27 in. 16 16 Use 28 in. Self-weight Beam: bw = 18 in. Slab: t = 7 in. thick 9.2.4.2 6.3.2.1 wb = (18 iin./12)(0.150 kip/ft3) = 0.53 kip/ft ws = (2.375 ft)(7 in./12)(0.150 kip/ft3) = 0.21 kip/ft Tributary width = 0.21 kip/ft Tributa 4.75 ft/2 = 2.375 ft (Fig. E2.1) Flange width The beam is poured monolithically with the slab RQRQHVLGHDQGZLOOEHKDYHDVDQ/EHDP7KH HIIHFWLYHADQJHZLGWKRQRQHVLGHRIWKHEHDPLV obtained from Table 6.3.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 the least of | ] A n /12 (6)(7 in.) = 42 in. (4.75 ft)(12)/2 = 28.5 in. Controls (36) ft(12))/12 = 36 in. Flange width: bf =  $\epsilon n/12 + bw$  bf = 28.5 in. + 18 in. = 46.5 in. American Concrete Institute – Copyrighted © Material –
www.concrete.org 6WHS/RDGVDQGORDGSDWWHUQV The service live load for public assembly is 100 psf per Table 4-1 in ASCE 7-10. To account for the ZHLJKWRIFHLOLQJVSDUWLWLRQV+9\$&V\WHPVHWF add 15 psf as miscellaneous service dead load. 163 The superimposed dead load is applied over a tributary width of 4.75 ft/2 + 1.5 ft width of B1 = 3.875 ft (refer to Fig. E2.1). The beam resists gravity load only and lateral forces are not considered in this example. 5.3.1 U = 1.4(0.53 kip/ft + 0.21 kip U = 1.2D + 1.6L U = 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.6L U = 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.6L U = 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.6L U = 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.6L U = 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.6((100 psf)(3.875 ft)/1000) = 1.58 kip/ft Controls 1RWH/LYHORDGLVQRWUHGXFHGSHU = 1.20 + 1.2(1.12 kip/ft)/1.4 + 1.Chapter 6 permits several analysis procedures to calculate the required strengths. For this example, assume an elastic analysis results ment at the face of girders: Mu = wuE2/16 = (1.58 kip kip/ft)(36 ft)2/16 = 128 ft-kip en is wuE2/8, so the mid midspan pan moment m The total moment is wuE2/16. This distribution assumes the girder remains uncracked. If the girder does crack, its stiffness is greatly reduced, which results in a higher moment l at midspan. To be conservative, this example assumes the total beam moment, wuE2/8, is resisted by the positive moment reinforcement and the supports resist wuE2/16. At the midspan: Mu = wuE2/8 = (1.58 kip/ft)(36 ft)2/8 = 256 ft-kip Beam B1 frames into girders on both ends. Because girders or walls and girders VXSSRUWLQJFRQFHQWUDWHGORDGVPD\WHQGWRURWDWHWKHHQGVXSSRUWVPD\EHFRQVLGHUHGOHVVWKDQ¿[HGHQG VXSSRUWV)RUDVLQJOHVSDQEHDPZLWK& [HGHQGVXSSRUWVWKHQHJDWLYHPRPHQWDWKHVXSSRUWZRXOGEH (1/12)wu&2)RUWKLVFDVHWKHPRPHQWDFKLHYHGZRXOGKDYHWREHWUDQVIHUUHGWRWKHJLUGHUHI&FLHQWO,QUHDO terms, the girder may tend to slightly rotate or may endure cracking which would reduce the rigidity and the ¿[LW\RIWKHEHDPWR]LUGHUMRLQW7RDFFRXQWIRUWKLVDVVXPHDPRPHQWRI wuE2 to account for a lower FDSDFLW\WRWUDQVIHUPRPHQWRI wuE2 to account for a simple support beam. Sc conservatively, use a positive midspan moment of (1/8)wuE2. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 164 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E2.2—Shear and moment envelopes. American Concrete Institute - Copyrighted © Material - Www.concrete.org Beams CHAPTER 7—BEAMS 164 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E2.2—Shear and moment envelopes. www.concrete.org CHAPTER 7—BEAMS 165 Beams Step 5: Moment design 9.3.3.1 The code requires a beam to be designed with steel strain at design strength condition. For reinforced beams, such as this example, reinforcing bar strain is usually not a controlling issue. 21.2.1(a) Assume the beam will be tensioned controlled with DPRPHQWVWUHQJWKUHGXFWLRQIDFWRURI7 KLV assumption will be checked later. 20.6.1.3.1 Calculate effective depth assuming No. 3 stirrups, No. 6 longitudinal bars, and 1.5 in. cover: The effective depth assumption will be tensioned controlled with DPRPHQWVWUHQJWKUHGXFWLRQIDFWRURI7 KLV assumption will be checked later. 20.6.1.3.1 Calculate effective depth assuming No. 3 stirrups, No. 6 longitudinal bars, and 1.5 in. cover: The effective depth of one row of longitudinal reinforcement is d = h - cover - db, stirrups db,long/2 22.2.2.1 n at which nominal The concrete compressive strain moments are calculated is:: İcu = 0.003 22.2.2.2 JWK FUHWHLQAH[X DYDUL 7KHWHQVLOHVWUHQ]WKRIFRQFUHWHLQAH[X DYDUL 7KHWHQVLOHVWUHQ]WKRIFRQFUHWHQVLOHVWUHQ]WKRIFRQFUHWHQVLOHVWUHQ]WKRIFRQFUHWHQVLOHVWUHQ]WKRIFRQFUH strength, ACI 318 18 ne For calculating no nominal neglects sil strength. ngth. the concrete compressive stress distribution is inelastic at high stress. The Code permits any stress distribution to be assumed in design if shown to result in predictions of nominal strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive strength and is obtained from Table 22.2.4.3. For fcg SVL 22.2.2.4.1 22.2.2.4.3 d = 28 in. - 1.5 in. - 0.375 in. - 0.75 in./2 = 25.75 in., say, d = 25.7 in.  $\beta 1 = 0.85 - 0.05(5000 \text{ psi} - 4000 \text{ psi}) = 0.8 1000 \text{ psi}$  American Concrete Institute - Copyrighted © Material - www.concrete.org 166 22.2.1.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Find the equivalent concrete compressive depth, a by equating the compression force to the tension force within the beam cross section: C=T 0.85fcgba = Asfy At midspan 0.85(5000 psi)(b)(a) = As(60,000 psi) = 0.304 As 0.85(5000 psi)(46.5 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For negative moment: b = bf = 18 in. a = As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) At support For nin.) Fig. E2.3—Section reinforcement and compression block at midspan and at support. American Concrete Institute – Copyrighted © Material – www.concrete.org 167 'HVLJQWKHEHDPIRUWKHPD[LPXPÀH[XUDOPRPHQW at the midspan and the face of supports. Mu, support = wuE2/16 = 128 ft-kip Mu, midspan = wuE2/8 = 256 ft-kip 9.5.1.1 The beam strength must satisfy the following equations at each section along its length:  $[]Mn \cdot Mu []Vn \cdot Vu$  Calculate required reinforcement area based on the assumptions above: Midspan (0.9)(60 ksi) As ( 0.304 As ) 25.7 in. – | \ 12 2 | / a \ ( M u \le \varphi M n = \varphi As f y | d - | \ 2/ 256 ft-kip \le No. 6 bars db = 3/4 in. and As = 0.44 in.2 As, req'd = 2.24 in.2; use six No. 6 Supports 128 ft-kip  $\leq$  (0.9)(60 ksi) As 12 0.784 As ( | 25.7 in. - 2 | As = 1.13 in.2; use three No. 6 21.2.2 9.3.3.1 Check if calculated strain is greater than 0.005 in./in. (tension controlled), but not At midspan. a = As f y 0.85 f c'b and dc = a  $\beta$ 1 (0.304)(6) in.2) = 0.80 in. a = 0.30 0.304As = (0.304)(6)(0.44 a/0. in. = 0.304)(6)(0.44  $1.0\ 0$  in c = a/0.8 in. ZKHUHu1 = 0.8 WKHUHIRUHEHDPVHFWLRQEHKDYHVDVDQ/VKDSH HEHDPV c < hfWK  $\epsilon t = \epsilon c$  (d - c) c Note that b = 18 in. for positive moments.  $\epsilon t = 0.003$  (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHAt support: As f y a and c = a = 0.003 (25.7 iii. - 1.0 in.) = 0.074 0 in. 1.0 in.) = 0.074 0
in. 1.0 in.) = 0.074 0 in. 1.0 in.] 0.85 f c'b β1 a = 0.784As = (0.784)(3)(0.44 in.2) = 1.03 in. c = a/0.8 = 1.29 in. ZKHUHù1 = 0.8 εt = ε cu (d - c) c εt = 0.003 (25.7 in. - 1.29 in.) = 0.057 1.29 in. 7KHUHIRUHDVVXPSWLRQRIXVLQJ[LVFRUUHFW American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 168 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E2.4—Strain distribution across beam section. 9.6.1.1 9.6.1.2 Minimum reinforcement ratio The provided ratio The provided rei 000 psi Controls (b) As = 200 bw d fy As = 200 (18 in.) = 1.54 in.2 (iin.)(25.7)(60,000 \text{ psi } 60 \text{ Because fcg}:SVL(TD FRQWUROV (TD RQWUROV At mids midspan: 6)(0.44 in in.2) = 2.64 in.2 > As,min = 1.63 in.2 As,m support: As,prov (supp) = 1.76 in.2 > As(min) = 1.63 in.2 OK American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS []shear = 0.75 Beams Step 6: Shear design Shear strength 21.2.1(b) Shear strength reduction factor: 169 9.5.1.1 9.5.3.1 22.5.1.1 []Vn • Vu Vn = Vc + Vs 9.4.3.2 Design shear force is taken at the face of the support because the vertical reaction causes vertical tension rather than compression (Fig. E2.5). CondiWLRQF KRZHYHULVQRWVDWLV¿HGUHIHUWR note at end of this step). Vu = 28.5 kip 22.5.5.1 Vc = 2 f c'bw d Vc = 2 & KHFNLI\_Vc•Vu \_Vc Vu \_Vc = 2 & KHFNLI\_Vc•Vu \_Vc•Vu \_Vc\bulletVu \_Vc•Vu \_Vc\bulletVu (0.75)(65.4 kip) = 49 kip []Vc = 49 kip > Vu = 28.5 kip OK 9.6.3.1 () 5000 psi (18 in.)(25.7 in.)/1000 = 65.4 kip Minimum area of shear reinforcement ement is required in []V Vc Vu NLS! NLS! []Vc = 49 kip/2 = 24.5 kip all regions where Vu! []V weve good ood d engine i However, engineering practice calls for vidi minimum nimum she providing shear reinforcement over the full b m sp beam span. vide No. 3 stirrups at 12 in. on center where; Provide n. < d/2 = 25.7 in. /2 = 12.8 in. OK 12 in. 22.5.1.2): (Vu  $\leq \phi$  Vc + 8 f c'bw d) %\LQVSHFWLRQWKLVUHTXLUHPHQWLVVDWLV2HG Note: The shear could be taken at distance d from the face of the girder if hanger reinforcement is provided in the girder as outlined in a paper by A. H. Mattock and J. F. Shen, 1992, "Joints between Reinforced Concrete Institute – Copyrighted © Material – www.concrete.org 170 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Torsion Fig. E2.6—Forces transferred from slab to edge beam. Calculate the design load at face of slab to beam connection: 22.7.4.1(a) wu = 1.2(0.21 kip/ft + (15 psf)(2.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft Calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(9.375 ft)/1000) + 1.6(0.1 ksf)(4.75 ft)/2 wu = 0.71 kip/ft calculate the design unit torsion at beam center: tu = (0.76 kip/ft)(in./12) = 0.53 ft-kip/ft Design torsional force: old torsion Tth: Therefore, check threshold Tu = (0.5 (0.53 ft-kip/ft)(18 ft) = 9.6 ft-kip (Acp2) Tth =  $\lambda$  f c' | | pcp / in. = 21 in. hw = 28 in. - 7 in where Fig. E2.7—L-beam geometry to resist torsion. Fi Acp \*\* bihi is the area enclosed by outside perimeter Acp = (18 in.)(28 in.) + (21 in.)(7 in.) = 651 in.2 of concrete. pcp  $^{m}$ bi + hi)pc is the perimeter of concrete gross area. pcp = 2(18 in. + 21 in. + 7 in. + 21 in.) = 134 in. 2 7KHRYHUKDQJLQJADQJHGLPHQVLRQLVHTXDOWRWKH ( (651 in.2 ) 2 ) VPDOOHURIWKHSURMHFWLRQRIWKHEHDPEHORZWKHVODE Tth = (1.0) 5000 psi | 134 in. | (21 in.) and four times the slab thickness (28 in.). Tth = 223,636 in.-lb = 18.6 ft-kip Therefore, use 21 in. (refer to Fig. E2.7). (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] ( []Tth = (0.75)(18.6 ft-kip) = 14.0 ft-kip Therefore, use 21 in. (refer to Fig. E2.7). (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU]
(21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQUOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQUOVWUHQJWKUHGXFWLRQIDFWRU] (21.2.1c 7RUVLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHGXFWLRQUOVWUHQJWKUHQJWKUHQJWKUHGXFWLRQUOVWUHQJWKUHQJWKUHQJWKUHQJWKUHQJWKUHQXFWLRQUOVWUHQJWKUHQJWKUHQXFWLRQUOVWUHQJWKUHQXFWLRQUOVWUHQJWKUHQJWKUHQXFWLRQUOVWUHQJWKUHQ Step 8: Reinforcement detailing Minimum bar spacing 9.7.2.1 Minimum clear spacing between the horizontal No. 25.2.1 6 bars must be the greatest of {db | 4 / 3(d) agg [ 1 in. 3/4 in. 4/3(3/4 in.) = 1 in. Assume maximum aggregate size 3/4 in. Check if six No. 6 bars can be placed in the beam's web. Therefore, clear spacing between horizontal bars must not be less than 1 in. bw, req'd = 2(1.5 in. + 0.75 in.) + 3.75 in. + 5 in. bw, req'd = 2(cover + dstirrup + 0.75 in.) + 3.75 in. + 5 in. bw, req'd = 2(1.5 in. + 0.75 in.) + 3.75 in. + 3.75 in. + 3.75 in. + 3.75 in.) +spacing between bars (Fig. E2.8). Fig. E2 E2.8—Bottom ottom rei reinforcement layout. 9.7.2.2 24.3.1 24.3.2 Maximum bar spacing at the tension face must not exceed the lesser of ( 40, 000 )  $s = 15 | -2.5c \langle f s | / (40,000 psi ) s = 15 | -2.5c \langle f s | / (40,000 psi ) s = 15 | -2.5c \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (40,000 psi ) s = 12 | \langle f s | / (4$ maximum spacing concept is intended to OLPLWAH[XUDOFUDFNLQ]ZLGWKV1RWHWKDWcc is the FRQFUHWHFRYHUWRWKHAH[XUDOEDUVQRWWKHWLHV 18 in. spacing is provided, therefore OK American Concrete Institute – Copyrighted © Material – www.concrete.org Controls 172 9.7.3 9.7.1.2 25.4.2.2 25.4.2.4 9.7.1.3 25.5.2.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Bar cutoff Development length of a No. 6 bar: Ad = f y $\psi$ t  $\psi$ e 25 $\lambda$  f c' db Top ( (60,000 psi)(1.3)(1.0) Ad = | (0.75 in.) = 33.1 in. , (25)(1.0) 5000 psi ) say, 36 in. where zt EDUORFDWLRQzt = 1.3 for top bars, because more than 12 in. of fresh concrete is placed below Bottom WKHPDQGzt = 1.0 for bottom bars, because not more ((60,000 psi)(1.0)(1.0) than 12 in. of fresh concrete is placed below them. Ad = | (0.75 in.) = 25.4 in., (25)(1.0) 5000 psi / ze FRDWLQJIDFWRUze = 1.0, because bars are uncoated say, 30 in. Splice length of No. 6 reinforcing bar Per Table 25.5.2.1 splice length is ( $\epsilon st$ ) = 1.3( $\epsilon d$ ) Top tension reinforcement RLQWIRUQHJDWLYHPRPHQW diagram: iMmaxiwu(x)2/2 + Vux = 0 Top: 1.3 $\epsilon d$  = (1.3)(33.1 in.) = 43.0 in., say, 4 ft 0 in. Bottom: 1.3 $\epsilon d$  = (1.3)(25.4 in.) = 33 in., say, 3 ft 0 in. (-128 28 fftkip) kip)) - 1.58 1 58 kip/ft ki k x2 + (28.5 kip)x = 0.2 x = 5.3 ft, say, 5 ft 6 in. he girder face two No. 6 can be cutoff At 5.5 fft from the an the remainder der two N and No. 6 bars will be extended overr the full beam spa span to support stirrups. 9.7.3.8.4 At least one-third of the bars resisting negative moment at a support must have an embedment OHQJWKEH\RQGWKHLQÀHFWLRQSRLQWWKHJUHDWHVWRId, 12db, and En/16. Two of three No. 6 bars are extended over full beam length and one No. 6 bars is terminated EH\RQGWKHLQÀHFWLRQSRLQWDGLVWDQFHHTXDOWRWKH embedment length (27 in.). For No. 6 bars: d = 25.7 in. 12db = 12(0.75 in.) = 9 in. En/16 = (36 ft)(12)/16 = (36 ft 27 in. Controls 66 in. + 27 in. = 93 in. Therefore, the middle two No. 6 bar will be terminated at 7 ft 9 in. (93 in.) from face of support. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS Bottom tension reinforcement Bars must be developed at points along the span where bent or terminated tension bars are no longer required to UHVLVWAH[XUH Six No. 6 bars are required to resist the factored moment at the midspan. Beams 9.7.3.2 9.7.3.3 173 2 x =  $2(0.44 \text{ in.})(60 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist a factored moment } (0.9)(60 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist } (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist } (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist } (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist } (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist } (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / | (256 \text{ ft-kip}) - 1.58 \text{ kip/ft } 2.2 \text{ Two No. 6 bars can resist } (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / (0.9)(50 \text{ ksi}) / (\times | 25.7 \text{ in.} - \langle 2(0.85)(5 \text{ ksi})(46.5 \text{ in.}) / (0.9)(50 \text{ ksi}) / (0.9)(5$ located at a section x from midspan. 9.7.3.3 x = 14 ft = 168 in. A bar must extend beyond the point where it is no ORQJHUUHTXLUHGWRUHVLVWAH[XUHIRUDGLVWDQFHHTXDO For No. 6 bars: 1) d = 25.7
in. Controls to the greater of d or 12db. 2) 12db = 12(0.75 in.) = 9 in. Therefore, extend four No. 6 bars the greater of the development length (30 in.) from the maximum moment at midspan and a distance d = 25.7 in. from the theoretical th cutoff point. 3.7 in. say 16 ft-6 in. > d = 25.7 in. = 193.7 refo e extend four four N Therefore, No. 6 bars > 1/4 six No. = 125.7 in. = 125.7the support, but not less than  $\mathcal{E}d = 30$  in. from m the theoretical theo cutoff point (Fig. E2.7). Note: These calculations requirements. ulat are performed to present the codee re ments In practice, all longitudinal bottom bars are extended into the support rather than terminating them 1 ft 6 in. ffrom the support as shown by calculations. Step 9: Integrity reinforcement Integrity reinforcement 9.7.7.2 (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWEHVDWLV¿HGDERYHE\H[WHQGLQJWZR At least one-fourth the maximum positive moment No. 6 bars into the girders. bars, but not less than two bars must be continuous. 9.7.3.8.2 rth positive tenses tenses tenses tenses that be continuous. 9.7.3.8.2 rth positive tenses tens on bars A minimum of on one-fourth tension o the support minimum 6 in. must extend into /RQJLWXGLQDOEDUVPXVWEHHQFORVHG stirrups along the clear span of the beam. American Concrete Institute - Copyrighted © Material - www.concrete.org 174 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E2.9—End span reinforcement cutoff locations. Step 10: Detailing Fig. E2.10—Beam reinforcement details. American Concrete Institute - Copyrighted © Material - www.concrete.org 175 Beam Example 3: Single interior girder beam Determine the size of a one-span beam (B2) built integrally with a 7 in. slab of a seven-story building. The beam frames into two girder beams (B3) as shown in Fig. E3.1. Design and detail the beam. Given: Load— Service live load D = 15 psf Service live load D = 15 psf Service live load L = 100 psf Concentrated loads Pu = 28.5 kip (PD = 14.4 kip and PL NLS ORFDWHGIWLQVRXWKDQGQRUWKRI&ROXPQ/LQHV%DQG' respectively. (Refer to Example 2.) Material properties— fcg SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi Span length: 28 ft Beam width: 18 in. Fig. E3.1—Plan of Beam B2. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 176 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Material requirements 9.2.1.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements of ACI 318-14. The designer determines the durability classes. Please refer to Chapter 4 of SP-17 for an in-depth discussion of the Categories and Classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing \$&,LQWRMREVSHFL¿FDWLRQV Calculation By specifying that the concrete mixture shall be in accordance with ACI 301-10 and providing the expoVXUHFODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 5000 psi. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Step 2: Beam geometry Beam depth 9.3.1.1 Beam depth cannot be calculated using Table nto it (concen9.3.1.1, because two beams frame into trated loads). For framing simplicity, choose a beam deeper than the beams' ddepths, framing into it to allow for ease off cons construction and placement of n. reinforcement. Try:: h = 30 in. 24.2.2 9.2.4.2 6.3.2.1 FW LOOEHFKHFNHGDQGFRPSDUHG 7KHEHDPGHAHFWLRQZLOOEHFKHFNHGDQGFRPSDUHG to Table 24.2.2. Flange width oli with the slab on both sides for the remainder of the beam. At the maximum positive moment, the beam will behave DVDQ/EHDP7KHUHIRUHWKHHIIHFWLYHADQJHZLGWK on one side of the beam is the least from Table 6.3.2.1. 2QHVLGH ] 6hslab | of web is  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf =  $\frac{1}{2}$  sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} sw /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least of |] A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHADQJHZLGWKbf = \frac{1}{2} su /2 the least 2QERWKVLGHVRIWKHRSHQLQJWKHEHDPLVSODFHG monolithically with the slab and will behave as a 7EHDP7KHADQJHZLGWKRQHDFKVLGHRIWKHEHDP is obtained from Table 6.3.2.1. 6.3.1 Each side | J A n /8 8(7 in.) = 56 in. 7KHUHIRUHADQJHZLGWKbf = En/8 + bw + En/8 bf = 39.75 in. + 18 in. 39.75 in. = 97.5 in. (16.5 ft)(12)/2 = 99 in. ((28 ft)(12) - 18 in.)/8 = 39.75 in. American Concrete Institute - Copyrighted © Material - www.concrete.org Controls 6WHS/RDGVDQGORDGSDWWHUQV Self-weights of B2 Beam: beam width b = 18 in. 177 wb = [(18 in.)(30 in.)/(144)](0.150 kip/ft3) = 0.56 kip/ft3) 7ULEXWDU\ORDGEHWZHHQ\*LUGHU%DQG&ROXPQ/LQH 3 (refer to Fig. E3.2). The load is transferred to B2 WKURXJK%HDP%DORQJ&ROXPQ/LQH&VSDQQLQJ (15 ft 6 in. clear span). For lobbies and assembly areas, the uniform design live load is 100 psf per Table 4-1 in ASCE 7-10. To account for weights from ceilings, partitions, and +9\$&V/WHPVDGGSVIDVPLVFHOODOHRXVGHDG load. Fig. E3. E3.2—Beams B B1 B and B4 framing into B2. Dead load: hick) supported by Beam B4: Ps = (7 in./12)(14 Slab self-weight; i deep, ep. 18 in. wide by 300 in. Note: 7 in. is the slabthickness (118 in.) (30 in. - 7 in.) (16.5 ft - 1 ft) 3 PB = | | | | | | | (0.15 kip/ft) | 12 | | | 112 2 = 3.3 .3 kip Superimposed dead load at B2 midspan:  $\mathbb{P} = 10.1 \text{ kip} + 3.3 \text{ kip} + 1.7 \text{ kip} = 15.1 \text{ kip} / LYHORDG & RQFHQWUDWHGORDGEHWZHHQ&ROXPQ/LQHDQG girder$ transferred at midspan Beams B1 frame into Beam B2 at 6 ft-3 in. and IWLQIURP&ROXPQ/LQH%)LJ( The beams' factored reactions were calculated in Example 2 and were found to be: 28.5 kip in Step 4. 7KHEHDPUHVLVWVJUDYLW\ORDGRQO\/DWHUDOIRUFHV are not considered in this problem. PL = (0.1 ksf)(14 ft)(16.5 ft)/2 = 11.6 kip Pu =  $28.5 \text{ kip } 5.3.1 \text{ Distributed load: } wu = 1.4\text{D} 5.3.1\text{b} wu = 1.2\text{P} + 1.6\text{L} 5.3.1\text{b} wu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.4(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.4(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip }
5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ kip } 5.3.1\text{b} Pu = 1.2(15.1 \text{ kip}) = 21.1 \text{ k$ American Concrete Institute – Copyrighted © Material – www.concrete.org Beams CHAPTER 7—BEAMS 178 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Analysis Beam B2 is monolithic with supports. 9.4.1.2 Chapter 6 permits several analysis procedures to calculate the required strengths. For this example, calculate the required strengths. beam moment at VXSSRUWVXVLQJFRHI¿FLHQWVIURP7DEOH% Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: . aspx?ItemID=SP1714DA Mu = wuE2/12+PuE/8 + Pua2b/E2+Puab2/E2 and the analysis shows the beam shear is Vu = wuE/2 +Pu/2 RP\$SS 8VLQJFRHI¿FLHQWVIURP\$SSHQGL[%5HLQforced Concrete Design Handbook Design Aid - n be downloaded from: ncr g g/store/product p tail. . SP DA, assuming m ximum aspx?ItemID=SP1714DA, maximum id moment is at midspan: Mu = (0.94 kip/ft)(28 ft)/2 + (36.7 kip)(28 ft)/8 + (28.5 kip)(6.25 ft)/8 + (28.5 k  $f_{1}(21.75 \text{ ft})/(28 \text{ ft})^2 + (28.5 \text{ kip})(6.25 \text{ ft})(21.75 \text{ ft})^2/(28 \text{ ft})^2 = 328 \text{ ft-kip Nu} = (0.94 \text{ kip/ft})(28 \text{ ft})^2 + (28.5 \text{ kip})/2 = 60 \text{ kip Note that the } (2)(28.5 \text{ kip})/2 = 60 \text$ however, its stiffness is greatly reduced and redistribution of moments occurs. Assume that the moments at supports are reduced by 15 percent: Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." Mu = (0.85)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and its for a construction of moments at supports are reduced by 15 percent." is concluded that redistribution of design bending moments by up WRGRHVQRWUHVXOWLQSHUIRUPDQFHLQIHULRUWRWKDWRIEHDPVGHVLJQHGIRUWKHGLVWULEXWLRQRIEHQGLQJ moments predicted by the elastic theory, either at working loads or at failure." (Mattock, A. H., 1959, "Redistribution of Design Bending Moments in Reinforced Concrete Continuous Beams," Proceedings, Institution of Civil Engineers/RQGRQ 9SS H. Scholz, however, limits the moment redistribution to 20 percent is imposed to avoid excessive cracking at elastic service moments." (Scholz, H., 1993, "Contribution to Redistribution of Moments in Continuous Reinforced Concrete Beams," ACI Structural Journal91R Mar.-Apr., pp. 150-155.) American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS 179 Refer to the torsion diagram in Fig. E3.3. Beams B1 and B4 frame into Beam B2; therefore, assume that (B1) reaction of 28.5 kip and 36.7 kip are applied at the face of Beam B2, but in opposite directions. Ignoring the distributed load IURPWKHVODE%HDP%LVVXEMHFWHGWRWRUVLRQ From B1: Tu = (28.5 kip)(9 in.) = 27.6 ft-kip 12 Fig. E3.3—Shear, moment, and torsion diagrams. American Concrete Institute – Copyrighted © Material – www.concrete.org 180 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 5: Moment design 9.3.3.1 The Code does not allow a beam to be designed with steel strain at design strength less than 0.004 in./in. The intent is to ensure ductile behavior at the nominal condition. For usual reinforced beams, such as this example reinforcing bar strain is not a
controlling issue. 21.2.1(a) \$VVXPLQIWKHEHDPVZLOOEHWHQVLRQFRQWUROOHG0.9 [] This assumption will be checked later. 9.7.1.1 20.6.1.3.1 Calculate the effective depth assuming No. 3 stirrups, No. 6 bars, and 1.5 in. cover: d = h - cover - dtie - db/2 22.2.2.1 The concrete compressive strain at which nominal moments are calculated is: ic = 0.003 22.2.2.2 QFUHWHLQAH[XUHLVDYDUL 7KHWHQVLOHVWUHQ]WKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value iis approximately 10 to 15 percent of the concr concrete compressive strength. ength, ACI 318 neglects For calculating nom nominal strength, nsil strength. ngth. the concrete tensile d = 30 in. - 1.5 in. - 0.375 in. - 0. explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcoZLWKDGHSWKRI a "1c" 1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3. 22.2.2.4.1 For fcq SVL  $\beta$ 1 = 0.85 - 22.2.2.4.3 Find the equivalent concrete compressive depth, a, by equating the compressive depth, a set (5000 psi) (0.05(5000 1000 psi American Concrete Institute – Copyrighted © Material – www.concrete.org 22.2.1.1 181 As (60,000 psi) = 0.317 As 0.85(5000 psi)(44.5 in.) For positive moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. br positive moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. br positive moment: b = br = 44.5 in. a= For negative moment: b = br = 44.5 in. br positive moment Section reinforcement ent at midspan dspan and at ssupport. ort. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7-BEAMS 182 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) 'HVLJOWKHEHDPIRUWKHPD/LPXPAH/XUDOPRPHOW at the midspan and the face of supports. 9.5.1.1 The beam strength must satisfy the following inequalities at each section along its length:  $[Mn \cdot Mu \ Vu \ DOFXODWHUHTXLUHGAH[XUDOUHLQIRUFHPHQWDUHD using the following equation: Midspan 21.2.2 9.3.3.1 a) (Mu \le \varphi Mn = \varphi As fy | d - | (2/0.317 As) (304 ft-kip \le (0.9)(60 ksi) As | 27.7 in. - (2 |) No. 6 bars; db = 0.75 in. and As = 0.75 i$ 0.44 in.2 As = 2.47 in.2; use six No. 6 Note that b = 18 in. for positive moments and b = 44.5 in. for positive moments. Supports Check if calculated strain is greater than 0.005 in./ in. (tension-controlled), but not less than 0.004 in./ in. Refer to Fig. E3.5. a= As f y 0.85 f c b and c = ZKHUHů1 = 0.8  $\epsilon t = \epsilon cu (d - c) c a \beta 1 0.784 As (279 ft-kip \le (0.9))$ (60 ksi) As  $27.7 \text{ in.} - \sqrt{2}$  // As = 2.31 in.2XVH11R Midspan p 7 s = (0.3 (0.317)(6)(0.44)(6 (in.2) = 0.84 \text{ in.} a = 0.317 \text{ a}/0. = 0.084 \text{ in.} a = 0.317 \text{ a}/0. = 0.084 \text{ in.} a = 0.317 \text{ a}/0. = 0.084 \text{ in.} a = 0.317 \text{ a}/0. = 0.084 \text{ s} = (0 (0.784)(6)(0.44)(6 (in.2) = 0.84 \text{ in.} a = 0.317 \text{ a}/0. = 0.084 \text{ s} = (0 (0.784)(6)(0.44)(6 (in.2) = 0.84 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 > 0.005 1.05 \text{ in.} = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.003 \text{ o}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.034 \text{ s} = 0.0317 \text{ a}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.0317 \text{ a}/7.7 \text{ in.} - 1 (27.7 1.05 \text{ in.}) = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text{ s} = 0.076 \text in.2) = 2.07 in. a = 0.784A c = a/0.8 = 2.07/0.8 = 2.59 in. et = 0.003 (27.7 in. - 2.59 in.) = 0.029 > 0.005 2.59 in. 7KHUHIRUHWKHDVVXPSWLRQRI/LVFRUUHFW Fig. E3.5—Strain distribution across beam section. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS Minimum reinforcement area The reinforcement area must exceed the minimum required at every section along the length of the beam. (a) As = 35000 psi (18 in.)(27.7 in.) = 1.76 in.2 60,000 psi 200 bw d fy Because fco!SVL(TD FROWUROV Step 6: Shear design Shear strength 21.2.1(b) Shear strength reduction factor: 9.5.1.1 $[Vn \cdot Vu \text{ At midspan: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} =
1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ OK At support: As(prov.)} = 2.64 \text{ in.} 2 > \text{As(min)} = 1.76 \text{ in.} 2 \text{ otherwise and a beta at a support at a suppor$ causes ha compression mpression (Fig. E .5). C tension rather than E3.5). CondiR DUHVDWLV¿HG & RQG Fig. E3 Fi E3.6—Shear-critical ar-critical ar-criti &KHFNLI[Vn • Vu [Vc = (0.75)(70.5 kip) = 52.9 kip (Vc + 8 f c'bw d) (Vu ≤  $\varphi$  70.5 kip + 8 5000 psi(18 in.)(27.7 in.)) ≤ 212 kip Section dimensions are satisfactory. Note: The shear could be taken at distance d from the face of the girder if hanger reinforcement is provided in the girder, as outlined in a paper by A. H. Mattock and J. F. Shen, "Joints between Reinforced Concrete Institute – Copyrighted © Material – www.concrete.org 184 22.5.10.1 22.5.10.5.3 22.5.10.5.6 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Shear reinforcement Transverse reinforcement is required at each section where  $Vu \perp \Box Vc$  where  $Vs = \Box Vs \cdot NLS$  NLS Av f yt d  $Vs \ge s$  Av (8.5 kip)  $\ge = 0.0051$  s (60 ksi)(27.7 in.) Av  $V \ge s$  s f yt d 9.7.6.2.2 6.4  $kip = 8.5 kip 0.75 Check maximum allowable stirrup spacing: 4 f c bw d = 4 () 5000 psi (18 in.)(27.7 in.) = 140.5 kip say, 141 kip Is Vs \le 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? 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Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4 f c bw d ? Vs = 8.5 kip < 4$ /in. > 0.005 in.2 /in. || s || 12 in. pr prov OK 9.6.3.3 UH FHPHQWPXVWEH WOHDV 6SHFL¿HGVKHDUUHLQIRUFHPHQWPXVWEHDWOHDVWWKH larger of: As ,min s = 0.75 f c' bw Av , min f yt s psi  $\geq 0.75$  5000 ps 18 in. = 0.016 in.2 /in. 60,000 psi Controls and As ,min s = 50 bw f yt Av , min s = 50 18 in. = 0.015 in.2 /in. 60,000 psi Provided: Av , min Av 2(0.11 in.2)  $\ge = 0.018$  in.2 /in. > = 0.016 in.2 /in. > = Determine portion of slab to be included with the beam for the torsional design: T-section between a and b and between a and betwee pcp = 2(18 in. + 23 in. + 23 in. + 23 in. + 23 in. + 23 in. + 7 in.) = 188 in. () ( 862 in. 2 2 ) (1.0 5000 psi | Ttth = (1.0) | (188 in. |) 9.5.1.1 9.5.1.2 Refer to Fig. E3.3 for ttorsional value Tu near supports. 279,474 4 4 in. iin.-lb lb = 223.3 ft-kip Tth = 27 21.2.1 c JWK XFWLRQIDFWRU7 BUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU3 [ft ft-kip) kip)) = 17.5 ft-kip Th = ((0.75)(23.3 | th = 117.5 ft-kip p > Tu = 7.6 ft-kip OK ||T sion reinforcement or cement is not required between a and b Torsion een d and e. and between a d b Torsion een d and e. and between a d b Torsion een d and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b Torsion een d and e. and between a and b
Torsion een d and e. and between a and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. and b Torsion een d and e. a VPDOOHURIWKHSURMHFWLRQRIWKHEHDPEHORZWKHVODE (23 in.) and four times the slab thickness (28 in.). Therefore, use 23 in. (refer to Fig. E3.8). Fig. E3. ]/ Tth = 244,699 in.-lb = 20.4 ft-kip ()() []Tth = (0.75)(20.4 ft-kip) = 15.3 ft-kip Refer to Fig. E3.3 for torsional value Tu at midspan. []Tth = 15.3 ft-kip × Tu = 13.8 ft-kip ()() []Tth = (0.75)(20.4 ft-kip) = 15.3 ft-kip × Tu = 13.8 ft-k HANDBOOK-SP-17(14) Step 8: Reinforcement detailing Fig. E3.9-Longitudinal and transverse reinforcement. 9.7.2.1 Minimum top bar spacing From Appendix A of SP-17(14) Reinforced Concrete Design Handbook Design Aid - Analysis Tables, p g which can be downloaded from: six No. 6 bars can be placed in one layer within an 18 in. wide beam. 9.7.2.1 25.2.1 so be calculated as shown as follows forr bo Bar spacing can also bottom bars. Minimum bottom m bbar spacing cing g itudinal Minimum clear sp spacing db | 4/3(d) agg | 1 in. | Clear spacing cing g itudinal Minimum clear sp sp cing g itudinal Minimum clear sp sp cing g itudinal Minimum clear sp sp cing g itudinal Minimum clear sp sp cing g itudinal Minimum clear sp cing g itudinal Minimum clear sp cing g itudinal Minimum clear sp cing g itudinal Minimum clear sp cing g itudinal Minimum clear sp citud clear sp cing g itudinal Minimum clear sp cing g i between horizontal bars must not be less than 1.0 in. 0.75 in. (3/4 in.) = 1 in.  $4/3(3/4 \text{ Controls Check if six No. 6 bars can be placed in the beam's web. bw, req'd = 2(1.5 in. + 0.375 in.) + 5(1.0 in.)$ for reinforcement placement in Beam B2. Fig. E3.10 for reinforcement placement. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 9.7.1.2 Maximum bar spacing at the
tension face must not exceed the lesser of ( 40, 000 )  $s = 15 | -2.5cc \setminus f s | / (40,000 \text{ psi}) s = 15 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 | -2.5(2 \text{ in.}) = 10 \text{ in.} (40,000 \text{ psi}) s = 10 \text{ in.} (40$ || (40,000) s = 12 | (f s |) (40,000 psi) s = 12 | = 12 in. (40,000 psi) / 7KLVVSDFLQJLVWROLPLWÀH[XUDOFUDFNLQJZLGWKV where cc = 2 in. is the least distance from surface of 1.8 in. spacing is provided, therefore OK deformed reinforcement to the tension face. Development length of No. 6 reinforcing bar 7KHVLPSOL¿HGPHWKRGLVXVHGWRFDOFXODWHWKH Top bars development length of No. 6 bars: f ywt we 25.4.2.2 Ad = 25.4.2.2 ORZWKH FRQFUHWHLVSODFHGEHORZWKHPz e is coating factor; use bbars are uncoated DQGze = 1.0 because 9.7.1.3 25.5.2.1 9.7.3 Controls 20 $\lambda$  f c' db einforcing 2. splice ce length is 1.3(Per Table 25.5.2.1, 1.3(Ed). (60,000 psi)(1.3)(1.0) Ad = | (0.75 in.) = 33.1 in. (25)(1.0) 5000 psi / say, 36 in. Bottom bars ( (60, (60, 000 ((60, psi)(1.0)(1.0)) Ad = | (0.75 in.) = 25.5 in. (25)(1.0) 0 5000 00 psi / (25)(1 say, 30 in. i To 3)(33)(1 iin.) = 43.0 in., say, 4 ft 0 in. Top:: 1.3 1.3 Ed = (1.3)(25.5 in.) = 33.2 in., say, 3 ft 0 in. Bottom: Bar cutoff Bottom tension reinforcement Four No. 6 bottom bars are terminated beyondment Beam B1 a distance equal to the development length of 30 in. Four No. 6 lengths:  $\mathcal{E} = 14 \text{ ft} + 2(1.5 \text{ ft}) + 2(2.5 \text{ ft}) = 22 \text{ ft Extend two No. 6 bottom bars the full length of the beam and develop into the girder beams along & ROXPQ/LQHV%DQG'DWHDFKHQGZLWKDKRRN American Concrete Institute – Copyrighted © Material – www.concrete.org$ Beams 9.7.2.2 24.3.1 24.3.2 187 188 9.7.3.2 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Top tension reinforcement must be developed at points of maximum stress and points of maximum st the factored moment at the support. )RXU1REDUVZLOOEHWHUPLQDWHGDWWKHLQÀHFWLRQ point. 9.7.3.8.2 – (279 ft-kip) – 0.94 kip/ft x2 + 60 kip(x) = 0.2 x = 4.8 ft, say, 5 ft At least one-third of the negative moment reinforcement at a support must have an embedment OHQJWKEH\RQGWKHSRLQWKHJUHDWHVWRI For No. 6 bars: d, 12db, and En/16. 1) d = 27.7 in. Controls 2) 12db = 12(1.0 in.) = 12 in. 3) En/16 = 21 in. Therefore, extend four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. Place four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. Place four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. Place four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. Place four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. Place four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. = 67.7 in. Place four No. 6 bars a distance d beyond WKHLQÀHFWLRQSRLQW 60 in. + 27.7 in. = 67.7 in. = Section B) Step 9: Integrity reinforcement Integrity reinforcement rc t 9.7.2 H RQGLWLRQVPXVWE VDWLV (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWE VDWLV) (LWKHURQVPXVWE VDWLV) (LWKHU ,QWKLVH[DPSOHERWKDUHVDWLV¿HG At least one-fourth the maximum positive moment reinforcement, but not less than two bars must be continuous. 7KLV FR 7KLVFRQGLWLRQZDVVDWLV¿HGDERYHE\H[WHQGLQJWZR No. 6 bottom reinforcement bars into the support. 2 No. 6 > (1/4)6 No. 6 /ROILWXGLODOUHLOIRUFHPHOWPXVWEHHOFORVHGE\ closed stirrups along the clear span of the beam.
7KLVFROGLWLRQLVVDWLV2HGE\H[WHQGLQIVWLUUXSVRYHU the full length of the beam. Refer to Fig. E3.9. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS 189 24.2.3.1 Beams 6WHS'HÀHFWLRQ 9.3.2 &DOFXODWHGHÀHFWLRQOLPLW, PPHGLDWHGHÀHFWLRQLVFDOFXODWHGXVLQJHODVWLF GHÀHFWLRQDSSURDFKDQGFRQVLGHULQJFRQFUHWH cracking and reinforcement for calculating stiffness. Modulus of elasticity: 19.2.2.1 Ec = 57, 000 f c' psi 24.2.3.4 7KHEHDPLVVXEMHFWHGWRDIDFWRUHGGLVWULEXWHG force of 0.94 kip/ft or service dead load of 0.58 kip/ft and 0.15 kip/ft service live load. (19.2.2.1b) Ec = 57, 000 5000 psi /1000 = 4030 ksi 7KHEHDPLVDOVRVXEMHFWHGWRFRQFHQWUDWHGORDGV from Beam B1 at 6 ft 3 in. and 21 ft 9 in. from &ROXPQ/LQH%IUDPLQJLQWRLWKDYLQJWKHIROORZLQJ reaction; factored 28.5 kip or service live load of 14.4 kip and 7 kip service live load. Also, Beam B2 LVVXEMHFWHGWRDFRQFHQWUDWHGORDGDWPLGVSDQIURP Beam B4 of 36.7 kip factored orr 15.1 kip dead service load and 11.6 kip service live load. XDWLRQ 7KHGHÀHFWLRQHTXDWLRQIRUGLVWULEXWHGORDGZLWK GV & [LW\DWERWKHQGV ZE4/384EI ed load at mi For concentrated midspan: 3E3/192EI Note: w and P are service loads. q Ie is the effective moment of inertia given by Eq. (25.2.3.5a): 24.2.3.5 3 [ / M ]3 / M ] Ie = | cr | I g + | 1 - | cr | | I cr | Ma / | ] For simpl simplicity, assume that the beam is rectangular for the calculation of the moment of inertia (conservative): Ig = bh3 (18 in.)(30 in.)3 = 40,500 in.4) = 120 ft-kip (15 in.)(12,000) Ma is the moment due to service load. For moment calculation, refer to table below: )RUGHÀHFWLRQFDOFXODWLRQXVHPRPHQWVREWDLQHG IURPHODVWLFDQDO\VLV&RHI¿FLHQWVIURP7DEOH% Reinforced Concrete Design Aid - Analysis Tables, which can be downloaded from: . aspx?ItemID=SP1714DA are used to calculate the moments. The beam is assumed cracked; therefore, calculate the moment of inertia of the cracked section, Icr. American Concrete Institute - Copyrighted © Material - www.concrete.org 190 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Determine neutral axis of the cracked section: bc 2 + (n - 1) As' (c - d') 2 where c is the uncracked remaining concrete depth n = Es/Ec nAs (d - c) = For c values, refer to the table below n = 29,000 ksi/4030 ksi = 7.2 Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - d') 2 + nAs (d - c) 2 3 For Icr values, refer to the table below As 2 2 0.88 in. 2 2.64 in. 2 0.88 in. 2 2.64 in. 2 0.88 in. 2 0 kip 10 ft-kip Dead: 123 ft-kip 101 ft-kip 123 ft-kip 101 ft-kip 123 ft-kip 64 ft-kip 75 ft-kip 64 ft-kip 75 ft-kip 7

concentrated load at midspan and Ma = Pa2b/E2, where a is the distance of the concentrated load to the left support 24.2.3.6 )RUFRQWLQXRXVEHDPVRUEHDPVc[HGDWERWKHQGV RWK HQGV (positive and negative moments), the Code permits to take Ie as the average of values obtained from Eq.(24.2.3.5a) for the critical positive and negative moments. I e, avg = I e, left @ supp + I e, right @ supp + I e, right @ supp + I e, right @ supp + I e, avg = 9200 in.4 + 15, 400 in.4 + 9200 in.4 + 9 0.7le,midspan + 0.15(Ie,[email protected] + Ie,[email protected]) Ie,avg = 0.7(15,400 in.4 + 0.15(9200 in.4 + 9200 in.4) Ie,avg = 13,540 in.4 American Concrete Institute - Copyrighted © Material - www.concrete.org 191, PPHGLDWHGHÀHFWLRQV 'HÀHFWLRQV 'HÀ (12)3 = 0.04 in. 384(4030 ksi)(11, 267 in.4) 'HAHFWLRQGXHWRWRWDOFRQFHQWUDWHGORDG PD+L = 14.4 kip + 7 kip = 21.4 kip  $\Delta$  C . L. = (26.7 kip)(28 ft)3 (12)3 = 0.12 in. 192(4030 ksi)(11, 267 in.4) At midspan: 'HAHFWLRQDWPLGVSDQGXHWR%DWIWLQDQG IWLQIURP&ROXPQ/LQH% PD+L = 14.4 kip + 7 kip = 21.4 kip  $\Delta$  C . L. = (26.7 kip)(28 ft)3 (12)3 = 0.12 in. 192(4030 ksi)(11, 267 in.4) At midspan: 'HAHFWLRQDWPLGVSDQGXHWR%DWIWLQDQG IWLQIURP&ROXPQ/LQH% PD+L = 14.4 kip + 7 kip = 21.4 kip  $\Delta$  C . L. = (26.7 kip)(28 ft)3 (12)3 = 0.12 in. 192(4030 ksi)(11, 267 in.4) At midspan: 'HAHFWLRQDWPLGVSDQGXHWR%DWIWLQDQG IWLQIURP&ROXPQ/LQH% PD+L = 14.4 kip + 7 kip = 21.4 kip  $\Delta$  C . L. = (26.7 kip)(28 ft)3 (12)3 = 0.12 in. 192(4030 ksi)(11, 267 in.4) At midspan: 'HAHFWLRQDWPLGVSDQGXHWR%DWIWLQDQG IWLQIURP&ROXPQ/LQH% PD+L =  $14.4 \text{ kip} + 7 \text{ kip} = 21.4 \text{ kip} \Delta$  C . L. = (26.7 kip)(28 ft)3 (12)3 = 0.12 in. 192(4030 ksi)(11, 267 in.4) At midspan: 'HAHFWLRQDWPLGVSDQGXHWR%DWIWLQDQG IWLQIURP&ROXPQ/LQH% PD+L =  $14.4 \text{ kip} + 7 \text{ kip} = 21.4 \text{ kip} \Delta$  C . L. = (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 = 0.12 in. (26.7 kip)(28 ft)3 (12)3 (12)3 (12)3 (12)3 (12)3 (12)3 (  $2(21.4 \text{ kip})(6.25\text{ft}) 2 (14 \text{ ft}) = 0.08 \text{ in. Equation is obtained from: https:// www.concrete.org/store/productdetail.aspx?ItemID=SP1714DA: 24.2.2$ 7RWDOGHÀHFWLRQ VWWKHPD[LPXPDOORZ &KHFNWKHGHÀHFWLRQD]DLQVWWKHPD[LPXPDOORZ.2.2: able limits in Table 24.2.2: "D+L = 0.04 in. + 0.12 in. + 0.08 in. = 0.24 in. UWLQ DWWDFKHGWRQRQ XFWXUDO )RUÀRRUVXSSRUWLQ]RUDWWDFKHGWRQRQVWUXFWXUDO ot likely y to be damaged by y large elements and not 0 GHAHFWLRQE/480 hallo eam could have been selected. But A shallower beam be ause of Beam m B1 (28 in. deep) that is framing into because it, a 30 in. deep beam was chosen. in./ft)/480 t)/4 ) = 0.7 in. "all. = (28 ft)(12 in LQ!!" BEAMS 192 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) /RQJWHUPGHAHFWLRQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHHQWRWDOGHAHFWLRQV (HAHFWLRQDQGGHDGORDGGHAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHHQWRWDOGHAHFWLRQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHHQWRWDOGHAHFWLRQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHHQWRWDOGHAHFWLRQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHHQWRWDOGHAHFWLRQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHHQWRWDOGHAHFWLRQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHIQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHIQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHIQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH EHWZHIQV 'HAHFWLRQVGXHWROLYHORDGDUHWKHGLIIHUHQFH ksi)(11, 267 in.4) Concentrated load at midspan:  $\Delta$  conc = (15.1 kip)(28 ft)3 (12)3 = 0.07 in. 192(4030 ksi)(11, 267 in.4) Concentrated load at 6.25 ft and 21.75 ft, respectively.  $\Delta$  C . L = 7RWDOGHDGORDGGHÅHFWLRQ "D = 0.04 in. + 0.05 in. = 0.16 in. + 0.05 in. = 0.16 in. + 0.05 in. = 0.16 in. + 0.07 in. &DOFXODWHORQJWHUPGHÀHFWLRQ  $λΔ = 24.2.4.1.3 2(14.4 \text{ kip})(6.25 \text{ ft})(14 \text{ ft}) = 0.05 \text{ in.} \xi 1 + 50 p' λΔ = 2.0 = 1.84 0.88 \text{ in.} \text{ in } 2 1 + 50 (18 8 \text{ in.})(27.7 \text{ in.}) 2. the time-dependent factor ad duration tion of the time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad duration time-dependent factor ad du$ more than tha 5 ye for sustained load years: h /RQJWHUPGHAHFWLRQGXHWRVXVWDLQHGORDGLV "T 3" "i %\LQVSHFWLRQWKLVLVVDWLV2HG %\LQVSHFWL
American Concrete.org CHAPTER 7—BEAMS 193 Beams Step 11: Detailing Fig. E3.11—Beam reinforcement details. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS 193 Beams Step 11: Detailing Fig. E3.11—Beam reinforcement details. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS 193 Beams Step 11: Detailing Fig. E3.11—Beam reinforcement details. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS 193 Beams Step 11: Detailing Fig. E3.11—Beam reinforcement details. Concrete Institute – Copyrighted © Material – www.concrete.org 194 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Beam Example 4: Continuous six-bay edge beam built integrally with a 7 in. slab on the exterior of the building. Design DQGGHWDLOWKHEHDP, JQRUHRSHQLQJVDW&ROXPQ/LQHVDQG Given: fcg SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi Beam width: 18 in. Beam height: 30 in. six span perimeter beam. ACI 318-14 Discussion Step 1: Material requirements 9.2.1.1 The mixture proportion must satisfy the durability. requirements of Chapter 19 and structural strength requirements of ACI 318-14. The designer determines the durability classes. Please refer to Chapter 4 of SP-17 for an in-depth discussion of the Categories and Classes. Calculation By specifying that the concrete mixture shall be in accordance with ACI 301-10 and providing the expoVXUHFODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL expoVXUHFODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV?HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL expoVXUHFODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV?HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL expoVXUHFODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV?HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL experience with local mixtures, the compressive vertex experience with local mixtures, the compressive vertex experience with local mixtures, the compressive vertex experience with local mixtures, the compressive vertex experience with local mixtures, the compressive vertex experience with local mixtures experience with loca referencing \$&,LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. American Concrete Institute – Copyrighted © Material – www.concrete.org Step 2: Beam geometry 9.3.1.1 Beam depth ,IWKHGHSWKRIDEHDPVDWLV¿HV7DEOH\$&, 318 permits a beam design without having to FKHFNGHÀHFWLRQVLIWKHEHDPLVQRWVXSSRUWLQJRU attached to partitions or other construction likely to EHGDPDJHGE\ODUJHGHÀHFWLRQV2WKHUZLVHEHDP GHÀHFWLRQVPXVWEHFDOFXODWHGDQGWKHGHÀHFWLRQ OLPLWVLQPXVWEHVDWLV¿HG Self-weight Beam: Slab:  $9.2.4.2 \ 6.3.2.1 \ 195 \ The beam has six continuous spans.$  Taking the controlling condition of having one end continuous:  $h=A=(36 \ ft)(12 \ in./ft) = 23.4 \ in. 18.5 \ Use \ 30 \ in. wb = [(14 \ ft - 15 \ in./12)/2)(7 \ in./12)](0.150 \ kip/ft) = 0.56 \ kip/ft$ Facade: assume facade weight is 35 psf spanning wcladding =  $(35 \text{ psf})(12 \text{ ft})/1000 = 0.42 \text{ kip/ft } 12 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 6.3.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } | A n /12 2 \text{ ft-0 in. vertically Flange width The beam is obtained from Table } 8.2.1. 2QHVLGH ] 6hslab | of web is } sw /2 \text{ the least of } 8.2.1. 2QHVLGH ] 8.2.1. 2QH$ (6)(7 in.) = 42 in. [((14 ((14 ft)(12) - 15 in.)/2 = 76.5 in. (34 ft)(1 ft)(12)/12 Controls 2)/12 = 34 in. in Flange width: bf En/12 + bw 6WHS/RDGVDQGORDGSDWWHUQV QV OR SVILQRI¿FHVDQG 10. This psf in corridors per Tablee 4-1 in ASCE 77-10. example will use 65 psf as an average aas thee actual layout is not provided. To account for the weight of ceilings, partitions, DQGPHFKDQLFDO+9\$& V/VWHPVDGGSVIDV miscellaneous dead load. The beam resists gravity load only and lateral forces are not considered in this problem. bf = 34 in. + 18 8 in. = 52 in. 5.3.1 U = 1.40 wu = 1.4(0.56 kip/ft + 0.42 kip/ft + 0.42 kip/ft + (15 psf)((14 ft)/2)/1000)= 2.3 kip/ft U = 1.2D + 1.6L wu = 1.2(2.3 kip/ft)/1.4 + 1.6(65 psf/1000)(14 ft)/2 = 2.7 kip/ft Controls American Concrete Institute – Copyrighted © Material – www.concrete.org Beams CHAPTER 7—BEAMS 196 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Analysis 9.4.3.1 The beams are built integrally with supports; therefore, the factored moments and shear forces (required strengths) are calculated at the face of the supports. 9.4.1.2 6.5.1 Clear span: En = 36 ft – 2 ft = 34 ft Chapter 6 permits several analysis procedures to calculate the required strengths. The beam required strengths can be calculated at the face of the supports. 9.4.1.2 6.5.1 Clear span: En = 36 ft – 2 ft = 34 ft Chapter 6 permits several analysis procedures to calculate the required strengths. are prismatic Beams are prismatic /RDGVXQLIRUPO\GLVWULEXWHG L"D 6DWLV¿HGQRFRQFHQWUDWHGORDGV 65 psf < 3(87.5 psf + 15 psf + beam self-weight Three spans minimum 6 spans > 3 spans Difference between two spans does not exceed 20 percent. Beams have equal clear span lengths 34 ft-0 in. OK \$OO¿YHFRQGLWLRQVDUHVDWLV¿HGWKHUHIRUHWKHDSSUR[L\$OO¿YH mate procedure is used. 6.5.4 6.5.2 Fig. E4.2—Shear and moment diagrams. 6.5.3 Note: The moments calculated using the approximate method cannot be redistributed in accordance with Section 6.6.5.1. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 197 Beams The slab load is eccentric with respect to the edge beam center. Therefore, the beam needs to resist a torsional moment (Fig. E4.3). Fig. E4.3—Torsion forces. /RDGDWVODEEHDPLQWHUIDFH wu = [((1.2)((7 in./12)(0.15 ksf)) + (1.6)(0.065 ksf))][(14 ft - 1.5 ft/2)/2 + 3)in./12] wu = 1.56 kip/ft ng the beam length is (Fig. E4.4): The torsional moment along kip/ft)(18 in./2/12) = 1.17 ft-kip/ft)(34 ft)/(18 in./2/12) = 1.17 ft-kip/ft)(34 ft)/( www.concrete.org 198 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Moment design 9.3.3.1 The Code does not permit a beam to be designed with steel strain less than 0.004 in./in. at design strength. The intent is to ensure ductile behavior at the factored condition. In most reinforced beams, such as this example, reinforcing bar strain is not a controlling issue. 21.2.1(a) The design assumption is that beams will be WHQVLRQFRQWUROOHG7 KLVDVVXPSWLRQZLOOEH checked later. 9.7.1.1 20.6.1.3.1 Determine the effective depth assuming No. 4 stirrups, No. 6 bars, and 1.5 in. cover: 2QHURZRIUHLQIRUFHPHQW d = h - cover - dtie - db/2 22.2.2.1 d = 30 in. - 1.5 in. - 0.5 in. - 0.75 in./2 = 27.6 in. The concrete compressive strain at nominal moment strength is: Icu = 0.003 22.2.2.2 QFUHWHLQAH[XUHLVDYDULable property and is approximately 10 to 15 percent mpre of the concrete compressive strength. ACI 318 cret tensilee strength to calculating ulating neglects the concrete th. nominal strength.
22.2.2.3 ent concrete com ressi Determine the eq equivalent compressive stress distribution is inelastic at high stress. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRI 22.2.2.4.3 22.2.1.1 a \u00fclosel 1.22.2.4.3 22.2.1.1 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fclosel 1.22.2.2.4.3 a \u00fcl obtained from Table 22.2.2.4.3.  $\beta 1 = 0.85 - = 0.81000$  psi For fcg"SVL Find the equivalent concrete compression force to the tension force within the beam cross section (Fig. E4.5): C=T 0.85fcgba = Asfy 0.85(5000 psi)(b)(a) = As(60,000 psi) For positive moment: b = bf = 52 in. a = As (60, 000 psi) = 0.271As 0.85(5000 psi)(52 in.) For negative moment: b = bw = 18 in. a = As (60, 000 psi) = 0.784 As 0.85(5000 psi)(18 in.) American Concrete Institute - Copyrighted © Material - www.concrete.org 199 Beams CHAPTER 7—BEAMS Fig. E4.5—Section compressive block and reinforcement at midspan and at support. 7KHEHDPLVGHVLJQHGIRUWKHPD[LPXPAH[XUDO moments obtained from the approximate method above. 7KH¿UVWLQWHULRUVXSSRUWZLOOEHGHVLJQHGIRUWKH larger of the moments on eithe either side of the column; 9.5.1.1 n stre The beam design strength must be at least the h aat each h section along its length required strength (Fig. E 4.6):  $[Mn \cdot Mu = \varphi As f y | d - | \langle 2 \rangle$  Each No. 6 bar has a db = 0.75 in. and an As = 0.44 in.2 21.2.2 9.3.3.1 Check if the calculated strain exceeds 0.005 in./in. (tension-controlled—Fig. E4.7) but not less than 0.004 in./in. As fy a and c = a = f b  $\beta$  0.85 c' 1 ZKHUHù1 = 0.8 (calculated above) Note that b = 18 in. for negative moments and 57.75 in. for positive moments (refer to Fig. E4.5).  $\epsilon t =$  Number of No. 6 bars Mu, ft-kip As, req'd, in.2 Req'd Prov. MU1 195 1.59 3.6 4 MU2 312 2.6 5.91 6 MT 5.36 6 284 2.36 ME + 223 1.81 4.11 5 MI+ 195 1.57 3.57 4 Table 4.2—Strain in tension bars Mu, ft-kip As, prov, in.2 a, in. İs, in./in İs > 0.005? MU1 195 1.76 1.38 0.045 Y MU2 312 2.64 2.07 0.029 Y MT 284 2.64 2.07 0.029 Y ME + 223 2.20 0.60 0.108 Y MI+ 195 1.57 3.57 4 Table 4.2—Strain in tension bars Mu, ft-kip As, prov, in.2 a, in. İs, in./in İs > 0.005? MU1 195 1.76 1.38 0.045 Y MU2 312 2.64 2.07 0.029 Y MT 284 2.64 2.07 0.029 Y ME + 223 2.20 0.60 0.108 Y MI+ 195 1.57 3.57 4 Table 4.2—Strain in tension bars Mu, ft-kip As, prov, in.2 a, in. İs, in./in İs > 0.005? MU1 195 1.76 1.38 0.045 Y MU2 312 2.64 2.07 0.029 Y MT + 223 2.20 0.60 0.108 Y MI+ 195 1.57 3.57 4 Table 4.2—Strain in tension bars Mu, ft-kip As, prov, in.2 a, in. İs, in./in İs > 0.005? MU1 195 1.76 1.38 0.045 Y MU2 312 2.64 2.07 0.029 Y ME + 223 2.20 0.60 0.108 Y MI+ 195 1.57 3.57 4 Table 4.2—Strain in tension bars Mu, ft-kip As, prov, in.2 a, in. İs, in./in İs > 0.005? MU1 195 1.76 1.38 0.045 Y MU2 312 2.64 2.07 0.029 Y MT + 223 2.20 0.60 0.108 Y MI+ 195 1.57 3.57 4 Table 4.2 1.76 0.48 0.136 Y 7KHUHIRUHDVVXPSWLRQRIXVLQJ[LVFRUUHFW ε cu (d - c) c American Concrete Institute - Copyrighted © Material - www.concrete.org 200 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E4.7—Strain distribution across beam section. 9.6.1.1 9.6.1.2 Minimum reinforcement The reinforcement area must be at least the minimum required reinforcement area at every section along the length of the beams. As = 3 f c' fy bw d Equation (9.6.1.2a) 2a) controls 60,000 psi All calc calculated reinforcement areas exceed the m imu required red reinforminum reinforcement area. Therefore, OK American Concrete Institute - Copyrighted © Material - www.concrete.org 201 External spans are relatively equal: 53 kip versus 46 kip; therefore, the continuous beam will be designed for 53 kip of shear force 9.4.3.2 9.5.1.1 9.5.3.1 22.5.1.1 22.5.1.1 22.5.1.1 Because conditions (a), (b), and (c) of 9.4.3.2 are VDWLV¿HGWKHGHVLJQVKHDUIRUFHFULWLFDOVHFWLRQLV taken at a distance d from the face of the support (Fig. E4.8) The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must satisfy: []Vn • 9u Vn = Vc + Vs Fig. E4.8] The controlling factored load combination must sati kip/ft)(27.6 in./12) = 47 kip Vc = (2) 5000 psi(18 in.)(27.6 in.) = 70.3 kip Vc = 2 f c'bw d 21.2.1(b) Shear strength reduction factor:  $\varphi Vc = \varphi 2 f c'bw d []Vc = 52.7 refo shear hear reinfo Therefore, reinforcement is not required. 9.6.3.1 Code$ requires that ha minimum mum shear rein reinforcement rcement NLS! V = 1/2(52.7 kip) = 26.4 kip ed over sectional ssonal dimensions satisfy atisfy Eq. (22.5.1.2): 22.5.1.2 Vu  $\leq \varphi(Vc + 8 \text{ f c'bw d})$  herefore, provide minimum shear reinforcement rcement NLS! V = 1/2(52.7 kip) = 26.4 kip ed over sectional ssonal dimensions satisfy atisfy Eq. (22.5.1.2): 22.5.1.2 Vu  $\leq \varphi(Vc + 8 \text{ f c'bw d})$  herefore, provide minimum shear reinforcement rcement NLS! over be length. (Refer to torsion calculation Step 7). full beam %\LQVSHFWLRQWKLVFRQGLWLRQLVVDWLV2HGDQGVHFWLRQ dimensions are satisfactory. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 202 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Torsion design Torsion design 9.4.4.3 Calculate the torsional moment at d from the face of the support: tu = 1.17 ft-kip/ft and Tu = 20 ft-kip (27.6 in.)/12 = 17.3 ft-kip/ft (27.6 in.)/12VPDOOHURIWKHSURMHFWLRQRIWKHEHDPEHORZWKHVODE (23 in.) and four times the slab thickness (28 in.). Therefore, 23 in. controls (Fig. E4.9). 22.7.4.1 Calculate the threshold torsion value: (Acp2) Tth =  $\lambda$  f c' | | \ Pcp / Fig. E4.9. L-beam geometry to resist torsion. where Acp is the area enclosed by outside perimeter of concrete cross section; and Acp = (1 (18 in.)(30 in.) + (23 in.)(7 in.) = 701 in.2 meter of o concrete cross section pcp = 2(18 in. + 2(22(23 in.) n + 7 in.) = 142 in. pcp is the outside perimeter ((701 in.2) 2) 5000 = 20.4 ft-kip ((1000 psi | Tth = (1.0) (1200 pcm) = 142 in. pcp is the outside perimeter ((701 in.2) 2) 5000 = 20.4 ft-kip ((1000 psi | Tth = (1.0) (1200 pcm) = 142 in. pcm) = 142 in. pcmft-kip) = 15.3 ft-kip 9.5.4.1 an be ignored; gnored; gnored; gnored; gnored; gnored; gnored of torsion can LS ! []T Tth = 15.3 ft-kip [email protected] NLS![]T NG ional effects effect cannot be neglected and reinforceTorsional
ment and detailing requirements for torsion must be considered. American Concrete Institute – Copyrighted © Material - Copyrighted © Material - Copyrighted Concrete Institute – Copyrighted Concrete Inst www.concrete.org CHAPTER 7—BEAMS 22.7.3.2 Torsion reinforcement Calculate cracking torsion: (Acp2) Tcr =  $4\lambda$  fc' | | Pcp / Beams 9.5.1.1 9.5.4.2 22.7.5.1 203 ((701 in.2) 2) = 81.5 ft-kip Tcr = 4(1.0) 5000 psi | 142 in. | Check if cross section will crack under the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip < Tcr = 87 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducing Tu to Tcr is not constructed and the torsional moment. Tu = 17.3 ft-kip Reducin required. OK Fig. E4.10—E4.10—Aoh area. 2[(8 in in. - 2(1.5 2(1 in iin.) - 0.5 in.)] = 82 in. (3 in. - 2(1.5 in (14.5 in.)(26.5 26 5 in in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in.))] = 82 in. (3 in. - 2(1.5 in oh 2 () (17.3 ft-kip)(12 × 103)(82 in.) 46,000,000 lb + (18 // 8 in.)(2 in) (27.6 1.7(384.25 in.2) 2 where ph is the perimeter of centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the perimeter of centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the perimeter of centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement; and Aoh is the area enclosed by centerline of the outermost closed transverse torsional reinforcement  $b \le (0.75)$  + 8 5000 psi | (18 in.)(27.6 in.) / 115 psi < 530 psi OK Therefore section is adequate to resist torsion. Calculate required transverse: Tn = 2 Ao At f yt s cot  $\theta$  (22.7.6.1a) Tu @ d  $\varphi$  = 2(327 in.2) At (60 ksi) (17.3 ft-kip)(12)  $\le$  Tn = cot 45 0.75 s At/s = 0.007 in.2/in. /RQJLWXGLQDO Tn = 2 Ao AA f y ph cot  $\theta$  (22.7.6.1b) Tu @ d  $\varphi$  = 2(327 in.2) AA (60 ksi) (17.3 ft-kip)(12)  $\leq$  Tn = cot 45 0.75 82 in. ZKHUH"  $\varsigma$ "XVH $\varsigma$  GHJUHHV AE •LQ2 Ao = 0.85(384.25 in.2) = 327 in.2 22.7.6.1.1 American Concrete Institute – Copyrighted © Material – www.concrete.org 204 9.5.4.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The required area for shear and torsional transverse reinforcement are additive: Av + t = 0 in.2 / in. + 2(0.007 in.2 / in. ) = 0.014 in.2 / in. + 2(0.007 in.2 / in. ) = 0.014 in.2 / in. + 2(0.007 in.2 / in. ) = 0.014 in.2 / in. + 2(0.007 in.2 / in. ) = 0.014 in.2 / in. + 2(0.007 in.2 / in. ) = 0.014 in.2 / in. = 0.014
in.2 / in. = 0.014 in.2 / in. = 0 &DOFXODWHWKHPD[LPXPVSDFLQJRIVWLUUXSVDWd from the column face. 9.7.6.3.3 9.6.4.2 0D[LPXPVSDFLQJRIWUDQVYHUVHWRUVLRQDOUHLQIRUFH Assume No. 4 stirrup ment must not exceed the lesser of ph/8 and 12 in. ph = 82 in. calculated above ph/8 = 82 in./8 = 10 in. < 12 in.; use 10 in. Check maximum transverse torsional reinforcement: (Av + 2At)min/sPXVWEHJUHDWHUWKDQ 0.75 fc' (Av + t) min 18 in. = 50 = 0.016 in. SVL s bw fyt use calculations showed that at shear Av = 0 in.2 because quired. Minimum shea reinforcement is not required. Solve fyt use calculations showed that at shear Av = 0 in.2 because quired. Minimum shea reinforcement is not required.UHLQIRUFHPHQWLVKRZHYHUSURYLGHG n2 At = (2)(0.20 in.2) = 0.4 in. 8VHWZROHJVIRUWRUVLRQDOUHLQIRUFHPHQW Provided: ded: Av + t (A) = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in. = 0.04 in. 2/in. = 10 s Av + t (A) = 0.04 in.f c'Acp fy fy (A) - |t| ph (s) fy AEPLQ = 55000 psi(701 in.2) (25(18 in.) (SVL NVL = 3.5 in.2 AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (25bw) fyt - ph |fy (fyt) AEPLQ = 5 f c'Acp fy (fyt) AEPLQ7KHORQJLWXGLQDOUHLQIRUFHPHQWPXVWEHDGGHGWRWKH AH[XUDOUHLQIRUFHPHQW American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS 9.7.5.1 9.7.5.2 Torsion longitudinal reinforcement, AE, must be distributed around the cross section and the portion of AE that needs to be placed where As is needed is added to As found in Step 5, Table 4.1. Assume that two No. 6 bars will be added at each side face and the remainder will be divided equally between top and bottom of beam with one in each corner. "AE = (2.49 in.2(LQ2)) = 0.7 in.2 Add 0.7 in.2/2 = 0.35 in.2 to Mi and M+ from Step 5, Table 1. Table 4.3—Total longitudinal reinforcement at tension side The spacing of longitudinal torsional sional reinforce2 in. on cen ment should not exceed 12 center and the minimum diameter must be greater than 0.042 se re times the transverse reinforcement spacing, but not less than 3/8 in. Beams 9.5.4.3 205 Number of No. 6 bars As, req'd, in.2 "AE/2, in.2 As "AE, in.2 Req'd Prov. MW1 1.59 0.35 1.94 4.4 5 MW2 2.6 0.35 2.95 6.7 7 MT 2.36 0.35 2.71 6.2 7 ME + 1.81 0.35 2.16 4.9 5 MI + 1.57 0.35 1.92 4.4 5 n. sp g between longitudinal reinforcement is 12 in. spacing VD V¿HG WR)LJ( 10 in.) = 0.42 in. dbb, min in = (0.042)(10 db, No.6 = 0.75 in. n. > db, min = 0.42 in. o.6 6 OK Typical span reinforcement or t due to torsi torsion moment oment the face of the support to zero at span mid-length. equired over a distance equal to:  $x = \phi$ Tth A n /2 Tu x = ((20 ft-kip) - (15.3 ft-kip))(34 ft/2) = 4 ft 20 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) = 4 ft-kip) (34 ft/2) /RQJLWXGLQDOWRUVLRQDOUHLQIRUFHPHQWPXVWEH developed beyond this length a minimum of xo +d (Fig. E.11). xo +d = 14.5 in. + 27.6 in. = 42.1 in. Therefore, bars due to torsion moment must extend a minimum distance of: (4 ft)(12) + 42.1 in. = 90.1 in., say, 7 ft 6 in. from the face of the support on both sides of the span. Note: Bars can be discontinued per the calculations above. Practically, however, bars are extended over the full length of the beam. Stirrups at 12 in. on center. Develop longitudinal reinforcement at face of support. American Concrete Institute Copyrighted © Material – www.concrete.org 206 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E4.11—Typical torsion reinforcement detailing Minimum top bars: 9.7.2.1 At maximum and interior negative moments 25.2.1 The clear spacing between the horizontal bars must be at least the greatest of: 1 in. | Clear spacing greater of { db | 4/3(d) agg [ 1 in. 0.75 in. 4/3(3/4 in.) = 1 in. Assume 3/4 in.] = 1 in. Assume 3/4
in.] = 1 in. Assume 3/4 in.] = 1 in. layer in the beam's m web. b. bw, req'd = 2(coverr + dstirrup rup + 0.75 in.) + 4.5 in. + 6 in. q'd d = <math>2(1.5 in. + 8 in. = 16 in. < 18 OK Therefore seven No No. 6 bars can be placed in one layer in the 18 in. beam web. Fig. E4.12—Top bar layout. Note: a preferred solution would be to bundle a few bars together to provide larger spacing between them and to allow for improved concrete last the greatest of CHAPTER 7—BEAMS Minimum bottom bars. The clear spacing between the horizontal bars must be at least the greatest the greatest the greatest of of: 1 in. Clear spacing greater of  $\frac{1}{4b} \frac{4}{3}(\frac{1}{4})$  agg  $1 \text{ in.} 0.75 \text{ in.} \frac{4}{3}(\frac{3}{4} \text{ in.}) = 1 \text{ in.} & \text{KHFNLicYH1REDUVFDQEHSODFHGLQRQHOD}$  in the beam's web. Therefore, clear spacing between horizontal bars must be at least 1 in. bw, reg'd = 2(cover + dstirrup + 0.75 in.) + 4db + 4(1 in.)min, spacing bw, reg'd = 2(1.5 in. + 0.75 in.) + 3.0 in. + 4 in. = 14.3 in. < 18 in. OK (25.2.1) Refer to Fig. E4.13 for steel placement in beam web. 9.7.2.2 24.3.1 24.3.2 24.3.2.1 Beams 25.2.1 207 7KHUHIRUH¿YH1REDUVFDQEHSODFHGLQRQHOD\HULQ the 18 in. beam web. Fig. E4.13—Bottom ott reinforcement inforcement \left( 40,000 \right) | 15 | -2.5cc | \sqrt{fs} s = the lesser of \left\{ | (40,000) | | 12 | (\sqrt{fs} where fs = 2/3fy = 40,000 psi 7KLVOLPLWLVLQWHQGHGWRFRQWUROÀH[XUDOFUDFNLQ] width, where cc = 2 in. is the least distance from the No. 6 bar surface to the tension face. 000 \ (40,, 0 s = 15 | -2.5(2 in.) = 10 in. (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) (40,, 0 s = 15 | -2.5(2 in.) = 10 in. (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) (40,, 0 s = 15 | -2.5(2 in.) = 10 in. (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) (40,, 0 s = 15 | -2.5(2 in.) = 10 in. (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Controls (40,000 |) Cont$ s = 12 | = 12 in. ( 40, 000 |/ 2.3 in. spacing is provided; therefore, OK American Concrete Institute – Copyrighted © Material – www.concrete.org 208 9.7.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) %RWWRPUHLQIRUFLQJEDUOHQJWKDORQJ¿UVWVSDQ &DOFXODWHWKHLQÀHFWLRQSRLQWIRUSRVLWLYHPRPHQW Assume the maximum moment occurs at midspan )LJ( )URPHTXLOLEULXPWKHSRLQWRILQAHFtion is obtained from a freebody diagram: Mmaxíwu(x)2/2 = 0 Fig. E4.14—Typical span moment diagram. 7RSUHLQIRUFLQJEDUOHQJWKDORQJ¿UVWVSDQ At the exterior support: &DOFXODWHWKHLQAHFWLRQSRLQWIRUWKHQHJDWLYH momen diagram: iMmaxiwu(x)2/2 + Vux = 0 (223 ft-kip) - kip) - (2.7 kkip/ft)(x) p/ft 2/2 = 0 12.8 ft, say, 13 ft x = 12.86 HFWLRQSRLQWRIPD[L ,QAHFWLRQSRLQWRIPD[L ,QAHFWLRQSRLQWRIPD[L ,QAHFWLRQSRLQWRIPD]] + Vux = 0 (IWNLS ±NLSIW & DOFXODWHWKHLQAHFWLRQSRLQWRIPD[L ,QAHFWLRQSRLQWRIPD]] + Vux = 0 (IVNLS ±NLSIW & DOFXODWHWKHLQAHFWLRQSRLQWRIPD]] x = 0 (IVNLS ±NLSIW & DOFXODWHWKHLQWRIPD]] + Vux = 0 (IVNLS ±NLSIW & DOFXODWHWKHLQWRIPD]] + Vux = 0 (IVNLS ±NLSIW & DOFXODWHWKHLQWRIPD]] + Vux = 0 (IVNLS ±NLSIW & DOFXODWHWKHLQWRIPD]] + Vux = 0 (IVNLS ±NLSIW & DOFXODWHWKHLQWRIPD]] + Vux = 0 (IVNLS ±NLSIW & DOFXO  $x^2/2+46x = 0 = 0 = 4.96 \text{ ft}$ , say, 5 ft 0 in., QAHFWLRQSRLQWRIH[WHULRUQH]DWLYHPRPHQW  $M = (1WNLS \pm NLSIW x)^2/2+53x = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 0 = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQSRLQWRILQWHULRUQH]DWLYHPRPHQW  $L_2(2+53x) = 7.2 \text{ ft}$ , say, 7 ft 3 in., QAHFWLRQWHUL length of No. 6 bar 7KHVLPSOL¿HGPHWKRGLVXVHGWRFDOFXODWHWKH GHYHORSPHQWOHQJWKRID1REDU 7RSEDUV f y $\psi$ t  $\psi$ e (60,000 psi)(1.3)(1.0) Ad = | (0.75 in.) = 33.1 in. (25)(1.0) 5000 psi / say, 36 in. 25.4.2.2 Ad = 25.4.2.4 ZKHUHzt LVWKHEDUORFDWLRQzt = EHFDXVH PRUHWKDQLQRIIUHVKFRQFUHWHLVSODFHGEHORZ %RWWRPEDUV WRSKRUL]RQWDOEDUVDQGzt = EHFDXVHQRWPRUH ( (60,000 psi)(1.0)(1.0) WKDQLQRIIUHVKFRQFUHWHLVSODFHGEHORZERWWRP A d = |
(0.75 in.) = 25.4 in. ( (25)(1.0) 5000 psi / horizontal bars say, 30 in. ze LVFRDWLQJIDFWRUDQGze = EHFDXVHEDUVDUH XOFRDWHG )LUVWVSDOWRSUHLOIRUFHPHOW 9.7.3.2 /HOIWKVDWWKHHIWHULRUVXSSRUW 5HLOIRUFHPHOWLVUHVVDOGDWVHFWLROVDOROIWKHVSDO ZKHUHEHOWRUWHUPLODWHGWHOVLROUHLOIRUFHPHOWLV UHLOIRUFHPHOWLV QRORQJHUUHTXLUHGWRUHVLVWÀH[XUH VWÀH[XUH )RXU1REDUVDUHUHTXLUHGWRUHVLVWWKHEHDP HTXLUHG IDFWRUHGQHJDWLYHPRPHQWDWWKHH[WHULRUFROXPQ PRP IDFH &DOFXODWHDGLVWDQFHxIURPWKHIDFHRIWKHFROXPQ DQF RPWKHIDFHRI FROXPQ UHVXI&FLHQWWRU VLVWWK ZKHUHWZR1REDUVDUHVXI¿FLHOWWRUHVLVWWKH OW IDFWRUHGPRPHOW 9.7.3.3 20λ f c´ dd XVW QGEH\RQGWKH WLRQDW 5HLQIRUFHPHQWPXVWH[WHQGEH\RQGWKHVHFWLRQDW ZKLFKLWLVQRURQ] 100 HTXLUHGWRUHVLVWAHIXUHIRUD GLVWDOFHHTXDOWRWKHIUHDWHURId or 12db. x2 + 46 kip(x) = -2(0.44 in. 2) $2(0.44 \text{ in.2})(60 \text{ ksi}) \times (0.9)(60 \text{ ksi}) \times (0.9)(60 \text{ ksi}) = 27.6 \text{ in.} - 2(0.85)(5 \text{ ksi})(18 \text{ in.}) = 9 \text{ in.} 2)$  12db = 12(0.75 in 7KHUHIRUH 7KH in., say, 52 in. or 4 ft 4 in. from WKHIDFHRIWKHFROXPQ 7KHVXPRIWKHWKHRUHWLFDOFXWRIISRLQWDQGd FRQWUROV<sup>2</sup>H[WHQGWZR1REDUVIWLQIURPWKH LQWHULRUIDFHRIWKHH[WHULRUVXSSRUWVKRZQEROGLQ)LJ E4.16. 9.7.3.8.4 7KHLQAHFWLRQSRLQWLVFDOFXODWHGDERYHDWIWLQ PXVWEHH[WHQGHGDPLQLPXPRI IURPWKHIDFHRIWKHFROXPQ7KHUHPDLQLQJWZR 5 ft 0 in. = 87.6 WRDFWDVKDQJHUEDUVIRUVWLUUXSV American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 210 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) First span top reinforcement 9.7.3.2 9.7.3.8.4 /HQJWKVDWWKHLQWHULRUVXSSRUW Following the above, seven No. 6 bars are required to resist the factored moment at the iUVWLQWHULRUFROXPQIDFH Calculate a distance x from the face of the column ZKHUHWKUHH1REDUVDUHVXIiFLHQWWRUHVLVW the factored moment. (Four No. 6 bars will be discontinued). x2 + 53 kip(x) =  $-3(0.44 \text{ in.} 2)(60 \text{ ksi}) \times (0.9)$ (60 ksi) | 27.6 in. - 2(0.85)(5 ksi)(18 in.) |/ (-(312 ft-kip) - 2.7 kip/ft x = 3.1 ft, say, 3 ft - 3 in. Therefore, extend four No. 6 bars the greater of the development length (36 in.) and the sum of theoretical cutoff point (3.25 ft) and d. 39 in. + 27.6 in. = 56.6 in. The distance of 56.6 in. Shown bold in Fig. E4.16 from the exterior face of the exterior support controls. Say 5 ft 0 in. as shown in Fig. E4.16. Extend the remaining three No. 6 bars the longer of the development length (36 in.) from where the four No. 6 bars are cut off and d = 27.6 in. beyond the LQÀHF LQÀHF URANT LQÀHF dEH\RQGWKHLQAHFThe lon longer SRL RZQEROGL WLRQSRLQWVKRZQEROGLQ)LJ(2QHRIWKHWKUHH N No. 6 bbars will be termin terminated at 7 ft 3 in. + 27.6 in. § IW H UHPDLQLQ §IWLQ7KHUHPDLQLQJWZR1RWRSEDUVDUH ende and spliced liced at m extended midspan. American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 7—BEAMS 211 9.7.3.2 9.7.3.3 /HQJWKVIURPPLGVSDQWRZDUGWKHFROXPQ )ROORZLQJWKHVDPHVWHSVDERYH¿YH1REDUV are required to resist the factored moment. (Three No. 6 bars will be cut off).  $x^2 = 2(0.44 \text{ in.2})(0.9)(60 \text{ ksi}) 2 / 2(0.44 \text{ in.2})(60 \text{ ksi}) \times 27.6 \text{ in.} = 138.6 \text{ in.} = 111 \text{ in.}$  Therefore, extend the three No. 6 bars the longer of the development length (30 in.) and 111 in. + 27.6 in. = 138.6 in. = 111 in. Therefore, extend the three No. 6 bars the longer of the development length (30 in.) and 111 in. + 27.6 in. = 138.6 in. = 111 in. longer—shown bold in Fig. E4.16 from midspan. 9.7.3.8.2 A minimum of one-fourth of the positive tension reinforcement must extend into the support minimum 6 in. The 6 in. requirement is superseded by the integrity reinforcement must extend into the support minimum 6 in. The 6 in. requirement to develop the bar at the column face. ([WHQGWKHUHPDLQLQJEDUVWZR1REDUV;2YH No. 6) the greater of the development length (30 in.) from the three No. 6 bar cutoff and d = 27.6 in. beyond WKHLQÀHFWLRQSRLQWDQGDPLQLPXPRILQLQWRWKH suppo support. m bar length is the distance 6 in. The controlling bo bottom upport shown show wn iin bold in Fig. E4.16. into the support 9.7.3.8.3 HFWL m ent \$WSRLQWRILQÀHFWLRQd b for positive moment cem must be limited such ch tha tension reinforcement that Ed FH VDWLV IRUWKDWUHLQIRUFHPHQWVDWLV2HV Ad  $\leq$  Vu = 46 kip - (2.69 2.69 kip/f kip/ft)(4 ft) = 35.2 kip A hat location on assum At that assume two No. 6 bars are effective: Mn + Aa Vu where Mn is calculated assuming all reinforcement d at the h at the section is stressed to fy. Vu is calculated section. At the support, Ea is the embedment length beyond the center of the support. At the point of
LQAHFWLRQOLPLWHGWRWKHJUHDWHURId and 12db. 9.7.3.5 QWRI WLRQRFFXUVDWIWIURPWKHIDFHRIWKH coeffect of the support. At the point of LQAHFWLRQOLPLWHGWRWKHJUHDWHURIDFHRIWKH coeffect of the support. umn column. ,IEDUVDUHFXWRIILQUHJLRQVRIÀH[XUDOWHQVLRQ then stress discontinuity in the continuing bars will RFFXU7KHUHIRUHWKH&RGHUHTXLUHVWKDWÀH[XUDO tensile reinforcement must not be terminated in a WHQVLOH]RQHXQOHVVD E RUF LVVDWLV¿HG (a) Vu" ||Vn at the cutoff point (b) Continuing reinforcement must not be terminated in a WHQVLOH]RQHXQOHVVD E RUF LVVDWLV¿HG (a) Vu" ||Vn at the cutoff point (b) Continuing reinforcement provides double the DUHDUHTXLUHGIRUÀH[XUHDWWKHFXWRIISRLQWDQGWKH DUHDUHTXLUHGIRUÀH[XUHDWWKHFXWRIISRLQWDQGVu" []Vn. (c) Stirrup or hoop area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance 3/4d from the termination point. Excess stirrup or hoop area shall be at least 60bws/fyt. Spacing s shall not exceed dub). (2(0.44 in.2)(60 ksi) M n = 2 0.44 in.2 (60 ksi) 27.6 in. - (2)(0.85)(5 ksi)(52 in.) (17 ft - 9.25 ft) = 25.2 kip []Vn []Vc + Vs) where Vc is calculated in Step 6 []Vn = 0.75(70.3 kip + 0) = 52.7 kip []Vn = 2/3(52.7 kip) = 35 kip OK Vu NLS"[]Vn = 35 kip Because only one of the three conditions needs to be VDWLV2HGWKHRWKHUWZRZLOOQRWEHFKHFNHG American Concrete Institute – Copyrighted © Material – www.concrete.org Beams First span bottom reinforcement 212 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Integrity reinforcement, but at least two bars must be continuous. Two No. 6 bars are extended into the column region WZR1R!¿YH1R 2VDWLV¿HGUHIHUWR)LJ E4.16). 9.7.7.1b At least one-sixth the maximum negative moment reinforcement at the support, but at least two bars must be continuous. Four No. 6 bars are extended into the column region IRXU1R!VHYHQ1R 2VDWLV¿HGUHIHUWR)LJ E4.16). 9.7.7.1c /RQJLWXGLQDOUHLQIRUFHPHQWPXVWEHHQFORVHGE\ closed stirrups along the clear span of the beam. /RQJLWXGLQDOUHLQIRUFHPHQWLVHQFORVHGE\1RVWLUUXSV at 12 in. on center along the full beam length— VDWLV¿HG 9.7.7.1 9.7.7.1 a /RQJLWXGLQDOVWUXFWXUDOUHLQIRUFHPHQWPXVWSDVV through the region bounded by the longitudinal reinforcement of the column. 9.7.7.4 25.4.3.1 7KLVFRQGLWLRQLVVDWLV¿HGE\H[WHQGLQJWKHWZR1RWRS and bottom bars full length and through the column cores. Integrity reinforcement must be anchored to develop fy at the face of the support. Therefore, development length for deformed bars in tension terminating in a standard hook must be the greater of: At the exterior support, the No. 6 bars must be developed at the face. Calculate if a standard hook will allow a No. 6 bar to develop within the column. (f ywewcwr) (a) A dh = [ (0.75 in.) = 8.9 in. 50(1.0) 5000 psi / 50(1 ( 5 (b) 8db (c) 6 in. 8(0 7 in.) .) = 6 in. (b) 8(0.75 ( 6 in. (c) Controls 25.4.3.2 FR IDFW ecau bars 7KHUHIRUHWKHKRRN¿WVZLWKLQWKHFROXPQOK UHIR RRN¿WVZ ZKHUHze LVWKHFRDWLQJIDFWRUz e = 1.0 because are uncoated zc LVWKHFRYHUIDFWRUzt = 0.7 because bars are smaller than No. 11 and terminate with a 90-degree hook with cover on bar extension EH\RQGKRRN•LQ 9.7.7.5 9.7.7.6 zrLVWKHFRQ¿QLQJUHLQIRUFHPHQWIDFWRUDQGzr EHFDXVHEDUVDUHQRWFRQ¿QHGZLWKVWLUUXSV spaced at s"db. Splices are necessary in continuous structural integrity reinforcement. The beam's longitudinal reinforcement shall be spliced in accordance with (a) and (b): (a) Positive moment reinforcement shall be spliced at or near th support (b) Negative moment reinforcement shall be spliced at or near midspan Splice length = (1.3)(development length) Edc = 1.3(27 in.) = 35 in., say, 3 ft 0 in. Refer to Fig. E 4.17 Use Class B tension lap splice American Concrete Institute – Copyrighted © Material – www.concrete.org 213 Beams CHAPTER 7—BEAMS Fig. E4.16—End span reinforcement me cutoff utoff locations. Step 9: Internal spans Flexure reinforcement was calculated above in Step 5. Six No. 6 top bars are required at midspan 9.7.6.2.2 Shear and torsion reinforcement following the same calculation in Steps 6 and 7, No. 4 at 10 in. are required for minimum 7 ft 0 in. from face of each column. Space No. 4 stirrups at maximum 12 in. on center (d/2) for the remainder of the span. American Concrete Institute – Copyrighted © Material – www.concrete.org 214 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Detailing Fig. E4.17—Beam reinforcement details. tails. Fig. E4.17—Beam reinforcement details. E4.18—Sections. American Concrete Institute - Copyrighted © Material - www.concrete.org 215 Beam Example 5: Continuous transfer girder 'HVLJQDQGGHWDLODQLQWHULRUFRQWLQXRXVIRXUED\EHDPEXLOWLQWHJUDOO\ZLWKDLQVODE7KHVSDQEHWZHHQ&ROXPQ/LQHV DQG'LVDWUDQVIHUJLUGHUVXSSRUWLQJcYHVWRULHVDERYH Given: Load— Service additional dead load D = 15 psf Service live load L = 65 psf Girder, beam and slab self-weights are given below. Material properties— fcg SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi 3 QRUPDOZHLJKWFRQFUHWH Span length— Typical beam: 14 ft Girder: 28 ft Beam and girder width: 24 in. Column dimensions: 24 in. x 24 in. Fig. E5.1—Plan and elevation of transfer girder and beams. American Concrete.org Beams CHAPTER 7—BEAMS 216 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Material requirements 9.2.1.1 The mixture proportion must satisfy the durability requirements of Chapter 19 (ACI 318) and structural strength requirements. The designer determines the durability classes. Please refer to Chapter 4 of SP-17 for an in-depth discussion of the Categories and Classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Step 2: Beam geometry 9.3.1.1 Girder depth The transfer girder supports a column at midspan with tributary loads from the third level, four stories, and a roof. Therefore, the depth limits in DOFXODWHGGHÀHFFWLRQOLPLWVLQ WLRQVPXVWVDWLVI\WKHGHÀHFWLRQOLPLWVLQ de of the girder beam are Beams on either side RUP VXEMHFWHGWRXQLIRUPORDG7KHUHIRUHWKHGHSWK ntrolling limits in Table 9.3. 9.3.1.1 aree used and the controlling eam depth th is one end co tinuou condition for beam continuous: 9.2.4.2 6.3.2.1 Calculation By specifying that the concrete mixture shall be in accordance with ACI 301-10 and providing the TODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 5000 psi. Assume 48 in in. deep transfer girder. h = A (14 ft)(12 iin./ft) = = 9.1 in. 18.5 18.5 1 ams are in Because intended to provide continuity ssist the girder, der, they must m have enough stiffness for to assist th purpose. pur this U a beam b epth of 30 in. Use depth Flange width The transfer girder and beams are poured monoeam. lithically with the slab and will behave as a T-beam. 7KHHIIHFWLYHADQJHZLGWKRQHDFKVLGHRIWKH transfer girder is obtained from Table 6.3.2.1. 7UDQVIHUJLUGHUADQJHZLGWK Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | of web is | S w / 2 the least of | ] A n /8 (8)(7 in.) = 56 in. (does not control) ((28 ft)(12) - 24 in.) /8 = 39 in. Controls Each side | 8hslab | 0 the least of | ] A n /8 (8 side 8hslab of web is Sw / 2 the least of A n/8 Flange width: bf = 8 - 8 + 8 + 8 = 102 in. 8 - 24 in. 3 -
24 in. 3 - 24 in. 3Beams 6WHS/RDGVDQGORDGSDWWHUQV Applied load on transfer girder 7KHVHUYLFHOLYHORDGLVSVILQRI&FHVDQG psf in corridors per Table 4-1 in ASCE 7-10. This example will use 65 psf as an average live load as the actual layout is not provided. To account for the ZHLJKWRIFHLOLQJVSDUWLWLRQVDQG+9\$&V\VWHPV add 15 psf as miscellaneous dead load. Dead load. Dead load: 7UDQVIHUJLUGHUVHOIZHLJKWZLWKRXWADQJHV Concentrated load on girder from column above (Fig. E5.2): Wgir =  $[(24 \text{ in.})(48 \text{ in.})](0.150 \text{ kip/ft3})/144 = 1.2 \text{ kip/ft3})/144 = 1.2 \text{ kip/ft3}/144 = 1.2 \text{ kip/ft3})/144 = 1.2 \text{ kip/ft3} = 44.1 \text{ kip Column weight per level: } 7 \text{ in.} / 3 \text{ Pcol} = (2 \text{ ft})(2 \text{ ft}) | 12 \text{ ft} - | (0.15 \text{ kip/ft3})/144 = 1.2 \text{ kip/ft3})/144 = 1.2 \text{ kip/ft3}/144 = 1.2 \text{ kip/ft3} = 44.1 \text{ kip Column weight per level: } 7 \text{ in.} / 3 \text{ Pcol} = (2 \text{ ft})(2 \text{ ft}) | 12 \text{ ft} - | (0.15 \text{ kip/ft3})/144 = 1.2 \text{ kip/ft3}/144 = 1.2 \text{ kip$ Total dead load aapplied PSSDLL = ((14 ft)(36 6 ft)(0 ft)(0.015)(01 kip/ft2) = 7.6 kip ght on both ends of the girder: Beams self-weights(44.1 kip + 14.7 ki kip + 7.6 kip)(5) PD = (44 43 kip = 433 / LYHORDG 7KHVHUYLFHOLYHORDGLVSVILQRI&FHVDQG psf in corridors per Table 4-1 in ASCE 7-10. This example will use 65 psf as anaverage, as the actual layout is not provided. A 7 in. slab weighs 88 psf service dead load. ws =  $[(24 \text{ in.})(30 \text{ in.}) + (60 \text{ in.} - 24 \text{ in.})(7 \text{ in.})] \times [(0 [(0.150 \text{ kip/ft3}) / 144] = 1.03 \text{ kip/ft2}) = 18 \text{ kip} (15) L = Lo$ 0.25 + |K LL AT | where L is reduced live load; Lo is unreduced live load; KLL is live load element factor = 4 for internal columns and 2 for internal columns and 2 for interior beam (ASCE 7 Table 4-2); AT—tributary are and KLLAT•IW2 KLL = 4; 4(36 ft)(14 ft) = 2016 ft2 > 400 ft2 Fourth to seventh reduced live load; Lo is unreduced live load; KLL is live load element factor = 4 for internal columns and 2 for interior beam (ASCE 7 Table 4-2); AT—tributary are and KLLAT•IW2 KLL = 4; 4(36 ft)(14 ft) = 2016 ft2 > 400 ft2 Fourth to seventh reduced live load; Lo is unreduced live load; KLL is live load element factor = 4 for internal columns and 2 for interior beam (ASCE 7 Table 4-2); AT—tributary are and KLLAT•IW2 KLL = 4; 4(36 ft)(14 ft) = 2016 ft2 > 400 ft2 Fourth to seventh reduced live load; Lo is unreduced live load; KLL is live load element factor = 4 for internal columns and 2 for interior beam (ASCE 7 Table 4-2); AT—tributary are and KLLAT•IW2 KLL = 4; 4(36 ft)(14 ft) = 2016 ft2 > 400 ft2 Fourth to seventh reduced live load; KLL is live load element factor = 4 for internal columns and 2 for internal column 2016 ft 2 / L = 0.038 ksf > 0.4Lo = 0.026 ksf OK American Concrete Institute - Copyrighted © Material - www.concrete.org 218 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) KLL = 2; 2(36 ft)(14 ft) = 1008 ft2 > 400 ft2 and L · Lo = 26 psf Reduced third level live load on beam: () 15 L = (0.065 kip/ft 2) | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 0.047 ksf 2 | 0.25 + = 1008 ft / L = 0.047 ksf > 0.4Lo = 0.026 ksf OK Concentrated live load to column per level: PL,col = (14 ft)(36 ft)(0.047 kip/ft2) = 23.7 kip  $\stackrel{\text{\tiny T}}{=}$  PL = 4PL,col + PL,3rd + PRoof 5.3.1 Concentrated live load on girder:  $\stackrel{\text{\tiny T}}{=}$  PL = (23.7 kip) + (19.2 kip)(4 levels) + (18 kip) = 119 kip The transfer girder resists gravity only and lateral forces are not considered in this problem. Transfer girder Distributed: wu = 1.4(433 kip) = 607 kip U = 1.4D The superimposed dead load is calculated above FHQWUDWHGORDG/LYHORDG DQGLVLQFOXGHGLQWKHFRQFHQWUDWHGORDG/LYHORDG LGWKRI LVDSSOLHGRYHUWKHZLGWKRIWKHJLUGHUIW = 1.20 + 1.65 kip/ft + 1.6(0.065 ksf)(2 ft) Distributed: ki = 1.65 kip/ft 1.2(433 kip) p) + 11.6 6 ((119 kip) = 710 kip Controls Pu = 1.2 B ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60
in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 14 kip/ft + (15 psf)(60 in./12)/1000) wu = 1.2 H ms Beams 1.4(1.03 = 1.55 kip/ft kip/ft)/1.4 + 1.6((65 psf)(60 in./12)/(1000) wu = 1.2(1.55 kip = 1.85 kip/ft Controls Fig. E5.2—Column C/4 tributary area. American Concrete Institute – Copyrighted © Material – www.concrete.org Step 4: Analysis 9.4.3.1 The beams are built integrally with supports; therefore, the factored moments and shear forces (required strengths) are calculated at the face of the supports. The beams were analyzed as part of a frame. The moment and shear diagram obtained from a software are presented below (Fig. E5.3). Notes: 1. Factored moments and shear forces are shown at faces of columns. 6SDQ%'LVVXEMHFWHGWRODUJHFRQFHQWUDWHGORDGDWPLGVSDQDQGUHODWLYHO\VPDOOGLVWULEXWHGEHDPVHOI weight. Therefore, the appearance of a straight line moment diagram. Fig. E5.3—Shear and moment diagram. Fig. E5.3—Shear and moment diagrams. American Concrete Institute – Copyrighted © Material – www.concrete.org 219 Beams CHAPTER 7— BEAMS 220 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Moment design 9.3.3.1 The code does not permit a beam to be designed with steel strain less than 0.004 in./in. at nominal strength. The intent is to ensure ductile behavior. In most reinforced beams, such as this example, reinforcing bar strain is not a controlling. issue. 21.2.1(a) The design assumption is that beams will be tension-controlled; therefore, the moment reduction IDFWRULV7 KLVDVVXPSWLRQZLOOEHFKHFNHG later. 20.6.1.3.1 Determine the effective depth assuming No. 5 stirrups and No. 11 bars for the transfer girder positive moment and No. 9 bars for the transfer girder negative moment. Assume No. 4 stirrups and No. 6 and No. 9 bars for beams positive and negative moments, respectively. Girder beam and beams one rows of reinforcement with onee bar spacing between the an one layer of bars at two rows of reinforcement with one bars one rows of reinforcement with onee bar spacing between the supports. Beams one rows of reinforcement with onee bar spacing between the an one layer of bars at two rows of reinforcement with one bar spacing between the supports. only d = h - cover - dttie - 3db/2 positive moment negative d = 30 in. - 1.5 in. - 0.5 in. - 1.128 in./2 = 27.4 in. American Concrete Institute - Copyrighted © Material - www.concrete.org 22.2.2.1 The concrete compressive strain at nominal moment strength is calculated at: İcu = 0.003 in./in. 22.2.2.7 KHWHQVLOHVWUHQJWKRIFRQFUHWHLQÀH[XUHLVDYDULable property and is approximately 10 to 15 percent of the concrete compressive strength. ACI 318 neglects the concrete tensile strength to calculate nominal strength. 22.2.2.4.1 22.2.2.4.3 22.2.1.1 221 Beams CHAPTER 7—BEAMS The concrete compressive strength to calculate nominal strength. Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRIa ulcZKHUHul ive strength and is a function of concrete compressive .2.4.3. is obtained from Table 22.2.2.4.3. For fcg SVL  $\beta$  = 0.85 – ent concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force to the ension by equating the concrete compressive depth a Find the equivalent sion force within the equivalent sion force to the equivalent sion force within the e  $0.85(5000 \text{ Transfer girder: For positive moment: b = bf = 60 \text{ in. } a = As} (60,000 \text{ psi}) = 0.235 \text{ As } 0.85(5000 \text{ psi})(102 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ Transfer girder and beams: For negative moment: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ transfer girder and beams: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ transfer girder and beams: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ transfer girder and beams: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ transfer girder and beams: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{
transfer girder and beams: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ in.}) \text{ transfer girder and beams: } b = bf = 60 \text{ in. } a = As (60,000 \text{ psi}) = 0.238 \text{ As } 0.85(5000 \text{ psi})(24 \text{ i$ in.) 0.05(5000 psi - 4000 psi) 0.05( = 0.8 1000 000 psi Fig. E5.4—Section compression block and reinforcement locations. American Concrete Institute - Copyrighted © Material - www.concrete.org 222 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The transfer girder and beams are designed for the  $PD[LPXPAH[XUDOPRPHQWVVKRZQLQWKHDERYH moment diagram (Step 4). 9.5.1.1 The beams' design strength must be at least the required strength is calculate the required strength at each section along their lengths: <math>Mn \cdot Mu = \phi As f y | d - | \langle 2 \rangle$  RUWRSQHJDWLYHUHLQIRUFHPHQWXVH2QHOD\HURITERSET (Marcinet at each section along their lengths:  $Mn \cdot Mu = \phi As f y | d - | \langle 2 \rangle$  RUWRSQHJDWLYHUHLQIRUFHPHQWXVH2QHOD\HURITERSET (Marcinet at each section along their lengths:  $Mn \cdot Mu = \phi As f y | d - | \langle 2 \rangle$  RUWRSQHJDWLYHUHLQIRUFHPHQWXVH2QHOD\HURITERSET (Marcinet at each section along their lengths:  $Mn \cdot Mu = \phi As f y | d - | \langle 2 \rangle$  RUWRSQHJDWLYHUHLQIRUFHPHQWXVH2QHOD\HURITERSET (Marcinet at each section along their lengths) at each section along the se 9 bar; db = 1.128 in., As = 1.0 in.2, and d = 43.7 in. Minimum reinforcement use: Two layers of No. 11 bar; db = 1.41 in., As = 1.56 in.2, and d = 43.7 in. Minimum reinforcement must st be at least the minimum required reinforcement must st be at least the minimum reinforcement must st be at least Use [email protected] = 45 in. n. > [email protected] = 43.7 in. Transfer girder: Mu, ft-kip As, req'd, in.2 Req'd Select 9.4 9.4 10 17.51 11.22 12 9.5 10 i 1782 Max. M Hax. M Number of bars er rrequired ed minimum rei orcement will yield higher reinforcement area: As = 3 f c' fy bw d Because fcg!SVL(TD RQO\DSSOLHV) 9.7.2.3 24.3.2 24.3.2.1 As = 3 5000 psi si (24 in.)(45 in.) = 3.73 in.2 in 660,000 psi ded reinforcement area exceeds the minimum required. Therefore, OK The girder is 48 in. deep. To control cracks within the web, ACI 318 requires skin reinforcement area exceeds the minimum required. 24 in. Spacing of skin reinforcement in girder must not exceed the lesser of: (40, 000) s = 15 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc and f s | / (40, 000) s = 12 | - 2.5cc a and fs = 2/3fy = 40 ksi Place two No. 8 skin reinforcement at girder middepth: 24 in. and the second pair of No.8 bars at 33.5 in. from the top of the girder. American Concrete Institute - Copyrighted © Material - www.concrete.org Controls 9.7.2.3 Skin reinforcement can be used in the strength calculation of the girder. Positive reinforcement at midspan: 223 (( 2.58 in.) ) || || 33.5 in. - 2 /|  $\phi$ M n = (0.9)(60 ksi)(2)(0.79 in.2) | 2.58 in.) || ( 24 in. + - || || 2 / ||  $\Box$ Mn = 4685.8 in.-kip = 390.5 ft-kip, say, 390 ft-kip Using strain compatibility, try two No. 8 bars in two layers on both sides of the girder. Assume reinforcement is yielding. It will be checked later. Re-evaluating the positive in.2 = 9.9 oose 100 No. N 11 bbars. bar Choose Negative reinforcement rcem att the supports pp aye of skin reinforcement d the two sides in the top hhalf of the girder. Exten Extend ars over the full length ngth of No. 8 middepth sk skin bars he top half at the support upport the girder and usee ffor the n both sides of o the girder two layers of No. 6 bbars on girder. Assume that skin reinforcement reach yielding (will be checked later). Calculate the provided moment from the skin reinforcement at girder midspan: 5.29 in.  $\langle \phi M n = (0.9)(60 \text{ ksi})(2)(0.79 \text{ ksi})(2)(0 \text{ in. } 2) | 24 \text{ in. } - | 2 / 5.29 \text{ in. } \rangle () (60 \text{ ksi})(2)(0.79 \text{ ksi})(2)(0.79 \text{ ksi})(2)(0 \text{ in. } 2) | 24 \text{ in. } - | 2 / 5.29 \text{ in. } \rangle () (60 \text{ ksi})(2)(0.44 \text{ ksi})(2) | 0.44 \text{ ksi})(2) + (0.9)(60 \text{ in. } 2) | 31.25 \text{ in. } - | 2 / 5.29 \text{ in. } \rangle () (407 \text{ ft-kip}) = 407.1 \text{ ft-kip}, \text{ say, } 407 \text{ ft-kip}, \text{ say, } 407 \text{ ft-kip}, \text{ say, } 407 \text{ ft-kip}, \text{ say, } 407 \text{ ft-kip} = 407.1 \text{ ft-kip}, \text{ say, } 407 \text{ ft-kip}, \text{
say, } 407 \text{ ft-kip}, \text{ say, }$ following equation: As,No.11 = 7.42 in.2 a) ( $\phi$ M n ≥ M u =  $\phi$ As f y | d - || 2) Required No. 9 bars = 7.42 in.2/1.0 in.2 = 7.49 Choose 8 No. 9 bars. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 224 21.2.2 9.3.3.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Check if the calculated strain exceeds 0.005 in./in. (tension controlled), but not less than 0.004 in./in.  $a = As f y 0.85 f c'b and c = a \beta 1 ZKHUH$  $u1 = 0.8 \epsilon s = Transfer girder: 0.003 (d - c) c Mu$ , ft-kip As, prov, in.2 a, in. c, in. İs, in./in. b = 24 in. for negative moments and 102 in. for girder positive moments. Place eight No. 9 in one layer with d = 45.3 in. Check skin reinforcement strain (Fig. E5.5): 7KHUHIRUHWKHDVVXPSWLRQRIXVLQ][LVFRUUHFW Positive half, strain at middepth Check strain in the upper skin layer at 24 in. from the top: Fig. E5 F E5.5—Strain ain distribution depth.  $\varepsilon s = 0.003 \ 0. \ (24 \ 4 \ in. - 2.7 \ in.) = 0.024 > 0.005 \ 2.7 \ in. OK By inspection the other skin reinforcement layers is yielding and the assumption of using <math>0.9 = []s \ correct.$  American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS of using  $0.9 = []s \ correct.$ 225 Beams Negative half, strain at supports B and D (Fig. E5.6) Check strain in the lower skin layer at 24 in. From the bottom: Fig. E5.6—Strain distribution. εs = 0.003 (24 in. - 5.9 in.) = 0.009 > 0.005 5.9 in. By inspection the other skin reinforcement layers is yielding and the assumption made of using 0.9 = []s correct. Beams Design beams for the maximum load condition. he ma Extend No. 9 top bars from Span BD to resist the IWNLSDQGIWNLSPRPHQWVDW&ROXPQ/LQHV I PRPHQWVDW&ROXPQ/LQHV B and D in Spans AB and respectively,, and No. 6 ns A d DE, respective bars to resist thee rest off the moments. ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ&ROXPQ/LQH() Number of bars Mu, ft-kip As,req'd in.2 Req'd Select Mí (No. 9) 935 8.33 8.33 9 M+ (No. 6) 293 2.38 5.42 6 6SDQ HANDBOOK—SP-17(14) Minimum reinforcement must be at least the minimum reinforcement area exceeds the code minimum reinforcement area at all locations Beams: As, min = 3 5000 psi (24 in.)(27.6 in.) = 2.34 in.2 60,000 psi Provide a minimum six No. 6 bars at all beam tension ORFDWLRQV([FHSWDW&ROXPQ/LQHV%DQG'ZKHUH transfer girder top reinforcement is extended over DGMDFHQWVSDQVWRUHVLVWWKHQH]DWLYHPRPHQWDQG6SDQ DE positive moment, where six No. 6 longitudinal bars is required. Note: Reduce the total number of No. 9 bars to eight No. 9 by extending the two top No. 6 skin bars from the girder into Beam DE to resist part of the negative moment. Calculate strain in No. 6 bars (Fig. E5.7). Fig. E5 Fi E5.7—Strain in diagram. εs = 0. 0.003 (20.5 20 5 iin. in - 5.9 in.) = 0.007 > 0.005 5.9 in. n. Therefore, herefore, rei reinforcement in the two No. 6 skin bars is vieldin vielding and the assumption made of using 0.9 = Is correct. 21.2.2 Check if the calculated steel strain exceeds 0.005 in./in. (tension controlled), but not less than 0.004 in./in. (refer to Fig. E5.8): As fy a and c = a =  $\beta 1$  0.85 f c'b ZKHUH11 = 0.8  $\epsilon s$  = Beams (only maximum moments are checked) M u, ft-kip As, prov, in.2 a, in. c, in. is, in./in. is > 0.005? Mí 935 8 4.7 5.9 0.011 Y M+ 293 2.64 0.62 0.78 0.103 Y 0.003 (d - c) c Note that b = 24 in. for negative moments, respectively. c < hf for both transfer girder and beams; therefore, the T-section members assumption for positive moments is correct. Fig. E5.8—Strain distribution across beam section. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS 227 Step 6: Shear design Beams Transfer girder Shear strength 9.4.3.2 Because conditions a), b), and c) of 9.4.3.2 are VDWLV¿HGWKHGHVLJQVKHDUIRUFHLVWDNHQDWFULWLFDO section at distance d from the face of the support (Fig. E5.9). use d = 45.3 in. at support The controlling factored load combination must satisfy: Fig. E5.9—Shear at the critical section. 9.5.1.1 22.5 (45 in./12) = 373.2 kip Vc = 2 f c'bw d Vc = 212.1(b) Shear strength reduction factor: []shear = 0.75 \varphi Vc = \varphi 114.6 kip []Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc = (0.75)(152.7 kip) = 114.6 kip (]Vc =
(0.75)(152.7 kip) = 114.6 kip (]Vc = (0.7 reinforcement is required. 22.5.1.2 tin shear ar reinforcemen Prior to calculating reinforcement, che check if na dimensions are satisfactory and by  $(85000 \text{ psi}(24 \text{ in.})(45 \text{ in.}) \ 1525227 \text{ ki kkip} + Vu \le \varphi \ 152.7 \ 1000 \text{ lb/kip} \ 1 \le 576.6 \text{ kip OK}$ , therefore, section dimensions are satisfactory American Concrete Institute – Copyrighted © Material – www.concrete.org 228 22.5.10.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Shear reinforcement satisfying equation 22.5.10.1 is required at each section where  $Vu | | Vc Vs \ge Vu - Vc \phi Vs \ge 373.2$  kip – 152.7 kip = 344.9 kip 0.75 Try a No. 5 bar, two legged stirrup 22.5.10.5.3 22.5.10.5.3 22.5.10.5.6 Spacing required for No. 5 stirrups: where Vs = Av f yt d (2)(0.31 in.2)(60,000 psi)(45 in.) s (1000 lb/kip) s = 4.85 in. 344.9 kip = s This is a relatively tight spacing. Use two No. 5 double stirrups side by side. This will yield a spacing of 9.7 in.; say, 8 in. spacing. 9.7.6.2.2 Calculate maximum allowable stirrups spacing: 4 f c'bw d = 4 () 5000 psi (24 in.)(45 in.) 1000 lb/kip = 305.5 kip First, does the beam transverse reinforcement value need to exceed the threshold value? Vs  $\leq$  4 f c'bw d = 305.5 kip OK uired shear strength is higher than Because the required alue the maximum stirr the threshold value, stirrup spacing is 4 aand 12 in. dd/4 = 45 in./4 [d/ in the lesser of d/4 in. < 12 in. RUF HVQRWYDU\VLJQ FDQWO\ %HFDXVHVKHDUIRUFHGRHVQRWYDU\VLJQ FDQWO\ %HFDXVHVKHDUIRUFHG 6SHFL¿HGVKHDUUHLQIRUFHPHQWPXVWEHDWOHDVW 0.75 f c' bw b and 50 w f yt f yt Use s = 8 in. = dd/4 = 11 in. < 12 in.  $\therefore$  OK Av , min s = 4(0.31 in.2) = 0.155 in.2 /in. 8 in. 6SDFLQJVDWLV¿HVWKHUHIRUHOK American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS 229 Beams 9.5.1.1 []Vn •Vu 9.5.3.1 22.5.1.1 Vn = Vc + Vs 9.4.3.2 are VDWLV¿HGWKHGHVL]QVKHDUIRUFHLVWDNHQDWFULWLFDO section at distance d from the face of the support (Fig. E5.10). []shear = 0.75 Beams 21.2.1(b) Shear strength Shear strength reduction factor: Fig. E5.10—Shear at the critical section [email protected] =  $Vu - (1.85 \text{ kip/ft})(27.4 \text{ in.}) = 93 \text{ kip} [Vc = (0.75)(93 \text{ kip}) = 69.7 \text{ kip} 9.6.3.1 \text{ & KHFNLI}[Vc \cdot Vu [email protected]]$ , kip res that minimum shear However, ACI 318 requires reinforcement must be provi provided, where: Beam 1 Beam 3 /2(6 kip)) = 34.9 kip Vu! Vc = 1/2(69.7 eam 4 Beam, V Vc • Vu ? /HIW 0 Y ht Right 21.7 Y uire shear reinfo emen Beam EF does no not require reinforcement; de minimum mum shear reinf rcem for however, provide reinforcement T refo shear Therefore, ar reinfor reinforcement is required where integrity. cret strength ngth is les concrete less than the factored shear force. Beam eam AB DE EF [email protected], kip Is Vu! [V ? 66.7 Y /HIW 95.9 Y Right 73.8 Y /HIW 0 N Right 21.7 N Provide minimum shear reinforcement over all beams spans. Shear reinforcement  $Vs \ge Vu \phi Vs \ge -Vc$  where Vs = Av f yt d s and Vc = 93 kip s = 95.9 kip - 93 kip = 34.9 kip 0.75 (2)(0.2 in.2)(60,000 psi)(27.4 in.) = 18.8 in. 34,900 lb Using No. 3 stirrups The spacing exceeds the maximum allowed d/2 = 13.7 in.; therefore, use 12 in. spacing over the full length of beam. By inspection provisions, 9.7.6.2.2 and 9.6.3.3 are VDWLV¿HG American Concrete Institute - Copyrighted © Material - www.concrete.org 230 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Reinforcement detailing Minimum bar spacing Bottom reinforcement—girder: 25.2.1 The clear spacing between the horizontal No.11 bars must be at least the greatest of: [1 in. | Clear spacing the greater of: {db | 4/3(d) agg | 1 in. 1.41 in. Controls 4/3(3/4 in.) = 1 in. Assume 3/4 in. maximum aggregate size. &KHFNLlcYH1REDUVUHVLVWLQ]SRVLWLYH moment) can be placed in the beam's web; refer to Fig. E5.11. Therefore, clear spacing between horizontal bars must be at least 1.41 in., say, 1.5 in. bw, reg'd = 2(cover + dstirrup + 1.0 in.) + 4db + 4(1.5 in.)min,spacing (25.2.1) bw, req'd = 2(1.5 in. + 0.625 in. + 1.0 in.) + 5.64 in. + 6.0 in. = 17.9 in. < 24 in. OK 7KHUHIRUH¿YH1REDUVFDQEHSODFHGLQRQHOD\HU in the 24 in. transfer girder web. ars: Spacing between longitudinal bars: sp = 24 in. - [2(1.5 in. + 0.625 in. + 1.0 in.) + 4(1.41 in.)] 4 = 3.0 in. Fig. E5.11—Bottom reinforcement layout one layer is shown. Bottom reinforcement—beams: Beams AB, DE, and EF are reinforced with six No. 6 bottom bars uniformly spaced. The calculated spacing LVLQ7KHUHIRUH2. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS Top reinforcement 7HQVLRQUHLQIRUFHPHQWLQADQJHVPXVWEHGLVWULEXWHGZLWKLQWKHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. Girder: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 231 %HFDXVHHIIHFWLYHADQJHZLGWKbf = 102 in. (Step 2), but not wider than: En/10. addi- Beams 24.3.4 addi- Be En/10 = (12 ft)(12)/10 = 14.4 in. < 60 in. SRUWLRORIWKHADQIH Use No. 6 placed in slab over bf for additional bonded reinforcement; refer to Fig. E5.12: This requirement is to control cracking in the slab due to wide spacing of bars across the full effective ADQIHZLGWKDQGWRSURWHFWADQIHLIUHLQIRUFHPHQW is concentrated within the web width. Bar spacing = Span AB BD 24 in. -  $[2(3.125 \text{ in.}) + 5(1.128 \text{ in.})] 5 \text{ DE EF} = 2.4 \text{ in. Prov. No. 9 No. 9 in web No.10 in En/10* No. 6 in outer portion* / 4 4 - 4 / † 5 † 5 - 4 R 5 † 2 6 R 8 6 2
6 R 8 6 2 6 R 8 6 2$ 6 bars Sec )LJ(2/RQJLWXGLQDOEDUVGLVWULEXWHGZLWKADQJH Note: Slab shrinkage and temperature reinforcement in WKHRXWHUSRUWLRQRIADQJHWRVDWLVI\\$&,6HFWLRQ Step 8: Development length Development length of No. 6, No. 9, and No. 11 bar 7KHVLPSOL¿HGPHWKRGLVXVHGWRFDOFXODWHWKH development length of No. 11 bars: f ywt we ( (60,000 psi)(1.0)(1.0) Ad = | (db) = 42.43db ( (20)(1.0) 5000 psi ) 25.4.2.2 Ad = 25.4.2.4 ZKHUHztLVWKHFDVWSRVLWLRQzt = 1.3, if more Top: 1.3db = 1.3(42.43 db) = 55 db than 12 in. of fresh concrete is placed below top KRUL]RQWDOEDUVDQGzt = 1.0, if not more than 12 in. No. 6 No. 9 of fresh concrete is placed below bottom horizontal Ed, in. 31.8 - 59.8 Use Ed, in. 36 - 60 d zeLVWKHFRDWLQIDFWRUDQGze = 1.0, because bars are uncoated No. 8 American Concrete Institute - Copyrighted © Material - www.concrete.org 232 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS, QÀHFWLRQSRLQWV 7KHPRPHQWGLDJUDPLQÀHFWLRQSRLQWVDUHFDOFXlated at both supports and at midspan (Fig. E5.13(a)). (a) Girder moment diagram Bottom bar length along girder &DOFXODWHWKHLQÀHFWLRQSRLQWIRUSRVLWLYHPRPHQW (Fig. E5.13(b)): Maximum moment at midspan: 3348 ft-kip - (1.68 kip/ft)(x)2/2 - 355x = 0 f 3 in. x = 9.2 ft, say, 9 ft mum positive moment occurs at Assume the maximum HTXL PWKHSRLQWR HFWLRQ PLGVSDO)URPHTXLOLEULXPWKHSRLOWRILOAHFWLRO m th lowing free bod is obtained from the following body diagram: Mmaxiwu(x)2/2 + Vux = 0 F. OF , QÀHFWLRQSRLQWDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (378 kip)x = 0 x = 4.76 ft, say, 4 ft 10 in. Right support: & DOFXODWHLQÀHFWLRQSRLQWDW6XSSRUW' []IWNLS ±NLSIW x)2/2 + (379.5 kip)x = 0 x = 4.8 ft, say, 4 ft 10 in. )LJ(2\*LUGHULQÀHFWLRQSRLQWORFDWLRQV American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 233 & KHFNWKHLQÀHFWLRQSRLQWIRU6SDQV\$% (DQG() Span AB: & DOFXODWHLQÀHFWLRQSRLQWDW6XSSRUW%)LJ E5.14(a)): 1/2/2 + Vux = 0 QSRLQ &DOFXODWHLQÀHFWLRQSRLQWDW6XSSRUW\$)LJ E5.14(b)): 2 + Vux = 0 iMmax iwu(x)2/2 Beams The moment at midspan. The moment at midspan. The moment at midspan. The moment at midspan. other support. D ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QAHFWLRQSRLQWORFDWLRQW6XSSRUW% (IWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QAHFWLRQSRLQW6XFDWAX x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 for spans threee and Following the same IRXUWKHLQAHFWLRQSRLQWVDUHFDOFXODWHGDW Span x-left 1 fx say, 2 ft 6 in. American Concrete Institute - Copyrighted © Material - www.concrete.org 234 THE REINFORCED CONCRETE DESIGN HANDBOOK -SP-17(14) Step 10: Cutoff locations Transfer girder 9.7.3.2 9.7.3.3 Support Bars must be developed at locations of maximum stress and locations of maximum stress and locations are required to UHVLVWAH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to Terminated tension bars are no longer required to UHVLVWAH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to Terminated tension bars are no longer required to UHVLVWAH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to Terminated tension bars are no longer required to UHVLVWAH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to Terminated tension bars are no longer required to UHVLVWAH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to Terminated tension bars are no longer required to UHVLVWAH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to Terminated tension bars are no longer requi &ROXPQ/LQHV%DQG' The two end moments at the supports are close, so the calculation will be applied at one end only. Calculate a distance x from the column face where four No. 9 bars can be discontinued and the contribution from the skin reinforcement is ignored. Cutoff point: x2 + 379.5 kip(x) = -4(1.0 in.2) (4(1.0 in.2)(60 ksi)) (0.9)(60 ksi)) (0.9)(60 ksi) ksi) 45 in. - 2(0.85)(5 ksi)(24 in.) // -1803 ft-kip - 1.69 kip/ft x = 2.69 ft, say, 2 ft 9 in. from column face Note: Skin reinforcement are extended over the full girder length and are properly developed or VDWWKHVXSSRUWV H[WHQGHGLQWRWKHDGMDFHQWVSDQVDWWKHVXSSRUWV For No. 9 bars: 1) d = 45 in. Controls 2) 12d db = 12(1) 12(1.128 in.) = 13.5 in. refo extend x tend t d th the ffour No. 9 bars the greater of the Therefore, elop p length (63 in. - Step 8) from the column development f e and d from m theoretic face theoretical cutoff point (33 in.) n. + 45 in. = 78 3 in. >  $\mathcal{E}d = 63 \text{ in.} 33 \text{ in.} 78.3 \text{ T}$  refo 78.33 in. Controls Con Therefore, Four No. 9 bars can be terminated 80 in. from the face s of thee column, shown bold in Fig. E5.15. 9.7.3.8.4 At least one-third of the bars resisting negative moment at a support must have an embedment OHQJWKEH\RQGWKHLQAHFWLRQSRLQWWKHJUHDWHVWRId, 12db, and En/16. For No. 9 bars: 1) d = 45 in. Controls 2) 12db = 12(1.128 in.) = 13.5 in. 3) En/16 = (28 ft - 2 ft)/16 1.625 ft = 19.5 in. Extend the remaining four No. 9 bars the larger of the development length (63 in.) beyond the theoretical cutoff point (33 in.) and
d LQEH\RQGWKHLQAHFtion point (4 ft 10 in.) (Step 9) shown bold in Fig. E5.15). 63 in. + 33 in. = 96 in. 58 in. + 45 in. = 103 in. Controls Extend the remaining four No. 9 bars 8 ft 8 in. from the face of the column. The four No. 9 bars will be, however, extended over the full length of the girder. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS Transfer girder bottom bars Following the same steps above, ten No.11 bars and four No.8 skin bars are required to resist the factored moment at midspan. Calculate a distance x from the midspan where four No.11 bars can resist the factored moment. Note: Skin bars are extended over full girder length. Beams 9.7.3.2 9.7.3.3 235 x2 - (355 kip)x = 4(1.56 in.2) (60 ksi)  $\times (0.9)(60 ksi) \times (0.9$ Therefore, extend six No.11 bars the larger of the development length (60 in. - Step 8) and a distance d beyond the theoretical cutoff point (6 ft 0 in. Controls) 72 in. + 43.7 in. = 115.7 in., say, 9.75 ft from maximum positive moment at midspan (Fig. E5.15). Extend the remaining four No. 11 bars at least the longer of 6 in. into the column or Ed = 60 in. past the theor theoretical cutoff point (Fig. E5.15); in = 132 in. < (13 ft)(12) + 6 in. = 162 in. 60 in. + 72 in. um controls, however, it is The 6 in. into the column them. 9.7.3.8.2 urt of the he positive tensi n bars must At least one-fourth tension ars > 1/4(10 bars) = 2.5 2 bars 4 bars extend into the co column at least 6 in. American Concrete Institute - Copyrighted © Material - www.concrete.org OK 236 9.7.3.8.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 3RLQWRILQAHFWLRQRFFXUVDWIW±IW \$ be limited such that Ed for that bar from the column face. VL]HVDWLVcHV Vu = 379.5 kip - (1.68 kip/ft)(3.75 ft) = 373.2 kip At that location, four No. 11 bars are effective: (4(1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | 43.7 in. - (2)(0.85)(5 ksi)(24 in.) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2)(60 ksi) | / (1.56 in.2kip This length exceeds  $\mathcal{E}d = 60$  in., therefore, OK where Mn is calculated assuming all bars at the section are stressed to fy. Vu is calculated at the section. The term la is the embedment length EH\RQGWKHSRLQWRILQÀHFWLRQOLPLWHGWRWKHJUHDWHU of d and 12db. 9.7.3.5, IEDUVDUHFXWRIILQUHJLRQVRIÀH[XUDOWHQVLRQ then a bar stress discontinuity occurs. Therefore, LOHEDUVPXVWQRW WKHFRGHUHTXLUHVWKDWAH[XUDOWHQVLOHEDUVPXVWQRW be terminated in a tensile zone unless (a), (b), or (c) LVVDWLV¿HG c point (a) Vu" []Vn att the cutoff Continuing bars prov provides double the area required HF SRLQWDQGE Vu " []V []V II. IRUÀH[XUHDWWKHFXWRIISRLQWDQGE V oop area in excess of that required for (c) Stirrup or hoop vided along each terminated erm shear and torsionn is provided nce 3/4d 3/4 from rom th bar or wire over a distance the termination rup or hoop op area shall bbe att least point. Excess stirrup 60bws/fyt. Spacing s shall not exceed d/(8ub)

153.8 8 + 34 + 3 + 344.5 = 498.3 kip Vn = Vc + Vs = 153  $\forall V = 153 \forall V = 15$ > 0.192 in.2, therefore, OK Because only one of the three conditions needs to be VDWLV¿HGWKLVUHTXLUHPHQWLVVDWLV¿HG American Concrete Institute – Copyrighted © Material – www.concrete.org 9.7.7.2 9.7.7.3 237 Integrity reinforcement At least one-fourth of the maximum positive moment bars, but at least two bars, must be continuous and developed at the face of the column. 7KLVFRQGLWLRQZDVVDWLV¿HGDERYHE\H[WHQGLQ]IRXU No.11 bars into the support. 4 bars/10 bars = 2/5 > 1/4 OK Beam longitudinal bars must be enclosed by closed stirrups along the clear span. 7KLVFRQGLWLRQLVVDWLV¿HGE\H[WHQGLQ]VWLUUXSVDWLQ on center over the full length of the beam. Beam structural integrity bars shall pass through the region bounded by the longitudinal column bars. Four No. 11 bars are extended through the columns' dimensions (24 in.). Therefore, beam longitudinal reinforcement must be offset to clear column reinforcement. Fig. E5.15—Girder bar cutoff locations. American Concrete Institute – Copyrighted © Material – www.concrete.org Beams Spans AB and DE 9.7.3.2 Beams spanning between column A and B and DQG(DUHVXEMHFWHGWRFRPSDUDEOH IDFWRUHGAH[XUHDQGVKHDUIRUFHV7KHUHIRUHWKH two beams). To simplify detailing, simply extend eight No. 9 bars and two No. 6 skin bars (span DE only) from the transfer girders to resist the negative moment in WKHDGMDFHQWEHDPV6WHS Calculate a distance x from the column face where four No. 9 bars can resist the factored moment.  $x^2 + 93.1 \text{ kip}(x) = -4(1.0 \text{ in.} 2)(60 \text{ ksi}) - 27.4 \text{ in.} 12 \text{ in.}/\text{ft} | (2(0.85)(5 \text{ ksi})(24 \text{ in.}) | ) - (906 \text{ ft-kip}) - (1.85 \text{ kip/ft}) x = 4.9 \text{ ft}, \text{ say}, 5 \text{ ft} \text{ At } 5 \text{ ft} 0 \text{ in.} \text{ in.} \text{ from the column face, four No. 9 can be$ cut off and the remainder four No. 9 bars can resist ctored mome the factored moment. 9.7.3.3 uto bars must extend beyond nd the The four No. 9 cutoff d to resist location where the they are no longer required VWD TXDOWRWKHJUHDW URId or AHJWHDWHURId 12db. rs: For No No. 9 bars: 11) d = 27.4 2 in. Contro Controls 2) 12d 2d db = 12(1.128 28 iin.)) = 13.54 in. 9.7.3.8.4 tive At least one-third of the bars resisting nnegative moment at a support must have an embedment OHQJWKEH\RQGWKHLQÀHFWLRQSRLQWWKHJUHDWHVWRId, 12db, and En/16. refo extend tend four No. 9 bars the greater of the Therefore, ment length (63 in.) Step 8) from the column development face and the sum of theoretical cutoff point and d: 2 in. = 87.4 in. > 63 in. Controls 60 in. + 27.4 Say, 90 in or 7 ft 6 in. For No. 9 bars: (a) d = 27.4 in. Controls 60 in. + 27.4 Say, 90 in or 7 ft 6 in. For No. 9 bars: (b) 12db = 12(1.128 in.) = 13.54 in. (c) En/16 = (28 ft - 2 ft)/16 = 1.625 ft = 19.5 in. Extend the remainder four No. 9 bars: (b) 12db = 12(1.128 in.) = 13.54 in. (c) En/16 = (28 ft - 2 ft)/16 = 1.625 ft = 19.5 in. length (63 in.) beyond the theoretical cutoff point (5 ft) and d LQEH\RQGWKHLQAHFtion point (11 ft 0 in.).  $\mathcal{E} = 63$  in./12 + 5 ft = 10.25 ft  $\mathcal{E}$  11.0 ft + 27.4 in./12 = 13.3 ft Controls Therefore, extend the remainder bars over the full length of the beam; refer to Fig. E5.16 Spans AB and DE. At the Support E, where there is no negative moment, provide minimum reinforcement of six No. 6 bars extending minimum the development OHQJWKLQ RQERWKVLGHVRIWKHFROXPQ2IWKH six No. 6 bars, extend two bars over the full length to support the stirrups (hanger bars); refer to Fig. E5.16. American Concrete Institute – Copyrighted © Material – www.concrete.org )RUWKHSRVLWLYHPRPHQWUHJLRQ¿YH1RERWWRP bars are extended over the full span length for Beams 1, 3, and 4. Step 12: Splicing and bar spacing 9.7.7.5 Splices are necessary for continuous bars. The bars shall be spliced in accordance with (a) and (b): (a) Positive moment bars shall be spliced at or near the support (b) Negative moment bars shall be spliced at or near midspan 9.7.2.2 Maximum bar spacing at the tension face must not 24.3.1 exceed the lesser of 24.3.2 (40,000) s = 15 | -2.5cc (fs | 239 Beams CHAPTER 7—BEAMS Splice length = (1.3)(development length) No. 11: Edc = 1.3(60 in.) = 78 in. = 6 ft 6 in. No. 6: Edc = 1.3(36 in.) = 46.8 in., say, 48 in. = 4 ft 0 in. No 9: Edc = 1.3(63 in.) = 81.9 in., say, 84 in. = 7 ft 0 in. No. 6: Edc = 1.3(42 in.) = 54.6 in., say, 57 in. = 4 ft 9 in. (40,000 psi) s = 12 | = 12 in. (40,000 psi) s = 12 | = 12 in. (40,000 psi) WUROAH[XUDOFUDFNLQ] 7KLVOLPLWLVLQWHQGHGWRFRQWUROAH[XUDOFUDFNLQ] he cover tto the longitudinal width. Note that cc is the bars, not to the tie. Spacing satisfy the maximum bar sp cing requirement; ement; the spacing therefore, OK American Concrete Institute – Copyrighted Material - www.concrete.org 240 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. E5.16-Longitudinal reinforcement cutoff locations. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS 241 Beams Step 13: Detailing )LJ(2%HDPEDUGHWDLOV1RWH)LJXUHFRQWLQXHGRQQH[WSDJH American Concrete Institute - Copyrighted © Material - www.concrete.org 242 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E5.17(cont.)—Beam bar details. Notes: 30DFH¿UVWVWLUUXSDWLQIURPWKHFROXPQIDFH 2. The contractor may prefer to extend two No.7 top reinforcement over the full beam length to replace the two No. 5 hanger beams. Bars should be spliced at mid-length American Concrete Institute - Copyrighted © Material - www.concrete.org 243 Beams CHAPTER 7-BEAMS Fig. E5.18-Sections. American Concrete Institute - Copyrighted © Material - www.concrete.org 244 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Beam Example 63RVWWHQVLRQHGWUDQVIHUJLUGHUVXSSRUWLQJ¿YHVWRULHVDERYHEXLOWLQWHJUDOO\ZLWKDLQVODE \*LUGHUWHQGRQVZLOOQRWEHVWUHVVHGXQWLOFRQFUHWHFRPSUHVVLYHVWUHQJWKUHDFKHVWKHVSHFL¿HGfci = 4000 psi. Assume the tendon center of gravity at the column. Tendon will be composed of 1/2 in. diameter individually coated and sheathed seven-wire prestressing strands. Given: Load— Service additional dead load D = 15 psf Service live load L = 65 psf Girder, beam and slab self-weights are given below. Material properties— fcg SVLQRUPDOZHLJKWFRQFUHWH fci = 4000 psi fy = 60,000 psi fy
= 60,000 psi fy = 60 girder width: 24 in. Column dimensions: 24 in. x 24 in. Area of 1/2 in. diameter strand = 0.153 in.2 Fig. E6.1—Plan and partial elevation of third level transfer girder and beams. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing \$&,LQWRMREVSHFL¿FDWLRQV Calculation By specifying that the concrete mixtures, the compressive and experience with ACI 301-10 and providing the expoVXUHFODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 5000 psi. The engineer must specify the transfer stress—in this case, 4000 psi. There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Step 2: Beam geometry Girder depth 9.3.1.1 The transfer girder supports a column at midspan. Assume 48 in. deep transfer girder. ght and its The column load includes self-weight rd level, the four stories tributary loads from the third above it, and the roof. Because of this large concenpth lim trated load, the depth limits in Table 9.3.1.1 cannot FXODW EHXVHGDQGFDOFXODWHGGHAHFWLRQVPXVWVDWLVI\WKH VLQ GHAHFWLRQOLPLWVLQ Flange width 9.2.4.2 rde is poured oured monolithically with 6.2.3.1 the slab and will bbehavee as a T-beam. R6.3.2.3 It is allowed per ACI 318-14 comment to ignore the ADQJHZLGWKUHTXLUHPHQWVEDVHGRQH[SHULHQFHDQG past performances. 'HWHUPLQDWLRQRIDQHIIHFWLYHADQJHZLGWKIRU prestressed T-beams is therefore left to the experience DQGMXGJPHQWRIWKHOLFHQVHGGHVLJQSURIHVVLRQDO 6.3.2.1 Use bf = 8h + bw + 8h bf = (8)(7 in.) + 24 in. + (8)(7 in.) = 136 in. American Concrete Institute – Copyrighted © Material – www.concrete.org Beams ACI 318-14 Discussion Step 1: Material requirements of Chapter 2 of SP-17 for an in-depth discussion of the categories and classes. 245 246 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E6.2—Transfer girder geometry. A, in.2 y, in. 1 (2)(56 in.)(7 in.) = 784 2 (48 in.)(24 in.) = 1152 TM 1936 Center of gravity: yt = A(y - yt)2, in.4 3.5 2744 116,690 3201 119,891 24 27,648 79,361 221,184 300,545 30,392 196,051 224,385 420,436 30,392 in.2 = 15.7 in. from top of girder 1936 in.2 yb = 48 in. - 15.7 in. = 32.3 in. from bottom of girder Section modulus: = 420,436 in. = 26,779 in. 15.7 in. = 32.3 in. from bottom of girder Section modulus: = 420,436 in. = 26,779 in. 15.7 in. = 32.3 in. from bottom of girder Section modulus: = 420,436 in. = 26,779 in. 15.7 in. = 32.3 in. Top American Concrete.org Ix, in.4 \*\*Ii, in.4 Ay, in.3 CHAPTER 7-BEAMS 247 6WHS/RDGV Dead load: 7UDQVIHUJLUGHUVHOIZHLJKWZLWKRXWADQJHV Beams Applied load on transfer girder 7KHVHUYLFHOLYHORDGLVSVILQRI¿FHVDQG psf in corridors per Table 4-1 in ASCE 7-10. This example will use 65 psf as an average live load, as the actual layout is not provided. To account for the ZHLJKWRIFHLOLQJVSDUWLWLRQVDQG+9\$&V/VWHPV add 15 psf as miscellaneous dead load. wb = [(24 in.)(48 in.) [(0.150 kip/ft3)/144] = 1.2 kip/ft Slab weight per level: 7 in.] (3 Pcol = (2 ft)(2 ft) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) = 6.85 kip (12 / Typical beam weight (refer to a structure)) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft - | (0.15 kip/ft ) | 12 ft plan): 18 in. × (30 in. - 7 in.) × (36 ft - 2 ft) (18 in.) (23 in.) (34 ft)(0.15 kip/ft 3) = 14.7 kip Pbm = | 12 | | | 12 | | el: Miscellaneous dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 3) = 14.7 kip Pbm = | 12 | | | 12 | | el: Miscellaneous dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 3) = 14.7 kip Pbm = | 12 | | | 12 | | | 12 | | | 12 | | el: Miscellaneous dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level: PSDL = ((14 ft)(36 ft)(0.015 kip/ft 2) = 7.6 kip ed on ggirder: Total dead load per level psf: 36 ft)(0 ft)(0.035)(0 kip/ft2) = 18 kip PLL,Roof ooff = (14 ft)(36 - p ASCE SCE 7 live load with th excepTotal live load per level: (15)L = Lo | 0.25 + | K LL AT / ( ) 15 L = (0.06 kip/ft2) | 0.25 + 0 (0.065 = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | (
2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.038 ksf2 | ( 2016 ft / L = 0.ksf > 0.4Lo = 0.026 ksf OK where L is reduced live load; Lo is unreduced live load; KLL is live load element factor = 4 for internal Reduced third level live load; KLL is ft2 L =  $0.047 \text{ ksf} > 0.4Lo = 0.026 \text{ ksf OK } 2(36 \text{ ft})(14 \text{ ft}) = 1008 \text{ ft} > 400 \text{ ft} 2 \text{ and } L \cdot Lo = 26 \text{ psf Concentrated live load at third level:} [email protected] = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level:} [= (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 19.2 \text{ kip Concentrated live load at third level:} [= (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load applied on girder (four levels):} ^{14} L = (19.2 \text{ kip})(4) + (23.7 \text{ kip Total live load at third level:} [= (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 19.2 \text{ kip Concentrated live load at third level:} [= (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load applied on girder (four levels):} ^{14} L = (19.2 \text{ kip})(4) + (23.7 \text{ kip Total live load at third level:} [= (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = 23.7 \text{ kip Total live load at third level} = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = (14 \text{ ft})(36 \text{ ft})(0.047 \text{ kip/ft}) = (14 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(36 \text{ ft})(3$ kip) + (18 kip) = 119 kip American Concrete Institute – Copyrighted © Material – www.concrete.org 248 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Material properties 20.3.1.1 Post-tensioned strands: ASTM A416 20.3.2.2 fpu = 270,000 psi 20.3.2.5.1 Stress in tendon immediately after force transfer: 0.7 fpu = 189,000 psi 20.3.2.6.1 & RQVLGHULQJSUHVWUHVVORVVHVDVVXPH¿QDOVWUHVVLQ tendons is: 0.65fpu = 175,000 psi R20.3.2.1 24.5.2.1 Modulus of elasticity is assumed for design use: Concrete strength at initial stressing Ep = 28,500,000 psi fcg SVL fcig = 4000 psi  $VVXPHDPD[LPXPFRQFUHWHAH[XUHVWUHVVDV 12 f c' = 530 psi < ft \le 12 f c' = 530 psi < ft \le 12 f c' = 530 psi < ft \le 12 f c' = 530 psi < ft \le 12 f c'$ 7.5 f cci 3 f ci can be exceeded. Location ation Al All Type yp Stress limits at sservice vice loads: 7.5 f ci = 474 psi Load condition L dition Concrete compressive stress limits i d load l Prestress plus sustained 0.45fcg SVL Prestress ress plus total tota load 0.60fcq SVL Step 5: Design assumptions The girder is designed using unbonded single-strand tendons, as it is the preferred construction method in the United States. Bonded tendons may be preferred construction method in the united states. the beam is analyzed as part of a frame. The factored moments and shear forces (required strengths) are calculated at the face of the supports. The moment and shear diagrams at different stages are obtained from PTData software. 7KHJLUGHULVVWUHVVHGLQWZRVWDJHV\$WWKH¿UVWVWUHVVLQJVWDJHIHZWHQGRQVDUHVWUHVVHGDIWHUFRQFUHWH UHDFKHVDFRQFUHWHFRPSUHVVLYHVWUHQJWKRIPLQLPXPSVLDQGVXEMHFWHGRQO\WRLWVVHOIZHLJKW%HIRUH stressing the remaining tendons, the girder will be supporting three levels—dead load only. At the second VWDJHWKHUHPDLQLQJWHQGRQVDUHVWUHVVHGDQGFRQVWUXFWLRQRIWKHXSSHUARRUVFRQWLQXHV American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7-BEAMS 249 Beams Step 6: Post-tensioning design Find the total number of strands required to resist the total load The required prestress force to support the total dead and live load is calculated at service load condition to satisfy the limit in the selected Class in Table 24.5.2.1. The service load and 119 kip concentrated live load is 433 kip concentrated live load is 433 kip concentrated live load and 119 kip concentrated live load is 433 kip concentrated live load and 119 kip concentrated live load is 433 kip concentrated live load is 433 kip concentrated live load is 433 kip concentrated live load and 119 kip concentrated live load is 433 kip concentrated live load live balanced by the harped post-tensioned tendons. From geometry, refer to Fig. E6.3: 0.75(433 kip) = 324.8 kip, say, 325 kip = 2Fsins () 28.3 in. θ = tan -1 | = 10.3 degrees (13 ft)(12 in./ft) |/ F = Calculate number of strands in the tendon. (Step 4) fpe = 0.65fpu = 175 ksi 325 kip = 908 kip 2sin10.3 Required ired tendon area ar a = (908) kip/(175 ksi) = 5.19 in.2 Number mber off strands t d = 55.19 in.2/(0.153 in.2) = 33.9 ds. Therefore,
Therefore, There addressed in PTI TAB.1-06. The lump-sum losses were used herein to determine effective prestress force. ACI 423.3R can be used to FDOFXODWHUH¿QHGWLPHGHSHQGHQWORVVHV )LJ(<sup>2</sup>7HQGRQSUR¿OH Moments at support and midspan for dead load, live load, and post-tensioning are obtained from PTData, Fig. E6.4, 6.5, and 6.6, respectively American Concrete Institute - Copyrighted © Material - www.concrete.org 250 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E6.5—Service live load moment diagram. Fig. E6.5 Tension (+) Note: When comparing compression stresses, absolute values are used. Moment due to balance load at midspan: At transfer: 7 Assume zero dead load present and all 34 diameter M bal = (1805 ft-kip) = 2106 ft-kip) = 2106 ft-kip (910) kip)  $(2106 \text{ ft-kip})(12) \text{ f top} = - + = 0.395 \text{ ksi } 1936 \text{ in.} 2 26, 779 \text{ in.} 3 \text{ Midspan stresses: Top: f top} = - P M + A \text{ Stop Check if actual stress is less than allowable stress (Step 4): Bottom: f bot = - P M - A \text{ Sbot ftop} = 395 \text{ psi} < \text{fall} = 474 \text{ psi f top} = - OK (7/6)(910 \text{ kip}) (2106 \text{ ft-kip})(12) + = -2.49 \text{ ksi } 1936 \text{ in.} 2 13, 017 \text{ in.} 3 \text{ fbot SVL}$  fall = 2400 psi Check if actual stress is less than allowable stress (Step 4): Bottom: f bot = - P M - A \text{ Sbot ftop} = 395 \text{ psi} < \text{fall} = 474 \text{ psi f top} = - OK (7/6)(910 \text{ kip}) (2106 \text{ ft-kip})(12) + = -2.49 \text{ ksi } 1936 \text{ in.} 2 13, 017 \text{ in.} 3 \text{ fbot SVL} if actual stress is less than allowable stress (Step 4): P M + A Sbot Bottom: f bot = -7(-345 ft-kip) = -403 ft-kip(403 ft-kip)(12) - = -0.729 ft-kip(12) ksi 1936 in.2 26, 779 in.3 ftop = 729 psi < fall = 2400 psi f bot = - OK (7/6)(910 kip) (403 ft-kip)(12) + = -0.177 ksi 1936 in.2 13, 017 in.3 Check if actual stress is less than allowable stress (Step 4): fbot = 177 psi < fall = 474 psi OK Conclusion: No stage stressing is required. Stress all tendons after girder beam concrete attained 4000 psi compressive strength. At service: Midspan: MTL = MD + ML Service moment at midspan: MTL = (26 (2618 ft-kip) + (614 ft-kip) = 3232 ft-kip []WNLS MBall []W "M = MTL[]MBal M IWNLS [] "M IWNLS Top: f top = - P M - A Stop f top = - Check if actual st stress iss less than allow allowable ble stress s (Step 4): P M + A Sbot Bottom: f bot = - Check if actual stress is less than allowable stress - Class T (Step 4): Support: 910 kip (1427 (142 ft-kip)(12) - = -1.109 ksi 2 1936 in. 26, 26 779 in.3 ftop = 11 1109 psi < fall = 22 2250 psi f botot = - OK 910 kip (1427 ft-kip)(12) + = 0.845 ksi 1936 13, 017 in.3 19 in.2 fbot = 845 psi < fall = 848 psi Service moment at support: OK 779 in.3 ftop = 414 psi < fall = 2250 psi f bot = - OK 910 kip (124 ft-kip)(12) - = -0.548 ksi 1936 in.2 13, 017 in.3 fbot = 548 psi < fall = 2250 psi American Concrete Institute - Copyrighted © Material - www.concrete.org OK Beams Moment due to balance at support: Support stresses: Top: f top = - 251 252 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Conclusions and summary: All service load stresses are acceptable. Initial stressing: 24.5.2.1 24.5.3.2 2. Top í í 2. Bottom 845 848 2. At service: Support Midspan Step 6: Design strength (a) Flexure Factored loads agrams are obtained from Shear and moment diagrams PTData Fig. E6.7: 5.3.1 VSD 0RPHQWDWPLGVSDQ ts gravity ty only and later The beam resists lateral forces are not considered in this problem. U = 1.4D U = 1.2D + 1.6L From
PTData, secondary moments are: Fr m M From Moment diagrams, Fig. E6.4 1. ft-kip) = 3665 ft-kip Mu = 1.2(2617.7 Controls M2 = 329.5 ft-kip) + 1.6(613.7 ft-kip) = 4123 ft-kip Mu = 1.2(2617.7 Controls M2 = 329.5 ft-kip) = 4123 ft-kip Mu =  $1.2(2617.7 \text{$ Concrete Institute – Copyrighted © Material – www.concrete.org 21.2.1(a) It is assumed that the girder is tension controlled, 0.9 = []This assumption will be checked later. Determine the effective depth assumption will be checked later. Determine the effective depth assumption will be checked later. Determine the effective depth assumption will be checked later. moment strength is: İc = 0.003 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and is approximately 10 to 15 percent of the concrete tensile strength. ACI 318 neglects the concrete tensile strength to calculate nominal strength. strength: 22.2.2.4.1 22.2.2.4.3 253 Beams CHAPTER 7—BEAMS Transfer girder: d = 48 in. - 1.5 in. - (1.128 in.)/2 = 45.4 in istribution is The concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distribution is the concrete compressive stress distr reasonable agreement with the results of comprehensive an ttests, the he Code allow tests. Rather than allows thee use of ect lar compressive stress ress ddistrian equivalent rectangular gZLWKDGHSWKRI GHS bution of 0.85fcgZ unction of concrete concr e compressive stress ress ddistrian equivalent rectangular gZLWKDGHSWKRI GHS bution of 0.85fcgZ unction of concrete concr e compressive stress ress ddistrian equivalent rectangular gZLWKDGHSWKRI GHS bution of 0.85fcgZ unction of concrete concr e compressive stress ress ddistrian equivalent rectangular gZLWKDGHSWKRI GHS bution of 0.85fcgZ unction of concrete concr Table 22.2.2.4.3: For fcg SVL  $\beta 1 = 0.85\ 85 - 0.05(5000\ psi - 4000\ psi)\ 0.05(50 = 0.8\ 1000\ psi$  Fig. E6.8—Section compression block and reinforcement locations. American Concrete Institute – Copyrighted © Material – www.concrete.org 254 20.3.2.4.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) For unbonded tendons and as an alternative to a more accurate calculation, the stress in postWHQVLRQHGWHQGRQVDWQRPLQDOÀH[XUDOVWUHQJWKLV the least of: (a) fse + 10,000 + fcgdp) (b) fse + 60,000 (c) fsy if fse = 175 ksi > 0.5 fpu = 135 ksi En/h = (26 ft)(12)/48 in.) = 6.5 < 35 and where Aps (34)(0.153 in.2)  $\rho p = = 0.00087$  bd p (136 in.)(44 in.) 9.6.2.3 The strands minimum area of GHIRUPHGUHLQIRUFHPHQWLVUHTXLUHGWRHQVXUHAH[ural behavior at nominal girder strength, rather than tied arch behavior. In addition, the reinforcing bar should limit crack width and spacing. To calculate the minimum area: As,min = 0.004Act where Act is the area of that hat part of the cross section HQVLRQ EHWZHHQWKHÀH[XUDOWHQVLRQIDFHDQGWKHFHQWURLG n of the gross section Calculate design moment strength of se section nm n at midspan with only nly PT tendons: C=T 0.85 fcgba = Apsfps + Asfy (a) 175 ksi + 10 ksi + 5 ksi/(100(0.00087)) = 242.5 ksi (b) 175 ksi + 60 ksi = 235 ksi Controls (c) fsy = 0.9 fpu = (0.9) (270 ksi) = 243 ksi Therefore, use fps = 235 ksi At midspan: Act = (24 in.)(48 in. - 15.7 in.) = 775.2 in.2 As, min = (0.004)(775.2 in.2) = 3.16 in.2 As, prov rov = (4)(0.79 a = OK (34)(0.153 in.2)(235 ksi) + (4)(0.79 in.2)(60 ksi)(34 (0.85)(5 ksi)(136 in.) (0.85 a = 2.44 in. < hf = 7 in. 7KHUHIR) 7KHUHIRUHFRPSUHVVLRQEORFNLQADQJH C = 2.44/0.8 = 3.05 Check that section is tension controlled: Is the strain in bars closest to the tension face are greater than 0.005. (d) Deformed bars:  $\varepsilon s = |-1| \varepsilon cu \langle c / (45.4 \text{ in.}) \rangle \varepsilon s = |-1| (0.003) = 0.042 \text{ in./in.} > 0.005 \langle 3.05 \text{ in.} / \text{Alternatively: c/d} = (30.5 \text{ in.})/(44 \text{ in.}) = 0.069 < 3/8 \text{ Therefore}$ use 0.9 = [] Fig. E6.9—Strain distribution over girder depth. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 255 a) a) ( ( M n = Aps f ps | d p - | + As f y | d - | ( ( 2 / 2 / 2.44 in.) ( M n = (5.20in .2)(235 ksi) | 44 in. - | ( 2 / 2.44 in.) ( + (4)(0.79in .2)(60 ksi) | 45.5 in. - | ( 2 / 0.9 = [] calculated above []Mn = (0.9)(52,277 in.-kip + 8395 in.-kip) = 54,605 in.-kip = 54,605 in.-kip = 417 ft-kip Mn = 1.2(298 ft-kip) + 1.6(171 ft-kip) = 1.40 U = 1.40
U = 1.40 U631 ft-kip Controls From PTData, secondary moments: Mu (IWNLSIWNLS (IWNLS 20.3.2.4.1 6WUHVVLQSRVWWHQVLRQHGWHQGRQVDWQRPLQDOÀH[XUDO QHGWHQ strength is the least of: st of (a) fse + 10,000 + fcgd.dp) (a) 175 ksi + 10 ksi + 5 ksi/(100(0.0067)) = 192.0 ksi Controls (b (b) fse + 60,000 (b) 175 ksi + 60 ksi = 23 235 ksi (c (0.9)(270 ksi) = 243 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c (0.9)(270 ksi) = 243 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi > 0.5 fpu = 135 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi (c) fsy (c) fsy (c) fsy = 0.9 fsu = (0.9)(27 if fse = 175 ksi (c) fsy (c) fs support: As,min = 0.004Act At supports: Act = (136 in.)(7 in.) + (24 in.)(8.7 in.) = 1161 in.2 As,min = (0.004)(1161 in.2) = 4.64 in.2 where Act is the area of that part of the cross section As,prov = (4)(1.27 in.2) = 5.08 in.2 Calculate design moment strength of section at midspan with only PT tendons: C=T 0.85 fcgba = Apsfps + Asfy a= OK (34)(0.153 in.2)(192 ksi) + (4)(1.27 in.2)(60 ksi) (0.85)(5 ksi)(24 in.) a = 12.78 in. 7KHUHIRUHFRPSUHVVLRQEORFNLQADQJH C = 12.78/0.8 = 15.98 in. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams &DOFXODWHAH[XUDOVWUHQJWKRIVHFWLRQ 256 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Check that section is tension-controlled: Is the strain in bars closest to the tension face are greater than 0.005? (d) Deformed bars:  $\varepsilon s = |-1|\varepsilon cu \langle c \rangle (45.4 \text{ in.}) \varepsilon s = |-1|(0.003) = 0.0055 \text{ in./in.} > 0.005 \langle 15.98 \text{ in.} \rangle$ Therefore, use 0.9 = []Fig. E6.10 - Strain distribution over girder depth. & DOFXODWHÀH[XUDOVWUHQJWKRIVHFWLRQ a) a) ( (Mn = Aps f ps | dp - | + As f y | d - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 in. - | (1.27 4)(1.27 in .2)(60 ksi) | 45.5 i $(25,868\ 868\ iin.-kip\ k + 11,921\ in.-kip) = 34,010\ 3\ n.-kip\ in.-kip\ []Mn = 22834\ ft-kip\ kip\ < Mu = 302\ ft-kip\ OK\ American\ Concrete\ Institute\ - Copyrighted\ (b)\ Shear\ design\ Shear\ strength\ Vu = (u.2)\ (1.2\ kip/ft)(13\ ft)\ + \ (1.2\ ln)\ (1.2\ kip/ft)(13\ ft)\ + \ (1.2\ kip/ft)(13\ ft$ = 376.4 kip 9.4.3.2 Because conditions a), b), and c) of 9.4.3.2 are VDWLV¿HGWKHGHVLJQVKHDUIRUFHLVWDNHQDWFULWLFDO section at distance h/2 from the face of the support (Fig. E6.11). Fig. E6.11—Factored shear at the critical section. (1.65 kip/ft)(24 in./12) = 373 kip [email Drotected/2 = (3/6.4 kip) - (1/5) ||shear arr = 0.75 21.2.1(b)9.5.1.1 uctio factor: Shear strength reduction []Vn•Vu 9.5.3.1 22.5.1.1 Vn = Vc + Vs 22.5.5.1 22.5.2.1  $\varphi$ Vc =  $\varphi$ 2 f c'bw d d = dp = 0.8h = 0.8(48 in.) = 37.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip
[]Vc•Vu Vc = 97.97.7 kip < [email protected]/2 = 373 kip []Vc•Vu Vc = 97.9 the cross-sectional dimensions satisfy Eq. (22.5.1.2): NG Therefore, shear reinforcement is required.  $Vu \le \varphi(Vc + 8 \text{ f c'bw d}) / 8(5000 \text{ psi})(24 \text{ in.})(38.4 \text{ in.}) Vu \le \varphi | 97.7 \text{ kip} + | 1000 \text{ lb/kip} (J = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le \varphi(Vc + 8 \text{ f c'bw d}) = 489 \text{ kip Vu} = 373 \text{ kip} \le$ www.concrete.org Beams CHAPTER 7—BEAMS 258 22.5.8.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Check if: Apsfse •Apsf pu + Asfy) (0.153 in.2)(34) (175 ksi) = 910 kip 0.4((0.153 in.2)(34)(175 ksi) = 38.4 -1.909 9 -1072.7 684 326 2 .8 374.8 -255 38.4 -4.694 2353 6 -2353.6 684 326 3 373. 373.2 118 38.4 10.078 62.9 15,262.9 684 326 4 371.5 37 491 38.4 2.422 2.4 1852.5 5 6684 326 6 368.2 36 1231 38.4 0.958 0.95 692.9 684 326 7 366.6 6.6 1598 38.4 4 00.734 533.7 37 684 326 8 364.9 1964 38.4 0.595 controlled by Eq. (22.5.8.2c), shown shaded in table above. Both equations are greater than Eq. (22.5.5.1). American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS 259 9.5.3 22.5.10.1 Beams Vu > []Vc at all sections along the girder length. Therefore, shear reinforcement is required. Try No. 4 stirrups; As = 2(0.2 in.2) = 0.4 in.2 and dp,min = 38.4 in. x, ft Vu, kip []Vc Eq. (22.5.8.2a) and (22.5.8.2c), kip 1 377 244 132 5.23 5 2 375 244 130 5.30 5 3 373 244 122 5.66 5 8 365 244 121 5.73 5 9 363 244 119 5.81 5 10 362 243 119 5.82 5 11 360 216 144 4.81 4 12 358 207 151  $4.72 4 13 357 209 148 5.10 4 14 355 212 143 5.23 .2 4 s = [Vs = Vu(Vc, kip \phi Av f y d p Vu - \phi Vc, in. sprov'd., in. Shear reinforcement ach section on where Vu > [V] c is required at each Vs ≥ Vu - Vc <math>\phi$  22.5.10.5.3 22.5.10.5.3 22.5.10.5.3 vhere Vs = Avfytd/s 9.7.6.2.2 cing: Calculate maximum allowable stirrup spacing: First, does the beam transverse reinforcement value need to exceed the threshold value? p 0 75 = 2282 kip Vs = 212 kip/0.75 Vs  $\leq 4$  f c'bw d = 4(5000 psi)(24 in.)(38.4 in.) = 260 kip The required shear strength is less than the threshold value; therefore, provide maximum stirrup spacing as the lesser of 3h/8 and 12 in. Vs = 282 kip > 4 f c'bw d = 261 kip OK 3h/8 = (3)(48 in.)/8 = 18 in. > d/4 = 12 in. Controls 30DFH¿UVW1RVWLUUXSDWLQIURPWKHIDFHRI support. Place No. 4 stirrups at 4 in. on center in the middle 6 ft of the girder. Place No. 4 stirrups at 5 in. on center on both sides of the midsection. American Concrete Institute – Copyrighted © Material – www.concrete.org 260 9.6.3.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6SHFL $\dot{c}$ HGVKHDUUHLQIRUFHPHQWPXVWEHWKHOHDVWRI the greater of (c) and (d) and (e) (c) 0.75 f c' (d) 50 (e) bw , and f yt 80 f yt d s Av ,min bw f yt Aps f pu Av ,min s d bw  $\geq$  0.75 5000 psi = 50 24 in. = 0.021 in.2 /in. 60,000 psi 24 in. = 0.02 in.2 /in. 60,000 psi (0.153 in.2)(34)(270 ksi) 38.4 in. = 0.01 in.2 /in. 80(60 ksi)(38.4 in.) 24 in. Controls to develop ductile behavior. Provided: 5 in. vSDFLQJVDWLVcHV.: OK American Concrete Institute – Copyrighted © Material – www.concrete.org Step 7: Reinforcement detailing Minimum longitudinal bar spacing 25.2.1 The clear spacing between longitudinal No. 10 bars: 1 in. | Clearing spacing greater of db | 4/3(d) agg [ Check if four No.10 bars (resisting positive moment) can be placed in the beam's web. Top bar layout: bw, req'd = 2(cover + dstirrup +1.0 in.) + 3db +3(1.5 in.)min, spacing (25.2.1) 261 1 in. 1.27 in. Controls 4/3(3/4 in.) = 1 in. assuming a 3/4 in. maximum aggregate size Therefore, clear spacing between horizontal bars must be at least 1.27 in. + 4.5 in. = 14.3 in. + 4.5 in. = 14.3 in. + 24 in. Therefore, clear spacing between horizontal bars must be at least 1.27 in. + 0.5 in. + 1.0 in.) + 3.81 in. + 4.5 in. = 14.3 in. + 2.5 in. = 14.3 in. = 14.3 in. = 14.3 in. = 14.3 in 7HQVLRQUHLQIRUFHPHQWLQADQJHVPXVWEHGLVWULEXWHGZLWKLQWKHHIIHFWLYHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded
reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKH[FHHGVE cement is required in the outer tional bonded reinforcement is required in the outer tional bonded reinforcement jH SRUWLRQRIWKHADQJHZLGWKHI] iddepth for additional onal Use No. 5 placed cem bonded reinforcement. nt is to control cracking in n the slab This requirement aci of bars across the full ll ef due to wide spacing effective GWR WHFWADQJHLIUHL IRUFHPHQW ADQJHLIUHL IRUFHPHQW ith thee web width. is concentrated within Fig. E6.12— Bottom reinforcement layout: Bottom bar layout: bw,req'd = 2(cover+ dstir + 1.0 in.) + 3 db + 3(1.5 in.) bw,req'd = 2(1.5 in.+0.5 in.+1.0 in.) + 3 in. + 4.5 in. = 13.5 in. < 24 in. OK American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7—BEAMS 262 9.7.2.2 24.3.1 24.3.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Maximum bar spacing at the tension face must not exceed the lesser of (40, 000) s = 15 | - 2.5cc f s | / (40,000 psi | S = 15 | - 2.5cc f s | / (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.) = 10 in. (40,000 psi | S = 15 | - 2.5(2 in.bars, not to the tie. Skin reinforcement The transfer girder is 48 in. deep > 36 in. Although the Code does not require skin reinforcement is placed a distance h/2 from the tension face. (40,000 psi) s = 12 = 12 in. (40,000 psi) /RQILWXGLQDOEDUVSDFLQJVDWLVI\WKHPD[LPXPEDU spacing requirement; therefore, OK Use two No. 5 bars each face side as shown in Fig. E6.15 Fig. E6.13—Skin reinforcement in girder. Step 8: Bar cutoff Development length Extend top and bottom deformed bars over the full length of the beam. This will ensure better resistance to creep stresses and control of cracks in the girder. \ Therefore, development length calculation is not required into the girder. A = 25.4.2.2 Ad = 25.4.2 Ad = 2 than 12 in. of fresh concrete below horizontal reinforcePHQW2WKHUZLVHXVH )RUWRSEDUVzt = 1.3; Ed = (1.3)(42.43 db) No. 8 bars: Ed = 42.43(1.0 in.) = 42.43 in. ave lop bars, development length is: 1.3Ed = (1.3)(42.43 in.)(1.27 in.) = 70 in., say, 6 ft 0 in. 9.7.7.5 f y  $\psi$ t  $\psi$ e 20 $\lambda$  f c' db are top bars. development length is: 1.3Ed = (1.3)(42.43 in.)(1.27 in.) = 70 in., say, 6 ft 0 in. 9.7.7.5 f y  $\psi$ t  $\psi$ e 20 $\lambda$  f c' db are top bars. (60,000 psi)(1.0)(1.0) Ad = (db) = 42.43db (20)(1.0) 5000 psi / Girder is a single 26 ft long span; therefore, reinforcement splicing is not required. American Concrete Institute – Copyrighted © Material – www.concrete.org 9.7.7.2 9.7.7.3 25.8.1 263 Integrity reinforcement The girder is an internal member; therefore, reinforcement splicing is not required. 7KLVFRQGLWLRQZDVVDWLV¿HGDERYHE\H[WHQGLQJDOO HLWKHUD RUE RIPXVWEHVDWLV¿HG,QWKLV four No. 10 bars into the support. OK H[DPSOHFRQGLWLRQD LVVDWLV¿HGE\KDYLQJDWOHDVW one-quarter of the positive moment bars, but not less than two bars continuous. Beam structural integrity bars must pass through the region bounded by the longitudinal column bars. Two No. 8 bottom bars are extended and placed between column reinforcement. Because the columns and girder are of the same dimension (24 in.), the two inner girder bars are selected to be the integrity bars. Post-tensioning detailing Anchorages for tendons must develop 95 percent of fpu when condition. 25.9 Post-tensioning anchorage design and detailing is usually provided by the post-tensioning supplier, as well as the detailed tendon layout. Step 9: Detailing )LJ(2%HDPEDUGHWDLOV1RWH3ODFH¿UVWVWLUUXSDWLQIURPWKHFROXPQIDFH American Concrete Institute – Copyrighted © Material www.concrete.org Beams CHAPTER 7—BEAMS 264 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E6.15—Sections. American Concrete beam Design and detail an interior, simply supported precast beam supporting factored concentrated forces of 15 kip located at 4ft 6 in. from each end and a continuously distributed factored force of 4.6 kip/ft. The beam is supported on a 6 in. ledge. Given: Material properties—fcg SVLQRUPDOZHLJKWFRQFUHWH 3 fy = 60,000 psi Load—Pu1 = 15.0 kip at 4 ft 6 in. from each support wu = 4.6 kip/ft Span length: 18 ft Beam width: 14 in. Bearing at support: 6 in. Bearing at concentrated load: 10 in. Fig. E7.1—Simply supported precast concrete beam. ACI 318-14 Discussion Step 1: Material requirements of Chapter 19 (ACI 318 318-14) and structural strength requirements. The designer quirements determines the durability rabil classes. Please refer to Chapter 3 of thiss Ha Handbook in-depth discuss for an in-d gor and nd classes. sion of the categories IH VSHFL¿FDWLRQWKD LVFR \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKD LVFR \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKD LVFR \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL \$&,LVDUHIHUHQFHVSHFL
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Calculation specifying that the concrete mixture shall be in By specif accordance with ACI 301-10 and providing the exposure classes, Chapter 19 (ACI 318-14) requirements lasses, Chapter 19 (ACI 318-14) requirements, and Based ex erien with local 1 l mi i experience mixtures, the compressive VWU QJWK FUHWHLVVS VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW le leastt 40 4000 psi. Assume 22 in. deep beam with American Concrete.org Beams CHAPTER 7—BEAMS 266 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Analysis The beam is simply supported and the loads are symmetrical. Therefore, the maximum shear and moment are located at supports and midspan, respectively. Fig. E7.2—Shear and moment diagrams. Vu , max = wu (A) 2 + (Pu)(x1) 8 Step 4: Bearing 16.2.6.2 The minimum seating length th of the precast beam on the wall ledge is thee greate greater of: En/180 and 3 in. 22.8.3.2 Vu, max = (4.6 kip/ft)(18 ft) + (15 kip) = 56.4 kip 2 M u, max = (4.6 kip/ft)(18 ft) + (15 kip)(4.5 ft) = 254 ft-kip 8 En/180 and - (18 ft)(12 iin./ft)/180 n /f /ft = 1.2 in. < 3 in. vide 6 in., n., therefore therefore OK. Provided h Bearing strength h at seat and con entra Check bearing str strength concentrated load f The supporting surface (ledge) is wider on three of the four sides. Therefore, condition (c) applies: 0.85(4000 psi)(14 in.)(10 in.)/1000 = 285.6 kip >> 56.4 kip OK A 10 in. wide beam rests on the precast beam: 0.85(4000 psi)(14 in.)(10 in.)/1000 = 476 kip >> 15 kip OK A 10 in. wide beam restswww.concrete.org CHAPTER 7—BEAMS 267 Beams Step 5: Moment design 9.3.3.1 The Code does not permit a beam to be designed with steel strain less than 0.004 in./in at nominal strength. The intent is to ensure ductile behavior. In most reinforced beams, such as this example, reinforcing bar strain is not a controlling issue. 21.2.1(a) The design assumption is the beams will be tensioned controlled, 0.9 = []This assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked later. 20.6.1.3.3 Determine the effective depth assumption will be checked la 22.2.2.1 The concrete compressive strain at nominal moment strength is calculated at: İcu = 0.003 22.2.2.2 WKRI 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULd is about 10 to 15 per able property and percent off the ess strength. rength. ACI 31 concrete compressive 318 neglects nsi strength to calculate the concrete tensile calculate nomi nominal strength. Use db = 1 in. cover d = 22 in. - 1.0 in. - 0.375 in. - 1.0 in. /2 = 20.1 in., say, 20 in. uiv concrete compressive stress at nominal strength: 22.2.2.4.3 22.2.1.1 The concrete compressive stress distribution is inelastic at high stress. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in reasonable agreement with the results of compressive strength in r and is obtained from Table 22.2.2.4.3. For fcg"SVL  $u_1 = 0.85$  Find the equivalent concrete compressive depth a by equating the compressive depth a by equating the compression force to the tension force to the tension force to the tension force within the beam cross section: C = T 0.85fcgba = Asfy 0.85(4000 psi)(14 in.) American Concrete Institute – Copyrighted © Material – www.concrete.org 268 9.5.1.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The beam's design strength at each section along its length:  $[Mn \cdot Mu ]$   $Vn \cdot 9u$  Calculate the required reinforcement area: a) ( $\phi M n \ge M u = \phi As f y | d - | \langle 2 \rangle$ 1.26 As (254 ft-kip)(12) = (0.9)(60 ksi)As = 0.79 in. - (2)/A No. 8 bar has a db = 1.0 in. and an As = 0.79 in. 2 As, req'd = 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables,
which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Aid - Analysis Tables, which can be downloaded from: aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Aid - Analysis Tables, which can be downloaded from aspx?ItemID=SP1714DA, four No. 8 bars require a 3.1 in. 2 Ver Reinforced Concrete Design Aid - Analysis Tables, which can be dominimum of 11.5 in. wide beam. Therefore, 14 in. width is adequate. Check if the calculated strain exceeds 0.004 in./in. (tension controlled). 9.3.3.1 a = As fy 0.85 f c'b and c = a  $\beta$ 1 6 s = ((1.26)(4)(0.79 in.2) = 3.98 in. a = 1.26A / 0.85 = 3.98 in. (0.85 0.8 = 4.68 in. c = a/0.85 ZKHUHů1 = 0.85 Et =  $\epsilon$  cu (d - c) c  $\epsilon$ t = 0.003 0 (20 4.68 in.) = 0.01 20 in. -4. 4.68 4. in. Therefore, refo assumption mption of using 0.9 = []s correct. Fig. E7.3—Strain distribution across beam section. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS Minimum reinforcement at every section. along the length of the beam. As , min = 3 f c' As , min = 3 f c' As , min = 3 f c' As , min = 3 4000 psi (14 in.)(20 in.) = 0.93 in.2 Controls 60,000 psi As , min = 200 (14 in.)(20 in.) = 0.93 in.2 Controls 60,000 psi As , min = 200 (14 in.)(20 in.) = 0.93 in.2 Controls 60,000 psi As , min = 34000 psi (14 in.)(20 in.) = 0.93 in.2 Controls 60,000 psi As , min = 200 (14 in.)(20 in.) = 0.93 in.2 Controls 60,000 psi As , min = 0.93 in.2 Controls 60,000 psi As , min = 0.93 in.2 Controls 60,000 psi As , min = 0.93 in.2 Controls 60,000 psi As , min = 0.93 in.2 Controls 60,000 psi As , min = reinforcement area at all positive moment locations. Top reinforcement While not required by Code, top bars are needed to stabilize the beam's stirrups. Use two No. 5 continuous bars. Step 6: Shear design Shear strength 21.2.1(b) tor: Shear strength 21.2.1(b) tor: Shear strength 21.2.1(b) tor: Shear strength reduction factor: 9.5.1.1 []Vn•Vu 9.5.3.1 22.5.1.1 Vn = Vc + Vs 9.4.3.2 Because conditions ion (a), (b), and (c) of 9.4.3.2 4.3.2 are LJQ DUIRUFHLVWDNHQ WGLVWDQFH d from the face of th the support (F (Fig. E7.4). [shearr = 0.7 0.75 Fig. E7.4] + Section. Vu = (56.4 kip) - (4.6 kip/ft)(20 in./12) = 48.7 kip 22.5.5.1 22.5.1.2 Vc = 2 f c bw d Vc = (2)( 4000 psi)(14 in.)(20 in.) = 35.4 kip Check if [VC•Vu [Vc = (0.75)(35.4 kip) = 26.6 kip < Vu = 48.7 kip NG Therefore, shear reinforcement, check if the cross-sectional dimensions satisfy Eq. (0.75)(35.4 kip) = 26.6 kip < Vu = 48.7 kip NG Therefore, shear reinforcement is required. Determine required Vu on each side of Pu /HIWRIPu: VXE = Vu - wux1 - Pu Prior to calculating shear reinforcement, check if the cross-sectional dimensions satisfy Eq. (0.75)(35.4 kip) = 26.6 kip < Vu = 48.7 kip NG Therefore, shear reinforcement is required. (22.5.1.2):  $Vu \le \varphi(Vc + 8 \text{ f c 'bw d}) VXE = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 56.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 36.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 35.7 \text{ kip Vu}, r = 36.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 36.4 \text{ kip} - (4.6 \text{ kip/ft})(4.5 \text{ ft}) = 36.4 \text{ kip} - (4.6 \text{ kip/$ DESIGN HANDBOOK—SP-17(14) 9.5.3 22.5.10.1 Shear reinforcement Transverse reinforcement satisfying equation 22.5.10.1 is required at each section where  $Vu > []Vc 22.5.10.5.3 []Vs \cdot Vu \pm []Vc where \phi Vs = []Vs \cdot 7 kip] - (26.6 kip) = 22.1 kip \phi Av f yt d s Assume a No. 3 bar, two legged stirrup 22.1 kip = (0.75)(2)(0.11 in.2)(60,000 psi)(20 in.) s = (0.75)(2)(0.11 in.2)(2)(0.11 in$ 8.9 in. 22.5.10.5.6 Calculate maximum allowable stirrup spacing: First, does the required transverse reinforcement value exceed the threshold value? Vs = Vs  $\leq 4$  f c'bw d = 4 (4000 psi)(14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1
kip = 29.5 kip (14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 kip (14 in.)(20 in.) = 71 threshold value, the maximum stirrup d = 20 in./ d/2 in./2 = 10 in. lesser of d/2 and 24 in. d = 10 in., therefore, OK Use s = 7 in. < d/2 9.7.6.2.2 u No. 3 stirrups at 7 in. on It is unnecessary to use th of the beam. center over the ful full length mu spacing acing is 10 in., ddetermine termine the Since the maximum value of: []Vn []Vc[]Vs with s = 10 in. ip + 26.6 kip  $\phi$ Vn = 2 (0.75)(2)(0.11 in.2)(60 ksi)(20 in.) (0.75) 10 in. [Vn = 46.4 kip 56.4 kip - 46.4 kip 56.4 kip - 46.4 kip 56.4 kip - 46.4 kip 56.4 kip - 26.6 kip / 2 = 1.6 ft, say, 2 ft 4.6 kip/ft Find  $\varepsilon$ v1 = ([email protected] - 46.4 kip 56.4 kip 56.4 kip - 46.4 kip 56. [Vc/2)/wu A v1 = Compute x1 + &v1 = 4.5 ft + 2 ft = 6.5 ft = 78 in. antil (Vu < 44.5 kip and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and use s = 10 in. until [Vu < 44.5 kip] and u (3 in.+ 35 in.+ 50 in.) = 88 in. > 78 in. OK The beam middle section of length: (18 ft)(12) - 2(88 in.) = 40 in. does not require shear reinforcement. However, extend No. 3 stirrups over the remaining length of 40 in. at 10 in. on center as good practice. Note: It is also good practice to add stirrups near a concentrated load. Place six No. 3 stirrups at 4 in. centered on each concentrated load. American Concrete Institute - Copyrighted © Material - www.concrete.org Beams CHAPTER 7-BEAMS 272 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) 6WHS'HÀHFWLRQ 9.3.2.1, PPHGLDWHGHÀHFWLRQLVFDOFXODWHGXVLQ]HODVWLF GHÀHFWLRQDSSURDFKDQGFRQVLGHULQJFRQFUHWH 24.2.3.1 cracking and reinforcement for calculating stiffness. 24.2.3.4 Modulus of elasticity: 19.2.2.1 Ec = 57, 000 f c' psi (19.2.2.1b) Ec = 57, 000 f c' psi (19.2.1b) The beam also resists a concentrated load of 15 kip or service dead load of 7.2 kip and 3.9 kip service live load.
7KHGHAHFWLRQHTXDWLRQHT inertia ggiven ven by · I is the equivalent .5 equation (25.2.3.5a): [(M)](M) I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I cr |M| I e = |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I g + |1 - |cr| I beam is assumed cracked, therefore, calculate the moment of inertia of the cracked section, Icr. Calculate the concrete uncracked depth, c: nAs (d - c) = where n = bc 2 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(3.16 in.2)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n - 1) As' (c - a') 2 n = 29,000 ksi = 8 3600 ksi (8)(20 in. - c) = Es and Asg LQ2 Ec Solving for c: c = 6. - a) 2 + nAs (d - c) 2 3 Icr = 5975 in.4 American Concrete Institute - Copyrighted © Material - www.concrete.org (14 in.) c 2 2 24.2.2 'HÀHFWLRQGXHWRGLVWULEXWHGORDG  $\Delta$  distr = 5(2.23 kip/ft + 1.2 kip/ft)(18 ft) 4 (12)3 = 0.38 in. 384(3600 ksi)(5975 in.4) 'HÀHFWLRQGXHWRFRQFHQWUDWHGORDG  $\Delta$  conc = (7.2 kip + 3.9 kip)(18 ft) 3 (12)3 = 0.18 in. 28.3(3600 ksi)(5975 in.4) 7RWDOGHÀHFWLRQE/240 24.2.4.1.1 "all. = (18.0 ft)(12 in./ft)/240 = 0.9 in. "all." all. = (18.0 ft)(12 in./ft)/240 = 0.9 in. "all." all. = (18.0 ft)(12 in./ft)/240 = 0.9 in." all." LQ!" LQOK & DOFXODWHORQJWHUPGHÀHFWLRQ  $\lambda \Delta = 24.2.4.1.3 273 \xi 1 + 50 \rho' \lambda \Delta =$  From Table 24.2.4.1.3, the time dependent factor ation of more than 5 years: for sustained load duration h 7KHUHIRUHORQJWHUPGHÀHFWLRQLV WHUP T 3" i 2.0 = 1.8 0.62 in.2 1 + 50 (14 in.)(20 in.) T = (1 + (1 + 1.8)(0.56 .56 in.) = 1.6 in. H7 WHUPGHÀ 1RWH7KHORQJWHUPGHAHFWLRQGXHWRVXVWDLQHG lo ding exceedss E/240. E loading Therefore, it is recommended amb the beam 1 iin. to camber Step 8: Details American Concrete Institute – Copyrighted © Material – www.concrete.org Beams CHAPTER 7—BEAMS 274 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Beam Example 8: Determination of closed ties required for the beam shown to resist shear and torque 'HVLJQDQGGHWDLODVLPSO\VXSSRUWHGSUHFDVWHGJHEHDPVSDQQLQJIWLQ7KHEHDPLVVXEMHFWHGWRDIDFWRUHGORDGRI kip/ft. Structural analysis provided a factored shear and torsion of 61 kip and 53 ft-kip, respectively. The torsional moment is determinate and cannot be redistributed back into the structure. Given: fcg SVLQRUPDOZHLJKWFRQFUHWH 3 fy = 60,000 psi d = 21.5 in. Vu = 61 kip Tu = 53 ft-kip wu = 4.72 kip/ft Fig. 8.1—Beam subjected to determinate torque. ACI 318-14 Discussion Step 1: Section properties Determine section properties ropertie for torsion. 9.2.4.4 Acp = bwh 22.7.6.1 n.) (-3.5 in.) Aoh = (bw -3.5 in.) (h 22.7.6.1 n.) (+2.7.6.1.1 Ao = 0.85Aoh 9.2.4.4 pcp = 2(bw + h) 22.7.6.1 in + h -3.5 in.) ph = 2(bw -3.5 in.) (+2.7.6.1 n.) (-3.5 in.) (+2.7.6.1 n.) (-3.5 in.) (+2.7.6.1 n.)
(+2.7.6.1 n.) (+2256 in. 2 2 Ao =  $0.85(256\ 0.8\ in.\ n.\)$  = 218 in.2 pcp =  $2(16\ 2(\ in.\ +\ 24\ in.\)i)$  = 80 in.  $2(1\ in.\ -\ 3.5\ in.\)$  = 66 in. ph =  $2(16\ (\ (384\ in.2\)\)2\)$  4(0.75)(1.0)(5000\ psi) |  $\phi$ Tcr = 4(0.75) (80 in. |) = 391,000 in.-lb 21.2.1c where torsion strength reduction factor 22.7.4.1 9.5.4.1 0.75 = a Calculate threshold torsion Tth = 0.25Tcr 22.7.7.1 Is section large enough? 22.5.1.2 []Tcr = (391,000 in.-lb)/(12,000 in.-lb)/(12,000 in.-lb)/(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip Because Tu = 53 ft-kip > 8.2 ft-kip Because Tu = 53 ft-kip) = 8.2 ft-kip Because Tu = 53 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip Because Tu = 53 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip Because Tu = 53 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip Because Tu = 53 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip Because Tu = 53 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip (16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip (16 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip (16 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 8.2 ft-kip (16 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 ft-kip)] = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(12 in./ in.2)2] = 0.377 ksi Calculate limit =  $\varphi(2 \text{ f c}' + 8 \text{ f c}')$ /LPLW 0.75(2 + 8)(5000 psi) = 0.53 ksi Is f v2 + f vt2 < limit 0.53 ksi Therefore, section is large enough. American Concrete Institute – Copyrighted © Material – www.concrete.org Step 3: Calculate shear and torsion reinforcement Required shear tie area/spacing:  $9.5.1.1 \text{ Av} (Vu - 2\varphi \lambda \text{ f c'} (bw \text{ d}) 22.5.1.1 = s \varphi \text{ f y d} 22.5.5.1 22.5.10.5.3 21.2.1b 275 \text{ Av} (61 \text{ kip} - 2(0.75)(1.0)(5000 \text{ psi})(21.5 \text{ in.}) = s (0.75)(60,000 \text{ psi})(21.5 \text{ in.})$ + 2At/s) At (53 ft-kip)(12 in./ft) = 0.0324 in. 2/in. s = 0.09 in. 2/in. s = 0.40 in. 2/(0.09 in. 2/in.) = 4.44 in. Use No. 4 ties for which (Av + 2At) = 0.40 in. Calculate s = 0.40/(Av/s + 2At/s). Use 4 in. se reinforceme Check mining the content of the contef yt (A 2A) 3/8 in. OK 8VH1RORQJLWXGLQDOEDUV3ODFH¿YH1RLQ bottom, two No. 5 in top. American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 7—BEAMS 277 Beams Step 5: Beam detailing Notes: %RWWRPUHLQIRUFLQJEDUVVXPPDWLRQRIAH[XUHDQGWRUVLRQUHLQIRUFHPHQWUHTXLUHPHQWV 2. Side reinforcing bar due to torsional moment American Concrete Institute - Copyrighted © Material - www.concrete.org 278 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Beam Example 9: Determine closed ties required for the beam of Example 8 to resist shear and torque Use the same data as that for Example 8, except that the factored torsion of 53 ft-kip is not an equilibrium requirement, but because the structure is indeterminate, can be redistributed if the beam cracks. Given: fcg SVLQRUPDOZHLJKWFRQFUHWH 3 = 1.0 fy = fyt = 60,000 psi bw = 16 in. h = 24 in. Vu = 61 kip Tu = 53 ft kip Fig. E9.1—Beam subjected to torque. ACI 318-14 Discussion Step 1: Section properties torsion. rties for tors 9.2.4.4 Acp = bwh 22.7.6.1 in + h - 3.5 in.) (h - 3.5
in.) (h - 3.5 in old torsion: on: Calculate threshold 2 22.7.4.1a (Acp)  $\phi$ Tth =  $\phi\lambda$  f c' | | pcp / 9.5.1.2 21.2.1c Torsion strength reduction factor 9.5.1.1 0.75 = [Check if Tu > []Tth 22.7.5.1 Calculate cracking torsion 9.5.1.1 Calculate 218 in.2 pcp = 2(16 2( in. + 24 in.) = 80 in. 2(1 in. - 3.5 .5 in. i + 24 2 in. - 3.5 in.) = 66 in. ph = 2(16 ( (384 in.2 ) 2 )  $\phi$ Tth = (0.75)(1.0)( 5000 psi) | = 97, 750 in.-lb ( 80 in. |) []Tth = 8.15 ft-kip Tu IWiNLSOK Design section to resist torsional moment. ( Acp2 )  $\phi$ Tcr =  $\phi$ 4 $\lambda$  f c' | | \ pcp / []Tcr = 32.6 ft-kip Check if Tu > []Tcr? Tu = 53 ft-kip > []Tcr = 32.6 ft-kip In statically indeterminate structures where Tu > []Tcr. Use Tu = 32.6 ft-kip and design for torsional reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 7—BEAMS Check if cross-sectional dimensions are large enough. 2 2 (Vu) (Tu ph) (Vc) || b d || + || 1.7 A2 ||  $\leq \varphi$  || b d + 8 f c' || w w oh 2 () (32.6 ft - kip)(12,000)(66 in.) 61,000 lb || (16 in.)(21.5 in.) || + || || 1.7(256 in.2) 2  $\leq (0.75)(25000 \text{ psi} + 85000 \text{ psi}) (177.3 \text{ psi}) 2 = 292 \text{ psi} \leq 530 \text{ psi}$  OK Therefore, section is large enough. Step 3: Torsional reinforcement Find area/spacing: 2 Ao At f y 9.5.4.1 22.7.6.1.2  $\varphi$ Tn =  $\varphi$  9.5.1.1 22.5.5.1 Calculate shear tie area/spacing: ng:  $\Box$ Vn =  $\Box$ Vc +  $\Box$ Vs • Vu = 61 kip and 22.5.10.5.3 21.2.1b  $\varphi$ Vs =  $\varphi$  s cot  $\theta \ge$  Tu = 32.6 ft-kip =  $0.0199 \text{ in.} 2 / \text{in.} \text{ Av fy d s spaci Calculate total tie area/spacing (Av/s + 2At/s) 9.6.4.2 At (32.6 ft-kip)(12,000) = s 2(0.75)(218 in.2)(60,000 psi)(cot45^\circ) Try No. 4 ties and calculate s: 61,000 lb 6 b - 2 50 5000 psi(16 in.)(21.5 in.) Av 0.75 = (60,000 psi)(21.5 in.) (60 s Av = 0 0.0253 in.2 / \text{in.} 0 s Av A + 2 t = 0.0253 in.2 / \text{in.} + 2(0.0199 in.2 / \text{in.}) s s = 0.065$ in.2 /in. s = 0.4 in.2 = 6.1 in. 0.065 in.2 /in. Use s = 6 in. Torsional longitudinal reinforcement 22.7.6.1b Tn = 2 Ao AA f y ph cot  $\theta$  (32.6 ft-kip)(12 in./ft) 2(218 in.2 ) AA (60 ksi) = cot 45° 0.75 66 in. AE = 1.32 in.2 American Concrete Institute – Copyrighted © Material – www.concrete.org 2 Beams 22.7.7.1 279 280 9.6.4.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The minimum torsional longitudinal reinforcement  $\mathcal{E}$ min, must be at least the lesser of: (a) 5 f c'Acp fy ( 25bw ) f yt (A) - t | ph ( s) fy ( 25bw ) f yt (A) - t | ph ( s) fy ( 2500 psi)(384 in.2 ) ( 25(16 in.) ) 60 ksi ( 66 in.) 60,000 psi ( 66 in.) - | | ( 60,000 psi ) 60,000 psi ) 60,000 psi 60 ksi = 1.82 in.2 A ESURY. = 1.32 in.2 > AEUHT (G = 0.12 in.2 > AEUH reinforcement spacing must n. not exceed the lesser of ph/8 and 12 in. DOEDUV3 8VH1ROROJLWXGLQDOEDUV3ODFH¿YH1RLQ bottom and two No.. 4 in each side face. Excess LQW AH[XUDOFDSDFLW\LQWRSDWd from support can serve n top. in place of threee N No. 4 in Step 4: Beam detailing ) ph/8 = 66 in./8 = 8.25 in. < 12 in. er to Beam Example 8. Refer Notes: 1. Bottom bars summation of moment and torsion reinforcement requirements. 2. Side bar due to torsional moments American Concrete one-way or two-way slabs spanning between columns or walls, or both columns and walls. They can be built out of cast-in-place (CIP) concrete, precast elements with end strips formed of CIP topping,
interconnected precast elements with end strips formed of CIP topping. WRWUDQVIHUZLQGHDUWKTXDNHAXLGRUODWHUDOHDUWKSUHVVXUH forces to the lateral-force-resisting system, such as shear walls, or both (ACI 318-14, Section 12.2). For dual system structures such as shear walls, or both (ACI 318-14, Section 12.2). shear walls deform in a bending mode (cantilever), also as shown in Fig. 8.1(a). Diaphragms maintain compatible deformations between the two systems, thus tying the entire structure together (Fig. ateral support to 8.1(a) and (b)). Diaphragms maintain compatible deformations between the two systems, thus tying the entire structure together (Fig. ateral support to 8.1(a) and (b)). percent of a column axial force must be resisted by uate lateral support to the diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for colu columns n high-rise building diaphragms, but mu must be checked for columns n high-rise building diaphragms, but mu must be checked for columns n high-rise building diaphragms, but mu must be checked for columns n high-rise building diaphragms, but mu must be checked for columns n high-rise building diaphragms, but mu must be checked for columns n high-rise building diaphragms, but mu m Checks can mns (a) Slab-bearing force at face of ccolumns (b) Adequacy of diaphragm slab rreinforcement anchored into columns at edge connections (c) Adequate diaphragm buckling strength to resist the bracing forces 8.2—Material Concrete compressive strength for diaphragms and collectors resisting lateral forces must be at least 3000 psi (ACI 318-14, Section 19.2.1.1). Steel strength for longitudinal and transverse bars is limited to 60,000 psi (ACI 318-14, Section 12.5.1.5). 8.3—Service limits The minimum diaphragm slabs thickness must satisfy the requirements of Section 7.3.1 (ACI 318-14) for two-way slabs. The GLDSKUDJPWKLFNOHVVPXVWDOVREHVXI/FLHOWWRUHVLVWLOSODOH moment, shear, and axial forces (Section 12.5.2.3 of ACI 318-14). 8.4—Analysis Diaphragm slabs must resist gravity loads and lateral in pla force combinations simultaneously. For concrete in-plane slabs, ASCE 7-1 7-10 (Section 12.3.1.2) permits the assumption igid diaphragm if the diaphragm aspect ratio, which is of a rigid n to depth ratio, iss 3 or less for seismic design and 2 the span-to-depth (A or less for wind loading (ASCE 7-10, Section 27.5.4) if the VWUX H KD LJQL¿FDQW KRUL]RQWDO LUUHJXODULWLHV:KLOH VWUXFWXUH KDV QR VLJQL¿FDQW structu es aare expected ected to bbehave inelastically during an structures ake, it is expected ected tha earthquake, that rigid diaphragms will perform elastic lly uunder all ll load co elastically conditions. Diaphragm slabs are commonly deep beam that resists lateral forces HOZLWK DWWKHÀRRUOHYHOZLWKV\VWHPFROXPQVDQGZDOOVDFWLQJDV supports forr the deep beam. he diaphragm reinforcement resisting tension due to The AH[XUH AH[XUH LLV SODFHG DW WKH WHQVLRQ HG]H SHUSHQGLFXODU WR WKH Fig. 8.1—Shear wall and moment frame dual system deformation. American Concrete Institute - Copyrighted © Material - www.concrete.org Diaphragms CHAPTER 8-DIAPHRAGMS 282 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 8.4a—Diaphragm tension-compression and shear forces. DSSOLHGIRUFH7HQVLRQHGJHVDUHLGHQWLcHG as chords. Because earthquake and wind forces are reversible, equal reinforcement should be provided at both chords (Fig. 8.4a). Building edge beams, if provided, are often designed as the diaphragm's chord (Section 8.4.1 of this Handbook). &KRUGV DUH DVVXPHG WR UHVLVW DOO WKH ÀH[XUDO WHQVLRQ IURP the diaphragm in-plane bending moment resulting from the lateral load. If edge beams are not provided, the slab acts as a deep rectangular beam resisting bending in the plane of the slab, with the chord tension reinforcement placed within h/4 of the tension face, where h is the diaphragm shear forces are resisted by the system columns and walls. The beams or slab sections that transfer VKHDUDUHLGHQWL¿HGDVFROOHFWRUV7KHF according to their relative lateral stiffness. Flexible diaphragms are modeled with rigid supports (Fig. 8.4b(b)). If all supports have equal )LJE<sup>2</sup>5LJLGDQGÀH[LEOHGLDSKUDJP American Concrete Institute - Copyrighted © Material - www.concrete.org 283 Fig. 8.4.1a—Collector having same width as shear wall— forces are reversible. Fig. 8.4c—Rigid diaphragm lateral ter force rce distribution. lateral resistance, the lateral force rce can be distributed tto the he tributary butary areas. columns and walls according to their G WK G E \$OVR¿QLWHHOHPHQWDQGVWUXWDQGWLHPHWKRGFDQEHXVHG WR DQDO\]H GLDSKUDJPV 7KH ¿QLWH HOHPHQW PHWKRG VKRXOG FRQVLGHU GLDSKUDJP AH[LELOLW\ 6HFWLRQ RI \$&, 318-14). 8.4.1 Collectors, are designed to collect lateral forces from the diaphragm and transfer them to the seismicforce-resisting system, or to transfer lateral loads from a shear wall into the diaphragm. Collectors can be the full length of the diaphragm, but not necessarily (ACI 318-14, 6HFWLRQ & ROOHFWRUV FDQ EH GH¿QHG DV D VHFWLRQ within the depth of the diaphragm or as a beam as part of the diaphragm. Collectors, as part of a rigid diaphragm, are expected to perform elastically during an earthquake event. Collectors parallel to a shear wall (an have the same width as a shear wall (Fig. 8.4.1a) or be wider. Collectors eccentric to the wall plus one-half the length of the shear wall (Seismology and Structural Standards Committee 6(\$2& \$&,6HFWLRQ5)LJERI this Handbook). Collectors with the same width as the wall will simply transfer the slab lateral forces by axial compression or tension to the shear wall. Collectors having a width wider than the shear wall length through shear fric-Fig. 8.4.1b—Collector wider than the shear wall—forces are reversible. tion. An eccentricity results between the resultant force in the sole transfer UHJLRQDGMDFHQWWRWKHZDOO\$GHTXDWHUHLQIRUFHPHQWPXVWEH SURYLGHGWRUHVLVWWKHVHVWUHVVHV6(\$2& Collectors, like rigid diaphragms, are expected to behave elastically under axial and compression forces. Reinforcement is usually placed at mid-depth in collectors, and is placed ZLWKLQWKHVODEWKLFNQHVV6(\$2& 8.5—Design strength Diaphragms in Seismic Design Categories (SDCs) D through F are designed for stability, strength, and stiffness under factored load combinations; its thickness must be at least that required of that member (ACI 318-14. Diaphragms in Seismic Design Categories) D through F are designed for stability, strength, and stiffness under factored load combinations; its thickness must be at least that required of that member (ACI 318-14. Diaphragms in Seismic Design Categories) D through F are designed for stability, strength and stiffness under factored load combinations; its thickness must be at least that required of that member (ACI 318-14. Diaphragms in Seismic Design Categories) D through F are designed for stability, strength are designed for stability, strength are designed for stability, strength are designed for stability, strength are designed for stability, strength are designed for stability. 14. Section 12.3.1). The shear forces and bending moments American Concrete Institute - Copyrighted ©
Material - www.concrete.org Diaphragms CHAPTER 8-DIAPHRAGMS 284 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) resulting from the effects of lateral loads are considered simultaneously. Diaphragms are designed to resist the design seismic force calculated from the structural analysis, Fpx, which must be at least (ASCE/SEI 7-10, Section 12.10.1.1): n Fpx =  $\Sigma$  Fi i=x n  $\Sigma$  Wi W px i= x where Fpx is the diaphragm design force at level x. The design force at level x. The design force at level x is Fi; Wi is the weight tributary to the diaphragm at a start of the diaphragm design force at level x. The design force at level x. The design force at level x is Fi; Wi is the weight tributary to the diaphragm design force at level x. The design force at level x. level x. The force calculated from this equation need not exceed 0.45SDSIWpx, but needs to be at least 0.25SDSIWpx (ASCE/SEI 7-10, Section 12.10.1.1). Collectors in SDCs C through F are designed for the largest of (a) through (c): (a) Fx obtained from structural analysis using load combiQDWLRQV ZLWK RYHUVWUHQJWK IDFWRU of ASCE/SEI 7-10, Section 12.10.1.1). 7-10, Section 12.4.3.2 rstrength factor (b) Fpx using load combinations of 3) fo ASCE/SEI 7-10, Section 12.4.2.3) forces Fx are applied to DOOARRUOHYHOVFRQFXUUHQWO\ applied one level at a time me Forces Fpx and Fpx,min "are ap ns ion (Moehle eet al. to the diaphragm under consideration tre of a diaphrag m is 2010)," The nominal shear strength diaphragm 8- Section ection 12.5.3.3 and nd Vn = Acv (2) f c' + pt f y) (ACI 318-14, us satisfy f Vn ≤ 8 f c'Acv the cross-sectional dimensions must (ACI 318-14, Section 12.5.3.4). In ACI 318-14, Sections 12.5.3.3 and 12.5.3.3 and 12.5.3.4) Acv is the ss gross area of concrete section bounded by web thickness and section length in the direction of shear force considered, DQGdt is the ratio of area of distributed transverse reinforcement to gross concrete area positioned perpendicular to the GLDSKUDJPAH[XUDOUHLQIRUFHPHQW7KHUHGXFWLRQIDFWRU[RU diaphragms is 0.75 (ACI 318-14, Section 12.5.3.2). In SDC ' (RU) [FDQQRW H[FHHG WKH YDOXH FRUUHVSRQGLQ] WR WKH seismic-force-resisting system it is part of if the nominal strength of the member. The least value is 0.6 (ACI 318-14, Section 21.2.4.1). At diaphragm discontinuities, such as openings and reentrant corners, the design needs to consider the dissipation or transfer of edge (chord) forces. When combined with other forces in the diaphragm, the local design strengths should be within the shear and torsion strength of the diaphragm. 8.6—Reinforcement detailing Generally, chord and collector reinforcement is placed around diaphragm mid-depth. It is common practice (Moehle et al. 2010) to reinforcement, but at least one bar on any side. /DUJHURSHOLOJVUHTXLUHDPRUHULJRURXVDODO\VLV Fig. 8.6a—Reinforcement detail around opening within diaphragm. 8.6a—Collector and chord reinforcement Table 8.6a—Co requirements SDCs ments in SD s D through F for splice R Requirement cing Spacing 33db+LQ ver Cover 22.5db+LQ 0.75 f c' 0 Transverse Greater t off 50bw s f yt bw s f yt ACI 318-14 18.12.7.6(a) 18.12.7.6(b) \$URXQG ODUJH RSHQLQJV RU RWKHU GLVFRQWLQXLWLHV FRQ¿QHment reinforcement (ties) should be placed around the chord bars surrounding the opening (Fig. 8.6a). To properly transfer forces between the diaphragm and columns or walls, chord bar splices should be Type 2 and chord bar spacing should satisfy the requirements of Table 8.6a. Chord bars in KLJKHUVHLVPLF]RQHVPXVWEHFRQ¿QHGZLWKFORVHGKRRSVRU spirals per Table 8.6b. Chords at openings need to be proportioned to resist the sum of the factored axial forces acting in the plane of the diaphragm and the force obtained by dividing the factored moment at the section by the distance between the chords at the section. A collector parallel to a shear wall has its critical connection at the face of the shear wall to develop and transfer the lateral force to wall reinforcement (Fig. 8.6b). The collector reinforcement is in addition to the horizontal diaphragm reinforcement required to resist the shear force (Moehle et al. 2010). Collector reinforcement must comply with Section 20.2.1 of ACI 318-14 with two exceptions: (a) Collector or chord reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams must satisfy ASTM A706/706M, Grade 60. Reinforcement placed within beams mu Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS 285 Table 8.6b—Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and compressive stress Transverse reinforcement requirements for tension and comp requirements Details ACI 318-14 Diaphragms Single or overlapping spirals per Sections 25.7.3.5 and 25.7.3.5 and 25.7.3.6 of ACI 318-14 Circular hoops or rectangular hoops or rectangular hoops with or without crossties spaced not more than 14 in. 18.12.7.5 > 0.2fcq (> 0.5fcq if forces are DPSOL¿HGWRDFFRXQWIRU over-strength) Yes ment spacing aalong lengt Transverse reinfo reinforcement length of the diaphragm iis the smallest of: D 2QHIRXUWKP PXPPHPEHUGLP VLRQ D 2QHIRXUWKPLQLPXPPHPEHUGLPHQVLRQ (b) 6db of smallest malle longitudinal reinfor cement men 14 - hx DQGLQ  $\cdot$  so" "LQ (c) so = 4 + 3 Rectilinear hoop Spiral or circular hoop Ash = 0.09 sbc f c' f yt (Ag) f'  $\rho$ s = 0.45 | -1 c \ Ach |/ f yt No 18.12.7.5 Greater of:  $\rho s = 0.12$  Fu = 52 kip OK NS:  $\Box Vn = 513$  kip >> Fu = 70 kip OK EW: T refo diaphragm phragm ha Therefore, has adequate strength to resist th later the lateral inertia fforce and shear reinforcement is not uired dt = 0. required; 12.5.3.4 th Vn, must nnot exceed: The nominal shear sstrength, 8(72 72 ft)(10 in.)(12 in in./ft)(1.0) 4000 psi = 4372 kip 1000 lb/kip Vn = 8 Acv  $\lambda$  f c Vn = Acv is the diaphragm gross area less the 6.0 ft overhang. %\LQVSHFWLRQWKLVLVVDWLV2HGOK American Concrete Institute – Copyrighted © Material – www.concrete.org Diaphragms 12.5.3.3 296 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Diaphragm lateral force distribution N-S /DWHUDOIRUFHLVGLVWULEXWHGWRWKHZDOOVDVIROORZV refer to Fig. E1.4: 12.4.2.4(a) 12.5.1.3(a) Diaphragm is a beam supported by idealized rigid supports with a depth equal to full diaphragm depth. The wall and frame forces and the assumed direction of torque due to the eccentricity are shown in Fig. E1.4. 12.4.2.4 The diaphragm reactions per unit length (Fig. E1.4.): Fig. E1.4.

systems in the N-S direction. Design force: 95 kip Force equilibrium (L) (L) qL | + qR | = Fpx, deses (NS) (2/(2/(3/) From statics solvee equations e ions (I) systems in the N-S direction. Design force: 95 kip Force equilibrium (L) (L) (QL | + qR | = 95 kip (2 | / 2 | / (1) Moment equilibrium rium (L) (L) (QL | + qR | = 95 kip (2 | / 2 | / (1) Moment equilibrium rium (L) (QL | + qR | = 95 kip (2 | / (2 | / (1) Moment equilibrium rium (L) (QL | + qR | = 95 kip (2 | / (2 | / (1) Moment equilibrium rium (QL | + qR | = 95 kip (2 | / (2 | / (1) Moment equilibrium rium (QL | + qR | = 95 kip (2 | / (2 | / (2 | / (1) Moment equilibrium rium (QL | + qR | = 95 kip (2 | / (2 | / (2 | / (1) Moment equilibrium rium (QL | + qR | = 95 kip (2 | / (2 | / (2 | / (1) Moment equilibrium rium (QL | + qR | = 95 kip (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / (2 | / ( and (II) ffor qL and qR: qL (218 ft )2 + q 2 (218 ft )2 + q 2 (218 ft )2 + q 2 (218 ft )2 + q R = 0.87 kip/ft 44 kip/ft (II)  $\frac{1}{2}$  (I)  $\frac$ derivative of the moment equation expressed as a function of x (unknown distance) dM/dx = 0 297 x = 114.3 ft Mmax = 2595 ft-kip Diaphragms Draw the shear and moment diagrams (Fig. E1.5). Note: In an Aug. 2010 National Institute and Standards Technology (NIST) report, GCR 10917-4, "Seismic Design of Cast-in-Place Concrete Diaphragms," Chords, and Collectors," by Moehle et al. states that, "This approach leaves any moment due to the frame forces along column lines A and F unresolved. Sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." In this example the small moment due to the frame forces (0.2 kip) is ignored. Fig. E1. E1.5—Shear Shear an and dm moment diagrams. ced engineers neers sometime implify Note: Experienced sometimes simplify ributing the load uniformly: the calculationss bby distributing axim moment of: Resulting in a maximum q = 95 kip 44 kip/ft (21 kip/ft)(218 ft) 2 = 2614 ft-kip 8 Note: Both approaches, in this example, result in close maximum moments m (less than 1 percent), but at different locations (114.3 ft versus 109 ft). M max = Shear diagram for the second approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed approach is a straight line with equal shear force at both ends. In this example, the detailed 2595 ft-kip Chord reinforcement resisting tension must be located within h/4 of the tension edge of the balcony or it can be placed along WKHH[WHULRUIUDPHRI&/\$ Placing chord reinforcement along the exterior frame is a simpler and cleaner load path for the forces in the diaphragm. Crack control reinforcement should be added in the balcony slab for crack control. American Concrete Institute – Copyrighted © Material – www.concrete.org 298 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Assume tension reinforcement is placed in a 3 ft strip at both north and south sides of the slab edges DW&/V\$DQG) 3 ft < h/4 = 18 ft OK Chord forces are usually highest furthest from the geometric centroid, in this case, edge of the overhang. To prevent cracking, place chord reinforcement at the outside edge of the overhang. The maximum chord tension force is calculated at IWHDVWRI&/ Tu = Mu B - 3 ft Tu = 12.5.2.2 Tension due to moment is resisted by deformed bars conforming to Section 20.2.1 of ACI 318-14. 12.5.1.5 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFLiHG\LHOG strength and 60,000 psi. 12.5.1.1 Required reinforcement []Tn = []fyAs•Tu 22.4.3.1 The building is assigned ssig to SDC B. Therefore, uire s for chord spa g and Chapter 18 requirements spacing for nt of Section 18 2.7.6 of transverse reinforcement 18.12.7.5 RU o for or chord design is not re2YHUVWUHQJWKIDFWRUö quired. Therefore, use the compression stress limit in provision 18.12.7.5 of 0.2fcg 2595 ft-kip = 37.6 kip 72 ft - 3 ft y = 60,000 psi As, req eq 'd = 37,600 lbb = 0.70 in. / HVVWKDQIWDVVXPHG7KHUHIRUHOK Choose reinforcement check if provided reinforcement area is greater than required reinforcement area is greater than required reinforcement. area: Try two No. 6 chord bars. As,prov. = 2(0.44 in.2) = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2 As,prov. = 0.88 in.2
As,prov. = 0.88 in.2 As,p engineer has several options: 1. Place chord reinforcement outside the web width 2. Place chord reinforcement within the web width Fig. E1.6—Chord reinforcement reinfo along CLs A and F. American Concrete Institute - Copyrighted © Material - www.concrete.org Diaphragms The engineer has two options for providing chord reinforcement along the exterior frames: 1. Excess amount of reinforcement used in the beams to resist gravity loads could be used to resist the difference. 2. The chord force is resisted with additional reinforcement. 300 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Collector reinforcement N-S 12.5.4.1 Collector elements transfer shear force: vu @ F = Fu @ F From Step 6: Fu = 52.2 kip B In diaphragm: vu @ F = In wall: vu @ F = Fpx Ldiaph Fpx Lwall vu @ F = 52.2 kip = 0.72 kip/ft(22 ft) = -15.8 kip + (1.14 kip/ft)(22 ft) = -15.8 kip + (1.14 kip/ftkip - (0.7 GXH PEHUURXQGLQ] §NLSGXHWRQXPEHUURXQGLQ] rc diagram, gram, the maxim m ax Per collector force must be transferred diaphragm collector to the shear wall (Fig. E1.7). The building is assigned to SDC B. Therefore, the collector force and its connections to the shear wall will not be multiplied by the system overstrength factor,  $\delta o = 2.5$  (ASCE/SEI 7-10, Table 12.2-1). 12.5.4.2 Collectors are designed as tension members, or both. Fig. E1.7—Collector force diagram. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS Tension is resisted by reinforcement as calculated above. Required reinforcement:  $[Tn = [fyAs \cdot Tu As, req 'd = Tu 16,000 lb = = 0.3 in. 2 0.9 fy (0.9)(60,000 psi)$ \$OWKRX]KRQH1REDUVXI¿FHVWZR1REDUVDUH provided to maintain symmetry of load being transferred from the slab into the wall. 18.12.7.5 Check in the slab into the wall. 18.12.7.5 Check i collector compressive force exceeds 0.2fco. Calculate minimum required collector width using 0.2fco wcoll. > CColl . 0.2 f c'tdiaph This results in compressive stress on the concrete diaphragm collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(Calculate minimum required collector being relatively low. The section is additional reinforcement. wcol in.) Use 12 in. wide collector (same width as shear wall). Provide two No. 5 bars at mid-depth of slab to prevent additional out-of-plane bending stresses in the slab. Space the two No. 5 bars at 8 in. on center starting at 2 in. ffrom the edge of the diaphragm within the 12 in. wide collector/shear wall (Fig. E1.8). ssion an The collector compression and tension forces are ateral force-resisting system transferred to the lateral within its width (she (shear wall). Therefore, no eccenin-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and
no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present an and no-in-plane bendi tricity is present and no-in-plane bendi tricity is pre design collector iss nnot required. Fig. E1.8—Collector reinforcement. 12.5.3.3 Check slab shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  Is the provided shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = (0.75)(2)(1.0) 21.2.4.2$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [ $0.75 = \mathbb{N}c \text{ OENLS} 12.5.1.1$  [0.75 =(from Step 7) 12.5.3.4 By inspection, the diaphragm shear design force VDWLV¿HVWKHUHTXLUHPHQWRI6HFWLRQRI\$&, 318-14. 12.4.2.4 () 5000 psi (28 ft)(12)(7 in.) φVc = φAcv 8λ f c' American Concrete Institute – Copyrighted © Material – www.concrete.org OK Diaphragms 12.5.1.1 22.4.3.1 301 302 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS/DWHUDOIRUFHGLVWULEXWLRQLQGLDSKUDJP(: Design force: 140 kip 12.4.2.4(a) 12.5.1.3(a) Design moments, shear, and axial forces are calculated assuming a simply supported beam with depth equal to full diaphragm length (refer to Fig. E1.9). Fig. E1.9—Seismic forces orc in the lateral force resisting ems in the he E-W dir systems direction. le effect of acci ntal Because of the neg negligible accidental rti force ce is uniformly ddistributed stribu torsion, the inertial hra width (1140 kip) 1 94 kip/ft ki qL = | = 1.94 (72 ft |) Shear ar fo force: V = (1.94 kip/ft)(36 ft) = 70 kip Maximum moment is located at midlength. x = 36 ft Draw the shear and moment diagrams. (Fig. E1.10). M max = wA 2 (1.94 kip/ft)(72 ft) 2 = = 1260 ft-kip 8 8 Fig. E1.10—Shear and moment diagrams. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 12.5.2.3 Mu = 1260 ft-kip Chord reinforcement must be located within h/4 of the tension edge of the diaphragm. h/4 = 218.0 ft / 4 = 54.5 ft Assume tension reinforcement is placed within a 1 ft strip of the slab. 1 ft < h/4 = 54.5 ft Diaphragms Step 12: Chord reinforcement E-W Calculate chord reinforcement Maximum moment is calculated above (Fig. E1.8). 303 OK Chord force The maximum chord tension force is at midspan: Tu = 12.5.2.2 12.5.1.5 12.5.1.1 22.4.3.1 Mu L - 1 ft Tu = 1260 ft-kip = 5.8 kip 218 ft - 1 ft Chord reinforcement Tension due to moment is resisted by deformed EDUVFRQ¿UPLQJWR6HFWLRQRI\$&, SHFL¿HG\LHOG 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFL¿HG\LHOG strength and 60,000 psi. fy = 60,000 psi emen Required reinforcement [] Tn = []fyAs•Tu As, req eq 'd = Tu 5800 lb = 0.1 in.2 = 0.9 fy (0.9)(60,000 psi) (0.9)(6 Along column lin lines 1 and 7, two No. 5 bars ars ccollecut are provided rovided to resist inertia nerti force tor reinforcement tio These has a bars can be used sed for in the N-S direction. ent in the E-W directi refer to chord reinforcement direction (refer Fig. E1.8). As,prov. = (2)(0.31 2)(0.31 1)(0.31 in.2) = 0.62 in.2 > 0.1 in.2 /V 0D[LPXPVKHDULQWKH(:GLUHFWLRQRFFXUVDW&/V 1 and 7: Unit shear force in frame: vu @1,7 = L 70 kip = 0.32 kip/ft 218 ft Step 13: Collector reinforcement & ROOHFWRUDORQJ&/V\$DQG) The continuous reinforced concrete frame over the full length of the building acts as a collector. Note: Provide continuous reinforcement with tension splices (Step 15). 12.5.3.7 In cast-in-place diaphragms, where shear is transferred from the diaphragm to a collector, or from the diaphragm to a collector. adequate to transfer that force. American Concrete Institute – Copyrighted © Material – www.concrete.org OK 304 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 14: Shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement (shrinkage area of shrinkage and temperature reinforcement 12.6.1 The minimum area of shrinkage and temperature reinforcement (shrinkage area of shrinkage (0.0018)(7 in.)(12 in./ft) = 0.15 in.2/ft Spacing of S+T reinforcement is the lesser of 5h and 18 in.: Note: Shrinkage and temperature reinforcement at positive moments at midspans and top reinforcing bars to resist negative moments at columns), continuity between them. (a) 5h = 5(12 in.) = 60 in. (b) 18 in. Controls Step 15: Reinforcement detailing Reinforcement spacing 12.7.2.1 Minimum and maximum spacing of chord and collector reinforcement must satisfy 12.7.2.1 and 12.7.2.2 m spacing of Section 25.2 limits minimum (a) 1 in. (b) 4/3 dagg. (dagg = 3/4 in.) in. (c) db (No. 5) Minimum spacin spacing splice must be at be
at be least the largest of: ud db (a) Three longitudinal (b) 1.5 in. (c) cc+PD[>db, 2 in.] 3(625 in.) = 1.875 in. 3(0.625 11.5 in. trols 2 in. Controls American Concrete Institute – Copyrighted © Material – www.concrete.org Controls CHAPTER 8—DIAPHRAGMS 305 Diaphragms Step 16 Details Fig. E1.11—Diaphragm chord and collector reinforcement. American Concrete Institute – Copyrighted © Material – www.concrete.org 306 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Rigid Diaphragm Example 1 for structure DQGGHVLJQGDWD\$QDO\]HDQGGHVLJQWKHVHFRQGOHYHOARRUGLDSKUDJPZLWKDIWLQ[IWLQRSHQLQJDVVKRZQLQ)LJ E2.1. For diaphragm building elevation, material properties, and design criteria, refer to Diaphragm Example 1. Fig. E2.1—Eight story building. g. ACI 318-14 Discussion scus Step 1: Material requirements Refer to Diaphragm m Example mple 11. Step 2: Slab geometry 6DWLV¿HGSHU'LDSKUDJP([DPSOH 6WHS/DWHUDOIRUFHV For lateral forces and design forces calculations, refer to Diaphragm will be designed for the seismic load in this high and the seismic load in this high and the seismic load in the seismic load in this high and the seismic load in this high and the seismic load in the seismic load in this high and the seismic load in this high and the seismic load in the seismic load in this high and the seismic load in example. East-West (E-W): 116 kip American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8-DIAPHRAGMS 307 Diaphragm: Take the point of origin at F1 (Fig. E2.2). Fig. E2.2-Mass center and rigidity gid center nter location ex excluding uding accidentall tor torsion. 'HWHUPLOH&20 XWK DXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHRSHO OJ7DNLOJWKHPPHOWDUHDDURXOGFR 7KH&20KDVVKLIWHGWRWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVRXWKEHFDXVHRIWKHVR (36 ft)(14 ft) 7KHUHIRUHWKH&20LVORFDWHGDWIWLQHDVWRI&/DQGIWQRUWKRI&/) GIWQRUWKRI / ) GIWARI / ) G ft - 36.8 ft = 1.2 ft Accidental torsion ASCE 7-10 (third edition), commentary Section C12.10.1 requires an additional moment caused by an assumed displacePHQWRI&20\$VKLIWRIPLQLPXPRI¿YHSHUFHQWRIWKHEXLOGLQJGLPHQVLRQSHUSHQGLFXODUWRWKHGLUHFWLRQRIVHLVPLF forces in addition to the actual eccentricity is considered, referred to as accidental eccentricity. ex =  $\pm (0.05)(218 \text{ ft}) = 3.9 \text{ ft}$  ey1 = (0.05)(78 ft) = -3.9 ft 6WHS/DWHUDOV/VWHPVWLIIQHVVFDOFXODWLRQV For wall and moment frame stiffness calculations refer to Diaphragm Example 1, Step 6. American Concrete Institute – Copyrighted © Material – www.concrete.org 308 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Lateral resisting system forces Force in walls and moment-resisting frames are given by the following equations: Fuxi = Fuyi = kix  $\sum$  kix kiy  $\sum$  kiy Fpx ± ki d i Fpx ey  $\sum$  ki di2 Fpy ± ki d i Fpx ey  $\sum$  ki di2 where di is the distance (xi or yi) of each wall from the COR. Fpx, y = 80 kip and  $Fpx_x = 114$  kip are second-story lateral forces obtained from Example 1, Step 3 (Table), N-S and E-W directions, respectively. Mass accidental eccentricities are ex = 10.9 ft + 1.2 ft = -2.7 ft are calculated in Step 4 of this example. The torsional force is calculated by multiplying the lateral inertia force by the corresponding eccentricity: NS: Ty = Fpyex = (81 kip)(±10.9 ft) = ±883 ft-kip EW: Tx1 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 =
Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft) = +592 ft-kip Tx2 = Fpxey1 = (116 kip)(+5.1 ft)2 + (10.5)(109 ft) 2] (2 Fu, wall, min = 8.1 (10.5)(218 ft/2) t/2) (81 kip) (883 ft-kip) = 36.5 p) - (8 ft-k 5 kip p (10.5 + 10.5) (2)[(1.0)(39 ft) 2] 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Therefore, use 58.0 kip. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS 309 Step 7: Check shear force in diaphragm In-plane shear in diaphragm Nominal shear strength in E-W direction The diaphragm slab is cast-in-place concrete, therefore, shear force is calculated from Eq. (12.5.3.3) Vn, N = (218 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn, E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn, E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn , E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn , E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn , E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn , E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv is the diaphragm gross area less the 6.0 ft over- Vn , E = (72 ft)(12 in./ft)(7 in.) 2(1.0) 5000 psi + 0 Vn = Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt = 0 Acv ( $\lambda$  fc' +  $\rho$ t fy) = 2,589,708 lb 2590 kip Ignoring the strength in N-S direction dt =KDQJUHIHUWR6WHSIRUFODUL¿FDWLRQ = 855,316 lb 855 kip (12.5.3.2 21.2.4.2 Design shear strength in E-W direction \$SSO\LQJWKHVKHDUVWUHQJWKUHGXFWLRQIDFWRU0.75 []at the north and south ends along column lines []Vn, N = (0.75)(2590 kip) = 1940 kip A and F. 12.5.3.2 21.4.2.1 \$WWKHHDVWDQGZHVWHQGVDORQJ&ROXPQ/LQHV DQGWKHVKHDUVWUHQJWKUHGXFWLRQIDFWRUBXVW not exceed the least value for shear used for the vertical components of the primary seismic-forceUHVLVWLQJV/VWHP7KHUHIRUHDCheck if factored shear force is less than design shear strength calculated d in Step 7. 12.5.1.1 22.5.1.2) Design shear strength in N-S direction  $\nabla n_E = (0.75)(855 \text{ kip}) = 641 \text{ kip } >> Fu = 58.0 \text{ OK}$  (:  $\nabla n = 641 \text{ kip } >> Fu = 58.0 \text{ OK}$  (:  $\nabla n = 641 \text{ kip } >> Fu = 58.0 \text{ OK}$  (:  $\nabla n = 641 \text{ kip } >> Fu = 58.0 \text{ OK}$  (:  $\nabla n
= 641 \text{ kip } >> Fu = 58.0 \text{ OK}$  (:  $\nabla n = 641 \text{ kip } >> Fu = 58.0 \text{$ 12.5.3.3 (310 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Second-level diaphragm lateral force: 81 kip 12.4.2.4(a) 12.5.1.3(a) Diaphragm depth satisfying equilibrium requirements. The wall forces and the assumed direction of torque due to the eccentricity are shown in Fig. E2.3. 12.4.2.4 The distribution of the applied force on the diaphragm Fig. E2.3.—Forces in the structural resisting systems is calculated by using qL and qR as the left and right due to a seismic force in the N-S direction. diaphragm reactions per unit length (Fig. E2.3): Force equilibrium (L)(L)qL| + qR| = Fpx, des (NS) 2/2/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) 2/21/21/218 ft (218 ft qL| + qR| = 81 kip (I) (21/21/218) ft (218 ft qL| + qR| = 81 kip (I) (21/218) ft PHQWE\WDNLQJW H¿UVW )LQGWKHPD[LPXPPRPHQWE\WDNLQJWKH¿UVW ed as a derivative of the moment equation expressed function of x (unknown distance) dM/dx = 0 qL (218 ft) 2 6 R 6 (218 ft) 2 + q 2 (218 kip/ft x = 114.455 ft; Mmax = 2213 ft-kip ermine Draw the shear and moment diagrams and determine the moment and shear forces at opening (Fig. E2.4). Note: In an Aug. 2010 National Institute and Standards Technology (NIST) report, GCR 10917-4, "Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors," by Moehle et al states that, "This approach leaves any moment due to the frame forces along column lines A and F unresolved. Sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." In this example the small moment due to the frame forces (0.2 kip) are ignored. Fig. E2.4—Shear and moment diagrams. American Concrete Institute - Copyrighted © Material - www.concrete.org (II) CHAPTER 8—DIAPHRAGMS Note: Experienced engineers sometimes simplify the calculations by using uniformly distributed load: q = Resulting in a maximum moment of: M max = 311 81 kip = 0.372 kip/ft 218 ft Note: Both approaches, in this example, result in similar maximum moment (2213 ft-kip versus 2210 ft-kip), but at different locations (114.5 ft versus 109 ft). In this example the detailed approach is presented. Step 9: Chord reinforcement resisting tension must be located within h/4 of the tension edge of the diaphragm. h/4 ft versus 109 ft). = 72.0 ft/4 = 18 ft Note: Chord reinforcement can be placed either in the exterior edge of the balcony or it can be placed along WKHH[WHULRUIUDPHRI&/\$ Placing chord reinforcement should be added in the balcony slab for crack control. Assume tension reinforcement rement is placed in a 2 ft strip at both north and so south sides of the slab edges 2 ft < h/ h/4 = 18 ft OK DW&/VDQG Chord force rd tension on force that mu Maximum chord must be rresisted n is: by the chord at mi midspan Tu = Mu B - 2 ft Tu = 22 ft-kip 2213 kip 3 kip = 31.6 72 ft - 2 ft American Concrete Institute – Copyrighted © Material – www.concrete.org Diaphragms (0.372 kip/ft)(218 ft) 2 = 2210 ft-kip 8 312 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Chord forces at opening in the diaphragm results in local bending of the diaphragm results in local bending of the diaphragm segments on either side of the opening (refer to Fig. E2.5). 1. The diaphragm sections above and below the RSHQLQJDUHLGHDOL]HGDV¿[HGHQGEHDPV 2. The applied loading on the sections above and below the opening are based on the internal moment in the diaphragm secWLRQVDGMDFHQWWRWKHRSHQLQJ 4. The calculated tension and compression secondary chord forces are added to the primary Fig. E2.5—Idealization of sections above and below tension and secondary chord forces. opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening
is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening is located at midlength of the building Force at east and west ends of opening force at east and west ends of opening force at east and west ends of opening force at east and west ends of opening force kip/ft - 0.25 kip/ft)(91 ft) forces at left and right edges of the opening are: qu' @ beg = 0.25 kip/ft + 218 ft = 0.35 kip/ft + 218 ft = 0.35 kip/ft + 218 ft = 0.25 kip/ft)(127 ft) es north and south of the masses qu' @ end = 0.25 kip/ft + 218 ft = 0.35 kip/ft + 218 ft ft 8 percent opening. Therefore, 38 percent and 62 percent of = 0.39 kip/ft the overall applied trape trapezoidal load will be distributed to the north and south section over this portion Force ce north n of opening p g m, respectively. ctively. of the diaphragm, qqg[email protected] (0.35 kip/ft) = 0.13 kip/ft @bN N = (0.38)(0.35 qg[email protected] 0  $39 \text{ kip/ft} \text{ kip/ft} \text{ p} = 0.15 \text{ kip/ft} \text{ (aeN N} = (0.38)(0.39 \text{ ude at the east aand d we The unit forces m magnitude west ends ve us 0.15 of the opening are closee (0.13 kip/ft versus Fo ce south so p g Force of opening) kip/ft versus Fo ce south so p g Force of opening are closee (0.13 kip/ft versus Fo ce south so p g Force of opening) kip/ft versus (0.35 ki kip/ft) = 0.22 kip/ft (abSS = (0.62)(0.35 kip/ft) = 0.22 kip/ft) = 0.22 kip/ft$ south of opening). Therefore, the average unit qo[email protected] = (0.62)(0.39 kip/ft) = 0.24 kip/ft force of 0.14 kip/ft and 0.23 kip/ft will be used for calculating the diaphragm moment segments north and south of the opening (Fig. E2.6). Fixed end moment can be obtained from computer-aided design software programs or from the Reinforced Concrete Design Handbook Design Aid - Analysis Tables, which can be downloaded from: . aspx?ItemID=SP1714DA Fig. E2.6-Moment diagram of sections at opening. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8-DIAPHRAGMS 313 The secondary chord forces are obtained from the moment diagram. Assuming a 1 ft strip (< B/4) at each end of the span between opening and diaphragm edge: Secondary chord force north of opening: 15 ft-kip = 0.7 kip 21 ft Secondary chord force south of opening: Nu D Tu-, op , N = Tu in.2 (0.9)(60,000 psi) Refer to the following discussion. The engineer has two options for providing chord reinforcement at the opening: 1. The beams are designed for 1.2D + 1.6L. For seismic, the governing load combination is (1.2 + 0.2SDS) D + 0.5L + E. The demand from 1.2D + 1.6L is usually higher than the gravity portion of the moments under seismic, (1.2 + 0.2SDS)D + 0.5L. The two loads are proportioned and then the balance reinforcement is used to carry seismic chord/collector forces. 2. The chord force is resisted with additional reinforcement (conservative). VXVHGDVWKHUHTXLUHGUHLQIRUF ,QWKLVH[DPSOHWKH¿UVWRSWLRQLVXVHGDVWKHUHTXLUHGUHLQIRUFHPHQWLVQH]OLJLEOH%HDPWRSUHLQIRUFHPHQWLVQH]OLJLEOH%HDPWRSUHLQIRUFHPHQWLV continuous. Diaphragm edge Total moment to be re resisted is the sum of the main dary chord force and the secondary force: Tu,Total = Tu1 + Tu2 Tu,total,N tal,N N = Cu,total,N N = 31.1 kip + 0.7 kip = 31.8 kip, say, 32 kip 12.5.2.2 12.5.1.5 12.5.1.1 22.4.3.1 Tu, Total = Tu1 + Tu3 Chord reinforcement: n Tension due to moment is resisted by deformed bars conforming to Section 20.2.1 of ACI 318-14. kip + 0.7 kip = 31.8 kip, say, 32 kip Tu, total, S = 31.1 ki 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFL¿HG\LHOG strength and 60,000 psi. fy = 60,000 psi Required reinforcement []Tn = []fyAs•Tu As , req ' d = 32,000 lb = 0.59 in.2 (0.9)(60,000 psi) The building is assigned to SDC B. therefore, ACI 318-14 Chapter 18 requirements for chord design is not required reinforcement of 18.12.7.5 2YHUVWUHQJWKIDFWRUo, for chord design is not required reinforcement of 18.12.7.5 and transverse reinforcement of 18.12.7.5 and transverse reinforcement []Tn = []fyAs•Tu As , req ' d = 32,000 lb = 0.59 in.2 (0.9)(60,000 psi) The building is assigned to SDC B. therefore, ACI 318-14 Chapter 18 required reinforcement of 18.12.7.5 and transverse
reinforcement of 18.12.7.5 and transverse reinforcement of 18.12.7.5 and transverse reinforceme Therefore, use the compression stress limit in Provision 18.12.7.5 of 0.2fcg Required chord width: wchord > CChord 0.2 f c'hdiaph Choose reinforcement area is greater than required reinforcement area is greater than required chord bars. (0.2)(5000 psi)(7 in.) /HVVWKDQWKHDVVXPHGIWOK Try two No. 5 chord bars limit in Provision 18.12.7.5 of 0.2fcg Required chord width: wchord > CChord 0.2 f c'hdiaph Choose reinforcement area is greater than required reinforcement area is greater than required chord bars. As,prov. = 2(0.31 in.2) = 0.62 in.2 As,prov. = 0.62 in.2 As,prov. = 0.62 in.2 As,req'd = 0.59 in.2 American Concrete Institute - Copyrighted © Material - www.concrete.org Diaphragms Tu , opening = 314 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Collector reinforcement N-S 12.5.4.1 Collectors transfer shear forces from the diaphragms to the vertical walls at both east and west ends along column lines 1 and 7 (Fig. E2.2). Collectors extend over the entire diaphragm: vu @ F = Fu @ F From Step 6: Fu = 44.5 kip = 0.62 kip/ft 72 ft vu @ F = 44.5 kip = 1.59 kip/ft 28 ft Force at diaphragm to wall connection Wall west end: F7/D.5 = -(0.62 kip/ft)(22 ft) = -13.6 kip + (0.97 kip/ft)(28 ft) = 13.5 kip + (0.97 kip/ft)(28 ft) = -13.6 kip + (0.97 kip/ft)force must be transferred from diaphragm shear wall (Fig. E2 E2.7). orc and d its connections to o the shear The collector force ied by the syste wall will not be mu multiplied system overASCE/SEI 77-10, Table VWUHQJWKIDFWRUo o = 2.5 (ASCE/SEI 12.2-1), because this is not a special structural wall. 12.5.4.2 12.5.1.1 22.4.3.1 Collectors are designed as tension members, or both. Tension is resisted by reinforcement:  $\Box Tn = \Box fyAs \cdot Tu As$ , reg ' d = Tu 13,600 lb = = 0.25 in.2 0.9 f y (0.9)(60,000 psi) \$OWKRX]KRQH1REDUVXI¿FHVWZR1RDUHSURvided to maintain symmetry. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS Check if collector compressive force exceeds 0.2fcg wcoll . > CColl . 0.2 f c'tdiaph This results in compressive stress on the concrete diaphragm collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. The collector compression and tension forces are transferred to the lateral force-resisting system within its width (shear wall). Therefore, no eccentricity is present and no in-plane bending occurs. wcoll > 13,600 lb = 1.9 in. (0.2)(5000 psi)(7 in.) Use 12 in. wide collector (same width as shear wall). Provide two No. 5 bars at mid-depth of slab to prevent additional out-of-plane bending stresses in the slab. Space the two No. 5 bars at 8 in. on center starting at 2 in. from the edge of the diaphragm within the 12 in. wide collector/shear wall (Fig. E2.8). ACI 318 permits to discontinue the collector along the length of the shear wall where transfer of design collector is not required. Fig. E2.8—Colle E2.8—Colle E2.8—Colle E2.8—Colle E2.8—Colle content. 12.5.3.3 21.2.4.2 Check slab shear strength along walls: L = 28 ft and slab thickness t = 7 in. From  $\varphi Vc = \varphi Acv 2\lambda fc' []$  []Vc = (0.75)(2)(1.0)(5000)(28 ft)(12(7 in.))[]Vc = 249,467 lb ~ 249 kip 12.5.1.1 Is the provided shear strength adequate? [Vc = 249 kip > Vu = 44.5 kip (from Step 7) 12.5.3.4 By inspection, the diaphragm shear design force VDWLV¿HVWKHUHTXLUHPHQWRI  $\varphi$ Vc =  $\varphi$ Acv 8 $\lambda$  f c' 12.4.2.4 American Concrete Institute – Copyrighted © Material – www.concrete.org OK Diaphragms 18.12.7.5 315 316 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS/DWHUDOIRUFHGLVWULEXWLRQLQGLDSKUDJP(: Design force: 114 kip 12.4.2.4(a) 12.5.1.3(a) Design moments, shear, and axial forces are calculated based on a beam with depth equal to full diaphragm length satisfying equilibrium requirements. The wall forces are calculated based on a beam with depth equal to full diaphragm length satisfying equilibrium requirements. The wall forces and the assumed direction of torque due to the eccentricity are shown in Fig. E2.9. The distribution of the applied force on the diaphragm is uniform because of the negligible effect of accidental torsion (Fig. E2.9): F E2 Fig. E2.9—Forces orces in the structural resisting systems ismic for due to a seismic force in the E-W direction. kip / (116 ki qL = | = 1.6 kip/ft \ 72 ft |) Shear force: V = (1.6 kip/ft \ 72 ft |) kip.ft)(36 ft) = 58 kip x = 36 ft; Mmax = 1131 ft-kip American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS 317 Diaphragms Maximum moment is taken at midspan. Draw the shear and moment diagrams. Step 12: Chord reinforcement EEE 2.10. Fig. E2 E2.10. W 12.5.2.3 Maximum moment me is calculated above above: Mu = 11 1131 ft-kip p men resisting isting tension m st be Chord reinforcement must located within h/4 of the tension reinforcement must located within h/4 of the tension reinforcement must located within h/4 of the tension reinforcement must located within h/4 of the tension edge of the slab edges DW&/VDQG 1 ft < h/4 = 54.5 ft OK Chord force Maximum chord tension force that must be resisted by the chord at midspan: Tu = Mu B - 1 ft Tu , 1 = 1131 ft-kip = 5.2 kip (218 ft - 1 ft) American Concrete Institute - Copyrighted © Material - www.concrete.org 318 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Chord forces at opening The</p> opening in the diaphragm results in local bending of the diaphragm segments on either side of the opening (Fig. E2.11). 1. The diaphragm sections to the east and west the opening are based on the relative mass of each section (1:1). Fig. E2.11—Idealization of sections above and below 3. The secondary chord forces are calculated opening. based on the internal moment in the diaphragm secWLRQVDGMDFHQWWRWKHRSHQLQJ Force east and west of opening 4. The calculated tension and compression secondary chord forces are added to the primary ( 1.6 kip/ft ) qu' @ bE = qu' @ b bW = | || = 0.8 kip/ft tension and secondary chord forces. 2 The opening is located at mid-length of the buildLQJARRUSODQLQWKH(:GLUHFWLRQ7KHORDGRQWKH north and south sections of the diaphragm bound by the opening are equal to one half of the overall is portion of the applied trapezoidal load over this diaphragm (Fig. E2.12). Because forces at both ends of openings are close, a uniform load is assu assumed. men can be obtained fro Fixed end moment from com computerig software ware programs or from aided aided design cre Design esign Handbook Design Aid Reinforced Concrete ed at: - Analysis Tables,, w which can be downloaded . aspx?ItemID=SP1714DA Fig. E2.12—Moment diagram of sections at opening. The secondary chord forces are obtained from the moment diagram. Assuming a 2 ft strip (< B/4) at each end of the span between opening and diaphragm edge: Tu,opening = Mu/D Tu+,op ,S
= 13.1 ft-kip = 0.15 kip 89 ft Total moment to be resisted is the sum of the main chord force and the secondary chord force: Tu,total = Tu1 + Tu,O2 Tu,total,N = 5.2 kip + 0.15 kip = 5.35 kip Use 5.4 kip American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS 12.5.1.5 12.5.1.1 22.4.3.1 Chord reinforcement: Tension due to moment will be resisted by deformed bars conforming to Section 20.2.1 of ACI 318-14. 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFL¿HG\LHOG strength and 60,000 psi) 2QH1REDUVDWLV¿HVWKHUHTXLUHPHQW7KHUHTXLUHG collector reinforcement in the N-S direction, however, requires two No. 5 bars. Therefore, provided reinforcement is adequate and no additional reinforcement is required. Step 13: Collector reinforcement is required. Step 13: Collector reinforcement is required. where shear is In cast-in-place diaphragms, he di transferred from the diaphragm to a collector, or agm or collector liector to a sh from the diaphragm to a collector liector to a sh from the diaphragm to a collector liector to a sh from the diaphragm shear wall, d sshrinkage age reinforcement is usually ns that at for adequate to transfer force. rat einforcement shear wall, d sshrinkage age reinforcement rin and temperature 12.6.1 The minimum shrinkage temperature 24.4.3.2 Reinforcement, AS+T: 12.5.3.7 24.4.3.3 AS+T•Ag (0.0018)(7 in.)(12 in./ft) = 0.15 in.2 AS+TT = (0.00 Spacing of S+T reinforcement is the lesser of 5h and 18 in. 5h = 5(12 in.) = 60 in. 18 in. Controls Note: Shrinkage and temperature reinforcement may be part of the main reinforcing bars resisting diaphragm in-plane forces and gravity loads. If provided reinforcement is not continuous (placing bars to resist negative moments at columns), continuity between top and bottom reinforcing bars to resist negative moments at midspans and top reinforcing bars to resist negative moments at columns). between them. American Concrete Institute - Copyrighted © Material - www.concrete.org Diaphragms 12.5.2.2 319 320 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 15: Reinforcement detailing Reinforcement spacing 12.7.2.1 Chord and collector reinforcement minimum and maximum spacing must satisfy 12.7.2.1 and 12.7.2.2. 25.2.1 18.12.7.6a 12.7.2.2 Section 25.2 requires minimum spacing of (a) 1 in. (b) 4/3dagg. (c) db No. 5 Minimum spacing 1.0 in. Controls 4/3(3/4 in.) aggregate = 1.0 in. 0.625 in. (c) cc•PD[>db, 2 in.] 3(0.625 in.) = 1.875 in. 1.5 in. 2 in. Controls Maximum spacing is the smaller of 5h or 18 in. 18 in. Controls Edge reinforcement The opening has four beams are not constructed around the opening perimeterr a m minimum of two No. 5 is rou the opening as sh n in the recommended around shown ex d a minimum of itss develop- Fi Fig. E2.13—Twoo No. 5 reinforcement r around opening. ment length. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8— DIAPHRAGMS 321 Diaphragms Step 16: Details Fig. E2.14—Typical diaphragm to wall section. ODULW\ 1RWH60DEUHLQIRUFPHQWQRWVKRZQIRUFODULW\ Fig. 2.15A—Collector reinforcement in shear walls along CL 1 and 7. American Concrete Institute – Copyrighted © Material – www.concrete.org 322 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 2.15B—Chord reinforcement at supports. Fig. 2.15D—Chord reinforcement at a opening. Fig. 2.15D—Chord reinforcement r at balcony edge. American Concrete Institute – Copyrighted © Material – www.concrete.org 323 Rigid Diaphragm Example 3: Lateral force distribution of a rigid diaphragm to shear walls—A three-story wood apartment building is built on a normalweight reinforced concrete one-story slab. The slab is 200 ft x 90 ft with fcg SVLDQG fy = 60,000 psi. Assume that the structure is located in an active earthquake region Seismic Design Category (SDC) D and WKDWWKHVHLVPLFDQDO\VLVRIWKHVWUXFWXUDODQDO\VLVEDVHGRQ\$6&(UHVXOWLQJLQDEDVHVKHDUFRHI¿FLHQWRI7KHVODE supporting the wood structure is 10 in. thick and the wall lengths, height, and thicknesses are shown as follows. Assume the weight of the wood frame building imparts an equivalent uniform dead load of 135 psf to the slab. In addition, add a 10 psf miscellaneous dead load to the slab. Refer to Fig. E3.1 for geometric information. This example will determine the seismic forces that are resisted by the shear walls, design the diaphragm, chords, and collecWRUVWRUHVLVWWKHVHIRUFHVDQGWUDQVPLWWKHPWRWKHZDOOVDQGWKHQGHWDLOWKHADWZRUNDFFRUGLQJO\ Given: Project data— Diaphragm size 200 ft 0 in. x 10 in. Wall 2: 30 ft 0 in. x 10 in. Wall 3: 30 ft 0 in. x 10 in. X 10 Parking structure (top of slab) height is 12 ft above the foundation Concrete—fcg SVL fy = 60,000 psi Fig. E3.1—Slab —Slab that su supports po a four-story wood building. por Seismic criteria—SDC D CS = 0.316 ateral-force-res ing system are not ot sh Note: Nonparticipating columns in the lateral-force-resisting shown for clarity clarity. ACI 318-14 Discussion Step 1: Material requirements 7.2.2.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements of Chapter 19 and structural strength requirements of Chapter 19 and structural strength requirements (ACI 318-14). specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW 4000 psi \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318-14. ACI encourages referencLQJ\$&,LQWRMREVSHFL¿FDWLRQV There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Step 2: Slab geometry 12.3.1.1 \$VVXPHWKDWGLDSKUDJPWKLFNQHVVVDWLV¿HVWKH requirements for stability, strength, and stiffness under factored load combinations. Given: h = 10 in. American Concrete Institute – Copyrighted © Material – www.concrete.org Diaphragms CHAPTER 8—DIAPHRAGMS 324 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS/DWHUDOIRUFHV The lateral force is obtained by multiplying the self-weight of the reinforced concrete slab, wood frame building dead load, and the contribution RIWKHVKHDUZDOOVE\WKHEDVHVKHDUFRHI¿FLHQW Gravity loads The reinforced concrete slab, wood frame building dead load, and the contribution RIWKHVKHDUZDOOVE\WKHEDVHVKHDUFRHI? f(10 in./12 in/ft)(150 lb/ft3) = 2,250,000 lb = 2250 kip + 10 psf)(200 ft)(90 ft) = 2610 kip = 4860 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip = 4860 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip = 4860 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip = 4860 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame
building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft)(90 ft) = 2610 kip Shear wall self-weight of wood frame building dead load: WD = (135 psf + 10 psf)(200 ft) $= (L)(H/2)(tw)(\hat{U}c)$  W1 = (90 ft)(12 ft/2)(8 in.)/(12 in./ft)(150 lb/ft3) W1 = 54,000 lb = 54 kip W4 = (28 ft)(12 ft/2)(10in.)/(12in./ft)(150 lb/ft3) W1 = 54,000 lb = 30 kip W5 = 30,000 Total gravity dead direction: ead load in the N-S direction Wi = (L)(H/2)(tw)(\hat{U}c) W = 44860 kip + 54 kip + 21 kip + 30 kip = 4965 kip  $W \text{ ft/2} \text{ lb/ft3} W2 = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W2} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ W3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ W3} = 22,500 \text{ lb} = 22.5 \text{ kip} \text{ V3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ kb/ft}) \text{ W3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ kb/ft}) \text{ W3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in.})/(12 \text{ in./ft})(150 \text{ kb/ft}) \text{ w3} = (30 \text{ ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(10 \text{ in./ft})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ ft/2})(12 \text{ f$ from ASCE7-10 Section 12.8.1: V = CsW Cs is calculated using ASCE 7-10 Eq. (12.8-11). The diaphragm design forces Fpx are calculated per ASCE 7-10 Eq. (12.10-1). Fpx and Fpy must be in accordance with ASCE 7-10 Eq. (12.10-2) and (12.10-3). Calculations not shown here as it is outside the scope of this Handbook. Equivalent lateral force at the concrete level is: Fx = 363.1 kip American Concr &RQVHUYDWLYHO\WKHZHLJKWRIDOOZDOOV[]SDUDOOHODQGSHUSHQGLFXODU[]WRWKHGLUHFWLRQRIWKHDQDO\VLVFDQEHLQFOXGHG,QWKLV example, the contribution of wall weights parallel to the applied seismic force is considered in the calculation of diaphragm shears. Walls perpendicular to the applied seismic force are included in determining the lateral force of concrete diaphragms. 6WHS&HQWHURIPDVV&20 Determine center of mass of walls is shown in Table E.1: Table and ycg are the centerr of mass of each wall. For example: nates xcg = 55 ft + 90 ft/2 = 100 ft and y = 90 ft - (8 in./12)/2 = 89.67 ft Wall 1 has the following coordinates: ina i 2)/2 = 0.41 0.417 ft and y = 30 ft + 30 ft ft/2 = 45 ft Wall 2 has the following coordinates: xcgg = 0 ft + (10 in./12)/2 Center of mass of all walls:  $x1 = y1 = \sum$  Wi xcg , i  $\sum$  Wi  $\sum$  Wi ycg ,i  $\Sigma$  Wi = 15,234 ft-kip 101.6 ft =1 150 kip = 7377.2 ft-kip = 49.2 ft 150 kip Center of mass of the slab is: x2 = 200 ft/2 = 45 ft /RFDWLRQRIFHQWHURIPDVVRIWKHVODEDQGZDOOVFRPELQHG xm = (4860 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(100 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  Wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  wi xi = = 100.05 ft W 4860 kip +
150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  wi xi = = 100.05 ft W 4860 kip + 150 kip  $\Sigma$  i and ym = (4860 kip)(45 ft) + (150 kip)(101.6 ft)  $\Sigma$  i and ym = kip)(49.2 ft)  $\Sigma$  Wi yi = = 45.13 ft 4860 kip + 150 kip  $\Sigma$  Wi where 4860 kip and 150 kip are the weight of the slab and walls, respectively. American Concrete Institute – Copyrighted © Material – www.concrete.org 300 7377.2 Diaphragms Fpy = 745 kip Fpx = 726 kip 326 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS&HQWHURIULJLGLW\&25 DQGODWHUDOV\VWHPVWLIIQHVV Determine center of rigidity )URPWKHODWHUDODQDO\VLVWKHGLDSKUDJPLVDVVXPHGULJLGDQGWKHUHIRUHGLDSKUDJPÀH[LELOLW\LVQRWFRQVLGHUHG7KHUHIRUH ODWHUDOIRUFHVDUHGLVWULEXWHGWRVKHDUZDOOVLQERWKGLUHFWLRQVLQSURSRUWLRQWRWKHLUUHODWLYHVWLIIQHVVHV/DWHUDOGLVSODFHPHQW LVWKHVXPRIÀH[XUDODQGVKHDUGLVSODFHPHQWV Apply a lateral force of 1 kip is applied at the top of a cantilevered wall as shown in Fig. E3.2. The wall's lateral displacePHQWXQGHUDXQLWORDGZKLFKLVUHODWHGWRLWVVWLIIQHVVLVWKHVXPRIÀH[XUDODQGVKHDUGLVSODFHPHQWV "Flexure Shear (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± and E = 3,605,000 psi + 3EI AG  $\Delta$  Flexure  $\Delta$  Shear (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± and E = 3,605,000 psi + 3EI AG  $\Delta$  Flexure  $\Delta$  Shear (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph = = AG (Lt) 0.4 E 3 (h) 3P | L/Et )LJ(± (h) 4P | 3 3 L/Ph Ph = = Et 3EI L3t 3E 12 1.2 Ph (1.2) Ph ( ±&DQWLOHYHUZDOOGHÀHFWLRQ Rigidity ki "i (refer to Table E.2) Table E.2. Determining walls' relative stiffnesses t, in. "i × 10-4, in. ki "i × 104, 1/in. 0.1333 8 0.14 7.043 0.4000 10 0.40 2.476 30 0.4000 10 0.40 2.476 28 0.4286 10 0.44 2.252 40 0.3000 10 0.28 3.576 Wall no. Height h, ft /HQJWKL, ft h/L 1 12 90 2 12 30 3 12 4 12 5 12 Table E.3—Determining walls' rigidity Wall no. Direction x, ft y, ft kix kiy (kiy)x (kix)y 631.551 x - 89.677.043 - 2 y 0.417 - 2.476 1.03 - 3 y 199.58 - 2.476 494.16 - 4 x - 10.0 2.252 - 2.252 5 x - 10.0 3.576 - 2.476 494.16 - 4 x - 10.0 2.252 - 2.476 494.16 - 4 x - 10.0 2.252 - 2.52 5 x - 10.0 3.576 - 2.476 1.03 - 3 y 199.58 - 2.476 494.16 - 4 x - 10.0 2.252 - 2.52 5 x - 10.0 3.576 - 2.476 494.16 - 4 x - 10.0 2.252 - 2.52 5 x - 10.0 3.576 - 2.476 494.16 - 4 x - 10.0 2.252 - 2.52 5 x - 10.0 3.576 - 2.476 1.03 - 3 y 199.58 - 2.476 1.03DIAPHRAGMS 327 Calculate the system's center of rigidity:  $\sum$  kiy xi 495.19 ft/ft xr = = 100 ft 4.95/ft  $\sum$  kiy 689.83 ft/ft  $\sum$  kix yi = 53.6 ft 12.87 1/ft  $\sum$  kix vi = 53.6 ft 12.87 1/ft  $\sum$  kix Torsional eccentricity is the difference between the system's center of rigidity and its center of mass (Fig. E3.3): ex = xr - xm = 100.05 ft - 100.02 ft = 0.03 ft, which is negligible ey = yr - ym = 53.6 ft - 45.1 ft = 8.5 ft Fig. E3.3—Locations of the system's m's ce center of mass. s. ASCE 7-10 requires shifting thee ce center of mass. s. ASCE 7 = 0 ft  $\pm (0.05)(200$  ft)  $= \pm 10$  0 ft ey = 8.5 ft  $\pm (0.05)(90$  ft) ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft + 4.5 = 13 ft ey = 88.5 ft - 4.5 ft = 4 ft .5 ft - 4.5 ft = 4 ft .5 ft - 4.5 ft = 4 ft .5 to direct lateral shear force are calculated by:  $Fvx = Fpx Fvy = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: ki xi T 2 x \sum ki xi Ftx = ki yi Fty = \sum ki yi 2 Ty The torsional moment are calculated by: <math>Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Tx = ki yi 2 Ty The torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional
moment are calculated by: Tx = ki yi 2 Ty The torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Tx = ki yi 2 Ty The torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy \sum kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix \sum kix kiy E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix E Kiy E Kiy In-plane wall forces due to torsional moment are calculated by: Fvx = Fpy kix E Kiy E$  $Fpxey1 = (726 lb)(\pm 4 ft) = \pm 2904 ft-kip Tx = Fpxey2 = (726 lb)(\pm 13 ft) = \pm 9438 ft-kip The in-plane diaphragm force is the sum of the direct lateral shear force, Fvi, and the torsional moment, Fti (refer to Tables E.4, E.5. and E.6): Fu = Fvi + Fti Table E.4-Determining wall shear due to seismic forces in the eN N-S direction direction Wall no kix kiy$ ddxi, ft dyi, ft kid ki((d)2 Fvi, kip Fti, kip Ftotal, kip Ftotal, kip 704 0 - 36.08 4.09 254.09 9166.8 0 27.3 - 27.3 2 0 2.476 - 99.66 - 46.56 246.56 553.6 24,553.6 37 372.5 26.5 - 26.5 399.0 4 2.25 0 - 43.6 - 185 - 98.185 4280.2 0 - 10.5 10.5 -10.5 5 3.57 0 - -43.6 - 155.91 6796.4 0 - 16 - 16.7 16.7 - 16.7 16.7 - 16.7 16.7 - 16.7 16.7 = 4.000 ft walls 4 and 5: dyi = 53.6 ft - 10 ft = 43.6 ft Table E.5-Determining wall shear due to seismic forces in the E-W direction ey1 = 4 ft Wall no kix kiy xi, ft yi, ft kid ki(d)2 Fx, kip Fx2\*, kip Fdesign, kip 386.7 1 7.043 0 - 36.08 254.09 9166.8 397.2 -10.6 2 0.00 2.476 -99.6 - -246.565 24.553.7 0.0 10.2 10.2 3 0.00 2.476 99.6 - 246.565 24.553.7 0.0 10.2 10.2 3 0.00 2.476 99.6 - 246.565 24.553.7 0.0 10.2 10.2 3 0.00 2.476 99.6 - 246.565 24.553.7 0.0 -10.2 4 2.252 0 - -43.6 -98.185 4280.2 127.0 4.1 131.1 5 3.576 0 - -43.6 -98.185 4280.2 127.0 4.1 131.1 15 3.576 0 - -43.6 -98.185 4280.2 127.0 4.1 131.1 15 3. seismic forces in the E-W direction ey2 = 13 ft Wall no kix kiy xi, ft yi, ft kid ki(d)2 Fx, kip Fx1\*, kip Fdesign, kip 362.8 1 7.043 0 - 36.08 254.09 9166.8 397.2 - 34.5 2 0.00 2.476 99.6 - 246.565 24,553.7 0.0 33.5 3 0.00 2.476 99.6 - 246.565 24,553.7 0.0 -33.5 4 2.252 0 - 43.6 -98.185 4280.2 127.0 13.3 140.4 5 3.576 0 - 43.6 -155.91 6796.4 201.7 21.2 222.9 12.87 4.952 0 69,350.9 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 329 where d is the distance (dxi or dyi) from the center of each wall to the center of rigidity. Fx1 is the additional shear force due to eccentricity of 13 ft. Fx2 is the additi eccentricity of 4 ft. Notes: • dxi and dyi are the distances of a wall from the center of rigidity in the x- and y-direction. • If torsional moment reduces the magnitude of the direct lateral shear on a wall, then it is ignored. The wall design shear, kip 1 90.00 2 kip N-S, load Design, shear, kip 1 90.00 2 kip N-S, load Design, shear, kip 1 90.00 2 kip N-S, load Design, shear forces are summarized in Table E.7. Wall no. Wall length, ft E-W load, kip N-S, load Design, shear, kip 1 90.00 2 kip N-S, load Design, shear forces are summarized in Table E.7. Wall no. Wall length, ft E-W load, kip N-S, load Design, shear forces are summarized in Table E.7. Wall no. Wall length are the distances of a wall from the center of rigidity in the x- and y-direction.  $30.00\ 387\ 27\ 387\ 33.5\ 399\ 3\ 30.00\ 33.5\ 399\ 3\ 30.00\ 33.5\ 399\ 3\ 90\ 4\ 28.00\ 5\ 40.00\ 140\ 11\ 140\ 223\ 17\ 223\ Step\ 7$ : Diaphragm shear strength is calculated North and south from Eq. (12.5.3.3) Vn = (90\ ft)(12\ in./ft)(10\ in.) (2)(1.0)\ 4000\ psi\ 12.5.3.3\ Vn = Acv\ 2\lambda\ fc'\ +\rho\ fy\ () ,QWKLVH[DPSOH¿UVWFKHFNWKHGLDSKUDJPVWUHQJWK UVWFK eme therefore, refore, ignor without reinforcement; ignore the buti of reinforcement; ignore the buti
of reinforcement; ignore the buti of reinforcement; ignore the but ignore the b ACI 318-14, ] must not exceed the least value for shear used for the vertical components of the primary seismic-force resisting system: 12.5.1.1 22.5.1.2 Check if design shear strength exceeds the factored shear force. 18.12.9.2 The nominal shear force due to seismic forces ) = 1,366,104 1 366 10 lb = 1366 kip East and westt Vn = (2 (200 ft)(12 2 in./ft)(1 in.) (2)(1.0) 4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip > Vu = 387 kip OK 8(10 in.)(90 ft)(12 in./ft) (4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip > Vu = 387 kip OK 8(10 in.)(90 ft)(12 in./ft) (4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip > Vu = 387 kip OK 8(10 in.)(90 ft)(12 in./ft) (4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip > Vu = 387 kip OK 8(10 in.)(90 ft)(12 in./ft) (4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip > Vu = 387 kip OK 8(10 in.)(90 ft)(12 in./ft) (4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip > Vu = 387 kip OK 8(10 in.)(90 ft)(12 in./ft) (4000 psi (= 3,035,787 877 lb = 33036 kip 16] Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip Vn = (0.6)(1366 (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip Vn = (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip Vn = (0.6)(1366 kip) = 1821 kip EW: [V [V = 40.0] Vn = 820 kip Vn = (0.6)(1366 kip) = 1821 kip EW: [V = 40.0] Vn = 1821 kip EW: [ psi 1000 lb/kip Vn, NS = 1366 kip < 5464 kip Vn, EW = 3036 kip < 5464 kip American Concrete Institute - Copyrighted © Material - www.concrete.org OK OK ) = 5464 kip ) 330 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Diaphragm lateral force distribution N-S 12.4.2.4 Diaphragm is assumed rigid (ACI 318-14, Sec12.5.1.3 tion 12.4.2.4(a)). The refore, the diaphragm depth (ACI 318-14, Section (12.5.1.3(a)). The wall forces and the assumed direction of the torsional moment are shown in Fig. E3.4. Refer to previous Step 5 for moment qL = 2.6 2.61 kip/ft, say, 2.6 kip/ft 4.84 kip/ft, ft, say, 4.8 kip/ft qR = 4.8 6 R 6 Note: In an Aug. 2010 National Institute and Standards Technology (NIST) report, GCR 10-917-4, "Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors," by Moehle et al. states that, "This approach leaves any moment due WRWKHIUDPHIRUFHVDORQJFROXPQOLQHV&/ \$DQG F unresolved. Sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." Vmax = 399 kip Mmax = 21,106 ft-kip, say, 21,100 ft-kip Fig. E3.5—Shear and bending moment diagrams due to a lateral seismic force in the N-S direction. Note: Experienced engineers simplify the calculations by using uniformly distributed load: q = 745 kip/200 ft = 3.723 kip/ft, say 3.72 kip/ft Calculate a maximum moment: M max = (3.73 kip/ft)(200 ft) 2 = 18,650 ft-kip 8 American Concrete Institute – Copyrighted © Material – www.concrete.org 331 Notes: • The difference in maximum moment between the two approaches is 13.3 percent and at different locations (110 ft versus 100 ft). • Shear diagram for the second approach is a straight line with maximum shear force: 9 NLSIW IW NLS Step 9: Chord reinforcement N-S R12.1.1 Assume the slab behaves like a beam with compression and tension forces at the near and far edges, respectively: Cchord Tchord = M/d 12.5.2.3 ACI 318 suggests placement of chord reinforcement within an arbitrary width of h/4 of the diaphragm tension edge (Fig. E3.6). h/4 = 90 ft - 1/2(22.5ft) = 78.75 ft 21,100 ft-kip = 268 kip 78.75 ft 21,100 ft-Section 20.2.1 of ACI 318-14. VSHFL¿HG\LHOG 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFL¿HG\LHOG strength and 60,000 psi. fy =
60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi. fy = 60,000 psi tension face. As Assume om arm is approximately ou sides des of the slab edges: es: at both north and south  $(0.95)(90 \ 5)(9 \ ft) = 85 \ 85.5 \ ft$  The calculated tension force is: Tu = Mu 0.95 B Required tension force is: Tu = Mu 0.95in.2 (0.9)60 ksi Per provision 18.12.7.5, the required chord width for the concrete compressive strength limit of 0.2 fc hdiaph wchord > 247 kip = 30.9 in. (0.2)(4000 psi)(10 in.) Note: The chord force does not need to be increased and by the overstrength factor. wchord = 30.9 in. (h/4 = 90 ft/4 = 22.5 ft OK Say, 32 in. Note: Although it is permissible to place bars within 22 ft-6 in. (h/4) of diaphragm width, it is recommended to place bars close to the tension end, where it is most effective. Since load is reversible, chord reinforcePHQWLVSODFHGDWERWKQRUWKDQGVRXWKRIGLDSKUDJPHQGVUHIHUWR)LJ(7KH¿QDOOD\RXWRIEDUVZLOOEH FRRUGLQDWHGZLWKWKHFROOHFWRUUHLQIRUFHPHQWGXHWRLQHUWLDOIRUFHLQWKH(DVW]:HVW(: GLUHFWLRQ American Concrete.org Diaphragms CHAPTER 8-DIAPHRAGMS 332 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. E3.6-Suggested chord reinforcement at the north and south edges of the diaphragm. Step 10: Collectors design N-S Collectors transfer shear forces from the diaphragm to the vertical walls at both east and west ends (Fig. E3.4). Assume collectors transfer shear force path should be designed that is caing aall forces from the diaphragm pable of transmitting to the collector and into the vertical ele elements. 12.5.4.1 agm sshear Unit shear forcee iis the maximum diaph diaphragm gm ddepth, B = 90 ft: divided by the dia diaphragm vu @ F = Fu @ F From Step 6 (Table E E.7): Fu = 399 kip mS B In slab: vu @ F = 399 kip = 4.43 kip/ft 90 ft In wall: vu @ F = 399 kip = 13.3 kip/ft 30 ft Check if the concrete shear strength excluding reinforcement exceeds the factored shear:  $\varphi vc = \varphi 2$  f c'btdiaph  $\varphi vc = (0.6)(2 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{ in.}/\text{ft})(10 \text{ in.}) = 9.1 \text{ kip/ft} 4000 \text{ psi})(12 \text{$ temperature and shrinkage reinforcement in each direction to increase shear capacity (assuming two-way slab).  $\rho t = NG \ 0.31 \ in.2 = 0.00194 \ (10 \ in.)(16 \ in.$ 333 Force at diaphragm to wall connection The proportional diaphragm force that the collector transfers to walls connection is (Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7): Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW
§NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end: []NLSIW IW §NLS Fig. E3.7]: Wall north end the walls is shown wn in Fig. g. E3.7: 18.12.2.1 12.5.4.2 R12.5.4 18.12.7.5 22.4.3 for is then multiplied by by the system Systems ctu walls alls in SDC D (ASCE SCE with special structural 7-10, Table 12.2-1). Tu =  $\circ o TCollIII = \circ o CCColl = (2.5)(133.2 \text{ kip}) = 333 \text{ kip}$ Collectors are designed as both tension and compression members. 7KHUHDUHOREHDPVDOROJWKH&/VRDSRUWLRORI the slab is used as a collector width for tension reinforcement is GHWHUPLOHGE\HOJLOHHULOJMXGJHPHOW\$&, provides in the commentary that a collector width cannot exceed approximately one-half the contact length between the collector and the vertical element measured from the face of the vertical element. beff = 30 ft/2 + 10 in = 15.8 ft = 190 in. The collector width, however, is chosen such that 2.5Ccoll 333 kip wcollector = = 16.7 in. the limiting stresses are not exceeded. When the 0.5 ft (2 ksi)(10 in.) 'c tension and compression collector forces are increased by the overstrength factor, then the limiting wcollector = 16.7 in. < 190 in. concrete compressive collector width: Therefore, part of the slab, beff, is needed to resist the collector force. Required reinforcement area to resist collector force: ΩT 333 kip = 6.2 in.2 As = o coll = [] Tn = []fyAs•Tu 0.9 f y 0.9(60 ksi) Note: Collector reinforcement may be varied along the length of the diaphragm based on required strength and terminated where not required. In this example, the reinforcement is extended over the full length of the diaphragm. American Concrete Institute – Copyrighted © Material - www.concrete.org Diaphragms Wall south end: []NLSIW IW ±NLS 334 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) A number of bars are placed in line with the wall. The balance is distributed across the width of the collector element. In this case, for the 10 inch thick wall, two No. 8 bars in line with the wall will result in a reasonable bar spacing of 6 in. Therefore, two No. 8 bars are centered on the wall for As, line = 1.58 in. 2 The balance of the required reinforcement is: As, bal = 6.2 in. 2/15.0 ft = 0.31 in. 2/ft Try eight No. 5 top and bottom spaced over 15 ft. As, prov. = (2)(8)(0.31 in.2) = 4.96 in. 2 As, prov. = 4.96 in.2 > As, req'd = 4.62 in.2 OK Design collector region: The collector region: The collector and benddiaphragm perpendicular ing in the plane off the ddiaphragm due to eccentric pres orces (Fig. E3.8). E tension and compression forces 9 wall Mu = Tdisteten + Cddistecompp - 9E where Tdist is thee portion orti off the force resisted by As, d s, dist; Cdistt is the p compression collector force resisted by slab outside the wall; and V is the shear strength of the diaphragm. Where the collector element is in tension, the concrete contribution to V is neglected, Vc = 0. [Vs [fyd,ttw(wcomp - twall) Assume No. 5 @ 16 in. on center is provided: For more in-depth understanding refer to: Structural Engineer Association of California 6(\$2& 6HLVPROR] & RPPLWWHH <sup>3</sup>&RQcrete Slab Collectors," from the Aug. 2008 SEAOC %OXH%RRN6HLVPLF'HVLJQ5HFRPPHQGDWLRQV Compilation, Structural Engineers Association of California, Sacramento. Fig. E3.8—Diaphragm segment plan. []Vs = 0.6(0.00193(60 ksi)(10 in.)(16.7 in. - 10 in.) = 4.7 kip pt = 0.31 in.2 = 0.00193 > 0.0018 (10 in.)(16 in.) American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 335 Tension force in the slab (outside the wall geometry) is proportional to collector width: ( 4.62 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 247 kip Tdist = | ( 6.2 in.2 ) (333 kip) = 24 16.7 in. - 10 in. Cdist = | 1/(333 kip) = 137 kip (16.7 in. Moment arm: ecomp = Mu = Tdisteten + Cdistecomp - 9E Mu = (247 kip)(95 in.) + (134 kip)(8.35 in.) - (4.7 kip)(29.5 ft)(12 in.//ft) Mu = 22,920 in.-kip (0.9)(60 ksi)(0.9)(29.5 ft)(12 in./ft) As, reg'd = 1.33 in.2 Use thre three No. o. 6 dowels at each end of the wall. R er to Fig. E3.9. 9 Refer American Concrete Institute – Copyrighted © Material – www.concrete.org Diaphragms (15 ft)(12 in./ft) 10 in. + = 95 in. 2 2 336 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Shear transfer design: A number of bars are placed in line with the wall, which results in transferring portion of the diaphragm force in tension and direct bearing of slab against the wall in compression. The diaphragm and shear transferring portion to collector area. Vu for diaphragm and shear transfer design is then calculated as follows: Vu = 247 kip + 134 kip + (4.43 kip/ft)(30 ft) = 514 kip 6.2 in.2 is the area of two No. 8 bars placed in-line with the shear wall. 12.5.3.7 Shear from the diaphragm is transferred by shear friction to the wall with dowels placed perpendicular to the wall-slab interface: Vn 3\$vffy 22.9.4.2 Assume that diaphragm ragm slab is placed against hardened wall concrete that is clean, free of laitance, te th y rroughened ned to a full am itude of and intentionally amplitude )URP7DEOH 3 DSSUR[LPDWHO\LQ)URP7DEOH 3 21.2.1(b) ID RI[EHF VHWKH 8VHDUHGXFWLROIDFWRURIFEHFDXVHWKH OR PHPEHU2WKHUZ HINLS" VKHDULQWHUIDFHLVQRWDPHPEHU2WKHUZLVHINLS (0.75)(1.0)Avf(60 ksi) n = (0.75) The wall is Ewall = 30 ft long. 42 in.2 or o Avf/Ewall = 11.42 in.2/(30 ft) = 0.38 in.2/ft Avf = 11.42 Try No. 6 aat 12 in. on center. As, prov. = 0.44 in.2/ft OK The required 0.08fcg Ac (c) 1600Ac (0.2)(4000 psi) = 800 psi OK (480 + 0.08(4000 psi)) = 800 psi OK Therefore, Vn must not exceed: (800 psi)(10 in.)(30 ft)(12 in./ft) = 2880 kip > Vn 1000 lb/kip American Concrete Institute - Copyrighted © Material - www.concrete.org 337 Diaphragms CHAPTER 8—DIAPHRAGMS Collector reinforcement at Shear Walls 2 and 3 Enlarged collector reinforcement at Wall 2 Fig. E3.9—Collector reinforcement for lateral force in the N-S direction. American Concrete Institute – Copyrighted © Material – www.concrete.org 338 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Diaphragm lateral force distribution E-W The wall forces and the assumed direction of torque due to accidental eccentricity are shown in Fig. E3.10. The distribution of the diaphragm force is calculated by using qL and qR as the left and right diaphragm reactions per unit length (Fig. E3.10). Case I: Eccentricity at ey = 4 ft ation of Refer to Step 5 of this example for calc calculation eccentricity. Force equilibrium F E3 Fig. E3.10—Shear ear wall forces due to a seismic force he E ction at ey = 8.5 ft. in the E-W direction  $L \left( L \right) \left( 2 \right) \left( 3 \right) = (F4 + F5)(80 ft) - F2(200 ft) \left( 90 ft \right) \left( 1 \right) \left( 2 \right) \left( 2 \right) \left( 1 \right) \left( 2
\right) \left( 2 \right) \left($ ft) (II) (90 ft)2 2(90 ft)2 + gR 6 6 = (208 kip + 131 kip)(80 ft) - (10 kip)(200 ft) gL F2. F4, and F5 are per Table E.5. Solve equations (I) and (II) for gL and gR; gR = 2.5 kip/ft American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8-DIAPHRAGMS 339 Draw the shear and moment diagrams for the diaphragm assuming simply supported beam behavior (Fig. E3.11). The maximum moment is located at 40.5 ft from the south end of the diaphragm. Diaphragms Vmax = 6542 ft-kip Note: Moehle et al. also state in NIST report number GCR 10-917-4 that, "For a rectangular diaphragm of uniform mass, a trapezoidal distributed force having the same total force and centroid is then applied to the diaphragm. The resulting shears and moments are acceptable for diaphragm inertia lateral force) unresolved; sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." Fig. E3.11—Shear 3.11—Shear 3.11—Shear and nd moment diagrams for Case I. Case II. Eccentricity at  $e_y = 13$  ft us Step 5 for calculation of Refer to previous eccentricity. m Force equilibrium (L) (L) qL | + qR | = Fpx, des (EW) (2/(90 ft) (90 ft) (90 ft) qL | + qR | = 726 kip (2/(2 ft) (1) (1) (2 ft) (2 f Institute - Copyrighted © Material - www.concrete.org 340 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Draw the shear and moment diagrams for the diaphragm assuming simply supported beam behavior (Fig. E3.12). The maximum moment diagrams for the diaphragm assuming simply supported beam behavior (Fig. E3.12). ft-kip, say, 6500 ft-kip Fig. E3.12—Shear E3.12—and moment diagrams for Case II Case I Case II Case I Case I Case II Case ignoring oring the accidental torsion, the correNote: Taking the approach of equivalent distributed sponding shear and moment forcess are: ned. Shear: 317.6 kip at walls W4 and W5 combined. Maximum moment: 6252 ft-kip &RPSDULQIWKHPRPHQWVRIWKHWZRDSSURDFKHVZH¿QGWKDWWKHGLIIHUHQFHLVOHVVWKDQSHUFHQWQHJOLJLEOH Experienced engineers will usually continue the design using the uniformly distributed inertia force. This example, howHYHUZLOOXVHWKHGHWDLOHGDSSURDFKDSSO\LQJWKH¿YHSHUFHQWDFFLGHQWDOWRUVLRQ Force summary in the E-W direction Case I Case II Maximum moment, ft-kip 6540 6500 I W1 shear force, kip 387 363 I W4 shear force, kip 131 140 II W5 shear force, kip 208 223 II American Concrete Institute - Copyrighted © Material www.concrete.org Controlling case CHAPTER 8—DIAPHRAGMS 341 Step 12: Chord reinforcement E-W R12.1.1 Assume the slab behaves like a beam with compression and tension forces at the near and far edges, respectively: Cchord = Tchord = M/d Chord reinforcement resisting tension must be located within h/4 of the tension edge of diaphragm. h/4 = 200 ft/4 = 50 ft d = 175 ft Diaphragms Assume that tension reinforcement will be placed within wall thickness. Therefore, moment arm is approximately 200 ft - 1/2 (10 in./12) = 199.58 \text{ ft} at both east and west sides of the slab edges: d = 199.58 ft Chord force The maximum chord tension force is at midspan: Tu = 18.12.7.5 Mu d T = 6540 ft-kip = 32.8 kip 199.58 ft Calculate the required chord width for the calculated concrete compressive strength limit of 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 f c't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in.
(0.2)(400 si) Note: Chord 0.2 fc 't wchord > 22.8 kip - 4.1 in. (0.2)(400 si) Note: Chord 0.2 fc 't wchor loChord reinforcement must aphragm. cated within h/4 of the tension edge of diaphragm. Check if required calculated width is less than wall thickness: wchordd = 4.2 in. < tw = 10 in. Tension due to moment is resisted by deformed bars conforming to Section 20.2.1 of ACI 318-14. 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFL¿HG\LHOG strength and 60,000 psi. fy = 60,000 psi Required reinforcement []Tn = []fyAs • Tu As , req ' d = OK 32.8 kip = 0.61 in.2 (0.9)60 ksi Note: The chord reinforcement placed in the east and west ends is compared to the partial collector reinforcement at east and west ends is compared to the partial collector reinforcement placed in the east and west ends is compared to the partial collector reinforcement placed in the east and west ends is compared to the partial collector reinforcement placed in the east and west ends is compared to the partial collector reinforcement placed in the east and west ends is compared to the partial collector reinforcement placed in the east and west ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the partial collector reinforcement placed in the east ends is compared to the placed in the east ends is compared to the placed in the east ends in the east ends is compared to the placed in the east ends in the east ends is compared to the east ends in the east ends in the east ends is compared to the east ends in the eas placed within the wall thickness that exceed the required chord reinforcement. Therefore, OK. Step 13: Diaphragm shear strength. American Concrete Institute - Copyrighted © Material - www.concrete.org 342 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 14: Collector design E-W Wall 1 12.5.4.1 Collectors transfer shear forces from the diaphragm and transfer them axially to wall W1 (Fig. E3.13). In this example, assume collectors extend over the full length of the diaphragm. Unit shear force: vu @ F = 12.5.4.1 From Step 6 (Table E.5): Fu = 387 kip = 1.94 kip/ft 200 ft In Wall W1: vu @ F = 387 kip = 4.3 kip/ft 90 ft Walls 4 and 5 Collectors transfer shear force: vu @ F = Fu @ F B From Step 6 (Table E.6) E.6): b: Fu = 140 40 kip + 2223 kip = 363 kip F = 363 kipp = 1.82

kip/ft k 200 ft Walll 4: Fu = 140 W 40 kip In slab: vu @ F = 40 kip 140 = 5 kip/ft 28 ft Wall 5: Fu = 223 kip = 5.58 kip/ft 40 ft American Concrete.org CHAPTER 8—DIAPHRAGMS 343 Step 15: Collectors design N-S Force at diaphragm to wall connection The proportional diaphragm force that the collector transfers to walls connection is (Fig. E3.13): Diaphragm end: -106.6 kip/ft)(40 ft) = -72.8 kip/ft)(40 ft) = -72.8 kip/ft)(40 ft) = -72.8 kip/ft)(28 ft) = 16.2 kip/ft)(28 ft) = 16.2 kip/ft)(28 ft) = 106.6 kip/ft)(28 ft) = 162.6 kip/ft)(28 ft) = 162.6 kip/ft)(28 ft) = 106.6 kip/ft)(28 ft) = 162.6 kip/ft)(28 ft) = 106.6 kip/ft)(28 ft) Wall 5 west end: NLS[]NLSIW IW ±NLS Wall 5 east end: /ft(40 ft) = 71.8 kip - (1.82 Fig. E3.13—Collector forces in the E-W direction. 18.12.2.1 The collector force that is transferred to the walls is shown in Fig. E3.13—Collector force is then multiplied by the system RYHUVWUHQJWKIDFWRUoo = 2.5 for building systems with special structural walls in SDC D (ASCE/SEI 7, Table 12.2-1). Wall 1 Wall 5 2.5Tu 268 -268 -182 -41 -197 -180 American Concrete Institute - Copyrighted © Material - www.concrete.org 344 12.5.4.2 18.12.7.5 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Collectors are designed as tension and compression members. 7KHUHDUHQREHDPVDORQJ&/VRSRUWLRQRIWKH slab is used as a collector. The collector width is determined by engineerLQJMXGJHPHQWDQGFKRVHQVXFKWKDWKHOLPLWLQJ stresses are not exceeded. When the tension and compression collector forces are increased by the overstrength factor, then the limiting concrete compressive stress is 0.5fcg&DOFXODWHWKHFRPSUHVVLYH collector width: wchord = 2.5Ccoll/0.2fcgt Wall 1 Wall 5 wcoll, in. 17 17 9 2 9.9 9 is wcoll > tw? Y Y N N N N For Wall 1, the required collector width (17 in.) is wider than the wall thickness (8 in.). Part of the seismic force is resisted by reinforcement placed in-line with the shear wall and direct bearing of slab against wall in compression. Therefore, two No. 6 in-line with the shear wall and direct bearing of slab against wall in compression. sides of the wall and apacity at the wall-to-slab interface to transfer seismic forces to the walls. It is widths within the walls widths walls widths walls will 1 and 99.9 in., respectively) are narrower hs (10 in.), therefore, place reinforcement ent w within the walls ls widths. than the walls widths walls Wall 1 Required area off ccollector reinforcement: or reinforcement in ment along th the length of the diaphragm phra may ay be var varied based on required strength ement is eextended over the full length of the and terminated when not required. In this example, the reinforcement diaphragm. The collector width, as suggested by ACI 318 commentary, is an arbitrary width equal to half the wall width. beff = 80 ft/2 + 8 in./(12 in./ft) = 45.67 ft 1. Place bars within the arbitrary width of 45.67 ft. Spreading bars over more than half the diaphragm width is not practical. 2. Use the calculated chord reinforcement in the E-W direction over h/4. This option is acceptable as the required chord and collector calculated reinforcement is approximately equal, 5 in.2. The collector is wider than the wall, therefore, longitudinal and transverse reinforcement must be provided to transfer forces from the collector and chord reinforcement are placed in a beam beam. In this example, this option is used. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS 345 The required collector reinforcement to resist the gravity dead and live load is calculated above. Therefore, a total of 8.8 in. 2 must be placed within the beam to resist gravity loads combined with either the calculated chord force or collector force due to inertial forces in the N-S and E-W directions, respectively (Fig. E3.14A). Note: Typically, gravity loading is (1.2 + 0.2SDS)D + 0.5L, which is usually less than the previous case. Therefore, it may be possible to count on a portion of provided gravity reinforcement for seismic collectors. The beam to the wall (8 in.). Mu = (268 kip)(2 in.) = 536 in.-kip = 0.5 kip (89 ft)(12 ft) (0.5 kip)(1000 compression)lb/kip) = 0.01 in.2 (0.9)(60,000 psi) Note: The force is very small and the corresponding reinforcement is negligible. Therefore, it is assumed that the result of the eccentricity, large force, or shorter wal wall length), reinforcement is required erly developed at both ends of the wall and extending into nto the diaphragm a minimum and placed and properly length equal to thee dev development length of the bars. Wall 4: cto width th is less than th Required collector the wall thick  $\Omega T$  182 kip = 3.37 in.2 m may ay be placed wi in th ness. Reinforcement within the wall As = o coll = f 0.9 ksi) 0.9(60 ks y width: Try four No. o. 9 bars: in 2 > As, req'd = 2.94 in 2.29.4.4 Shear friction reinforcement is not required as the collector force is already developed into the wall. The value of Vn across the assumed shear plane must not exceed the lesser of the following limits: (a)  $0.2fc_{0}Ac$  (b)  $(480 + 0.08fc_{0}Ac$  (c) 1600Ac(0.2)(4000 psi) = 800 psi OK (480 + 0.08(4000 psi)) = 800 psi 1600 psi OK Therefore, Vn must not exceed: (800 psi)(10 in.)(28 ft)(12 in./ft) = 2688 kip > Vn 1000 lb/kip Wall 5: Required collector width is less than the wall thickness. Reinforcement may be placed within the wall width: As =  $\Omega \text{ oTcoll } 197 \text{ kip} = 3.65 \text{ in.} 2\ 0.9 \text{ fy} (0.9(60 \text{ ksi}) \text{ Try four No.} 9)$ bars: As, prov. = 4.0 in.2 > As, req'd = 3.65 in.2 Shear friction reinforcement is not required as the collector force is already developed into the wall. American Concrete Institute - Copyrighted © Material - www.concrete.org Diaphragms Gravity load calculation is not provided in this example. 346 THE REINFORCED CONCRETE DESIGN HANDBOOK -SP-17(14) Step 16: Shrinkage and temperature reinforcement 12.6.1 Shrinkage and temperature 24.4.3.2 reinforcement is the lesser of 5h and 18 in. (a) 5h = 5(15 in.) = 75 in. (b) 18 in. Controls Note: Shrinkage and temperature reinforcement may be part of the main reinforcing bars resisting diaphragm in-plane forces and gravity loads. If provided reinforcement is not continuous (placing bars to resist negative moments at columns), the engineer must ensure continuity between top and bottom reinforcing bars by providing adequate splice lengths between them. Step 17: Reinforcement detailing Development 25.4.2.2 Chord and collector reinforcement are extended 25.4.10.2 over full length and width of the edges of the 12.7.3.2 diaphragm. Therefore, development length will be ngths. Calculated only to determine splice lengths. Development length of shear transfer reinforcement Ad = 25.5.2.1 18.12.7.6 12.7.2.1 25.2.1 18.12.7.6 12.7.2.2 f y  $\psi$ t  $\psi$ e Ed Ed, in. Use Ed, in. 25 $\lambda$  f c' N 5 No. 23.4 24 N 6 No. 28.5 30 Splices ild engths are longe Because the building lengths longer than a ng length th of the No. 8 lo ngitud standard shipping longitudinal plic will be needed. U reinforcement, splices Usee Class B splice: 1.3(Ed) The center-to-center spacing of the longitudinal bars for collector and chords at splices and anchorage zones is but not less than 2 in. (1.3) A d = (1 60,000 0,000 psi 0) 4000 psi 25(1.0) (1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. Say, 56 in. (4 ft 8 in.) 3db = 3(1.0 in.) = 3.0 in. minimum spacing (2.5)(1.0 in.) = 55.6 in. (5 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in.
(5 in.) 56 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in.) 56 in. (5 in.) 56 in. (5 in.) 56 in.) 52.5 in. cover Therefore, transverse reinforcement is not required. Reinforcement spacing Chord and collector reinforcement minimum and maximum spacing of (a) 1 in. (b) 4/3dagg. (c) db No. 9 Minimum spacing 1.128 in., say, 1.25 in. Collector reinforcement spacing at splice must be at least the larger of: 3(1.128 in.) = 3.384 in. say 3.5 in. a. At least three longitudinal db b. 1.5 in. c. cc PD[>db, 2 in.] Maximum spacing is the smaller of 5h or 18 in. 18 in. Controls American Concrete Institute – Copyrighted © Material – www.concrete.org Controls CHAPTER 8—DIAPHRAGMS 347 Diaphragms Step 18: Detailing Fig E3.14—Diaphragm reinforcement ment de detailing. Section A—Edge beam gravity and collector or chord reinforcement. American Concrete.org 348 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Section B—Chord/collector reinforcement at south end of diaphragm. Section C— Collector reinforcement along Walls 4 and 5. Section D—Collector reinforcement along Walls 2 and 3. Note: Wall reinforcement not shown for clarity for Walls W4 and W5 the detail is similar, however, alternate dowels to either side of the wall. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 8—DIAPHRAGMS 349 Step 19: Discussion There is no consensus among engineers on how to distribute the diaphragm inertia force. Based on discussions with several respected engineers, the main approaches are as follows: This example follows the ASCE 7-10 recommendation. The 5 percent eccentricity is excluded in the analysis. The diaphragm inertia force is uniformly distributed to the diaphragm. The shear forces in the lateral-force-resisting system due to the equivalent lateral force. American Concrete Institute – Copyrighted © Material – www.concrete.org Diaphragms ASCE 7-10 (third printing) Section 12.10.3.4, recommends shifting the center of mass by a minimum of 5 percent of the building dimension in either direction and perpendicular to the seismic loading, referred to as accidental eccentricity. This ¿YHSHUFHQWWRUVLRQDOHFFHQWULFLW\LVDSSOLHGLQDGGLWLRQWRWKHFDOFXODWHGJHRPHWULFHFFHQWULFLW\+RZHYHUWKH\$6&( recommendation is located in the commentary, therefore, it is not mandatory. 350 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) When center of rigidity do not coincide, the lateral-force-is example: (W1 (27.3 kip), W4 (10.5 kip), and W5 (16.7 kip)) for a seismic force acting in the N-S direction. Drawing the moment diagram shows a discontinuity at the center of rigidity. Moehle et al. also state in NIST Report No. GCR 10-917-4 that: "For a rectangular diaphragm of uniform mass, a trapezoidal distributed force having the same total force and centroid is then applied to the diaphragm design. Note that this approach leaves any moment due to (shear walls perpendicular to the diaphragm inertia lateral force) unresolved; sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." 2WKHUHQJLQHHUVLQFRUSRUDWHWKHPRPHQWGXHWRVKHDUZDOOVSHUSHQGLFXODUWRWKHGLDSKUDJPLQHUWLDODWHUDOIRUFH7KLVUHVXOWV LQGLVFRQWLQXLW\MXPS LQWKHPRPHQWGLDJUDPDVVKRZQLQ)LJ((TXLOLEULXPLQWKHV\VWHPLVREWDLQHGE\GUDZLQJWKH moment diagram due to the shear force (Fig. E3.15): Fig. E3.15). Fig. E3.15): Fig. E3.15: Fig. E3.15 www.concrete.org CHAPTER 8—DIAPHRAGMS 351 Diaphragms In this example, the moment diagram. Since Case I resulted in a slightly higher moment, the calculations for that case follow: Fig. E3.16—Free body diagram. Mx = (13) kip + 208 kip)(x IW NLSIW x2 > NLSIW x2 > NLSIWNLSIW x2 > NLSIW x2 Chapter 10 in ACI 318-14 apply for the design of nonprestressed, prestressed, and composite columns. Heading limits the following requirements to a particular type of column. The word "nonprestressed" is typically in reference to castin-place columns. and "prestressed" with precast columns. While ACI 318-14 covers post-tensioned columns, they are not commonly used. Chapter 14 in ACI 318-14 covers the design of plain concrete pedestals. 9.3—Design limits 9.3.1 General—Concrete columns offer architects an opportunity to create various cross-sectional shapes for the aesthetic purposes. Unusual cross-sections, however, DUH PRUH GLI¿FXOW WR DQDO\]H 6HFWLRQ LQ\$&, permits the designer to use an effective cross-section for analysis. For Fig. 9.3.1—Permitted cross-section for analysis. For Fig. 9.3.1—Permitted cross-section for analysis. section (Fig. 9.3.1). The key point is that a using a portion of an oversized column or wall that is easier to analyze is permitted. Section 10.3.1.2 in ACI 318-14 allows an oversized column is that a using a portion of an oversized column is that a using a portion of an oversized column or wall that is easier to analyze is permitted. detailed considering the actual cross section. Note that the minimum area of steel (refer to 9.6 of this Handbook) is 0.01Ag based on the smaller effective area, but Ag cannot be less than half ar the actual area. Note that the shape of the column can be determined by the building architecture but it must always equirements of of ACI A AC 318-14. meet the requirements zingg It is i not economical to have a unique 9.3.2 Initi Initial sizing—It de for eeach column olumn in th design the building. The following guidelp in economical mical colu lines help columns sizes for the entire use only three bu building. t same me streng 2. Use the strength of concrete for all columns at a l. Structures Structur under six stories or less commonly level. co use one concrete strength for the full height of the bui building. Note section 15.3 in ACI 318-14 has addiWLRQDOGHVLJQUHTXLUHPHQWVDWWKHARRUMRLQWVLIWKH concrete strength in the columns exceeds 1.4 times WKHFRQFUHWHVWUHQJWKRIFRQFUHWHVVWHP 3. Proportion the column cross sections and concrete strength is needed, increasing the VWUHQJWKRIFRQFUHWHLVXVXDOO\PRUHHICFLHQWWKDQ increasing reinforcement area. The analysis and design of columns is an iterative process. To begin, the designer assumes sectional properties in order to run an analysis. An initial column area, Ag, can be estimated by dividing the maximum factored axial load by 0.4fcg for ordinary columns or 0.3fcgIRUFROXPQVLQKLJKVHLVPLF areas. Columns are usually rectangular, )RU WKH ¿UVW LWHUDWLRQ D SHUFHQW UHLQIRUFHPHQW UDWLR LV evenly distributed around the column perimeter. An effective moment of inertia, leff, of concrete members is used in the analysis to account for cracking at the nominal condition. The simple leff values in Table 6.6.3.1.1(a) in ACI 318-14 are generally used and the cross-section properties are assumed to be uniform for the length of the member. With these American Concrete Institute – Copyrighted ©
Material – www.concrete.org Columns 9.1—Introduction The column chapter; analysis to determine the required strength; design area of reinforcement needed to exceed the required; and detailing. The analysis must be consistent with ACI 318-14, will remind the designer of limits and rules for columns that should be accounted for in their analysis model. The requirements for column design start in Section 10.5 in ACI 318-14. A column is always part of the ODWHUDO IRUFHUHVLVWLQJ V\VWHPV /)56 7KH PRVW FRPPRQ lateral design forces are seismi and wind. For buildings designed to resist seismic forces, the requirements of Chapter 10 in ACI 318-14 for columns that are part of an ordinary, intermediate, termediate or special moment frame system. There are also seismic requirements lso sei IRUFROXPQVWKDWDUHQRWSDUWRID/)567KHVHLVPLFUHTXLUH/)5 ments are intended to increase col column ductility to ac accommodate the large displacementss tthat are during re expected dur ga maximum design earthquake. Fo For buildings dings that resist only nly wind forces, columns are designed gn byy Chapter 10 in ACI ZKHWKHUWKH\DUHGHVLJQDWHGDVSDUWRID/)56R RW There are no additional requirements and how they impact a column's design and detailing. A review of basic engineering principles is provided so that a designer car effectively design columns by hand or computer using only this Handbook. 354 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) )LJD<sup>2</sup>3"HIIHFWV assumptions, an initial analysis is run and, subsequently, VHFWLRQ SURSHUWLHV RU UHLQIRUFHPHQW DUHD DUH DGMXVWHG DV necessary. Fig. 9.4.1b—Effects of slenderness on a column. 9.4—Required strength 9.4.1 General—The required strength is calculated using nd analysis the factored load combinations in Chapter 5 and procedures in Chapter 5 and procedures in Chapter 5 and procedures in Chapter 5 and procedures in Chapter 6 (ACI 318-14). Three methods of VLV HODVWLF VHFRQG DQDO/VLV HODVWLF pter 3 of this Handbook. permitted as discussed in Chapter ose all columns mu Regardless of the method chosen, must be checked for slenderness effects are associated slenderness. oci with higher slenderness effects are associated slenderness. dimension, or columns with limited end restraint. As a column becomes more slender, larger lateral deformations and deformation along the length occur due to the applied load. The column must support additional moment created by the column axial load acting on the deformed column, also known as second-order moments. There are two types of second-order moments related to the P-Delta effects shown in Fig 9.4.1a: 1) second-order moments due to translation of the column stabilizes. If the slendernessed ue to this geometric nonlinearity until the column stabilizes. If the slendernessed ue to translation of the column stabilizes and the slendernessed ue to translation of the column stabilizes. ratio is very high, the column becomes unstable and cannot resist the axial load, refer to Fig. 9.4.1b). 9.4.2 Slenderness ratio, N&u/r, where & is unsupported column length; k is effective length IDFWRUUHAHFWLQJHQGUHVWUDLQWDQGODWHUDOEUDFLQJFRQGLWLRQV and rLVWKHUDGLXVRIJ\UDWLRQUHAHFWLQJWKHVL]HDQGVKDSH of a column cross section. The column's unsupported length & u is the clear distance between the underside of the beam, slab, or column capital DERYHDQGWKHWRSRIWKHÅRRUEHORZ7KHXQVXSSRUWHGOHQJWK may be different framing conditions and corresponding unsupported lengths )LJD<sup>2</sup>8QVXSSRUWHGFROXPQOHQJWK Eu. (Eu). Each coordinate and x and yVXEVFULSWLQWKH¿JXUHLQGLcates the plane of the frame in which the stability of column is investigated. The effective length factor k UHAHFWV WKH FROXPQ DQG EHWZHHQ DQG 'IRU XQEUDFHG columns. Most columns have end restraints that are neither SHUIHFWO\KLQIHGQRUIXOO\¿[HG7KHGH]UHHRIHQGUHVWUDLQW GHSHQGV RQ WKH ARRU VWLIIQHVV UHODWLYH WR WKH FROXPQ VWLIIness. Jackson and Moreland alignment charts, given in Fig. R6.2.5 in ACI 318-14, can be used to determine the factor k for different values of relative stiffness at column ends. The VWLIIQHVV UDWLRV zADQG zB XVHG LQ WKH FKDUWV VKRXOG UHAHFW concrete cracking, and the effects of sustained loading. %HDPV DQG VODEV DUH AH[XUH GRPLQDQW PHPEHUV DQG PD\ American Concrete Institute – Copyrighted © Material – www.concrete.org 355 Columns CHAPTER 9—COLUMNS IIH OHQJWKIDFWRUN UFROXPQVD E D DUHIRUQR )LJE<sup>2</sup>(IIHFWLYHOHQJWKIDFWRUNIRUFROXPQVD E DQGF DUHIRUQRQVZD\IUDPHV and (d), (e), and (f) are for sway fram frames. Fig. 9.4.2c—Radius of gyration for circular, square, and rectangular sections. FUDFNVLJQL¿FDQWO\PRUHWKDQFROXPQVZKLFKDUHFRPSUHVsion dominant members. The reduced moment of inertia values given in Section 6.6.3.1.1 of ACI 318-14 should be used to determine k. Tables D1.1 through D1.5 in the supplement to this Handbook, ACI Reinforced Concrete Design Handbook Design Aid – Analysis Tables (ACI SP 17DA), provide design aids for the calculation of the effective length factor, stiffness, and moment of inertia. The radius of gyration introduces the effects of crosssectional size and shape to slenderness. A section with a higher moment of inertia. rLVGHiQHGLQ6HFWLRQRI&, DQG shown in the following Eq. (9.4.2) r = I A (9.4.2) It is permissible to use r = 0.3h for square and rectangular sections, where "h" is the dimension in the direction stability is being considered. This is shown in Fig. 9.4.2c. 9.4.3 First-order analysis<sup>2</sup>)RU D iUVWRUGHU HODVWLF analysis, Section 6.6.4 in ACI 318-14 provides a moment PDJQL¿FDWLRQ PHWKRG ZKLFK FRQVHUYDWLYHO\ DFFRXQWV IRU slenderness. This method is shown in Example 9.1 of this +DQGERRN, Q D ¿UVWRUGHU HODVWLF DQDO\VLV WKH EXLOGLQJ frame is analysed once and results are used for input to the PRPHQWPDJQL¿FDWLRQPHWKRG American Concrete Institute - Copyrighted © Material - www.concrete.org 356 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) 9.4.3.1 Sway or nonsway frames-The designer needs to determine if the column is in a sway or nonsway frame. \$IUDPHLVQRQVZD\LILWLVVXI¿FLHQWO\VXSSRUWHGE\ODWHUDO bracing, such as structural walls. Structural walls used for elevator shafts, stairwells, partial building enclosures, or interior stiffening elements provide substantial drift control and lateral bracing. In many cases, even a few structural walls can brace a multi-story, multi-bay building. Sway PXVW EH FKHFNHG IRU HDFK GLUHFWLRQ DQG ARRU\$&, provides three methods to determine if the lateral stiffness is VXI¿FLHQWWRGHVL]QDWHWKHIUDPHDVQRQVZD\ 1. Section 6.2.6, ACI 318-14: Columns are nonsway if the gross lateral stiffness of the walls (bracing elements) in a story is at least 12 times the gross lateral stiffness of the columns in that story. This is a simple, conservative hand calculation. 2. Section 6.6.4.3(a), ACI 318-14: Columns are nonsway if the increase in column end moments. 3. Section 6.6.4.3(b), ACI 318-14: Columns are nonsway if the increase in column end moments. the stability index "Q" does not exceed 0.05 as shown in Eq. (9.4.3.1): Q = ∑ Pu ∆ o ≤ 0.0 0.05 Vus A c (9.4.3.1) axial load ad acting on al all the ZKHUH <sup>™</sup>Pu is total factored ax O IIDFWRUHG HG VWRU\ VKHDU "o is columns in a story, Vus LV WRWDO KH RIWKHVWRU\UHO YH ODWHUDOVWRU\GULIWGHAHFWLRQRIWKHWRSRIWKHVWRU\UHODWLYH G LIW "o should h to the bottom of that story) due to Vus6WRU\GULIW" be computed using section properties taking into account the presence of cracked regions along the member, refer to Section 6.6.3.1 in ACI 318-14. 9.4.3.2 Column slenderness<sup>27</sup>KHPRPHQWPDJQL¿FDWLRQ method states that for columns in sway or nonsway frames, secondary effects may be neglected if the slenderness ratios are below the limits given in Section 6.2.5 in ACI 318-14. If these limits are exceeded in a nonsway frame, then secondorder effects due to translation (P- $\Delta$ ) may be ignored, but the second order effects along the member (P-δ) given in 6.6.4.5 in ACI 318-14 need be considered. If the slenderness ratio limits are exceeded in a sway frame, column (P-Δ) and (P-Δ) effects are to be calculated. Section 6.6.4.6 in \$&,LVFRPSOHWHG¿UVWWRFDOFXODWHWKHDPSOL¿HGPAPHOWV are then used in Section 6.6.4.5 in ACI 318-14 to calculate the second-order effects. Such a program must be capable of the iterative calculations to determine: (a) Second-order moments (P-Δ) due to the laterally GHÀHFWHGVWUXFWXUH (b) Second-order moments (P-& GXHWRGHÀHFWLRQDORQJ the length of the column needs to be divided into at least two segments WR FDOFXODWH D
GHÀHFWLRQ DORQJ LWV OHQJWK\$ ZD\ WR FKHFN a computer program for this effect is given in the ACI Q&A, Using an Elastic Frame Model for lations (Frosch 2011). Slenderness effects are nonlinear, so the principle of superposition does not apply for calculating the analysis. Section 6.7.1.1 in ACI 318-14 states, "An elastic secondRUGHU DQDO\VLV VKDOO FROVLGHU the second-order moments. The forces for a load combination must be combined before running WKH LQAXHQFH RI D[LDO ORDGV presence of cracked regions along the length of the member, DQG HIIHFWV RI ORDG GXUDWLRQ '7KH PRPHQW PDJQL¿FDWLRQ method in Section 6.6.4 in ACI 318-14 accounts for these properties by using stiffness reduction factors, fk. The method uses two different factors for the two different types of slenderness effects, P- $\Delta$  and P- $\delta$ , refer to R6.6.3.1.1, R6.6.4.5.2, and R6.6.4.6.2 in ACI 318-14. A lower stiffness reduction in its second-order analysis. These programs should be reviewed for how they account for stiffness reduction. —Design strength stren 9.5—Design PDMRULW\RIUHLQIRUFHG FH 7KHPDMRULW\RIUHLQIRUFHGFRQFUHWHFROXPQVDUHGHVLJQHG WAH[ D[LDOIRUFH WRUHVLVWAH[XUHD[LDOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[LDOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[LDOIRUFH WRUHVLVWAH[XUHD[LDOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[LDOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[DOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[DOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[DOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[DOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAH[ D[DOIRUFHGFRQFUHWHFROXPQVDUHGHVL]QHG WAHI]QHG WAH[ D[DOIRUFHGFRQFUHW ession. As can be to combined ffere combinations binations of moment and accompanying seen, different xial force orce result in different column nominal strengths F UHVS J VWUDLQ SSUR¿OHV ZKLOH DOVR DIIHFWLQJ WKH DQG FRUUHVSRQGLQJ t pression i controlled behavior. The combinatension or compression men a column's tion of moment nal strength is traditionally presented by "column internominal action dia diagrams." Interaction diagrams are constructed by computing moment and axial force nominal strengths, as VKRZQEHORZIRUGLIIHUHQWVWUDLQSUR¿OHV Pn = Cc + Cs1 + Cs2 - Ts (9.5a) Mn = Ccx2 + Cs1x1 + Cs2 + Tsx3 (9.5b) As the strains vary using Eqs. (9.5a) and (9.5b) from pure compression to pure bending, a nominal strength curve can be created as shown by the outer curve in Fig. R10.4.2.1 in \$&,7KHQRPLQDOVWUHQJWKLVDGMXVWHGWRWKHGHVLJQ VWUHQJWKE\PXOWLSO\LQJE\WKHDSSURSULDWH[] FWRUV7KH[] factor for comp controlled sections is 0.75 for spirals DQG IRU RWKHU WLH FRQ¿JXUDWLRQV 7KH []DFWRU IRU DOO tension-controlled sections is 0.9. This factor varies linearly from 0.65 or 0.75 to 0.9 through the transition zone shown in Fig. 9.5. The design strength curve is shown by the inner curve in Fig. R10.4.2.1 in ACI 318-14. The interaction diagram is the plot of both the nominal and design strength curves shown on a graph. An electronic spreadsheet is provided as a supplement to this Handbook to demonstrate how to make an interaction diagram (ACI SP17DAE). The key points of the diagram American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS 357 are: pure compression (zero moment), pure tension (zero moment), pure tension (zero moment), pure bending (zero axial force), reinforcement stress at 0.0fy, reinforcement stress at 0 contained interaction diagrams. These diagrams have been retained in a supplement to this Handbook titled, Reinforced Concrete Design Handbook titled, Rei Because shear design in columns Hand review Beam Chapter 8 of this Handbook for this informaFFX FROXPQVDQGW HWLRQ6LJQL¿FDQWWRUVLRQUDUHO\RFFXUVLQFROXPQVDQGW HWLRQ6LJQL A content of this informaFFX FROXPQVDQGW HWLRQ6LJQL A content of the set of the set of the set of the set of this informaFFX FROXPQVDQGW HWLRQ6LJQL A content of the set of th should reviewed. 9.6—Reinforcement limits The minimum column vertical reinforcement ratio is 0.01Ag. This amount is enough to keep the reinforcement ratio is 0.01Ag. This amount is about the maximum that can be realistically provided at the perimeter of a concrete section that would also meet the minimum cover and spacing requirements. This percentage was set in ACI 318-63 when butt splices were common. For present day construction, lap splices are more common and for many SURMHFWVEDUVDUHVSOLFHGDWWKHERWWRPRIWKHFROXPQVWDUWLQJ DWWKHARRU,QWKLVFDVHWKHPD[LPXPSHUFHQWRIUHLQIRUFHment is 4 percent, since the maximum percent of reinforcement at a section is 8 percent. If the lap splices
are staggered, the maximum percent of reinforcement can increase up to 6 percent. As stated earlier, usual reinforcing ratios are in the range of 1 to 2 percent. There are cases where more reinforcement is necessary, such as to meet a required strength. If the column design routinely requires reinforcement is necessary, such as to meet a required strength. detailing 9.7.1 General—Section 10.5 in ACI 318-14 provides minimum column reinforcement area. Section 10.7 in ACI 318-14 provides limitations on the location, spacing, and splicing of longitudinal reinforcement and the location, spacing, geometry, and type of transverse reinforcement. General requirements such as concrete cover, development length, and splice lengths, are covered in Chapters 20 and 25 in ACI 318-14 for all members. Note that many detailing provisions of this chapter are related to how columns are constructed. 9.7.2 Longitudinal bars 9.7.2.1 Spacin Spacing—The minimum number of bars in a on 10.7.3.1 in ACI 318-14. Square columns must have a minimum of four bars. Circular columnss must have three or six depending if the tie is triangular tri gular or circular. cular. Eigh Eight bars are suggested, however, to ensure column is given in Se Section tangular must have a minimum of four bars. ens re that th the design esign mo moment is achieved regardless of LWLRQ RI UHLQIRUFHPHQW QIRUFHPH LQ WKH & COUNTRY Content in Se spacing Section 25.2 in ACI 318-14. A max um sspacing g requirem maximum requirement in Chapter 10 in ACI 318-14 ly given. Section 18.7.5.2(e) in ACI 318-14. 14, is not explicitly however, states that ffor columns in a special moment frame, thee spacing of longitudinal bars laterally supported by the corner of a crosstie or hoop leg, hx, shall not exceed 14 in. around the perimeter of the column. Note that this spacing is further reduced to 8 in. for conditions given in Section 18.7.5.2(f) in ACI 318-14. Typically bars are evenly spaced around the perimeter, as this helps to create a more stable column cage during construction. Designers will often place bars only at the corners of square or rectangular columns to reduce the ¿HOG ZRUN QHFHVVDU\ WR SODFH WLHV %XQGOHG EDUV DUH RIWHQ needed to meet the required area of steel for this arrangement. Note that Section 18.7.5.2 in ACI 318-14 makes this practice impractical for columns in special moment frames GXHWRWKHLQOLPLWDWLRQ, IFRQ¿QHPHQWRIWKHFRQFUHWH core is necessary or desired, evenly spaced longitudinal bars DUH KHOSIXO , I PRUH EDUV DUH UHTXLUHG WR UHVLVW AH[XUH DW D particular location, some bars can be added to the evenly spaced column bars. 9.7.2.2 Splicing—Splice locations, lengths, and types VKRXOGEHLQFOXGHGRQWKHVWUXFWXUDOGUDZLQJV/DSVSOLFHV are the most common splice type due to their ease of fabrication and construction. Mechanical connectors and buttwelded splices are helpful where bar arrangement becomes congested but they can require additional erection time. End-bearing splices are not common today but are some- American Concrete Institute – Copyrighted © Material – www.concrete.org Columns Fig. 9.5—Column section analysis. 358 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 9.7.2.2a—Column splice locations ons (Fanella 2007). 2007) times used in bundled bar arrangements ents that are only in binatio compression under all load combinations. IFR VDWHDFKARRU D XVXDOO\ORFDWHGDWWKHERWWRPRIFROXPQVDWHDFKARRULID HG /DS VSOLFHV IRU FROX PQV PLGKHLJKW VSOLFH LV QRW UHTXLUHG FROXPQV in buildings assigned to SDC D, E, or F are required too be n length according n ACI 318-14. Bottom and nd Sections 18.7.4.3 and 18.14.3.3 in ns are illustrated t mid-height column splice locations in Fig. 9.7.2.2a. For lap splices than are about one-third to one-half the he story height, it may be more economical to lap-splice the EDUVHYHU\RWKHUARRU&RQFUHWH5HLQIRUFLQJ6WHHO,QVWLWXWH (CRSI) 2011). The length of lap splices should be noted on WKHVWUXFWXUDOGUDZLQJV/DSVSOLFHVYDU\ZLWKWKHEDUGLDPeter, concrete strength, bar spacing concrete cover, position of the bar, distance from other bars, and if the bar is tension RU FRPSUHVVLRQ /DS VSOLFHV DUH QRW SHUPLWWHG IRU 1R and 18 bars, except for transferring compression (only) to a footing with dowels (ACI 318-14, Section 16.3.5.4). To maintain bars in the corners of rectangular column, longitudinal bars that are lap spliced are usually offset bent into the column size does not change in column size or not. Circular columns size does not change in column size does not change in column size does not change in column size or not. and are placed not more than 6 in. from the point of the bend (ACI 318-14, Section 7\SLFDOO\WKUHHFORVHO\VSDFHGWLHVLVVXI¿FLHQWWR resist the lateral force created by the bend and one of the ties may be part of the regularly spaced ties. Separate splice bars and more ties may be necessary where the column section changes 3 in. or more. Examples of offset bent splices are illustrated in Fig. 9.7.2.2b. Where there is a reduction of orcement, longit reinforcement, longit udinal bars from the column below are \ WH PLQDWHG ZLW KLQ LQ LQ RI WKH WRS RI WKH ¿QLVKHG W\SLFDOO\ WHUPLQDWHG ZLWKLQ \$&, XQOHVV GHVLJQ UHTXLUHV RWKHUZLVH Col n the column bar area in column bar area in column bar area in column bar area in column bar area typical Bundled typically groups of larger bars that SDQ WZR WZR VW H EHDULQJ VSOLFHV LQ EXQGOHG VSDQ VWRULHV /DS DQG HQG bars require quire staggering gering the individual bars. For this reason, bbundled bars preassem preasem pre The one and two-piece tie arrangements shown provide maximum rigidity for column cages preassembled on the site before erection. The spacing of ties required for shear is shown in Table 10.7.6.5.2 in ACI 318-14. 9.7.3.2 Spirals—Spirals are used primarily for circular columns, piers, and caissons. Spiral reinforcement can be plain or deformed bars or wire. The term "spiral" used in the code is more than a geometric description of a circular tie. It GH¿QHVWKHUHTXLUHGSLWFKUHLQIRUFHPHQWDPRXQWVSOLFLQJ and termination, which are listed in Section 25.7.3 in ACI 318-14. A continuouslywound bar or wire not meeting all of the requirements of 25.7.3 is simply a continuous circular tie. The spiral pitch is between 1 and 3 in., inclusive, and is typically given in 1/4 in. increments. The spiral size and pitch should meet the volumetric reinforcement ratio, rs (Eq. (25.7.3.3) in ACI 318-14). The continuation of spirals into ARRUMRLQWVLVDFFRUGLQJWR7DEOH7KHPLQLPXP diameters to which standard spirals can be formed is given in Table 9.7.3.2 (ACI 315-99). American Concrete.org 359 Columns CHAPTER 9-COLUMNS Fig. 9.7.2.2b-Column splice details (ACI 315-99). Table 9.7.3.2-Minimum diameters o spiral reinforcement Spiral bar diameter, in. Minimum outside diameter that can be formed, in. 3/8 9 1/2 12 5/8 15 3/4 30 9.8—Design steps 1. Determine an initial size of the column is often dictated by the architect; otherZLVHVTXDUHRUURXQGFROXPQVDUHFRPPRQ¿UVWHVWLPDWHV (c) Reinforcement ratios are typically 1 to 2 percent 2. Run initial analysis to determine column loads. (a) If second-order effects are not accounted by the computer program, use the Moment 0DJQL¿FDWLRQPHWKRGLQ6HFWLRQLQ ACI 318-14 to calculate these effects 3. Create a moment-interaction diagram using an electronic spreadsheet or commercial software. Check the required moment and axial load strengths against the design strength curve. American Concrete Institute – Copyrighted © Material – www.concrete.org 360 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 9.7.3.1—Standard column ties (ACI 315-99). 0DNHDGMXVWPHQWVDVQHFHVVDU\WRWKHLQLWLDOFROXPQ size and reinforcement. Rerun the analysis, if necessary, until design strength is greater than required strength for all load cases. 5. Check shear strength and minimum shear reinforcement requirements. 6. Detail column longitudinal and transverse requirements showing all bar locations, spacing, splices, and bar terminations. It is common to use typical column details and sections along with a column schedule table. REFERENCES American Concrete Institute (ACI) ACI 315-99—Details and Detailing of Concrete Reinforcement ACI 318-63—Building Code Requirements for Structural Concrete
and Commentary ACI 318-14—Building Code Requirements for Structural Concrete and Commentary ACI 318-14—Building Code Requirements for Structural Concrete and Commentary ACI 318-14—Building Code Requirements for Structural Concrete and Commentary ACI 318-14—Building Code Requirements for Structural Concrete and Commentary ACI SP-17DA-14—Reinforced Concrete Design Handbook Design Aid – Analysis Tables; https://www.concrete.org/store/product/store/product/store/product/store/store/product/store/sto ItemID=SP1714DA ACI SP-17DAE-14— Interaction Diagram Excel spreadsheet; American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 9—COLUMNS https://www.concrete.org/store/productde/sp1714DAE \$PHULFDQ & RQFUHWH (OVWLWXWH) DUPLQJWRQ + LOOV 0,9 No. 6, June, pp. 79-80. Concrete Reinforcing Steel Institute (CRSI), 2011, "Detailing Concrete Columns," Concrete International, \$PHULFDQ & RQFUHWH, QVWLWXWH )DUPLQJWRQ + LOOV 0,9 No. 8, Aug., pp. 47-53. Columns Authored documents Fanella, D., 2007, Seismic Detailing of Concrete Buildings633RUWODQG&HPHQW\$VVRFLDWLRQ6NRNLH,/SS Frosch, R., 2011, "Using an Elastic Frame Model for Column Slenderness Calculations," Concrete Institute – Copyrighted © Material – www.concrete.org 362 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 9.9 Example 1: Column analysis \$QDO\]H¿UVWÀRRULQWHULRUFROXPQLQRQHGLUHFWLRQDWORFDWLRQ(IURPWKHH[DPSOHEXLOGLQJJLYHQLQ&KDSWHURIWKLV +DQGERRN7KHPRPHQWPDJQL¿FDWLRQPHWKRGLVXVHGZLWKD¿UVWRUGHUDQDO\VLV7KHFROXPQ¶VIDFWRUHGIRUFHVDQGPRPHQWV DUHIURPD¿UVWRUGHUIUDPHDQDO\VLVXVLQJKDQGFDOFXODWLRQV7KHEXLOGLQJZDVDOVRDQDO\]HGE\¿UVWRUGHUDQGVHFRQGRUGHU elastic methods using commercial software for comparison purposes. The factored moments are from an analysis of the moment frame along Grid E. Three common controlling load combinations are considered. Given: Materials— 6SHFL¿HGFRQFUHWHFRPSUHVVLYHVWUHQJWK fcg= 5 ksi 6SHFL¿HGCVWUHQJWK fcg= 5 ksi 6SHFL¿HGCVWUHQJWK fcg= 60 ksi Modulus of elasticity of concrete, Ec = 4030 ksi Loading— /RDGVFRQVLGHUHG /RDG Combination /RDGEUHDNRXW Vu, kip Pu, kip (Mu)top, kip-in. Dead d + live + sn snow Dead + wind + live l e + snow Dead D + EQ + live + snow 2D L+0.5S (i) U = 1.2D+1.6L+0.5S (U=1.2D+1.0U+0.5S U=1.2D+1.0U+0.5S U=1.2D+1.0U+0.2S (ii) 1.2D + 1.6L + 0.5S 0 867 1.2D + 1.0L + 0.2S 0 789 1.0E 22 0 24 12 ±418 12 ±1740 [] [] (Mu)bot, kip-in. ±650 ±2328 7KHORDGIDFWRURQ'LQ/RDG&RPELQDWLRQLLL LVLQFUHDVHGDVUHTXLUHGE\\$6&(E\SDS. Note that d = 1.0 for buildings in SDC B. \* Reference: SP-17 Supplement, Reinforced Concrete Design Handbook Design Aid – Analysis Tables, found at https:// www.concrete.org/store/productdetail.aspx?ItemID=SP1714DA, ACI 318-14 Discussion Calculation Step 1—Determine initial column size Estimate the maximum load at the interior column, (XVLQJ/RDGFRPELQDWLRQL 5.3.1 U = 1.2D + 1.6L + 0.5S Pu = 867 kip area = = 434 in.2 force by  $0.4fcg 0.4 \times 5$  ksi h = 434 = 20.8 in. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS Complete a rough check on slenderness: Concrete columns become very slender when NEu/r exceeds 45. Since this column is likely to be in a sway frame, assume a k = 1.5 and determine a size WKDWVDWLV¿HVNEu/r < 45. r = 0.3h and kAu = 45 r Eu =  $18 \times 12 - 30 = 186$  in. Rearranging terms and solve for h:  $h = 1.5 \times 186 = 20.7$  in.  $0.3 \times 45$  It is common for an engineer to choose a large enough column sizes are often rounded to the nearest increagainst sidesway, a NEu/r limit of 22 would allow ment of 2 in.) the engineer to ignore slenderness. This example, however, will consider slenderness and show the full set of calculations needed for a slender column. Step 2—Sway or nonsway moment frame Fig. R6.2.6 \$ARZFKDUWKHOSVWKHHQJLQHHUGHWHUmine column analysis options. This example shows the full extent of the provisions when done by hand 7KHVHFDOFXODWLRQVFRXOGHDVLO\EHSURJUDPPHGLQDVSUHDGVKHHW7KH¿UVWVWHSLQWKHARZFKDUWLVWRGHWHUmine if the structure is a sway or nonsway greatly tly reduces he Code provides three the required calculations. The ame to be considered as options to permit the frame nonsway greatly tly reduces he Code provides three the required calculations. The ame to be considered as options to permit the frame nonsway greatly tly reduces he Code provides three the required calculations. re ar 6.2.5 1) The stiffness of al all the bracing elements in a There are no bracing elements in the direction of the men frame. story are at least 12 timess the stiffness of all the moment n of evaluation. Bracing racing columns in the di direction al means ans walls but bra es ar elements in the direction of the men frame. order moment 6.6.4.3(a) 2) The increase off co column moments due Second moments are calculated in Step 4. The second-order effects does not exceed 5 percent. WKH¿UVWRUGHUHQGPRPHQWV effects 6.6.4.3(b) 3) Q, in accordance with Section 6.6.4.1, does not  $\Sigma$  Pu  $\Delta$  o 6.6.4.4.1 exceed 0.05. Q is determined for a single story and Q = Vus A c FRQWUROOLQJORDGFRPELQDWLRQ'LIIHUHQWARRUVRI where, the same moment frame have different Q values. (30) (7) A c = 18 × 12 - | + | = 204.5 in. (2) (2) Ec is the height of the column from center-to-center of the joints. To calculate Q, the load case must consider lateral Check Load Combination (ii), which has a larger axial load than Load Combination (ii), which has a larger axial load than Load Combination (iii), which has a larger axial load than Load Combination (iii), which has a larger axial load than Load Combination (iii). The value was derived from the loads given in Chapter 1 of this Handbook. Vus is the total factored horizontal story shear for Vus = 775 kip Load Combination (iii). The value was calculated from the lateral forces calculated in Chapter 1 of this Handbook. American Concrete Institute – Copyrighted © Material – www.concrete.org Columns 6.2.5 6.2.5. 363 364 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) all VWRUGHUVWRU\GHAHFWLRQGHWHUPLQHGE 3 3 Vus [( 2 × A c ) ( A c )] an elastic analysis. For this hand calculation exΔo = + || 3 Σ EI || 3 [/ |] DPSOHDVLPSOHDSSUR[LPDWLRQRIGHÀHFWLRQLVSURYLGHG7KH¿UVWVWRU\RIDEXLOGLQJLVRIWHQDVVXPHG to have a hinge at 0.678c. The following equation Ec = 4030 ksi SURYLGHVGHÀHFWLRQDWDGLVWDQFHE to the hinge. I = 24 × 243/12 = 27,650 in.4 3 TEI = 37 columns × 0.5 × 4030 × 27,650 V × (A)  $\Delta$  = us = 2.06 × 109 kip-in.2 3  $\Sigma$  EI 6.6.3.1.2 The value for stiffness should be reduced for cracking. A value of 0.5 is commonly used for all members in an analysis calculated by hand. åo = 0.36 in. Q= 1RWHWKDWWKHGHAHFWLRQIURPD¿UVWRUGHUHODVWLF analysis from software that accounts for the relative stiffness of all the members is 0.64 in. The advanRQRIODWHUDOGHAHFWDJHRIDPRUHDFFXUDWHFDOFXODWLRQRIODWHUDOGHAHFtion is discussed in furtherr detail in Step 4. Step 3—Check to see if slenderness can bbe neglected ndern ratio, NEu/r. The notation Compute the slenderness RUWH JWK7KHARRU RRU EuLVWKHXQVXSSRUWHGOHQJWK7KHARRUWRARRU WD HÀRRUDWWKHVHF QGOHYHOLV GLVWDQFHLVIWDQGWKHÀRRUDWWKHVHFRQGOHYHOLV 30 in. deep. To calculate the ef effective ve length factor k for the th column, the member framing att the top mbe stiffnesses ffnesses framin DQGERWWRPMRLQWVQHHGWREHFDOFXODWHG 25, 700 × 0.36 = 0.058 > 0.05 775 × 204.5 Therefore, th frame is sway. PRUHDFFXUDWH UVWRUGHUDQDO VLVVKRZVWKDW Q = 25, 700 × 0.64 = 0.104 775 × 204.5  $\varepsilon_u = 18$  ft × 12 in./ft - 30 in. = 186 in. T-Beam column): Team (at top of colum bf = 12 120 in. (calculated calculated in Beam Ex. 1) bw = 18 in. n. hf = 7 in. h = 30 in. For rectangular sections, I = bh3/12.  $P_{1} = 10 \times 7 \times 3.5 + 18 \times 23 \times 18.5 = 8.5$  in.  $120 \times 7 \times 18.5 = 8.5$  in.  $120 \times 7 \times 3.5 + 18 \times 23 \times 18.5 = 8.5$  in.  $120 \times 7 \times 3.5 + 18 \times 23 \times 18.5 = 8.5$  in.  $120 \times 7 \times 5.02$  12  $18 \times 233 + 18 \times 23 \times 10.02 = 84,100$  in. 4 12 Slab (at bottom of column): h = 7 in. b = 14 ft  $\times 12$  in. = 168 in. See the plan in Chapter 1 for plan dimensions. Islab = 168 × 73/12 = 4800 in 4 Column (all levels): h = 24 in. b = 24 in. lcol = 24 × 243/12 = 27,650 in 4 American Concrete Institute – Copyrighted © Material – www.concrete.org ) CHAPTER 9—COLUMNS Table 6.6.3.1.1(a) & DOFXODWHDGMXVWHGEI values. For this part, a more detailed values for EI are used. 365 Ec = 4030 ksi R6.2.5 Joint at top of column: Factor kUHAHFWVFROXPQHQGUHVWUDLQWFRQGLWLRQV which depends on the relative
stiffness of the col78 × 106 + XPQVWRWKHARRUPHPEHUVDWWRSDQGERWWRPMRLQWV 204.5 168 \$WWKHWRSMRLQWWKHFROXPQVIUDPHLQWREHDPVDQG Ψ A = 119 ×  $106\ 119 \times 106\ DWWKHERWWRPMRLQWWKHFROXPQVIUDPHLQWRDWZR + 432\ 432\ way\ slab.$  Find the ratio of column stiffness to beam, right] Joint at bottom of column: Fig. R6.2.5 Read k from the nomograph. nom For a sway sw frame, rame, key set for the nomograph. Stiffness to beam, right] Joint at bottom of column: Fig. R6.2.5 Read k from the nomograph. nom For a sway sw frame, rame, key set for the nomograph. kk§ (Note: k§ (N te: For F a nonsway onsway frame, fra 6.2.5.1 Determine the rad radius off gyr gyration, r Ig = 27, 27,650 in.4 Ag = 57 576 in.2 r = 6.2.5 Ig 78 × 106 + 204 204.5  $\Psi$ B =  $4.8 \times 106 + 204 204.5 \Psi$ B = 23.0 r = Ag Note that Section 6.2.5.1 also allows an approximation of 0.3 times the width of the column in the direction of the frame, which would be  $0.3 \times 24 = 7.2$  in this case. Check to see if slenderness can be neglected for sway frame, there are two limits that must be met: (M) kAu kAu  $\leq 34 + 12$  | 1 | and  $\leq 40$  r r (M2 / 8VH/RDG&RPELQDWLRQLLL WRcQGM1 and M2. 27,560 60 = 6.9 576 For a sway frame,  $2.2 \times 186 = 59 > 22$  6.9 Slenderness cannot be neglected. Note: For a nonsway frame: k A u 0.9 × 186 = 24 r 6.9 (1752)  $24 \le 34 + 12$  = 40.9 and  $24 \le 40 \setminus 2352$  |/ therefore, slenderness could be neglected if this condition occurred. American Concrete Institute – Copyrighted © Material www.concrete.org Columns 0.35(EcI) beam =  $0.35 \times 4030 \times 84,100 = 119 \times 106$  kip-in.2 0.25(EcI) slab =  $0.25 \times 4030 \times 4800 = 4.8 \times 106$  kip-in.2 0.70(EcI) col =  $0.70 \times 4030$  x  $27,650 = 78 \times 106$  kip-in.2 366 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4—Determine second order effects for PåVZD For a sway frame, the secondary moments at the )URP/RDG&RPELQDWLRQLLL DERYH end of the column due to differential movement of the ends of column must be calculated before Mns Ms calculated before Mns Ms calculating deformations along the length. Note that 1.0E 1.2D + 1.0L + secondary moments are often called "Pa'XSSHU 0.2S case delta) effects by most reference materials, but M1NLSLQ 12 ±1740 are labeled "Pis ORZHUFDVHGHOWD LQ\$&, M NLSLQ -24 2 6.6.4.6.1 ± 7KH&RGHSURYLGHVDFRQVHUYDWLYHPHWKRGWRHVWL mate this effect. The equations are M1 = M1nsisM1s M2 = M2nsisM2s 6.6.4.6.2 Eq. D :KHUHWKH¿UVWRUGHUPRPHQWVGXHWRJUDYLW\ORDGV IRUDVLQJOHORDGFRPELQDWLRQM1ns] are added to WKH¿UVWRUGHUPRPHQWVGXHWRODWHUDOORDGVM1s) PXOWLSOLHGE\WKHVZD\PRPHQWPDJQL¿FDWLRQIDF WRUjs. 7KHVZD\PRPHQWPDJQL¿FDWLRQIDF  WRUjs. 7KHVZD) (FDWLRQIDF WRUjs. 7KHVZD VRIWZDUHWKDWXVHVDVHFRQGRUGHUDQDO/VLV American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS R6.6.4.4.2). Eleff may be calculated from one of three equations in 6.6.4.4.4. Use Eq. (6.6.4.4.4a) since the column reinforcement is not known at this point of the design. Key points about Eleff are (a) Commentary Section R6.6.4.4.4, explains the differences in the equations E)RUVZD\IUDPHV\u00fcds LVVXEVWLWXWHGIRU\u00fcdns and is 0.0 for short term lateral loads (c) Ise in Eq. (6.6.4.4.4b) may be calculated using Table D.4.5 in the supplement to this Handbook, see reference at the start of this example.  $Pc = \pi 2$  (EI) eff (kAu)2 (EI) eff =  $0.4 Ec Ig 1 + \beta ds 0.4 \times 4030 \times 27$ , 650 kip-in.2  $1 + 0 = 45 \times 106$  (isparity between the two results (1.06 versus 1.54) is un unusual. This disparity

indicates that the n. column stiffness stiff ss m 24 in. may be outside normal bounds d th for stabl stable results, an and the engineer should consider reasi the he column ssize. increasing Discussion: HE F\*UHJRUDQG+D H LWLVVX JHVW åo iin Eq. ,QWKHUHIHUHQFHE\0DF\*UHJRUDQG+DJH LWLVVXJJHVWHGWKDWå Eq ((6.6.4.6.2a) be taken from a \VL QJDFRPSXWHUSU JUDPWKDWDFFRX WVIR HPEHUVWL ¿UVWRUGHUDQDO\VLVXVLQJDFRPSXWHUSURJUDPWKDWDFFRXQWVIRUWKHPHPEHUVWLIQHVVHV\$VQRWHGLQ6WHS HUD HFWLRQLVLQ DQGWKHYDOXHI Q becomes b s 0.104. Thus, T WKHVRIWZDUHODWHUDOGHÀHFWLRQLVLQDQGWKHYDOXHIRUQ the revised is, for Eq. (6.6.4.6.2a) is  $\delta s = 1 = 1.121 - 0.104$  The MacGregor and Hage (1977) reference is informative and describes the Q method. A few helpful suggestions from the reference are (a) For Q values between 0.05 and 0.2, the error in second-order moments will be less than 5 percent E åo/H (story height) should be less than 1/500 for nonsway frames and less than 1/200 for sway frames at factored loads F <sup>M</sup>Pu<sup>M</sup>Pcr in Eq. (6.6.4.6.2b) should be less than 0.2, QWKLVH[DPSOH<sup>M</sup>Pu<sup>M</sup>Pcr is 0.26 > 0.2, so the results from Eq. (6.6.4.6.2b) are becoming questionable. Q

FDOFXODWHGIURPGHÅIFWLRQVED/UVWRUGHUFRPSXWHUDQDOV/LVLZKLFKUVLQWKHUDQJHXXJJHVWHGEQ0FGregor and Hage; thus, the second order-moments calculated by the Q method should be within 5 percent of WKHDFWXDQJIWKH86 from the computer analysis was not available, is would be prudent to use Eq. (6.6.4.6.2b) DQGLQFUHWKHVTXDUHFROXPQVLJHWRVDWLVI<sup>™</sup> Pu<sup>™</sup>Pc < 0.2 & QDOFXODWHWKHVDQJUKJHS = 1.2 + 1950 = 1962 kip-in. M2 = M2nsjM2s = 2.4 + 2610 = 2634 kip-in. American Concrete Institute - Copyrighted © Material - www.concrete.org Columns 6.6.4.4.2 & 66.4.4.4 & 367 - 2620 kip-in. Step - 2620 kip-in. St

2QHFDQVHHWKDWWKHKDQGFDOFXODWLRQWR¿QGWKHVHFRQGRUGHUPRPHQWLVPRUHFRQVHUYDWLYHWKDQLQDFRPSXWHUDQDO\VLV The second-order moment increase for the computer is 10 percent which is lower than the 18 percent calculated by the moPHQWPDJQL¿FDWLRQPHWKRG1RWLFHWKDWKHQPHWKRGXVHGLQ(TD ZDVDLGHGZLWKDao computed by a computer DQDO\VLV,IWKDWGHAHFWLRQZDVQRWDYDLODEOH(TE ZRXOGKDYHEHHQXVHGDQGWRWDOLQFUHDVHRISHUFHQWZRXOG have been required which is more than the 40 percent allowed. Thus, a larger column would have been selected as suggested in the discussion in Step 4. American this Handbook. The column is part of an ordinary moment frame. Example LVWKHGHVLJQRIWKHFROXPQDQDO\]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL¿HGWRPDWFKWKHUHVXOWVRIDQDQDO\]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL¿HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGUNGVKDYHEHHQPRGL?HGWRPDWFKWKHUHVXOWVRIDQDQDO]HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHQPDQDQDO]HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDYHEHHQPRGL?HGWNGVKDY elasticity of steel, Es = 29,000 ksi 6SHFL¿HGFRQFUHWHFRPSUHVVLYHVWUHQJWK fcq NVL Modulus of elasticity of concrete, Ec = 4030 ksi Normalized maximum size of aggregate is 1 in. Loading— /RDGFRPELQDWLRQV (i) U = 1.2D + 1.6L + 0.5S (ii) U = 1.2D + 1.0E1.0L + 0.2S Pu, kip 890 800 818 486 Mu, kip-in. 0 651 2401 2401 Vu, kip 0 5 18 18 7KHVRIWZDUHDGMXVWVVHLVPLFORDGFRPELQDWLRQVDVUHTXLUHGE\\$6& s productdetail.asp p ?Item Ite Reference: SP-17 Supplement, Interaction Diagram Excel spreadsheet found at ACI 318-14 Discussion D Calculation C Step 1—Find the required areaa of longitudinal reinforcement tudinal reinfor ment l en ection 22.4 for thee calculation of Pn and Mn. Section The Code references nces Section 22. 22.2 for strain limits. evaluated by y making aan interaction tera diagram diagram. A tutorial spreadsheet is proThe interaction off Pn and Mn is evaluat m for a giv vided with this manual that demonstrates how to make a diagram given section and reinforcement The section properties and geometry were determined in the analysis from Column Example 1. The next step is to assume a quantity, size, and location of the longitudinal reinforcement. The design of columns is RIWHQDQLWHUDWLYHSURFHVV7KH¿QDOGHVLJQPD\VKRZWKDWDODUJHUVHFWLRQRUPRUHUHLQIRUFHPHQWLVQHHGHG in which case, the analysis will need to be redone. The spreadsheet analyzes rectangular or square colXPQVIRUFRPELQHGD[LDODQGÅH[XUDOVWUHQ]WK7KH designer inputs the number of steel layers in the colXPQFURVVVHFWLRQ7KHVSUHDGVKHHWSODFHVWKH¿UVW and last layer as close to the outer face as permitted (concrete cover plus tie bar size). 10.6.1.1 The remaining layers are evenly spaced between the outer layers. The spacing of the bars within each layer is similar to the placement of the layers. The similar to the placement of the bars within each layer is similar to the placement of the bars within each layer. © Material – www.concrete.org CHAPTER 9—COLUMNS 1 22.2.2.4.3 1 25.7.2.2 1 20.6.1.3.1 The design moments are not very large, so try the minimum area of reinforcement and assume a uniform distribution of bars around the perimeter. /RQJLWXGLQDOEDUVDUHW\SLFDOO\ODUJHUEDUV1RDQG greater, to make stable column cages for erection and to reduce the number of ties at a section. 1 21.2 1 Try 8 bars, one in each corner and one on each side (3 layers: 3 bars, 2 bars, 3 bars) Area of a No. 8 bar is 0.79 in.2; therefore, try (8)-No. column. 8 bars The spreadsheet does a sectional strength analysis using an equivalent rectangular stress distribution Section properties and geometry DFRUGLOIWR6HFWLRO7KHuLVDIXOFWLRORI No. lavers: 3 fc' which is automatically calculated and displayed ' = 5.000 psi f c for the user's information. No. 4 ties are a common starting size since they provide good initial shear strength and are rigid enough to provide column cage stability during erection. Concrete cover protects the reinforcement from HFWLRQ7KHPLQLFRUURVLRQDQGSURYLGHV¿UHSURWHFWLRQ7KHPLQLFRUURVLRQDQGSURYLGHV¿UHSURWHFWLRQ7KHPLQLn Section 20.6. mum cover is provided in The spreadsheet calculates alcula the distances to the cem c iss needed eede for later e layers of reinforcement, which y us he tie bar size, ccover er to tie, calculations, by using the al bbar size. ze. and longitudinal 125.2.3 As,min =  $0.01 \times 242 = 5.76$  in.2 "u 0.80 b= 24 in. h= 24 in. fy = 60 ksi ksi Es = 29,000 Tie bar size: 4 C Clear cover to tie = 1.50 /RQJED/VL]H 8 Number of bars per layer /D\HU D\HU di, in. N No. long. bars Asi, in.2 3 2.500 00 3 2.37 2 12.000 000 2 1.58 1 1 500 21.500 3 2.37 M 6.32 The minimum bar spacing is calculated and displayed for the user's information. Key variables needed for design are displayed for the user's information. Key variables needed for design are displayed for the user's information. tensioncontrolled sections. in. Bar spacing checks d1 = 21.50 in. c/c bar sp. (h) = 9.50 in. 2. c/c bar sp. (b) = 9.50 in. 2. Min. clear sp. (in.) = 1.50 in. (25.2.3) 6WUDLQGH $\stackrel{?}{c}QLWLRQV$  fy/E = 0.00207 lcu = 0.003 Ductile Strain = -0.005 Brittle Strain = -0.005 Br calculating the Pn and Mn for incremental changes in the net tensile strain of 0.0007 was chosen to illustrate the calculation of Pn and Mn in the following steps. 9DU\İt from pure compression, İt equal to fy/Es at Find Pn and Mn for İt = 0.0007 all layers, to pure tension, -fy/Es at all layers. American Concrete Institute - Copyrighted © Material - www.concrete.org Columns 10.7.3.1 371 372 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 2. Calculate c, the distance from the extreme comSUHVVLRQ¿EHUWRWKHQHXWUDOD[LVIRUWKH]LYHQİtE\ XVLQ]VLPLODUWULDQ]OHV c=  $c = -0.003 \times 21.5 = 28.09$  in.  $0.0007 - 0.003 - \varepsilon$  cu × d1  $\varepsilon$  t -  $\varepsilon$  cu 3. Calculate aWKHGHSWKRIWKHHTXLYDOHQWVWUHVV EORFN [ $\beta \times c = min$ ] 1 [h [  $0.80 \times 28.09 = 22.47$  in. a = min] 1 [h ] 0.8 $0.00172 = 0.003 \ 0.003 \times 0.003 \times 0.003 \times 0.003 \times 10^{-0.000} = 0.00273 \ 28.1 \ 10^{-0.0000} = 0.00273 \ 28.1 \ 10^{-0.0000} = 0.00207 \ 10^{-0$ 1RWHWKDWSRVLWLYHVWUDLQLQGLFDWHVWKDWWKHEDULVLQ Fs2 ii±ii NLS compression. The force is adjusted to account for the Fs3 ii±ii NLS displaced concrete. ,IIsi"WKHQFsi Isi × Asi × Es 7. Calculate Po. Po = Cc<sup>™</sup>Fsi Po NLS Note that Pn, max is not applied in this spreadsheet XOWLOWKHGHVLJOVWUHOJWKFXUYHLVFDOFXODWHG 8. Calculate di, the moment arm to forces Cc and each Fsi. dCc = di = h a - 2 2 h - di 2 24 22.47 - = 0.77 in. 2 2 d1 = 12 - 21.5 = -9.5 in. d2 = 12 - 21.5 in. d2 = 12 - 21.5 in. d2 = 12 - 21.5 in. d2 = 12 - 21373 9. Calculate Mi, the moment for each force about the center of the section. MCc =  $dCc \times Cc M Cc = 0.77 \times Mi = di \times Fsi M 1 = -9.5 \times M2 = 0 \times 2292 = 147$  ft kip 12 108 = 0 ft kip 12 108 = 0 ft kip 12 108 = 0 ft kip 12 12.2 12.4.2.1 Mn = 147 - 30.3 + 0 + 105 = 222 ft kip Mn = MCc<sup>™</sup> Mi The spreadsheet changes the strain in small increDesign Strength Interaction Diagram: ments to create a smooth plot of a nominal strength interaction diagram. The values are then multiplied by ] and the limits on axial strength are applied eraction diagram. The values are then multiplied by ] and the limits on axial strength interaction diagram. IRUWLHV7KH[DFWRUIRUDOOWHQVLRQFRQWUROOHG is fac sections is 0.9. This factor varies linearly from 0.65 he transition tr n zone show ich crecre to 0.9 through the shorp change n the right ht is for this column exe The diagram on ads from the fou ample and it has the loads four given load ted combinations plotted. Use se eight No. 8 bars ba evenly spaced around the met perimeter. Note that points for load combinations (i), (ii), and (iii) are to the left of the "Stress = 0 fy'/LQHPHDQQHPHDQ ing all the bars remain in compression. Point iv) is for a load combination with lighter gravity loads. ,WLVMXVWWRWKHULJKWRIWKHOLQHPHDQLQJDIHZEDUV will be in tension. Thus, a tension splice is required. Another line could be drawn on the diagram showing where the tensile bar stress is 0.5fy. That line delineates whether a Class A or B splice is required. 10.7.5.2.2 For this example, all the bars are going to be spliced at one location so a Class B splice is always required. Step 2—Find the required area and geometry of transverse reinforcement 1 The Code references Section 22.5 for the calculation /RDG&RPELQDWLRQLLL 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLL 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLL 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLL 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLL 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of Vn. The PulLYHQLQ 22.5 of Vn. The PulLYHQLQ/RDG&RPELQDWLRQLA 22.5 of V ASCE 7-10. The Pu value shown KHUHLVFKDQJHGWRUHAHFWSDS in the upward position for the lowest axial load associated with this load combination. 18.3.3 Note that Eu (186 in.) is greater than 5c1 (120 in.); therefore, the additional shear requirement for ordinary moment frame columns does not apply. American Concrete Institute Copyrighted © Material – www.concrete.org Columns 10. Calculate Mn. 374 Eq. (22.5.5.1) THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The shear force resisted by the column is minimal in this case. It is conservative to ignore the effect of the compression load on shear resistance. Use Eq. WR¿OGVc. 21.5 = 73.0 kip  $\geq$  18 kip 1000 Vc = 2λ f c'bw d Vc = 2 × 1.0 5000 × 24 × 10.6.2 1 21.2 Minimum shear reinforcement is required when Vu ! Vc Check for minimum shear reinforcement 18 kip 8 0.5 × 0.75 × 73 = 27 kip 1 25.7.2 Ties are not needed for shear but they are necessary for lateral support of longitudinal bars. Thus, tie requirements must meet the geometry requirements of 25.7.2 and location requirements of 10.7.6.2 Column ties are not necessary for shear resistance. 10.7.6.2 25.7.2.2 The minimum tie bars size is No. 3 for longitudinal bars No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 3 for longitudinal bars No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 3 for longitudinal bars No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum tie bars size is No. 4 tie was chosen as dis No. 4 tie was chosen as discuss requirements controls ample. Use No. 4 ties. acing shall not exceed the least of: The maximum spacing in Controls. (a)  $166 \times 1.00$  in. = 16 in. ongi l bar (b)  $48 \times 0.50$  in. transverse se bar (b)  $48 \times 0.50$  in. transverse se bar (c) 24 in in. (c) h or b 25.7.2.3 Fig. R25.7.2.3 Fig. R25.7.2.3 requires that support from ties is provided for bars at every column corner and also bars with greater than 6 in. clear on each side. ort in this Thus, every vertical bar needs lateral support example U Use s = 16 in. on center (o.c.) i o.c. Use N No. 4 ties @ 16 in. Ties with 90-degree standard hooks are acceptable for this case. A diamond-shaped tie is used to support

bars along the sides. This is desirable because it provides a strong column cage for erection; however, it becomes a fabrication problem when the column is rectangular and this tie becomes a fabrication problem when the column is rectangular and this tie becomes a fabrication problem. It is common to use alternative tie geometry, such as cross ties, for columns that are not square. 6WHS<sup>2</sup>&KHFNWKHMRLOW 7KHWUDQVIHURIFROXPQD[LDOIRUFHWKURX]KWKHMRLQWPXVWEHFKHFNHG,WLVFRPPRQWRKDYHKLJKHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQVDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWHYMUHQWKHYNUHQWYUHQQWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHFROXPQNDQGDORZHUVWUHQJWK 15.3 FRQFUHWHLQWKHYNUHQJWK 15.3 FRQFUHWH difference. This building uses the same fc'IRUERWKWKHFROXPQVDQGARRUV\VWHPVRQRDGGLWLRQDOFDOFXODWLRQVRUDGMXVWPHQWVDUH necessary. 15.2.2, 6HFWLRQVDQGUHTXLUHWKDWEHDPFROXPQMRLQWVEHGHVLJQHGDQGGHWDLOHGIRUVKHDULI 15.2.3, moment is being transferred to the column. Section 15.4 does not require minimum shear reinforcement 15.2.4, WKURXJKWKHMRLQWLVUHVWUDLQHGLQDFFRUGDQFHZLWKRUDQGLVQRWSDUWRIDVHLVPLF 15.2.5, force-resisting system. Note that this exception may be superseded by Chapter 18 as noted in Section 15.2.3. 15.4.1 This column is part of an ordinary moment frame; therefore, it is part of the seismic-force-resisting system. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 9-COLUMNS Section 15.4.2 applies which requires that minimum shear reinforcement is provided in the joint region even if minimum shear reinforcement is not required in the column. Av shall be at least the greater of:  $0.75 \text{ f}' 50 15.4.2.1 15.4.2.2 \text{ At each tie, Av} = 0.20 \text{ in.} 2 \times 4 \text{ legs} = 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.34 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.} 2 \text{ 60, 000 bs fy } 50 \times 24 \times 16 = 0.32 \text{ in.} 2 \leq 0.80 \text{ in.}$ full depth of the deepest member. Find the maximum spacing in joint: s = 30 = 15 in. 2 Controls Thus, two ties are required along the depth of the joint at a spacing not greater than 15 in. ACI 352R Section 15.2.2 of the Code states that, "the shear resulting from moment transfer shall be considered in the GHVLJQRIWKHMRLQW 7KH&RGHGRHVQRWSURYLGHVSHFL¿FUHTXLUHPHQWVWRPHHWWKLVUHTXLUHPHQWLQ&K,I shear strength requirements of S Section 18.18.4 of the Code can be used. Note LVQRWVLJQL¿FDQWO\LQFUHDVHGE\WKHLQFUHDVHR WKDWMRLQWVKHDUVWUHQJWKLVQRWVLJQL¿FDQWO\LQFUHDVHGE\WKHLQFUHDVHGE\WKHDUUHLQIRUFHPHQWDVH[SODLQHGLQ KHVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULVVRORZIRUWKLVH]DPSOHWKDWDFKHFNRQVKHDULV understanding of jointt des for a full joint design, reference A ACI 562R, "Recommendations eam mn Connection n Monolithic Rein Reinforced Concrete Structures." ice and joint at Level 2 Step 4—Detail the column splice RE ODSVSOLFHVIRUWK YHUW PHQ ARRUOHYH ,WLVFRPPRQWREHJLQODSVSOLFHVIRUWKHYHUWLFDOUHLQIRUFHPHQWDWWKHARRUOHYHOLQRUGLQDU\PRPHQWIUDPHV ice aree at one location, a Class B tensio tension lap length iss provide provided and checked against the compression lap len length as a minimum. I Est is the greater of: Determinee Estt for a No. N 8 bar:  $\hat{1}$  (a) 1.3Ed zt = ze = zs 3 25.5.2 (b) 12 in. in. db = 1.0 in cb = 1.5 + 0.5 + 1.0/2 = 2.5 in. 10.7.5.2.2 n = 3, number of longitudinal bars along the splitting plane where, s = 16 in., spacing of ties  $\hat{1}$  f Atr = 4 tie legs × 0.2 in.2 = 0.8 in.2  $\psi$ t  $\psi$ e $\psi$  s 3 y = d A d b  $\hat{1}$  40 × 0.8 25.4.2 K tr = = 0.67 |\ d || 16 × 3 b and, K tr = 40 Atr sn ( cb + K tr ) ( 2.5 + 0.67 ) || d || = || 1.0 || = 3.17 ≤ 2.5 b Ad = 3 60,000 1.0 1.0 = 25.4 in. 40 1.0 5000 2.5 Est = 1.3 × 25.4 = 33.0 in. American Concrete Institute – Copyrighted © Material – www.concrete.org Columns 15.4.2 375 376 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) | For fy equal to 60 ksi, Esc is the greater of: 1 (a) 0.0005fy db = 30db 25.5.5.1 (b) 12 in. Check compression lap splice length. 10.7.5.2.1 For ease of construction, use Est = 33 in. for all splices and splice at every level. 10.7.4.1 10.7.6.4 A common way of expressing this splice on the structural drawings is to make a lap splice table and reference it in the detail. Note that it is common to splice at every other story to save time on labor in WKH¿HOG /DWHUDOVXSSRUWRIRIIVHWEHQGVLVSURYLGHGE\ column ties at bend. Ties need to resist 1.5 times the horizontal component of the computed force in the incline, 1 in 6 (9.5 degrees). The calculations shows that one additional No.4 tie can laterally support one No. 8 bar at the offset.  $Esc = 30 \times 1.0 = 30$  in. Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tie strength: No. 8 = (fyAst) = 60 \times 0.79 = 47.4 kip Horizontal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times
sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at the bend:  $Pu = 1.5 \times 47.4 \times sin(9.5^\circ) = 11.7$  kip Nominal tension at te for each vertical bar at its offset. The current tie detail meets this requirement. 10.7.6.2 15.4.1 7KH&RGHUHTXLUHVWKHcUVWWLHVWDUWLQJRQDQ\OHYHO to be within, s/2, of the top of the slab. It is good construction practice to start the tie at 2 or 3 in. ARRUE IURPWKHWRSRIWKHARRUEHJLQQLQJRIWKHFROXPQ ocee with the typical tie spacing. cage) and then proceed HF QFDJHERWWRP WKHARRU \$WWKHWRSRIWKHFROXPQFDJHERWWRPRIWKHARRU UH GWKURXJKEHDPVODEMRLQWLI OE UHQRWFRQ¿QHGE WKHF WKHORQJLWXGLQDOEDUVDUHQRWFRQ¿QHGE WKHFROXPQFDJHERWWRPRIWKHARRU UH GWKURXJKEHDPVODEMRLQWLI OE UHQRWFRQ¿QHGE WKHF WKHORQJLWXGLQDOEDUVDUHQRWFRQ¿QHGE WKHF WKHORQJLWXGLQDOEDUVDUHQRWFRQ¿QHGE WKHFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRWFRQźQHGE WKHFROXPQFDJHERWWRPRWFRQŚQHGE WKHFROXPQFDJHERWWRPRWFROXPQFDJHERWWRPRWFROXPQFDJHERWWRPRIWKHFROXPQFDJHERWWRPRWFROXPQFDJHERWWRPRWFROXPQFDJHERWWRPRWFROXPQFDJHERWWRPRWFR on all four sid sides. In WV EHDP GHSWK E Z WKH IUDPHLQWRWKHEHDPGHSWKEHORZWKH VODEGRHVQRWSURYLGHWRLQWFRQ¿QHPHQWDQG KH &RGH WLHVDUHQRWUHJLRQ7KH&RGH requires the last tie to be within RQWDOUHLQIRUFHPHQWLQWKHFRQ¿QHGFRQFUHWH Again, it is good construction practice to place a tie DWRULQEHORZWKHFRQ¿QLQJFRQFUHWH /RFDWHWKHWRSRIIVHWEHQGDWWKHWLHMXVWEHORZWKH slab. Provide a tie at the bottom offset bend. This WZRWLHVLQWKHMRLQWUHJLRQLQUHTXLUHG spacing. The tie requirements are not additive in this case. Step 4—Discussion and summary 7KHJLQFROXPQLVVDWLVIDFWRU\IRUGHVLJQ7KHPLQLPXPUHLQIRUFHPHQWHLJKW1RVLVVXI&FLHQWWR resist the factored loads and moments. Shear is very low and only the minimum tie area and spacing, No 4 WLHVDWLQRQFHQWHUDUHUHTXLUHGIRUVXSSRUWRIWKHORQJLWXGLQDOUHLQIRUFHPHQW2LOOZRUNIRUWKHUHPDLQLQJÅRRUVVLQFHWKHFROXPQVJHWVKRUWHUDQGWKHORDGV GHFUHDVH\$VPDOOHUFROXPQZLOOZRUNIRUWKHXSSHUARRUVEXWWKHFRVWRIDFKDQJHLQIRUPZRUNPD\QRW overcome the cost of the small amount of concrete that is saved. For this building, the architect may want to VDYHVRPHVSDFHRQWKHXSSHUARRUVDQGWKHUHPD\EHDFRPSURPLVHEHWZHHQIXQFWLRQDOLW\DQGH[SHQVH American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 9-COLUMNS 377 Column Example 3: Column for an intermediate moment frame (IMF 'HVLJQDQGGHWDLOWKH¿UVWÅRRULQWHULRUFROXPQDWORFDWLRQ(IURPWKHH[DPSOHEXLOGLQJJLYHQLQ&KDSWHURIWKLV+DQGERRN The column is part of an IMF. ([DPSOHLVDFRQWLQXDWLRQRI([DPSOHVDQG7KHORDGVKDYHEHHQPRGL¿HGWRPDWFKWKH results of an analysis from commercial software capable of second order elastic analysis and for Seismic Design Category C. Given: Loading— /RDG&RPELQDWLRQV (i) U = 1.2D + 1.0E + 1.0L + 0.2S (ii) U \*= 0.9D + 1.0E + 1.0L + 0.2S Pu, kip 890 800 848 56 456 Mu, kip-in. 0 651 3228 3228 Vu , kips 0 5 25 25 7KHVRIWZDUHDGMXVWVVHLVPLFORDGFRPELQDWLRQVDVUHTXLUHGE\\$6&(TXLUHGE\\$6&(TXLUHGE\\$ Reference: SP-17 Supplement, Interaction Diagram Excel spreadsheet found at productdetail.asp p ?Item Ite ACI 318-14 Discussion Calculation 6WHS<sup>2</sup>LVFXVVLRQRQPRGL¿FDWLRQRI([DPSOHIRUDQ,0) FDW ([DPSohirudQ,0) FDW ([DPsohirudQ,0) requirements The le demonstrates onstrates onstrates the colu mn design requi men for an IMF. Th h design requirements are I VLPLODUWRDQRUGLQDU/PRPHQWIUDPH20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRRSVDUHUHTXLUHGLQWKHSODVWLFKLQJHUHJLRQ QR DU/PRPHQWIUDP 20) H[FHSWWKDWKRXZ E0, DWDUHGXFHGVSDFLQJ7KHDGGLWLRQDOGHVLJQUHTXLUHPHQWVIRUDQ,0)LQFUHDVHVWKHUHVSRQVHPRGL&FDHG FLQJ7KHDGGLWL DOGHVLJQUHTXLUHPHQWVIRUDQ,0)LQFUHDVHLQVHLVPLF signed to SDC B and required for SDC C. base shear. An IMF is permitted to be used for structures assigned ear is so low that it is not economical to require For the column designed in Examples 1 and 2, the shear IWKHUHGXFHGVKHDU WKHHIWUDGHWDLOLOIWRIDLOWKHEHOH¿WRIWKHUHGXFHGVKHDU)RUWKLVH[DPSOHEXLOGLQ]LV DQDO\]HGIRUDUHJLRQDVVLJQHGWR6'&&DQG/RDG&RPELQDWLRQLLL DUHUHYLVHG Step 2-Find the required area of longitudinal reinforcement The same column from Example 2 is checked usDesign Capacity Interaction Diagram: ing the interaction diagram spreadsheet referenced at the start of this example. See Example 2 for a detailed discussion on the right is for this example and it has the loads from the four given load combinations plotted. Eight No. 8 bars evenly spaced around the perimHWHULVVXI¿FLHQW American Concrete Institute - Copyrighted © Material - www.concrete.org Columns Materials— 6SHFL¿HGFRQFUHWHFRPSUHVVLYHVWUHQJWK fcg NVL Modulus of elasticity of concrete, Ec = 4030 ksi Normalized maximum size of aggregate is 1 in. 378 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3—Find the required area of transverse reinforcement 17KHVKHDUKDVLQFUHDVHGLQ/RDG&RPELQDWLRQLLL 22.5 but it is still very low. The concrete design strength FDOFXODWHGLQ([DPSOHLVVXI&FLHQWWRFDUU\WKH 10.6.2 load. 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn
shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, there are additional shear requirements. []Vn shall be at least the lesser of: 18.4.3.1 For columns in IMFs, LOOXVWUDWHVKRZWR¿QGMn (National Institute of Standards and Technology (NIST), "Seismic Design of Reinforced Concrete Special Moment Frames: A Guide for Practicing Engineers," NIST GCR 8-917-1).  $\varphi Vc = 27$  kip  $\geq Vu = 25$  kip 2 The range of nominal moments is shown in the foloRZLQ]¿JXUH interaction diagram spreadsheet allows the user The interact In this example, the range of Mn does not contain input an axial load value and it will calculate the he maximum to inp the balance point of the curve. Thus, the QRPLQDOPRPHQWVHHWKH<sup>3</sup>6HOHFW\$[LDO/RDG<sup>'</sup>WDE value for Mn is at the load combination with the greatest Mn. For Pu = 848 kip, Mn = 9373 kip-in. For Pu = 456 kip, Mn = 9373 kip-in. kip, Mn = 7994 kip-in. Use, Mn = 9373 kip-in. Use, Mn = 9373 kip-in. Vu = 18.4.3.1(b) 25 Shear with the overstrength factor on applied. M nt + M nb 2 × 9373 = = 101 kips Au 186 on = 3 (Table 12.2-1, ASCE 7) Vu = 3 × 25 = 75 kip Controls American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS Shear reinforcement is required. The equation for shear reinforcement is NG Av f yt d From Example 2 Area of a No. 4 bar = 0.20 in 2 s Av = 4 legs × 0.20 = 0.80 in 2 d = 24 - 2.5 = 21.5 in. fyt = 60 ksi When shear is required, there is a limit on spacing of The maximum spacing is the lesser of d/2 or 24 in. d 21.5 for smax = = 10.75; use s = 10in. Vs =  $10.7.6.5.2 \text{ } \varphi Vc / 2 = 27 \text{ kip} < Vu = 75 \text{ kip} Vs \leq 4 \text{ f } c'bw d = 4 5000 \times 24 \times 21.5 = 145.9 \text{ kip} 1000 2 2 \text{ Calculate Vs Vs} = 0.80 \text{ in.} 20.75 \text{ } c' 50 \text{ bs fy } Vc / 2 = 27 \text{ kip} < Vu = 75 \text{ kip Vs} \leq 4 \text{ f } c'bw d = 4 \text{ legs} \times 0.25 = 0.80 \text{ in.} 20.75 \text{ } c' 50 \text{ ls fy}$  $5000 \times 244 \times 10 = 0.21$  in.2  $\leq 0.80$  in.2 OK 60, 000 50  $\times 24 \times 10 = 0.20$  in iin.2  $\leq 0.80$  in.2 OK 0.20 60, 60 000 bs fy Step 4—Find the required geometry from Example 2 is acceptable for Typical cal section along  $\varepsilon_0$ . all locations except in the plastic hinge region, Eo. Section 18.4.3.3 requires that hoops are used in the plastic hinge region, Eo. 1 25.7.4 18.4.3.3 Hoop geometry is similar to ties but the tie must be closed with seismic hooks at each end. Hoop is IXUWKHUGH¿QHGLQ&KDSWHURIWKH&RGH,WVWDWHV that the bend must not be less than a standard 135-degree hook. Another common tie arrangement is one exterior hoop with one cross tie for the middle bars in each direction. The plastic hinge Eo is the greatest of (a) 1/6×186 = 31 in. (b) 24 in. (c) 18 in. American Concrete Institute - Copyrighted © Material - www.concrete.org Controls. Columns 1 22.5.10.5.3 379 380 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 18.4.3.3 The maximum spacing, so, along the plastic hinge Find the maximum spacing for so region is the smallest of (a)  $8 \times 1 = 8$  in. Controls. (a) 8db, smallest longitudinal bar (b) 24db, hoop (b)  $24 \times 0.5 = 12$  in. (c) 0.5h or 0.5b (c)  $0.5 \times 24 = 12$  in. (d) 12 in. (d) 12 in. (d) 12 in. (d) 12 in. 18.4.3.5 2XWVLGHE0 the column spacing for so region is the smallest of (a)  $8 \times 1 = 8$  in. Controls. (a) 8db, smallest longitudinal bar (b) 24db, hoop (b)  $24 \times 0.5 = 12$  in. (c) 0.5h or 0.5b (c)  $0.5 \times 24 = 12$  in. (d) 12 in. (d) 12 in. 18.4.3.5 2XWVLGHE0 the column spacing for so region is the smallest of (a)  $8 \times 1 = 8$  in. Controls. (a) 8db, smallest longitudinal bar (b) 24db, hoop (b)  $24 \times 0.5 = 12$  in. (c) 0.5h or 0.5b (c)  $0.5 \times 24 = 12$  in. (d) 12 in. (d) 12 in. 18.4.3.5 2XWVLGHE0 the column spacing for so region is the smallest of (a)  $8 \times 1 = 8$  in. Controls. (a) 8db, smallest longitudinal bar (b) 24db, hoop (b)  $24 \times 0.5 = 12$  in. (c) 0.5h or 0.5b (c)  $0.5 \times 24 = 12$  in. (d) 12 in. (d) 12 in. 18.4.3.5 2XWVLGHE0 the column space in (a) (a) (b) (a) (b) (b) (b) (b) (b) (c) (b) (c) (transverse reinforcement is Thus, the transverse reinforcement is provided as shown in Example 2 but with a 10 in. No. 4 hoop at 8 in. along Eo and No. 4 ties at 10 in. outside of Eo spacing for shear. 6WHS<sup>2</sup>&KHFNWKHMRLQW 15.2.4 )URP([DPSOHWZRWLHVDUHUHTXLUHGDORQ]WKHGHSWKRIWKHMRLQWDWDVSDFLQ]QRWJUHDWHUWKDQLQ 18.4.4 refers the engineer to the Chapter 15 so there are no additional requirements according to the Code for an IMF. 18.4.2 The shear required for design is larger for the IMF. As discussed in Step 3 of Example 2, the provisions RI6HFWLRQFDQEHXVHGWRFKHFNWKHFRQFUHWHVKHDUVWUHQJWKWKURXJKWKHMRLQW1RWHWKDW6HFWLRQDVGH¿QHGLQ\$&,5\$Q,0)FRXOGEHGHVLJQHGDV a Type 1 connection in ACI 352R. 9DOXHVDGDSWHGIURP%HDP([DPSOHLQ&KDSWHURI Calculate column shear force Vu associated with the nominal moments and related shears calculated WKLV+DQGERRN5HLQIRUFHPHQWLVPRGL¿HGWRPHHW the beam requirements for IMFs. Use seven No. 7 2. The column in accordance with Section 18.4.2. EDUVDWWKH EDUVDWWKH7EHDPADQJHDQGWKUHH1REDUVDW ated with the following shear can be approximated he T-beam ste the stem. JLQAHFWL HTXDWLRQDVVXPLQJLQAHFWL RQSRLQWVRFFXUDWWKH ghts: column midheights: Mnll = 6520 0 kip-in. h] [ Mnrr = 2960 kip in. | (Mnl + Mnr) + (Vul + Vur) 2 ] | Vull = 77 kip Vu, col = [ Vurr = 30 kip A Also, Where E is the e distance d ce between the mid-height id-height of Also WKHFROXPQDERYHDQGEHORZWKHMRLQWLW 188 ft + 14 ft is unconservative to ignore the slab for this check; A =  $\times$  12 in./ft = 192 in. 2 therefore, consider the full effective width, bf, of h = 24 in. the T-Beam in the calculations. Vu , col 7KHVKHDUDWWKHFHQWHURIWKHMRLQWLV 24  $\left[ \left| \left[ (6520 + 2960) + (77 + 30) 2 \right] \right] = 100 \text{ J}$  $52 \text{ kip } 192 \text{ T1} = 3 \times 0.60 \times 60 = 108 \text{ kip } \text{T2} = 7 \times 0.60 \times 60 = 252 \text{ kip } \text{Vj} = T2 + C1 - \text{Vu C1} = T1 = 108 \text{ kip } \text{C2} = T2 = 252 \text{ kip } \text{Vj} = 252 + 108 - 52 = 308 \text{ kip } \text{American Concrete Institute} - Copyrighted © Material - www.concrete.org CHAPTER 9-COLUMNS 7KHHIIHFWLYHDUHDRIWKHMRLQWAj, is calculated by the multiplying the column$ depth, h, by the effective width which is the lesser of: bw,beam = 18 in. hcol = bcol = 24 in. x = Fig. R18.8.4 18.8.4.1 21.2.4.3 (a) bw,beam + hcolumn (b) bw,beam + 2x (c) bcolumn where x is the smaller distance between the edge of the column. &DOFXODWHWKHVKHDUVWUHQJWKRIWKHMRLQW7DEOH 18.8.4.1 21.2.4.3 (a) bw,beam + hcolumn (b) bw,beam + 2x (c) bcolumn where x is the smaller distance between the edge of the column. Vn for several conditions. This MRLQWLVFRQ¿QHGE\EHDPVRQWZRRSSRVLWHIDFHV Vn = 15 $^{\circ}$  f c'Aj 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (b) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in.
- 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 in. - 18 in. = 3 in. 2 (IIHFWLYHMRLQWZLGWK (a) 18 + 2 × 3 = 24 in. (c) 24 i Type 1 connection in ACI 352R a higher shear strength is permitted. Vn = 20 $\lambda$  f c'Aj 6WHS<sup>2</sup>'HWDLOWKHFROXPQVSOLFHDQGMRLQWDW/HYHO example is an extension of Example 2 but the seismic loads where increased for a Seismic Design Category C. The difference in design resulted in a 135-degree hooks. The column size and longitudinal reinforcement remained the same. American Concrete Institute - Copyrighted © Material - www.concrete.org Columns 18.8.4.3 381 382 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Column Example 4: Column for a special moment frame (SMF) 'HVLJQDQGGHWDLOWKH¿UVWÅRRULQWHULRUFROXPQDWORFDWLRQ(IURPWKHH[DPSOHEXLOGLQJ]LYHQLQ&KDSWHURIWKLV+DQGERRN The column is part of an SMF. ([DPSOHLVDFRQWLQXDWLRQRI([DPSOHVDQG7KHORDGVKDYHEHHQPRGL¿HGWRPDWFKWKH results of an analysis from commercial software capable of second order elastic analysis and for Seismic Design Category D. Given: Materials— 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL²HG\LHOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL?HOGVWUHQJWKfcg NVL Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL?HOGVWUHQJWKfcg NVL Modulus of elasticity of RDGFRPELQDWLRQV (i) U = 1.2D + 1.6L + 0.5S (ii) U = 1.2D + 1.0W + 1.0L + 0.5S (iii) U = 1.2D + 1.0E + 1.0L + 0.2S (iv) U = 0.9D + 1.0E + 1.0E + 1.0L + 0.2S (iv) U = 0.9D + 1.0E + 1.0L + 0.2S (iv) U = 0.9D + 1.0E + 1.0L + 0.2S (iv) U = 0.9D + 1.0E + 1.0L + 0.2S (iv) U = 0.9D + 1.0E + 1.0L + 0.2S (iv) U = 0.9D + 1.0E + 1. Calculation 6WHS<sup>2</sup>'LVFXVVLRQRQPRGL¿FDWLRQRI([DPSOHIRUDVSHFLDOPRPHQWIUDPH60) RQRI([DPSOHIRUDVSHFLDOPRPHQWIUDPH60) RQRI([DP (IMF) de moment design and are repeated here for and detailing requirements equ ents for a SMF increases creases the in rma information. UHVSRQVHPRGL¿FDWLRQFRHI¿FLHQW5IUP an IMF to 8 for an SMF. permitted F. A SMF is perm tted to be C umn Column: es aassigned ned to SDC B and C and h = b = 24 in used for structures required for SDC D, E, and F. For this example, the Eight No. 8 longitud longitudinal bars example building is analyzed for a region assigned No. 4 hoops WR6'&'DQG/RDG&RPELQDWLRQVLLL DQGLY 1 are revised. This example starts with the column designed in Example 3. Beam: h = 30 in. bf = 120 in. bw = 18 in. 6HYHQ1RORQJEDUVDWEHDPADQJH Three No. 7 long. bars at beam stem Step 2—Check dimensional limits and axial load 18.7.2.1 The column cross section shall be at least 12 in. b) b/h•ZKHUHh•b h LQ•LQOK b/h •OK American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9-COLUMNS 18.8.2.4 18.7.5.2(f) 7KHEHDPFROXPQMRLQWPXVWEHGHHSHQRXJKWR The largest longitudinal reinforcement in the beam must be 20 times the largest longitudi- 20db = 20 × 0.875 = 17.5 in. nal reinforcement in the beam. h •LQOK 7KHGHSWKRIWKHMRLQWVKDOOQRWEHOHVVWKDQRQHKDOI hbeam 30 WKHGHSWKRIDQ\EHDPIUDPLQJLQWRWKHMRLQW = = 16 in. 2 2 The arrangement of the longitudinal reinforcement is effected if hcolumn LQ•LQOK Pu = 872 kip Ag = 576 in.2 Pu > 0.3AgfcgRU fcg!SVL 872 kip >  $0.3 \times 576 \times 5 = 864$  kip 5000 psi 8 10,000 psi 8 10,000 psi The value hx shall not exceed 8 in. if this occurs; otherwise, it shall not exceed 8 in. hx for the cross section in Examples 2 and 3 is The area of steel in the current column is Ast =  $8 \times 0.79$  in. 2 = 6.31 in.  $2 \left[ 24 - (1.5 \times 2) - (0.5 \times 2) -
(0.5 \times 2$ 1.0] 9 5 in in in.; 8 in. hx = [ = 9.5 Try ry using 4 bars at each side using No. 7s 2 77.20 2 in.2 Thus, either the cross-section ss-sect can be increased or the Ast = 12 × 0.60 in.2 = 7.2 ould be rearranged to satisfy this reinforcement should hx = [24 - (1.5 × 2) - (0. (0.5 × 2) - 875]/3 requirement. LQ LQ"LQ er level and the aaxial ial lo This column is at a lower load 3Agfcg7KHXSS g7KHXSS g7KHXSS g7KHXSS g7KHXSS HUOHYHO HYHO is slightly greater th than 0.3A xc i It is therefore columns will not exceed this li limit. recommended to rearrange the reinforcement. The reinforcement for the upper levels can switch back o be more to the previous cross-section if it is found to economical. Step 3—Find the required area of longitudinal reinforcement l Using the Interaction Diagram spreadsheet referl enced at the start of this Example, the revised column section is analyzed for the four load combinations given for this example. 18.7.4.1 For SMFs, the maximum amount of longitudinal reinforcement, Ast, is reduced to 0.06Ag. The minimum amount of Ast stays the same at 0.01 Ag. U twe Use twelve No. 7 longitu longitudinal bars evenly spaced a und the perimeter. rimeter. around Design Capacity Interaction Diagram: Twelve No. 7 bars evenly spaced around the perimeter. rimeter. around Design Capacity Interaction Diagram: Twelve No. 7 bars evenly spaced around the perimeter. rimeter. around Design Capacity Interaction Diagram: Twelve No. 7 bars evenly spaced around the perimeter. Twelve No Institute - Copyrighted © Material - www.concrete.org OK Columns 18.8.2.3 383 384 18.7.3.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 7KHAH[XUDOVWUHQJWKRIFROXPQLQD60)PXVW satisfy: (6) S M nc > | | S M nb \5/ 18.6.3.2 The beam design is not part of this example. The EHDPXVHGIRUWKHRUGLQDU/PRPHQWIUDPH20) and IMF does not meet all the requirements for a 60)7KHEHDPLVPRGL¿HGKHUHDVQHFHVVDU/WR IXOO/GHPRQVWUDWHWKHFROXPQDQGMRLQWGHVLJQ Mn is the minimum nominal moment in the range of the interaction curve related to the minimum axial loads for the seismic load combinations. For more information on how to calculate Mn, refer to Step 2 in Example 3. The Interaction Dia Diagram Spreadsheet allowss the ad value and it will ll caluser to input an ax axial load na moment, ment, see the "S ect A culate the nominal "Select Axial /RDG WDE From Example 3, the nominal moments in the beam are: Mn = 6520 kip-in. Mnr = 2960 kip-in. Add one more No. 7 bars to the bottom of the beam so that the positive moment at least one-half the negative PRPHQWDWWKHMRLQWIDFH Mnl = 6520 kip-in. For Pu = 872 kip, p, Mn = 9 F Pu = 432 kip, p, Mn = 8 8380 kip-in. For U the lowest value, Mn = 8380 kip-in. Use 6 5  $(2 \times 8380) \ge (6520 + 3940) \cdot OK \cdot Step 4$ —Determine the geometry of the transverse reinforcement required for the entire columns are required for the entire columns or spirals for 18.7.5.1 circular columns are required for the entire columns are required for the entire column 18.7.5.2 height. The hoops at the plastic hinge,  $\mathcal{E}_0$ , and the splice must meet geometry requirements of Section 18.7.5.2 There are six conditions that geometry of the section must satisfy. The cross-section shown on the right meets these conditions. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9-COLUMNS 385 Note that hoops are closed ties with standard hooks at the end that are at least 135 degrees. Crossties a permitted to support the longitudinal bars between the corners. Where crossties are used they shall be alternated end for end along the longitudinal bar. For this example, every bar needed support. Since there are an even number of bars, overlapping hoops provide the least number of pieces that still provide the QHFHVVDU\VXSSRUWDQGFRQ¿QHPHQW 14 - hx (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4 + ( | (3) / (14 - 6.4) (c) 4in. (a) n (b) 6 × 0.875 = 5.25 in. Controls. t column height unless noted Use s = 5.25 in. along the erwi from m the follow otherwise following checks. m of transverse rein orcement require ti hinge hi Step 6—Check the minimum amount reinforcement required alon along the plastic length, & 2 18.7.5.4 ate than n 0.3AgfcgDVVKRZQLQ6WHS VKRZQLQ6 Assh = 0. 0.20 in. × 4 = 0.80 in.2 Since Pu is greater greatestt of: bc = 24 - (2 × 1.5) = 21 in. Ash/sobc shall be gre Ag = 24 × 24 = 576 in. in 2 2 2 Ach = 21 = 441 in. 5000 + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0; use 1.0. 25, 000 12 kn = = 1.2 12 - 2 kf = (A f (a) 0.3 | g - 1 | c (Ach / f yt (b) 0.09 f c' f yt (c) 0.2k f kn (576 ) 5 - 1 | = 0.0077 (a) 0.3 | (441 / 10.00) + 0.6 = 0.8 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 < 1.0 nal bars around the perimeter of the column core that are laterally supported by the corner of hoops or seismic hooks. American Concrete Institute – Copyrighted © Material -
$Ach 5 = 0.0075 60 (c) 0.2 \times 1.0 \times 1.2 872 = 0.0079 Controls 60 \times 441$  where kf = f c' + 0.6  $\ge$  1.0 and 25, 000 kn = nl nl - 2 nl is the number of lon www.concrete.org Columns Notice that Section 18.7.5.2(e) and (f) can impact the design of the column as it did in this example. Step 5—Determine the maximum spacing of the transverse reinforcement 18.7.4.3 The maximum spacing, so, along the plastic hinge 18.7.5.1 Maximum spacing for so length,  $\mathcal{E}_0$ , and splice regions is the smallest of: 18.7.5.3 (a) 0.25 × 24 = 6 in. (a) 0.25 hor 0.25b (b) 6 × 0.875 = 5.25 in. Controls (b) 6db, smallest longitudinal bar 386 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Find the required maximum spacing for this amount of reinforcement Ash  $\geq 0.0079$  or so bc so  $\leq$  Ash 0.80 = = 4.8 in. 0.0079bc 0.0079 × 21 Use s = 4.5 in. along & Step 7—Check the minimum amount of transverse reinforcement required for shear 18.7.6.1.1 Section 18.7.6.1.1 S .QWVDWHDFKHQGRIWKHFROXPQ7KHVHMRLQWIRUFHVVKDOOEHFDOFXODWHGXVLQJWKHPD[LPXP SUREDEOHÅH[XUDOVWUHQJWKVMpr, at each end of the column associated with the range of factored axial forces, ROXPO7KHFROXPOVKHDUVOHHGORWH[FHHGWKRVHFDOFXODWHGIURPMRLQWVWUHQ]WKVEDVHG on MprRIWKHEHDPVIUDPLQ]LQWRWKHMRLQW,QQRFDVHVKDOOVe be less than the factored shear calculated by analysis of the structure." 7KH¿UVWSDUWRI6HFWLRQLVYHU\VLPLODU Using the Interaction generate to how Mn is calculated in Step 3 above, except that the curve for fy = 75 ksi. on diagram 1.25fy is used to generate the interaction ards and Technology (National Institute of Standards gn of Rei (NIST), "Seismic Design Reinforced Concrete ames: A Guide for Practicing Special Moment Frames: 7\*& (QJLQHHUV) 1,67\*&5 7KLVPRGL¿HG d th bable moment is called the probable moment, Mpr. 8VHWKH<sup>3</sup>6HOH 8VHWKH<sup>3</sup>6HOH FW\$[LDO/RDG VKHHWLQWKHVSUHDGVKHHW WR ¿Q WR¿QGM pr. For Pu = 872 kip, Mpr = 10,284 kip-in. Use Mpr = 10,284 kip-in. Use Mpr = 10,284 kip-in. Ve = M prt + M prb Au = 2 × 10, 284 = 111 kip 186 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 9-COLUMNS The second part of Section 18.7.6.1.1 is similar to KRZWKHFROXPQVKHDUZDVFDOFXODWHGIRUWKHMRLQW in Step 3 of Example 3. Calculate column shear force Ve associated with the probable moments and related shears of the beam. The column shear can be approximated with the following equation DVVXPLQJLQÅHFWLRQSRLQWVRFFXUDWWKHFROXPQPLGheights: 387 The probable Mpr1 = 8010 kip-in. Mpr2 = 4920 kip-in. Ve1 = 85 kip Ve2 = -22 kip Also, 18 ft + 14 ft × 12 in./ft = 192 in. 2 h = 24 in. A = () Ve, col 24  $\left[ \left[ \left( 8010 + 4920 \right) + (85 - 22) 2 \right] \right] = 71$  kip 192 where  $\mathcal{E}$ is the distance between the mid-height of WKHFROXPQDERYHDQGEHORZWKHMRLQW1RWHWKDWLW is unconservative to ignore the slab for this check; therefore, consider the full effective width, bf, of the T-beam in the calculations. The last part of Section 18.7.6.1.1 8.7 the analysis. not be less than Vu from Ve = 34 kip l 22.5.10.5.3 The requirement permits the use of the shear pe ar calUse Ve = 111 kip culated for the sec second part of Section 118.7.6.1.1, 7.6.1, 7.6 for shear Vc = 73 kip [] Area of No. 4 bar = 0.20 in.2 Av = 4 legs × 0.20 = 0.80 in.2 d = 24 - 2.44 = 21.56 in. 22.5.1.1 The equation for shear reinforcement is Vs = Av f yt d s = 5.25 in. from Step 5 Vs = s [] Vn [] Vs + Vc • Ve 0.80 × 60 × 21.56 = 197 kips 5.25 [] Vn NLS•Ve = 111 kip Step 8—Summarize the amount and spacing of transverse the amount and spacing of transverse the amount and spacing of transverse the amount and space to the space to
the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the space to the s reinforcements along the height of the column 18.7.5.1 Find Eo The plastic hinge length, Eo, is the greatest of: (a) 24 in. (c) 18 i American Concrete Institute – Copyrighted © Material – www.concrete.org Columns Ve,col h] [ | M pr1 + M pr 2 + (Ve1 + Ve 2) 2 |] = A 388 18.7.4.3 | 25.5.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) /DSVSOLFHVVKDOOEHSODFHGZLWKLQWKHFHQWHUKDOI of the column. The hoop spacing along the splice length must meet Sections 18.7.5.2 and 18.7.5.3 but not 18.7.5.4. In the calculation of hoop spacing for  $\mathcal{E}$  or regions, 18.7.5.4 controlled but the difference is so small in this case that the same spacing used. Est is the greater of: (a) 1.3Ed (b) 12 in. 10.7.5.2.2 where, Ad = and K tr = ø t ø eø s 3 fy 40  $\lambda$  f' (cb + K tr c |  $\langle db \rangle$  | / db 40 Atr sn Along the splice, use No. 4 hoops at 4.5 in. Determine Est for a No. 7 bar: zt = ze = zs 3 db = 0.875 in. cb = 1.5 + 0.5 + 0.875/2 = 2.44 in. n = 4, number of bars along the splitting plane s = 4.5 in., spacing of ties Atr = 4 tie legs × 0.2 in.2 = 0.8 in.2 Ktr =  $(cb + Ktr)(1.94 + 1.78)||=|| = 4.25 \le 2.5 (db)(0.875)/Ad = 1$  For fy equal to 60 0 kksi, Esc iss the greater of the splitting plane s = 4.5 in., spacing of ties Atr = 4 tie legs × 0.2 in.2 = 0.8 in.2 Ktr =  $(cb + Ktr)(1.94 + 1.78)||=|| = 4.25 \le 2.5 (db)/(0.875)/Ad = 1$  For fy equal to 60 0 kksi, Esc iss the greater of the splitting plane s = 4.5 in., spacing of ties Atr = 4 tie legs × 0.2 in.2 = 0.8 in.2 Ktr =  $(cb + Ktr)(1.94 + 1.78)||=|| = 4.25 \le 2.5 (db)/(0.875)/Ad = 1$  For fy equal to 60 0 kksi, Esc iss the greater of the splitting plane s = 4.5 in. mid mid-height The end result is hoop spacing of 4.5 in. at & regions and along the splices are permitted. Type 1 mechanical splices are permitted. Type 1 mechanical splices are not permitted within a distance equal to twice the column depth from the bottom of beam or top of slab. Type 2 mechanical splices do not have a location restriction for cast-in-place SMFs. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 9-COLUMNS 6WHS<sup>2</sup>&KHFNWKHMRLQW 18.8 7KHFROXPQVKHDUVWUHVVHVDWWKHMRLQWLVFDOFXODWHG 18.8.2.1 in Step 7. The column shear is calculated from the SUREDEOHPRPHQWLQWKHEHDPVZKHUHWKHAH[XUDO tensile reinforcement is 1.25fy. 7KHVKHDUDWWKHFHQWHURIWKHMRLQWLV Vj = T2 + C1 - Vu 389 Ve, col = 71 kip T1 = 4 × 0.6 × 60 × 1.25 = 180 kip T2 = 7 × 0.875 × 60 × 1.25 = 315 kip 18.8.4.3 7KHHIHFWLYHDUHDRIWKHMRLQWAj, is calculated by the multiplying the column depth h by the effective width which is the lesser of: Fig. R18.8.4 (a) bw, beam + hcolumn (b) bw, beam + 2x (c) bcolumn where x is the smaller maller distance between the edge of dge of the column. the beam and edge bw, beam = 18 in. hcol = bcol = 24 in. x =MRLQWLVFRQ¿QHGE\EHDPVRQWZRRSSRVLWHIDFHV Vn = 15 × 1.0 × 5000 × 576 = 611 kips 1000 [V Vn î NLS•NLS Joint is OK. Step 10—Discussion and summary This examples 2 and 3 but the seismic loads where increased for a Seismic Design Category D. The difference in design resulted in a rearrangement of the longitudinal bars and a differHQWKRRSDUUDQJHPHQW7KHDPRXQWRIWUDQVYHUVHUHLQIRUFHPHQWLVDOPRVWWLPHVWKDWRIDQ,0)7KHFROXPQVL]HZDVVXI&FLHQWIRUDOOWKUHHW\SHVRIPRPHQWUHVLVWLQJIUDPHV American Concrete Institute - Copyrighted © Material - www.concrete.org Columns C1 = T1 = 180 kip C2 = T2 = 315 kip Vj = 315 + 180 - 71 = 424 kip 390 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete.org 10.1—Introduction The scope of ACI 318-14, Chapter 11 addresses nonprestressed and prestressed cast-in-place and precast reinforced concrete walls. In addition to Chapter 11, Section 18.10 of ACI 318-14 provides design and detailing requirements for special cast-in-place walls forming part of a building's lateral-forceUHVLVWLQJV\VWHP/)56 GXHWRWKHLUKLJKLQSODQHVWLIIQHVV Although structural walls are also part of the gravity-forceresisting system, they are often lightly axially loaded. They are often lightly axially loaded. They are often lightly axially loaded. they resist a large portion of the total lateral forces acting on the structure through in-plane shear. In ACI 318, all walls are referred to as structural walls. equirements QSUHVWUHV LGHQWLI/WZRFDWHJRULHVRI/)56IRUQRQSUHVWUHVVHGFDVWLQ place walls: 1. An ordinary cast-inplace stru structural wall, permitted DC A, B, and C, which whi is in seismic design categories (SDCs) ian with th Chapter 11 of ACI designed and det iled 2. A special structural wall, which detailed 1 and 18 of ACI 318-14. 31 4. in compliance with Chapters 11 dF Special walls are required in SDCs D, E, and F, but can be constructed in all seismic categories. Seismic requirements are intended to increase wall e strength and ductility to accommodate the large displacements demands expected during a maximum design earthquake. Walls are laterally connected to diaphragms and vertically to foundations or support elements. In seismic and nonseismic design, the connections to diaphragms are designed to remain elastic, and the energy from the lateral forces is absorbed by the structural wall. In seismic design, connection to the foundation can be the point of maximum wall moment and yielding of vertical reinforcement is expected. 10.2—General 10.2.1 Distinguishing a column from a wall—The design of a wall can be so similar to a column that the question of when a rectangular column becomes a wall is often delibHUDWHG )RU VSHFLDO PRPHQW IUDPHV FROXPQV DUH GH¿QHG as having a minimum aspect ratio of 0.4 in 18.7.2.1(b) of ACI 318-14. Although this limit is necessary to achieve the expected behavior, it might not be the best limit for the consideration of column or wall design. Expected behavior is similar to beams. Walls usually have high axial loads and their shear behavior is similar to one-to make the shear behavior is similar to one-to make the shear behavior is similar to be a similar to be a similar to be a similar to one-to make the similar to be a
similar to be a way slabs for out-ofplane loads, with unique shear behavior for in-plane loads. For further discussion and information regarding unique shear behavior for in-plane loads in walls, refer to Moehle (2015). 10.2.1.1 Longitudinal reinforcement—In general, longitudinal reinforcement—In general, longitudinal bars require lateral support to prevent buckling of the bars due to axial compression. In a wall, if longitudinal reinforcement is required for axial strength, or if Ast exceeds 0.01Ag, then Section 11.7.4.1 of ACI 318-14 states the longitudinal reinforcement must be supported by transverse ties. This requirement could be used as a practical limit to determine whether the member should be designed as a column or a wall. If the wall requires heavy reinforcement (exceeding 0.01Ag), a tie bar would be required at every intersection of longitudinal and transverse reinforcement, which would VLJQL¿FDQWO\ DIIHFW WKH UHTXLUHG DPRXQW RI FRQVWUXFWLRQ labor. Designing this same member as a column could be more ppractical. 10.2.1.2 Shea Shear aspect ratios—The next limit to consider ear. Most walls have a length-to-thickness ratio of at is shear. hese aspect ratios, ati it is easy to see how the shear atio least 6. For these ffer from a column for either in-plane or outbehaviorr wi will differ of-p or smaller as a wall or column, ng oon the shear hear forc depending force applied and the direction of hear, except xce as limited mited by 18.7.2.1(b) of the Code. For shear, aspec atio under er 2.5, the member is likely to be designed aspect ratios as a column. Further ddiscussion on this topic is given in 03). Carcia (2003). 2.2 Wall lay 10.2.2 layout—Shear walls should be located within DDEXLOGLQJSODQWRHI¿FLHQWO\UHVLVWODWHUDOORDGLQJ/RFDWLQJ EXLOGLQ shear walls in the center half of each building is generally a good location for resisting lateral forces (Fig. 10.2.2(a)). This arrangement, however, can restrict architectural use of space. Although shear walls are commonly located at the ends of a building, such wall locations will increase slab restraint and shrinkage stresses, especially in long buildings such as parking structures that are exposed to large temperature changes (Fig. 10.2.2(b)). Symmetrical wall DUUDQJHPHQWVSURYLGHJRRGAH[XUDODQGWRUVLRQDOVWLIIQHVV Walls at the perimeter resist torsional forces most effectively. Walls away from the perimeter, however, could have a higher tributary area and, consequently, larger gravity axial force to resist uplift or overturning. They are, however, less HI¿FLHQWLQUHVLVWLQJKRUL]RQWDOWRUVLRQ An unsymmetrical arrangement, however, does not usually provide predictable torsional stiffness due to their eccentricity (Fig. 10.2.2(c)). Such a shear wall layout should be designed explicitly for torsion. A symmetrical arrangement is preferable to avoid designing walls for torsion. 10.2.3 :DOO FRQ¿JXUDWLRQV—Shear walls could have VHYHUDO JHRPHWULF FRQ¿JXUDWLRQV LQFOXGLQJ SODQH ADQJHG or channel sections. A plane wa is rectangular with American Concrete Institute - Copyrighted © Material - www.concrete.org Walls CHAPTER 10—STRUCTURAL REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 10.2.2—Shear wall layouts. Fig. 10.2.3a—Shear wall cross sections. se s. )LJE<sup>2</sup>'H¿QLQJVKHDUZDOOKHLJKWOHQJWKDQGGHSWK or without enlarged ends (boundary elements). Flanged VKHDU ZDOOV DUH RIWHQ 7 RU /VKDSHG VHFWLRQV 5HLQIRUFHG concrete walls around building elevator core shafts and stairwells are typically in a C- or U-shape (channel shear wall) (Fig. 10.2.3a). Notation uused to describe the wall dimensions are shown in Fig. 10.2.3b; whe where h is the wall thickness, hw is the wall thickness, hw is the wall type. The bas n se based on several factors, in including functionality, constructabilit tability, eco economy, and seism seismic performance (Moehle et al 011) For low-rise buildings building (Fig. 10.2.4(a)), squat, solid 2011). predominantly used (hw/Ew < 2). As the building walls are re pr nantly use increases (hw/ N OO E Ew • PDNLQJZDOOVEHFRPHPRUHVOHQGHU)LJF Perforated walls (Fi (Fig. 10.2.4(b)) are acceptable, but ding on a wall's w depending opening percentage, the wall strength r could be reduced. A row of vertically aligned openings in a slender wall results in dividual continuous wall sections, termed "coupled walls" because they behave as two individual continuous wall sections connected by coupling beams (Fig. 10.2.4(d)). 10.2.5 Design limits—Minimum wall thicknesses are as shown in Table 10.2.5. For walls designed by the alternative method for out-ofplane loads in Section 11.8 of ACI 318-14. ACI 551.2R provides the following slenderness limits: D 2QHOD\HURIUHLQIRUFHPHQWDWWKHFHQWHURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIUHLQIRUFHPHQWDWWKHFHQWHURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIUHLQIRUFHPHQWDWWKHFHQWHURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 does the following slenderness limits: D 2QHOD\HURIWKHZDOO Ec/h" (b) Two layers of reinforcement, one at each face of the wall, Ec/h" ACI 318 not provide separate thickness limits for structural walls resisting in-plane lateral forces. The NEHRP Technical Brief No. 6 (Moehle et al. 2011) suggests the following minimum wall thickness: (a) Special structural walls: 8 in. (b) Special structural walls: 8 in. (b) Special structural walls resisting in-plane lateral forces. The NEHRP Technical Brief No. 6 (Moehle et al. 2011) suggests the following minimum wall thickness: (a) Special structural walls: 8 in. (b) Special structural walls: 8 in. (c) Coupled shear walls— American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 10-STRUCTURAL REINFORCED CONCRETE WALLS 393 Fig. 10.2.4-Shear wall types. Table 11.3.1.1) Minimum thickness h Bearing\* Greater of: Exterior basement and foundation\* 4 in. (a) 1/25 the lesser of unsupported length and unsupported height (b) 4 in. (c) unsupported 1/30 the lesser of unsupported neight (d) 7.5 in. (e) \* 2QO\DSSOLHVWRZDOOVGHVLJQHGLQDFFRUGDQFHZLWKWKHVLPSOLCHGG PHWKRG GR i. Coupling beam designed d aas a special
special special special special special special special special special special special special special spe pecial moment bbeam: am: 14 in. d w ceii. Coupling beam designed with diagonal reinf reinforcement: 16 in. There is no code limit on the overall building drift. \*HQHUDOO\WKHUHODWHUDOGHAHFWLRQLQDQ\RQHVWRU\ should not exceed 1/500 of the story height. 10.3—Required strength Chapter 5 in ACI 318-14 provides the load combinations necessary to design a shear wall for moment, shear, and axial force. Section 12.4 in ASCE 7 has additional seismic load combinations to consider and load effects, such as the overVWUHQJWKIDFWRUo. 10.3.1 Methods of analysis—ASCE 7 allows for three different types of analysis for determining the lateral seismic forces: (TXLYDOHQW/DWHUDO)RUFH\$QDO\VLV(/) 7KHHTXLYDlent lateral force analysis is the simplest method of analysis DQG LV VXI¿FLHQW IRU WKH PDMRULW\ RI VWUXFWXUHV EXLOW XVLQJ an approximated fundamental period Ta, which can be conservative. For long periods greater than 3.5 seconds, this method is not acceptable. 2. Moda Response Spectrum Analysis (MRS): The modal response spectrum analysis accounts for the elastic dynamic behavior of the structure and determines the building period. The calculated base shear FDOFXODWHGXVLQJWKH(/)PHWKRG7KHEDVHVKHDUKRZHYHU VKRXOG EH VFDOHG WR D PLQLPXP RI SHUFHQW RI WKH (/) base shear. 33. S smi Response nse History Analysis (SRH): The seismic espon e hi response history method of a three-dimensional model is used to anal analyze thee building. E 7-10 7 Table bl 12 2 provides the required response ASCE 12.2-1 PRGL¿FDWLRQ FRHI¿FLHQW R DQG GHAHFWLRQ DPSOL¿FDWLRQ UHTXLUHGLQWKHDQDO\VLV7KHUHOHYDQWFRHI¿FLHQWV factors CdUHTXLUH are listed for shear walls in Tables 10.3.1a through 10.3.1c. 6HFWLRQVDQGLQ\$&,DGGUHVV¿UVWRUGHU elastic analysis, second-order elastic analysis, and second-order inelastic analysis, respectively. Section 18.2.2.1 in ACI 31 14 requires that the interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions be considered in the analysis. )RU ADQJHG ZDOOV WKH HIIHFWLYH ADQJH ZLGWK RI D ZDOO varies depending on the anticipated deformation level and whether it is in tension or compression. Tests (Wallace 1996) KDYH VKRZQ WKDW WKH HIIHFWLYH ADQJH ZLGWK LQFUHDVHV ZLWK LQFUHDVLQJ GULIW OHYHO DQG WKH HIIHFWLYH ADQJH ZLGWK KDV little ADQJH ZLGWK KDV little ADQJH ZLGWK KDV little ADQJH ZLGWK KDV little ADQJH ZLGWK KDV IIIHFWLYH ADQJH ZLGWK KDV little ADQJH ZLGWK KDV IIIHFWLYH ADQJH ZLGWK KDV little ADQJH ZLGWK KDV IIIHFWLYH ADQJH K effect on the strength and deformation capacity of the wall: therefore, to simplify design, a single value of effective ADOIHZLGWKLVXVHGLOERWKWHOVLRO 6HFWLRO LO \$&. GH¿OHV WKH HIIHFWLYH ADOIHZLGWK WKDW H[WHOGV IUR] WKH IDFH RI WKH ZHE RI / 7 & RU RWKHU ADQIHG VHFWLRQV DV WKH OHVVHU RI RQHKDOI WKHGLVWDQFHWRDQDGMDFHQWZDOOZHEDQGSHUFHQWRIWKH WRWDO ZDOO KHLJKW7KH IXOO ADQIH ZLGWK DQG QRW WKH HIIHF- American Concrete Institute – Copyrighted © Material – www.concrete.org Walls Wall type 394 THE CED CONCRETE DESIGN HANDBOOK—SP-17(14) Table 10.3.1a—Design coefficients for shear walls in bearing wall systems Seismic-forceresisting system 5HVSRQVHPRGL¿FDWLRQ FRHI¿FLHQWR factor Cd Special reinforced concrete shear walls 5 5 2UGLQDU\UHLQforced concrete shear walls 4 Intermediate precast shear walls 4 2 UGLQDU/SUHFDVW walls 3 3 Table 10.3.1b—Design coefficients for shear walls in building frame systems Seismic-forceresisting system 5HVSRQVHPRGL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ FRHI¿FLHQWR factor Cd Special reinforced concrete shear walls 6 5 2 UGLQDU/UHLQforced concrete shear walls in building frame systems Seismic-forceresisting system 5HVSRQVHPRGL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL¿FDWLRQ 'HAHFWLRQDPSOL 'A state s 5 4.5 Intermediate precast shear walls 5 4.5 2UGLQDU/SUHFDVW walls 4 4 Table 10.3.1c—Design coefficients for shear cie walls in dual systems with special moment frames capable of resisting at least 25 percent of prescribed seismic forces Seismic-forceresisting system 5HVSRQVHPRGL¿FDWLRQ FRHI¿FLHQWR factor Cd Special reinforced concrete shear walls 7 5.5 2UGLQDU/UHLQforced concrete shear walls 6 5 WLYHADQJHZLGWKPD/EHXVHGLQGHWHUPLQLQJWKHWULEXWDU/ gravity loads that resist uplift (ASCE 7). 10.4—Design strength Walls are a versatile building element used in a variety of ways that determine how the design. A wall is typically very long in one plan dimension, compared to the orthogonal dimension making the smaller wall slender. This slenderness can control the design if there are large loads applied laterally along the smaller wall dimension h/RDGV applied in this direction are commonly called out-of-plane ORDGV/RDGVDSSOLHGODWHUDOO\DORQJWKHODUJHUZDOOGLPHQsion Ew are commonly called in-plane loads. Rarely do out-of-plane and in-plane loads are always present. The designer typically designs a wall for the two conditions discussed for axial load with out-of-plane and those with in-plane loads (10.2.1). 10.4.1 Design for axial load—Wall design for axial load is similar to column design. The wall slenderness is considHUHGE\XVLQJWKHPRPHQWPDJQL¿FDWLRQPHWKRGLQ6HFWLRQ RI\$&, IRU D ¿UVWRUGHU DQDO\VLV RU E\ XVLQJ a computer program that accounts for P" HIIHFWV XVLQJ D second-order analysis. The design is completed according to Section 22.4 of ACI 318-14, which can be quickly evaluated using an interaction diagram generated by software or an electronic spreadsheet. If the resultant of all axial loads is located in the middle third of the wall thickness h, then DVLPSOL¿HGHTXDWLRQLVSHUPLWWHGLQRI\$&, where moment can be ignored and Pn is directly calculated. The wall section considered effective to resist a concentrated gravity load is the width of the bearing plus four times the wall thickness. The effective width cannot cross a YHUWLFDOZDOOMRLQWXQOHVVWKHMRLQWLVGHVLJQHGWRWUDQVIHUWKH load (ACI 318-14, Section 11.2.3.1). 10.4.2 Axial and out of-plane loads—Walls should be analyzed for combined axial and out-of-plane loads. For walls that are part of a multistory building lateral load em, combined axial and out-of-plane loads. For walls that are part of a multistory building lateral load em, combined axial and out-of-plane loads. checked ed using S slenderness Section 6.6.4 of ACI 318-14 for D¿UVW UGHU LVRUE\XVLQJDFRPSXWHUSURJUDPWKDW IHFWVXVLQ accountss for P"HIIHFWVXVLQJDVHFRQGRUGHUDQDO\VLV For a tall one-story ry buildin building with long shear walls, such as a warehouse, war ouse combined bined axia axial and out-of-plane loads typically h walll ddesign. i T control the There are many one-story commergs that use exterior concrete walls to support roof cial buildings IURPWKHDGM ORDGVIURPWKHDGMDFHQWLQWHULRUED rack storage or second-story mezzanines. The wall thickness is usually made as slender as possible for economy. The design of these walls is typically completed
in accordance with Section 11.8 of ACI 318-14. Alternative methods for out-of-plane slender wall analysis include ACI 318-14, or by using a Finite Element Analysis (FEA) that accounts for P"HIIHFWV The alternative method has several limitations as stated LQ 6HFWLRQ RI \$&, /LPLWV WKDW JHQHUDOO\ control this method, including its derivation use, and worked examples, is given in ACI 551.2R. 10.4.3 Axial and in-plane loads, squat walls—Walls are W\SLFDOO\SDUWRIWKH/)56GXHWRWKHLUODUJHLQSODQHVWLIIness. In squat walls (hw/Ew < 2), the predominant wall failure mode is diagonal shear. Shear applied to the top of the wall is delivered to the base through compressive struts. Diagonal cracks form along the struts at inclined angles of approximately 38 degrees (Barda et al. 1977), as shown in Fig. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 395 10.4.3. The vertical (longitudinal) reinforcement is mostly effective in resisting this type of shear through shear friction. Separation at the crack engages the vertical reinforcement in tension, creating a clamping force and increased resistance to shear. The vertical reinforcement in tension, creating a clamping force and increased resistance to shear. reinforcement begins to provide a portion of the resistance. At a height-to-length ratios exceeds 2.5, the horizontal reinforcement provides most of the shear strength and the shear strength and the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength and the shear strength and the shear strength and the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength and the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides most of the shear strength ratio exceeds 2.5, the horizontal reinforcement provides at ZKHUHd tVKDOOEHDWOHDVWdt is calculated according to Section 11.5.4.8 of ACI 318-14. If this value is less than a reinforcement ratio of 0.0025, then the minimum reinforcement ratio of 0.0025, then the minimum reinforcement ratio of 0.0025, then the minimum reinforcement ratio of 0.0025. If the required shear reinforcement exceeds 0.0025, en Eq. (11.6.2) of ACI 318-14. assures that enough longitudinal ded ffor shear-friction resis(vertical) reinforcement is provided " d.E computed from m tance in squatter walls. At hw/EEw " qui by y 11.5.4.8. How However, ver, (T FRXOGH[FHHGdt required KDW EH[FHHGdt required KDW EH[FHHGdt required KDW CH]] hw/Ew ! DQG" cement that cha es minimum amount of longitudinal re reinforcement changes / w off 22.5. At OLQHDUO\IURPdt required at an hw/Ew > 2.5, the transverse (horizontal) reinforcement is fully HQJDJHGDQGDPLQLPXPDdE of 0.0025 is provided. WK )RURUGLQDU\VWUXFWXUDOZDOOVWKHD[LDODQGAH[XUDOVWUHQ]WK is calculated according to Section 22.4 of ACI 318-14, as discussed in 10.4.2 of this Handbook. Concrete shear strength Vc is calculated according to Section 11.5.4.8 (ACI 318-14), and shear reinforcement strength Vc is calculated according to Section 11.5.4.8 (ACI 318-14). The maximum nominal shear strength Vn is, according to Section 11.5.4.3 Vn ≤ 10 f c'hd )RU VSHFLDO VWUXFWXUDO ZDOOV WKH D[LDO DQG ÀH[XUDO strength may also be calculated according to Section 12.4 of ACI 318-14, which is discussed in 10.4.2 of this Handbook. Section 18.10 in ACI 318-14 implies that a wall is squat when the height to-length ratio is less than 2.0. For walls with hw/Ew < 2.0, the displacement method of Section 18.10.6.2 (ACI 318-14) to determine if boundary elements are necessary. If boundary elements are not necessary, the design of a special structural wall is similar to an ordinary structural wall. A more detailed discussion about boundary elements is given in 10.4.4 of this Handbook. Key differences in squat wall shear design for special structural walls are Fig. 10.4.1—Effective wall horizontal length. Walls (h)  $\rho A \ge 0.0025 + 0.5$  ( $\rho t = 0.0025$ ) (11.6.2) Aw / Fig. 10.4.3— Shear 4.3— in squat w walls. calculated over Acv instead of hd. Acv is (a) Shear hear stress is calcul ngth of th typically the length the wall, Ew, multiplied by the width of eb, h (ACI 318-14, Section 18.10.4.1). the wall web, b) Minimum reinforcement is approximately the same: (b) i F i. For ordinary structural walls, the minimum reinforcement is according to Table 11.6.1 (ACI 318-14) for Vu OHVV WKDQ [Vc RU [hd3 fc' (ACI 318-14, Section 11.6.1). ii. For special structural walls, the minimum reinforcement is according to Table 11.6.1 for Vu less than Acv3 fc' (ACI 318-14, Section 18.10.2.1). 2WKHUZLVHWKHPLQLPXPUHLQIRUFHPHQWUDWLRIRUdtDQGd& is 0.0025 (ACI 318-14, Section 18.10.2.1). Section 18.10.2.1). (c) For hw/Ew dE VKDOO EH DW OHDVW dt (ACI 318-14, Section 18.10.4.3). (d) For Vu greater than 2 Acv3 fc or hw/Ew • WZR curtains of reinforcement are required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQIRUFHPHQWPXVWH[WHQGEw beyond the point where it is no longer required (ACI 318-14, Section 18.10.2.2). H /RQJLWXGLQDOUHLQI 18.10.2.3(a)). (f) At locations where yielding of longitudinal reinforcement is expected, development lengths shall be 1.25fy in tension (ACI 318-14, Section 18.10.2.3(b)). 10.4.4 Axial and in-plane loads, slender walls—The term "slender walls" is used if the predominant failure mode is ÀH[XUH%RXQGDU\HOHPHQWVDUHRIWHQUHTXLUHGZKLFKRIIHU LQFUHDVHG ÀH[XUDO VWUHQ]WK HQKDQFHG FXUYDWXUH FDSDFLW\ American Concrete.org 396 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 10.4.4—Calculation of neutral axis depth c (Moehle ehle et al. 2011). DQG EHWWHU GLVWULEXWLRQ RI ÀH[XUDO O FUDF FUDFNV WKDW SURPRWH increased displacement capacity (Mo (Moehle 2015). For walls ACI 318-14), 4), the design design for designed only by Chapter 11 (ACI D[LDO DQG ÀH[XUDO VWUHQ]WK LV DFF DFFRUGLQ] RI J WR 6HFWLRQ ACI 318-14, as discussed in 10.4.2 off this Handbook Handbook. For 0. special structural walls, Chapterr 18 of ACI 318-14 requires req ires special boundary elements: the displacement method of Section 18.10.6.2 and the stress-based method of Section 18.10.6.3 (ACI 318-14). The displacement method is the preferred method of design according to the NEHRP Technical Brief No. 6 (Moehle et al. 2011). This method assumes that: (a) The longitudinal bars at ends of the wall will yield at a single critical section (b) The wall has a constant cross section throughout its height (c) hw/Ew• G ju/hw• A special boundary element is required if the neutral axis depth c calculated exceeds Eq. (18.10.6.2) in ACI 318-14. c ≥ Aw 600 (1.5δ u /hw) (18.10.6.2) This equation was derived from the relationship shown in Fig. 10.4.4 of this Handbook (Moehle et al. 2011). The 1.5 PXOWLSOLHUWRiu was added in ACI 318-14 to more accurately
HYDOXDWHWKHGHÀHFWLRQRIWKHZDOODWWKHPD[LPXPFRQVLGHUHGHDUWKTXDNH7KHQRWDWLRQiu is the design displacement found through a linear elastic analysis and the section should include the effect of the corresponding axial load for the JLYHQODWHUDOVHLVPLFIRUFH7KHOLPLWRIju/hw•ZDV PRGL¿HGLQ\$&,7KLVORZHUOLPLWSURYLGHVLQFUHDVHG deformation capacity for a range of stiffer walls that did not previously require boundary elements. This method also has an additional requirement for the transverse reinforcement at special boundary elements. vertically above and below the critical section at least the greater of Ew and Mu/4Vu, except at the wall base as noted in 18.10.6.4(g) in ACI 318-14. At the foundation 12 in., or if the edge of the boundary element is within one-half the foundation depth from an edge of the footing, ties or hoops must extend into the foundation or support a distance equal to the development length of the largest vertical bar in the boundary element must be adequately developed in the foundation. The stress-based method is used for irregular walls or walls with disturbed regions, for example, around openings, according the NEHRP Technical Brief No. 6 (Moehle et al. 2011). The method requires special boundary elements if the effective compressive stress is less than 0.15 fcq7KH sed on a line arr eelastic analysis. model iss ba based linear Bo dary elements.—Special eleme 10.4.55 Boundary boundary elements are ¿ LQ6 GH¿QHGLQ6HFWLRQ\$&, :KHQWKHOLPLWV n 18.10.6.2 18. 0.6.2 or 18.10.6.3 at in are met, boundary elements are VWLOOU XLUH DUHGH¿QH VWLOOUHTXLUHGDQGDUHGH¿QH GDFFRUGLQJWR6HFWLRQ (A 318-14). 8 1 Iff a speci i boundary element is not required, (ACI special einforceme is required if the longitudinal reinboundary reinforcement PHQW UDWLR DW WKH ZDOO ERXQGDU\ d. H[FHHGV fy. IRUFHPHQW Boundary reinforcement is also required for the stress-based method where the effective compressive stress is between 0.15fco DQG fco DFFRUGLQ] WR 7KH UHTXLUHPHQWV for size and detailing of these requirements are described in Fig. 10.4.5 of this Handbook. Horizontal reinforcement in structural walls with boundary elements must be anchored into the core of the boundary element with hooks, headed bars, or straight embedment and also has to extend to within 6 in. of the end of wall Section 18.10.6.4. 10.4.6 Vertical wall segments and wall piers—A vertical wall segment is any portion of a wall that is bound by the outer edge of a wall and an edge of an opening, or the portion of a wall bound by the vertical edges of two openings. According to Chapter 11 of ACI 318-14, the design of nonseismic vertical walls because as that of walls. For special structural walls designed according to Chapter 18 of ACI 318-14, there are additional requirements. The nominal shear strength is reduced for the total cross section of a wall at the vertical wall segments. The calculated Vn may not exceed 8Acv fc' for the total Acv, as shown in Fig. 10.4.6 of this Handbook. For an individual vertical wall segment, Vn may not exceed 10Acv fc' . 9HUWLFDOZDOOVHJPHOWVDUHGHVLJOHGDVZDOOVFROXPOVRU wall piers according to the segment geometry as summarized American Concrete Institute - Copyrighted © Material - www.concrete.org 397 Walls CHAPTER 10-STRUCTURAL REINFORCED CONCRETE WALLS Fig. 10.4.5-Summary of boundary element requirements for special walls (Fig. R18.10.6.4.2 of ACI 318-14). in Table 10.4.6 of this Handbook. In many cases, the special structural wall requirements apply. If the wall segment is designed as a column, Section 18.7.4, 18.7.5, and 18.7.6 for columns or by alternative requirements given in Section 18.10.8 in ACI 318-14. Wall piers designed to these alternative requirements require: (a) Ve that can develop Mpr at the ends of the column or oo times the factored shear determined by analysis (18.7.6.1 and 18.10.8.1) (b) Hoops at a spacing not greater than 6 in. (c) Checking to see if the pier should include a special boundary element (d) Horizontal reinforcement above and below the wall SLHU WR WUDQVIHU WKH GHVLJQ VKHDU LQWR WKH DGMDFHQW ZDOO segments and coupling beams—A horizontal wall segment is any portion of a wall that is bound by the outer edge of a wall and an edge of an opening, or the portion of a wall bound by the horizontal edges of two openings. According to Chapter 11 of ACI 318-14, the design of nonseismic horizontal walls are designed as special structur special structural wall system, the segment is called a coupling beam. The coupling beam is separated into three categories in ACI 318-14. (a) If En/h < 2 and Vu · Acv fc', design the beam with diagonally placed bars for a more effective transfer of shear through the member (c) For other cases, the beam may be designed either as a special beam or with diagonal placed bars The design of a coupling beam is beyond the scope of this Handbook. For more information, reference Moehle et al. (2011) and Moehle (2015). American Concrete Institute – Copyrighted © Material – www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 10.5—Detailing Structural walls are, in general, thin, tall, wide members with reinforcement in both the horizontal (transverse) and vertical (longitudinal) directions. Properly designed and Fig. 10.4.6—Shear strength for vertical wall segments. Table 10.4.6—Governing design provisions sions for 8.10.1 in ACI vertical wall segments\* (Table R18.10.1 318-14) Length of vertical wall segment/wall thickness (Ew/bw) Clear height of vertical wall segment/wall thickness (Ew/bw) Clear height of vertical wall segment/wall thickness (Ew/bw) > 6.0 hw/Ew < 2.0 Wall Wall all Wall all Wall all Wall hw/Ew • Wall II ppier required uired Wall pier WRVDWLVI\VSHFL¿HG required to column design VDWLVI\VSHFL¿HG requirements; refer to Section 18.10.8.1 (ACI 318-14) 318-14) Wall \* hw is the clear height, Ew is the horizontal length, and bw is the web of the web of the wall segment. detailed shear walls in buildings have resisted seismic forces and sidesway effectively in past earthquakes. ,I VKHDU ZDOOV DUH WKH RQO\ PHPEHUV LQ WKH /)56 WKH\ XVXDOO\EHKDYHDVFDQWLOHYHUEHDPV¿[HGDWWKHEDVH7KH\ transfer moments, shear, and axial forces to the foundation. ,I WKH /)56 LQFOXGHV D VWLII IUDPH WKH ZDOO FRXOG EHKDYH more like a column, depending on relative stiffnesses and shear wall locations. In these cases, the shear wall will XVXDOO\FROOHFWWKHDUEXWWKHLUPRPHQWV are much less due to frame action. Reinforcement placed in the horizontal and vertical directions resists in-plane shear forces and limits cracking. For WDOOHUZDOOVWKHYHUWLFDOUHLQIRUFHPHQW /I VLJQL¿FDQW PRPHQW VWUHQJWK LV UHTXLUHG DGGLtional reinforcement is placed at the ends of a wall or within boundary elements (Fig. 10.5a and 10.5b of this Handbook). Although one curtain of reinforcement is permitted for ordinary shear walls, two curtains of reinforcement are recommended. It is advantageous to place the transverse reinforcement as the exterior layer to prevent longitudinal UHLQIRUFHPHQWIURPEXFNOLQJDQGSURYLGHEHWWHUFRQ¿QHPHQW to the concrete. The casting position of transverse reinforcement assumes more than 12 in. of concrete below the bar for DOFXODWLRORIGHY WKHFDOFXODWLRORIGHYHORSPHQWDOGVSOLFHOHOJWKVz t = 1.3). UVW VSOLFHRIYHUWLFDOUHLOIRUFHPHQWW\SLFDOO\RFFXUV ately above ove the ffoundation, where wall longituimmediately dina reinforcement info nt laps wit dinal with foundation dowel bars. These dowel lap th the w dowels, lapped with wall bars, provide the critical h ism of transferring ferring te mechanism tension and shear forces from the t the foundation. ion. The dowels should extend into the wall to IRXQG LRQ GHSWKVXI iRXQGDWLRQZLWKDGHSWKVXI iRXQGDWLRQXJKWREHIXOO\GHYHOn. F co oped for tension. For constructability purposes, dowels with ooks should extend to the bottom of the foundaZKHUHWKH\F
WLRQZKHUHWKH\FWLRQZKHUKHYKH longitudinal reinforcement be developed or spliced for 1.25fy if the foundation connection is where yielding of wall reinforcement is likely to occur as a result of lateral displacements. )LJD±±'HYHORSPHQWRIZDOOKRUL]RQWDOUHLQIRUFHPHQWLQFRQ¿QHGERXQGDU\HOHPHQW American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 399 Fig. 10.5b—Boundary element requirements for special walls. 10.6—Summary in advantages: ntages: structural walls have two main c uct because reinf rce1. They are relatively easy to construct reinforcent frd. ment detailing of walls is straightforward. tiff they usually m ni2. Because of their inherent stiffness, minit d nonstructural mize sway and damage in structural and elements, such as glass windows and building contents, for buildings exposed to high lateral loads. o: Structural and elements, such as glass windows and building contents, for buildings exposed to high lateral loads. mechanical requirements. 2. Shear walls carry large lateral forces resulting in the possibility of large overturning moments. Attention is required at the wall-foundation interface and foundation design. REFERENCES American Concrete Institute (ACI) ACI 318-14—Building Code Requirements for Structural Concrete and Commentary ACI 551.2R-10— Design Guide for Tilt-Up Concrete Panels American Society of Civil Engineers (ASCE) \$6& \$6&(^20LQLPXP'HVL]Q/RDGVIRU%XLOGLQJVDQG 2WKHU6WUXFWXUHV ed documents Authored a, F.; Hanson, nson, J. M.; and Corley, W. G., 1977, "Shear Barda, 6WUH KRI 5LVH:DOOVZ 6WUHQJWKRI/RZ5LVH:DOOVZLWK%RXQGDU\(OHPHQWV'ReinConc uctures in Seismic Zones, SP-53, Amerforced Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In ional9 Concrete Hills, MI, pp. 149-202. \*DU D/ 3&RQ \*DUFLD/(3&RQFUHWH4 \$4XHVWLRQV' Conc e In iona Design of Reinforced Concrete McGraw-H Education, New York, 760 pp. Buildings, McGraw-Hill Moehle, J. P.; Ghodsi, T.; Hooper, J. D.; Fields, D. C.; Ged and Gedhada, R., 2011, "Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams: A Guide for Practicing Engineers," NEHRP Seismic Design Technical Brief No.6, National Institute of Standards and Technology, Gaithersburg, MD, 37 pp. Wallace, J. W., 1996, "Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls," Earthquake Spectra 9 1R 0D SS GRL 10.1193/1.1585883 American Concrete Institute – Copyrighted © Material – www.concrete.org Walls The amount of required transverse reinforcement is found in Table 18.10.6.4(f) (ACI 318-14). The spacing of the transverse reinforcement is tighter at the base of the wall for a his distance & or Mu/4Vu at the lesser of 6 in. or 6db. Above this region, the spacing widens to 8 in. or 8db untill the reinforcetransverse reinforcement ratio drops below 400/fy, where transverse ment is not required (Fig. 10.5b). 400 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 10.7—Examples Shear Wall Example 1: Seismic Design Category B/wind—The reinforced concrete shear wall in this example is nonprestressed. This shear wall is part of the lateral force-resisting-system (a shear wall is at each end of the structure) in the NorthSouth (N-S) direction of the hotel (Fig. E1.1). Material properties are selected based on the code limits and requirements of &KDSWHUV DQG \$&, HQJLQHHULQJ MXGJPHQW DQG ORFDOO\ DYDLODEOH PDWHULDOV7KH VWUXFWXUH LV DQDO\]HG IRU DOO

UHTXLUHGORDGFRPELQDWLRQVE\DQHODVWLF'¿QLWHHOHPHQWDQDO\VLVVRIWZDUHPRGHOWKDWLQFOXGHVVKHDUZDOOIUDPHLQWHUDFWLRQ The resultant maximum factored moments and shears over the height of the wall are given for the load combination selected. This example provides the shear wall design only at the base. Given: Pu = 1015 kip In-plane— Vu = 235 kip Mu = 18,600 ft-kip Out-of-plane— Vu = 16 kip Mu = 60 ft-kip Material properties— fcg SVL fy = 60,000 psi )LJ(<sup>2</sup>%XLOGLQ]ÅRRUSODQ¿UVWÅRRU This example uses the Interaction Diagram spreadsheet aid found at . aspx?ItemID=SP1714DAE. ACI 318-14 Discussion Calculation Step 1: Geometry 11.3.1 This wall design example follows the requirements of Section 11.5.2, and, therefore, the wall thickness does not need to meet the requirements of Table 11.3.1.1 (ACI 318-14). The unsupported height controls; 18 ft < 28 ft From Table 11.3.1.1, the wall thickness must be at least the greater of 4 in. and the lesser of 1/25 the hreq'd = (18 ft)(12 in./ft)/25 = 8.64 in. IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (28 ft from end to end of the wall). A 12 in. wall is used in this design and the wall is assumed to be exposed to weather on the exterior of the structure. Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 401 6WHS/RDGVORDGSDWWHUQVDQGDQDO\VLVRIWKHZDOO 11.4 The structure is analyzed using 3D elastic Finite 7KHPD[LPXPIDFWRUHGD[LDOIRUFHÀH[XUDOPRPHQW Element Analysis (FEA) software that follows the and shear force at the base of the wall are listed in the analysis requirements of Section 11.4 and Chapters given sectively refer to Fig. E1.2 and E1.3 for in-plane AH[XUHDQGLQSODQHVKHDUDORQ]WKHKHLJKWRIWKH wall, respectively. )LJ(<sup>2</sup>,QSODQHAH[XUHDORQJWKHKHLJKWRIWKHZDOO J WKHK Fig. E1.3—In-plane shear along the height of the wall. Step 3: Concrete and steel material requirements (ACI 318-14). The designer determines the durability requirements of Chapter 19 and structural strength requirements (ACI 318-14). The designer determines the durability requirements of Chapter 19 and structural strength requirements (ACI 318-14). classes. Please refer to Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318-14. ACI encourages referencLQJ\$&,LQWRMREVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318-14. ACI encourages referencLQJ\$ designer can require, permit, or review if suggested by the contractor. By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW psi. American Concrete Institute – Copyrighted © Material – www.concrete.org 402 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 11.2.1.2 The reinforcement must satisfy Chapter 20 of ACI 318-14. The designer determines the grade of bar and if the bar should be coated by epoxy, galvanized, or both. 6WHS\$[LDODQGÀH[XUDOGHVLJQVWUHQ]WK 11.5, 11.5.2 7KHFRPELQHGD[LDODQGÀH[XUDOGHVLJQVWUHQ]WKRI a shear wall can be determined using an interaction diagram similar to a column interaction diagram. The wall interaction diagram is generated using the Interaction Diagram spreadsheet (link in the given section of this example). Refer to Column Example 9.2 in this Handbook for additional information about the Interaction Diagram spreadsheet. By specifying the rebar grade and any coatings, and that the rebar shall be in accordance with ACI 301, &KDSWHURI\$&, UHTXLUHPHQWVDUHVDWLV¿HG In this case, assume grade 60 bar and no coatings. An initial interaction diagram is made using No. 5 bars at 12 in. spacing throughout the wall (refer to \*Note below). It is assumed that all of the longitudinal reinIRUFHPHQWLVHIIHFWLYHUHVLVWLQJLQSODQHÀH[XUH 7KH¿UVWSDLURI1REDUVLVDVVXPHGWREHDWLQ from the end of the wall, the second pair is placed at 12 in. from the end of the wall, and the remaining pairs at 12 in. spacing at the end of the wall is to allow for cover on the end pairs of bars in the wall and to force the reinforcement is symmetrical. The reinforcement is symmetrical about the center of the wall and this bar layout is applied to both ends of the wall. To estimate an initial reinforcement area, the wall is Fig. E1.4 shows the resulting design strength interacDPRXQWRIÀH[XUDO diagram. The design strength interaction diagram y to resist th LQFOXGHVWKH[D reinforcement necessary the moment is LQFOXGHVWKH[DFWRU calculated. ion Diagra The Inte Interaction Diagram spreadsheet contains a sheet PHG W\$[LDO/R QDPHG<sup>3</sup>6HOHFW\$[LDO/RDG':KHQWKHXVHUHQWHUVD Pn, the ssheet calculates th the associated maximum Mn on th inter iagram ccurve and plots a point on the the interaction diagram in ract ram to sh interaction diagram show that point. th "Input Point" on the interacThiss po point is named the gram. The input point of Pn corresponding tion diagram. to a Pu of 1015 kips calculates a point on the design strength interaction diagram. The input point is plotted as a solid triangle. The open triangle indicates where the example load resultants are and shows that this iteration does satisfy required strength. Further iterations are unnecessary. \* Note: For constructability, No. 5 bars are selected for the vertical reinforcement. Smaller bars will work for WKHVWUHQJWKRIWKHZDOOEXWDUHRIWHQWRRAH[LEOHWRHI&FLHQWO\ZRUNZLWKLQDYHUWLFDOSRVLWLRQ American Concrete Institute – Copyrighted © - www.concrete.org Fig. E1.4—Design strength interaction diagram. Step 5: Shear design strength (in-plane) 11.5.4.5 therefore, by inspection it will not control the wall 21.2.1 design. 11.5.1.1 In-plane shear, Vu is given as 235 kips Equation gives: Vn = Vc + Vs Vc = 2 f c'(h)(d) where ss and dcw = (0.8)(336 h = 12 in., wall th thickness in.) = 268.8 in. From Table 21.2.1(b) .1(usee shear strength (out-of-plane) 11.5.1 As shown in Step 4, the layers of No. 5 vertical 11.5.3.1 ZDOOUHLQIRUFHPHQWVDWLVcHVWKHLQWHUDFWLRQHTXDWLRQ 21.2.2 for in-plane bending. The resultant of the out-of-plane moment, Mu = 60 ft-kip is within the middle third of the wall. This allows Section 11.5.3.1 to be used to check the outof-plane strength of the wall. Vc = 2 (403) 5000 00 psi (12 (1 in.)(268.8 in.) = 456 kip [Vc = (0 [ kip []Pn = (0.65)(9090 kip) = 5900 kip NLS•NLSOK American Concrete Institute – Copyrighted © Material – www.concrete.org Walls CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 404 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Reinforcement limits 11.6 %HFDXVHWKHLQSODQHVKHDUH[FHHGV[]Vc, 11.6.2 applies. 11.6.2 Assuming 12 in. for spacing in both directions, the Vu•]Vc minimum reinforcement results in: D DQGE PXVWEHVDWLV2HG D dE must be the greater of the three values below: dE 0.0025 ( 0.62 in.2 ) dE•±hwEw idt±  $\rho A \ge 0.0025 + 0.5( 2.5 - 92ft/28 ft) \times | -0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in.) + 0.0025 | ( 12 in$ (12 in.) dEIURPHTXDWLRQQHHGQRWH[FHHGdt in accordance with Eq. (11.5.4.8) dE dt(T \* E d dtE Minimum reinforcement in both the longitudinal and horizontal directions. As, prov. LQ2/ft > As, req'd LQ2/ft OK \* 11.5.4.8 Equation (11.5.4.8) requires zero reinforcement for Equatio strength becau because the concrete strength is adequate in shear. Step 8: Reinforcement detailing rrequirements uirements at 12 in. meets the detailing rrequirements at 12 in. the horizontal direction construction. ire n for ease of con ructio 11.7.4.1 nfor t is not req Reinforcement required for shear strength; the for the maximum spacing for horizontal bars Similarly, HGh FDQQRWH[FHHGhLQ RULQ7KHLQVSDFLQ] ontal bars are less than these limits. of horizontal h > 10 in., therefore two layers are required. Two layers are required. Two layers are required. Two layers are required. Two layers are required. reinforcement is needed to resist strength interaction diagram is two No. 5 bars at 12 in. D[LDOORDGVWLHVDUHUHTXLUHGWRFRQ¿QHWKHYHUWLFDO on center spacing. The Ag within this length is 144 in.2 UHLQIRUFHPHQW7KHUHLQIRUFHPHQWUDWLRIRUWKHÀH[ Ast is 0.62 in.2 ural vertical reinforcement at the wall ends needs to The ratio of Ast to AgLV7KLVLVOHVVWKDQWKH 0.01 and therefore ties
are not required by 11.7.4.1. be calculated to determine if ties are required. The maximum factored moment is IWNLSRULQOE7KHIDFWRUHGD[LDO stress on the concrete due to the combined loads is: 1 OE^LO IW LQIW `LQOEîLQLQ4 SVL This is below the design strength of concrete and thus steel is not needed to resist the axial load. Therefore, ties are not required by Section 11.7.4.1. Refer to Fig. E1.5 and E1.6 for wall elevation and section cut at the ends of the wall. American Concrete Institute – Copyrighted © Material – www.concrete.org / CHAPTER 10 -STRUCTURAL REINFORCED CONCRETE WALLS 405 Walls Step 9: Detailing Fig. E 1.5—Vertical bar distribution. Fig. E 1.5 The reinforced concrete shearwall in this example is nonprestressed. This shearwall is part of the lateral force-resistingsystem (a shearwall is at each end of the structure) in the North-South (N-S) direction of the hotel (Fig. E2.1). Material properties are selected based on the code limits and requirements of Chapters 19 and 20 (ACI 318-14), engineering judgment, and ORFDOO\DYDLODEOHPDWHULDOV7KHVWUXFWXUHLVDQDO\]HGIRUDOOUHTXLUHGORDGFRPELQDWLRQVE\DQGHODVWLF'¿QLWHHOHPHQWDQDO\VLV software model that includes shearwall-frame interaction. The resultant maximum factored moments and shears over the height of the wall are given for the load combination selected. This example provides the shearwall design and detailing at the base of the wall. Given: Forces and moments at the wall base— Pu = 1015 kip In-plane— Vu = 32 kip Mu = 37,200 ft-kip JLJ(2%XLOGLQJÅRRUSODQ¿UVWÅRRU ) This example shows the design and detailing of a special shear wall due to in-plane forces, including a seismic ial structural shea ERXQGDU\HOHPHQWDWKHRXWRISODQHIRUFHVLVYHUL¿HG,QWKLVH[DPSOHRQO\ one loading condition is checked. In a typical design, several load combinations require checking. This example uses the Interaction Diagram spreadsheet aid found at . aspx?ItemID=SP1714DAE. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS Calculation hreq'd = (18 ft)(12 in./ft)/25 = 8.64 in. Example shear wall h = 12 in > hreq'd = 8.64 in. OK 7KHPD[LPXPIDFWRUHGD[LDOIRUFHAH[XUDOPRPHQW 7KHPD[LPXP and shear force at the bbase of the wall are listed in the en se tion at the start tart of this example. given section American Concrete Institute – Copyrighted © Material – www.concrete.org Walls ACI 318-14 Discussion Step 1: Geometry 11.3.1 This wall design example follows the requirements of Table 11.3.1.1 (ACI 318-14). However, the thickness chosen is an appropriate design starting point. Note that where special boundary elements are required, the special boundary element will be thicker. From Table 11.3.1.1, the wall thickness must be at least the greater of 4 in. and the lesser of 1/25 the lesser of 1/25 the lesser of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHOHYDWHGARRU DQGWKHXQVXSSRUWHG length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKH¿UVWHGHYGW length of the wall (18 IWIRUWKHµgw length design and the wall is 20.6.1.3.1 assumed to be exposed to weather on the exterior of the structure. Concrete cover is 1-1/2 in., which is in accordance with Table 20.6.1.3.1 (ACI 318-14). 6WHS/RDGVORDGSDWWHUQVDQGDQDO/VLVRIWKHZDOO 11.4 The structure is analyzed using sing the assumptions and on 11.4. requirements of Section The structure was analyz analyzed using 3D elastic Finite Element Analysiss (FE (FEA) software that follows the me off Section 11.4 of o ACI 318analysis, LJ DQG(IRULQ SODQHAH[ure and in-planee sshear along the height off the wall, respectively). 407 408 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) )LJ(<sup>2</sup>,QSODQHÀH[XUHDORQ]WKHKHLJKWRIWKHZDOO Fig. E2.3—In-plane shear along g th the height ight of the wal wall. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS Step 3: Concrete and steel material requirements 11.2.1.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements (ACI 318-14). The designer determines the durability classes. Please refer to Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. 409 By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW psi. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318-14. ACI encourages referencLQJ\$&,LQWRMREVSHFL¿FDWLRQV There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. The reinforcement must satisfy Chapter 20 of ACI 318-14. de of bar and if the The designer determines the grade y epoxy or galvanized, or rebar should be coated by both. DFWLRQ 6WHSD\$[LDODQGÀH[XUDOGHVLJQ HQJWK 7KHFRPELQHGD[LDODQGÀH[XUDOGHVLJQVWUHQJWK mined using an in eraction of a shearwall iss interaction lumn interaction diag diagram similarr tto a column diagram. By specifying the rebar grade and any coatings, and that the rebar shall be in accordance with ACI 301-10, &KDSWHU
&KDSWHU &KDSWHU &KDSWHU &KDSWHU &KDSWHU &KDSWHU &KDSWHU &KDSWH wall interaction generated using the m spreadsheet ht (link in the given i Interaction Diagram spreadsheet. American Concrete Institute - Copyrighted © Material - www.concrete.org Walls 11.2.1.2 410 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHSE\$[LDODQGAH[XUDOLQWHUDFWLRQGLD]UDPLQSODQH 11.1.2, Section 11.1.2 requires that special structural walls 18.10.5 be designed in accordance with Chapter 18 of ACI 318-14. Chapter 18 covers all requirements necessary to design the wall. An initial interaction diagram is generated using No. 8 bars at 12 in. spacing throughout the wall. It is assumed that all of the longitudinal reinforcement is HIIHFWLYHWRUHVLVWLQSODQHAH[XUH 7KH¿UVWSDLURI1REDUVLVDVVXPHGWREHDWLQ from the end of the wall, the second pair is placed To estimate an initial reinforcement area, the wall is at 12 in. from the end of the wall, and the remaining DVVXPHGDVDFDQWLOHYHUDQGWKHDPRXQWRIAH[XUDO pairs at 12 in. spacing. The wall is symmetrical about reinforcement necessary to resist the moment is the center of the wall and this bar layout is applied at calculated. both ends of the wall. Fig. E2.4 shows the resulting design strength interaction diagram. The design strength interaction diagram LQFOXGHVWKHIDFWRU7KLVVSUHDGVKHHWFRQWDLQVDVKHHW QDPHG<sup>3</sup>6HOHFW\$[LDO/RDG<sup>'</sup>:KHQWKHXVHUHQWHUVDPu, WKHVKHHWFDOFXODWHVWKHDVVRFLDWHGPD[LPXP]Mn on the design strength interaction diagram curve and plots a point on the design strength interaction diagram. It also generates a corresponding maximum Mn on the gen streng interaction diagram (not shown). nominal strength hee ""Input Point" on the interaction This point is called the gram Th i t po i of Pu of 1015 kips calculates diagram. Thee input point D[LP ]M Mn point o on the interaction diagram of DPD[LPXP]M 4 200 ft-kips. The input point is plotted as a solid 40,200 tri ngle along the h iinteraction t triangle curve. The example 5 kip and an Mu of 37,200 ft-kip. The has a Pu of 1015 op n tri ndicates where the example Pu and Mu are and shows that this iteration does satisfy required strength; therefore, further iterations are unnecessary. Fig. E2.4—Design strength interaction diagram. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 10-STRUCTURAL REINFORCED CONCRETE WALLS 411 6WHSF\$[LDOÀH[XUDODQGVKHDURXWRISODQH 11.5.1 As shown in Step 4b, the layers of No. 8 vertical ZDOOUHLQIRUFHPHQWVDWLV¿HVWKHLQWHUDFWLRQHTXDWLRQ for in-plane bending. 11.5.3.1 The resultant of the out-of-plane moment, Mu = 120 ft-kip is within the middle third of the wall. Eccentricity of the resultant load: (1015 kip)(e) = 120 ft-kip is within the middle third of the wall. This allows Section 11.5.3.1 The resultant of the out-of-plane moment, Mu = 120 ft-kip is within the middle third of the wall. in. e < 2 in.  $( kA 2 Pn = 0.55(fc')(Ag) | 1 - | c | | ( 32h / / ( (0.8)(202 in.) 2 Pn = 0.55(5 ksi)(12 in.)(336 in.) | 1 - | | | ( 32(12 in.) / Pn = 9090 kip <math>\varphi$ Pn = (0.65)(9090 kip) = 5900 kip 22.5.5.1, 21.2.1 From Table 21.2.2(b) use axial strength reduction factor: []gn strength calculaFor out-of-plane shear, the design tion is:  $\varphi Vc = \varphi(2)$  f c'bw d From Table 21.2.1(b) (b) uuse shear strength reduction factor:  $\Box OK$  Vu = 32 kip The axial and moment design strength over a one foot section oof the wall is adequate. Walls 21.2.2  $\varphi Vc = ((0.75)(2)()(5000 \text{ ppsi})(336 \text{ in })(9.25 \text{ i$ THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Seismic reinforcement detailing 18.10.1 This structure is using a special structural wall to resist the lateral loads applied. Two curtains of steel are used, the distributed reinforcement ratios are met, and the forces are determined within code allowed analysis methods. Therefore, requirements 18.10.1 through 18.10.3 (ACI 318-14) are met. 18.10.2 7KHGLVWULEXWHGZHEUHLQIRUFHPHQWUDWLRVd EDQGdt, for structural walls must be at least 0.0025, except that if Vu does not exceed Acv3¥fcgWKHQd EDQGdt are permitted to be reduced to the values in Section 11.6 of ACI 318-14. This example provides No. bars in the horizontal direction in each face at 12 in. This provides 0.40 in.2 per foot in the horizontal direction. The transverse reinforcement ratio is: dt = 0.4 in.2/(12 × 12) in.2 = 0.0028 > 0.0025 OK This example provides No. 8 bars in the vertical direction. The transverse reinforcement ratio is: dt = 0.4 in.2/(12 × 12) in.2 = 0.0028 > 0.0025 OK This example provides No. 8 bars in the vertical direction. The transverse reinforcement ratio is: dt = 0.4 in.2/(12 × 12) in.2 = 0.0028 > 0.0025 OK This example provides No. 8 bars in the vertical direction.  $in.2/(12 \times 12)$  in.2 = 0.0110 > 0.0025 OK 18.10.2.2 The code requires two curtains of distributed reinforcement if; Vu > 2 Acv3¥fcgRUhw/Ew hw/Ew IW IW ! Two curtains are required and are provided. 18.10.3 lculated from the factored Vu = 470 kip The code allows the Vu calculated n shear. loads to be the design Mu = 37,200 ft-kip 18.10.4 th oof special al structural walls is af The shear strength afeigh to length ength ratio of th fected by the height the wall. The code limitss Vn to: ( $Vn = Acv \alpha c \lambda f c' + \rho t f y$ ) OK Vn = 40 4032 0 in.2 (2 5000 psi) 1248 kip = 12 ere where (12 in.)(28 28 ft)(12 iin./ft) = 4032 in.2 Acv = (1 ZKHUHIc is 2.0 for hw/lw+DQGYDULHVEHWZHHQ 2.0 and 3 for hw/Ew < 2.0. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 10-STRUCTURAL REINFORCED CONCRETE WALLS 413 21.2.4, 21.2.4.1 7KH[DFWRUIRUVSHFLDOVWUXFWXUDOZDOOVLVGHWHUmined by Sections 21.2.4 and 21.2.4.1. From the analysis of the structure, the maximum axial load under seismic loading combinations for this wall is approximately 1200 kip. Pu = 1200 kip From the interaction diagram, Mu corresponding to Pu of 1200 kip is: Nu = 2 × 41,860 ft-kip/18 ft = 4650 kip 18.10.4.4 In this example, the code limits on shear strength based on concrete strength of Vn"Acw¥fcg ZLOODOVROLPLWWKH[DFWRUWR Max shear: Vn = 4032 in.2 (10 5000 psi) = 2851 kip The Vu calculated from the nominal moment strength of the shear wall is greater than the maximum code alORZHGVKHDUVWUHQJWK7KHUHIRUHXVHD[DFWRURI []Vn NLS NLS!NLS2. 18.10.6, 18.10.6.1 'HVLJQIRUAH[XUHDQGD[LDOIRUFHDUHWKHVDPHDV for a non-seismic structural wall. Requirements of Section 18.10.5 are met through the AH[XUDODQGD[LDOLQWHUDFWLRQGLDJUDPGHVLJQSURFHVV in Step 4b. VRQZDOOADQ]HV This w 7KHFRGHSURYLGHV]HRPHWULFOLPLWVRQZDOOADQ]HV wall is rectangular in plan and does not have end ADQJHV ments ar Special boundary elements are often required in special structural walls tto resist the large compression forces att the ends of the walls during an nt. earthquake event. en are required in accordance 6.2 or 18.10.6.3 (ACI 31 -14) Sections18.10.6.2 318-14). American Concret Institute - Copyrighted © Material - www.concrete.org Walls 18.10.5 414 18.10.6.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Section 18.10.6.2 (a) hw/Ew•DUHFRQWLQXRXVIURPEDVH RIVWUXFWXUHWRWRSRIZDOODQGDUHGHVLJQHGWRKDYH DVLQJOHFULWLFDOVHFWLRQIRUÀH[XUHDQGD[LDOORDGV 2WKHUZLVH6HFWLRQDSSOLHV hw/Ew IW IW IWOK :DOOLVFRQWLQXRXVIURPERWWRPRIVWUXFWXUHWRWRSRI wall. OK :DOOLVGHVLJQHGKDYLQJDVLQJOHFULWLFDOVHFWLRQIRU VKHDUDQGÀH[XUHOK 7KHUHIRUH6HFWLRQZLOOEHXVHG JHVDVSHFLDOERXQGDU\ HOHPHQWLIWKHQHXWUDOD[LVGHSWKFDOFXODWHGIRUWKH IDFWRUHGD[LDOIRUFHDQGIDFWRUHGPRPHQWLV]UHDWHU WKDQWKHYDOXHLQ(T Check if. 7KHLQWHUDFWLRQGLD]UDPVSUHDGVKHHWFDOFXODWHVWKH QHXWUDOD[LVGHSWKcZKLFKLVLQ 7KHVRIWZDUHWKDWDODO\IHVWKHVWUXFWXUHSUHVHOWVWKH GHÀHFWLROGDWDIRUWKHVWUXFWXUH7KHYDOXHRI $\mu$ URP WKHVRIWZDUHUVOHHGHG  $\delta$  1/hw).
IVRDVSHFLDOERXOGDU\HOHPHOWLVOHHGHG  $\delta$  1/hw). IVRDVSHFLDOERXOGDU\HOHPHOWLVOHHGHG  $\delta$ /hw = (2.4 in.)(5) (92 ft)(12 in./ft) iu/hw = 0.0109 7KHUHIRUHWKHYDOXHGHWHUPLQHGIURP(T is: c  $\geq$  336 in. = 34.25 in. 600(1.5 × 0.0109) %HFDXVHcRILQLVJUHDWHUWKDQLQ6HF cRILQLV WLRQGRHVUHTXLUHDERXQGDU\HOHPHQW Q GRHVUHT ERX \HOHPHQWWUDQV UVHUHLQ E 7KHVSHFLDOERXQGDU\HOHPHQWWUDQVYHUVHUHLQ WH YHUWLFDOO\DERYH DQGEHORZ LR HDVWWKHJUHDWHUR & WD DQG WKHFULWLFDOO\DERYHDQGEHORZ LR HDVWWKHJUHDWHUR & WD DQG WKHFULWLFDOVHFWLRQDWOHDVWKHJUHDWHUR & WD DQG WKHFULWLFDOVHFWLRQDWOHDVWKHJUHDWHUR & WD DQG WKHFULWLFDOVHFWLRQDWOHDVWKHJUHDWHUR & WD DQG WKHFULWLFDOVHFWLRQDWOHDVWKHJUHDWHUR & WD DQG WKHFULWLFDOVHFWLRQDWOHDVWKHJUHDWHUR & WD DQG WKHFULWLFDOVHFWLRQDWOHDVWKHJWH & WD DQG WKHFULWHUNDVHFWLWHY & WD DQG WKHFULWC & WD DQG W Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 415 18.10.6.2 or 18.10.6.3, (a) through (h) must be VDWLV¿HG a) The boundary element must extend horizontally IURPWKHH[WUHPHFRPSUHVVLRQ¿EHUDGLVWDQFHDW least the greater of c - 0.1Ew and c/2, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent ZLWKiu. 18.10.6.4(a) requires that the special boundary element extend a minimum from the extreme compresVLRQ¿EHU c . 0.1Ew = 67.85 in. - (0.1)(336 in.) = 34.25 in. or c/2 = 67.85 in./2 = 33.925 in. The example rounds the length of the boundary element to 34 in. E :LGWKRIWKHAH[XUDOFRPSUHVVLRQ]RQHb, over the horizontal distance calculated by 18.10.6.4(a), LQFOXGLQJADQJHLISUHVHQWVKDOOEHDWOHDVWhu/16. LVVDWLV $\dot{c}$ HGE\PDNLQJWKHZDOOWKLFNHU over the boundary element to meet the requirement of hu/16 = 216 in./16 = 13.5 in. Therefore, the wall thickness will be 14 in. at the boundary elements. (c) For walls or wall piers with hw $\mathcal{E}w$ •2.0 that are se of structure to effectively continuous from the base ve a single critical secsec top of wall designed to have [LDOORDG WLRQIRUAH[XUHDQGD[LDOORDGVDQGZLWKFE w • 3/8, DOFRP ZLGWKRIWKHAH[XUDOFRPSUHVVLRQ]RQHb over the d in S length calculated Section 18.10.6.4(c) does not apply because the example assum that the wall and boundary element will be assumes reevaluated at teel required; ttherefore, refo the wall is considered of steel LJQH IRU VHYHUDO FULW L GHVLJQHGIRUVHYHUDOFULWLFDOVHFWLRQVIRUÀH[XUHDQG al loa axial loads. HF WKHERXQGDU\HO PHQ G ,QADQIHGVHFWLRQVWKHERXQGDU\HOHPHQWVKDOO FWL DQIHZLGWKLQFR SUHVVLRQ LQFOXGHWKHHIIHFWLYHADQJHZLGWKLQFRPSUHVVLRQ b and shall extend at least 12 in. into the web. Section 18.7.5.3, except the value hx in Section 18.10.6.4(e) imposes spacing requirements on the transverse reinforcement. The boundary element transverse reinforcement shall satisfy Section 18.7.5.2(a) through (e) and Section 18.7.5.3, except the value hx in Section requires that the geometry and Section 18.10.6.4(e) ing of the vvertical and crosstie reinforcement shall spacing meet the requirements of a special structural column. American Concrete Institute - Copyrighted © Material - www.concrete.org Walls 18.10.6.4(a), 18.10.6.4(a) through (d) impose additional geo(b), (c), (d) metric requirements upon the special boundary elements. Section 18.10.6.4 states the following: 416 18.10.6.4 (e) 18.7.5.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Transverse reinforcement shall be in accordance with (a) through (f): (a) Transverse reinforcement shall be in accordance with (a) through (f): (b) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) through (f): (c) through (f): (c) the following: 416 18.10.6.4 (e) through (f): (c) through (f Bends of rectilinear hoops and crossties shall engage peripheral longitudinal reinforcing bars. (c) Crossties of the same or smaller bar size as the KRRSVVKDOOEHSHUPLWWHGVXEMHFWWRWKHOLPLWDWLRQ of Section 25.7.2.2 of ACI 318-14. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement and around the perimeter of the cross section. (d) Where rectilinear hoops or crossties are used, they shall provide lateral support to longitudinal reinforcement in accordance with Sections 25.7.2.3 of ACI 318-14. d such that the (e) Reinforcement shall be arranged spacing hx of longitudinal bars laterally supported sstie or hhoop leg shall not by the corner of a crosstie nd the perimeter of the column. exceed 14 in. around 18.10.6.4(e) 18.7.5.3 18.7.5.3 Spacing reinforcement g oof transverse sverse reinforc ment shall not exceed the sm smallestt of (a) through ((c): ): D 2QHIRXUWKRIWKHPLQLPXPFROXPQGLPHQVLRQ RI PLQLPXPFROXPQ GLPH (b) Six times thee diameter d ter of the smallest smalle longitulongitu dinal bar (c) so, as calculated by: (14 - hx) so = 4 + | (3 |) (18.7.5.3) Section 18.7.5.2 requires that the transverse boundary element reinforcement satisfy essentially the same requirements as those of a special concrete column. +RZHYHUWRVDWLVI\WKH¿UVWSDUWRIH WKH limit of hxIURP6HFWLRQH LVPRGL¿HG The permitted hx is less than 2/3 of the 14 in. width of the boundary element.  $2/3 \times 14$  in. = 9-1/3 in. This permitted hx is for both across the width of the element. The distance between the two curtains of No. 8 bars in a 14 in. thick wall is approximately:  $14 \text{ in.} = (2)(1.5 \text{ in.}) - (2)(0.5 \text{ in.}) - (2)(0.5 \text{ in.}) + (2)(0.5 \text{ in.}) + (2)(1.5
\text{ in.}) + (2)(1.5 \text{$ in.) - 1 in. = 9 in. Therefore, for the thickness of the boundary element, there is no need to add vertical reinforcement. To satisfy this requirement for hx along the length of the element, reinforcement is spaced at 7 in. for the ¿UVWLQWRUHGXFHWKHVSDFLQJWREHORZLQ Note that for this example, the interaction diagram FDOFX FDOFXODWLRQVZHUHPRGL¿HGWRPDWFKWKLVGHWDLOLQJ However, this added precision may not be neces/LQDGHVLJQRI H7 VDU/LQDGHVLJQRI H7 UWKHLUVSHF The sec second part 18.10.6.4(e) and Section 18.7.5.3 T rt of 18.10 determines a maximum spacing of the transverse reinde ermi i forcement eme in thee special boundary element. D LVPRGL¿HGWRRQHWKLUGWKHPLQLPXP PRGL¿HGW column umn dimension by 18.10.6.4(e): 14 in./3 = 4.67 in. 18.7.5.3(b): = 6 in. 18.7.5.3 (c): (14 - 9) so = 4 + | = 5.67 in. (3 |) The value of so from Eq. (18.7.5.3) shall not exceed The example uses a spacing of 4 in. for the transverse reinforcement in the special boundary element and all 6 in. and need not be taken less than 4 in. longitudinal bars in the special boundary element are engaged by a crosstie. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL REINFORCED CONCRETE WALLS 18.10.6.4(f) sets requirements for the area of reinforcement for the ties in the boundary element. (f) The amount of transverse reinforcement for the ties in the boundary element. 18.10.6.4(f), Eq. (a): (Ag) f - 1 | c 0.3 | Ach / fyt (476) 0.3 | -1 = 0.176 > 0.09 | 300 | Ag = (14 in.)(34 in.) = 476 in. 2 Ach = (14 in.)(34 in.) = 45000 psi (4 in.)(12 in.) 60,000 psi Ash = 0.7 in.2 No. 4 ties at 4 in. vertically satisfy 18.10.6.4(g) Section 18.10.6.4(g) requires that the transverse re- 7KLVVHFWLRQLVVDWLV2HGE\H[WHQGLQ]WKHWUDQVYHUVH inforcement extend into the wall support base when reinforcement a minimum of 12 in. into the foundation the critical section occurs at the wall base. element below the base of the wall. 18.10.6.4(h) Section 18.10.6.4(h) requires res that the horizontal reinforcement in the wall be developed within the core of the boundary element. ry ele 18.10.6.5 Does not apply. 18.10.7, 8, and 10 Do not apply. 18.10.9 Section 18.10.9.1 Construction joints in structural ZDOOVVKDOOEHVSHFL¿HGDFFRUGLQJWR6HFWLRQ and contact surfaces shall be roughened consistent with condition (b) of Table 22.9.4.2 of ACI 318-14. Final sketch of structural wall using the special boundary elements development length of the No.4 bars being used The development length being used The development length being used The development length being used The development length being used The development length being used The development length being used The development length be development length be an adve than the 34 in. depth boundary th of the he boundary element. Extending the No.4 lateral ral bbars through hrough the boundary element. iin. of th 6HFWLRQLVVDWLV¿HGE\UHTXLULQJWKDWDOO 6HFWLRQ construction joints in the wall be roughened to approximately a 1/4 in. amplitude in the construction documents.) LJV(DQG(VKRZWKH¿QDOFRQ¿JXUDWLRQRIWKH wall if special boundary elements were required. American Concrete.org Walls 18.10.6.4(f) 417 418 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E 2.5—Final layout of special boundary element reinforcement. Fig. E2.6—Elevation of wall with special boundary elements. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 11.1—Introduction The foundation is an essential building system that transfers column and wall forces to the supporting soil. Depending on the soil properties and building loads, the engineer may choose to support the
structure on a shallow foundations are not covered in this Handbook. Shallow foundation systems include isolated footings that support individual columns (Fig. 11.1(a) and (b), combined footings that support two or more columns (Fig. 11.1(c)), strip footings that support a tank wall, and mat footings (Fig. 11.1(d)), ring footings (Fig. 11.1(d)), ring footings that support a tank wall, and mat footing (Fig. 11.1(f)) that support several or all columns or walls. In this chapter, isolated, combined, and continuous footing examples are presented. A pile cap design is presented in the Strut-and-Tie Chapter of this Handbook. Foundationss 11.2—Footing design typically consulting with a geotechnical engineer who furnishes information in a geotechnical report. Important information that a geotechnical report should include are the: D 6XEVXUIDFH SUR¿OH ZKLFK SURYLGHV SK\VLFDO FKDUacteristics of soil, groundwater, rock, and other soil elements (b) Shear strength parameters to determine the stability of sloped soil (c) Frost depth to determine the bearing level of footing below frost penetration level (d) Unit weights, which is the weight of soil and water per unit volume, used to determine the additional load RQDIRRWLQJVWUXFWXUHZKHQEDFN¿OOHG (e) Bearing capacity, which is the maximum allowable loads Predicte settlement, which is the anticipated vertical (f) Predicted f g over time movement of a footing Fig. 11.1—Shallow foundation types. American Concrete.org 420 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 11.3.1a—One-way shear critical section in footings. /LTXHIDFWLRQZKLFKLVDQLPSRUWDQWVRLOFKDUDF istic if the building's structure 1) and a factored tored loads (ACI (ACI 318-14, Section R13.2.6.1) lat moments ments and forc 318-14, Section 5.3.1) to calculate forces on oc evel; the service load oad the columns and walls at the footing level; oo earing areas an analysis is used to calculate footing pressures are assumed to be uniform or to vary linearly; bearing pressure is measured in units of force per unit area, such as pounds per square foot (b) The effect of anticipated differential vertical settlePHQW EHWZHHQ DGMDFHQW IRRWLQJV RQ WKH VXSHUVWUXFWXUH are considered (c) Footings need to be able to resist sliding caused by any horizontal loads (d) Shallow footings, assumed not to be able to resist tension, should be able to resist overturning moments from compression reactions only; overturning moments are commonly caused by horizontal loads H /RFDO FRQGLWLRQV RU VLWH FRQVWUDLQWV VXFK DV SUR[imity to property lines or utilities, are adequate. 4. Design and detail the footing in accordance with ACI 318-14, Chapter 13. During this step, the previously selected geometry is checked against strength requirements of the reinforced concrete sections. The step-by-step structural design process for concentrically loaded isolated footings follows: 11.3—Design steps 1. Find service dead and live column loads: ACI 318 14, Section R13.2.6 The footing geom geometry is selected using service loads. vice dead loa ro column D = service load from ervic li ve load l d fro ffrom column L = service load from site constr nts and calculate culate the other such that: b ×  $\mathcal{E}$ •Areq constraints 22. Calculate lcul the he design (factored) column load U: ACI on 5.3.1 318-14, Section DLQWKHDOOR 2EWDLQWKHDQUQUU 2EWDLQWKHDQUQUU 2EWDLQWKHDQUQUU 2EWDLQWKHDQUQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKHDQUU 2EWDLQWKH soil pressure based on initial footing: qu = U/EE 5. Check one-way (beam) shear: The critical section for one-way shear extends across the width of the footing: qu = U/EE 5. Check one-way (beam) shear: The critical section for one-way (beam) section for one-way (beam) section for one-way (beam) section for 8.4.3.2. The shear is calculated assuming the footing is cantilevered away from the column or wall ACI 318-14, Section 8.5.3.1.1. For masonry wall (Fig. 11.3.1a(b) right side). 127(,IWKHFDOFXODWHGRQHZD\IDFWRUHGVKHDUH[FHHGV the one-way shear design strength, then increase the footing thickness. Footings are typically not designed with shear reinforcement. 6. Check two-way (slab) shear: (a) Determine the dimensions of loaded area for: i) Rectangular concrete columns, the loaded area for: i) Rectangular concre www.concrete.org CHAPTER 11—FOUNDATIONS 421 Fig. 11.3.1b—Two-way o-way shear critical section in footings. s. beff = b f + Foundationss ii) Steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of
the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the fac faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face faces of the steel columns, the effective tive loaded d area is assumed nd to be halfway between the face face faces of the stee 11.3.1b(b)): bbpp - b f 2 where bfLVWKHZLGWKRIFROXPQADQJHDQGbbp is base plate side. (b) Calculate the shear critical section, located at a distance of d/2 outside the loaded area (ACI 318-14, Section 13.2.7.2) (c) Calculate the factored shear force for two-way shear stress, vu (d) Compare vuWRWZRZD\GHVLJQVWUHVV[vn (ACI 318-14, Section 22.6.5.2). 127(, I WKH GHVLJQ VKHDU VWUHVV LV OHVV WKDQ IDFWRUHG shear stress, then increase footing thickness and repeat steps are designed and detailed for moment in one direction and the same reinforcing is placed in the other direction. For rectangular footings the reinforcing must be designed and detailed in each direction. The critical section Fig. 11.3.1c—Column load distribution in footing. for moment extends across the width of the footing at the face of the column. ACI 318-14, Sections 13.2.6.4 and 13.2.7.1. D &DOFXODWHSURMHFWLRQx, from the column face (Fig. 11.3.1d): x =  $\mathcal{E}/2 - c/2$ , where c is the smaller dimension perpendicular to the critical section in each direction (b) Calculate total factored moment, Mu, at the critical section (c) Calculate required As. ACI 318-14, Sections 13.3.2.1 and 7.6.1.1, specify a PLQLPXPAH[XUDOUHLQIRUFHPHQWAs must be met, and 7.7.2.3 VSHFL¿HVDPD[LPXPEDUVSDFLQ]RILQ 8. Check the load transfer from the column to the footing per ACI 318-14, Section 16.3 (Fig. 11.3.1e) (a) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (b) Check the load transfer from the column to the footing per ACI 318-14, Section 16.3 (Fig. 11.3.1e) (b) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (b) Check the load transfer from the column to the footing per ACI 318-14, Section 16.3 (Fig. 11.3.1e) (b) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (b) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (b) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the footing concrete: ACI 318-14, Section 16.3 (Fig. 11.3.1e) (c) Check the load transfer from the column to the 14, Section 22.8.3.2 (b) Calculate the load to be transferred by reinforcement (usually dowels): Bdowels = Pu±Bn American Concrete Institute - Copyrighted © Material - www.concrete.org 422 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 11.3.1f—Bar distribution in short direction. Fig. 11.3.1d—Moment critical section in short direction in short direction. footing at rein reinforced concrete column face. be uniformly spaced in each direction (ACI 318-14, Sections 13.3.2.2 and 13.3.3.2): w (i) In a band of width Bs centered on column: 2 A /b + 1 ((total tal # of bars)) (round uup to an interger) #bars ars iin Bs = (i Re (ii) Remaining g bars should be uniformly spaced in er pportionss of footi outer footing). The remaining g bars should be uniformly spaced in er pportions of ACl 318-14, Section 9.6.1 9 (refer to Fig. 11.3.1f). (g) Check development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Section 25.4. The development length; Ed, in tension per ACI 318-14, Sect ,I[]Bn•Pu only a minimum area of reinforcement is required ACI 318-14, Section 16.3.4.1. (c) Calculate the required reinforcement area and choose bar size and number. (d) Check dowel embedment into footing for compression; Edc. Hooks are not considered effective in compression and are used to stabilize the dowels during construction. (e) Dowels must be long enough to lap with the column bars in compression, Esc: ACI 318-14, Section 25.5.5 (f) Choose bar size and spacing: For square footings, As must be furnished in each direction. The same size and number of bars should 11.4—Footings subject to eccentric loading In addition to vertical loads, footings often resist lateral loads or overturning moments. These loads are typically from seismic or wind forces. 2YHUWXUQLQJPRPHQWVUHVXOWLQDQRQXQLIRUPVRLOEHDULQJ pressure under the footing, where soil-bearing pressure is larger on one side of the footing than the other. Nonuniform soil bearing can also be caused by a column located away from the footing's center of gravity. If overturning moments are small in proportion to vertical loads, that is, the total applied load is located within the kern (e"E/6), then the entire footing bottom is in compression and a P/A ± M/S analysis is appropriate to calculate to
calculate to calculat the soil SUHVVXUHVZKHUHWKHSDUDPHWHUVDUHGH¿QHGDVIROORZV P = the total vertical service load, including any applied loads along with the weight of all of the foundation American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS 2. Calculate the total service overturning moment M, measured at the footing bottom 3. Determine whether P/A exceeds M/S 4. If P/A is less than M/S, then the maximum bearing pressure equals P/A + M/S and the minimum bearing pressure distriEXWLRQ LV VWUXFWXUDOOV LQHIcFLHQW 7KH PD[LPXP EHDULQJ SUHVVXUH VKRZQ LQ WKH cJXUH LV FDOFXODWHG DV IROORZV maximum bearing pressure = 2 P/[(B)(X)] where X = 3( $\epsilon/2 - e$ ) and e = M/P. 11.5—Combined footing If a column is near a property line or near a pit or a mechanical equipment in an industrial building, a footing may not be able to support a column concentrically and the eccentricity is very large. In such a case the column footing LVH[WHQGHGWRLQFOXGHDQDGMDFHQWFROXPQDQGVXSSRUWERWK on the same footing, called combined footing (Fig. 11.5). The combined footing is sized to have the resultant force of the two columns within the kern, or preferably to coincide with the center of the footing area. The combined footing can be rectangular, tr trapezoidal, or having a strap, connecting n column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 2 Calculate C lcula service ce column loads, P1 and P2 (ACI 318-1 Section Sec 3.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-14, R13.2.6) 318-1 2. column load resultant location Cen r fo gular foo Center for rectangular footing:  $xc = P1 x1 + P2 x2 \sum Pi$  If P1 is much larger than P2, then trapezoidal combined footing may be used. Fig. 11.4—Footing under eccentric loading. Fig. 11.5—Common types of combined footing may be used. www.concrete.org Foundationss components, and also including the weight of the soil located directly above the footing. A = the area of the footing bottom. S = the section modulus of the footing bottom. If overturning moments are larger, that is, the total applied load falls outside the kern, e > E/6, then P/A - M/S analysis requires the soil to resist tension (upward movement of the footing), which is not possible. This soil is only able to transmit compression. The following are typical steps to calculate footing bearing pressures if nonuniform bearing pressures if nonuniform bearing pressures if nonuniform bearing pressures if nonuniform bearing pressures are present. and assumes that overturning moments are parallel to one of the footing's principal axes. These steps should be combination with the maximum P usually causes the maximum bearing pressure while the load combination with the minimum P usually is critical for overturning. 1. Determine the total service vertical load P 423 424 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Determine combined footing coincides with the force resultant or is at least within E/6 of the service vertical load P 423 424 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Determine combined footing length from construction constructio force resultant. A B1 + 2 B2 c x = -3 B1 + B2 2 3. Determine combined footing dimensions assuming uniform bearing Combined rectangular footing design step presented above. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 425 11.6—Examples Foundation Example 1: Design of a seven-story building 'HVLJQDQGGHWDLODW\SLFDOVTXDUHIRRWLQJRIDVL[ED\E\ ¿YHED\VHYHQVWRU\EXLOGLQJIRXQGHGRQVWLIIVRLOVXSSRUWLQJD LQVTXDUHFROXPQ7KHEXLOGLQJKDVDIWKLJKEDVHPHQW7KHERWWRPRIWKHIRRWLQJLVIWEHORZ¿QLVKHGJUDGHUHIHUWR)LJ E1.1). The building is assigned to Seismic Design Category (SDC) B. Given: Column load— Service dead load D = 541 kip Service live load L = 194 kip Seismic load E = ±18 kip Material properties— Concrete compressive strength fcg NVL Steel yield strength fy = 60 ksi 1RUPDOZHLJKWFRQFUHWH3 Density of concrete = 150 lb/ft3 Fig. E1.1—Rectangular foundation plan. ACI 318-14 Dis Discussion Step 1: Foundation type 13.1.1 The bottom of the footing baseng is 3 ft below the he base ment slab. Therefore, itt is considered a shallow ref hallo foundation. 13.3.3.1 Calculation C Ca The footing will bee ddesigned detailed with ed and detail th the DSSOLFDEOHSURYLVLRQVRI&KDSWHU2QHZD\VODEV and Chapter 8, Two-way slabs of ACI 318-14. American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss Allowable soil-bearing pressures— D only: qall.D = 4000 psf D + L: qall,D+L = 5600 psf D + L + E: qall.Lat = 6000 psf 426 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 2: Material requirements Concrete compressive strength 19.2.1.1 The value of concrete compressive strength at a 19.2.1.3 JLYHQDJHPXVWEHVSHFL¿HGLQWKHFRQWUDFWGRFXments. Table 19.2.1.1 provides a lower concrete compressive strength limit of 2500 psi. 19.3.1 19.3.2 Provided: fcg SVL!fgc,min = 2500 psi OK Exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either
assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure categories and classes The engineer must either assign exposure ready-mix supplier can proportion the concrete mixture proportions in the concrete mixture proportions in the contract documents. Based on the exposure classes, the concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Concrete mixtures must satisfy the most restrictive requirements of Table 19.3.2.1. Category F The foundation is placed below the frost line, there- Class F0 elements—freezes. The mum fcq SVL SV SVL ing and thaving cycles. Therefore, class F0 applies. Minimum 19.3.2.1 PHQ PXVWEHVDWL IRU) 0L[WXUHUHTXLUHPHQWVWKDWPXVWEHVDWLV¿HGIRU) ble 19.3.2.1. 2.1. are listed in Table 19.3.2.1. are listed in Table 19.3.2.1. 2.1. are listed in Table 19.3.2.1. 19.3.2.1 Category W The footing may be in contact with water and low permeability is not required. : lw/cm)max = none and fcg SVL 19.3.1.1 19.3.2.1 Category C The concrete is exposed to moisture and there is no external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore the class is C1. Mixture requirements for C1 are listed in Table 19.3.2.1. Air con content is not external source of chlorides; therefore t required and there are no limits on ccementitious menti materials & lw/cm)max = none and fcq SVL Therefore, there is no restrictive minimum concrete compressive strength is 2500 psi, and no limits on the w/cm. Therefore, in the MXGJPHQWRIWKHOLFHQVHGGHVLJQSURIHVVLRQDOXVHSVLFRQFUHWHFRPSUHVVLYHVWUHQJWK E 2WKHUSDUDPHWHUVVXFKDVPD[LPXPFKORULGHLRQFRQWHQWDQGDLUFRQWHQWDUHH[SRVXUHVSHFL¿FDQGWKXVQRWFRPSDUHG with other exposure limits. (c) The fcgXWLOL]HGLQWKHVWUHQJWKGHVLJQPXVWEHDWOHDVWZKDWLVUHTXLUHGIRUGXUDELOLW\ American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS Step 3: Determine footing dimensions 13.3.1.1 To calculate the footing base area, divide the service load by the allowable soil pressure. area of footing = total service load ( $\Sigma$  P) allowable soil pressure qa 427 The unit weights of concrete and soil are 150 pcf and 120 pcf; close. Therefore, footing: D 541 kip = 135 ft 2 qall ., D 4 ksf (D + L) qall ., D 4 ksf (D + L) qall ., D + L = Controls 541 kip + 194 kip = 131 ft 2 5.6 ksf D + L + E 541 kip + 194 kip + 194 kip + 194 kip + 194 kip + 194 kip + 194 kip = 131 ft 2 5.6 ksf D + L + E 541 kip + 194 kip + 1 (0.7)18 kip = 125 ft 2 qall ., Lat 6 ksf Assuming a square footing. A = The footing thickness is calculated in Step 5, footing design. Therefore, try a 12 x 12 ft square footing. Foundationss 135 ft 2 = 11.6 ft American Concrete Institute – Copyrighted © Material – www.concrete.org 428 THE REINFORCED CONCRETE DESIGN
HANDBOOK—SP-17(14:10) and the step 5, footing design. Step 4: Soil pressure Footing stability Because the column doesn't impart a moment to the footing, the soil pressure under the footing is assumed to be uniform and overall footing is assumed to be uniform and overall footing is assumed to be uniform and overall footing stability is assumed to be uniform and overall footing is assumed. from the applied factored loads. 5.3.1(a) /RDG&DVH, U = 1.4D U = 1.4D U = 1.4D U = 1.4D U = 1.4D U = 1.2D + 1.6(194 kip) = 960 kip qu = 5.3.1(d) U = 1.2D D + E + L /RDG&DVH, 9U / RDG&DVH, 9U = 0.9D + E 13.3.2.1 144 ft 2 = 6.7 ksf Controls 861 kip 86 ksf = 66.0 0 ksf Controls 861 kip 86 ksf = 66.0 0 ksf Controls 861 kip 86 ksf = 66.0 0 ksf Controls 861 kip 86 ksf = 66.0 0 ksf Controls 861 kip 86 ksf = 66.0 0 ksf Controls 861 kip 86 ksf = 66.0 ksf Controls 861 kip 86 ksf = 66.0 ksf Controls 861 kip 86 ksf Controls 861 kip 86 ksf Controls 861 kip 86 ksf Controls 861 kip 86 ksf Controls 861 kip 86 ksf Controls 8 ft 2 U =  $0.9D \ 0.9D + 1.0E = 0.9(541 \text{ kip}) + 18 \text{ kip} = 505 \text{ kip} \text{ qu} = \text{The load combinations include the seismic uplift force. In this example, uplift does not occur. 960 kip U = <math>1.2D \ 1.2 + 1.0E \ E + 11.0L \ 0L = 1.2(541 \ \text{kip}) + 18 \ \text{kip} + 194 \ \text{kip} = 861 \ \text{kip} \ 1.0(194 \ \text{qu} = 5.3.1(e) \ 757 \ \text{kip} = 5.3 \ \text{ksf} \ 144 \ \text{ft} \ 2 \ 505 \ \text{kip} \ 144 \ \text{ft} \ 2 \ = 3.5 \ \text{ksf}$ 1RWH7KHIXOOGH¿QLWLRQRIE includes not only earthquake loads dues to overturning, but also earthquake loads due to vertical acceleration of ground as per ASCE 7-10, Section 12.4.2. Because the footing is square, it will only be designed in one direction. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11-FOUNDATIONS 429 6WHS2QHZD/VKHDUGHVLJQ Fig. E1.2—One-way shear in longitudinal direction. 21.2.1(b) 7.5.1.1 Shear strength reduction factor: []Vn • Vu ]]shear = 0.75 7.5.3.1 22.5.1.1 Vn = Vc + Vs Assume Vs = 0 (no shear reinforcement) Vn = Vc + Vs Assume Vs = 0 (no shear reinfor critical al section fo for one-way way nce d from m the face of th shear at a distance the column 1.2 (refer to Fig. E1.2). uld either er assume a valu The engineer could value for d that WK E\LW RQRU VDWLV¿HVWKHVWUHQJWK(T E\LWHUDWLRQRU equate Eq. (22.5.5.1) to Eq. (7.5.1.1) and solve for d. Foundationss 20.6.1.3.1 ,QWKLVH[DPSOHWKH¿UVWDSSURDFKLVIROORZHG Assume that the footing is 30 in. thick. The cover is 3 in. to bottom of reinforcement. Assume that No. 8 bars are used in the both directions and design for the more critical case (upper layer). Therefore, the effective depth d: d = 30 in. - 3 in. - 1 in. - 1 in./2 = 25.5 in. (A c)  $\varphi$ Vn  $\geq$  Vu = | - - d bqu 2 2 / (12 ft 24 in. 25.5 in.) (12 ft)(6.7 ksf) Vu = | - 2(12 in./ft) 12 in./ft  $| / (2 = 231 \text{ kip } \phi \text{Vc} = 0.75(2) 4000 \text{ psi}$  (12 ft)(25.5 in.)(12 in./ft) = 348 kip - 2(12 in./ft) = 348 kip - 2(12 in./ft) = 348 kip - 2(12 in./ft) (12 ft)(6.7 ksf) Vu = | - 2(12 in./ft) = 348 kip -DESIGN HANDBOOK—SP-17(14) Step 6: Two-way shear design The footing will not have shear reinforcement. Therefore, the nominal shear strength for this twoway footing is simply the concrete shear strength for this twoway footing is simply the concrete shear strength for this two and propagate at 45 degrees away and down from the column corners. The area of concrete that resists shear is calculated at an average distance of d/2 from column face on all sides (refer to Fig. E1.3). Fig. E1.3—Two-way shear. bo = 4(c + d) bo = 4(dept average of the effective depth in the two orthogonal alcula directions when calculating the shear strength ut in this example the ssmaller of the footing, but effective depth is used. 8.4.2.3.4
22.6.5.1 22.6.5.2 he strength ength equations forr The two-way shear IRR VPXVWEHVDWLV¿ DQGWKH QRQSUHVWUHVVHGIRRWLQJVPXVWEHVDWLVcHGDQGWKH alu of (a), a), (b), and (c ntrols: least calculated value (c) controls: (a) vc =  $4\lambda$  f c' vc = 1/4 (b) vc =  $2 + 1/\lambda$  f c'  $\beta/\sqrt{4}$  (vc =  $1/2 + 1/\lambda$  f c'  $\beta/\sqrt{4}$  (vc = 1/2 $21.2.1(b) (\alpha d) (c) vc = |s + 2| \lambda fc' b (o) (4000 psi) = 452 psi (198 in.) vc = |+2| (1.0)(4000 psi) = 452 psi (198 in.) |x = 40, considered interior column Equation (a) controls; vc = 253 psi Vc = 4\lambda fc' b d Vc = Use a shear strength reduction factor of 0.75: 'c o 8.5.1.1 4(1.0)(4000 psi)(198 in.)(25.5 in.) = 1277 kip 1000 lb/kip [] \varphi Vc = (0.75) 4\lambda fb d [] Vc = (0.75) 4\lambda fb d$ 0.75(1277 kip) = 958 kip Vu = qu [(a) 2 - (c + d) 2] 2 ( 24 in. + 25.5 in.) Vu = (6.7 ksf) (12 ft)(12 ft)(12 ft) - (12 ft)(12 ft) - (12 ft)(12 ft) - (12 ft)(12 ft) - (12 ft)(12 ft)(12 ft)(12 ft) - (12 ft)(12 ft)(12 ft)(12 ft)(12 ft)(12 ft)(12 ft)(12 ft)(12 ft) - (12 ft)(CHAPTER 11—FOUNDATIONS 431 Step 7: Flexure design 13.2.7.1 The code permits the critical section to be at the face of the column (refer to Fig. E1.4). Fig. E1.4—Flexure in the longitudinal direction. 2 (A - c) Mu = gu | (b)/2 2 2.2.1.1 Set concrete compression mpre force equal tto the steel mn face: C = T tension force at the column 22.2.2.4.1 nd T = Asfy C = 0.85 fcgba and a = 7.5.2.1 As fy 'c 0.85 f b and 24 in. (12 ft - 12 in./ft | (12 ft)/2 = 1005 ft-kip (ksf) | Mu = (6.7 | 2 | | ) a = As (60 ksi) s) = 0.15 As 0.85(4 4 ksi)(12 ft) fa  $(\phi M n = \phi As fy | d - 1 | 2 / 2 / 3.1.1 22.2.2.2 21.2.1(a)$  Substitute for a in the equation above. 8.5.1.1(a) 6HWWLQJ[]Mn • Mu = 1005 ft-kip and solving for As: Use moment strength reduction factor from Table 21.2.1. 0.15) [As  $(\phi M n \ge (0.9) \text{ As} (60 \text{ ksi}) | 25.5 \text{ in.} - (2 | As \cdot LQ2 \text{ Use } 13 \text{ No. 8 bars} (13 \times 0.79 = 10.27 \text{ in2}) distributed uniformly across the entire 12 ft width of footing: 8.6.1.1$ &KHFNWKHPLQLPXPUHLQIRUFHPHQWUDWLRdl = 0.0018 As,min = 0.0018 (12 ft)(12 in./ft)(30 in.) = 7.8 in.2 As,prov = 10.27 in.2 > As,min = 7.8 in.2 Check if the assumption of tension controlled beKDYLRUDQGWKHXVHRI[LVFRUUHFW 21.2.1(a) American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss 2 432 21.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) To answer the question, the calculated tensile strain in reinforcement is calculated from similar triangles (refer to Fig. E1.5):  $\epsilon t = 22.2.2.4.1 \ 22.2.4.1 \ 22.4.1 \ 22.4.1
\ 22.4.1 \ 22.4$ (0.79 in.2) = 1.03 in. c = 1.03 in. c = 1.03 in. = 1.21 in. 0.85 0.003 (25.5 in. - 1.21 in.) = 0.06 1.21 in.) = 0.06 1.21 in. it = 0.06 > 0.005 ct = 6HFWLRQLVWHQVLRQFRQWUROOHGDQGWKHDVVXPSWLRQRI0.9 = [] is correct Fi E1 Fig. E1.5—Strain ain distribution distributincluding footing self-weight and slab self-weight and live load above footing: Footing self-weight less soil self-weight and assume 40 psf live load: Ws = (12 ft)(12 ft) (0.5 ft(0.15 kcf) + 0.04 ksf) = 16.6 kip Total weight on supporting soil: WT = 541 kip + 194 k 10.8 kip + 16.6 kip = 762.4 kip Calculate actual soil pressure: qa = 762.4 kip = 5.3 ksf < qall = 5.6 ksf (12 ft)(12 ft) American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 11—FOUNDATIONS 433 Step 8: Transfer of column forces to the base 13.2.2.1 Factored column forces are transferred to the footing 16.3.1.1 by bearing on concrete and through reinforcement. 22.8.3.2 (b) and Bn = 22.8.3.2 (b) and Bn = 22.8.3.2 (b) and Bn = 22.8.3.2 (c) and Bn = 22.8.3.2 (b) and Bn = 22.8.3.2 (c) and Bn = 22.8.3.transferred to the founColumn factored ng and through hrough reinforcement, reinforc ment, usuation by bearing ov dowel area of at le st 0. ally dowels. Provide least 0.005Ag and at least four bbars. 16.3.5.4 25.4.9.2 A2 [(12 ft)(12 in./ft)]2 = =6>2 A1 (24 in.) 2 Therefore, Eq. (22.8.3.2(b)) controls. [bearing ov dowel owel area of at le st 0. ally dowels. Provide least 0.005Ag and at least four bbars. 16.3.5.4 25.4.9.2 A2 [(12 ft)(12 in./ft)]2 = =6>2 A1 (24 in.) 2 Therefore, Eq. (22.8.3.2(b)) controls. [bearing ov dowel owel area of at le st 0. ally dowels. Provide least 0.005Ag and at le psi)(24 in.)2 [Bn = (0.65)(2)(0.8 [Bn = 22546 6 kip ki > 960 96 kkip (Step 4) OK (24 in.)2 = 2.88 in.2 As,dowel owell = 0.005(24 Use eight eig No. 6 bars Bars are in compression for all load combinations. Therefore, the dowels must extend into the footing a compression development length, Edc, the larger of the two expressions and at least 8 in. (refer to Fig. E1.6):  $\int fy \psi r \, db = \frac{3.5 \text{ in.}}{1.00000 \text{ psi}(1.0)} = 14.3 \text{ in.}$  Controls  $\mathcal{E}dc = 0.0003(60,000 \text{ psi})(1.0)(0.75 \text{ in.}) = 13.5 \text{ in.}$  The footing depth must satisfy the following inequality so that the No. 6 dowels can be developed within the provided depth:  $25.3.1 \text{ h} \cdot \text{Edc} + r + \text{dEGZE} + 2\text{db}$ , bars + 3 in. where r = radius of No. 6 bent = 6db hreq'd = 24.1 in. + 6(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75 in. + 2(0.75 in.) + 0.75
in. + 2(0.75 in.) + 0.75 in. + 2(0.75 Foundationss A2 ≤ 2.0 A1 where A1 is the bearing area of the column and A2 is the area of the supporting footing that is geometrically similar to and concentric with the loaded area. Check if 434 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E1.6—Reinforcement development length. Step 9: Footing details Development length 13.2.8.3 Flexural reinforcement bar development is required cb + K tr 3.5 in. + 0 = 3.5 13.2.7.1 at the critical section. This is the point of maximum Mo. 6: db 1.0 in. factored moment, which occurs at the column face. Bars must extend at least a tension development cb + K tr = 2.5 Use maximum maximu length beyond the critical section. db 25.4.2.2 25.4.2.4 () 3 fy  $\psi t \psi e \psi s$  Ad = ( db 40  $\lambda$  fc' cb + K tr ( db // 3 60,, 000 psi p (1.0)(1.0)(1.0) Ad = ( db = 28.5 db 2.5 \ 440 (1.0) 4000 psii / ZKHUHzt = casting g position; on; No.8 N 8 ba bars: Ed = 28.5(1 in.) - 28.5 in. > 12 in. 2. 7KHUHIRUH2. zt = 1.0 because not more than 12 in. of fresh concrete below horizontal reinforcement ze FRDWLOIIDFWRUze = 1.0, because bars are uncoated longitudinal direction:  $\mathcal{E}d$  in the lon  $\mathcal{E}d$ , prov. = 57 in. >  $\mathcal{E}d$ , reg'd = 28.5 in. OK Use straight No. 8 bars in both directions, zs EDUVL[HIDFWRUzs = 1.0 for No. 7 and larger cb = spacing or cover dimension to center of bar, whichever is smaller Ktr = transverse reinforcement index It is permitted to use Ktr = 0. However, the expression: cb + K tr must not exceed 2.5. db American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS 435 Step 10: Detailing Foundationss Fig. E1.7—Footing reinforcement detailing. American Concrete Institute - Copyrighted © Material - www.concrete.org 436 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Foundation Example 2: Design and detail of a continuous footing, founded on stiff soil, supporting a 12 in. concrete wall. The footing is located in Seismic Category D and is 3 ft-0 in. EHORZ¿QLVKHGJUDGH([SRVXUHWRIUHH]LQ]DQGWKDZLQLVXHUHIHUWR)LJ( Given: Wall load— Service due to resisting wind loads) 6HLVPLF27E = ±6 kip/ft (Wall vertical force due to resisting seismic forces) Note: the wall has no out-of-plane moments or shears. Material properties— Concrete = 150 lb/ft3 Allowable soil-bearing pressures— D only: qall., D = 3000 psf /(Tall, E = 5000 psf /(Tall, E = 5000 psf Fig. E E2.1)d elevatio elevation of continuous footing. ACI 318-14 Discussion scuss Step 1: Foundation type 13.1.1 7KLVVWULSIRRWLQJLVIWEHORZ¿QLVKHGJUDGH Therefore, it is considered a shallow foundation. 13.3.2.1 The footing will be designed and detailed with the DSSOLFDEOHSURYLVLRQVRI&KDSWHU2QHZD\VODEV and Chapter 9, Beams, of ACI 318-14. 13.2.3.1 Foundation resisting earthquake forces must comply with Section 18.2.2.3 of ACI 318-14. Calculation Ca American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11–FOUNDATIONS Step 2: Material requirements 13.2.1.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements (ACI 318-14). The designer determines the durability classes. Please see Chapter 4 of this Handbook for an indepth discussion of the categories and classes. 437 By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVWSVL \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLV coordinated with ACI 318-14. ACI encourages UHIHUHQFLQ]\$&,LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. area of footing = total service load ( S P kip/ft ) allowable wable soil pressure pres re ga The footing thickness is calculated in Step 4, footing design. D + L 25 kip/ft + (0.75)(0.6) (6.4 kip/ft) = 7.5 ft 5 ksf D + 0.75 L + (0.75)(0.6) (0.7)(0.6) kip/ft) = 7.5 ft 5 ksf U se B = 10 ft American Concrete Institute Copyrighted © Material – www.concrete.org Foundationss Example 1 of this chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. Step 3: Determine footing dimensions g width, ddivide the service 13.3.1.1 To calculate the footing gnoring the foot Ignoring footing selfweight: DOORZ ORDGSHUIRRWE\WKHDOORZDEOHVRLOSUHVVXUH/RDG D 255 kip/ft ki /f combinations are obta obtained from ASCE7-10, = = 8.3 ft q 3 ksf sf all , D Section 2.4. 438 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Footing design Wall stability Because there is no out-of-plane moment, the soil pressure under the footing is assumed to be uniform and overall wall stability is assumed. 7.4.2.1 5.3.1 The footing cantilevers on both sides of the wall are designed as one-way slabs. Calculate the soil pressures resulting from the applied factored loads. /RDG&DVH, U = 1.4D U = 1.4D U = 1.4D = 1.4(25 kip/ft) = 35 kip/ft or 3.89 ksf/RDG&DVH, U = 1.2D + 1.6L U = 1.2D + 1.6L U = 1.2D + 1.6L U = 1.2D + 1.6L U = 1.2D + 1.0W W = 0.9(25.9(9U = 0.9D kip/ft) + 6.4 kip/ft / 12.5 kip/ft / 1 $3.04\ 3\ 4\ ksf = 28.9\ U = 1.2D + E + L\ /RDG\&DVH9U\ 1.2 + 1.0E\ E + 1.0L\ 1.0L\ U = 1.2D\ 1.2\ ft) + 6\ kip/ft\ strute - Copyrighted @ Material$ www.concrete.org CHAPTER 11—FOUNDATIONS 439 One-way shear design—Shear strength reduction factor:  $[]Vn \cdot Vu ]$  shear = 0.75 7.5.3.1 22.5.1.1 22.5.1.1 22.5.1.2 And satisfying: Vu''[]Vn 7.4.3.2 Vu is calculated at d from the face of the wall. 21.2.1(b) 7.5.1.1 13.2.7.1 13.2.7.1 13.2.7.2 y wall, the critic Note that for a m masonry critical sec section ay between ween center and face ce of is located halfway masonry wall. 7.4.3.2 In this example the second approach is followed: Foundationss The engineer can either assume a value for d that VDWLV¿HVWKHVWUHQJWK(T E\LWHUDWLRQRU equate Eq. (7.5.1.1) to Eq. (22.5.5.1) and solve for d. Fig. E2.2—Shear —Shear cr critical section. (B t) Vu = | - d | (5.26 ksf) (2 / 2 d in ft  $\varphi Vc = 0.75(2)(4000 \text{ psi})(d)$ , where d is in inches 13.3.1.2 20.6.1.3.1 Bottom reinforcement must have an effective depth of more than 6 in. Concrete cover must satisfy Table 20.6.1.3.1. Use c = 3 in. Total footing depth: h = d + db + c Solving these two equations for d = 15.0 in. + 3 in. = 20 in. American Concrete Institute – Copyrighted © Material – www.concrete.org 440 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Flexure design Note: Masonry wall is shown in Fig. E2.2 and E2.3 to indicate
that for masonry walls, the critical section for shear or moment are not the face of the masonry wall, but is tw/4 from the face of the masonry walls, the factored moment are not the face of the masonry wall but is 22.2.2.4.1 Set the concrete compression strength equal to the steel tension strength: C = T; 0.85fcqED = Asfy 7.5.1.1 Fig. E2.3—Moment critical sections. Mu = (5.26 ksf)(10 ft/2-1.0 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. Mu = (5.26 ksf)(10 ft/2)/2 = 53.3 ft-kip/ft  $\Pi$ Moment critical sections. barss No. 8 at 12 in. on center. If these bars QRW SURYLGHF DUHQRWKRRNHGSURYLGHFDOFXODWLRQVWRMXVWLI\WKHXVH of straight traig bars. 7.6.1.1 Check if As exceeds ds the minimum: As,min ≥ 0.0018Ag 21.2.1(a) Check if the section is tension controlled and the XVHRI/LVFRUUHFW 21.2.2 To answer the question, the tensile strain in. c= 1.16 in. 0.85 = 1.37 in. 0.003 (16.5 in. - 1.37 in.) = 0.033 1.37 in. It = 0.033 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[] t = (d - c) where: c = a/and a = 1.47As OK American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 441 Fig. E2.4—Strain distribution across footing 9HULI\WKDWWKHDOORZDEOHVRLOSUHVVXUHLVQRWH[ceeded when including footing self-weight and soil self-weight (20 in.) WF = (10 (1 ft) | (0.15 kip/ft 3 - 0.12 kip/ft 3) = 0.5 kip (12) ting so Total weighton su supporting soil: WT = 25 kip/ft + 12.5 kip kip/ft + 12.5 kip kip/ft + 12.5 kip kip/ft + 1.6 kip/ft = 39 39.6 ft Calculate actual soil unit weights

multiplied by the footing volume. American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss (422 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Footing details Shrinkage and temperature reinforcement along length of footing 7.6.4.1 7.6.1.1 The area of shrinkage and temperature reinforcement:  $AS+T = (0.0018)(20 \text{ in}, (10.0 \text{ ft})(12 \text{ in}, /ft) = 4.3 \text{ in}.2 \text{ AS}+T-\text{ Ag Ten No. 6 bottom longitudinal bars will satisfy the requirement for shrinkage and temperature reinforcement in the long direction. Development length for the botting reinforcement beyond the critical tension section. 25.4.2 5.4.2.3 (1) a fy <math>\psi$  w [t es] db dd = [] 40 \ f c' cb + Kt m [] (db ]/ 3 60, 000 psi (1.2)(1.0)(1.0) (1.0)

VXEMHFWWRORDGLQJWKDWLQFOXGHVDQRYHUWXUQLQJPRPHQW7KHERWWRPRIWKHIRRWLQJLVIWEHORZ¿QLVKHGJUDGHUHIHUWR)LJ[ Given: Wall load = 15 kip/ft (including CMU wall weight) Horizontal wind shear V = 3.0 kip/ft (strength level applied at 1 ft above grade) Material properties— Concrete compressive strength fco SVL Steel yield strength fy = 60,000 psi 1RUPDOZHLJKWFROFUHWH3 Density of concrete = 150 lb/ft3 Soil data— gall = 4000 psf qu, permitted = 6000 psf Qu, permitted = 6000 psf Qu, permitted = 6000 psf Qu, permitted = 100 lb/ft3 Fig. E E3.1—Plan 1— and d elevatio elevation of continuous footing. ACI 318-14 Discussion Step 1: Foundation type 13.1.1 7KLVVWULSIRRWLQJLVIWEHORZ¿QLVKHGJUDGH Therefore, it is considered a shallow foundation. 13.3.2.1 C Calculation cula The footing will be designed and detailed with the DSSOLFDEOHSURYLVLRQVRI&KDSWHU2QHZD\VODEV and Chapter 9, Beams, of ACI 318-14. American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 11—FOUNDATIONS Step 2: Material requirements (ACI 318-14). The designer determines the durability classes. Please see Chapter 4 of this Handbook for an indepth discussion of the categories and classes. 445 By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVWSVL \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLV coordinated with ACI 318-14. ACI encourages UHIHUHQFLQJ\$&,LQWRMREVSHFL¿FDWLRQV There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. ck nd then th The footing thickness iss also assumed aand KF DWLRQVLQ6WHS RRWLQ HUL2HGWKURXJKFDOFXODWLRQVLQ6WHS)RRWLQ design. 13.3.1.2 The footing thickness must be such that the bottom reinforcement has an effective depth of at least 6 in. Try 15 in. footing foo thickness. American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss Example 1 of this chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. Step 3: Determine footing wid width a: A = 1(7) (7) =7 ft2/f Area: /ft tion modulus: lus: S = 1(Section 1(7 ft)/(7 ft)/(6 = 8.167 ft3/ft 446 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Footing design Wall stability Because there is an out-of-plane (overturning) lateral force on the stem wall, the overall wall stability must be checked. To calculate the stability of a footing, the total vertical load is calculated and the resulting moment (MR) is compared to the resulting overturning moment (MOTM). Commonly, engineers require MR•MOTM to consider a footing: W soil = | Weight of concrete wall pier pier WWRPRIIRRWLQ ORFDWLRQ 9HUWLFDOGLVWDQFHIURPERWWRPRIIRRWLQJWRORFDWLRQ H = 3 ft + 1 ft = 4 fft of applied lateral wind force must be multiplied by 0.6 (ASCE7-10 Section 2.4.1) to convert to service load level. MOTM = (0.6)(W)(H) = (0.6)(W)(H(3.0 kip/ft)(4 ft) = 7.2 ft-kip/ft (wind load) The resisting moment, MR, is calculated as the product of vertical load by distance from the centerline to edge of footing: MR = P(B/2) MR = (17.7 \text{ kip/ft})(7 \text{ ft/2}) = 61.8 \text{ ft-kip/ft} (WINLSIW IWNLSIW KTO ensure footing stability, the following inequality PXVWEHVDWLV¿HG MR > 1.5MOTM American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 447 Step 5: Calculate soil pressure is calculated from service forces and moments transmitted by foundation to the soil. To calculate soil pressure, the location of the vertical service resultant force is determined. The distance to the resultant from the front face of stem: e = M otm  $\Sigma P = Check$  if resultant falls within the middle third (kern) of the footing. 8.4 ft - kip = 0.47 ft 17.7 kip B/6 = 7 ft/6 = 1.17 ft > e = 0.47 ft 0K Because e''B/6, the footing imposes compression to the soil across the entire width. The resulting soil pressure must be less than the allowable bearing pressure provided by the geotechnical report.  $q_{1,2} = M \text{ OTM } \sum P \pm A \text{ S } q_{11,2} = 17.7 \text{ kip } 8.4 \text{ ft } f - \text{kip}(6) \pm (7 \text{ ft})(1 \text{ ft}) + (1 \text{ ft})(7 \text{ ft}) 2 \text{ qmax} = 22.53 \text{ ksf } sf + 11.03 \text{ 03 } \text{ ksf} = 3.56 \text{ ksf} < q_{all} = 4 \text{ ksf } 2 \text{ ksf} - 1.0 \text{ 1.03 } \text{ ksf} = 1.49 \text{ ksf} > 0 \text{ ksf } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ ksf} + 11.03 \text{ OS } \text{ ksf} = 3.56 \text{ ksf} < q_{all} = 4 \text{ ksf } 2 \text{ ksf} - 1.0 \text{ 1.03 } \text{ ksf} = 1.49 \text{ ksf} > 0 \text{ ksf } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ American Concrete Institute - Copyrighted } OK \text{ qmin} = 2.52 \text{ q$ www.concrete.org Foundationss nimu soil pressures are Maximum and minimum calculated by: 448 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Factored loads 13.3.2.1 The footing is designed as one-way slab. Calculate the soil pressures resulting from the applied factored loads. 5.3.1a 5.3.1d /RDG&DVH, U = 1.4D Use = P = 17.7 kip and MOTM = W = 12 ft-kip /RDG&DVH, U = 1.2D/A + 1.0W/S + 0.5L/A where S is the section modulus (Step 2) e = 1.0(12 ft-kip)/(7 ft) = 3.53 ksf < qu = 6 ksf OK 1.2D/A = 1.2(17.7 kip/ft)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(8.167 ft3) = 1.47 ksf 0.5L = 0 e = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf
1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.03 ksf 1.0W/S = 1.0(12 ft-kip)/(7 ft) = 3.0(12 ft-kip)/(7 ft) = 3.0(12 ft-kip)/(7 ft) = 3.0(12 ft-kip)/(7 ft) = 3.0(12 ft-kip)/(7 ft) = 3.0(12 ft kip/(1.2(17.7 kip)) = 0.56 ft < (7 ft)/6 = 1.67 ft Because e < B/6, the footing bearing pressure varies as follows (refer to Fig. E3.2):  $qu = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum)  $qu,min = 1.2(D/A) \pm 1.0(W/S)$  qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum) qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum) qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (maximum) qu,max = 3.04 ksf + 1.47 ksf = 4.51 ksf (max  $3.04 \text{ ks ksf} - 1.47 \text{ ksf} = 1.57 \text{ ksf} (\text{minimum}) = 6 \text{ ksf OK } qu, \text{max} = 4.51 \text{ ksf} < qu, \text{permitted perm er 0 D/A} = 0.9(17.7 7.7 \text{ kip/ft})/(7 \text{ kip/st}) = 0.75 \text{ ft } e = 1.0(12 \text{ q1}, 2 = 2.27 \text{ ksf} \pm 1.5 \text{ ksf} qu, \text{max} = 3.75 \text{ ksf} (\text{maximum}) < qu, \text{permitted perm er 0 D/A} = 0.9(17.7 7.7 \text{ kip/ft})/(8 (1.0\text{W/S ft-kip}))/(8$ 6 ksf 0 ksf (minimum) qu,min = 0.81 OK Fig. E3.2—Soil pressure distribution under factored loads, critical shear section, and critical moment section, and critical shear strength reduction factor: 7.5.1.1 [Vn • Vu 7.5.3.1 Vn = Vc + Vs 22.5.1.1 22.5.5.1 Vc =  $2\lambda$  f c'bw d 449 [shear = 0.75 Assume Vs = 0 (no shear reinforcement) Vn = Vc 3 20.6.1.3.1a Effective depth: d = height - cover - db/2 Assume db = 1 in. and c = 3 in. Therefore:  $\varphi Vc = \varphi 2\lambda$  f c'bw d Calculate factored soil pressure at distance d from face of wall: - qu,min ( B twall (q) qu)  $d = qu,min + |u,max|/|(2 + 2 + d|/(B d = 15 in. - 3 in. - 0.5 in. = 11.5 in. \phi Vc = 0.75(2)() 4000 \text{ psi}(12 in.)(11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf} - 1.57 \text{ ksf})(7 \text{ ft} 1 \text{ ft} 11.5 in.) = 13.1 \text{ kip/ft}(4.51 \text{ ksf})(7 \text{ ft} 11.$ 0.96 ft) = 8.3 kip/ft 2 Note: weight off footing g and earth abov above foot footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss Calculate factored ed sshear force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 22.5.1.2 &KHFNLI\_Vc•Vu = 8.3 kip/ft 2 Note: weight off footing ly not deducted from sshear ar force are conservatively created by qu. 7.4.3.2 & 0.5.1 &
0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & 0.5.1 & ce at d from face off wall: Vu 450 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Flexural strength Flexure design 13.2.7.1 The footing factored moment is calculated at the face of the wall (refer to Fig. E3.3). Fig. E3.3—Moment critical section. 5.3.1a 5.3.1d Calculate Mu at face of wall from U = 1.4D/A. (Step 6) qu = 3.53 ksf Mu = 3.53 ksf (3.5 ft - 0.5 ft) 2 = 15.9 ft - kip/ft 2 Calculate Mu at face of wall: U = 1.2D/A + 1.0W/S + 0.5L/A qu, wall = qu, min + qu, max - qu, max - qu, min + qu, max - qu, min + qu, max - qu, min + qu, max - qu, min + qu, max - qu, min + qu, max - qu, min + qu, max - qu, ma | - wall | (2 3 2 ) 5.3.1 f Mu = 2 33.25 ks (3.5 33.55 ft - 0.5 ft) 2 2 sf - 3.25 3 25 kks 4.51 ks + (3.5 ft - 0.5 ft) 2 = 18.4 ft - kip/ft 3 Controls By inspection load condition U = 0.9D/A + 1.0W/S does not control.&DOFXODWHWKHUHTXLUHGDUHDRIÀH[XUDOUHLQIRUFHPHQW Set concrete compression strength equal to steel tension strength equal to steel tension strength equal to steel tension strength 22.2.2.4.3 7.5.1.1 22.3.1.1 C=T 0.85fcgED = Asfy Mu" Mn = 0.9As(60,000 psi)(12 in. - 1.47As/2) •IW±NLSIW As • LQ2/ft American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 9.6.1.2 451 Check As against the minimum: As , min = 200 bd fy As , m > As, req'd = 0.475 in.2/ft 21.2.1(a) Check if the section is tension controlled and the XVHRI[LVFRUUHFW 21.2.2 To answer the question, the tensile strain in reinforcement is calculated from similar triangles (refer to Fig. E3.4): 22.2.1.2 Et = EC (d - C) C 1.47(0.48) in.2) = 0.83 in. 0.85 0.003 (12 in. - 0.83 in.) = 0.040 0.83 in.) = 0.040 0.83 in. It = 0.040 > 0.005 Section is tension controlled DQG[]ct = Foundationss where: c = au1 and a = 1.47As c = OK Fig. E3.4—Strain distribution through depth of footing. American Concrete Institute - Copyrighted © Material - www.concrete.org 452 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 9: Footing details 7.6.4.1 Shrinkage and temperature reinforcement:  $AS+T \cdot Ag AS+T = (0.0018)(15 \text{ in.})(7 \text{ ft})(12 \text{ in.}) = 2.27 \text{ in.} 2 \text{ Eight No.} 5 \text{ bottom longitudinal bars (area = 2.48 in.2)}$ VDWLV¿HVWKHUHTXLUHPHQWIRUVKULQNDJHDQGWHPSHUDWXUH UHLQIRUFHPHQWSODFHGSHUSHQGLFXODUWRWKHÅH[XUDO reinforcement. 13.2.7.1 25.4.2.3 25.4.2.4 Development is calculated at the maximum factored moment and the code permits the critical section to be located at the wall face. Bars must extend at least a tension development length beyond the critical section. () 3 fy  $\psi t \psi e \psi s$  | Ad = | db | 40  $\lambda$  f c' c + K tr || db |/ (3 60, 000 psi (1.0)(1.0)(1.0) Ad = | db 2.5 (40 (1.0) 4000 psi / = 28.5 28 5db = 28.5(0.875 in.) = 25 in. where ation zt = casting location; se nnot more ore than 12 in. of o fresh zt = 1.0, because ced below w horizontal rei forcem concrete is placed reinforcement cto  $z_e = coating$  factor;  $z_s = 1.0$ , because bars are uncoated or;  $z_s = 1.0$ , because bars are uncoated or;  $z_s = 1.0$ , because bars are uncoated or;  $z_s = 1.0$ , because bars are larger than No. 7 cb = spacing or cover dimension to center of bar, which ever is smaller Ktr = transverse reinforcement index. It is permitted to use Ktr = 0. cb + K tr For an event index. No. 7 bar: 25.4.2.3 But the expression: greater than 2.5. 25.4.2.1 The development length is the greater of the calculated value of Eq. (25.4.2.3) and 12 in. db must not be taken cb + K tr db = 3.44 in + 0 = 3.93 0.875 in. Use maximum value of 2.5 Ed = 25 in. 12 in. OK Check if No. 7 can be developed using straight bars, without hooks. Ed provided perpendicular to the wall: Ed,prov. = ((7 ft)(12 in./ft) - 12 in.)/2 - 3 in. > Ed,prov. = 33 in. > Ed,prov. = 33 in. > Ed,prov. = 33 in. > Ed,prov. = 33 in. > Ed,prov. = (17 ft)(12 in./ft) - 12 in.)/2 - 3 in. Ed,prov. = 33 in. > Ed,prov. = (17 ft)(12 in./ft) - 12 in.)/2 - 3 in. Ed,prov. = (17 ft)(12 in./ft) - 12 in.)/2 - 3 in. > Ed,prov. = (17 ft)(12 in./ft)(12 in./ft) - 12 in.)/2 - 3 in. > Ed,prov. = (17 ft)(12 in./ft)(12 in./ft)(12 in./ft)(12 in./ft)(12 in./ft)(12 in./ft)(12 in./ Institute – Copyrighted © Material – www.concrete.org 454 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Foundation Example 4—Design of a rectangular spread footing founded on stiff soil, supporting an 18 in. square column. The bottom of the IRRWLQJLVIWEHORZ¿QLVKHGJUDGHUHIHUWR)LJ( Given: Column load— Service dead load D = 200 kip Service live load L = 100 kip Factored wind W = ±175 kip (Axial column force due to the building frame resisting the wind load) Material properties— Concrete compressive strength fcg SVL Steel yield strength fy = 60,000 psi
1RUPDOZHLJKWFRQFUHWH3 Density of Soil = 120 lb/ft3 Density of Soil = 120 lb/ft3 Allowable service level soil bearing pressures— D only: qall,D = 4000 psf D + L + W: qall.Lat = 8000 psf Fig. E4.1—Rectangular footing plan. ACI 318-14 Procedure Computation mpu on Step 1: Foundation type 13.1.1 7KLVIRRWLQJLVIWEHORZ¿QLVKHGJUDGH7KHUHIRUH RZ¿QLVKHGJUDG 7KH it is considered a shallow footing. ow fo 13.3.3.1 The footing will bee ddesigned detailed with ned and detail ith the DSSOLFDEOHSURYLVLRQVRI&KDSWHU2QHZD\VODEV and Chapter 8, Two-way slabs, of ACI 318-14. American Concrete Institute – Copyrighted © - www.concrete.org CHAPTER 11—FOUNDATIONS Step 2: Material requirements 13.2.1.1 The mixture proportion must satisfy the durability requirements (ACI 318-14). The designer determines the durability classes. Please see Chapter 4 of this Handbook for an indepth discussion of the categories and classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLV coordinated with ACI 318-14. ACI encourages UHIHUHQFLQ]\$&,LQWRMREVSHFL¿FDWLRQV 455 By specifying that the concrete mixture shall be in accordance with ACI 318-14. ACI encourages UHIHUHQFLQ]\$ and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 4000 psi. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. area of footing  $\geq 5.3.4$ totall service load ( $\Sigma$  P) allowable all le soil pressure pressu qa Wservice = (0.6)W = (0.6)(175 kip) = 105 kip on The uni unitt weights of con concrete and soil are 150 pcf and se. Therefore, footing self-weight will gnor for initial siz be ignored sizing. Actual soil pressure is ch cked end of Step 6: checked D qall, D = (D + L) qall, D + L 200 kip ip = 5 50 ft 2 4 ksf = Foundationss Example 1 of this chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. Step 3: Determine footing dimensions oting aarea, divide the service 13.3.1.1 To calculate the footing able soil pressure. load by the allowable 200 kip + 100 kip = 51.7 ft 25.8 ksf D + L + W 200 kip + 105 kip = 50.6 ft 28 ksf qall, Lat Controls The lateral wind force must be multiplied by 0.6 (ASCE 7-10, Section 2.4.1) to convert to service load level. Assume that there is a constraint on the width of the footing (B) of 5.5 \text{ ft.} (50.6 \text{ ft} 2)/(5.5 \text{ ft}) = 9.2 \text{ ft say } 10 \text{ ft } \text{ ys } 10 \text{ ft } \text{ ys } 10 \text{ ft } \text{ ys } 10 \text{ ft } \text{ ys } 10 \text{ ft } \text{ ys } 10 \text{ ft } \text{ ys } 10 \text{ ft } 100 \text{ kip} ft The footing thickness is calculated in Step 4, footing design. Aprov. = 55 ft2 > Areg'd = 50.6 ft2 % E = (55 ft)/(10 ft) = 0.55 American Concrete Institute – Copyrighted © Material – www.concrete.org 456 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Factored soil pressure Footing stability 13.3.3.2 Because there is no out-of-plane moment, the soil pressure under the footing is assumed to be uniform and overall footing stability is assumed. 7KHIRRWLQJLVGHVLJQHGIRUÅH[XUHDVRQHZD\VODE (Step 5) and checked for two-way punching shear (Step 6). Calculate soil pressure qu =  $\Sigma$  Pu area Factored loads Calculate the soil pressures resulting from the column factored loads. 5.3.1(a) /RDG&DVH, U = 1.4(200 kip) = 280 kip qu = 5.3.1(b) /RDG&DVH, U = 1.2(1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(1) (kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(200 kip) + ((1 (1.0)(175 0)(100 kip) + 1.0(100 kip) + 1.0(100 kip) = 515 kip qu = 5.3.1(d) U = 1.2(100 kip) + ((1 (1 (1.0)(175 0)(100 kip) + 1.0(100 kip) + 1.0(100 kip) + 1.0(100 kip) 5.3.1(e) 280 kip = 5.1 ksf 55 ft 2 51 kip 515 = 9.4 ksf 555 ft 2 Controls kip 1.0(175 kip) = 355 kip U = 0.9(200 kip) + qu = 355 kip U =7KHIRRWLQJLVUHFWDQJXODULQSODQ7KHUHIRUHLWQHHGVWREHGHVLJQHGLQERWKGLUHFWLRQV2IFRXUVHWKHORQJHUGLUHFWLRQZLOO have larger moments and thus is the more critical condition. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS 457 6WHS2QHZD\VKHDUGHVLJQ Fig. E4.2—One-way shear in longer direction. /RQJGLUHFWLRQ Shear strength reduction factor: []shear = 0.75 7.5.1.1 7.5.3.1 22.5.1.1 []Vn•Vu Vn = Vc + Vs Assume Vs = 0 (no shear reinforcement) Vn = Vc + Vs the face of the column (refer to Fig. E4.2). 21.2.1(b) 13.3.1.2 g is 28 in. thick. Assume that the effooting 20.6.1.3.1 n. to bottom ttom of reinforcement. As The cover is 3 in. reinforcement. As The cover is 3 in. reinforcement. As The cover is 3 in. reinforcement. As The cover is 3 in. reinforcement. As The cover is 3 in. reinforcement. 24.625 in. say, d = 24.5 in. (A c ) Vu = | - - d | bqu (2 2 / (10 ft 18 in. 24.5 in.) (5.5 ft)(9.4 ksf) Vu = | - (2 2(12 in./ft) / (1000 lb/kip) = 153.4 kip 7.5.1.1, V\_U Vn > Vu? UV = 114 kip American Concrete Institute - Copyrighted © Material - www.concrete.org OK Foundationss The engineer assumess a valu value
of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV¿HGDQHZYDOXHRId is sele selected. 458 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Short direction Check one-way shear. The effective depth for the short direction is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV¿HGDQHZYDOXHRId is sele selected. 458 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Short direction Check one-way shear. The effective depth for the short direction is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV¿HGDQHZYDOXHRId is sele selected. 458 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Short direction Check one-way shear. The effective depth for the short direction is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV¿HGDQHZYDOXHRId is sele selected. 458 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Short direction Check one-way shear. The effective depth for the short direction is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV¿HGDQHZYDOXHRId is sele selected. 458 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Short direction Check one-way shear. The effective depth for the short direction is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV¿HGDQHZYDOXHRId is sele selected. 458 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Short direction Check one-way shear. The effective depth for the short direction is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV?HOXHRID is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (7.5.1.1) is not YDOX VDWLV?HOXHRID is a value of d then checks 2.5.5 strength by Eq. (22.5.5.1). If Eq. (23.5.5.1) is not YDOX VDWLV?HOXHRID is a value of d then checks 2.5.5 strength by Eq. (23.5.5.1). If Eq. (23.5.5.1) i  $20.6.1.3.1\ 13.2.7.2\ 7.5.1.1\ d = h - cover - db - db/2\ d = 28\ in. - 3\ in. - 0.75\ in. d = 23.875\ in., say, d = 23.5\ in.$  Therefore, assumed depth is adequate:  $h = 28\ in. (b\ c\ )\ \phi Vn \ge Vu = | - - d\ |\ Aqu\ (2\ 2\ )\ Fig.$  E4.3—One-way shear in short direction. 7.4.3.2 Distance of critical shear plane from center of footing (refer to Fig. E4.3): c 18\ in. 23.5 in. +d = + = 2.71 ft 2 2 (12 in./ft ) 12 in./ft Half of footing width: b 5.5 ft = = 2.75 ft 2 2 7KHUHIRUHRQHZD\VKHDULQWKHVKRUWGLUHFWLRQLV2. by ins inspection because the critical shear plane is at the edge of the fo footing. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS 459 Step 6: Two-way shear design 13.3.3.1 The footing will be without shear reinforcement. 13.2.7.2 Therefore, the nominal shear strength for two-way punching shear theory, the nominal shear strength is equal to the concrete strength for two-way punching shear theory. inclined cracks are assumed to originate and propagate at 45 degrees away and down from the column on all sides (refer to Fig. E4.4). ACI 318 permits the engineer to take the average of Fig. E4.4—Two-way shear. the effective depth in the two orthogonal directions 24.5 in. + 23.5 in. when designing the footing. d = 24 in. 2 bo = 4(18 + 24) = 1168 8 iin. in bo = 4(c + d) al schear strength rength is the least value Two-way nominal of (a), (b), and (c) (c): (a) vc = 4\lambda f c' vc = 4(14(1.0) (4) (b) vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) (vc = 2 + |\lambda f c' \beta/ (4) ( 4000 psi = 379.5 psi ZKHUH $\$ LVUDWLRRIWKHORQJVLGHWRVKRUWVLGHRI FROXPQ $\$  () ( $\alpha$  d) (c) vc = | s + 2 |  $\lambda$  f c' (bo / (40)(24.5 in.) vc = | + 2 | (1.0) (166 in. / 22.6.5.2 (a) Vc = 4 \lambda f c' bo d Vc = 21.2.1(b) Use a shear strength reduction factor of way shear strength is adequate. American Concrete Institute - Copyrighted © Material - www.concrete.org Foundationss 8.4.2.3.4 22.6.5.1 22.6.5.2 460 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Calculate the service-level soil pressure: B = 5.5 ft, L = 10 ft, h = 28 in. Weight of displaced soil by the footing. Weight of soil above. footing: The footing weight is added to the dead load: Wftg IW LQ NFIíNFI = 3.85 kip, say, 4 kip (36 in. -15 in.) (5.5 ft)(10 ft)(0.120 kip/ft 3) Wsoil =  $1 \cdot 12$  in./ft (1 = 4.4 kip  $D^* = 200$  kip + 4 ki Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 11—FOUNDATIONS 461 Step 7: Flexure design /RQJGLUHFWLRQ 13.2.7.1 The long direction in the rectangular footing will generate larger moments because of the column (refer to Fig. E4.5). Fig. E4.5— Flexure in the long direction. 2 (A - c) M u = qu | (B)/2 2 | 18 in. (10 ft - 12 in./ft | (5.5 ft)/2 = 467 ft-kip M u = (9.4 ksf) | 2 | 1 Set the concrete compression strength equal to the steel tension strength: C = T a = 22.2.2.4 22.2.4.3 C = 0.85 fcgba and T = Asfy and fcg SVL  $u_1 = 0.85 7.5.2.1 22.3.1.1 a$  ( $\phi M n = \phi As f y | d - 1 2 i d - 1
2 i d - 1 2 i d$ 22.2.2.4.1 As f y 0.85 f c'b = 0.28 As Substitute 0.28A As ffor a in the equation above. ove. 21.2.1(a) 8VHÀH[XUDOVWUHQJWKUHGXFWLRQIDFWRUIURP7DEOH 8.5.1.1 [a) 6HWWLQJ[]Mn•Mu and solving for As, where Mu = 467 ft-kip (0.28)As (0.9(467 ft-kip) = (0.9) As (60 ksi) | 24 in. - (2 | 8.6.1.1 Check the minimum area: As,min = (0.9) As (0.9(467 ft-kip) = (0.9) As (60 ksi) | 24 in. - (2 | 8.6.1.1 Check the minimum area: As,min = (0.9) As (0.9(467 ft-kip) = (0.9) As (0.9(46LVFRUUHFW To answer the question, the tensile strain in reinforcement is calculated and compared to the values in Table 21.2.2. The strain in reinforcement is calculated from similar triangles (refer to Fig. E4.6):  $\varepsilon = (d - c)$  where  $c = a\hat{u}1$  and a = 0.2848 c = 0.28(8)(0.6 in. 2) = 1.58 in. 0.85 st = 0.003 (24.5 in. - 1.58 in.) = 0.044 1.58 in. It = 0.044> 0.005 \$VDVVXPHGVHFWLRQLVWHQVLRQFRQWUROOHGDQG[]American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss 2 462 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E4.6—Strain distribution across footing section. American Concrete Institute www.concrete.org CHAPTER 11—FOUNDATIONS 463 Short direction Calculate moment in the short direction, at the column face. Note: the effective depth is less than that calculated for the long direction: d = 23.5 in. Fig. E4.7—Flexure in the short direction. 2 13.2.7.1 (b - c) Mu = qu | (A)/2 2 | 18 in. ) (5.5 ft - 12 in./ft | (10 ft)/2 = 188 ft-kip  $M u = (9.4 \text{ ksf}) \left[ 2 \right] \left[ \sqrt{22.2.2.4.1 \text{ Set compression force equal to tension force at the column face: } C=T C = 0.85 \text{fcgba and } T = \text{Asfy } a = 22.2.2.4.3 \text{ fcg SVL} \ u = 0.85 \text{ 7.5.2.1 a} \right] \left( \phi M n = \phi \text{As f y} \right] d - \left[ \sqrt{2} 22.3.1.1 \text{ Substitute } 0.147 \text{ A 7 s for a 7A 21.2.1(a) } Q XFWLRQIDFWRUIURP DEOH \right] d = 0.85 \text{ 7.5.2.1 a} \right] d = 0.85 \text{ 7.5.2.1 a} \left( \phi M n = \phi \text{As f y} \right) d = 0.85 \text{ 7.5.2.1 a} \right) d = 0.85 \text{ 7.5.2.1 a} d = 0.85 \text{ 7.5.2.1 a} d = 0.85 \text{ 7.5.2.1 a} d = 0.85 \text{ 7.5.2.1 a} \right) d = 0.85 \text{ 7.5.2.1 a} d = 0.85 \text{ 7.5.2.$ 8VHAH[XUDOVWUHQJWKUHGXFWLRQIDFWRUIURP7DEOH 8.5.1.1 [a) and solving lving for As: 6HWWLQJ[]Mn = Mu an 0.147As (60 ksi) 24.0 in. - (2) 22.2.2.4 As fy 0.85 f c'b = 0.147 As As = 1.75 75 in.2 8.6.1.1 13.3.3.3(b) Check minimum reinforcement area: As, min = 0.0018Ag As, min = 0.0018(10 ft)(12 in./ft) (12 in./(28 in) = 6.0 in.2 > As,req'd = 1.75 in.2 Use minimum required reinforcePHQWUsAs is distributed within a band width is equal to the length of the short side (5.5 ft). 7KHUHPDLQLQJUHLQIRUFHPHQW±UsAs) is distributed equally on both sides outside the band width. The remaining area of reinforcement must be at least the minimum reinforcement with the bars spacing not exceeding the smaller of 3h or 18 in.  $2 = 0.71 \ 10 \ \text{ft} + 15.5 \ \text{ft}$  Reinforcement with the bars spacing not exceeding the smaller of 3h or 18 in.  $2 = 0.71 \ 10 \ \text{ft} + 15.5 \ \text{ft}$  Band width = (6.0 in.2)(0.71) = 4.26 in.2 Use ten No. 6 bars distributed uniformly across the 5.5 ft band width Reinforcement area outside the central band = (6.0 in.2) - 10(0.44 in.2) = 1.6 in.2 American Concrete Institute - Copyrighted © Material - www.concrete.org Foundationss 2 464 7.6.1.1 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) The area of reinforcement outside the band width PXVWKRZHYHUVDWLVI\DWOHDVWWKHPLQLPXPÀH[ural reinforcement: As,min = 0.0018Ag As,min IWíIW LQIW LQ = 1.36 in.2 > As,req'd = 1.6 in.2/2 = 0.8 in.2 Use three No. 6 bars each side distributed uniformly outside the band width. As,min = 1.32 in.2§As,min = 1.36 in.2 OK Step 8: Column-to-footing connection 16.3.1.1 9HUWLFDOIDFWRUHGFROXPQIRUFHVDUHWUDQVIHUUHGWR the footing by bearing on concrete and the reinforcement, usually dowels. 22.8.3.2 The footing is wider on all sides than the loaded area. Therefore, the nominal bearing strength, Bn, is the lesser of the two equations. 22.8.3.2(a) Bn = (0.85 f c'A1) 22.8.3.2(b) and Bn = 2(0.85fcg\$1) Check if A2 A1 A2 A1  $\leq 2.00$  w where A2 [(5.5 ft)(12 in. in in./ft)]2 = 3.6 > 2 = A1 (18 in.) 2 Therefore, T refo Eq. (22.8.3.2(b)) controls. umn and A2 is the area of the A1 is the area of the A1 is the area of the A1 is the area of the A1 is the area of the A1 is the area of the A1 is the area of the A1 is the area of the A1 is the area of the A1 is the
area of the A1 is the A1 is the area of the A1 is the area of the A1 is the area 16.3.4.1 Provide minimum dowel area of 0.005Ag and at least four bars. This requirement is to ensure ductile behavior between the column. Bars are in compression for all load combinations. Therefore, the bars must extend into the footing a compression development length, Edc, the larger of the two and at least 8 in.: 16.3.5.4 25.4.9.2 25.3.1 A dc fyyr db | =  $\frac{1}{50\lambda}$  fc' | (0.0003 fy d) yr b [ Bn = (0.65)(2)(0.85)(4000 psi)(18 in.)2 Bn = (0.65)(2)(0.85)(4000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 in.) = 14.2 in. Controls Edc = (0.0003 in.2/lb)(60,000 psi)(0.75 i = 13.5 in. Edc = 14.2 in. (controls) > 8 in. OK The footing depth h must satisfy the following inequality so that the vertical reinforcement can be developed: h Edc + r + db, dwl + 2db, bars + 3 in. where r = radius of No. 6 bent = 6db hreq'd = 14.2 in. + 6(0.75 in.) + 0.75 in. + 2(0.75 in.) + 3 in. = 23.95 in. < hprov. = 28 in. OK American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS Assume that the column is reinforced with four No.8 bars. 25.5.5.4 25.4.9.1 25.4.9.1 25.4.9.1 25.4.9.2 As mentioned above, bars are in compression for all load cases. Therefore, the compression lap splices is the larger of the two conditions: 1. The development length, Edc, of the larger bar and 465 Compression development length for No.8 bars is: A dc = 60,000 psi 50 4000 psi (1.0 in.) = 18 in. 25.5.5.1 2. The compression lap splice of the smaller bar Esc = 0.0005(60,000 psi)(0.75 in.) = 22.5 in. Use Esc = 24 in. > 12 in. OK Therefore extend No.6 bars is: A dc = 60,000 psi (1.0 in.) = 18 in. 25.5.5.1 2. The compression lap splice of the smaller bar Esc = 0.0005(60,000 psi)(0.75 in.) = 22.5 in. Use Esc = 24 in. > 12 in. OK Therefore extend No.6 bars is: A dc = 60,000 psi (1.0 in.) = 18 in. 25.5.5.1 2. The compression lap splice of the smaller bar Esc = 0.0005(60,000 psi)(0.75 in.) = 22.5 in. Use Esc = 24 in. > 12 in. OK Therefore extend No.6 bars is: A dc = 60,000 psi (1.0 in.) = 18 in. 25.5.5.1 2. The compression lap splice of the smaller bar Esc = 0.0005(60,000 psi)(0.75 in.) = 22.5 in. Use Esc = 24 in. > 12 in. OK Therefore extend No.6 bars is: A dc = 60,000 psi (1.0 in.) = 18 in. 25.5.5.1 2. The compression lap splice of the smaller bar Esc = 0.0005(60,000 psi)(0.75 in.) = 22.5 in. Use Esc = 24 in. > 12 in. OK Therefore extend No.6 bars is: A dc = 60,000 psi (1.0 in.) = 18 in. 25.5.5.1 2. The compression lap splice of the smaller bar Esc = 0.0005(60,000 psi)(0.75 in.) = 22.5 in.bars 24 in. into the column. Step 9: Footing details Development length 13.2.8.1 the critical section. 25.4.2.4 25.4.2.1 (3 60)  $60,000\ 60,$ , psi (1.0)(1.0)(1.0) Ad = | db = 28.5db 2.5 0 (1 4000 (1.0) 0) 40 400 0 ps ppsi (40 / where VLW zt = 1.0, because not ot m more zt FDVWLQJSRVLWLRQz sh concrete crete below hor ontal than 12 in. of fresh horizontal reinforcement ze FRDWLQJIDFWRUze = 1.0, because bars are uncoated ger zs EDUVL]HIDFWRUzs = for No. 6 and larger cb = spacing or cover dimension to center of bar, whichever is smaller Ktr = transverse reinforcement index Foundationss 25.4.2.3 () 3 f y  $\psi t \psi e \psi s$  Ad = | db | 40  $\lambda$  f c' c + K tr || db |/ It is permitted to use Ktr = 0. But the expression: cb + K tr must not exceed 2.5. db No. 6: cb + K tr 3.44 in. + 0 = = 4.59 0.75 in. db use maximum value of 2.5 The development length must be the greater of the calculated value of Eq. (25.4.2.2) and 12 in. No. 6 bars: 28.5(0.75 in.) = 22 in. > 12 in. 7KHUHIRUH2. Ed in the long direction: Ed, prov. = ((10 ft)(12 in./ft) - 18 in.)/2 - 3 in. Ed, prov. = 48 in. > ld, req'd = 22 in. OK use straight No. 6 bars in long direction Ed in the short direction: No. 6 bars: 28.5(0.75 in.) = 22 in. > 12 in. 7KHUHIRUH2. Ed in the long direction Ed in the short direction: No. 6 bars in long direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short direction Ed in the short dir Ed, prov. = ((5.5 ft)(12 in./ft) - 18 in.)/2 - 2 in. Ed, prov. LQ • Id, req'd = 22 in. OK American Concrete Institute - Copyrighted © Material - www.concrete.org 466 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Detailing Fig. E4.8—Footing reinforcement ent ddetailing. Square footing If the problem was solved as sq square footing, then in Step dimensions would have been selected: tep 3, the following following following following for 3 in. x 7 ft 3 in. x 7 ft 3 in. x 7 ft 3 in. x 7 ft 3 in. x 7 ft 3 in. a ft 3 in. x 7
ft 3 in. x 7 design (10 No.6 each direction). Distribution of reinforcement within a centrall band doe does not apply to square footings. Development lengths and dowel calculations are not affected. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 467 Foundation Example 5: Design of a combined footing. Design and detail a rectangular combined footing, founded on stiff soil, supporting two building columns, oriented as shown LQ)LJ(7KHERWWRPRIWKHIRRWLQ]LVIWEHORZ¿QLVKHGJUDGH The columns only transmit axial force and neither shear nor moment is transmitted from the frame above into the footing. The soil reaction to column loads is assumed to be uniform across the footing bearing area. Given: Exterior column load— Service dead load D1 = 150 kip Service live load L1 = 100 kip c1 x c1 = 18 in. x 18 in. Material properties— Concrete compressive strength fcq = 4000 psi Steel yield strength fcq Density of concrete = 150 lb/ft3 Allowable soil bearing pressure qa = 5000 psf under all loads ACI 318-14 Procedure Calculation cula Step 1: Foundation type 13.1.1 7KLVIRRWLQJLVIWEHORZ¿QLVKHGJUDG 7KH it is considered a shallow footing. ow fo Step 2: Material requirements 13.2.1.1 The mixture proportion must satisfy the durability By specifying ying that the concrete mixture shall be in requirements of Chapter 19 and structural accordance dance with ACI 301 and providing the exposure strength requirements (ACI 318-14). The designer FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLVcHG FODVVHV&K determines the durability classes. Please refer Based on durability and strength requirements, and to Chapter 4 of this Handbook for an in-depth experience with local mixtures, the compressive discussion of the categories and classes. VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDW least 4000 psi. \$&,LVDUHIHUHQFHVSHFL¿HGDWGD\VWREHDW least 4000 psi. \$&,LVDUHIHUHQFHVSHFL? encourages referencing \$&,LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Example 1 of this chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. American Concrete Institute - Copyrighted © Material - www.concrete.org Foundationss Interior column load D2 = 260 kip Service live load L2 = 160 kip c2 x c2 = 20 in. x 20 in. 468 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 3: Determine footing dimensions Service loads 13.1.1 To calculate the footing area, assume the columns 13.3.1.1 are supported on isolated square footings. Divide the column: Areq'd = (D1 + L1)/qa Interior column: Areq'd = (D2 + L2)/qa The unit weights of concrete and soil are 150 pcf and SFIFORVH7KHUHIRUHIRRWLQJVHOI[]ZHLJKWZLOOEH checked later: Areq'd = (100 kip + 150 kip)/5 ksf = 50 ft 2 Use 7 ft 3 in. x 7 ft 3 in. x 9 needs external bracing to remain stable. This can be supplied by a moment connection between the exterior and the interior footing, but in this case, the two footings are simply combined. 13.2.6.2 ted in Step 5, TwoThe footing thickness is calculated way shear design. on of the resultant of the two Determine the location ads bby taking the moments about service column loads olumn. the center of the exterior column.  $xc = 13.3.1.1 \ 13.3.4.3 \ P1 \ x1 + P2 \ x2 \ Pi \ xc = (2600 \ kip + 160 \ l kip)(10 \ ft) = 6.3 \ ft (150 \ kip + 160 \ kip) + (260 \ kip + 160 \ l kip)(10 \ ft) = 6.3 \ ft (150 \ kip + 160 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip + 160 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip + 160 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ kip)(10 \ ft) = 6.3 \ ft (150 \ ft) = 6.3 \ ft (150 \ ft) = 6.3 \ ft (150 \ ft) = 6.3 \ ft (15$ can be assumed as uniform under the two
column loads: 2(7.3 ft) = 14.6 ft, Distribution of bearing pressure under combined footing must be consistent with the soils properties and structure. The footing must be consistent with the soils properties and structure. The footing must be consistent with the soils properties and structure. The footing must be consistent with the soils properties and structure. The footing must be consistent with the soils properties and structure. The footing must be consistent with the soils properties and structure. American Concrete Institute - Copyrighted © Material - www.concrete.org Fig. E5.2—Combined footing dimensions. Step 4: Design forces Calculate the soil press pressures resulting from the 13.2.6.1 uding footin applied factored loads including footing selfn. thick footing. footing weight. Assumee 2 ft 6 in. eig Footing self-weight:  $469 (0. \text{kip/ft3})(15)(1 \text{ ft})(9.5)(1 \text{ ft})(9.5 \text{ ft}) = 53.5 \text{ kip } W = (0.15 \text{ Se Not ollows. See Note that follows. bo combined mbined footing: (0 kip/ft p/ft3)(15 \text{ ft})(9.5 \text{ ft}) = 42.8 \text{ kip } W = (0.12 \text{ Total dead load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip } W = (0.12 \text{ Total dead load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip } + 42.8 \text{ kip deadd load} = 150 = 506.3 \text{ kip Total live load: } 15 \text{ kip } + 260 \text{ kip } + 53.5 \text{ kip }$ live load = 100 kip + 160 kip = 260 kip 5.3.1a /RDG&DVHU = 1.4D qu = 1.4(506.3 kip) = 47.3 kip/ft (15 ft) 5.3.1b /RDG&DVHU = 1.2D + 1.6L qu = 1.2(506.3 kip) + 1.6(260 kip) = 68.2 kip/ft (15 ft) 13.2.6.1 13.3.4.3 Controls Distributed soil pressure per square area below combined footing (refer to Fig. E5.3): qu = 1024 kip = 7.2 kip/ft 2 (15 ft)(9.5 ft) (15 ft) 13.2.6.1 13.3.4.3 Controls Distributed soil pressure per square area below combined footing (refer to Fig. E5.3): qu = 1024 kip = 7.2 kip/ft 2 (15 ft)(9.5 ft)(9.5 ft) (15 f ft) Note: This is a conservative approach. The footing concrete displaces soil. Therefore, the actual load on soil is the difference between the concrete Institute – Copyrighted © Material – www.concrete.org Foundationss CHAPTER 11—FOUNDATIONS 470 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E5.3—Shear and moment di diagrams. ms. nd in rior column faces, re Note: -231 kip and 295 kip aree sshear forces taken at th thee exte exterior and interior respectively. 159 ft-kip and 111 P QWWDNHQDWWKHH HULRUFROXPQI HDQ LQWHULRUF IWNLSDQGIWNLSDUHAH[XUHPRPHQWWDNHQDWWKHH[WHULRUFROXPQIDFHDQGERWKLQWHULRUFROXPQIDFHV American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 471 Step 5: Two-way shear design 13.3.4.1 Design of combined footing must satisfy the requirements of Chapter 8 for two-way slab of ACI 318-14. Check footing two-way shear strength at both columns. 22.6.1.4 22.6.4.1 Shear strength reduction factor: Exterior column is three sided. From the free body diagram, the direct shear force, Vug, is the result of the factored column force less the factored soil pressure force within the critical shear perimeter (refer to Fig. E5.4 and Fig. E5.5). Therefore, Vug = Pu1 - quA1 where A1 = (c2 + d)((c1 + d)/2 + e) d LQíLQíLQ LQ shear = 0.75 Foundationss 21.2.1b Fig. E5.4 and Fig. E5.4 in. A1 = (18 in. + 26.4 in.) + 12 in. = 1518.5 in. 2 (J 2 Assume No. 9)bars and e = 1 ft edge distance from the column centerline (refer to Fig. E5.4) 5.3.1 Solving for Vug, where Pu1 = 1.2D1 + 1.6L1 Pu1 = (1.2)(150 kip) + (1.6)(100 kip) = 340 kip Fu = quA1 Fu = Substituting into: Vug = Pu1 - quA1 Vug = 340 kip - 76 kip = 264 kip (7.2 ksf) (1518.5 in.2) = 76 kip 144 in.2 /ft 2 American Concrete Institute -Copyrighted © Material - www.concrete.org 472 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) The Code requires the footing moment at the critical shear centroid is transferred into the column by GLUHFWAH[XUHDQGE\HFFHQWULFLW\RIVKHDU Calculate the centroid axis of shear perimeter (refer to Fig. E5.4 and Fig. E5.5) where x = 2(b1) 2/2 where 2b1 + b2 c d and + 2 2 b2 = c + d b1 = e + x = 2(34.2 in.) 2/2 = 10.4 in. 2(34.2 in.) + 44.4 in. 18 in. 26.4 in. = 12 in. + From the free body diagram (refer to Fig. E5.4 and Fig. E5.5), summing the factored column load and soil pressure force about the critical sectioncentroid: 8.4.4.2.1 8.4.4.2.3 (b) Mu\* = Pu1 (b1 - e - x) - Fu | 1 - x | 2 / The maximum shear stress tress du due to direct shear and nt tra shear due to moment transfer is: vu = Vug Ac + (34.2 in.) - 76 kip | - 10.4 in. | (2 / Mu\* = 3503 3 03 in.-kip in kip y v M u\*c (Eq. q. (R8.4.4.2.3)) Jc Ac = bod; is the area ea of concrete oncrete within the critical section bo; (refer to Fig. E5.4 and Fig. E5.5). 8.4.4.2.3 M u\* = (340 kip)(34.2 in. - 12 in. - 10.4 in.) 2(34.2 in.))(26.4 in.) = 2978 in.2 Ac = (44.4 in. + 2(34 The shear perimeter moment of inertia Jc is: 2 | d3 2 b3 / b ) ] J c = 2 | b1 + d1 + (b1d) | 1 - x | + b2 dx ( / 12 2 | ] [ 12 [ (26.4 in.)3 (34.2 in.)3 J c = 2 | (34.2 in.) + 26.4 in. 12 12 | 2 ( 34.2 in.) + 26.4 in.) ] + (34.2 in.)(26.4 in.) | - 10.4 in. | (2 / I] + (44.4 in.)(26.4 in.)(10.4 in.) 2 = 488,727 in.4 8.4.2.3 Z 7KHSRUWLRQRIWKHPRPHQWLVWUDQVIHUUHGE\VKHDUUVWUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUUHGE\VKHDUVVUDQVIHUU 0.63 = 0.37 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 8.4.4.2.1 Solving for vu from Eq. (R8.4.4.2.3) above: where c = b1 - x c = 34.2 in. - 10.4 in. = 23.8 in. 22.6.5.1 22.6.5.2 473 264 kip (0.37)(3503 in. -
kip)(23.8 in.) + 2978 in.2 488, 727 in.4 = 0.089 ksi + 0.063 ksi = 0.152 ksi vu = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0.063 ksi = 0.089 ksi + 0. 7ZRZD\VKHDUVWUHQJWKHTXDWLRQVPXVWEHVDWLV¿HG and the least value controls:  $\left(\alpha \text{ s d}\right) + 2 \left| \left| \left| \right| \right| \right|$  bo  $\left| \left( 4 \right) \left| \right| \left| \left| \left| 4 \right| \right| \right| \left| \left| 1 \right| \right|$  is = 30, edge column 22.6.5.2  $\varphi vc = \varphi 4\lambda$  f c' 21.2.1 Shear strength reduction factor: 0.75 2+ 4 4 = 2+ = 6 > 4 \beta 1 Therefore, 4 NG NG Controls  $\varphi vc = (0.75)(4)$ (1) 4000 psi = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi |vc| = 190 psi THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Interior column: The maximum factored shear force at the critical section is equal to the factored soil pressure within the critical section (refer to Fig. E5.6): 22.6.1.4 Vu = Pu2 - qu(c2 + d)(c2 + d) 5.3.1 where Pu2 = 1.2D2 + 1.6L2 Vu = (1.2)(260 kip) + 1.6 strength Eq. (22.6.5.2(a)) controls: bo = (4)(20 in. + 26.4 in.) = 185.6 in. Check if the f c' factors are less than 4. 4 is used if the other factors are less than 4. 4 is (0.75)(4)(1.0) (1.0) (1.0) (1.0) (1.0) (1.0) (1.0) (1.0) (1.0) (1.0) (20 in + 26.4 in)(20.4)(20 in + 26.4 in)(26.4 in) = 929.5 kip, 930 kip ip, say, 938.5.1.1 & KHFNLI (Vc = 930 k ip)(4)(20 in + 26.4 in)(26.4 in) = 929.5 kip, 930 kip ip, say, 938.5.1.1 & KHFNLI (Vc = 930 k ip)(4)(20 in + 26.4 in)(26.4 in) = 929.5 kip, 930 kip ip, say, 938.5.1.1 & KHFNLI  $(Vc = 930 \text{ k ip})(4)(20 \text{ in})(4)(20 \text{
in})(4)(20 \text{ i$ FOUNDATIONS 475 6WHS2QHZD/VKHDUGHVLJQ 2QHZD/VKHDUGHVLJQ 13.3.2.1 2QHZD/VKHDUVWUHQJWKLVFDOFXODWHGDWDGLVWDQFH 13.2.7.2 d from the interior column face (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is permitted to be calculated. &KHFNLI[]Vc with d = 26.4 in. exceeds Vu = 295 kip (refer to Fig. E5.7) 7.4.3.2 where the maximum shear force is pe E5.3(a)). 7.4.1.1 7.4.3.2 Calculate required strength from column factored loads less soil pressure Vu"[email protected] – qu(c2/2 + d) (26.4 in.) Vu = 295 kip - 68.2 kip/ft = 145 kip (12 in./ft ) Calculate shear strength and verify that it exceeds the calculated required strength from column factored loads less soil pressure Vu"[email protected] – <math>qu(c2/2 + d) (26.4 in.) Vu = 295 kip - 68.2 kip/ft = 145 kip (12 in./ft ) Calculate shear strength and verify that it exceeds the calculated required strength from column factored loads less soil pressure Vu"[email protected] – <math>qu(c2/2 + d) (26.4 in.) Vu = 295 kip - 68.2 kip/ft = 145 kip (12 in./ft ) Calculate shear strength and verify that it exceeds the calculated required strength from column factored loads less soil pressure Vu"[email protected] – <math>qu(c2/2 + d) (26.4 in.) Vu = 295 kip - 68.2 kip/ft = 145 kip (12 in./ft ) Calculate shear strength and verify that it exceeds the calculated required strength from column factored loads less soil pressure Vu"[email protected] – <math>qu(c2/2 + d) (26.4 in.) Vu = 295 kip - 68.2 kip/ft = 145 kip (12 in./ft ) Calculate shear strength and verify that it exceeds the calculated required strength from column factored loads less soil pressure Vu"[email protected] – <math>qu(c2/2 + d) (26.4 in.) Vu = 295 kip - 68.2 kip/ft = 145 kip (12 in./ft ) Calculate shear strength and verify that it exceeds the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculated required strength for the calculatShear strength is calculated ed from:  $QG[\phi Vc = \phi 2 f c'bw d DQG7.5.1.1 [Vc Check if Vu]V Vs = 0$  (foundations are usually sized such that shear reinforced is not required). Therefore,  $Vn = Vc \phi Vc = (0.75)(2)$  () 40 4000 0 pps psi (9.5 ft)(12 in./ft)(26.4 in.) = 2285.5 5 kip Vu NLSa[V [V Vc = 285.5 kip/2 = 143 kip Foundationss refo shear reinforced is not required). Therefore, reinforcement is not required. Fig. E5.7—One-way shear. 6XPPDU\7KHFRPELQHGIRRWLQJWKLFNQHVVRILQVDWLV¿HVERWKRQHZD\DQGWZRZD\VKHDUUHLQIRUFHPHQW American Concrete Institute – Copyrighted © Material – www.concrete.org 476 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Flexure design &DOFXODWHWKHÀH[XUDOUHLQIRUFHPHQWLQWKH FRPELQHGIRRWLQ] /RQ]LWXGLQDOGLUHFWLRQ 1RWHWKDWAH[XUDOWHQVLRQRFFXUVDWWKHWRSRIWKH IRRWLQ]EHWZHHQWKHWZRFROXPQVDQGDWWKHERWWRP RIWKHIRRWLQ]DWERWKLQWHULRUDQGH[WHULRUFROXPQV )L](E IPHQWEHWZHHQFROXPQV \$WWKHVHFWLRQRIPD[LPXPPRPHQWVHWWKHLQWHUQDO FRPSUHVVLRQIRUFHHTXDOWRLQWHUQDOWHQVLRQIRUFHPHQWDUHD 22.2.2.4 C=T 22.2.4.1 22.2.3.1 0.85fcg ba = Asfy 22.3.1.1 SVL IW LQIW a = AsSVL a = 0.155As URPPRPHQWGLDJUDP)LJ(E Mu"]Mn []Asfy(d - a Mu IWNLS 21.2.1a XFWLRQIDFWRUIURP7DEOH 8VHÀH[XUDOVWUHQJWKUHGXFWLRQIDFWRUIURP7DEOH 21.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MuDQ DQGVXEVWLWXWHIRUaLQWKHTXDWXWH HTXD 6HWWLQJ]Mn•M WLRQDERYH / in.-kip 0.155 As \ in. kip p / IWNLS IWN | ≥ As NVL | LQ - ( IW NLS | 1.2.1 9.5.1.1a MUDQ DQGVXEVWLWXWHIRUaLQWKHTXDWXWH IWNLS IWN | 2.2.1 9.5.1.1a MUDQ MUDQ WXWHIRUALQWKHTXDWXWH IWN | 3.5.1.1a MUDQ MUDQ MUQAQWX | 3.5.1.1a MUDQ MUQAQWXWH | 3.5.1.1a MUDQ MUQAQWX | 3.5.1.1a MUDQ MUQAQWX | 3.5.1.1a MUDQ MUQAQWX | 3.5.1.1a MUDQ MUQAQWX | (IW)(LQIW) (LQ = LQ2 SVL 200 bd E fy As min = 3 f c' D As mi min = 3 40 4000 psi (IW)(LQIW) (LQ = LQ2 SVL 200 bd E fy As min = 200 (IW)(LQIW) (LQ = LQ2 SVL fy bd As, min = 3 40 4000 psi (IW)(LQIW) (LQ = LQ2 SVL 200 bd E fy As min = 3 40 4000 p FRQWUROVEHFDXVHFRQFUHWHFRPSUHVVLYHVWUHQJWKfc9LVOHVVWKDQSVL 7KHUHIRUHPLQLPXPUHLQIRUFHPHQWFRQWUROV 8VH1RWRSFRQWLQXRXVDQGHYHQO\GLVWULEXWHG RYHUWKHZLGWKRIWKHIRRWLQJ As, prov. LQ2 LQ2 22.2.2.4.1 22.2.2.4.1 22.2.2.4.3 1) ( &KHFNLIVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKHXVHR a = | LQ2 = LQ2 \ / in.
[LVFRUUHFW 21.2.2 c = a β1 c = 1.55 in. = 1.82 in. 0.85 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 477 Calculate the strain in the tension reinforcement and compare to the minimum strain required for tension-controlled section: (d - c) ε t = 0.003 | \ c | / (  $26.4 \text{ in.} - 1.82 \text{ in.} \epsilon t = 0.003 | l| = 0.0405 > 0.005 (1.82 \text{ in.} Place bars such that the spacing between them does Therefore, section is tension controlled. not exceed 3h or 18 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH¿UVWEDUSODFHGDWLQIURPWKHHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH²UVWEDUSODFHGDWLQIURPWKHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH²UVWEDUSODFHGDWLQIURPWKHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH²UVWEDUSODFHGDWLQIURPWKHGJH Foundationss 7.7.2.3 American Concrete Institute – Copyrighted © 1.82 in. 30DFH²UVWEDUSODFHG$ - www.concrete.org 478 13.2.7.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Reinforcement at interior column: Mu = 216 ft-kip (Fig. E5.3(b)) 22.2.2.4 At the section of maximum moment, set the internal compression force equal to internal tension force to Solve for As As  $LQ \cdot LQ 2$  This is less than n th the minimum mum reinforc reinforcement ent area ve. calculated above. Therefore, refo use As,min = 110.0 in.2 > As,req,d = 1.83 in.2 9.6.1.2 0.85(4000 psi)(9.5 ft)(12 in./ft)a = As60,000 psi a = 0.155As Use ten No. 9 continuous ontinuous ontinuous bottom bars evenly di ribu overr the widt distributed width of the dist footing. 1.0 in.2) = 10 in.2 As = (10)(1.0 1)(a = 0.00.155 155)(10 in.2) = 1.55 in.2 115 (in.) Check if section is tension controlled and the use of [LVFRUUHFW 22.2.2.4.3 c = a  $\beta 1 c = 1.55$  in. = 1.82 in. 0.85 Calculate the strain in the tension reinforcement and compare to the minimum strain required for tension-controlled section:  $21.2.2 (d - c) \epsilon t = 0.003 | c | / (26.4 in. - 1.82 in.) \epsilon t = 0.003 | l = 0.0405 > 0.005 (1.82 in. Therefore, section is tension controlled. Note: The calculated factored interior column face (159 ft-kip) and the exterior column face (15$ column face (216 ft-kip). Therefore, minimum reinforcement area controls. Provide 10 No. 9 bottom bars over full length of combined footing and spaced at 12 in. on center < 3h = 90 in. and 18 in. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS 479 Transverse reinforcement In a combined footing, transverse moment distribution may be addressed similar to an isolated spread footing. A strip over the width of the footing itself. Darwin et al. (2015) and Fanella (2011) recommend the width of the strip to be half the effective depth (d/2) on either side of the footing from the face of columns. Calculate d to the center of the second layer. d = 30 in. -3 in. -1.128 in. -1.12distributed soil reaction is:  $qu^*,TI = 13.2.7.1\ 1.2(260\ kip) + 1.6(160\ kip) = 59.8\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\
6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ ft\ 8.5\ say,\ 6\ 60\ kip/ft\ 9.5\ say,\ 8.5\ say,\$ Fig. E5.8. 8.5.1.1 Calculate required reinforcement:  $\Box$ Mn  $\exists$ fyAsjd•Mu 460 ft-kip = 0.9As(60,000 psi)(0.9)(25.3 in.) As = 4.5 in.2 & RHI¿FLHQWRQd: j = 0.9 21.2.1 Flexural strength reduction factor: 9.6.1.2 Check if the minimum reinforcement area controls: As , min = 200 bd fy  $\exists$ As , min = 200 (45.3 in.) (25.3 in.) = 3.8 in.2 /ft 60,000 psi)(0.9)(25.3 in.) As = 4.5 in.2 & RHI¿FLHQWRQd: j = 0.9 21.2.1 Flexural strength reduction factor: 9.6.1.2 Check if the minimum reinforcement area controls: As , min = 200 bd fy  $\exists$ As , min = 200 (45.3 in.) (25.3 in.) = 3.8 in.2 /ft 60,000 psi)(0.9)(25.3 in.) As = 4.5 in.2 & RHI¿FLHQWRQd: j = 0.9 21.2.1 Flexural strength reduction factor: 9.6.1.2 Check if the minimum reinforcement area controls: As , min = 200 bd fy  $\exists$ As , min = 200 bd fy data bases at the factor at controls because concrete compressive strength fcgis less than 4444 psi. Required reinforcement is greater than the minimum required. Therefore use eight No. 7 spaced at 6 in. on center and placed within the calculated width 46.4 in. Check if section is tension controlled and the use of As, prov = (8)(0.6 in.2) = 4.8 in.2 > As, req'd = 4.5 in.2 | LVFRUUHFW 22.2.2.4.1 22.2.2.4.3 c= a  $\beta$ 1 a= (0.9)(60,000 psi)(4.8 in.2) = 1.68 in. 0.85(4000 psi)(45.3 in.) c= 1.68 in. = 1.98 in. 0.85 American Concrete Institute – Copyrighted © Material – www.concrete.org 480 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Calculate the strain in the tension reinforcement and compare to the minimum strain required for tension-controlled section:  $21.2.2 (d - c) \epsilon t = 0.003 | c | / (26.4 in. 1.98 in.) \epsilon = 0.003 | l = 0.035 > 0.005 (1.98 in. Therefore, section is tension controlled. Exterior column The factored co$ distributed soil reaction is:  $qu^{TI} = 13.2.7.1 d = 25.3 in. w = 12 in. + 18 in./2 + 25.3 in./2 = 33.7 in.$  Pu , int B  $qu^{*}$ , TI = 1.2(150 kip) + 1.6(100 kip) = 35.8 kip/ft)(4 ft)2 = 286 ft-kip 2 The factored moment at the column face is: Mu = 1 (35.8 kip/ft)(4 ft)2 = 286 ft-kip 2 The factored moment at the column face is: Mu = 1 (35.8 kip/ft)(4 ft)2 = 286 ft-kip 2 The factored moment at the column face is: Mu = 1 \* (bc) qu = 1 ( $2 \sqrt{22}/2bc$  = 0.5 ft 18 in. - = - = 4 ft 2 2 2 (12 in./ft) Refer to Fig. E5.8 8.5.1.1 red reinforcement: orcement: Calculate required  $[]Mn \cdot Mu$  []fyAsjjd d: j = 0.99 & RHi & FLHQWRQd: 21.2.1 on factor: Flexural strength red reduction 9.6.1.2 Check if the minimum reinforcement area controls: 286 ft-k ft-kip = 0.9A As(60,00 (60,000 psi)(0.9)(25.3 in.) As = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) 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(25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 200 bd fy As , min = 200 (33.7 in.) (25.3 in.) = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8 in.2 []As , min = 2.8in.2 /ft 60,000 psi Minimum reinforcement is equal to the calculated reTXLUHGUHLQIRUFHPHQW7KHUHIRUHXVH¿YH1RVSDFHG at 10 in. on center. As, prov = (5)(0.6 in.2) = 3.0 in.2 > As, req'd = 2.8 in.2 Check if section is tension controlled and the use of [LVFRUUHFW a \$1 22.2.2.4.1 c = 22.2.2.4.3 Calculate the strain in the tension reinforcement and compare to the minimum strain required for tension-controlled section:  $21.2.2 (d - c) \epsilon t = 0.003 | c | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} -
1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) \epsilon t = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) c = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 1.41 \text{ in.} = 1.66 \text{ in.} 0.85 (25.3 \text{ in.} - 1.66 \text{ in.}) c = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 0.003 | | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 0.003 | a = (0.9)(60,000 \text{ psi})(3.7 \text{ in.}) c = 0.003 | a = (0.9)(60,000 \text{ psi})($ Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS For sections outside the effective width at the exterior and interior columns, provide minimum reinforcement area. As , min = 481 200 (12 in.) (25.3 in.) = 1 in.2 /ft 60,000 psi Use No. 7 at 7 in. on center < 3h = 90 in. or 18 in. OK Foundationss Fig. E5.8—Footing width at columns for transverse reinforcement calculations. American Concrete Institute - Copyrighted © Material - www.concrete.org 482 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Footing details Development length of No. 9 top bars From the moment diagram in Fig. E5.3(b), the SRVLWLYHPRPHQWLQAHFWLRQSRLQWVDWWKHH[WHULRUDQG interior columns occur at 0.12 ft and 0.45 ft from the respective column centerlines (Fig. E5.9). Therefore, extend top bars to the edge of the footing. &KHFNLIWKHDYDLODEOHGLVWDQFHLVVXI&FLHQWWR develop the top bars at midspan in tension. The development length for No. 9 bar is calculated XVLOIDVLPSOL¿HGHTXDWLRQDVDOORZHGE\\$&, code rather than the more detailed Eq. (25.4.2.3a): Fig. E5.9—Longitudinal reinforcement of combined footing. 25.4.2.2 (casting position) d below more than 12 in. of fresh concrete is pla placed horizontal bars an and or = 1.0 0 because bars aaree unc uncoated ze (coating factor) 25.4.1.4 Check if 25.4.2.1 Check if development is less than 12 in. les than an 100 psi f c' is less ( (60, 000 psi)(1.3)(1.0) Ad = | | db = 61.7db = 69.6 in. (20)(1.0) 4000 psi | (20)(1 (2 Use 70 in. i = 5 ft 10 in. 4000 000 psi = 63.2 psi < 100 psi Ed = 69.6 in. = 69.6 in 5.8 ft > 12 in. 15 ft - 5.35 ft = 9.65 ft > 5.8 ft OK OK Enough distance is available length from maximum moment at midspan to the exterior column is less than the calculated development length,  $\mathcal{E}d = 5.8$  ft. 25.4.3.1 Therefore, a hook is required at the exterior support and must be the greater of: (f yweycyr) a. A dh = | db \ 50 \lambda f c' b. 8db c. 6 in. where: ze (coating factor) = 1.0 because bars are uncoated zc (cover) = 0.7 for No.9 bars with 3 in. side cover and 3 in. cover for the 90-degree hook zrFRQiQHPHQW EDUVDUHQRWFRQiQHG ( (60,000 psi)(1.0)(0.7)(1.0) A dh = | (1.128 in.) 50\lambda 4000 psi ( ) Edh = 14.98 in., say, 15 in. Controls 8(1.128 in.) = 9 in. 6 in. Therefore, No. 9 top straight bars can be placed full length and will be developed at the point of maximum moment at the interior column and 90-degree hook at the exterior column. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 483 Development of bottom bars /RQJLWXGLQDOEDUV1R The factored moment at the exterior column is negligible, 2 ft-kip at face of column, 32 ft-kip at column centerline (refer to Fig. E5.3(b)). Calculate the development length at the interior column Mu = 279 ft-kip at exterior face (Fig. E5.3(b)): 25.4.2.4 25.4.2.1 (f ywt we) Ad = | db \ 20 \lambda fc' | where zt FDVWLOJORFDWLROzt = 1.0, because bars are uncoated Check if development is less than 12 in. (60, 000 psi)(1.0)(1.0) Ad = | db = 47.4db (20)(1.0) 4000 psi / Bar size No. 7 Ed, in. 53.5 41.5 Use, in. 54 42 Both required development length exceeds 12 in. Therefore, OK Interior column: m footing edge, which Column is located 4 ft from ed calcu is less than the required development length of 54 in. = 4 ft 6 in. Therefore, OK Interior column: m footing edge, which Column is located 4 ft from ed calcu is less than the required development length of 54 in. = 4 ft 6 in. Therefore, OK Interior column: m footing edge, which Column is located 4 ft from ed calcu is less than the required development length of 54 in. = 4 ft 6 in. Therefore, OK Interior column: m footing edge, which Column is located 4 ft from ed calcu is less than the required development length of 54 in. = 4 ft 6 in. Therefore, OK Interior column: m footing edge, which Column is located 4 ft from ed calcu is less than the required development length of 54 in. = 4 ft 6 in. provide a om No. 9 bars at the interior colhook for the bottom 5 ft - 11. 11.83 ft = 3.17 ft OK Edh = 15 in. < 15 rio calculations ulations for top bars. rs. umn. Refer to prior ment length Eq. (25.4.2.3a) 5.4. d, th Note: That if thee more detailed develop development is used, then adequate distance is availing to bend them able to place thee N No. 9 bars without hav having them. Transverse reinforcement: or nt: ang at the interior int col From Fig. E5.8, thee overhang column is 3.92 ft = 47 in., which is greater than the required calculated development length = 42 in. Therefore, No.7 bars are placed straight. American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss 25.4.2.2 484 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 9: Column-to-footing connection Interior column forces are transferred to the footing is wider on all sides than the loaded area. Therefore, the nominal bearing strength, Bn, is the smaller of the two equations. (a) Bn = (0.85 f c'A1) A2 A1 and (b) Bn = 2(0.85fcqA1) A1 is the bearing area of the column and A2 is the area of
the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of the column and A2 is the area of combined footing long sides. 3.16 ft //2 = 1.58 ft = 19 in. < 30 in. footing thickness The sides of the pyramid tapered wedges are sloped A2 = [2(10 in. + (3.16 ft)(12 in./ft))]2 = 9063 in.2 4.7 > 2 = 4.76 A1 (20 in.) n.) 2 T refo Eq. (22.8.3.2 (b)) controls. 2(0.85 fcg A1) Bn = 2(0.21.2.1 The bearing strength reduction factor is 0.65: [bearing ringg = 0.65 [Bn = (0.65)(2)(0.85)(4000 psi)(20 in.)2 [Bn = 1768 kip > 1.2D + 1.6L = 600 kip 16.3.4.1 16.3.5.4 Column factored forces are transferred to the footing by bearing and through dowels. The minimum dowel area is 0.005 Ag and at least four bars across the interface between interior column and combined footing. The four No. 7 dowels must be developed in the footing depth. As, dowel = 2.0 in. 2 Bars are in compression for all load combinations. Therefore, the bars must extend into the footing at least a compression development length Edc, which is the larger of the following two expressions: 25.4.9.2 A dc  $\int fy\psi r db = \frac{1}{50\lambda} fc' = \frac{(0.0003)(60,000 \text{ psi})(1.0)}{(0.875 \text{ in.})} = 15.75 \text{ in.}$  American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 11—FOUNDATIONS 25.4.9.3 20.6.1.3.2 zr FRQ¿QLQJUHLQIRUFHPHQWIDFWRU zr EHFDXVHUHLQIRUFHPHQWLVQRWFRQ¿QHG The footing depth must satisfy the following inequality: h•Edc + r + db,dwl + db,#7 + db,#9 + 3 in. 485 hreq'd = 16.6 in. + 6(0.875 in.) + 0.875 in. + 0.875 in. + 1.128 in. + 3 in. = 27.728 in., say, 28 in. hreq'd = 28.0 in. < hprov. LQ2. 3 in. cover (refer to Fig. E5.10) Check development length of dowel s in the column is the greater of the development length. Assume that the column is reinforced with six No. 8 bars. db,dowel < db,column Therefore, the lap splice length must bbe the greater of a) and b): er of: a. No. 8 bars is the larger  $\int fy \psi r db = 18.0$  in. where zrFRQ¿QLQJUHLQIRUFHPHQWIDFWRU zr = 1.0, because stirrup spacing is greater than 4 in. (condition 3) b. The compression lap splice length of No. 7 is the larger of  $\left\{ \begin{array}{c} 0.0005 \text{ (fy db } \text{Esc} = \text{larger of } \\ 0.0005 \text{ (fy db } \text{Esc} = \text{larger of } \\ 0.0005 \text{ (fo } 0.000 \text{ psi})(0.875 \text{ in.}) = 26.25 \text{ in.} 12 \text{ in.} \text{ Use } 27 \text{ in.} = 2 \text{ ft } 3 \text{ in.} \text{ long lap splice. American Concrete Institute - Copyrighted © Material Concrete Institute - Copyrighted © Concrete Institute - Copyrighted © Concrete Institute - Copyrighted © Concrete Institute - Copyrighted © Concrete Institute - Copyrighted © Concrete Insti$ - www.concrete.org Foundationss 25.4.9.2 486 16.3.1.1 22.8.3.2 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Exterior column The column The column The column The column factored forces are transferred to the footing is wider on all sides than the loaded area. Therefore, the nominal bearing strength, Bn, is the smaller of the following two equations. (a) Bn = (0.85 f c'A1) A2 A1 and (b) Bn = 2(0.85 f c'A1) A2 is the area of the column and A2 is the sides. 1 ft /2 = 0.5 ft = 6 in. < 30 in. footing thickness The sides of the pyramid tapered wedges are sloped A2 = [2(9 in. + 3 in.)]2 = 576 in.2 = 1.33 13 340 kip [] The four No.6 dowels must be developed in the footing. As,dowel = 0.005(18 in.)2 = 1.62 in.2 The bars are in compression for all load combinations. Therefore, the bars must extend into the footing a compression development length Edc, which is the larger of the two following expressions: A dc  $\int fy \psi d y r b \left( (0.65)(0.8 \text{ psi})(18 \text{ in.}) 2 \phi Bn = (0.65)(0.85)(4000 576 \text{ in.}2 (18 \text{ in.}) 2 \phi Bn = (0.65)(0.85)(400 576 \text{ in.}2 (18 \text{ in.}) 2 \phi Bn = (0.65)(0.85)(18 \theta h) 2 \phi Bn = (0.65)(0.85)(18 \theta h) 2 \phi Bn = (0.65)(0.85)(18 \theta h) 2 \phi Bn = (0.65)(0.85)(18$ column. As =  $(4)(0.44 \text{ in.2}) = 1.76 \text{ in.2} > \text{As,dowel} = 1.62 \text{ in.2} OK A dc = (60,000 \text{ psi})(1.0) (50) 4000 \text{ psi}(0.75 \text{ in.}) = 14.2 \text{ in. Controls } \mathcal{E}dc = 0.0003(60,000 \text{ psi})(1.0)(0.75 \text{ in.}) = 13.5 \text{ in. where } zr FRQ2QLQJUHLQIRUFHPHQWIDFWRU zr EHFDXVHUHLQIRUFHPHQWLVQRWFRQ2QHG The footing depth h
must satisfy the following inequality: h*Edc + r$ + db,dwl + db,#7 + db,#9 + 3 in. 20.6.1.3 hreq'd = 14.2 in. + 6(0.75 in.) + 0.75 in. + 0.875 in. + 3 in. = 24.5 in., say, 25 in. hreq'd = 25 in. < hftq.prov. = 30 in. OK 3 in. cover (refer to Fig. E5.10) American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 11—FOUNDATIONS 487 Check development length of dowel reinforcement into the column. The length of dowels in the column is reinforced with six No. 8 bars. db,dowel < db,column 25.4.9.2 Therefore, the lap splice length must be at least equal to the larger of (a) and (b): (a) For column bars, at least the larger of: A dc 25.5.5.1 [fyyr db] =  $\frac{50\lambda fc'}{(0.0003 f \psi d) yr b}$  (60, 000 psi)(1.0) (50)(1.0) 4000 psi (0.75 in.) = 14.3 in. Controls 0.0003(60,000 psi)(0.75 in.) = 13.5 in. where r FRQ2QLQJUHLQIRUFHPHQWIDFWRU r EHFDXVHUHLQIRUFHPHQWIVQRWFRQ2QHG plice length of dowel (b) The compression lap splice rger of: must be at least the larger 0.0005(60,000 005( 00 psi)(0.75 in.) = 22.5 in. Controls 12 1 in. n. U 24 in. long lap splice. Step 10: Details Fig. E5.11—Combined footing dimensions and reinforcement. References Darwin, D.; Dolan, C., Nilson, A., eds., 2015, Design of Concrete Structures, McGraw-Hill Professional Publishing, 15th edition, New York, 786 pp. Fanella, D., ed., 2011, 5HLQIRUFHG&RQFUHWH6WUXFWXUHV\$QDO\VLVDQG'HVLJQ0F\*UDZ+LOO3URIHVVLRQDO3XEOLVKLQJ¿UVW edition, New York, 615 pp. American Concrete Institute – Copyrighted © Material – www.concrete.org Foundationss [] 0.00 0.0005 f y db Esc = larger of { [] 12 in. 488 THE REINFORCED CONCRETE DESIGN HANDBOOK -SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 9 781942 727378 An ACI Handbook The Reinforced Concrete Design Handbook A Companion to ACI 318-14 Volume 2: Special Topics SP-17(14) ACI SP-17(14) THE REINFORCED CONCRETE DESIGN HANDBOOK A Companion to ACI 318-14 VOLUME 2 BUILDING EXAMPLE RETAINING WALLS STRUCTURAL ANALYSIS STRUCTURAL ANALYSIS STRUCTURAL ANALYSIS STRUCTURAL SYSTEMS SERVICEABILITY STRUCTURAL ANALYSIS STRUCTUR COLUMNS STRUCTURAL REINFORCED CONCRETE WALLS FOUNDATIONS ACI SP-17(14) Volume 2 THE REINFORCED CONCRETE DESIGN HANDBOOK A Companion to ACI 318-14 Editors: Andrew Taylor Trey Hamilton III Antonio Nanni First Printing September 2015 ISBN: 978-1-942727-3- Errata as of September 2, 2016 THE REINFORCED CONCRETE DESIGN HANDBOOK Volume ~ Ninth Edition Copyright by the American Concrete Institute, Farmington Hills, MI. 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Fergusson Manager, Publishing Services; Barry Bergin Lead Production Editor; Carl Bischof Production Editors; Kelli Slavden, Kaitlyn Hinman, Tiesha Elam Graphic Designers; Ryan Jay, Aimee Kahaian Manufacturing; Marie Fuller www.concrete.org DEDICATION This edition of The Reinforced Concrete Design Handbook, SP-17(14), is dedicated to the memory of Daniel W. Falconer and his many contributions to the concrete industry. He was Managing Director of Engineering for the American Concrete Institute from 1998 until his death in July 2015. Dan was instrumental in the reorganization of Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14) as he served as ACI staff liaison to ACI Committee 318, Structural Concrete Building Code; and ACI Subcommittee 318-SC, Steering Committee 318-SC, Steering review comments were instrumental in the development of this Handbook. An ACI member since 1982, Dan served on ACI Committees 344, Circular Concrete Structures, and 373, Circular Concrete Structures, and 373, Circular Concrete Structures, and 374, Circular Concrete Structures, and 374, Circular Concrete Structures, and 374, Circular Concrete Structures, and 374, Circular Concrete Structures, and 374, Circular Concrete Structures, and 374, Circular Concrete
Structures, and 374, Circular Concrete Structures, and 374, Circular Concrete Structures, and 375, Circ MRLQLQJ\$&,'DQKHOGVHYHUDOHQJLQHHULQJDQGPDUNHWLQJSRVLWLRQVZLWK96/&RUS%HIRUH WKDWKHZDV3URMHFW(QJLQHHUIRU6NLGPRUH2ZLQJVDQG0HUULOOLQ:DVKLQJWRQ'&+H received his BS in civil engineering from the University of Buffalo, Buffalo, NY and his 06LQFLYLODQGVWUXFWXUDOHQJLQHHULQJIURP/HKLJK8QLYHUVLW\%HWKOHKHP3\$+HZDVD licensed professional engineer in several states. ,QKLVSHUVRQDOOLIH'DQZDVDQDYLGJROIHUHQMR\LQJRXWLQJVZLWKKLVWKUHHEURWKHUVZKHQHYHUSRVVLEOH+HZDVDOVRDQDFWLYH PHPEHURI2XU6DYLRU/XWKHUDQ&KXUFKLQ+DUWODQG0,DQGDGHGLFDWHGVXSSRUWHUDQGIROORZHURIWKH0LFKLJDQ6WDWH6SDUtans basketball and football programs. Above all, Dan was known as a devoted family man dedicated to his wife of 33 years, Barbara, his children Mark, Elizabeth, Kathryn, and Jonathan, and two grandsons

Samuel and Jacob. In his memory, the ACI Foundation has established an educational memorial. For more information visit . Dan will be sorely missed for many years to come. FOREWORD The Reinforced Concrete Design Handbook provides assistance to professionals engaged in the design of reinforced concrete Design Handbook provides assistance to professional memorial. EXLOGLQJVDQGUHODWHGVWUXFWXUHV7KLVHGLWLRQVRI Building Code Requirements for Structural Concrete (ACI 318-14). The layout and look of the Handbook have also been updated. The Reinforced Concrete Design Handbook now provides dozens of design examples of various reinforced concrete members, such as one- and two-way slabs, beams, columns, walls, diaphragms, footings, and retaining walls. For consistency, many of the QXPHULFDOH[DPSOHVDUHEDVHGRQD¿FWLWLRXVVHYHQVWRU\UHLQIRUFHGFRQFUHWHEXLOGLQ]7KHUHDUHDOVRPDQ\DGGLWLRQDOGHVLJQ examples not related to the design of the members in the seven story building that illustrate various ACI 318-14 requirements. Each example starts with a problem statement, then provides a design solution in a three column format—code provision UHIHUHQFHVKRUWGLVFXVVLRQDQGGHVLJQFDOFXODWLRQV2IROORZHGE\DGUDZLQJRIUHLQIRUFLQJGHWDLOVDQG¿QDOO\DFRQFOXVLRQ elaborating on a certain condition or comparing results of similar problem solutions. In addition to examples, almost all chapters in the Reinforced Concrete Design Handbook contain a general discussion of the related ACI 318-14 chapters. All chapters were developed by ACI staff engineers under the auspices of the ACI Technical Activities Committee (TAC). 7R SURYLGH LPPHGLDWH RYHUVLJKW DQG JXLGDQFH IRU WKLV SURMHFW 7\$& DSSRLQWHG WKUHH FRQWHQW HGLWRUV\$QGUHZ 7D\ORU 7UH\ Hamilton III, and Antonio Nanni. Their reviews and suggestions improved this publication and are appreciated. TAC also appreciates the support of Dirk Bondy who provided free software to analyze and design the post-tensioned EHDP H[DPSOH LQ DGGLWLRQ WR YDOXDEOH FRPPHQWV DQG VXJJHVWLRQV 7KDQNV DOVR JR WR -R\$QQ %URZQLQJ 'DYLG 'H9DOYH \$QLQG\D'XWWD&KDUOHV'RODQ0DWWKHZ+XVOLJ5RQDOG.OHPHQFLF-DPHV/DL6WHYHQ0F&DEH0LNH0RWD+DQL1DVVLI-RVH Pincheira, David Rogowski, and Siamak Sattar, who reviewed one or more of the chapters. Special thanks go to StructurePoint and Computers and Structures, Inc. (SAP 2000 and Etabs) for providing a free copy of their software to perform analyses of structure and members. Special thanks also go to Stuart Nielsen, who provided the cover art using SketchUp. The Reinforced Concrete Design DQFKRULQJWRFRQFUHWHEHDPVFROXPQVFUDFNLQJGHÀHFWLRQGLDSKUDJPGXUDELOLW\ÀH[XUDOVWUHQJWKIRRWLQJV frames; piles; pile caps; post-tensioning; punching shear; retaining wall; shear strength; seismic; slabs; splicing; stiffness; structural analysis; structural systems; structural systems; structural systems; structural systems; structural systems; structural analysis; structural systems; struc VOLUME 2: CONTENTS CHAPTER 12—RETAINING WALLS 12.1—General, p. 9 12.2—Design limits, p. 10 12.3—Applied forces, p. 11 12.4—Design strength, p. 12 12.5—Reinforcement limits, p. 13 12.7—Summary, p. 13 12.8—Examples, p. 15 CHAPTER 13—SERVICEABILITY 13.1—Introduction, p. 115 Shrinkage and temperature reinforcement - posttensioned, p. 119 <sup>2</sup>3HUPLVVLEOHVWUHVVHVLQSUHVWUHVVHGFRQFUHWHÀH[XUDO members, p. 120 13.10—Permissible concrete compressive stresses at service loads, p. 120 13.11—Examples, p. 121 <sup>2</sup>'HÀHFWLRQGHVLJQDLGVS CHAPTER 14—STRUT-AND-TIE MODEL 14.1—Introduction, p. 159 14.2—Concept, p. 159 14.3—Design, p. 159 14.4—Struts, p. 160 14.5—Ties, p. 163 14.7—Usual calculation steps and modeling consideration to apply strut-and-tie model, p. 164 14.8—Examples, p. 165 CHAPTER 15—ANCHORING TO CONCRETE 15.1— Introduction, p. 213 15.2—Materials, p. 213 15.3—Design assumptions, p. 216 <sup>2</sup>/LPLWDWLRQVRQLQVWDOODWLRQJHRPHWU\S 15.8—Examples, p. 218 12.1—General A retaining or cantilevered wall is a structural system that provides horizontal resistance to a soil mass and prevents it from assuming its natural slope. A retaining wall consists of a vertical stem and a horizontal resistance to the soil behind the wall, which is at higher elevation than the soil in front of the wall. The footing toe and heel transfers the lateral soil pressure to the soil strata under the retaining wall (Fig. 12.1a (a)). If retaining walls are placed close to property borderlines, the toe or heel can be eliminated (refer to Fig. 12.1a (b) and (c)). Reinforced concrete stems are designed to withstand horizontal soil pressures and surcharge loads. Earthquake loads for retaining walls are not addressed in this Handbook. For the purposes of this chapter, it is assumed that the geotechnical report states that the retaining wall geometry FL¿HG factors of safety (FS) are observed. Failuree of a retaining wall geometry FL¿HG factors of safety (FS) are observed. is ween the retaining wall associated with the interaction between geometry and the local soil properties. Traditional FS against these soil-related failures are shown as follows: D )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )6•DJDLQVWRYHUWXUQLQJIDLOXUH F )60 engineer and are not in the scope of this guide. 7KH UHLQIRUFHG FRQFUHWH VWHP DQG IRRWLQJ AH[XUH DQG shear design strength must be at least equal to the factored moments and shears determined from the analysis. Figure FGHSLFWVWKHGHAHFWHGVKDSHXQGHUORDGRIWKHVWHPKHHO toe, and key and related tension areas (shown by cracks): In situations where the factor of safety against sliding failure is low, and there are site constraints against lengthening the heel, a "key" can be constructed below the footing to increase sliding resistance, as shown in Fig. 12.1d. Typical retaining walls vary in height between 5 and 20 ft. o retaining walls beyond 20 ft, buttresses or counterforts For are usually prov provided. Counterforts are normally preferred as they create clean and an unobstructed view from the stem g. 112.1e). 2.1e). face (Fig. Fig. 12.1a—Retaining or cantilever wall types. Fig. 12.1b—Cantilever wall soil failure modes. American Concrete Institute - Copyrighted © Material - www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 10 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) )LJF<sup>2</sup>&DQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem—Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem—Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem—Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem—Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem—Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem - Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem - Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem - Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem - Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem - Retaining wall stem - Retaining wall stems are designed DV FDQWLOHYHUZDOOGHÅHFWHGVKDSHV Fig. 12.2-Design limits 12.2.1 Wall stem - Retaining wall stem -
Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining wall stem - Retaining thickness t is usually uniform. A tapered front face complicates the formwork construction and reinforcement layout, but requires less concrete material. The minimum recommended stem thickness is 8 in., but many engineers prefer a 10 in. minimum. For preliminary calculations, choose a stem thickness is 8 in., but many engineers prefer a 10 in. minimum. retained earth height h, as shown in Fig. 12.2.1a. Stem WKLFNQHVVLVYHUL¿HGE\VDWLVI\LQJVKHDUGHVLJQVWUHQJWKDQG moment design in addition to bending due to eccentric vertical loads, surcharge loads, and lateral earth pressure (Fig. 12.2.1b). A small axial load tends to increase the moment strength of the wall according to the interaction equation, so ignoring the axial load is conservative. 12.2.2 Wall footings—for preliminary calculations, the footing length can be estimated as 40 to 70 percent of the total height h of the stem. Usually the footing extends on both sides of the stem unless there is a physical constraint, such as a property line or an existing utility. The footing thickness is usually taken as at least equal to the stem thickness, and not les less than 12 in. (refer to Fig. 12.2.1a). 7KH IRRWLQJ W SURMH pressure. sure. HFWLRQ DZ 7KH IRRW IRRWLQJ SURMHFWLRQ DZD\ IURP WKH UHWDLQHG VRLO LV known as the th toe. The toe length can be estimated as one fourt o on d the over fourth to one third overall length of the footing (refer to Fig. 12.2.1a). orizontal fforce applied to the stem exceeds the If the horizontal ximum allow maximum allowable frictional force of the footing, the retainin retaining wall resistance to sliding should be increased. In this case, either the overall footing length is increased or a key can be constructed at the underside of the key in VKHDU UDWKHU WKDQ AH[XUDO ,I GHHSHU NH\V DUH XVHG WKH\ VKRXOGEHUHLQIRUFHGIRUAH[XUHDQGGHVLJQHGDVFDQWLOHYHUV resisting the soil passive force Hp shown in Fig. 12.1d. 7KH FULWLFDO VHFWLRQV IRU FDOFXODWLQJ WKH AH[XUDO VWUHQJWK for the toe and heel are the front and back face of the wall stem The critical sections for calculating shear strength for the toe and heel are taken at a distance d from the front and back face of the wall stem. 12.2.3 Buttresses or counterforts, and vertical sections for calculating shear strength for the toe and heel are taken at a distance d from the front and back face of the wall stem. reinforcement connects the footing to the buttresses or counterforts. If stirrups are used to provide shear strength, they should be detailed in accordance with ACI 318. If straight bars are used, their yield strength should be fully developed through straight embedment or hooks at the interface of the stem or footing and the buttress or counterfort American Concrete Institute - Copyrighted © Material - www.concrete.org 11 Retaining Walls CHAPTER 12—RETAINING WALLS Fig. 12.2.1b—Forces Fi 1 acting on the wall stem. height and unit weight of the retained soil. The lateral pressure is calculated by the Rankine or Coulomb theory as shown in Eq. (12.3a) and (12.3b): Fig. 12.2.1a—Retaining wall preliminary dimensions. The stem and buttress or counterfort are designed as FDQWLOHYHUEHDPV¿[HGDWWKHIRRWLQ]ZLWKDWULEXWDU\ORDG area equal to the distance between individual buttresses or counterforts. 12.3—Applied forces Retaining wall stems are designed to resist axial loads, including the weight of the stem and frictional forces due to VORSHGEDFN¿OODFWLQJRQWKHVWHPDQGWRUHVLVWEHQGLQJGXH to eccentric vertical loads, and lateral earth pressure. The horizontal soil pressures acting on the rear and front faces of the stem wall are referred to as the active and passive soil pressures, respectively, and they are proportional to the P = CaÛsh (12.3a) P = CPÛsh (12.3b) where Ca and CP DUH FRHI¿FLHQWV WKDW GHSHQG RQ WKH VRLO properties and angle of inclination of retained earth. Coef¿FLHQWVCa and CP can vary from approximately 0.3 for loose granular soil to 1 for cohesive soil. The engineer should refer to the soils engineering report for values of Ca and CP to use in design. This chapter considers two locations for a uniform surcharge load is located immediately behind the wall. To calculate the force on the wall, an equivalent height of soil is determined from h2 = waÛs, where wa is the VXUFKDUJHORDGSVI DQGÛs is the soil unit weight (pcf). The horizontal soil pressure is calculated based on adding the American Concrete Institute - Copyrighted © Material - www.concrete.org 12 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Fig. 12.3-Surcharge urc e load behind a ccantilever wall. equivalent height of soil to the stem em height ght as shown iin Eq. q (12.3c) (refer to Fig. 12.3(a)). (h2) H = H1 + H2 = Ca  $\gamma$  s + hhs  $\left| \left\langle 2 \right\rangle$  (12.3c) 2. The surcharge load will increase the pressure on the stem if a line at 45 degrees from the edge of the surcharge load is located at a distance from the vall. The surcharge load is located at a distance from the vall. State +LJKZD\DQG7UDQVSRUWDWLRQ2I&FLDOV\$\$6+72)LJ 5.5.2B) provides a graphical procedure to determine lateral pressure due to point load and line load applied at a distance from the stem. For a uniform load strip at a distance away IURPDQGSDUDOOHOWRWKHZDOOWKHVRLOSUHVVXUHRQWKHVWHPLV FDOFXODWHGDVVKRZQLQ(TG +6+72/5)' Section 3.11.6) (refer to Fig. 12.3(b)): H = H 1 + H2 (12.3d) 22w w [h( $\delta - \sin \delta \cos(\delta + 2\alpha)$ ]  $\pi$  H 1 = Ca  $\gamma$  s h2 2 (12.3e) (12.3f) ZKHUHĮDQGįDUHLQUDGLDQW ZKHUH Į D For in depth discussion on calculating the lateral pressure DJDLQVWDUHWDLQLQJZDOOIURPVXUIDFHVXUFKDUJHSRLQWOLQH RUVWULSORDGLQJUHIHUWR%RZOHV 12.4—Design strength 7KH VWHP KHHO DQG WRH PXVW VDWLVI\ GHVLJQ VWUHQJWK UHTXLUHPHQWV IURP\$&, []Sn • U. The two commonly used inequalities are (Eq. (12.4a) and (12.4b)): []Mn•Mu (12.4a) []Vn•Vu (12.4b) Equation (12.4c) only applies to the stem and only needs WREHFKHFNHGLIVLJQL¿FDQWD[LDOORDGH[LVWV ]Pn • Pu where and H2 = (12.4c) 12.5—Reinforcement limits )RUZDOOVWHPVLQRUJUHDWHULQWKLFNQHVVWZROD\HUV of horizontal and vertical reinforcement are required. The PD[LPXPVSDFLQJRIKRUL]RQWDODQGYHUWLFDOUHLQIRUFHPHQW is the smaller of 3t or 18 in. where t the wall thickness. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 12-RETAINING WALLS 13 Retaining wall reinforcement limits for cast-in-place stems are determined per Table 12.5 (Table 11.6.1 of ACI 318-14). Table 12.5-Minimum reinforcement for walls with in-plane Vu " 0.5-Vc Reinforcement type Deformed bars Welded wire reinforcement Bar/wire size fy, psi Minimum vertical rl Minimum horizontal rt "1R • 0.0012 0.002 Fig. 12.6—Typical expansion joint detail.) RUÀH[XUHWKHPLQLPXPOLPLWVDWVWHPWREDVHFRQQHFtion are determined per 9.6.1.2 of AC 318-14. 12.6—Detailing Many engineers specify weakened plane to form contracWLRQ MRLQWV DQG H[SDQVLRQ MRLQWV DW DSSUR[LPDWHO\ IW LQ RQ FHQWHU & RQWUDFWLRQ MRLQWV UHWHDWD DUHYHUWLFDOMRLQWVRUJURRYHVWKDWZHDNHQWKHFRQFUHWHDWD PRYH\$FRQWUDF VSHFL¿FORFDWLRQDQGDOORZVWKHFRQFUHWHWRPRYH\$FRQWUDFDPIHURQ WLRQMRLQWLVXVXDOO\DYHUWLFDOLQFKDPIHURQERWKVLGHVRI ment continuous through the stem with horizontal reinforcement RQVW WKHMRLQW([SDQVLRQMRLQWVDUHFRQVWUXFWHGZLWKMRLQW¿OOLQ] ss aand waterstops ater retained ear therefore the pressure on the stem. earth is not allowed to drain, the effect off hydrostatic water pressure must be added to that of earth pressure for design of the retaining wall. DO 7R DYRLG D EXLOGXS RI ZDWHU SUHVVXUH EDFN¿OO PDWHULDO EHKLQGWKHVWHPLVXVXDOO\VSHFL¿HGWREHZHOOGUDLQHGFRKHsionless, nonexpansive, noncorrosive material. Silts and FOD\VDUHQRWUHFRPPHQGHGDVEDFN¿OO7KHVWHPLVXVXDOO\
GUDLQHGE\ZHHSKROHV¿OOHGZLWKSHUYLRXVPDWHULDORURWKHU positive drains, preforated drains, pr and elevations. Counterfort walls are usually designed with at least one drain between counterforts. 12.7—Summary 1. Determine base thickness. 4. Estimate length of base. 5. Estimate length of toe. 6. Calculate vertical and horizontal loads. 7. Calculate overturning and restoring moments. 8. Check Chec wall against overturning, sliding, and bearing pressure. VLJQWKHVWHPI VKH 'HVLJQWKHVWHPIRUVKHDUDQGAH[XUH HVLJ WKH KHHO IR VKK 'HVLJQWKHVWHPI VKH 'HVLJQWKHVWHPIRUVKHDUDQGAH[XUH HVLJ WKH KHHO IR VKK 'HVLJQWKHVWHPI VKH 'HVLJQWKHVWHPI 'HVLJQWK 'HVLJQDUHLQIRUFHGNH\IRUÀH[XUHLISUHVHQW etail reinforcement cement per p ACI requirements; reinforce14. Detail ment layout, l yout development opment locations. 15. Provide drainage system. REFERENCES A American Association of State Highway and TransportaWLRQ2I&FLDOV\$\$6+72 <sup>3</sup>6WDQGDUG6SHFL¿FDWLRQIRU +LJKZD\%ULGJHV′HGLWLRQ\$\$6+72:DVKLQJWRQ'& American Association of State Highway and TransporWDWLRQ2I¿FLDOV\$\$6+72 WKLUGHGLWLRQZLWK ,QWHULP 5HYLVLRQV <sup>3</sup>/5)' %ULGJH 'HVLJQ 6SHFL¿cations Customary U.S. Units," AASHTO-LRFD 2007, \$\$6+72:DVKLQJWRQ'& Bowles, J., Foundation Analysis and Design, JWK edition, The McGraw-Hill Co, Inc., 1241 pp. American Concrete Institute - Copyrighted © Material - www.concrete.org 15 12.8—Examples Retaining Walls Example 1: Reinforced concrete basement wall The eight-story building has a 10 ft high, 10 in. thick normalweight reinforced concrete basement wall. The walls support a LQWKLFNOEIWVLGLQJ\$LQWKLFNOHYDWHGUHLQIRUFHGFRQFUHWHVODEVSDQQLQJIWVXEMHFWHGWRSVIVXSHULPSRVHG dead load, and 100 psf live load nonreducible is supported on a 5 in. ledge (refer to Fig. E1.1). The wall compressive strength is fcg SVLDQGVWHHO\LHOGVWUHQJWKLVfy = 60,000 psi. The retained earth behind the basement wall is assumed level and has the following soil data obtained from the geotechnical report: soil unit weight 120 pcf, angle of soil internal friction, 30 degree, FRHI¿FLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLODQGWKHDOORZDEOHVRLOEHDULQJSUHVVXUHSVI \$VVXPHDQHDUWKTXDNHODWHUDOIRUFHDWWKHGLDSKUDJPOHYHORINLSDQGWKDWWKHIURVWOLQHLVIWEHORZWKH¿QLVKHGJUDGH Given: Soil data— Ûs = 120 pcf [GHJUHHV µ = 0.5 qall = 2500 psf Concrete— fcq SVL 3a = 1.0 normalweight concrete fy = 60,000 psi Loads— 7KHHOHYDWHGVODELVVXEMHFWHGWR Superimposed dead load SDL = 15 psf ible) /LYHORDGL = 100 psf (non-reducible) S /DWHUDOORDGDWGLDSKUDJPOHYHO NLS Slab— Span = 24 ft Basement wall— h = 10 ft & ex = 72 ft Fig. E1.1—Basement wall. American Concrete Institute - Copyrighted © Material - www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 16 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Material requirements Concrete compressive strength 19.2.1.1 The value of concrete compressive strength at a 19.2.1.3 JLYHQDJHPXVWEHVSHFL¿HGLQWKHFRQWUDFWGRFXments. Table 19.2.1.1 provides a lower concrete compressive strength limit not less than 2500 psi. 19.3.1 19.3.2 Calculation Provided: fcg SVL!f gc,min= 2500 psi OK Exposure categories and classes The engineer must either assign exposure classes to the retaining wall with respect to Table 19.3.1.1 so the ready-mix supplier can proportion the concrete mixture, or use the classes to directly specify mixture proportions in the contract documents. Based on the exposure classes, the concrete mixtures must satisfy to the most restrictive requirements of Table 19.3.2.1. Concrete exposure classes, the concrete mixture proportions in the contract documents. Based on the exposure classes, the concrete mixture proportions in the contract documents. 19.3.1.1 19.3.2.1 19. 0L[WXUHUHTXLUHPHQWVWKDWPXVWEHVDWLV¿HGIRU te in Table able 19.3.2.1. Class F2 are listed Category S The geotechnical report usually provides the soil sulfate content. Assuming the sulfate content is in the range of moderate severity, the class is S1. Mixture requirements for S1 are listed in Table 19.3.2.1. Class S1 ximum w/ Maximum w/cm is 0.50 Minimu fcgLVSVL Minimum Allowable cementitious materials are ASTM C150 (Type II), ASTM C595 (Type IP, IS, or IT), and ASTM C157 (Type MS) Category W Class W0 The retaining wall is in contact with water, but assuming that a proper drainage system is provided to Minimum fcgLVSVL prevent water from being retained behind the wall, The maximum water soluble chloride ion content in low permeability is not required. nonprestressed concrete as percent by weight of cement is 0.3. Category C The concrete as
percent by weight of cement is 0.3. Category C The concrete as percent by weight of cement is 0.3. (w/cm)max = none and fcg SVL Conclusion: (a) The most restrictive minimum concrete compressive strength is 4500 psi, and (w/cm)max is 0.45. E 2WKHUSDUDPHWHUVVXFKDVPD[LPXPFKORULGHLRQFRQWHQWDQGDLUFRQWHQWDUHH[SRVXUHVSHFL¿FDQGWKXVQRWFRPSDUHG with other exposure limits. (c) The fcgXVHGLQWKHVWUHQJWKGHVLJQPXVWEHDWOHDVWWKHfcgUHTXLUHGIRUGXUDELOLW\ American Concrete Institute - Copyrighted © Material - www.concrete.org Step 2: Equivalent lateral pressure The geotechnical report provides the equivalent p = 40 pcf AXLGGHQVLW\WKHZDOOLVUHTXLUHGWRUHVLVW Step 3: Applied forces 7KHFDQWLOHYHUHGUHWDLQLQJZDOOLVDFRQWLQXRXV PHPEHU,WZLOOEHDQDO\]HGIRUWKHPD[LPXPORDG HIIHFWV7KHFDOFXODWLRQVDUHSHUIRUPHGRQDXQLW OHQJWKIW 7KHUHIRUHDOOFDOFXODWHGIRUFHVPRPHQWVDQGUHLQIRUFHPHQWDUHDVDUH EDVHGRQIWOHQJWKRIWKHUHWDLQLQJZDOO )RULQSODQHVKHDUKRZHYHUWKHIXOOEDVHPHQWZDOO OHQJWKLVXVHG Vertical loads 7KHYHUWLFDOZHLJKWRIWKHUHWDLQLQJZDOOVWHPDQGEDVH WKHZHLJKWRIVLGLQJVHOI ZHLJKWRIWKHHOHYDWHGVODEDQGWKHWULEXWDU\SDL and LWUDQVIHUUHGWRWKHZDOOUHIHUWR)LJ( Fig. E1 E1.2—Applied pplied forc forces on retaining wall. h 6WHPZDOOP Üconctstem tem h Base: P2 Üconctbbase b &XUWDLQZDOOP3 SFI IW IW IW OE P SFI IW IW IW OE P SFI IW IW IW OE P2 OEIWFXUWDLQZHLJKW FXUWDLQZ P3 P4LVWKHHOHYDWHGVODEVHOIZHLJKWDQGOLYHORDG CaÛsh2/2 ZKHUH Ca = - VLQ φ + VLQ φ and q = CaÛsh SFI IW OEIWIW American Concrete Institute - Copyrighted © Material - www.concrete.org 17 Retaining Walls CHAPTER 12-RETAINING WALLS 18 6.2.3 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) The basement wall can be modeled as a simply supported member at the bottom and laterally restrained at the top (refer to Fig. E1.3)-statically determinate. Summing moments about point "A" (refer to Fig. E1.3):  $\begin{pmatrix} x \\ x \end{pmatrix} - M - M$  Top + H B x - (Ca  $\gamma$  s x) | || = 0 \ 2/\ 3/ Calculate reactions: ons Fig. E1 F E1.3—Applied plied force forces on basement wall. Summation of ve vertical forces less base weigh weight of P2 = 300 lb.  $\Sigma$  P = 0 = PA - 4430 lb Summation of moments about Point A: "MA = 0 ( 400 lb/ft/ft ) ( 10 ft )  $\Sigma$ MA = | // \ || + 0 \ 2/\ 3/ + ( 475 ft - lb/ft ) - H B (10 ft) "MA = (7142 ft-lb) - HB(10 ft) = 0 The top lateral force is: = H B = 714 lb/ft, say, 710 lb/ft Summation of horizontal forces: "H = 0 HA + HB - (400 lb/ft/ft ) ( 10 ft )  $\Sigma$ MA = ( 1/ \ 1/ t )  $\Sigma$ MA = ( 1/ t lb/ft/ft)(10 ft)/2 = 0 HA = 2000 lb/ft - 710 lb/ft = 1290 lb/ft Moment, Shear, and Axial Diagrams shown in Fig. E1.4: Moment equation: Vx = H B - q (x) (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H B - q (x) x || h (2) Axial equation: Vx = H BCopyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 19 /RDGIDFWRU /RDGFRPELQDWLRQU = 1.6H controls; when lateral pressure acts alone. (Mu)max = 1.6 (2.35 ft-kip/ft Step 4: Wall design Basement concrete wall supported laterally at top and bottom is modeled as a hinged one-way slab UHIHUWR)LJ( DQGVXEMHFWHGWRODWHUDOIRUFHV as shown in Fig. E1.5. Therefore, the provisions in &KDSWHU2QHZD\EHDPV LQDGGLWLRQWR&KDSWHU :DOOV PXVWEHVDWLV¿HG 11.7.2.3 Flexure: The basement wall is assumed 10 in. thick. Therefore, two layers of reinforcement will be provided although not required by code because it is a basement wall. The main reinforcement in the stem for resisting the lateral earth pressure is vertical and placed at the inside face of the stem wall, away from the retained earth. Fig. E1.5—Soil lateral force on stem. 20.6.1.3.1 Adequate concrete cover protects rein reinforcement against oncrete cover is measured from moisture in the ncrete surfacee to the outermost surface of the steel er re ment applies, to which the cover requirement c = 2 in in. coverr 21.2.2 er is tension con rolled steel Assume that thee m member controlled; HWHWHQVLRQVWUDLQ ic = 0 and VWUDLQit FRQFUHWHWHQVLRQVWUDLQi IURP7DEOH22.2.2.2 22.2.1 [The concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003 ACI 318, Section 22.2.2.2 requires that the tensile VWUHQJWKRIFRQFUHWHVKDOOEHQHJOHFWHGLQAH[XUDO and axial strength calculations. 22.2.2.3 22.2.2.4.1
22.2.2.4.1 22.2.2.1 22.2 compressive stress distribution is inelastic at high stress. The actual distribution of concrete compressive stress is complex and usually not known explicitly. The Code permits any particular stress distribution of comprehensive tests The Code allows the use of an equivalent rectangular compressive strength and is obtained from Table 22.2.2.4.3. For fcg SVL  $\beta 1 = 0.85 - 0.05$  (f c' - 4000 psi) 1000 psi  $\beta 1 = 0.85 - 0.05$  (4500 psi - 4000 psi) = 0.825 1000 psi American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls 5.38 20 22.2.1.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Find the equivalent concrete compressive depth a by equating the compression force to the tension force within a unit length of the wall cross section: C=T C = 0.85fcgba and T = Asfy (0.85)(4000 psi)(12 in.)a = As(60,000 psi) Calculate required reinforcement area: a = As (60,000 \text{ psi}) = 1.31As (0.85)(4500 \text{ psi})(12 \text{ in.}) The wall is designed for the maximum moment FDOFXODWHGDWIWIURPWKHWRS6DWLVI( $\Box Mn \cdot Mu = 1.223 \times 21.2.1 \times 20.6.1.3.1 \times 7.5.1.1 a$ ) (Mn = f y As | d - | (2) 1.31As ) (Mn = (60, 000 \text{ psi}) As | 7.625 \text{ in.} - (1.233 \times 10.6.1) \times 10^{-1} \text{ m}^{-1} 2 // 8VHVWUHQJWKUHGXFWLRQIDFWRUIRUAH[XUH []Assume No. 6 vertical reinforcement: d = tstem - cover - db/2 use 2 in. cover d = 10 in. - 2 in. - 0.75 in./2 = 7.625 in. 6XEVWLWXWLQJLQWR[]Mn•Mu Mu = 3.76 ft-kip = 45 in.-kip calculated ated in Step 3. 1.31As / (0.9) As (60 ksi) / 7.625 in. - = 45 in.-kip (2 // Solving for As As = 0.11) prevent a sudden den failure, the Code requires that IRU QWDUHDLVQRWOH KDQWKH WKHÀH[XUDOUHLQIRUFHPHQWDUHDLVQRWOHVVWKDQWKH greater of: 9.6.1.2 (a) As,min ≥ 3 f c' fy bw d As , min = 3 4500 00 psi (1 (12 in.)(7.625 in.) = 0.31 in.2 ,000 00 psii 60,000 ⇒ As , min = 00.31 3 in.2 Equation (9.6.1.2(a)) controls, because concrete compressive strength fcg = 4500 psi. 21.2.2 Check if the tension controlled assumption and the XVHRI[LVFRUUHFW" To answer the question, the tensile strain in reinforcePHQWPXVWEHcUVWFDOFXODWHGDQGFRPSDUHGWRWKH values in Table 21.2.2. The strain in reinforcement is calculated from similar triangles (refer to Fig. E1.6): 22.2.1.2 8VH1RDWLQRQFHQWHUWRUHVLVWAH[XUH As, provided = 0.33 in.2 > As, req'd = 0.31 in.2 and provide No. 4 at 16 in. on center at the opposite face  $\epsilon t = \epsilon c$  (d - c) c where c = a and a = 1.31As derived previously.  $\beta 1 c = (1.31)(0.33 in.2) = 0.524 in. 0.825 0.003 (7.625 in. - 0.524 in.) = 0.041 0.524 in.$  it = 0.041 > 0.005 Section is tension controlled and, DFFRUGLQJO\[] et = American Concrete Institute - Copyrighted © Material - www.concrete.org 21 Retaining Walls CHAPTER 12-RETAINING WALLS 22.4 22.4.2.1 Fig. E1.6-Strain distribution across stem. Axial (9 in.) (12 ft)(150 pcf) In addition to the lateral soil pressure, the basement Pu = 1.2(450 lb/ft) + 1.2 (12 in./ft) ZDOOLVVXEMHFWHGWRD[LDOIRUFHVIURPWKHIDFDGH + (1.2(15 \text{ psf}) + 1.6(100 \text{ psf}))(12 \text{ ft}) DERYHDQGWKH(10 in.)(12 in.) - (12 in./ft)0.48 in.2) Po = 0.85(4.5 ks ksi + (0.48 in.2)(60 ksi) p/ft Po = 486 kip/ft Apply an axial str strength limit factor of 0.8...0 Po =  $(0.8)(486\ 86\ kip/ft) = 389\ kip/ft 0.8P$  O Po =  $389\ kip/ft 0.8P$  O Po =  $4.3\ kip/$ (refer to Fig. E1.7):  $Vu = (1.0)VEQ/2 = 680 \text{ kip } / 2 = 340 \text{ ki$ 7KHUHIRUH [] Vn [] Vc [] Vn = (0.75)(1,112,810 lb) = 834,600 lb ,V [] Vn > Vu? [] Vn = 834,600 lb >> Vu = 340,000 lb OK 22.5.1.1 7.5.3.1 22.5.5.1 7.5.1.1 Therefore, shear reinforcement is theoretically not required. American Concrete Institute – Copyrighted © Material – www.concrete.org 22 7.4.3.2 THE REINFORCED CONCRETE DESIGN HANDBOOK -SP-17(14) 2XWRISODQHVKHDU The closest inclined crack to the support of the basement wall will extend upward from the face of the base. Accordingly, the Code permits design for a maximum factored shear force Vu at a distance d from the support for nonprestressed members when the conditions listed LQDUHVDWLV¿HG Therefore, the critical section for shear strength is taken at a distance d from the bottom of the stem: Assume Vs = 0 7.5.3.1 22.5.5.1 21.2.1  $\varphi$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12
in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 2 $\lambda$  f c'bd  $\varphi$ Vn = (0.75)(2) 4500 psi(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1  $\Box$ V $\Box$ Vc + Vu VDWLV¿HG"  $\Box$ Vc =  $\varphi$ Vn =  $\varphi$ 9200 lb > Vu = (1.6)(1290 lb) = 2060 lb OK shear reinforcement is not required 11.6.1 Provided minimum reinforcement as required by Table 11.6.1 because Vu'' to  $\text{LQ} \perp \text{Q} \perp \text$ As,l(provided) provid • LQ LQ LQ2/ft As,t•LQ LQ Use No No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face) and No. 4 at 18 in. on center at the moment face (in erior face) and No. 4 at 18 in. on center at the moment face) and No. 4 at 18 in. on c spacing of vertical bars is the lesser of: a. 3h = 3(10 in.) = 30 in. or b. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5(10 in.) = 50 in. or d. 18 in. (controls) 11.7.3.2 Maximum spacing of horizontal bars is the lesser of: c. 5h = 5ZLOOSURYLGHOLWWOHRXWRISODQHAH[XUDOUHVWUDLQW consequently the wall is assumed to hinge at the foundation. Therefore, no moment is being transferred from the foundation. Therefore, no moment is being transferred for transmitting unfactored forces to the soil. Areq'd = P qall Areq'd = 4730 lb = 1.89 ft 2 2500 psf say, 2 ft (24) in.) width of footing. Reinforce footing with four No. 4 continuous. American Concrete Institute - Copyrighted © Material - www.concrete.org 24 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Retaining Walls Example 2—Reinforced concrete cantilever wall 'HVLJQDQRUPDOZHLJKWUHLQIRUFHGFRQFUHWHFDQWLOHYHUZDOOWKDWUHWDLQVDOHYHOHDUWKEDQNIWKLJKDERYHWKH¿QDOHDUWK level as shown in Fig. E2.1.
Concrete mixture must satisfy durability and strength requirements. The grade is 60 (ASTM A615 GR60) 60,000 psi. The soil data, which would typically be taken from a geotechnical report, is assumed to be the following: VRLOXQLWZHLJKWSFIDQJOHRIVRLOLQWHUQDOIULFWLRQGHJUHHVFRHI¿FLHQWRIIULFWLRQGHJUHHVFRHI¿FLHQWRIIULFWLRQGHJUHHVFRHI] psf. \$VVXPHWKDWWKHFDQWLOHYHUZDOOLVQRWVXEMHFWHGWRDQ\RWKHUORDGDQGWKHIURVWOLQHLVIWEHORZWKHcQLVKHGJUDGH Given: Soil data— Ûs = 110 pcf [GHJUHHV 3 qall = 3000 psf Concrete— 3a = 1.0 (normalweight concrete) fy = 60,000 psi Cantilever wall— h1 = 15 ft h2 = 3 ft Fig. E2.1—Can E2.1—Can tilever retaining wall. ACI 318-14 Discussion Step 1: Material requirements 7.2.2.1 The mixture proportion must satisfy the durability requirements. The designer determines the durability classes. Please refer to Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. Calculation By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVWSVI \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLV coordinated with ACI 318. ACI encourages UHIHUHQFLQJLQWRMREVSHFL¿FDWLRQV There are several mixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Example 1 of this Chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. American Concrete Institute - Copyrighted © Material - www.concrete.org Step 2: Equivalent lateral pressure The geotechnical report provides the equivalent lateral pressure the wall is required to resist. Step 3: Preliminary cantilever wall data General criteria The preliminary retaining wall dimensions are determined from guidelines presented by Bowles LQWKHcIWKHGLWLRQRI<sup>3</sup>Foundation Analysis and Design," The McGraw-Hill Companies, Inc., 1996. Figure E2.2 shows the variables used in the example. p = 29.8 pcf Fig. E2.2—Retaining 2—Retain wall required dimensions. 13.3.1.2 20.6.1.3.1 2YHUDOOKHLJKWRIZDOOLV Estimating stem thickness, tstem: (0.07 to 0.12)h Estimating base thickness, tbase: (0.07 to 0.1)h, but at least 12 in. Engineers commonly specify the base be at least as thick as the stem. h = h1 + h2 = 15 ft + 3 ft = 18 ft tstem ~ 0.07(18 ft) = 1.26 ft, say, 1 ft 3 in. tbase ~ 0.07(18 ft) = 1.26 ft, say, 3 in. Assuming 3 in. cover, the effective depth is: IWLQ iLQ iLQ iWLQOK /HQJWKRIEDVHbtoe + tstem + bheel): (0.4 to 0.7)h bbase ~ 0.55 (18 ft) = 9.9 ft, say, 9 ft 3 in. /Heel length: bheel = bbase - btoe - tstem bheel = 9.25 ft - 2.5 ft - 1.25 ft = 5.5 ft American Concrete Institute - Copyrighted © Material - www.concrete.org 25 Retaining Walls CHAPTER 12—RETAINING WALLS 26 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Applied forces For out-of-plane moment and shear, the cantilevered retaining wall is assumed to be continuous, and a representative 1 ft strip is analyzed for the maximum load effects (refer to Fig. E2.3). 9HUWLFDOORDGV ht is the self-weight of the retainThe vertical weight of the soil ing wall: Fig. E2 E2.3 shows the fo forces applied to the re retaining E2.3—Applied lied force forces on retaining wall. Stem wall: P1 = Uconc on (tstem)(h - tbas base) Base: P2 = gconc(tbase)(b) Soil: P3 Us(h - tbase)(bheel) Total vertical load: (150 pcf)(1.25 ft) (1 10,134 lb MP = 3140 314 lb + 1734 lb + 10,134 lb = 15,008 lb MP American Concrete Institute - Copyrighted C Material - www.concrete.org CHAPTER 12-RETAINING WALLS 27 Retaining Walls Moments The self-weight of the retaining wall and the soil above the heel tend to counteract the overturning moment. Moments taken about the front edge of base (toe): Stem wall: M1 = P1(btoe + tstem/2) Base: M2 = P2(bbase/2) Soil: M3 = P3(b - bheel/2) Restoring moment: M1 = (3140 lb)(9.25 ft - 5.5 ft/2) = 65,871 ft-lb  $^{14}$  MR = 9813 ft-lb + 65,871 ft-lb = 83,704 ft-lb The retained soil behind the wall exerts lateral pressure (H) on the wall: H = CaUsh2/2 H = (0.271)(110 pcf)(18 ft)2/2 = 4829 lb where Ca = 1 - sin  $\varphi$  1 + sin  $\varphi$  Therefore, this lateral force tends to overturn the retaining wall about the front edge of the toe: MOTM = (4829 lb) | = 28,974 ft-lb ( 3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = (83,704 ft-lb) (3 |) "M = ( lb) - 28,974 ft-lb) = 54,730 ft-lb 7KHUHIRUHED 7KHUHIRUHEDVHJHRPHWU\LVVXI&FLHQWWRUHVLVWRYHUning. turning. Step 5: Soil pressure 13.3.1.1 The cantilever wall base iss checked using usi unfacufactored forces and d aallowable ble soil bearing pressure. ressure. il pressure, ure, the location off the vertiTo calculate soil rce
must be determined. cal resultant force nt to the fron The distance of thee rresultant front facee of stem:  $a = \Delta M \sum P = 3.65$  ft 15,008 lb Eccentricity is the difference between the resultant location and the base mid-length: e = bbase/2 - a e = 9.25 ft/2 - 3.65 ft = 0.98 ft Check if resultant falls within the middle third (kern) of the base. bbase 9 ft = 1.5 ft > e = 0.98 ft 6 6 Maximum and minimum soil pressure: Therefore, there is no uplift.  $q_{1,2} = \sum P \sum Pe \pm A S q_{1,2} = 15,008 lb (15,008 lb)(0.98 ft) \pm (9.25 ft)(1 ft) (9.25 ft) 2 / 6 q_1 = q_{max} = 2653 psf < q_{all} = 3000 psf q_2 = q_{max} = 591 psf > 0 psf Soil bearing pressure is acceptable. American Concrete Institute – Copyrighted © Material –$ www.concrete.org 28 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Stability requirements Calculate the factor of safety against overturning:  $FS = \sum MR \ge 2.0 M \text{ OTM } FS = 2.89 > 2.0 \text{ OK Calculate the factor of safety against sliding: } FS = \mu \ge P \ge 1.5 \sum H FS = (0.5)(15, 008 \text{ lb}) = 1.5 \sum H FS = (0.5)(15, 008 \text{ lb}) = 1.5 \sum H FS = (0.5)(15, 008 \text{ lb}) = 1.5 \sum H FS = 2.89 > 2.0 M \text{ OTM } FS = 2.89 > 2$ 1.55 (29.8 pcf)(18 ft)(18 ft) / 2 FS = 1.55 > 1.5 OK This calculation neglects the passive pressure against the toe (conservative). Conclusion The retaining wall preliminary dimensions are adequate to resist overturning, sliding, preventing uplift, and limiting pressure on the soil to less than the allowable provided soil pressure in the geotechnical report. In the following steps, the retaining wall is designed for strength. If any of the aforementioned determined dimensions are not satisfactory, then all the previous steps must be revised. Note: Unfactored loads were used to determine the stability of the retaining wall and to calculate the soil pressure. American Concrete Institute – Copyrighted © Material - www.concrete.org Step 7: Stem design The cantilevered concrete stem is a determinate member and is modeled as a 1 ft wide cantilever beam. 13.2.7.1 Flexure The maximum design moment in the stem is calculated at the face of the base foundation (refer to Fig. E2.4). 9HUWLFDOUHLQIRUFHPHQWLQWKHVWHPUHVLVWVWKHODWHUDO earth pressure and is placed near the face of the stem wall that is against Fig. E2.4—Soil lateral force on stem. moisture changes in soil. Cover is measured from the concrete surface to the outermost surface of the reinforcing bar. hstem = (15 ft) + (3 ft) - (1ft 3in.) = 16 ft 9 in. Stem height: 20.6.1.3.1 From Table 20.6.1.3.1, use 2 in. cover 21.2.2 sion controlled; Assume that the stem wall is tension minimum steel strain It DQG5.3.8 7.4.1.1 (a) /RDGIDFWRU n la e. U = 1.6H; when lateral pressure acts al alone. The moment is ta taken att the bottom of thee stem at the base. 7.5.1.1 6DWLVI\[]Mn•Mu 22.2.2.1 The concrete compressive strain at which nominal moments are calculated is: İc = 0.003 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQÀH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strength. For calculating nominal strength, ACI 318 neglects the concrete tensile strength. 22.2.2.3 Determine the equivalent concrete compressive stress for design: Hu =  $1.6 \ 1.6(29.8 \ \text{pcf})(16.75 \ \text{ft}/3) \ \text{ft}/3 = 37,347 \ \text{ft}-\text{lb} \ \text{Mu} = (6 = 44 \ \text{n.-lb} \ 448,163 \ \text{in.-lb} \ \text{The concrete compressive stress}$ concrete compressive stress is complex and usually not known explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of nominal strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRI American Concrete Institute - Copyrighted © Material - www.concrete.org 29 Retaining Walls CHAPTER 12—RETAINING WALLS 30 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 22.2.2.4.1 a u1cZKHUHu1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3 For fcg SVL 22.2.1.1 Find the equivalent concrete compressive depth a by equating the comp psi) = 1.31As 0.85(4500 psi)(12 in.) Calculate required reinforcement area: 7.5.2.1 22.3 a) (Mn = f y As | d - | (2) 1.31As ) (Mn = f y As | d - | (2) 1.31As ) (Mn = cover - db/2 use 2 in. cover 0 in./2 = 12.5 in. d = 12.5 in.
d = 12.5 in. d = 12.5 in. d = 12.5 in. d = 12.5 in. d = 12.5 in. d = 12.5 in 15 in. - 2 in. - 1.0 7.5.1.1 R n•M Mu 6XEVWLWXWLQJLQWR Mu LQOE Q LQNLSFDOFXODWHGDERYH LQNLSFDOFXOD HGDER As, req'd n.2 q'd d = 0.69 in. Solving for As: 9.6.1.2 The cantilevered retaining wall calculated required tensile reinforcement is usually very small compared to the member concrete section. 7KHVWHPUHLQIRUFHPHQWLVFKHFNHGDJDLQVWWKH K EHDPPLQLPXPUHTXLUHGAH[XUDOUHLQIRUFHPHQWDUHDEHFDXVHRIWKHODFNRI redundancy. The Code requires that the beam reinforcement area be at least the greater of: (a) As, min ≥ 3 f c' fy bw d Eq. (9.6.1.2(a) controls, because concrete compressive strength fcg SVL 21.2.2 1.31As / LQ L - NVL As | LQ = LQ - NLS ( 2 | As, min = 3 4500 psi (12 in.)(12.5 in.) = 0.5 in.2 60,000 psi As, prov. = 0.79 in.2 > As, min = 0.50 in.2 As, prov. = 0.79 in.2 > As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 As, prov. = 0.79 in.2 > As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 As, prov. = 0.79 in.2 > As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 As, prov. = 0.79 in.2 > As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 As, prov. = 0.79 in.2 > As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 & As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 & As, req'd = 0.69 in.2 & As, req'd = 0.69 in.2 & KHFNLIWKHVHFWLRQLVWHQVLRQFRQWUROOHGDQGWKH XVHRI[LVFRUUHFW] = 0.50 in.2 & As, req'd = 0.69 in.2 & As, req To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Assume concrete and nonprestressed reinforcement strain varying pro2 SRUWLRQDOWRWKHGLVWDQFHIURPWKHQHXWUDOD[LVUHIHU c = (1.31)(0.79 in.) = 1.25 in. 0.825 to Fig. E2.5): American Concrete Institute – Copyrighted © Material – www.concrete.org OK 22.2.1.2  $\epsilon t = \epsilon c (d - c) c$  where: c = a and a = 1.31As derived previously.  $\beta 1 0.003 (12.5 \text{ in.} - 1.25 \text{ in.}) = 0.027 1.25 \text{ in.} = 0.027 1.25 \text{ in.} = 0.027 1.25 \text{ in.} = 0.027 > 0.005 \text{ 6HFWLRQLVWHQVLRQFRQWUROOHGDQGDFFRUGLQJO}$ ned crack to the support of the ll w tend upward fr m the face cantilevered wall will extend from chi thee compression zzone ne approxi of the base. The lateral ant r between the fa applied to the cantilever face of th the om the face is transferred nsferred base and point d aw away from directly to the base bby compression in the cantilever above the crack. Accordingly, the Code permits design for a maximum factored shear force Vu at d a distance d from the support for nonprestressed members. Vu = 6689 lb calculated above For simplicity, the critical section for design shear strength in this example is calculated at the bottom of the stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) =  $2\lambda$  f c' bw d with Vs = 0 21.2.1 Shear strength reduction factor: 7 KHUHIRUH [Vn [Vc is:  $\varphi Vc = (0.75)(2)(4500 \text{ psi})(12 \text{ in.}) = 15,093 \text{ lb} > Vu = 6689 \text{ lb OK Shear reinforcement is not required. American Concrete Institute – Copyrighted ©$ Material - www.concrete.org 31 Retaining Walls CHAPTER 12—RETAINING WALLS 32 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Heel design ef vity loads is neglected as the soil pressure may not be linear as assumed (refer to Fig. E2.6). Fig. E2 18,515 lb 7.5.3.1 22.5.5.1 20.6.1.3.1 d = 15 in. - 3 in. - 0.5 in. = 11.5 in. 3.1, use 3 in. cover to the tension From Table 20.6.1.3.1, nd aassume No. o. 8 bars. reinforcement and Vc is: 7KHQ[]Vn []V []Vn []V [] v = 13,886 lb ete selfA load factor of 1. 1.2 is used for the conc concrete UE OOVHOIZHLJKW VXSHUZHLJKWDQGIRUEDFN¿OOVHOIZHLJKWDQGVXSHUimposed load of 360 psf. 11.5 in. ] ( 12 (150 pcf p f ) ] 5.5 5 5 ft - Vu = 1.2 ]/ (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 ft) | (1.25 f To increase Vn, two options are possible: 1. Reduce heel length 2. Increase base thickness Reducing the heel length will reduce the shear demand. However, this will also reduce the cantilevered retaining wall resistance against sliding and will reduce the length of the base in contact with soil, thus increasing the soil pressure as the change in soil pressure is a function of d2. In this example, the base thickness is increased by the factor Vu Vn Vu 14,411 lb = = 1.04  $\varphi$ Vn 13,886 lb American Concrete Institute – Copyrighted © Material – www.concrete.org NG The revised base thickness is: Minimum required base thickness: hreq'd = dreq'd + cover + db/2 33 dreq'd = 1.04(11.5 in.) = 11.96 in. say 12 in. hreq'd = 12 in. +3 in. + 0.5 in. = 15.5 in., say, 18 in. = 1 ft 6 in. 20.6.1.3.1 Use 3 in. cover to the soil. d = 18 in. - 3 in. (cover) - 0.5 in. (db/2) d = 14.5 in. 22.5.1.1 Revised shear strength of heel:  $\varphi Vc = (0.75)(2)(4500 \text{ psi})(12 in.)(14.5 in.) = 17,508 \text{ lb}$  The heel thickness increase will result in change to the calculations performed in Steps 4 through 8 above. 14.5 in.) | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in.
| 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 12 | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 14.5 in. | 142YHUWXUQLQJPRPHQW Safety factor against overtur overturning: Force causing sliding: ing: Resisting force: Safety factor against gain sliding: ing: Resisting force: Safety factor against gain sliding: ing: Resisting force: Safety factor against gain sliding: ding: ding: ding: bear strength: Factored shear: The provide shear: The provide shear strength: Factored shear: The provide shear: The MOTM M = 28,974 ft-lb FS = 2.89 482 lb H = 4829 33 3 OE OE Hfr 33 F = 1.55 1.5 FS qmax = 22652 psi qmin = 5593 psi 08 ft-lb Mu = 35,708 N 8 at 12 in. o.c. As = 0.66 in.2 No. []V N = 15, 15,903 lb Vu = 6492 lb []Vn > Vu OK American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 34 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE\WKHVXSHUimposed weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed Therefore, it is not included in WKHFDOFXODWLRQRIÀH[XUH 6.6.1.2 The cantilever heel maximum moment occurs at the stem face (refer to Fig. E2.7). Redistribution of moments cannot occur. Fig. E2.7). Redistribution of moments cannot occur. Fig. E2.7). Redistribution of moments cannot occur. Fig. E2.7).  $Mu = (1.2)(150 \text{ pcf})(1.5 \text{ ft})(5.5 \text{ ft})2/2 + (1.6)(110 \text{ pcf})(16.5 \text{ ft})2/2 \text{ Mu} = 48,000 \text{ ft}-\text{lb} = 576,081 \text{ in}-\text{lb} (0.85)(4500 \text{ psi})(12 \text{ in}.)(a) = As (60,000 \text{ psi}) = 1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As }) (60 60,000 \text{ psi}) \text{ As } |14.5 \text{ in} - |2| ||Mn \cdot Mu \text{ Mu} = 576 \text{ in}-\text{kip } p \text{ in} \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As }) (60 60,000 \text{ psi}) \text{ as } |14.5 \text{ in} - |2| ||Mn \cdot Mu \text{ Mu} = 576 \text{ in}-\text{kip } p \text{ in} \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As }) (60 60,000 \text{ ps } \text{ psi } \text{ Mn} = (60, \text{ psi})) \text{ As } |14.5 \text{ in} - |2| ||Mn \cdot Mu \text{ Mu} = 576 \text{ in}-\text{kip } p \text{ in} \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500 \text{ psi})(12 \text{ in}.) a / (Mn = fy \text{ As } |d - ||2/1.31 \text{ As } 0.85(4500$ above. 1.31As \ ( ((0.9)(60 9)(60 ksi) As | 14.5 14 5 in in. - = 576 in.-kip \ 2 / | Solving for As: As, req'd n.2 q'd d = 0.76 in. The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete section. Because the wall is structurA O DOO\GHWHUPLQDWHWKH&RGHUHTXLUHVWKDWWKHÀH[XUDO reinforcement area is at least the greater of: (a) As,min ≥ 3 f c' fy bw d Equation (9.6.1.2(a)) controls, because concrete compressive strength fcg SVL 21.2.2 Check if the section is tension controlled and the XVHRI[LVFRUUHFW 22.2.1.2 To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Concrete and nonprestressed reinforcement strain is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E2.5): As , min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 60,000 psi Use No. 8 at 12 in. on center (Fig. E2.8). As,prov. =  $0.79 \text{ in.} 2 > \text{As},\text{min} = 0.58 \text{ in.} 2 \text{ As},\text{prov.} = 0.79 \text{ in.} 2 > \text{As},\text{req'd} = 0.76 \text{ in.} 2 \text{ OK} \text{ c} = (1.31)(0.79 \text{ in.} 2) = 1.25 \text{ in.} 0.825 \text{ American Concrete Institute} - Copyrighted © Material - www.concrete.org & t = & c (d - c) c where: c = a and a = 1.31 \text{ As} \beta 1.35 0.003 (14.5 \text{ in.} - 1.25 \text{ in.}) = 0.0318 1.25 \text{ in.} \text{ if } = 0.0318 > 0.005$ 6HFWLRQLVWHQVLRQFRQWUROOHGDQGDFFRUGLQJO\[]ct = Fig. E2.8—Heel reinforcement. Step 9: Toe design 7KHWRHLVGHVLJQHGIRUÀH[XUHDQGVKHDUFDXVHGE\ the bearing pressure. he toe is usually The weight of the soil above the de or be rem neglected as it may erode removed (refer to Fig. E2.9). 21.2.1 22.5.5.1 7.4.3.2 5.3.1 Shear uc actor: Shear strength reduction factor: Fig. E2 Fi E2.9—Shear ar at toe side.  $\Box$  Assume Vs = 0 The critical section for shear strength is taken at a distance d from the stem.  $\varphi$ Vn =  $\varphi$ (Vc + Vs) =  $\varphi$ 2 $\lambda$  f c'bw d Vn = (2) 4500 psi(12 in.)(14.5 in.) = 23,344 lb 7KHUHIRUH [Vn:  $\Box$ Vn = (0.75)(23,344 lb) = 17,508 lb The applied force on the toe is from the soil-bearing pressure acting upwards and the applied shear is calculated at distance d from stem: (6.75 ft + 14.5 in./12) [email protected] 4 = (2652 psf - 593 psf) [] 1 + 593 psf 9.25 ft = 2365 psf For soil reaction on the toe portion, a load factor of 14.5 in. 1.6 is taken. For the self-weight of concrete portion, (2625 psf + 2365 psf) (2.5 psf)ft - Vu = 1.6 | || a load factor of 0.9 is used. || 2 12 / 14.5 in. || - (0.9)(150 pcf)(1.5 ft) | 2.5 ft - || 12 / Vu = 5184 lb - 262 lb = 4922 7.5.1.1,  $V \square Vn \cdot Vu V D W L V H G^{"} \square Vn = 17,508 \text{ lb} > Vu = 4922 \text{ OK}$  Steel reinforcement is not required. American Concrete Institute - Copyrighted © Material - www.concrete.org Retaining Walls CHAPTER 12-RETAINING WALLS 36 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Flexure Soil bearing pressure at stem: (6.75 ft) [email protected] stem , L = (2652 psf - 593 psf) + 593 psf) + 593 psf (9.25 ft) = 2096 psf 7.4.1.1 5.3.8(a) Moment due to soil: 5.3.1 Moment due to soil: 5.3.1 Moment due to soil: 5.3.1 Moment due to soil: 5.3.1 Moment due to soil: 5.3.1 Moment due to soil: 5.3.1 Moment due to soil:
5.3.1 Moment due to soil: 5.3.1 20.6.1.3.1 22.2.1.1 Concrete is placed against earth. Therefore, use 3 in. cover. Setting C = T to calculate required reinforcement: C = 0.85fcqba and T = Asfy Mu2 = 1.6 [(2096 psf (2.5 ft)2/2 + (2652 psf - 2096 psf)(2.5 ft)2/2 + (2652 psf - 2096 psf)(2.5 ft)2/2 + (2652 psf - 2096 psf)(2.5 ft)2/2 + (2652 psf - 2096 psf)(2.5 ft)2/2 + (2652 psf - 2096 psf)(2.5 ft)2/2 = 633 ft-lb Mu1 = (14,188 ft-lb) - (633 ft-lb) = 13,555 ft-lb = 162,665 in.-lb = 163 in.-kip d = 18 in. - 3 in. - 0.5 in. = 14.5 in. (0.85)(4500 psi)(12 in.)(a) = As (60,000 psi) a  $\prod Mn \cdot Mu = 2163 \text{ in.-kip kip calculated ulated above. } 1.31 \text{ As} / 9)(60 \text{ ksi}) \text{ As} | 14.5 14 5 \text{ in in.} - (0.9)(60 = 163 \text{ in.} - \text{kip} / 2 | ) \text{ As,reg'd } n.2 \text{ g'd } d = 0.21 \text{ in.} 2 \text{ As,provid rovid } d = 0.79 \text{ in.} / \text{ft} > \text{As,reg'd } OK \text{ Solving for } \text{As: Minimum required reinforcement is the greater of (a) and (b): } 9.6.1.2 (a) \text{ As,min} \geq 3 \text{ fc' fy bw d } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 = 163 \text{ in.} - \text{kip} / 2 | ) \text{ As,reg'd } OK \text{ Solving for } \text{As: Minimum required reinforcement is the greater of (a) and (b): } 9.6.1.2 (a) \text{ As,min} \geq 3 \text{ fc' fy bw d } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 = 163 \text{ in.} - \text{kip} / 2 | ) \text{ As,reg'd } OK \text{ Solving for } \text{As: Minimum required reinforcement is the greater of (a) and (b): } 9.6.1.2 (a) \text{ As,min} \geq 3 \text{ fc' fy bw d } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 = 163 \text{ in.} - \text{kip} / 2 | ) \text{ As,reg'd } OK \text{ Solving for } \text{As: Minimum required reinforcement is the greater of (a) and (b): } 9.6.1.2 (a) \text{ As,min} \geq 3 \text{ fc' fy bw d } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 = 163 \text{ in.} - \text{kip} / 2 | ) \text{ As,reg'd } OK \text{ Solving for } \text{As: Minimum required reinforcement is the greater of (a) and (b): } 9.6.1.2 (a) \text{ As,min} \geq 3 \text{ fc' fy bw d } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 \text{ solving for } \text{As: Minimum required reinforcement is the greater of (a) and (b): } 9.6.1.2 (a) \text{ As,min} \geq 3 \text{ fc' fy bw d } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 \text{ solving for } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ in in.} - (0.9)(60 \text{ solving for } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ solving for } \text{As , min} = 3 4500 \text{ psi } (12 \text{ in.})(14.5 \text{ solving for } \text{As , min} = 3 4500 \text{ solving for } \text{As , min} = 3 4500 \text{ solving for } \text{As , min} = 3 4500 \text{ solving for } \text{As , min} = 3 4500 \text{ sol$ in.) = 0.58 in.2 60,000 psi  $\Rightarrow$  As = 0.58 in.2 Equation (9.6.1.2(a)) controls, because concrete compressive strength fcg = 4500 psi. 21.2.2 To answer the question, the tensile strain in reinIRUFHPHQWPXVWEHcUVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. The strain in concrete and reinforcement is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E2.10). c= (1.31)(0.79 in.2) = 1.25 in. 0.825 American Concrete Institute – Copyrighted © Material – Copyrighted Concrete Institute – Cop www.concrete.org 22.2.1.2  $\epsilon t = \epsilon c (d - c) c$  where: c = a and a = 1.31As  $\beta 1 \epsilon t = 0.003 (14.5 \text{ in.} - 1.25 \text{ in.}) = 0.0318 1.25$  in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t =
0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.003 (14.5 in. - 1.25 in.) = 0.0318 1.25 in. t = 0.0318 1.25 in. = 0.0318 1.25 in. t = 0.0318 1.25 in. = 0.0318 1.25 in. t = 0.0318 1.25 in. = 0.0318 1.25 in. t = 0.0318 1.25 in. =that No. 5 bars infor transverse wall reinforcement. Per Table 11.6.1, dmin = 0.002 ve reinforcement einfo at 18 in in. on center (12 in.) As , prov. = 00.31 in.2 = 0.21 in.2 (18 in.] > As , reg 'd = 0.18 in.2 /face Provide vertical reinforcement at front face to support the transverse wall reinforcement. The reinforcement can be located at the top, bottom, or allocated between the two faces. Use No. 5 spaced at 18 in. on center Use No. 5 at 18 in. on center. As, min = (0.0018)(9 ft)(12 in.)(18 in.) = 3.5 in.2 Use 12 No. 5 bars distributed. American Concrete Institute – Copyrighted © Material – www.concrete.org 37 Retaining Walls CHAPTER 12—RETAINING WALLS 38 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Dowels 7.7.1.2 The development length concept is based on the 25.4.2 attainable average bond stress over the embedment length of reinforcement. Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. In application, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of critical sections for development RIUHLQIRUFHPHQWLQAH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLnates, or is bent. For the cantilevered wall, toe and heel reinforcement must be developed. Heel reinforcement must be checked for: 6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHOopment length in the toe section. nforcement in the stem to 2. Extension of toe reinforcement evelopment stem must m reinforcement pm length and controls. 25.4.2.1 ng Ed is the greater of Eq. Development length q. 4.2 off ACI 318-14 and 12 in.: (25.4.2.3) or (25.4.2.3) 25.4.2.2 (f y  $\psi$   $\psi$   $\psi$  1. A d = | db | 40  $\lambda$  f c' cb + K tr || db ]/ In this example Eq. (25.4.2.3a) will be used. 25.4.2.3 zt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement z = coating factor; uncoated z = 1.0, because not more than 12 in. of concrete is placed below bars z = 1.0, because not more than 12 in. of concrete is placed below bars z = 1.0, because hars are uncoated z = 1.0, because not more than 12 in. of concrete is placed below bars z = 1.0, because not more than 12 in. of concrete is placed below bars z = 1.0, because hars are uncoated z = 1to use Ktr = 0. But the expression: cb + K tr  $\leq$  2.5 db 2.5 in. + 0 = 2.5  $\leq$  2.5 1.0 in. must not be taken greater than 2.5. Therefore, use 2.5 in Eq. (25.4.2.3 7.7.1.3 25.5 25.5.1.1 25.5.2.1 7.7.2 25.2.3 39 Substitute in Eq. (25.4.2.3a) Development length of heel and toe reinforcement: The available length for toe bars (3ft 9 in.) is long enough to accommodate the required development length of (27 in.) for No. 8. (3 60,000 psi (1.0)(1.0) Ad = | (1.0) 2.5 (40 (1.0) 4500 psi ) = 26.8 in., say, 27 in. > 12 in. OK EdIWLQWRHOHQJWK (LQFRYHU IWLQ Note: the development length of No. 8 bar extended from the base into the stem must be checked Equals the length available refer to Fig. E2.10) OK against the splice length of stem reinforcement. The larger length for No. 8 bar extended from the toe is calculated previously as 27 in. Splice length = 1.3(27 in.) = 35.1 in. > 12 in. Use 3 ft for splice length The deformed bars are in tension and the ratio of provided reinforcement area is less than 2. Therefore, per Table 25.5.2.1, splice is Type B and the splice length is the greater of (1.3Ed) or 12 in. Therefore, extend No. 8 toe bar 3 ft into the stem. Refer to Fig. E2.11. en bars is the greater of: Clear spacing between (a) 1.5 in. (b) 1.5 db (c) 4/3 dagg (a) 1.5 in. 5 ((1.0 in.) n.) = 1.5 in (b) 11.5 in. (c) (4/3 (4/3)(1 in.) = 1.33 in. Assume 1 in. max maximum m aggregate size iss use used. refo 1.5 in. n. contro Therefore, controls American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 40 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 12: Details Weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight of steel per linear foot in stem only: No. 8 = 2.67 lb/ft Total length = (3 ft + 16.5 ft) = 19.5 ft Total weight = (3 ft + 16.5 ft) = 19.5 ft Total weight = (3 ft + 16.5 ft) = 19.5 ft Total weight = (3 ft + 16.5 ft) = 19.5 ft Total weight = (3 ft + 16.5 ft) = 19.5 ft Total weight = (3 ft + 16.5 ft) = 19.5 ft Total weight = (3 ft + 16.5Copyrighted © Material – www.concrete.org 41 Step 13: Alternate detailing The engineer may wish to extend the No. 8 bars coming from the base/toe higher into the stem reinforcement. Assume that No. 8 bars are extended a
distance 4 ft 6 in. into the stem. The factored applied moment at that level can be calculated from:  $Mu = Ca\hat{U}ho3/6$  Mu = (1.6)(0.271)(110 pcf)(16.5 - 4.5 ft)3/6 = 13,736 ft-lb The required calculated spacing matches the spacing of the No. 8 extended from the base. Use No. 5 at 12 in. on center. As, prov = 0.31 in.2/ft > As, req'd OK 11.6.1 & KHFNLISURYLGHGUHLQIRUFHPHQWDUHDVDWLVċHVWKH minimum required in Table 11.6.1. As, min = 0.0012(12 in.)(15 in.) = 0.22 in.2 As, prov = 0.31 in.2/ft > As, min OK 7.7.1.3 Required splice length: 25.5.1.1 The maximum bar size is No. 8, therefore splicing is permitted. 25.5.2.1 ars aare in tension nsion and the ratio The deformed bars nfo ent area to required required reinforcement han 2. Therefore forcement area is less than Therefore, per T Table 25.5.2.1, splice is Type B and the splic splice length is .3ld) or 12 in., where Ed iss the develthe greater of (1.3l cu previously. opment length calculated 25.4.2.3 Development of No. 5 bar ( 3 60, 60,000 psi (1.0)(1.0) Ad = | (0.625 in.) 2.5 440 (1.0) 4500 psi | = 16.8 in., say, 18 in. 7.7.1.3 25.5 25.5.2.2 Splice Compare development length of No. 5. If different bar sizes are spliced in tension, then the greater of Ed of the larger bar and Est of the smaller bar is used for the splice length. Est = 1.3(18 in.) = 23.4 in. Use Ed = 27 in. > Est = 23.4 in. use 27 in. Refer to Fig. E2.12. Extend No. 8 at 12 in. on center dowels coming from the toe 4 ft 6 in. into the stem. Splice No. 5 at 12 in. on center dowels coming from the toe 4 ft 6 in. into the stem. THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 14: Details Weight of steel per lineal foot in stem only: No. 5 = 1.043 lb/ft No. 8 = 2.67 lb/ft Total length of No. 5 = 1.043 lb/ft (14.25 ft) + (1.043 lb/ft (14.25 ft) = 26.9 lb/ft Fig. E2.12—Retaining wall reinforcement (alternate solution). Conclusion: (25.2 lb/ft) Alternate solution provides a saving of: 52.1 lb/ft - 26.9 lb/ft = 25.2 lb/ft or | 1 - = 51.5 percent. (52.1 lb/ft |) American Concrete Institute - Copyrighted © Material - www.concrete.org 43 Retaining Walls Example 325HLQIRUFHGFRQFUHWHFDQWLOHYHUZDOOUHWDLQLQJVORSHGEDFN¿OO Design a normalweight reinforced concrete cantilever wall that retains a 10-degree sloped earth bank 12 ft high above the ¿QDOHDUWKOHYHODVVKRZQLQ)LJ(7KHFRQFUHWHPL[WXUHPXVWVDWLVI\GXFWLOLW\DQGVWUHQJWKUHTXLUHPHQWV7KHJUDGHUHLQforcing bar is assumed at 60,000 psi. The soil data were obtained from the geotechnical report: soil unit weight of 120 pcf, angle RIVRLOLQWHUQDOIULFWLRQRIGHJUHHVFRHI¿FLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLORIDQGWKHDOORZDEOHVRLOEHDULQJ pressure of 3000 psf. \$VVXPHWKDWWKHFDQWLOHYHUZDOOLVQRWEHVXEMHFWHGWRDQ\RWKHUORDGDQGWKHIURVWOLQHLVIWLQEHORZWKH¿QLVKHGJUDGH Given: Soil data— Ûs = 120 pcf [GH]UHHV 3 i GH]UHHVORSH gall = 3000 psf Concrete - 3a = 1.0 normalweight concrete fy = 60,000 psi Cantilever ACI 318-14 Discussion cussion Step 1: Material requirements 7.2.2.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements. The designer determines the durability classes. Please see Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV Calculation C By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure accordan FODVVHV&KDSWHUUHTXLUHPHOWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHOIWKRIFROFUHWHLVVSHFL¿HGDWGD\VWREHDW least 4500 psi. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Example 1 of this Chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. Step 2: Equivalent lateral pressure The geotechnical report provides the equivalent p = 33.8 pcf lateral pressure the wall is required to resist. American Concrete Institute - Copyrighted © Material - www.concrete.org Retaining Walls CHAPTER 12-RETAINING WALLS 44 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 3: Preliminary cantilever wall data General criteria The preliminary retaining wall dimensions are determined from guidelines presented by Bowles LQWKH¿IWKHGLWLRQRI<sup>3</sup>Foundation Analysis and Design, The McGraw-Hill Companies, Inc., 1996. Figure E3.2 shows the variables used in the example. Fig. E3. E3.2—Determine retaining wall dimensions. RIZ V 2YHUDOOKHLJKWRIZDOOLV h = h1 + h2 = 12 2 ft + 3.5 ft = 15.5 ft m thickness: ess: (0.07 to 0.1) h Estimating base th thickness: 0.1) h, but at least 12 in. a IW IWVD/IW\_LQ tstemaIW 0 (15.5 5 ft) = 1.24 ft say 1 ft-6 in. tbase ~ 0.08 Engineers commonly specify the base to be slightly thicker than the stem. 13.3.1.2 20.6.1.3.1 Assuming 3 in. cover, the effective depth is, by inspection larger than 6 in. /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHKHHO WR h /HQJWKRIEDVHWRHK h /HQJWKRIEDVHWRHKHO WR h /HQJWKRIEDVHWRHKHO WR h /HQJWKRIEDVHWRHK h /HQJWKR &DOFXODWHWKHVRLOKHLJKWGXHWRVORSHGEDFN¿OODW heel end: ho IWIW WDQf IW American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 12-RETAINING WALLS Retaining Walls Step 4: Applied forces For out-of-plane moment and shear, the cantilevered retaining wall is assumed to be continuous, and a representative 1 ft strip is analyzed for the maximum load effects (refer to Fig. E3.3). Fig. E3. E3.3—Acting force forces on retaining wall. tem Soil height at stem: as end:: Soil heig the retaining wall (stem and base) and the weight of the soil above the heel. The soil weight over the toe is neglected as it may erode away or be removed: Stem wall: P1 Ûconc(tstem)(h [tbase] Base: P2 Ûconc(tbase)(b) Soil: P3 Ûs(h - tbase)(bheel) Weight of sloped soil above stem: 9HUWLFDOFRPSRQHQWRIODWHUDOSUHVVXUHH: Total vertical load: Moments The self-weight of the retaining wall and the soil above the heel tend to counteract the overturning moment): Stem wall: M1 = P2(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 = P3(bbase/2) Soil: M3 =
P3(bbase/2) Soil: M3 = P3(b(15.5 ft - 1.5 ft) = 2625 lb P2 = (150 pcf)(1.5 ft)(8.25 ft) = 1856 lb P3 = (120 pcf)(15.5 ft - 1.5 ft)(4.75 ft) = 7980 lb P4 = (120 pcf)(4.75 ft)2WDQf OE HV = P5 = { $(33.8 \text{ pcf})(16.34 \text{ ft})^2 VLQf$  OE  $^{11}\text{ P} = 2625 \text{ lb} + 1856 \text{ lb} + 7980 \text{ lb} + 239 \text{ lb} + 784 \text{ lb} = 13,484 \text{ lb M1} = (2625 \text{ lb})(2.25 \text{ ft} + 1.25 \text{ ft}/2) = 7547 \text{ ft} \text{ lb M2} = (1856 \text{ lb})(8.25 \text{ ft}/2) = 7656 \text{ ft} \text{ lb M3}$ = (7980 lb)(8.25 ft - 4.75 ft/2) = 46,883 ft-lb M4 = (239 lb)(8.25 ft - 4.75 ft/3) = 1593 ft-lb Moment of lateral soil pressure vertical component: M5 = (784 lb)(8.25 ft - 4.75 ft/2) = 46,883 ft-lb M5 = HV(bbase) American Concrete Institute – Copyrighted © Material – www.concrete.org 46 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Restoring moment: The retained soil behind the wall exerts lateral pressure (H) on the wall: H = CaÛsh2/2 where Ca is the Rankine's equation obtained from any soils textbook.  $^{m}MR = (7547 \text{ ft-lb}) + (46,833 \text{ ft lb}) + (1596 \text{ ft-lb}) + (46,833 \text{ ft lb}) + (1596 \text{ ft-lb}) + (46,833 \text{ ft lb}) + (1596 \text{ ft-lb}) + (46,833 \text{ ft lb}) + (1596 \text{ ft-lb}) +$ HFRVi Hh OE FRVf Therefore, this lateral force tends to overturn the retaining wall about the front edge of the toe: Summation of moments about the front of toe: MOTM = (70,417 ft-lb) = 44,638 ft-lb = 44,638 ft-lb = 44,638 ft-lb = 44,638 ft-lbStep 5: Soil pressure d cantilever wall 13.3.1.1 The aforementioned determined nfactored forces fo base is checked using unfactored and allowssure. able soil bearing pressure, the location of the vertice m cal resultant force must be determined. Itant to the from The distance off the vertice m cal resultant force must be determined negative.  $\Delta M \sum P a = 44, 44,638$  ft-lb b = 3.3 ft f 13,484 lb 13 Eccentricity is the difference between the resultant location and the base mid-length: e = bbase/2 - a e = 8.25 ft/2 - 3.3 ft = 0.83 ft 6 6 OK Therefore, there is no uplift. Maximum and minimum soil pressure:  $q_{1,2} = \sum P \sum Pe \pm A S q_{1,2} = 13,484 lb (13,484 lb)(0.83 ft) \pm (8.25 ft)(1 ft) (8.25 ft) 2 /6 q_{1,2} = 1634 psf \pm 987 psf q_1 = qmax = 2621 psf < qall = 3000 psf no uplift q_2 = qmax = 647 psf > 0 psf Soil bearing pressure is acceptable. American Concrete Institute – Copyrighted © Material – www.concrete.org 47 Step 6: Stability requirements$ Calculate the factor of safety against sliding:  $FS = \sum M Res \ge 2.0 M OTM FS = 70,147 \text{ ft-lb} = 2.72 > 2.0 OK Calculate the factor of safety against sliding: FS = <math>\sum \mu P \ge 1.5 \sum H FS = (0.5)(16.34 \text{ ft})(16.234 \text{ ft} / 2))(\cos(10^\circ))$  FS = 1.52 OK This calculation neglects the passive pressure against the toe

(conservative). Conclusion The retaining wall preliminary dimensions are adequate to resist overturning, sliding, preventing uplift, and limiting pressure on the soil to less than the allowable provided soil pressure in the geotechnical report. In the following steps, the retaining wall is designed for strength. If any of the aforementioned determined dimensions are not satisfactory, then all the previous steps must be revised. Note: Unfactored loads were used to determine the stability of the retaining walls CHAPTER 12—RETAINING WALLS 48 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Stem design The cantilevered concrete stem is a determinate member and is modeled as a 1 ft wide cantilever beam (refer to Fig. E3.4). 13.2.7.1 Flexure The maximum design moment in the stem is calculated at the face of the base foundation 9HUWLFDOUHLQIRUFHPHQWLQWKHVWHPUHVLVWVWKHODWHUDO earth pressure and is placed near the face of the stem wall that is against moisture changes in soil. Cover is measured from the concrete surface Fig. E3.4—Soil lateral force on stem. of the reinforcing bar. Stem height: Soil height at heel end: 20.6.1.3.1 21.2.2 hstem = (12 ft) + (3.5 ft) - (1.5 ft) = 14.0 ft hsoil IW WDQ f IW From Table 20.6.1.3.1, use 2 in. cover. sion controlled; Assume that the stem wall is tension minimum steel strain It DQG5.3.8 7.4.1.1 (a) /RDGIDFWRU when lateral teral pressure aacts alone. U = Hu = 1.6H; wh Hu =  $1.6(33.8 \ 1.6 \ pcf)(14.84 \ ft)/2(cos(100)) = 5864 \ lb$  The moment is ta taken att the bottom of the ste stem at the base:  $(5864 \ lb)(14.84 \ ft)/3 = 29,007 \ ft-lb = 348,087 \ in.-lb$  Mu =  $(57.5.1.1 \ 6DWLVI \ Mn \cdot Mu \ 22.2.2.1 \ The concrete compressive strain at which nominal PRPHQWVDUHFDOFXODWHGLVIC = 0.003 \ 22.2.2.2$ 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVD variable property and its value is approximately 10 to 15 percent of the concrete tensile strength. American Concrete tensile strength. For calculating nominal strength. ACI 318 neglects the concrete tensile strength. WALLS 22.2.2.4.1 Determine the equivalent concrete compressive stress is complex and usually not known explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive stress distribution of 0.85fcoZLWKDGHSWKRI a ù1cZKHUHಬ1 is a function of concrete compressive strength and is obtained from Table 22.2.4.3. Retaining Walls  $22.2.2.3 49 \ 0.05(4500 \ \text{psi} - 4000 \ \text{psi}) = 0.81000 \ \text{psi} \\ 22.2.2.4.3 \ \text{For fcg''SVL} \ \beta_1 = 0.85 - 22.2.1.1 \ \text{Find the equivalent concrete compressive depth a by equating the compressive dep$ 5(4500 psi)(12 psi)(2 iin.) ed re Calculate required reinforcement area: 7.5.2.1 22.3 a) (Mn = f y As | d - | (2/1.31As) (Mn = (6 si)As | 12 (60,000 \text{ psi}) 12.5 \text{ in.} - (2 | / 21.2.1 8VHVWUHQJWKUHGXFWLRQIDFWRUIRUÀH[XUH XFWL DFWRUIRUÀH[XUH XFWL DFWRUIRUÀH[XUH XFWL DFWRUIRUÀH[XUH XFWL DFWRUIRUÀH] (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = (6 si)As | 12 (60,000 \text{ psi}) 12.5 \text{ in.} - (2 | / 21.2.1 8VHVWUHQJWKUHGXFWLRQIDFWRUIRUÀH[XUH XFWL DFWRUIRUÀH[XUH XFWL DFWRUIRUÀH] (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/1.31As) (Mn = f y As | d - | (2/  $= 15 \text{ in.} - 2 \text{ in.} - 0.875 \text{ in.}/2 = 12.56 \text{ in.}, \text{ say, } d = 12.5 \text{ in.} 7.5.1.1 \text{ } 6XEVWLWXWLQJLQWR m Mu = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 ) OK The cantilevered retaining wall calculated above. As = 0.53 \text{ in.}/ft Solving for As 9.6.1.2 1.31As } (0.9)(60 \text{ ksi})As = 12.5 \text{ in.} - = 348 \text{ in.-kip } (2 \text{ or } 1.5$ required tensile reinforcement is usually very small compared to the member concrete section. The stem reinforcement is checked against the beam minimum reinforcement is checked against the beam minimum UHTXLUHGAH[XUDOUHLQIRUFHPHQWDUHDUDWKHUWKDQWKH one-way slab minimum reinforcement is usually very small compared to the member concrete section. reinforcement area at least: (a) As , min  $\ge 3$  f c' fy bw d As , min = 3 4500 psi (12 in.)(12.5 in.) = 0.5 in.2 60,000 psi Equation (9.6.1.2(a)) controls for concrete strength; As,prov. = 0.6 in.2/ft > As,min = 0.5 in.2/ft OK fcg!SVL American Concrete Institute – Copyrighted © Material – www.concrete.org 50 21.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Check if the tension controlled assumption and the XVHRI[LVFRUUHFW Use No.7 at 12 in. on center. To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Assume concrete and
nonprestressed reinforcement strain varying proportional to the distance from the neutral axis (Fig. E3.5): 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E3.5). 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E3.5). 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E3.5). 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 0.95 in. It = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E3.5). 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 0.95 in. It = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E3.5). 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 0.95 in. It = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E3.5). 22.2.1.2  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31(0.6 in.2) = 0.036 0.95 in. It = 0.036 0 Concrete Institute – Copyrighted © Material – www.concrete.org 7.4.3.2 Shear The closest inclined crack to the support of the base and point d away from the face of the base. The lateral load applied to the cantilevered wall will extend upward from the face of the base and point d away from the face of the base. from the face is transferred directly to the base by compression in the cantilever above the crack. Accordingly, the Code permits design for a maximum factored shear force Vu at a distance d from the support for nonprestressed members. For simplicity, the critical section for design shear strength in this example is calculated at the bottom of the stem: 51 Retaining Walls CHAPTER 12—RETAINING WALLS Vu = 1.6(33.8 pcf) (16.34 ft - 1.5 ft - 12.5 in./12) 2 cos10° 2 = 5070 lb 7.5.3.1 22.5.5.1 Vn = Vc + Vs =  $2\lambda$  f c'bw d with Vs = 0 Vn = (0.75)((0.75)(20,125 \text{ lb}) = 15,093 \text{ lb}) G", V[]Vn•VuVDWLV¿HG" 093 lb > Vu = 5070 5 lb []Vn = 115,093 7.5.1.1 OK ar re ement is not required. Shear reinforcement Step 8: Heel design de avity loads is neglected as the soil pressure may 6). not be linear as assumed (refer to Fig. E3.6). Fig. E3.6—Force on heel. 21.2.1 Shear strength is taken at a distance d from the bottom of the stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) =  $2\lambda$  f c'bw d 20.6.1.3.1 d = tbase - cover - db/2 [Vn = (2) 4500 psi(12) strength reduction for shear strength is taken at a distance d from the bottom of the stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) =  $2\lambda$  f c'bw d 20.6.1.3.1 d = tbase - cover - db/2 [Vn = (2) 4500 psi(12) strength reduction for shear strength is taken at a distance d from the bottom of the stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) =  $2\lambda$  f c'bw d 20.6.1.3.1 d = tbase - cover - db/2 [Vn = (2) 4500 psi(12) strength reduction for shear strength is taken at a distance d from the bottom of the stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) =  $2\lambda$  f c'bw d 20.6.1.3.1 d = tbase - cover - db/2 [Vn = (2) 4500 psi(12) strength reduction for shear strength reduction for in.)(14.5 in.) = 23, 345 lb with Vs = 0 where d = 18 in. - 3 in. - 0.875 in./2 = 14.56 in., say, 14.5 in. American Concrete Institute - Copyrighted © Material - www.concrete.org 52 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) From Table 20.6.1.3.1, use 3 in. cover to tension reinforcement and assume No. 7 bars. 7KHQ[]Vn []Vc is: Vn = (0.75)(23,345 lb) = 17,508 lb A load factor of 1.2 is used for the concrete selfZHLJKWDQGIRUEDFN¿OOVHOIZHLJKW Vu = (1.2)(1.5 ft) + (1.6)(120 pcf)(4.75 - 14.5 in./12)(1.5 ft) + (1.6)(17KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE\WKHVXSHUimposed weight of soil and self-weight of heel. The soil pressure may not be linear as assumed. Therefore, it is not included in WKHFDOFXODWLRQRIAH[XUHUHIHUWR)LJ( 6.6.1.2 The heel maximum moment occurs at the stem face. Redistribution of moments cannot occur. Fig. E3 E3.7-Heel critical moment section. 5.3.1 5.3.8 A load factor of 1.2 iss used for the concrete selfUVRLO ZHLJKWDQGIRUVRLOEDFN¿OO pcf) .55 fft) + 1.6(120 pcf)  $Mu = (1.2(150 \text{ pcf})(1.5 \times (14 \text{ ft} + ((16.34 \text{ ft} - 1.5 \text{ ft}))/2(1 \text{ ft}))(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75 \text{ ft})2/2 34 \text{ ft} - 1.5 \text{ ft})/2(1 \text{ ft})(4.75
\text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{ ft})(4.75 \text{$ 22.2.1.1 Setting C = T: 0. 5(45 psi)(12 12 in.)(a) in )(a) = As(60,000 psi) 0.85(4500 a = As (60,000 psi)) = 1.31As 0.85(45000 psi)(12 in.) 0.8 7.5.2.1 a) (Mn = fy As | d - | (2/1.31As) ( (60 000 00 p M n = (60, psi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31As) ( (0.9)(60 ksi) As | 14.5 in. - (2/2.1.31A - = 411 in.-kip (2 |/ Mu = 411 in.-kip calculated above. Solving for As: As = 0.54 in.2 Use No. 7 at 12 in. on center (Fig. E3.8) American Concrete Institute - Copyrighted © Material - www.concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete Institute - Copyrighted © Material - www.concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small compared to the member concrete.org 9.6.1.2 The cantilevered retaining wall calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile reinforcement is very small calculated tensile structurDOO\GHWHUPLQDWH(TD PXVWEHVDWLV $\partial$ HG (a) As ,min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 > As, min = 0.58 in.2 > LVFRUUHFW To answer the question, the tensile strain in reinIRUFHPHQWPXVWEHUVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Concrete and nonprestressed reinforcement strain is assumed to vary proportionally from the neutral axis. From similar triangles: 22.2.1.2 53  $\epsilon$ t =  $\epsilon$ c (d - c) c where: c = a and a = 1.31A .31As  $\beta$ :  $c = \epsilon t = (1.31)(0.6 \text{ in.} 2) = 0.925 \text{ in.} 0.825 0.003 (14 \text{ n.} - 0.925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 0 925 \text{ in.}) = 0.044 (14.5 \text{ in.} 0.925 \text{ in.}) = 0$ 54 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 9: Toe design 7KHWRHLVGHVLJQHGIRUAH[XUHDQGVKHDUFDXVHGE\ the bearing pressure. The weight of the soil above the toe is usually neglected as it may erode or be removed (refer to Fig. E3.9). Fig. E3.9—Shear at toe side. 21.2.1 Shear Strength reduction factor. 9.4.3.2 The critical section for shear strength is taken at a distance d from the stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) =  $2\lambda$  f c'bw d with Vs = 0 7KHUHIRUH[Vn: [Vn = 2 () 4500 psi (12 in.)(14.5 in.) = 23,345 lb [V Vn = ((0.75)(23,345 lb) = 17,508 lb The applied force on the toe is from the soil bearing ward pressure acting upwards. 7.4.3.2 Applied shear at ddistancee d from stem: 21 ppsf From Step 5: qmax = 2621 7 psf qminn = 647 [email protected] d = ((2621 psff - 647 psf) p in./12 ft/in.) in./1 ( 6 ft + 14.5 in./8 ||) + 647
psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is taken. For the self-weight of concrete portion, Vu = 1.6(( 1.6)(2621 psf + 14.5 in./8) ||) + 647 psf = 2372 psf ( 8.25 8 25 ft f 5.3.1 For soil reaction on the toe portion a load factor of 1.6 is 2372)/2)(2.25 ft - 14.5 in./12) a load factor of 0.9 is used. -0.9(150)(1.25 ft)(2.25 ft - 14.5 in./12) = 4161 lb - 176 lb = 3985 lb 7.5.1.1,  $V_{\Box}$  Vn • VuVDWLV $\dot{c}$ HG"  $\Box$ Vn = 17,508 lb > Vu = 3985 lb OK Shear reinforcement is not required American Concrete Institute - Copyrighted  $\odot$  Material - www.concrete.org Flexure Soil bearing pressure at stem: 55 ( 6 ft (11,342 ft-lb) - (513 ft-lb) IWOE LOOEDQNLS 20.6.1.3.1 22.2.1.1 Concrete is placed against earth. Therefore, use 3 in. cover. d = 18 in. - 3 in. - 0.5 in. = 14.5 in. Setting C = T to calculate required reinforcement: (0.85)(4500 psi)(12 in.)(a) = As(60,000 psi) = 1.31 As 0.85(4500 psi)(12 in.) 7.5.2.1 a) (Mn = f y As |d - || 2/2)compressive strength, fcq SVL 21.2.1 Check if the tension controlled assumption of the VHFWLRQDQGWKHXVHRI[LVFRUUHFW 21.2.2 To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. The strain in concrete and reinforcement is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E3.5): 22.2.1.2 As = 0.1 0.17 in.2 Provide Pr vide same reinforcement e  $(14.5 \text{ in.}) = 0.58 \text{ in.} 2\ 60,000\ \text{psi}\ \text{As,prov'd} = 0.6\ \text{in.} 2 > \text{As,req'd} = 0.17\ \text{in.} 2 = 0.95\ 0.825\ \epsilon t = 0.003\ (14.5\ \text{in.} - 0.95\ \text{in.}) = 0.043\ 0.95\ \text{in.}) = 0.043\ 0.95\ \text{in.} = 0.04$ REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E3.10—Toe reinforcement. Step 10: Minimum transverse reinforcement. Step 10: Minimum transverse reinforcement. Per Table 11.6.1, dmin = 0.002 2 As, m s, min = (0.002)(12 in.)(15 in.) = 0.36 in. nd tempe Distribute shrinkage and temperature reinforcement equally between thee fron front and back face of the stem wall. (As,min (s,min)backk = 0.36 in.2/2 = 0.18 in. 2 nov = 0.21 in.2 rov horizontally horizontally. 24.4.3 Provide vertical reinforcement at front face to support the transverse wall reinforcement. The reinforcement. The reinforcement can be located at the top, bottom, or allocated between the two faces. Use No. 5 at 18 in. on center vertically at the exterior face. As,min =
(0.0018)(8.25 ft)(12 in.)(15 in.) = 2.7 in.2 Use nine No. 5 bars distributed. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS Retaining Walls Step 11: Dowels 7.7.1.2 The development length concept is based on the 25.4.2 attainable average bond stress over the embedment length of reinforcement. Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections, the development length concept requires minimum lengths or extensions of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development RIUHLQIRUFHPHQWLQAH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLnates, or is bent. For the cantilevered wall, toe and heel reinforcement must be developed. Heel reinforcement is developed beyond the stem critical sections for the cantilevered wall, toe and heel reinforcement must be developed. section. Toe reinforcement must be checked for: PPRGDWHWKHGHYHO6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHO6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHO6XI and the stem must be ice length and the stem must be ice length and the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement in the stem must be inforcement inforcement in the stem must be inforcement in checked against the splice thee larg larger length controls. 25.4.2.1 th Ed is the greater of Eq (25.4.2.2) Development length or (25.4.2.3) of ACI 318-14 and 12 in.: 25.4.2.2 (fy\text{ ye} \ 1. A d = | db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db | 40 \larkstrian f c' cb + K tr || \ db 25.4.2.3 zt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ze = coating factor; uncoated zs = 1.0, because bars are uncoated zs = 1.0, because No.7 bars are used cb = 2 in. + 0.875 in./2 = 2.44 in. < 2.5 American Concrete Institute - Copyrighted © Material - www.concrete.org 58 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Ktr = transverse reinforcement index It is permitted to use Ktr = 0. However, the expression: taken greater than 2.5. cb + Ktr must not be db 2.44 in. + 0 = 2.79 > 2.5 0.875 in. Therefore, use 2.5 in Eq. (25.4.2.3a) Substitute into Eq. (25.4.2.3a): Development length of toe reinforcement: (3 60,000 psi (1.0)(1.0) Ad = (0.875 in.) 2.5 (40 (1.0) 4500 psi ) = 23.5 in., say, 24 in. The available length for toe bars (2 ft 3 in.) is long enough to accommodate the required development development length of toe reinforcement: (3 60,000 psi (1.0)(1.0) Ad = (0.875 in.) 2.5 (40 (1.0) 4500 psi ) = 23.5 in., say, 24 in. The available length for toe bars (2 ft 3 in.) is long enough to accommodate the required development development length of toe reinforcement: (3 60,000 psi (1.0)(1.0) Ad = (0.875 in.) 2.5 (40 (1.0) 4500 psi ) = 23.5 in.) as (2 ft 3 in.) as length of 24 in. for No. 7. 7.7.1.3 25.5 25.5.1.1 Available length < 2 ft 3 in. (toe length)-3 in. (cover) Ed < 2 ft length available OK Note: the development length in the stem must be checked against the splice length for th The required development the bars ously as extended from the toe iss calculated pre previously 24 in. 25.5.2.1 ar are in tension and th The deformed bars the ratio orc nt area to required required required required regulated previously 24 in.
25.5.2.1, splice is Type B and the splice length is the greater of (1.3Ed) or 12 in. Splice Sp ce llength: Est = (1 (1.3)(24 in in.)) = 31.2 31 in., say, 33 in. > 27 in. Controls sa Also,, it must satisfy 25.4.2.1(b) > 12 in. 7.7.2 25.2.3 Clear spacing between bars is the greater of: (a) 1.5 in. (b) 1.5(1.0 in.) = 1.5 in. (c) (4/3)(1 in.) = 1.33 in. Assume 1 in.maximum aggregate size is used. Therefore, 1.5 in. controls American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 59 Weight of steel per lineal foot in stem only: No. 7 = 2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft – 1.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft – 1.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 extending into stem from toe = 2 ft 9 in. Total weight = (2.044 lb/ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = (12 ft + 3.5 ft) = 14 ft No. 7 in stem = lb/ft/ft) (14 ft + 2.75 ft) = 34.24 lb/ft Fig. E3.11—Retaining wall reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org Retaining Walls Step 12: Details 60 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 13: Alternate solution The engineer may wish to extend the No. 7 bars coming from the base/toe higher into the stem to a level where smaller bar diameters, larger bar spacing, or both can be used for the stem. The factored applied moment at that level can be calculated from: Mu = CaÜho3/6 Mu = (1.6)(0.3)(120 pcf)(14.84 ft - 3 ft)3/6 = 15,934 ft-lb The required reinforcement area: As,req'd = 0.29 in.2/ft Use bar size such that the required calculated spacing matches the spacing of the reinforcement extended from the base. Use No. 5 at 12 in. on center As,prov = 0.31 in.2/ft > As,req'd = 0.29 in.2/ft 11.6.1 & KHFNLISURYLGHGUHLQIRUFHPHQWDUHDVDWLV¿HVWKH minimum required in Table 11.6.1. As,min = 0.0012(12 in.)(15 in.) = 0.22 in.2 As,prov = 0.31 in.2/ft > As,min = 0.22 in.2 OK 25.5.1.1 Required splice length: 25.5.2.2 The maximum bar size is No. 8, therefore splicing is permitted. OK izes are spliced liced in tens hen the If different bar sizes tension, then greater of Ed of the largerr bar and Estt of the smaller ce len bar is used for the splice length. 25.5.2.1 ar are in tension and th The deformed bars the ratio orc nt area to required re  $(0.625 \text{ in.}) 2.5 \setminus 40 (1.0) 4500 \text{ psi} = 16.8 \text{ in.}, \text{ say}, 18 \text{ in.} \text{ Et} = 1.3(16.8 \text{ in.}) = 21.9 \text{ in.}, \text{ say}, 22 \text{ in.} 25.5 25.5.2.2 \text{ Splice Compare development length of No. 7 at 12 in.} on center dowels coming from the toe 3 ft into the stem. Splice No. 5 at 12 in. on center. The overlap$ length is 2 ft. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 61 Weight of steel per linear foot in stem only: No. 5 = 1.043 lb/ft No. 7 = 2.044 lb/ft Total length of No. 7 = 5.0 ft + 2.0 ft = 11 ft Total weight = (2.044 lb/ft/ft)(5.0 ft) + (1.043 lb/ft Total length of No. 7 = 5.0 ft To lb/ft/ft (11 ft) = 21.7 lb/ft Fig. E3.12—Retaining wall reinforcement (alternate solution). Conclusion: (21.7
lb/ft - 21.7 lb/ft - 21.7 lb/ft - 21.7 lb/ft - 21.7 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft - 21.7 lb/ft = 12.5 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft or | 1 - = 36.5 percent. (34.2 lb/ft o REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Retaining Walls Example 4—Reinforced concrete cantilever wall retaining earth with surcharge load 'HVLJQDQRUPDOZHLJKWUHLQIRUFHGFRQFUHWHFDQWLOHYHUZDOOWKDWUHWDLQVDOHYHOHDUWKEDQNIWKLJKDERYHWKH¿QDOHDUWKOHYHO and supporting a storage area rated at 360 psf, as shown in Fig. E4.1. The concrete mixture must satisfy durability and strength requirements. The grade of reinforcing bar is assumed at 60,000 psi. The soil data were obtained from the geotechnical report: VRLOXQLWZHLJKWRISFIDQJOHRIVRLOLQWHUQDOIULFWLRQRIGHJUHHVFRHI¿FLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLORI and the allowable soil bearing pressure of 3000 psf. \$VVXPHWKDWWKHFDQWLOHYHUZDOOLVQRWVXEMHFWHGWRDQ\RWKHUORDGDQGWKHIURVWOLQHLVIWLQEHORZWKH¿QLVKHGJUDGH Given: Soil data—  $\hat{U}s = 120 \text{ pcf } 35 = [degrees 3 = 0.55 \text{ qall} = 3000 \text{ psf Concrete} - 3 a = 1.0 (normalweight concrete) fy = 60,000 \text{ psi Cantilever } 1- ver retain retaining wall. ACI 318-14 Discussion Step 1: Material requirements 7.2.2.1 The mixture proportion must$ satisfy the durability urability requirements of Chapter 19 and structural strength requirements. The designer determines the durability classes. \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV Calculation By spe specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVW psi. There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Example 2 of this Chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. Step 2: Equivalent lateral pressure the equivalent p = 32.5 pcf lateral pressure the wall is required to resist. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS harge from storage area is conThe uniform surcharge ival earth th height abo verted to an equivalent above thee top of the wall: hs = w ys Retaining Walls Step 3: Preliminary cantilever wall data General criteria The preliminary retaining wall dimensions are determined from guidelines presented by Bowles LQWKHcIWKHGLWLRQRI<sup>3</sup>Foundation Analysis and Design, The McGraw-Hill Companies, Inc., 1996. Figure E4.2 shows the variables used in the example. Fig. E4 E4.2—Retaining etaining wall required dimensions. hs = Retaining wall height: Equivalent wall height: Equivalent wall height due to surcharge: Estimating base thickness: (0.07 to 0.12)heqq Estimating base thickness: (0.07 to 0.1211.5 ft 11.5 ft + 3 ft = 14.5 ft heq = h + hs = 11 0.07(14.5 ft) = 1.02 ft, say, 1 ft stem ~ 0.07 tbase ~ 0.08 (14.5 ft) = 1.16 ft, say, 1 ft 3 in. Engineers commonly specify the base be at least as thick as the stem. 13.3.1.2 Assuming 3 in. cover, the effective depth is: IWLQ [LQ IW!LQOK 20.6.1.3.1 /HQ]WKRIEDVHWRHKHHO WR heq /HQ]WKRIEDVHWRHKHHO WR heq /HQ]WKRIEDVHWRHK heq /HQ]WKRIEDVHWRHKA Heq /HQ]WKRIEDVHWRHKHO WR heq /HQ]WKRIEDVHWRHKA heq /HQ]WKRIEDVHWRHKA heq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA hq /HQ]WKRIEDVHWRHKA h to b/3 Heel length: bheel = bbase - btoe - tstem bbase ~ 0.5 (14.5 ft) = 7.25 ft, use 7 ft 3 in.) - (2 ft 3 in.)
- (2 ft 3 in.) - (2 ft 3 in out-of-plane moment and shear, the cantilevered retaining wall is assumed to be continuous, and a representative 1 ft strip is analyzed for the maximum load effects (refer to Fig. E4.3). Vertical load is the self-weight of the retaining wall (stem and base) and the weight of the soil above the heel, including the storage orage area uniform load. The soil weight overr the toe is neglected as it may erode away or bee remo removed: Stem wall: P1 Uconc onc(tstem)(h i tbase) Base: P2 Uconc(ttbas base)(b) Soil: P3 Us(h + hs i tbase se)(bheel) (150 pcf)(1.0 f)  $(11.5\ 11.5\ ft + 33.0\ ft - 1.25\ ft)(4\ ft) = 6360\ lb\ P3 = (12\ oa\ Total\ vertical\ load: \mathbb{M} = 1538\ 15\ lb\ + 6360\ lb\ = 9257\ lb\ \mathbb{M} = 9257\ lb\$ + tstem/2) Base: M2 = P2(b/2) Soil: M3 = P3(b - bheel/2) M1 = (1538 lb)(2.25 ft + 1.0 ft/2) = 4229 ft-lb M2 = (1359 lb)(7.25 ft - 4.0 ft/2) = 33,390 ft-lb H2 = (1359 lb)(7.25 ft - 4.0 ft/2) = (1359 lb)(7.25 ft - 4.0 ft/2) = (1359 lb)(7.25 ft - 4.0 ft/2) = (1359 lb)(7.25 ft - 4.0 ft/2) = (1359 lb)(7.25 ft - 4.0 ft/2) = (1359 lb)(7.25 ft - 4.0 f the wall and the superimposed loads exert lateral pressure (H) on the wall: Due to retained earth: H1 = (Ca)( $\hat{U}s$ )(h2/2) Due to surcharge: H2 = (Ca)(the retaining wall about the front edge of the toe: MOTM = H1(h/3) + H2(h/2) 6XPPDWLRQRIPRPHQWV" M  $\stackrel{\text{\tiny M}}{=}$  (42,545 ft-lb) - (14,693 ft-lb) = 27,582 ft-lb
7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI $\stackrel{\text{\tiny C}}{=}$  14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 65 Retaining Walls Step 5: Soil pressure 13.3.1.1 The aforementioned determined cantilever wall base is checked using unfactored forces and allowable soil bearing pressure. To calculate soil pressure, the location of the vertical resultant force must be determined. The distance of the resultant to the front face of stem:  $a = \Delta M \sum P = 27,852$  ft-lb = 3.01 ft 9258 lb Eccentricity is the difference between the resultant falls within the middle third of the base. essure: Maximum and minimum soil pressure:  $q_{1,2} = \sum P \sum M \pm A$ S bbase 7.25 ft = 1.21 ft > e = 0.62 ft 6 6 OK Therefo there is no uplift. Therefore,  $q_{1,2} = 9257$  lb (9257 lb)(0.62 ft) ± (7.25 ft)(1 ft) (7.25 ft) 2 /6 q1 = qma 32 psf < qall = 3000 psf max = 1932 2 psf > 0 psf no uplift q2 = qmin i = 622 - bearing pressure is acceptable. Step 6: Stability requirements Calculate the factor of safety against overturning:  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 2.00 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = \sum MR \ge 2.0 \text{ M OTM } FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 2.90 \times 2.0 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safety against sliding}$ ;  $FS = 1.56 \times 1.5 \text{ OK Calculate the factor of safet$ Conclusion The retaining wall preliminary dimensions are adequate to resist overturning, sliding, preventing uplift, and limiting pressure on the soil to less than the allowable provided soil pressure in the geotechnical report. In the following steps, the retaining wall is designed for strength. If any of the aforementioned determined dimensions are not then all the previous steps must be revised. Note: Unfactored loads were used to determine the stability of the retaining wall and to calculate the soil pressure. American Concrete Institute - Copyrighted © Material - www.concrete.org 66 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Stem design The cantilevered concrete stem is a determinate member and is modeled as a 1 ft wide cantilever beam. 13.2.7.1 Flexure The maximum design moment in the stem wall that isplace of the stem wal against the retained soil. Adequate concrete surface of the reinforcement against moisture changes in soil. Cover is measured from the concrete surface of the reinforcing bar. Stem height: hstem = (9 ft) + (2.5 ft) = 10.25 ft 20.6.1.3.1 From Table 20.6.1.3.1 From Table 20.6.1.3.1, use 2 in. cover. 21.2.2 Assume that the member is tension contr controlled; minimum steel strain It DQG[]ral pressure ure due to surch ge load: Equivalent lateral surcharge 7.4.1.1 5.3.8(a) /RDGIDFWRU ne. U = 1.6H; when la lateral pressure acts al alone. d The moment is taken at the bottom of the stem and above the base (refer to Fig. E4.4). 7.5.1.1 6DWLVI\[]Mn•Mu 22.2.2.1 The concrete compressive strain at which nominal moments are calculated is: ic = 0.003 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete tensile strength. For calculating nominal strength. For calculating nominal strength. For calculating nominal strength. For calculating nominal strength. charge =  $(32.5 \text{ U} = 1.6[(98 \ 1.6 \ f)(11.5 \ ft - 1.25 \ ft) \text{ psf})(11.5 \ ft - 1.25 \ ft)(10.25 \ ft/2) = 1607 \ lb + 2732 \ lb = 4340 \ lb \ Mu = (1607 \ lb)(10.25 \ ft/2) + (2732 \ lb) = 17,570 \ ft - lb = 210,843 \ in.-lb, \text{ say}, 211 \ in.-kip \ American \ Concrete \ Institute - Copyrighted © Material - www.concrete.org \ CHAPTER \ 12-RETAINING \ WALLS$ 22.2.2.4.1 Determine the equivalent concrete compressive stress is complex and usually not known explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of nominal strength in reasonable agreement with the results of comprehensive tests. Rather that tests, the Code allows the use of an equivalent rectangular compressive strength and is obtained from Table 22.2.4.3.  $L_{2}(26WHPAH[XUDOUHLQIRUFHPHQW)$  (PAH[XUDO 0.05(4500 5(4500 ps psi - 4000 psi) = 0.825 1000 100 psi 22.2.2.4.3 VL For fcg"SVL 0.85 -  $\beta 1 = 0.8 22.2.1.1$  Find the equivalent concrete compressive depth, a, by equating the compression force to the tension force within a unit length of the wall cross section: C=T 0.85(4500 psi)(12 in.(a)) As (60,000 psi) a = As (60,000 psi) = 1.31As 0.85(4500 psi)(12 in.) 7.5.2.1 22.3 Calculate required reinforcement area: a) (Mn = fy As  $| d - | \langle 2 | \rangle$  21.2.1 8VHVWUHQJWKUHGXFWLRQIDFWRUIRUÀH[XUH 20.6.1.3.1 [Assume No. 6 vertical reinforcement: d = tstem - cover - db/2 use 2 in. cover 6XEVWLWXWLQJLQWR Mu = 211 in.-kip (0.9)(60 ksi)As 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. 1.31As (0.9)(60 ksi)As 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. -
2 in. - 0.75 in./2 = 9.625 in., say, 9.6 in. - 2 in. - 0.75 in./2 = 9.625 in./2 in. - 2 in. - 0.75 in./2 = 9.625 in./2 in. Copyrighted © Material - www.concrete.org 68 9.6.1.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The cantilevered retaining wall calculated required tensile reinforcement is usually very small compared to the member concrete section. The stem reinforcement is checked against the (9.6.1.2(a)) controls, because concrete compressive strength fcg SVL 21.2.2 Check if the tension controlled assumption and the XVHRI[LVFRUUHFW To answer the question, the tensile strain in GDQG UHLQIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQG compared to the values in Table 21.2.2. Assume ressed rreinforcement strain concrete and nonprestressed al to the distance from the varying proportional neutral axis (referr to Fig. E4.5): 22.2.1.2  $\epsilon$   $\epsilon$  t = c (d - c) c where: c = a and nd a = 1.31A .31As derived previously. viously.  $\beta$ 1 c=  $\epsilon$ t = (1.31)(0.44 in.2) = 0.70 in. 0.825 00.003 9.6 in. - 00.70 in.) = 0.0381 (9.6 0.70 in. 0. It = 0.03 0.0382 > 0.005 005 WLRQ VLRQFRQW 6HFWLRQLVWHQVLRQFRQWUROOHGDQG0.9 = [Fig. E4.5—Strain distribution across stem. American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 12—RETAINING WALLS Shear The closest inclined crack to the support of the cantilevered wall will extend upward from the face of the base reaching the compression zone approximately d from the face of the base. The lateral load applied to the cantilever between the face of the base by compression in the cantilever above the crack. Accordingly, the Code permits design for a maximum factored shear force Vu at a distance d from the support for nonprestressed members. Retaining Walls 7.4.3.2 69 For simplicity, the critical section for design shear strength in this example is calculated at the bottom of the stem: Vu = 1.6[(32.5 pcf)(3 ft)/(10.25 ft) + (32.5 pcf)/(10.25 ft)/(10.25 ft) + (32.5 pcf)/(10.25 ft)/(10.25 ft) + (32.5 pcf)/(10.25 ft)/(10.25 ft) + (32.5 pcf)/(10.25 ft)/(10.25[0.75 s: Therefore, [Vn = [Vc is: (0.75)(15,456 lb)] = 11,600 lb [Vn = (0.75)(15,456 lb)] = 11,600 rejuined American Concrete Institute - Copyrighted Waterial - www.concrete.org70 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Heel design Shear The base heel is designed for shear caused by the superimposed weight of soil, including self-weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed (refer to Fig. E4.6). Fig E4.6—Force on heel. 21.2.1 Strength reduction factor: 7.4.3.2 The critical section for shear strength is taken at a distance d from the face of the stem:  $0.75 = [Vn = (Vc + Vs) = 2\lambda f c'bw d 7.5.3.1 22.5.5.1 with Vs = 0 20.6.1.3.1 d = tbase - cover - db/2 2 5 in. - 3 in. - 0.875 87 in./2 = 11.56 in., say, 11.5 in. 875 d = 15 .1.3 use 3 in. cover to the ten$ From Table 20.6.1.3.1, tenmen and assume No. 6 bars. b. sion reinforcement Then, []Vn is: φVn = ((0.75)(2) 4500 p psi(12 in.)(11.5 in.) = 13,886 lb ete selfA load factor of 1. 1.2 is used for the conc concrete UE OOVHOIZHLJKWDQGIRUEDFN¿OOVHOIZHLJKWDQGVXSHUimposed load of 360 psf. 7.5.1.1 Is [] Vn•VuVDWLV¿HG" Vu  $1.2[(150\ 150\ pcf)(4\ pcf)(4\ oft - 11.5\ in./12)(1.25\ ft)] + 1.6[(120\ pcf)(10.25\ ft) + (360\ psf)] \times (4\ (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute - Copyrighted\ @Material - www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute - Copyrighted\ @Material - www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute - Copyrighted\ @Material - www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute - Copyrighted\ @Material - www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute\ - Copyrighted\ @Material - www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute\ - Copyrighted\ @Material - www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Concrete\ Institute\ - Copyrighted\ @Material - Www.concrete.org\ OK\ CHAPTER\ 12-RETAINING\ WALLS\ Retaining\ Walls\ Flexure (4.0\ ft - 11.5\ in./12) = 684\ lb + 7738\ lb = 8422\ lb\ American\ Am$ 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE\WKHVXSHUimposed weight of soil and self-weight of heel. The soil pressure may not be linear as assumed. Therefore, it is not included in WKHFDOFXODWLRQRIAH[XUHUHIHUWR)LJ( 6.6.1.2 The cantilever wall maximum moment and shear in the heel and toe of the base occur at the stem face. Redistribution of moments cannot occur. Fig. E4.7—Critical moment section in heel. 5.3.1 5.3.8 A load factor of 1.2 is used for the concrete and 1.6 IRUEDFN¿OOVHOIZHLJKWDQGVXSHULPSRVHGORDG 22.2.1.1 Setting C = T Mu1 = 1.2(150 pcf)(1.25 ft)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6[(360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 +
1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2/2 + 1.6](360 psf)(4.0)2 +  $(120 \text{ pcf})(10.25 \text{ ft})](4.0 \text{ ft})2/2 = 1800 \text{ ft}-\text{lb} + 20,352 \text{ ft}-\text{lb} = 22,152 \text{ ft}-\text{lb} = 265,824 \text{ in}-\text{lb} 0.85(4500 \text{ psi})(12 \text{ in}.)(a) = As(60,000 \text{ psi}) = 1.31 \text{ As} (60,000 \text{ psi}) \text{ As} | 1.5 \text{ in} - \langle 2 | / 21.2.1(a) 6WUHQJWKUHGXFWLRQIDFWRUIRUAH[XUH IDFWR 7.5.1.1 0.9 = [] + (120 \text{ pcf})(10.25 \text{ ft})](4.0 \text{ ft})2/2 = 1800 \text{ ft}-\text{lb} + 20,352 \text{ ft}-\text{lb} = 22,152 \text{ ft}-\text{lb} = 265,824 \text{ in}-\text{lb} 0.85(4500 \text{ psi})(12 \text{ in}.)(a) = As(60,000 \text{ psi}) = 1.31 \text{ As} (60,000$  $[]Mn \cdot Mu Mu = 265,825 \text{ in.-lb } n.- = 266 \text{ in.-kip}, calcu calculated ated above. 1.31As ] (60 \text{ ksi}) As [11.5 11 5 \text{ in iin.} - 0.9(60 \ge 266 \text{ in.-kip}, calcu calculated tensile reinforcement is very small compared to the member concrete section. Because the wall is h the h$ structurally determinate, the Code requires that AH[XUDOUHLQIRUFHPHQWDUHDLVDWOHDVWWKHJUHDWHURI (a) As , min ≥ 3 f c' fy bw d Equation (9.6.1.2(a)) controls, because concrete compressive strength fcg SVL 21.2.2 Check if the tension controlled assumption and the use of 0.9 = []s correct. To answer the question, the tensile strain ir UHLQIRUFHPHQWPXVWEHcUVWFDOFXODWHGDQG compared to the values in Table 21.2.2. Concrete and nonprestressed reinforcement strain is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E4.5): 22.2.1.2 et = cc (d - c) c where: c = a and a = 1.31As derived previously.  $\beta$ 1 71 As,min = 3 4500 psi (12) in.)(11.5 in.) = 0.46 in.2 60,000 psi  $\Rightarrow$  As,min = 0.46 in.2 Use No. 7 at 12 in. on center As,prov'd = 0.6 in.2 > As,min = 0.46 in.2 > As,req'd = 0.43 in.2 OK Refer to Fig. E4.8. c= (1.31)(0.60 in.2) = 0.95 in. 0.825 0.003 (11.5 in. - 0.95 in.) = 0.0333 0.95 in. It = 0.0333 > 0.005 ɛt = Section is tension controlled and 0.9 = [American Concrete Institute - 0.0333 0.95 in.] = 0.0333 0.95 in. It = 0.0333 > 0.005 ɛt = Section is tension controlled and 0.9 = [American Concrete Institute - 0.0333 0.95 in.] = 0.0333 0.95 in.] = 0.0333 0.95 in. It = 0.0333 0.95 in.] = 0.0333 0.95 in. Copyrighted © Material - www.concrete.org 72 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E4.8—Heel reinforcement. Step 9: Toe design 7KHWRHLVGHVLJQHGIRUÀH[XUHDQGVKHDUFDXVHGE\ the bearing pressure. The weight of the soil above the toe is usually neglected as it may erode or be removed (refer to Fig. E4.9). 21.2.1 Shear Shear strength reduction factor: Fig. E4.9—Shear at toe side.  $00.75 = [0r shea The critical section for shear strength is taken at a distance d from thee ste stem: 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) = 2\lambda f c'bw d Vn = (2) 4500 0 psi(12 in.)(11.5 in in.) = 18,515 lb with Vs = 0 Therefore []Vn: []Vn = (0 8,515 18,515)(0.75) []b) = 13,886 lb$ The applied force on the toe is from the soil bearing pressure acting upward. 7.4.3.2 Applied shear at distance d from stem: (Refer to Fig. E4.9): [email protected] d = (1932 psf - 622 psf) ( 5.0 ft + 11.5 in./12 ) [ | | + 622 psf = 1700 psf / 2.5 ft 5.3.1 For soil reaction on the toe portion a load factor of Vu = 1.6((1932 psf - 622 psf) ( 5.0 ft + 11.5 in./12 ) [ | | + 622 psf = 1700 psf / 2.5 ft 5.3.1 For soil reaction on the toe portion a load factor of Vu = 1.6((1932 psf + 1700 psf)/2) 1.6 is taken. For soil reaction on the toe portion a load factor of Vu = 1.6((1932 psf + 1700 psf)/2) 1.6 is taken. the self-weight of concrete portion, × (2.25 ft - 11.5 in./12) a load factor of 0.9 is used. - 0.9(150 pcf)(1.25 ft)(2.25 ft - 11.5 in./12) = 3753 lb - 218 lb = 3535 lb 7.5.1.1 Is [Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ Is} []Vn • VuVDWLV2HG" []Vn = 13,886 > Vu = 3535 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} 7.5.1.1 \text{ lb} bearing pressure at face of stem (refer to Fig. E4.9): 7.4.1.1 5.3.8(a) Moment due to soil: 5.3.1 For the self-weight of concrete portion, a load factor of 0.9 is used. Resultant moment: 20.6.1.3.1 22.2.1.1 ( 5.0 ft ) + 622 psf [email protected] stem , L = (1932 psf - 622 psf) | ( 7.25 ft ) = 1525 psf Mu2 = 1.6 [(1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)] = 1525 psf (2.25 ft) 2/2 SVI(SVI IW 2(2/3)) = 1525 psf
(2.25 1.6[(3860 ft-lb) + (1374 ft-lb) = 8374 ft-lb Mu1 = 0.9(150 pcf)(1.25 ft)(2= As(60,000 psi) a= 7.5.2.1 7.5.1.1 21.2.1(a) 73 a) (Mn = f y As | d - | 2/ [Mn • Mu As (60,000 psi) = 1.31As 0.85(4500 psi)(12 in.) 0.85(4 1.31As ) (900 psi)As | 1 1 1 in. - Mn = (6 (60,000 11.5 ) 2 | 1.31As ) (9)(60 ksi)As | 11.5 11 5 in in. - (0.9)(60 = 95.4 in.-kip \ 2 | ) ip calculated late above. Mu = 95.4 in.-kip fer to Fig. ig. E4.10): Solving for As (refer 9.6.1.2 As = 0.1 0.16 in.2 Minimum required reinforcement is calculated from Eq. (9.6.1.2(a)): (a) As , min = 3 f c' fy bw d As , min = 3 assumpWLRQDQGWKHXVHRI[].VFRUUHFW 21.2.2 To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. The strain in concrete and reinforcement is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E4.5): 22.2.1.2 Et = EC (d - c) c where:  $c = a \beta 1$  As, prov'd = 0.60 in.2 > As, min = 0.46 in.2 (Refer to Fig. E4.10.) c = (1.31)(0.60 in.2) = 0.95 in. 0.825 0.003 (11.5 in. - 0.95 in.) = 0.0333 0.95 in. it = 0.0333 > 0.005 Section is tension controlled. Therefore, assumption of 0.9 = []s correct.  $\epsilon t = and a = 1.31$  As OK American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 74 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E4.10—Toe reinforcement. Step 10: Minimum transverse reinforcement. Per Table 11.6.1, d. min = 0.002 As,min = (0.002)(12 in.)(12 in.) = 0.288 in. 2 Zry No. 4 at 16 in. on center n) / 12 in. in. 2 | 0 2 in As, prov = 0.15 in. 2 rov = 0.144 in. 2 / face rcement at front faceace tto Provide verticall rreinforcement sve wall reinforcement. 24.4.3 Base Assume that No. 5 bars or smaller will be used ent. The for temperature and shrinkage reinforcement is bars or smaller will be used ent. adequate. U No 4 at 18 8 in. on ccenter. Use No. As,min = (0.0018)(7.25 ft)(12 in.)(15 in.) = 2.35 in.2 Use eight No. 5 bars distributed. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS Retaining Walls Step 11: Dowels 7.7.1.2 The development length concept is based on the 25.4.2 attainable average bond stress over the embedment lengths are required because of the tendency of highly stressed bars to split relatively thin sections of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development RIUHLQIRUFHPHQWUHLQIRUFHPHQWWHUPLnates, or is bent. For the cantilevered wall, toe and heel reinforcement must be developed. Heel reinforcement is developed beyond the stem critical section. Toe reinforcement must be checked for: DFFRPPRGDWHWKHGHYHO 6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHO 6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked for: DFFRPPRGDWHWKHGHYHO 6XI and the stem must be checked against the stem must be checked against the splice the larger length controls. 25.4.2.1 Development length and 12 in: 25.4.2.2 (f ywt we) 1. A d = | db | 200 f c' | 25.4.2.3 () 3 f y wt wew s | 2. A d = | db | 40 \lambda f c' cb + K tr | | db | 25.4.2.4 In the splice the larger length controls. 25.4.2.2 (f ywt we) 1. A d = | db | 200 \lambda f c' | 25.4.2.3 () 3 f y wt wew s | 2. A d = | db | 40 \lambda f c' cb + K tr | | db | 25.4.2.4 In the splice the larger length controls. 25.4.2.2 (f ywt we) 1. A d = | db | 200 \lambda f c' | 25.4.2.3 () 3 f y wt wew s | 2. A d = | db | 40 \lambda f c' cb + K tr | | db | 25.4.2.4 In the splice the larger length controls. 25.4.2.1 Development length controls. 25.4.2.2 (f ywt we) 1. A d = | db | 200 \lambda f c' | 25.4.2.3 () | 3 f y wt wew s | 2. A d = | db | 40 \lambda f c' cb + K tr | | db | 25.4.2.4 In the splice the larger length controls. 25.4.2.1 Development length controls. 25.4.2.2 (f ywt we) 1. A d = | db | 200 \lambda f c' | 25.4.2.3 () | 3 f y wt wew s | 2. A d = | db | 40 \lambda f c' cb + K tr | | db | 25.4.2.4 In the splice the larger length controls. 25.4.2.4 In the splice the larger length controls. 25.4.2.4 In the splice the larger length controls. 25.4.2.4 In the splice the larger length controls. 25.4.2.4 In the splice the larger length controls. 25.4.2.4 In the splice the larger leng this example Eq. (25.4.2.3a) will be used. zt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ze = coating factor; uncoated zs = bar size factor; No. 7 and larger 75 zt = 1.0, because not more than 12 in. of concrete is placed below bars ze = 1.0, because bars are uncoated zs = 1.0, because No.7 bars are used American Concrete Institute – Copyrighted © Material – www.concrete.org 76 25.4.2.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) cb = spacing or cover dimension to center of bar, whichever is smaller Ktr = transverse reinforcement index It is permitted to use Ktr = 0. But the expression: greater than 2.5. cb + K tr must not be taken db 2.375 in. + 0 = 3.17 > 2.5 0.75 in. Therefore, use 2.5 in Eq. (25.4.2.3(a)). 25.4.2.3 Substituting into Eq. (25.4.2.3(a)). 25.4.2.3 (a)): Development length of heel and toe reinforcement: (3 60,000 psi (1.0)(1.0)(1.0) Ad = | (0.875 in.) 2.5 \ 40 (1.0) 4500 psi / The toe (2 ft) and stem (1ft) have enough space to accommodate the required development length of heel and toe reinforcement: (3 60,000 psi (1.0)(1.0)(1.0) Ad = | (0.875 in.) 2.5 \ 40 (1.0) 4500 psi / The toe (2 ft) and stem (1ft) have enough space to accommodate the required development length of (24 in.) for No.7. Note: the development length of No.7 reinforcement extended from the stem must be checked against the splice length controls. 7.7.1.3 25.5 25.5.1.1 = 23.5 in., say, 24 in. Available length: 27 in. + 12 in. - 3 in. = 36 in. > Ed Extend No. 7 over the full toe length less cover, see Fig. E4.11, therefore, OK Splice ar si The maximum bar size is No.7, therefore, splicing is permitted. ev ment length for N.7 The required development No.7 xt d from the toe is calculated alcula reinforcement extended above as 24 in. Sp ce llength = 1.3(24 in Splice in.) = 31.2 in. > 12 in. Use 2 ft 9 in. for splice length 25.5.2.1 fore, exten Therefore, extend No. 7 toe reinforcement 2 ft 9 in. The deformed bars are
in tension and the ratio into the ste stem. Refer to Fig. E4.11. of provided reinforcement area is less than 2. Therefore, per T Table 25.5.2.1 splice is Type B and the splice length is the greater of (1.3Ed) or 12 in. 7.7.2 25.2.3 Clear spacing between bars is the greater of: (a) 1.5 in. (b) 1.5db (c) 4/3dagg (a) 1.5 in. (b) 1.5(1.0 in.) = 1.3 in. From Step 1, 1 in. maximum aggregate size is used. Therefore, 1.5 in. (c) 4/3(1 in.) = 1.33 in. From Step 1, 1 in. maximum aggregate size is used. Therefore, 1.5 in. (c) 4/3(1 in.) = 1.33 in. From Step 1, 1 in. maximum aggregate size is used. only: No. 7 = 2.044 lb/ft No. 6 = 1.502 lb/ft Total length = (2.75 ft) + 9.0 ft = 11.75 ft Total weight = (2.044 lb/ft/ft)(9.0 ft) = 19.1 lb/ft Fig. E4.11—Retaining wall reinforcement. inf ment. American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls Step 12: Details 78 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 13: Alternate solution The engineer may wish to extend the No.7 reinforcement coming from the base/toe higher into the stem reinforcement. Assume that No.7 bars are extended a distance 5 ft 6 in. into the stem. The factored applied moment at that level can be calculated from: Mu = CaUsho3/6 The required reinforcement area: Use bar size such that the required reinforcement area: Use bar size such that the required in Table 11.6.1. Mu = CaUsho3/6 The required in Table 11.6.1. Mu (1.6)[(0.271)(120 pcf)(10.25 ft - 2.0 ft)3/6 + (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(3 ft)(10.25 ft - 2.0 ft)2/2] = 10,182 ft-16 As,req'd = 0.24 in.2/ft Vse No.5 (0.271)(120 pcf)(10.25 ft - 2.0 ft)2/2therefore splicing is permitted. 25.5.2.2 n, the If different bar si sizes aree spliced in tens tension, then the the larger er ba bar and Estt of the he sm smaller greater of Ed of th he splicee length. bar is used for the 25.5.2.1 The deformed bars are in tension and the ratio of provided reinforcement area to required reinforcement area is less than 2. Therefore, per Table 21.5.2.1 splice is Type B and the splice length is the greater of (1.3 Ed) or 12 in., where Ed is the develop Ad =  $| (0.625 \text{ in.}) 2.5 \setminus 40 (1.0) (1.0)(1.0) \rangle$  ment length of No.7 to splice length of No. 5 Est = 1.3(17.0 in.) = 22.1 in. Use Ed = 24 in. > Est = 22.1 in. use 24 in. See Fig. E4.12. Extend No.7 @ 12 in. o.c. dowels coming from the toe 4 ft 0 in. into the stem and splice the No. 7 bars with the No.5 bars. The overlap length is 24 in. (Step 11). American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 12—RETAINING WALLS 79 Weight of steel per lineal foot in stem only: No. 5 = 1.043 lb/ft No. 7 = 2.044 lb/ft/ft)(4 ft) + (1.043 lb/ft/ft)(7 ft) = 15.5 lb/ft Fig. E4.12—Retaining wall reinforcement in ment (alternate) (alternate) Conclusion: 5 5 lb/ft lb ( 15.5 2SWLRQ%SURYLGHVDVDYLQJRIOEIW ± OEIW OEIWRU | 1 – pe | = 19 percent. ( 19.1 lb/ft /ft ft ) American Concrete.org Retaining Walls Step 14: Details (alternate solution) 80 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Retaining Walls Example 5—Reinforced concrete cantilever wall retaining earth with surcharge load 'HVLJQDQRUPDOZHLJKWUHLQIRUFHGFRQFUHWHFDQWLOHYHOHDUWKEDQNIWKLJKDERYHWKH¿QDOHDUWKOHYHO and supporting a storage area rated at 330 psf as shown in Fig. E5.1. The concrete mixture must satisfy durability and strength requirements. The grade of reinforcing bar is assumed at 60,000 psi. The soil data were obtained from the geotechnical report: VRLOXQLWZHLJKWRISFIDQJOHRIVRLOLQWHUQDOIULFWLRQRIGHJUHHVDQGFRHI¿FLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLO and between soil and soil is 0.4 and 0.7, respectively Allowable soil bearing pressure obtained from the geotechnical report is given at 3000 psf. \$VVXPHWKDWWKHFDQWLOHYHUZDOOLVQRWVXEMHFWHGWRDQ\RWKHUORDGDQGWKHIURVWOLQHLVIWLQEHORZWKH¿QLVKHGJUDGH Given: Soil data— Û s = 110 pcf [GHJUHHV 3 concrete-soil = 0.4 3 soil-soil = 0.7 qall = 3000 psf Concrete-300 psf Concretea = 1.0 normalweight concrete fy = 60,000 psi Cantilever wall— h1 = 12 ft h2 = 2.5 ft Load— w = 330 psf Fig. E5.1—Cantilever Fig 1— lever reta retaining wall. ACI 318-14 Discussion Step 1: Material requirements 7.2.2.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements. The designer determines the durability classes. Please see Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. Calculation By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVWSVL \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 301, such as admixtures and pozzolans, which the designer can require, permit, or review if suggested by the contractor. Example 1 of this Chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. American Concrete Institute – Copyrighted © Material – www.concrete.org Step 2: Equivalent lateral pressure The geotechnical report provides the equivalent lateral pressure the wall is required to resist. Step 3: Preliminary cantilever wall data General criteria The preliminary retaining wall dimensions are determined from guidelines, Inc., 1996. Figure E4.2 shows the variables used in the example. p = 29.8 pcf The storage area applies uniform loading behind the retaining wall that is converted to an equivalent, imaginary earth height is: h = h1 + h2 = 12 ft + in. 110 11 psf tbase = 0.07(17.5 ft) = 1.23 ft, say, 1 ft 4 in. Assuming 3 in. cover, the effective depth is greater than 6 in. OK /HQJWKRIEDVHWRHKHHO WR h bbase = 0.45(17.5 ft) = 7.875 ft, say, 2 ft 3 in. Heel length: bheel = 8 ft - 2.25 ft - 1.25 ft = 4.5 ft American Concrete Institute - Copyrighted © Material - www.concrete.org 81 Retaining Walls CHAPTER 12—RETAINING WALLS 82 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Applied forces For out-of-plane moment and shear, the cantilevered retaining wall is assumed to be continuous, and a representative one foot strip is analyzed for the maximum load effects. Refer to Fig. E5.3 for YDULDEOHLGHQWL¿FDWLRQ 9HUWLFDOORDGV weight of the soil ding the storage area uniform above the heel, including ght oover the toe is neglected as it load. The soil weight oved: may erode away or bbe removed: Fig. E5. E5.3—Applied Applied forc forces on retaining wall. Stem wall: P1  $\hat{U}$ conc(tstem tem)(h (tbase) Base: P2 = qconc(tbase)(b)) Soil: P3  $\hat{U}$ s(h + hs (tbase se)(bheel) P1 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 (14.5 ft + 3 ft - 1.33 ft)(4.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 (14.5 ft + 3 ft - 1.33 ft)(4.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 (14.5 ft + 3 ft - 1.33 ft)(4.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 (14.5 ft + 3 ft - 1.33 ft)(4.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1 ft)(14.5 ft) P3 = (15 (150 pcf)(1.25 25 ft)(1
ft)(14.5 ft) P3 = (15 (150 pcf)(14.5  $(11\ 80\ lb = 8000\ Total\ vertical\ load: {}^{M}P = 2470\ lb + 1600\ lb + 8000\ lb = 12,070\ lb\ the\ self-weight\ of\ the\ retaining\ wall\ and\ the\ soil\ and\ distributed\ load\ above\ the\ heel\ tend\ to\ counteract\ the\ overturning\ moment.$ =  $(2470 \text{ lb})(2.25 \text{ ft} + 1.25 \text{ ft}/2) = 7100 \text{ ft-lb} \text{ M2} = (1600 \text{ lb})(8 \text{ ft}/2) = 6400 \text{ ft-lb} \text{ M3} = (8000 \text{ lb})(8 \text{ ft}/2) = 46,000 \text{ ft-lb} + (46,000 \text{ ft-lb}) + (6400 \text{ f$ (h2/2) H1 = (29.8 psf) (14.5 ft)2/2 = 3130 lb Due to surcharge: H2 = (Ca Ûs)(hs)(h) H2 = (29.8 psf) (3 ft)(14.5 ft) = 1300 lb American Concrete Institute – Copyrighted © Material – www.concrete.org 83 Ca is obtained from Rankine's formula found in soils engineering handbooks. Therefore, this lateral force tends to overturn the retaining wall about the front edge of the toe: MOTM =  $(3130 \text{ lb/ft})(14.5 \text{ ft/3}) + (1300 \text{ lb/ft})(14.5 \text{ ft/2}) = 24,550 \text{ ft-lb} - (24,550 \text{ ft-lb}) - (24,550 \text{ ft-lb}) = 34,950 \text{ ft-lb} - (24,550 \text{ ft-lb}) = 34,950 \text{ ft-lb} - (24,550 \text{ ft-lb}) = 34,950 \text{ ft-lb} + (1300 \text{ lb/ft})(14.5 \text{ ft/3}) + (1300 \text{ lb/ft$ aforementioned determined cantilever wall base is checked using unfactored forces and allowable soil bearing pressure. To calculate soil pressure, the location of the vertirmined. cal resultant force must be determined event thee resu resultant id-length: location and the ba base mid-length: e = bbase/2 - a Check if resultant falls within the middle third of the base. Maximum and minimum soil pressure:  $q_{1,2} = \sum P \sum M \pm A S 34$ , 34,950 ft-lb lb = 2.9 ft 12,070 lb 12 e = 8 ft/ ft/2 - 2.9 9 ft = 1.1 1 ft bbbase 8 ft f = 1.33 ft > e = 1.1 ft 6 6 Therefore, there is no uplift.  $q_{1,2} = 12,070$  lb  $(12,070 \text{ lb})(1.1 \text{ ft}) \pm (8 \text{ ft})(1 \text{ ft}) \pm (8 \text{ ft})(2
\text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})(2 \text{ ft})($ DESIGN HANDBOOK—SP-17(14) Step 6: Stability requirements Calculate the factor of safety against overturning: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  OTM FS = 59,500 ft-lb = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  Other FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS =  $\Sigma MR \ge 2.0 \text{ M}$  Other FS = 2.42 > 2.0 OK Calculate the factor of safety against sliding: FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 = 2.0 OK Calculate the factor of safety against sliding: FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 = 2.0 OK Calculate the factor of safety against sliding: FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 = 2.0 \text{ M} Other FS = 2.42 neglects the passive pressure against the toe (conservative). There is not enough frictional resistance to counteract the effects of the surcharge load and the EDFN¿OOODWHUDOIRUFHFRPSRQHQWVDJDLQVWVOLGLQJ Two solutions: 1. Increase the heel dimensions (length and maybe thickness) to increase the effect of gravity loads he sliding force. and thus increase resistance to the 2. Provide shear key to engage the soil and to essure aat the shear key large develop a passive pressure he late anough to resist the lateral force. XWL HKHHOOHQJWKDQ WKLFNQHVV )RUWKH¿UVWVROXWLRQWKHKHHOOHQJWKDQ WKLFNQHVV as to should be increased 10 ft 6 in. and 1 fft 9 in. to satisfy the st soliding g and to satisfy the he shear 1.5 factor against vely demand, respectively. In this example, the shear key solution approach is chosen. American Concrete org CHAPTER 12-RETAINING WALLS Retaining Walls Shear key is constructed at 3 ft 0 in. from the toe and has the following dimensions: Depth: 1 ft 3 in. Width: 1 f = 85 1 + sin  $\varphi$  1 1 = = 3.69 1 - sin  $\varphi$  Ca 0.271 Fig. E5 E5.4—Forces 4—Forces 4—Forces 4—Forces acting ctin tin on R-Wall with shear key. American Concrete Institute – Copyrighted © Material – www.concrete.org 86 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) /DWHUDOSDVVLYHSUHVVXUH 1 C p y s (tbase + hkey) 2 2 The sliding occurs at the bottom surface of the foundation along the path Hp = Hp = 1 (3.69)(110 pcf)(1.33 ft + 1.25 ft) 2 = 1351 lb 2 a-c + c-d + e-f (refer to Fig. E5.5). (a-c) = Eac = 3.0 ft (c-d) + (e-f) = Ecf = 5.0 ft Fig. E5.5 ). between conThe sliding along Path c-d + e-f IULFWL FUHWHDQGVRLOZLWKIULFWLRQFRHi FLHQW & DOFXlate soil pressure at P Point c. 2755 psf - 265 psf 8 ft qc = 1821 182 psf sf qc = ist force P: The friction resistance Pac = qa + qc A ac 2 Pac = Pcd+ef mP - Pac 22755 psf + 1821 ppsf (3 ft) = 6864 lb 2 Pcd+ef = 6864 lb 2 Pcd+ef = 6864 lb 2 Pcd+ef 12,070 lb - 6864 lb = 5206 lb 2ULWFDQEHFDOFXODWHGIURP Pcd + ef = qc + qf 2 A cd + ef Pcd + ef = 1821 psf + 265 psf (8 ft - 3.0 ft) = 5215 lb 2 Along Path cf, sliding occurs within soil layers ( $\mu = 0.7$ ), while along Path cf, sliding occurs within soil layers ( $\mu = 0.7$ ), while along Path cf, sliding occurs between concrete and soil ( $\mu = 0.4$ ) (refer to Fig. E5.5). Approximately equal. Difference can be attributed to rounding of results. P 31Pac32Pcd+ef P = (0.7)(6864 lb) + (0.4)(5215 lb) = 6891 lb Due to the shear key, the imposed lateral pressure increases. Due to retained earth: H1 = CaÛs(h + hkey)2/2 H1 = (29.8 \text{ pcf}) (3 \text{ ft})(14.5 \text{ ft} + 1.25 \text{ ft})2/2 = 3696 \text{ lb} Due to surcharge: H2 = CaÛshs (h + hkey)2/2 H1 = (29.8 \text{ pcf}) (3 \text{ ft})(14.5 \text{ ft} + 1.25 \text{ ft})2/2 = 3696 \text{ lb} Due to retained earth: H1 = CaÛs(h + hkey)2/2 H1 = (29.8 \text{ pcf}) (3 \text{ ft})(14.5 \text{ ft} + 1.25 \text{ ft})2/2 = 3696 \text{ lb} Due to surcharge: H2 = CaÛshs (h + hkey)2/2 H1 = (29.8 \text{ pcf}) (3 \text{ ft})(14.5 \text{ ft} + 1.25 \text{ ft})2/2 = 3696 \text{ lb} Due to surcharge: H2 = CaÛshs (h + hkey)2/2 H1 = (29.8 \text{ pcf}) (3 \text{ ft})(14.5 \text{ ft} + 1.25 \text{ ft})2/2 = 3696 \text{ lb} ft) = 1408 lb FS = µ1 PAC + µ 2 PCF + H p SH FS = 6891 lb + 1351 lb = 1.61 > 1.5 3696 lb + 1408 lb H p = 1351 lb calculated above. American Concrete.org OK Conclusion The retaining wall preliminary dimensions are adequate to resist overturning, sliding, preventing uplift, and limiting pressure on the soil to less than the allowable provided soil pressure in the geotechnical report. But the length of the retaining wall is not adequate to prevent sliding. Therefore, a shear key is provided. In the following steps, the retaining wall is designed for strength. If any of the aforementioned determined dimensions are not satisfactory, then all the previous steps must be revised. Note: Unfactored loads were used to determine the stability of the retaining wall and to calculate the soil pressure. American Concrete Institute - Copyrighted © Material - www.concrete.org 87 Retaining Walls CHAPTER 12-RETAINING WALLS 88 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 7: Stem design The cantilevered concrete stem is a determinate member and is modeled as a 1 ft wide cantilever beam. 13.2.7.1 Flexure The maximum design moment in the stem is calculated at the face of the base foundation (refer to Fig. E5.6). 9HUWLFDOUHLQIRUFHPHQWLQWKHVWHPUHVLVWVWKHODWHUDO earth pressure and is placed near the face of the stem wall that is against the retained soil. Adequate concrete cover protects reinforcement against moisture changes in soil. Cover is measured from the concrete surface to the outermost surface of the reinforcing bar. Fig. E5.6—Soil la lateral fforce on stem. Stem height: hstemm = ((12 ft) + (2.5 ft) - (1.33 ft) = 13.17 ft 20.6.1.3.1 6. use 2 in. cover. From Table 20.6.1.3.1, 21.2.1 er is tension con olled; steel Assume that the m member controlled; QG VWUDLQIt DQG[Equivalent lateral pressure due to surcharge load: 7.4.1.1 5.3.8(a) /RDGIDFWRU U = Hu = 1.6(H1\* + H2\*) when lateral pressure due to surcharge load: 7.4.1.1 5.3.8(a) /RDGIDFWRU U = Hu = 1.6(H1\* + H2\*) when lateral pressure acts alone. 7.5.1.1 6DWLVI\[]Mn•Mu 22.2.2.1 The concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strength. For
calculating nominal strength, ACI 318 neglects the concrete tensile strength. 22.2.3 Determine the equivalent concrete compressive stress for design. Psurcharge harge = (29.8 pcf)(3 ft) = 90 psf Hu = (1.896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) + (4135 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb Mu = (1896 lb)(13.17 ft/2) = 1896 lb + 4135 lb = 6031 lb =stem's self-weight restoring moment (conservative) American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 12-RETAINING WALLS 22.2.2.4.3 22.2.1.1 The concrete compressive stress distribution is inelastic at high stress. The actual distribution of concrete compressive stress distribution is inelastic at high stress. explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of ultimate strength in reasonable agreement with the results of comprehensive stress distribution of 0.85 fco ZLWKDGHSWKRI a  $\hat{u}1$  cZKHUH $\hat{u}1$  is a function of concrete compressive strength and is obtained from Table 22.2.4.3. For fco''SVLZKHUH $\beta 1 = 0.85 - 0.05(4500 \text{ psi} - 4000 \text{ psi}) = 0.825 1000 \text{ psi}$  Find the equivalent concrete compressive rectangle depth, a, by equating the compressive rectangle depth, a by equating the compressive rectangle depth. wall cross section: C = T where C = 0.85fcgba and  $T = AsTy 0.85(4500 \ 5(4500 \ psi)(12 \ in.)a)a = As(60,000 \ psi)(12$ 0.56 in.2 Use No. 7 at 12 in. on center. As, provided = 0.60 in.2/ft > As, req'd = 0.56 in.2 Solving for As: 9.6.1.2 OK The cantilevered retaining wall calculated required tensile reinforcement is usually very small compared to the member concrete section. The stem reinforcement is checked against the beam minimum UHTXLUHGÀH[XUDOUHLQIRUFHPHQWDUHDUDWKHUWKDQWKH one-way slab minimum reinforcement area is at least: (a) As , min = 3 f c' fy bw d As , min = 3 f c' f Material - www.concrete.org 90 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Equation (9.6.1.2(a)) controls, because concrete compressive strength fcg!SVL 21.2.2 Check if the tension controlled assumption and the XVHRI[LVFRUUHFW 22.2.1.2 To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH&UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Assume concrete and nonprestressed reinforcement strain varying proportional to the distance from the neutral axis (refer to Fig. E5.7):  $\epsilon t = \epsilon c (d - c) c$  where: c = a and  $a = 1.31As \beta 1 As$ , prov. = 0.6 in.2 > As, min = 0.5 in.22. c = (1.31)(0.60 in.2) = 0.95in. 0.825 0.003 (12.5 in. - 0.95 in.) = 0.037 0.95 in.] = 0.037 0.95 in.] = 0.037 > 0.005  $\epsilon$ t = 6HFWLRQLVWHQVLRQFRQWUROOHGDQG extend upward from the face of the base reaching the compression zone approximately d from the face of the base. The lateral load applied to the cantilever between the face of the base and the point d away from the face of the base. maximum factored shear force Vu at a distance d from the support for nonprestressed members. For simplicity, the critical section for shear strength in this example is calculated at the bottom of the stem: Retaining Walls 7.4.3.2 91 Vu = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(12)/2] = 5242 lb 7.5.3.1 22.5.5.1 Vn = 1.6[(29.8 pcf)(3 ft)(13.17 ft - 12.5 in)(13.17 $(Vc + Vs) = 2\lambda f c'bw d 21.2.1$  Shear strength reduction factor: 7KHUHIRUHUVn is: (0.75)(20,125 lb) = 15,093 lb > Vu = 5242 5 lb (Vn = (0.75)((0.75)(20,125 lb) = 15,093 with Vs = 0 Vn = 24500 psi(12 in.) = 20,125 lb OK ar re ement is not required. Shear reinforcement American Concrete Institute – Copyrighted © Material – www.concrete.org 92 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Heel design of heel. The soil pressure
counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed or may not be present. Therefore, conservatively, it is not included in the calculation of shear strength. Fig. E5.8—Force on heel. 7.4.3.2 The critical section for shear strength is taken at a distance d from the bottom of the stem (refer to Fig. E5.8): 7.5.3.1 22.5.5.1 Vn =  $(Vc + Vs) = 2\lambda f c'bw d Vn = Vc = 2(1.0) 4500 psi(12 in.)(12.5)$ in.) = 20,125 lb with Vs = 0 d = tbase - cover - db/2 d = 16 6 iin. - 3 in. - 0.875 in./2 = 12.56 in., say, 12.5 in. 20.6.1.3.1 6. use 3 in. cover to tensi From Table 20.6.1.3.1, tension ume No. 7 bars. reinforcement an and assume 21.2.1 duc factor: Shear strength reduction 7 7.4.3.2 [HUHIRUH] Vn is: Vn = 15,093 llb A load factor of 1.2 is used for the concrete and 1.6 IRUWKHVXUFKDUJHORDGDQGEDFN $\stackrel{:}{\circ}OOVHOIZHLJKW$  Vu = (1.2)(150 pcf)(4.5 ft - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$  - 12.5 in./12) + (330 psf)(4.5 ft - 12.5 in./12)] = 828 lb + 8016 lb + 1826 lb = 10,670 lb /UUVDWLV $\stackrel{:}{\circ}H$ American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 12—RETAINING WALLS 6.6.1.2 The cantilever wall is a determined system. Therefore, maximum moment and shear in the heel and toe of the base occur at the stem face and at distance, d, from stem face, respectively. Redistribution of moments does not occur. 5.3.1 5.3.8 A load factor of 1.2 is used for the concrete selfZHLJKWDQGIRUVRLOEDFN¿OODQGVXSHULPSRVHG load. 8004 lb is the weight of soil and self-selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and T = fyAs 22.2.1.1 Retaining Walls Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE] with the selfzed and Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXV weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed or may not be linear as assumed or may not be present. Therefore, conservatively, it is not included in the calculation RIÀH[XUH Mu1 =  $1.2(150 \text{ pcf})(1.34 \text{ ft})(4.5)2/2 + (1.6)(8000 \text{ lb/ft})(4.5 \text{ ft})/2 = 2442 \text{ ft-lb} + 28.800 \text{ ft-lb} = 31,242 \text{ ft-lb} \approx 375,000 \text{ in.-}$ lb (0.85)(4500 psi)(12 in.)(a) = As (60,000 psi) a = As (60,000 psi) (60, = 1.31As 0.85(4500 psi)(12 0.8 1 2 // Mu = 375 in.-kip, calculated above. Solving for As: 9.6.1.2 As, req'd = 0.57 in.2 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDÀH[ural member concrete section. Also, the cantilevered retaining wall is a statically determinate member with no possibility for moment redistribution. Because the wall is structurDOO\GHWHUPLQDWHWKH&RGHUHTXLUHVWKDWWKHÀH[XUDO reinforcement area is not less than the greater of: (a) As, min = 3 f c' fy bw d 93 As ,
min = 3 f c' fy bw d 93 As , min = 3 f c' fy bw d 93 As , min = 3 f c' fy bw d 93 As , min = www.concrete.org 94 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Use No. 7 at 12 in. on center (refer to Fig. E5.7). As,prov'd = 0.60 in.2 > As,reg'd = 0.57 in.2 21.2.2 22.2.1.2 Check if the tension controlled assumption and the XVHRI/LVFRUUHFW To answer the question, the tensile strain in reinIRUFHPHQWPXVWEHiUVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Concrete and nonprestressed reinforcement strain is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E5.7):  $\epsilon t = \epsilon c (d - c) c$  where: c = a and a = 1.31As  $\beta 1$  OK 2 c = (1.31)(0.6 in.) = 0.95 in. 0.825 0.003 (12.5 in. -0.95 in.] = 0.036 0.95 in. It = 0.036 > 0.005 ct = 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Step 9: Toe and shear key design 7KHWRHLVGHVLJQHGIRUÀH[XUHDQGVKHDUFDXVHGE] the bearing pressure. he toe is usually The weight of the soil above the de or be rem neglected as it may erode removed. 21.2.1 7.5.3.1 22.5.5.1 Shear edu factor: Shear strength reduction  $\square$  to for shear strength is taken at a The critical section m: distance d from the soil il bearing Shear ccalculation is similar to the shear calculation for pressure acting upward. heel. See Step 8 for heel shear

calculation. 7.4.3.2 Applied shear at distance d from stem: [email protected] d =  $(2755 \text{ psf} - 265 \text{ psf})(4.5 \text{ ft} + 12.5 \text{ in.}/12) + (1.25 \text{ ft} - 12.5 \text{ in.}/12) - (0.9)(150(1.33 \text{ ft})(2.25 \text{ ft} - 12.5 \text{ in.}/12) + (1.25 \text{$ concrete portion, = 4964 lb - 217 lb = 4747 lb OK a load factor of 0.9 is used. 7.5.1.1, V[Vn • VuVDWLV2HG" [Vn = 15,093 lb > Vu = 4747 lb OK Shear reinforcement is not required. American Concrete Institute - Copyrighted © Material - www.concrete.org 7.4.1.1 5.3.8(a) Flexure Soil bearing pressure at the face of stem: Moment due to soil load: 95 (5.75 ft) + 265 psf [email protected] stem , L = (2755 psf - 265 psf) | (8 ft) = 2055 psf Mu2 = 1.6 [(2055 psf (2.25 ft)2/2 SVI(SVI IW 2 (2/3)] = 1.6(5202 \text{ ft})2/2 = 454 \text{ ft}) = 12,104 \text{ ft} + 2363 \text{ ft}) = 12,104 \text{ ft} + 2363 \text{ ft} + 10 \text{ sc} = 12,104 \text{ ft} + 2363 \text{ ft} + 10 \text{ sc} = 1.6(5202 \text{ ft})2/2 \text{ sc} = 454 \text{ ft} + 10 \text{ sc} = 1.6(5202 \text{ ft})2/2 = 454 \text{ ft} + 10 \text{ sc} = 1.6(5202 \text{ ft})2/2 \text{ sc} = 1.6  $b_{1} = (454 \text{ ft-lb}) = 11,650 \text{ ft-lb} = 140,000 \text{ in.-lb}$  Concrete is placed against earth. Therefore, use 3 in. cover. Setting C = T to calculate required reinforcement d = 16 in. - 3 in. - 0.44 in. = 12.56 in., say, 12.5 in. (0.85)(4500 \text{ psi})(12 in.)(a) = As (60,000 \text{ psi}) = 1.31 \text{ As } 0.85(4500 \text{ psi})(12 in.) 7.5.2.1 a) (Mn = f y \text{ As } | d - | (2/1.31 \text{ As } )(2/1.31 \text{ As } )(2/1.31 \text{ As } )(2/1.31 \text{ As } )(3/1.31 \text{  $M n = (60,000 \ (60,00 \ psi)As | 12.5 \ in. - \langle 2 | / 21.2.1 \ Flexural strength reduction educt factor [] [] Mn \cdot Mu 1.31 As \rangle (9) As (60 \ ksi) | 12.5 \ 12.5 \ in. - \langle 0.9 \rangle = 140 \ in.-kip \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 As = 0.2 \ 0.21 \ in.2 \ 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDAH[ural member. The tensile \ calculated ated above. Solve for As: 9.6.1.2 \ As = 0.2 \ 0.21 \ in.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ 0.21 \ in.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ 0.21 \ in.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1.2 \ As = 0.2 \ red (above. Solve for As: 9.6.1$ reinforcement amount is very small compared to
the member concrete seci tion. Also, the cantilevered retaining wall is a statically determinate member with no possibility for moment redistribution. To prevent a sudden failure, WKH&RGHUHTXLUHVWKDWWKHÀH[XUDOUHLQIRUFHPHQW area is not less than the greater of: (a) As,min = 3 f c' fy bw d As , min = 3 4500 psi (12 in.)(12.5 in.) = 0.5 in.2 60,000 psi  $\Rightarrow$  As,min = 0.50 in.2 Use No.7 at 12 in. on center As,prov'd = 0.60 in.2 > As,req'd = 0.60 in.2 > As,req'd = 0.50 in.2 American Concrete Institute – Copyrighted © Material – www.concrete.org OK Retaining Walls CHAPTER 12—RETAINING WALLS 96 THE REINFORCED CONCRETE DESIGN HANDBOOK– SP-17(14) 21.2.1 Check if the tension controlled assumption and the XVHRI to the values in Table 21.2.2. To answer the question, the tensile strain in concrete and reinforcement is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E5.7): 21.2.1.2  $\varepsilon$  (d - c) c where: and a = 1.31As  $\varepsilon$ t = c = (1.31)(0.60 in.2) = 0.95 in. 0.825  $\varepsilon$ t = 0.003 (12.5 in. - 0.95 in.) = 0.036 0.95 in.] considered, as this will apply higher force on the shear key and it may not be present. HGSUHYLRXVO\ 3DVVLYHFRHI¿FLHQWZDVFDOFXODWHGSUHYLRXVO\ 2 = 359 lb 2 Hp = 1 (3.69)(11 111 )(1.33 ft + 1.25 ft) 2 = 1351 lb 2 alculate at top and bottom of Passive pressure is calculated shear key: Y: Top of shear key: Hp = 1 (C p)( $\gamma$  s))((thase) 2 2 Bottom of shear key: Hp = 1 (C p)( $\gamma$  s)(hase + hkey) 2 2 (359 lb + 1351 lb) Vu = (1.6) | |) (1.25 ft) = 1710 lb (2 7.4.3 5.3.1 Required shear design: 16.5.2.4 Check corbel dimensions if they satisfy the following requirements: VuPXVWQRWH[FHHGWKH least of: (a) 0.2fcgbd (b) (480+0.08fcg bd (c) 1600bd (a) 0.2(4500 psi)(12 in.)(11.56 in.) = 124,848 lb (b) (480 + 0.08(4500 psi))(12 in.)(11.56 in.) = 122,000 lb (c) (1600)(12 in.)(11.56 in.) = 222,000 lb (c) (1600)(12 in.)(11.56 in.) = 124,848 lb (c) (11.56 in.)shear resistance is calculated from Eq. (22.9.4.2). 16.3.3.5 22.9.4.2 21.2.1 22.9.3.1 Vs  $_3Avf$  fy ZKHUH3 33 DQG & KHFNLI Vn  $_2$  (22.9.4.2) Vs = (1.0)(0.6 in.2)(60,000 psi) = 36,000 lb Use 2 No. 5 continuous reinforcement at bottom of shear key. Vs = (1.0)(0.6 in.2)(60,000 psi) = 36,000 lb Use 2 No. 5 continuous reinforcement at bottom of shear key. Step 10: Minimum transverse reinforcement Stem 11.6.1 Assume that No. 5 bars or smaller will be used for temperature and shrinkage and temperature reinforcement (As,min) front = (0.002)(12 in.)(15 in.) = 0.36 in. 2/2 = 0.18 in. 2front and back face of the stem wall. (12 in.) As , prov. = 0.31 in.2 = 0.21 in.2 \ 18 in.] > As , req ' d = 0.18 in.2 / face Use No. 5 spaced at 18 in. on center placed horizontally. 24.4.3 Provide vertical reinforcement at front face to support the horizontal shrinkage and temperature reinforcement. Base Assume that No. 5 bars or smaller will be used for temperature and shrinkage reinforcement. The reinforcement can be located at the top, bottom, or o faces. allocated between the two Use No. 5 at 18 in. on center. As,min = (0.0018)(8ft)(1.33 ft)(12 in./ft)2 = 2.76 in.2 Equivalent to nine No. 5 bottom at the top, bottom, or anywhere within the base. Therefore, place three No. 5 bottom at the top a the toes end and six No. 5 top at the nd heel end. Step 11: Dowels 7.7.1.2 The development based on the nt llength concept is bas age bond d stress over the embedment 25.4.2 attainable average orc t. length of reinforcement. gth aree required beca Development based on the nt llength concept is bas age bond d stress over the embedment 25.4.2 attainable average orc t. length of reinforcement. gth aree required beca Development based on the nt llength concept is bas age bond d stress over the embedment 25.4.2 attainable average orc t. length of reinforcement. gth aree required beca Development based on the nt llength concept is bas age bond d stress over the embedment 25.4.2 attainable average orc t. length sections of restraining concrete. In application, the development length concept i requires minimum lengths or extensions off reinforcement. Such peak stresses generally occur at the points of critical sections for development RIUHLQIRUFHPHQWLQAH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLnates, or is bent. For the cantilevered wall, toe and heel reinforcement must be properly developed. Heel reinforcement must be checked for: 6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHOopment length in the to e section. 2. Extension of toe reinforcement in the stem to satisfy the required development length. Note: the development length and the larger length controls. American Concrete Institute
– Copyrighted © Material – www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 98 25.4.2.1 25.4.2.2 25.4.2.3 25.4.2.4 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Development length,  $\mathcal{E}d$ , is the greater of Eq 25.4.2.3 of ACI 318-14 and 12 in.: (f y t y e) 1. A d = | db | 40  $\lambda$  f c' b + K tr || db |/ In this example, Eq. (25.4.2.3a) will be used.  $z_t = bar$ location ze = coating factor zs = bar size factor zt = 1.0; because not more than 12 in. of fresh concrete is placed below horizontal reinforcement index It is permitted to use bars used are No. 7 cb = spacing or cover dimension to center of bar, whichever is smaller Ktr = transverse reinforcement index It is permitted to use Ktr = 0.25.4.2.3 press However, the expression: an 22.5. taken greater than cb + K tr must not be db 2.44 4 in in. + 0 = 2.79 > 22.5 0.875 875 in. Therefore, refo use 2.5 in Eq. (25.4.2.3a): 5.4. (0.875) 2.5 / 40 (1.0) 4500 psi = 23.5 in., say, 24 in. The available length for toe reinforcement (2 ft 3 in.) is long enough to accommodate the required development length of stem must be checked against the splice length of stem reinforcement and the larger length controls. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 25.5.2.1 25.2.3 Splice The maximum bar size is No. 7, therefore splicing is permitted. The required development length for No. 7 extended from the toe is calculated previously as 24 in. The deformed bars are in tension and the ratio of provided reinforcement area to required reinforcement area is less than 2. Therefore, per Table 25.5.2.1, splice is Type B and the splice length is the greater of: (a) 1.5 in. (b) 1.5db (c) 4/3dagg Assume, 1 in. maximum aggregate size is used. Step 12: Details Retaining Walls 7.7.1.3 25.5 25.5.5.1 99 Splice length = 1.3(24 in.) = 31.2 in. > 12 in. = 31.2 in. > 12 in. loss 2 ft 9 in. in. for splice length. Therefore, extend No. 7 to e reinforcement 2ft 9in. into the stem. Refer to Fig. E5.9. (a) 1.5 in. (b)  $1.5(0.875 \text{ in.}) = 1.33 \text{ in.} \text{ (c) } 4/3(1.0 \text{ in.$ Total weight = ((2.044 lb/ft/ft) (15.84 ft) = 32.4 lb/ft Fig. E5.9—Retaining wall reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org 100 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 13: Alternate solution The engineer may wish to extend the No. 7 reinforcement coming from the base/toe higher into the stem to a level where smaller bar diameters, larger bar spacing, or both, can be used for the stem reinforcement. Assume that level can be calculated from: Mu = CaUsho3/6 The required reinforcement area: Mu = (1.6)[(0.271)(110 pcf)]  $(13.17 \text{ ft} - 4.5 \text{ ft})3/6 + (0.271)(110 \text{ pcf})(3 \text{ ft})(13.17 \text{ ft} - 4.5 \text{ ft})2/2] = 10,559 \text{ ft}-\text{lb} \text{ As,req'd} = 0.19 \text{ in}.2/\text{ft} \text{ Use bar size such that the required calculated spac-Use No. 5 at 12 in. on center ing matches the spacing of No. 7 extended from the As,prov = 0.31 in.2/\text{ft} > \text{As,req'd} 0.19 in.2/\text{ft} \text{ base. } 11.6.1 \text{ WDUHDVDWLV} + 10.00 \text{ WLV} + 10.00 \text{ WLV} + 10.00 \text{ Hc}$ &KHFNLISURYLGHGUHLQIRUFHPHQWDUHDVDWLV¿HVWKH ble 11.6.1 minimum required in Table 7.7.1.3 ngth: Required splice length: 25.5.1.1 The maximum bar size is No. 7, theref therefore splicing is permitted. n, th If different bar siz sizes aree spliced in tens tension, then the he larger er bar and Estt of thee smaller greater of Ed of the bar is used for the sp splice length. 25.5.2.2 25.5.2.1 The deformed bars are in tension and the ratio of provided reinforcement area to required reinforcement area to required reinforcement area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice
length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., where & Content area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3&C) or 12 in., whe Development of No. 5 bar OK 0.0012(12 in.)(15 in.) = 0.22 in.2 As,min = 0.0012 ( 3 60,000 psi (1.0)(1.0)(1.0) Ad = | (0.625 in.) 2.5 \ 40 (1.0) 4500 psi | = 16.8 in., say, 17 in. Est = 1.3(17.0 in.) = 22.1 in. Use 24 in. > Est = 22.1 in. Use 24 in. > Es Fig. E5.10. Extend No. 7 at 12 in. on center dowels coming from the toe 4 ft into the stem. Splice No. 5 at 24 in. on center. The overlap length is 30 in. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 101 Weight of tension steel per linear foot in stem only: No. 5 = 1.043 lb/ft No. 7 = 2.044 lb/ft Total length of No. 7 = 4.5 ft Total length of No. 5 = 13.17 ft - 4.5 ft + 2.0 ft = 10.67 ft. Total weight = (2.044 lb/ft/ft)(4.5 ft) + (1.043 lb/ft/ft)(4.5 ft) + (1.043 lb/ft/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or 1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft = 12.1 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. (32.4 lb/ft)(10.67 ft) = 20.32 lb/ft or <math>1 - = 37.3 percent. lb/ft // American Concrete Institute - Copyrighted © Material - www.concrete.org Retaining Walls Step 14: Detailing 102 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Retaining Walls Example 6—Reinforced concrete cantilever wall retaining earth built at property line 'HVLIODORUPDOZHLIKWUHLOIRUFHGFROFUHWHFDOWLOHYHUZDOOWKDWUHWDLQVDOHYHOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿QDOHDUWKEDQNIWKLIKDERYHWKH¿ report: soil unit weight of 120 pcf, angle of soil LQWHUQDOIULFWLRQRIGHJUHHVFRHI¿FLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLORIDQGWKHDOORZDEOHVRLOEHDULQJSUHVVXUH of 3000 psf. \$VVXPHWKDWWKHFDQWLOHYHUZDOOLVEXLOWDWWKHSURSHUW\OLQHLVQRWVXEMHFWHGWRDQ\RWKHUORDGDQGWKHIURVWOLQHLVIWLQ EHORZWKH¿QLVKHGJUDGH Given: Soil data— Ûs = 120 pcf [GHJUHHV 3 qall = 3000 psf Concrete— 3 a = 1.0 (normalweight concrete) fy = 60,000 psi Cantilever wall— h1 = 8 ft h2 = 2.5 ft Fig. E6.1—Cantilever 1—Cantil retaining wall. ACI 318-14 Discussion Step 1: Material requirements 7.2.2.1 The mixture proportion must satisfy the durability classes. Please see Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. Calculation By specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFL¿HGDWGD\VWREHDWOHDVWSVL \$&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages
referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI 318. ACI encourages referencing LQWRMREVSHFL the contractor. Example 1 of this Chapter provides a more detailed breakdown on determining the concrete compressive strength and exposure categories and classes. American Concrete Institute - Copyrighted © Material - www.concrete.org Step 2: Equivalent lateral pressure the wall is required to resist. Step 3: Preliminary cantilever wall data General criteria The preliminary retaining wall dimensions (refer to Fig. E6.2) are determined from the text. 103 Retaining i i gL L-Wall W required dimensions. 20.6.1.3.1 13.3.1.2 RI LV 2YHUDOOKHLJKWRIZDOOLV h = h1 + h2 = 8 ft + 2.5 ft = 10.5 ft ic (0 07 to 0.12)h Estimating stem thickness: (0.07 to 0.12)h Estimating stem thickness: (  $0.08(10.5 \text{ Assuming 3 in. cover, the effective depth is: (1 ft 3 in.) - (3 in.) = 1 ft > 6 in. /HQJWKRIEDVHKHHO WR h Heel length: bheel = (5 ft 8 in.) - (10 in.) = 4 ft 10 in. American Concrete Institute - Copyrighted ©$ Material - www.concrete.org 104 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 4: Applied forces For out-of-plane moment and shear, the cantilevered retaining wall is assumed to be continuous, and a representative 1 ft strip is analyzed for the maximum load effects (refer to Fig. E6.3). 9HUWLFDOORDGV The vertical weight is the self-weight of the retaining wall (stem and base) and the weight of the soil above the heel. Fig. E E6.3—Applied forces on retaining wall. (hítbase) Stem wall: P1  $\hat{U}$ conc(ttstem)(h Base: P2  $\hat{U}$ conc(ttstem)(h Base: P2  $\hat{U}$ conc(ttstem)(h Base: P2  $\hat{U}$ conc(ttstem)(h Base) Stem wall: P1  $\hat{U}$ conc(ttstem)(h Base: P2  $\hat{U}$ conc(ttstem)(h Base) Stem wall: P1  $\hat{U}$ conc(ttstem)(h Base) Stem w 1063 lb P2 = (15 ((120 pcf)(10.5 ft - 11.25 ft)(5.67 ft - 0.83 ft) = 5372 lb P3 = (12 oa Total vertical load:  $^{\text{TM}}$  = 11  $^{\text{TM}}$  P 1152 lb + 1063 lb + 5372 lb = 7587 lb The self-weight of the retaining wall an and the soil nd tto counteract unteract the ov urning above the heel tend overturning moment. Moments taken about the front edge of base (stem) Stem wall: M1 = P1(tstem/2) Base: M2 = P2(bbase/2) Soil: M3 = P3(b - bheel/2) M1 = (1152 lb)(0.83 ft/2) = 478 ft-lb M2 = (1063 lb)(5.67 ft - 4.83 ft/2) = 478 ft-lb M3 = (5372 lb)(5.67 ft - 4.83 ft/2) = 478 ft-lb M3 = (537 wall: H1 = (Ca  $\hat{U}s$ )(h2/2) H1 = (0.271)(120 pcf)(10.5 ft)2/2 = 1793 lb/ft Therefore, this lateral force tends to overturn the retaining wall about the front edge of the stem: MOTM = (1793 ft)(10.5 ft/3) = 6276 ft-lb Summation of moments: "M  $\stackrel{\text{\tiny M}}{=}$  MOTM  $\stackrel{\text{\tiny M}}{=}$  MOTM  $\stackrel{\text{\tiny M}}{=}$  MOTM  $\stackrel{\text{\tiny M}}{=}$  (20,978 ft-lb) - (6276 ft-lb) = 14,702 ft-lb American Concrete Institute -Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 105 a=  $\Delta M \ge P$  Retaining Walls Step 5: Soil pressure 13.3.1.1 The aforementioned determined cantilever wall base is checked using unfactored forces and allowable soil bearing pressure. To calculate soil pressure, the location of the vertical resultant force must be determined. The distance of the resultant to the front face of stem: a = 14,702 ft-lb = 1.94 ft 7587 lb Eccentricity is the difference between the resultant location and the base mid-length: e = bbase/2 - a = 5.67 ft/2 - 1.94 ft = 0.90 ft Check if resultant falls within the middle third
of the base. bbase 5.83 ft = 0.97 ft > e = 0.9 ft 6 6 Maximum and minimum soil pressure: Therefore, there is no uplift.  $q_{1,2} = \sum P \sum Pe \pm A S q_{1,2} = 7587$  lb (7587 lb)(0.90 ft)  $\pm ft$ ) (5.67 ft)(1 f (5.67 ft) 2 /6 q\_{1,2} = 1338 psf  $\pm 12 1274 4 ppsf f < qall = 3000 psf q_1 = qma max = 2612 psf 4 psf > 0 ps psf, no tension q_2 = qmi min = 64 So Soil bea bearing pressure iis acceptable. Step 6: Stability requirements Calculate$ factor of safety overturning: y against overt ning:  $FS = \sum MR \ge 2.0 \text{ M OTM } 20,978 \text{ ft} \text{-lb FS } = 3.5 > 2.0 \text{ OK FS} = 3.$ (conservative). Conclusion: The retaining wall preliminary dimensions are adequate to resist overturning, sliding, preventing uplift, and limiting pressure on the soil to less than the allowable provided soil pressure in the geotechnical report. In the following steps, the retaining wall is designed for strength. If any of the aforementioned determined dimensions are not satisfactory, then all the previous steps must be revised. Note: Unfactored loads were used to determine the stability of the retaining wall and to calculate the soil pressure. American Concrete Institute – Copyrighted © Material – www.concrete.org 106 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Stem design The cantilevered concrete stem is a determinate member and is modeled as a 1 ft wide cantilever beam (refer to Fig. E6.4). 13.2.7.1 Flexure The maximum design moment in the stem is calculated at the face of the base foundation. face of the stem wall that is against the retained soil. Adequate concrete cover protects reinforcement against moisture changes in soil. Cover is measured from the concrete surface of the reinforcing bar. Fig. E6 E6.4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4—Soil 4= height: 20.6.1.3.1 6. use, 2 in. cover From Table 20.6.1.3.1 21.2.2 er is tension con olled; Assume that the m member controlled; []VWHHOVWUDLQIt DQG5.3.8 7.4.1.1 [a) /RDGIDFWRU U = Hu = 1.6H; when lateral pressure acts alone. The moment is taken at the bottom of the stem and above the base; h1 = 9.25 ft 7.5.1.1 6DWLVI\[]Mn•Mu 22.2.2.1 The concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate PRPHQWVDUHGHYHORSHGLVHTXDOWRIC = 0.003. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQAH[XUHLVDYDULable property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate property and its value is approximately 10 to 15 percent of the concrete compressive strain at which ultimate property at a percent of the concrete compressive strain at which ultimate property at a percent of the concrete compressive strain at a percent of the concrete compressive strain at a percent of the concrete compressive strain at a percent of the concrete compressive strain at a percent of the concrete compressive strain at a percent of the concrete compressive strain at a percent of the concrete compressive st tensile strength. Hu = 1.6( $1.6(32.5 \text{ pcf})(9.25 \text{ ft})/2 = 2225 \text{ lb Mu} = (2225 \text{ lb})(9.25/3) = 6860 \text{ ft-lb} \approx 82,300 \text{ in.-lb American Concrete Institute} - Copyrighted © Material - www.concrete.org CHAPTER 12-RETAINING WALLS 22.2.2.4.1 Determine the equivalent concrete compressive stress for design. The concrete compressive stress for design. The concrete compressive stress for design. The concrete compressive stress for design.$ distribution is inelastic at high stress. The actual distribution of concrete compressive stress is complex and usually not known explicitly. The Code permits any stress distribution to be assumed in design if shown to result in predictions of nominal strength in reasonable agreement with the results of comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRI a "1cZKHUH" is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3 For fcg"SVL 22.2.1.1 Find the equivalent concrete compressive depth, a, by equating the compression force and the tension force within a unit length of the wall cross section: C=T C = 0.85fcgba and T = As(60,000 psi) = 1.31As 0.85(4500 psi)(12 in.)(a) = As(60,000 psi) = 1.31As 0.85(4500 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 1.31As 0.85(4500 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825
1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psi) = 0.825 1000 psi)(12 in.)(a) = As(60,000 psEquating n moment strength and required mont strength, As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) As | 7.68 in. - M n = (60, (2 | ) 21.2.1 8VHAH[XUHVWUHQJWKUHGXFWLRQIDFWRU 20.6.1.3.1 [Assume No. 5 vertical reinforcement: d = tstem - cover - db/2 use 2 in. cover 7.5.1.1 (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) As | 7.68 in. - M n = (60, (2 | ) 21.2.1 8VHAH[XUHVWUHQJWKUHGXFWLRQIDFWRU 20.6.1.3.1 [Assume No. 5 vertical reinforcement: d = tstem - cover - db/2 use 2 in. cover 7.5.1.1 (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) As | 7.68 in. - M n = (60, (2 | ) 21.2.1 8VHAH[XUHVWUHQJWKUHGXFWLRQIDFWRU 20.6.1.3.1 [Assume No. 5 vertical reinforcement: d = tstem - cover - db/2 use 2 in. cover 7.5.1.1 (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) As | 7.68 in. - M n = (60, (2 | ) 21.2.1 8VHAH[XUHVWUHQJWKUHGXFWLRQIDFWRU 20.6.1.3.1 [Assume No. 5 vertical reinforcement: d = tstem - cover - db/2 use 2 in. cover 7.5.1.1 (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) As | 7.68 in. - M n = (60, (2 | ) 21.2.1 8VHAH[XUHVWUHQJWKUHGXFWLRQIDFWRU 20.6.1.3.1 [As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) As | 7.68 in. - M n = (60, (2 | ) 21.2.1 8VHAH[XUHVWUHQJWKUHGXFWLRQIDFWRU 20.6.1.3.1 [As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a) (M n = As f y | d - | (2/1.31As) ((000 0 psi) ps) (As is: 7.5.2.1 22.3 a6XEVWLWXWLQJLQWR Mn • Mu Mu = 82.3 in.-kip calculated above. Solving for As (refer to Fig. E6.4): 9.6.1.2 The cantilevered retaining wall calculated required tensile reinforcement is usually very small compared to the member concrete section. The stem reinforcement is checked against the beam minimum UHTXLUHGAH[XUDOUHLQIRUFHPHQWDUHDUDWKHUWKDQWKH one-way slab minimum reinforcement area at least the greater of: (a) As , min  $\geq 3$  f c' fy bw d d = 10 in. - 2 in. - 0.625 in./2 = 7.68 in. 1.31As ( (0.9)(60 ksi)As | 7.68 in. - = 82.3 in.-kip ( 2 | ) As = 0.20 in.2 No. 5 at 12 in. on center. As,prov'd = 0.31 in.2/ft > As,req'd = 0.2 in.2/ft As ,min = 3 4500 psi (12 in.)(7.68 in.) = 0.31 in.2 60,000 psi American Concrete Institute – Copyrighted © Material – www.concrete.org 108 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Equation (9.6.1.2(a)) controls, because concrete compressive strength, fcg = 4500 psi. 21.2.2 22.2.1.2 Check if the tension controlled assumption and the XVHRI[LVFRUUHFW To answer the question, the tensile strain in reinIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQGFRPSDUHG to the values in Table 21.2.2. Assume concrete and nonprestressed reinforcement strain varying proportional to the distance from the neutral axis (refer to Fig. E6.5): t = c (d - c) c where: c = a and a = 1.31As derived previously.  $\beta 1$  Use No. 5 at 12 in. on center. As, prov'd = 0.31 in.2/ft > As, req'd = 0.2 in.2IW2. c = (1.31)(0.31 in.2) = 0.49 in. 0.825 0.003 (7.68 in. - 0.49 in.) = 0.044 0.49 in. It = 0.044 > 0.005 t = 0.005 ct= 6HFWLRQLVWHQVLRQFRQWUROOHGDQG[]Fig. E6.5—Strain distribution across stem. 7.4.3.2 7.5.3.1 22.5.5.1 21.2.1 7.5.1.1 Shear The closest inclined crack to the support of the base. The lateral load applied to the cantilever between the face of the base and point d away from the face are transferred directly to the base by compression in the cantilever above the crack. Accordingly, the Code permits design for a maximum factored shear force Vu at a distance d from the support for nonprestressed members. For simplicity, the critical section for design shear strength in this example is calculated at the bottom of the stem:  $Vn = (Vc + Vs) = 2\lambda f c'bw d$  with Vs = 0 Shear strength reduction factor: 7KHUHIRUH[[Vn []Vc is: ,V[]Vn • VuVDWLVcHG" hstem = h - tbase = 10.5 ft - 1.25 ft = 9.25 ft Vu = 1.6(32.5 pcf)(9.25 ft) = 9273 lb []Vn = 9273 lb > Vu = 2225 lb OK Shear reinforcement is not required American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 12—RETAINING WALLS Retaining Walls Step 8: Heel design Shear The base heel is designed for shear caused by the superimposed weight of soil, including self-weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed (refer to Fig. E6.6). Fig. E6.6). Fig. E6.6). Fig. E6.6). Fig. E6.6). Fig. E6.6). 20.6.1.3.1 d = tbase - cover - db/2 [with Vs = 0 15 in. in - 3 iin.. - 0.375 0. 75 in. = 11.6 in. d = 15 6.1 use 3 in. cover to tension From Table 20.6.1.3.1, reinforcement. rei eme Assume No. 6 reinforcement s: 7KHQ[Vn [Vc is: A load factor of 1.2 is used for the concrete selfZHLJKWDQGIRUEDFN¿OOVHOIZHLJKW 7.5.1.1, V[]Vn •VuVDWLV¿HG" 109  $\varphi$ Vn =(0 =(0.75)2 4500 ps psi(12 in.)(11.6 in.) = 14,007 lb Vu = (1.2) (1.2)(150 pcf)(4.84 ft - 11.3 in./12)(1.25 ft) + (1.6)(120 pcf)(9.25 ft)(4.84 ft - 11.6 in./ft) Vu = 7750 lb OK Shear reinforcement is not required American Concrete Institute - Copyrighted © Material - www.concrete.org 110 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Flexure 7KHKHHOLVVXEMHFWWRAH[XUHFDXVHGE\WKH superimposed weight of soil and self-weight of heel. The soil pressure counteracting the applied gravity loads is neglected as the soil pressure may not be linear as assumed. Therefore, it is not LQFOXGHGLQWKHFDOFXODWLRQRIAH[XUHFDXVHGE] 6.6.1.2 The cantilever wall maximum moment and shear in the heel and toe of the base occur at the stem face. Redistribution of moments cannot occur. 5.3.1 5.3.8 A load factor of 1.2 is used for the concrete selfZHLJKWDQGIRUVRLOEDFN¿OO Mu1 =  $1.2(150 \text{ pcf})(4.84 \text{ ft})2/2 + 1.6(120 \text{ pcf})(9.25 \text{ ft})(4.84 \text{ ft})2/2 = (2634 \text{ ft-lb}) + (20,802 \text{ ft-lb}) = (2634 \text{ ft-lb}) + (20,802 \text{ ft-lb}) = (2634 \text{ ft-lb}) + (20,802 \text{ ft-lb}) = (2634 \text{ ft-lb}) + (20,802 \text{ ft-lb}) = (2634 \text{ ft-lb}) + (20,802 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) + (20,802 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634 \text{ ft-lb}) = (2634
\text{ ft-lb}) = (2634 \text{$ 23,347 ft-lb  $\approx 281,000$  in.-lb 22.2.1.1 Setting C = T 0.85(4500 psi)(12 in.)(a) = As(60,000 psi) = 1.31 As(Mu = 281 in.-kip, p, calculated lated above. As = 0.46 in. n.2 Solving for As: 9.6.1.2 The cantilevered retaining wall calculated d tensile reinforcement is very small compared to the member concrete section. To prevent a VXGGHQIDLOXUHWKH&RGHUHTXLUHVWKDWWKHAH[XUDO reinforcement area is least the greater of: (a) As , min  $\geq 3$  f c' fy bw d As,min = 3 4500 psi (12 in.)(11.5 in.) = 0.46 in.2 60,000 psi Use No. 7 @ 12 in. on center (refer to Fig. E6.7). Equation (9.6.1.2(a)) controls, because concrete compressive strength fcg SVL 21.2.2 As,prov'd = 0.6 in.2/ft > As,req'd = 0.46 in.2/ft OK Check if the tension controlled assumption and the tension controlled assumption assumption assumption assumption as the tension controlled assumption as the tension control as t XVHRI[LVFRUUHFW American Concrete Institute - Copyrighted © Material - www.concrete.org To answer the question, the tensile strain in UHLQIRUFHPHQWPXVWEH¿UVWFDOFXODWHGDQG compared to the values in Table 21.2.2. Concrete and nonprestressed reinforcement strain is assumed to vary proportionally from the neutral axis. From similar triangles (refer to Fig. E6.5):  $\epsilon t = \epsilon c$  (d - c) c where: c = a and a = 1.31As derived previously.  $\beta 1 c = 111 (1.31)(0.60 \text{ in.} 2) = 0.95 \text{ in.} 0.825 0.003 (11.6 \text{ in.} - 0.95 \text{ in.}) = 0.0336 0.95 \text{ in.} 1t = 0.0336 0.95 \text{ in.} 1t = 0.0336 > 0.005 \epsilon t = 6$ there is not toe at the base. Step 10: Minimum transverse reinforcement cement Stem 11.6.1 Assume that No. 5 bbars or smaller will be used for d sh ge reinforcement met and shrinkage reinforcement. Per Table 11.6.1: As,min (12 in.)(1 in.) = 0.24 in.2 in = (0.002)(12 ((As,min As,min)back = 0.24 in.2/2 = 0.12 in.2)) min)fr front = (A Distribute shrinkage and temperature reinforcement Try No. 4 at 18 in. oon center equally between the front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support the transverse wall reinforcement at front face to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to (12 in.) support to No. 4 spaced at 18 in. on center 24.4.3 Base Assume that No. 5 bars or smaller will be used for temperature and shrinkage reinforcement. The reinforcement. The reinforcement can be located between the two faces. (0.0018)(1.25 ft)(12 in./ft) = 1.84 in.2 8VH¿YH1RSODFHGDWWKHWRSDQGWZR1RDWWKH bottom as continuous nose bars for the dowel reinforcement extended into the stem (refer to Fig. E6.8). American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 112 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Dowels 7.7.1.2 The development length concept is stem. based on the attainable average bond stress over the embedment 25.4.2 length of reinforcement. Development lengths are required because of the tendency of highly stressed bars to split relatively thin sections of reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development
RIUHLQIRUFHPHQWUHLQIRUFHPHQ Heel reinforcement is developed beyond the stem critical section. Development length, Ed, is the greater of Eq. 25.4.2.1 (25.4.2.2) or (25.4.2.3) of ACI 318-14 and 12 in.: 25.4.2.2 (f y t y e) 1. A d = | db \ 200 \lapha f c' | 25.4.2.3 () 3 f y t y t y ey s | 2. A d = | db \ 40 \lapha f c' cb + K ttrr || db | 40 \lapha f c' cb + K ttrr || db | 25.4.2.4 In this example Eq. (25.4.2.3a) will be used. zt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ze = coating factor; uncoated zs = bar size factor; No. 7 and larger cb = spacing or cover dimension to center of bar, whichever is smaller Ktr = transverse reinforcement index. It is permitted to use Ktr = 0. 25.4.2.3 However, the expression: taken greater than 2.5. cb + K tr must not be db Note: the development length in the stem must be checked against the splice length of stem reinforcement and the larger length controls. zt = 1.0, because bars are uncoated zs = 1.0, because bars  $2.5\ 0.625\ \text{in.}\ 2.44\ \text{in.}\ +\ 0 = 2.79 > 2.5\ 0.875\ \text{in.}\ 2.25\ \text{in.}\ +\ 0 = 4.5 > 2.5\ 0.5\ \text{in.}\ \text{Therefore, use } 2.5\ \text{for all three bar sizes in Eq.}\ (25.4.2.3a)\ (3\ 60,000\ \text{psi}\ |\ No.\ 7\ 5\ 4\ \text{Ed,req'd, in.}\ 23.5\ 16.8\ 13.4\ \text{American Concrete Institute} - Copyrighted}\ (0\ \text{Material}\ -\ \text{www.concrete.org}\ \text{Ed,prov., in.}\ 24\ 18\ 15\ 7.7.1.3\ (3\ 60,000\ \text{psi}\ |\ No.\ 7\ 5\ 4\ \text{Ed,req'd, in.}\ 23.5\ 16.8\ 13.4\ \text{American Concrete Institute} - Copyrighted}\ (0\ \text{Material}\ -\ \text{www.concrete.org}\ \text{Ed,prov., in.}\ 24\ 18\ 15\ 7.7.1.3\ (3\ 60,000\ \text{psi}\ |\ No.\ 7\ 5\ 4\ \text{Ed,req'd, in.}\ 23.5\ 16.8\ 13.4\ \text{American Concrete Institute} - Copyrighted}\ (0\ \text{Material}\ -\ \text{www.concrete.org}\ \text{Ed,prov., in.}\ 24\ 18\ 15\ 7.7.1.3\ (3\ 60,000\ \text{psi}\ |\ No.\ 7\ 5\ 4\ \text{Ed,req'd, in.}\ 23.5\ 16.8\ 13.4\ \text{American Concrete Institute} - Copyrighted}\ (0\ \text{Material}\ -\ \text{www.concrete.org}\ \text{Ed,prov., in.}\ 24\ 18\ 15\ 7.7.1.3\ (1.0)\ (1$ 25.5 25.5.1.1 25.5.2.1 7.7.2 25.2.3 Splice The maximum bar size is No. 7, therefore splicing is permitted. No. 5 4 113 Est, req'd, in. 22 17.4 The No. 7 development in the stem must be checked against the splice length of the No. 5 bars in the stem. area is less than 2. Therefore, per Table 21.5.2.1, splice Type B is used and the splice length is the greater of (1.3Ed) or 12 in. (b) 1.5 (d) (1.5 in. (c) (4/3)(1 in.) = 1.33 in. Therefore, 1.5 in. controls. Fig. E6.8—Retaining wall reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org Est, prov., in. 24 20 Retaining Walls CHAPTER 12—RETAINING WALLS 114 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 13.2—Limitations on member thickness ACI 318-14 states that one-way slabs and beams whose section conforms to Table 13.2 of this Handbook (ACI 7DEOHV DQG GR QRW UHTXLUH GHAHFtions to be calculated if they are not supporting or attached to partitions or any construction or equipment that might be 7KH GDPDJHG RU PDGH G\VIXQFWLRQDO E\ ODUJH GHÀHFWLRQV KHUWKDQQRUPDOYDOXHVLQWKHWDEOHVUHTXLUHDGMXVWPHQWLIRWKHUWKDQQRUPDOement are used. For weight concrete and 60 ksi reinforcement 11 lb/ft3) members, lightweight concrete (90 lb/ft3"wc < 115 the obtained values from the tablee are multiplied by  $(1.65 - \text{rcem with yielding sstresss } 0.005wc) \ge 1.09$ , and for reinforcement ed from the tabl other than 60,000 psi, the valuess oobtained table are ). multiplied by (0.4 + fy/100,000). nes off nonprestressed e deflections fl ti beams and one-way slabs unless are calculated Minimum thickness h Simply supported Member Solid one-way slabs Beams or robbed one-way slabs 2QHHQG continuous Both ends continuous Cantilever Members not supporting or attached to partitions or other FRQVWUXFWLRQOLNHO\WREHGDPDJHGE\ODUJHGHAHFWLRQV E/20 E/18.5 E/28 E/21 E/10 E/8 Notes ELVWKHVSDQOHQJWKRIEHDPRURQHZD\VODEFOHDUSURMHFWLRQRIFDQWLOHYHULQ 9DOXHVJLYHQVKDOOEHXVHGGLUHFWO\IRUPHPEHUVZLWKQRUPDOZHLJKWFRQFUHWHDQG \*UDGHUHLQIRUFHPHQW)RURWKHUFRQGLWLRQVWKHYDOXHVVKDOOEHPRGL&HGDV follows: (a) For lightweight concrete having equilibriun density wc in the range of 90 to 115 lb/ft3, the values shall be multiplied by (1.65 - 0.005wc) but not less than 1.09 (b) For fy other than 60,000 psi, the values shall be multiplied by (0.4 + fy/100,000). 13.3—Immediate deflection behavior of beams or one-way slabs 7KHHODVWLFGHÀHFWLRQRIDEHDPRURQHZD/VODELVHYDOXated (Eq. (13.3a)) in member due to service loads at VWDJHGHÀHFWLRQLVFDOFXODWHGLQOE E = span length of beam, in. Ec = modulus of elasticity of concrete, psi. Ie = effective moment of inertia, in4. 7KH PRGXOXV RI HODVWLFLW\ RI FRQFUHWH DV GH¿QHG E\ ASTM C469, is the slope of the straight line connecting (fcc11) corresponding c to 0.00005 strain to stress (fc2) the stress (f esponding to 0.40f 0.4 cgDVIROORZVLQ(TE UHIHUWR corresponding 3a). Fig. 13.3a). Ec = f c 2 - f c 1 psi  $\epsilon$  2 - 0.00005 (13.3b) Mat ema Ec is expressed in ACI 318-14 Section Mathematically, 19.2. f concrete with unit weights 19.2.2.1 by Eq. (13.3c) for n 90 and d 1160 lb/ft3: varying bbetween Ec = 33wc1.5 f c (13.3c) For normalweight concrete (unit weight approximately 145 lb/ft3), the modulus of elasticity can be expressed by the VLPSOL¿HG(TG Ec = 57, 000 f c' (13.3d)). The ratio of the steel modulus of elasticity is taken as Es = 29 × 106 psi (refer to Fig. 13.3b). The ratio of the steel modulus of elasticity to concrete is termed as the modular ratio, n = Es/Ec. It varies from 9.3 for 3000 psi to 4.6 for 12,000 psi to 4.6 for 12,000 psi for normalweight concrete. (ODVWLFGHAHFWLRQVRIEHDPVDQGRQHZD\VODEVDUHFDOFXlated in
accordance with two assumptions: (a) Plane sections remain plane after bending (for example, strains are linearly distributed over the depth of the member) (refer to Fig. 13.3c) (b) Stress is calculated from Hooke's law; stress is equal to the product of modulus of elasticity and strain (1 = Eİ The internal moment of one-way slabs and beams in the elastic range (refer to Fig. 13.3c) is calculated by Eq. (13.3e) M 1/y American Concrete Institute - Copyrighted © Material - www.concrete.org (13.3e) Serviceability 13.1—Introduction ([FHVVLYHVWDWLFGHAHFWLRQVPLJKWQRWEHDOLIHVDIHW\LVVXH but are usually associated with excessive concrete cracking and sagging of structural members, which could interfere with the functioning of building components such as doors DQG ZLQGRZV, Q FDVHV RI H[FHVVLYH G\QDPLF GHAHFWLRQV occupants experience a bouncy feeling when walking on the surface, giving the perception of an unsafe structure. Therefore, engineers must design slabs and beams for adequate stiffness in addition to meeting strength requirements 116 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 13.3a—Stress-strain relationship for concrete compressive strength. Fig. 13.3b—Stress-strain relationship for reinforcement Grade 60. Because the modulus of steel elasticity is greater than concrete, steel is transformed to an equivalent concrete area based on equal axial stiffness AE in Eq. (13.3f). Ac , equiv Ec = As Es = Ac equiv = Es E As and s = n Ec Ec (13.3f)  $\Rightarrow$  Ac , equiv = nAs The cracking moment, Mcr, of a reinforced concrete beam or one-way slab (ACI 318-14, Eq. (24.2.3.5b)) is given by replacing the stress term,  $\sigma$ , in Eq. (13.3e) with the rupture stress, fr = 7.5 3 fc' (ACI 318-14, Eq. (19.2.3.1)), where modi¿FDWLRQIDFWRUλUHÀHFWVWKHUHGXFHGPHFKDQLFDOSURSHUWLHV of lightweight concrete relative to normalweight concrete and λ = 0.75 for lightweight concrete. Fig. 13.3c—Moment at a distance y from neutral axis. M cr = fr I g yt (13.3g) where yt in Eq. (13.3g) is the distance from the centroidal axis of the gross section, neglecting reinforcement, to the tension face. \$ UHLQIRUFHG FRQFUHWH EHDP ZKHQ VXEMHFWHG WR VPDOO ORDGLQJ ZLOO EHKDYH HODVWLFDOO\ DV VKRZQ IRU D ¿[HGHQG beam in Fig. 13.3c). As load increases beyond its cracking American Concrete Institute – Copyrighted © Material – www.concrete.org 117 )LJI±±0RPHQWYHUVXVÀH[XUDOVWLIIQHVVUHODWLRQVKLSRI a beam. )LJG±±/RDGYHUVXVGHÀHFWLRQRID¿[HGHQG Few references, however, use the term kd instead of c, because c is typically associated with the neutral axis at ultimate. The factor kLVREWDLQHGIURP(TM k = (ρn) 2 + 2ρn – ρn M Both will yield the same result. To calculate the cracked moment of inertia for T-beam cross sections, refer to the Table 13.6.1 and 13.6.2 at the end of this Design Aid, Tables chapter. use the cracked moment in ia oof a beam am cann of inertia cannot be calculated. ACI 318-14 provides equation as follows (Eq. (13.3k)) rovid s an approximate i ate t equ to calculate moment of inertia (ACI 318-14, ulate an effective ective mo 2.3 Eq. (2 (24.2.3.5a)) Fig. 13.3e—Rectangular beam transformed ans med cross secti section. ORDG AH[XUDO FUDFNV ¿UVW GHYHORS DW VXSSRUWV PD[LPXP moment location), reducing the overall effective stiffness. \$V ORDGLQJ LQFUHDVHV AH[XUDO FUDFNV ZLOO WKHQ GHYHORS at midspan, thus further reducing effective beam stiffness (refer to Fig. 13.3d). In an uncracked section, the full (or gross) moment of LQHUWLD LV XVHG WR FDOFXODWH GHAHFWLRQV +RZHYHU ZKHQ D beam section cracks, a reduced (cracked) moment of inertia must be calculated. To calculate the cracked moment of inertia of a rectangular beam cross section, the neutral axis, take the moment of both beam areas above and + |1 - | cr | | I cr \ Ma / | [ \ Ma / |] (13.3k) where Ma is the unfactored maximum moment at which the GHÀHFWLRQ LV FDOFXODWHG Mcr is the gross moment of inertia neglecting reinforcement; and Icr is the cracked moment of the transformed cross section. If Ma in Eq. (13.3i) is equal to or smaller than Mcr, the effective stiffness is the stiffness of the uncracked beam, EcIg. As Ma increases, the effective stiffness is close to the fully cracked section EcIcr, (refer to Fig. 13.3f). )RU D EHDP ZLWK WZR FRQWLQXRXV RU ¿[HG HQGV ZKHUH] cracking is predominantly near the supports and at the midspan, ACI 318-14, Section 24.2.3.6 allows the designer to take the average effective moment of inertia as calculated in Eq. (13.31) I e, avg = I e, left - sup + I e, midspan 3 (13.31) NUDEHDPZLWKRQH2[HGRUFRQWLQXRXVHQGWKHDYHUDJH effective moment of inertia as calculated in Eq. (13.31) I e, avg = I e, left - sup + I e, midspan 3 (13.31) NUDEHDPZLWKRQH2[HGRUFRQWLQXRXVHQGWKHDYHUDJH effective moment of inertia as calculated in Eq. (13.31) I e, avg = I e, left - sup + I e, midspan 3 (13.31) I e , avg = I e, left - sup + I e, midspan 3 (13.31) I e , avg = I e, left - sup + I e, midspan 3 (13.31) I e , avg = I e, left - sup + I e, midspan 3 (13.31) I e , avg = I e, left - sup + I e , midspan is given by Eq. (13.3m) American Concrete Institute - Copyrighted © Material - www.concrete.org Serviceability CHAPTER 13—SERVICEABILITY 118 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) I e, avg = I e, sup + I e, midspan (13.3m) 2 ACI 435R-95 recommends alternate equations to calculate the average effective moment of inertia in a beam with WZR ¿[HG RU FRQWLQXRXV HQGV (T Q DQG ¿[HG RU FRQWLQXRXVDWRQHHQG(TR Ie,avg = 0.70Ie,midspan + 0.15[e,sup (13.30) For a beam subjected to various load cases (distributed, FRQFHQWUDWHG DQG YDULDEOH WKH GHÀHFWLRQ IRU Analysis Tables, which org/sto g loaded from: . DVS[",WHP,' 63'\$. flec calculation 13.4—Time-dependent deflection ur calculates ulates an imme iate The aforementioned procedure immediate Q RI ORDG DG %HDPV KRZ YHU GHÀHFWLRQ XSRQ WKH DSSOLFDWLRQ KRZHYHU ustained loads ((dead ead support loads over their service li life. Sustained uch as inn warehouses and loads and sustained live loads such libraries) result in concrete undergoing creep strains and therefore the cross section curvature increases. The immeGLDWH GHAHFWLRQ RI D UHLQIRUFHG FRQFUHWH EHDP LQFUHDVHV due to time-dependent factors such as creep, shrinkage, and temperature strains and therefore the cross section curvature increases. (as shown in Fig. 13.4a).  $PXOWLSOLHUIRUWKHDGGLWLRQDOGHAHFWLRQGXHWRVXVWDLQHG ORDGLQJLVJLYHQLQ$&,(T <math>\lambda \Delta = LJ D \pm kKDQJH LQ FXUYDWXUH RI D FUDFNHG UHLQIRUFHPHQWUDWLRDQGh is the time-background-content of the time-back$ dependent factor for sustained loads (as shown in Fig. 13.4b and Table 13.4a). Table 13.4a—Time-dependent factor for sustained loads Time h 3 months 1.2 12 months 1.4 2 years 1.7 5 years 2.0 E±± OWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU
)LJE±±0XOWLSOLHUIRU )LJE±0XOWLSOLHUIRU )LJE±±0XOWLSOLHUIRU )LJE±0XOWLSOLHUIRU )LJE± smaller the reep strain rain in concrete t bbecause forces transfer from concrete creep RUFH HFDXVH UUHLQIRUFHPHQW %HFDXVH nor al te tures, the transfer of force away from the at normal temperatures, FRQ H UHV Q D GHFUH FRQFUHWHUHVXOWVLQDGHFUHDVHLQFUHHSGHÀHFWLRQ7KHWRWDO ÀHFWLRQLVW ORQJWHUPGHÀHFWLRQFDOFXODWHGIURP(T (13.3a). 7KH LPPHGLDWH DQG ORQJWHUP GHÀHFWLRQV DUH WKHQ FRPSDUHG WR WKH PD[LPXP DOORZDEOH GHAHFWLRQV SURYLGHG in Table 13.4b (ACI 318-14, Table 24.2.2). If the calculated GHAHFWLRQ H[FHHGV WKH PD[LPXP DOORZDEOH WKHQ DFWLRQ VKRXOGEHWDNHQIRUH[DPSOHLQFUHDVHFWLRQ 'HVLJQ \$LG 'HAHFWLRQ DFWLRQ VKRXOGEHWDNHQIRUH[DPSOHLQFUHDVHFWLRQ H] to calculate cracking moments of rectangular, T-, and L-sections with and without negative reinforcement, cracking moment of inertia, modulus of elasticity for various concrete strengths, and other helpful graphs and tables. 13.5—Distribution of flexural reinforcement, effective moment of inertia, modulus of elasticity for various concrete strengths, and other helpful graphs and tables. FUDFN FRQWURO LV DGGUHVVHG LQ \$&, HLWKHU through prestressing or by reinforcing bar spacing limits. Research illustrates that the crack width in nonprestressed slabs and beams is primarily related to reinforcing steel spacing. Wide spacing of reinforcement can result in large American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 13—SERVICEABILITY 119 Type of member 'HÀHFWLRQVREHFRQVLGHUHG 'HÀHFWLRQ limitation Flat roofs not supporting or attached to nonstructural elements OLNHO\WREHGDPDJHGE\ODUJHGHÀHFWLRQV. PPHGLDWHGHÀHFWLRQXHWROLYHORDGL E/180\* Floors not supporting or attached to nonstructural elements likely WREHGDPDJHGE\ODUJHGHÀHFWLRQGXHWROLYHORDGL & /360 7KDWSDUWRIWKHWRWDOGHÀHFWLRQGXHWRDOO VXVWDLQHGORDGVDQGWKHLPPHGLDWHGHÀHFWLRQGXHWRDQ\DGGLWLRQDO live load)† E/480‡ 5RRIRUÀRRUFRQVWUXFWLRQVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWLRQVWUXFWLRQVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWLRQVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWLRQVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWLRQVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWLRQVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWLRQVXSSRUWLQJRU + LPLWQRWLQWHQGHGWRVDIHJXDUGDJDLQVWSRQGLQJ3RQGLQJVKRXOGEHFKHFNHGE\VXLWDEOHFDOFXODWLRQVRIGHAHFWLRQVGXHWR ponded water, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained loads, camber, and considering long-term effects of all sustained long-term effects of all sustained long-term effects of all sustained long-term effects of all sustained long-term effects of all sustained long-term effects of construction tolerances and reliability of provisions for drainage. † /RQJWHUPGHAHFWLRQVKDOOEHGHWHUPLQHGLQDFFRUGDQFHZLWK\$&,6HFWLRQVKDOOEHGHWHUPLQHGLQDFFRUGDQFHZLWK\$&,6HFWLRQVKDOOEHGHWHUPLQHGLQDFFRUGDQFHZLWK\$ relating to timeGHAHFWLRQFKDUDFWHULVWLFVRIPHPEHUVVLPLODUWRWKRVHEHLQJFRQVLGHUHG ‡ /LPLWFDQEHH[FHHGHGLIDGHTXDWHPHDVXUHVDUHWDNHQWRSUHYHQWGDPDJHWRVXSSRUWHGRUDWWDFKHGHOHPHQWV § /LPLWVKDOOQRWEHIUHDWHUWKDQWROHUDQFHSURYLGHGIRUQRQVWUXFWXUDOHOHPHQWV/LPLWPD\EHH[FHHGHGLIFDPEHULVSURYLGHGVRWKDWWRWDOGHAHFWLRQPLQXV camber does not exceed limit. crack widths, while tighter spacing results in smaller crack widths. While the crack control provisions in ACI 318-14 generall result in acceptable outcomes, actual crack widths in structures are highly variable. cement spac The Code provisions that limit reinforcement spacing are o gene intended to control crack widths to generally acceptable nt is stressed to 60 percent values, assuming the reinforcement FNZ, QDSDSHUL RI\LHOG:K\OLPLWAH[XUDOFUDFNZLGWKV", QDSDSHULQWKH HU /RRN DW &UDF QJ 1999 ACI Structural Journal 3\$ 3\$QRWKHU &UDFNLQJ d Concrete," rete, appearance.' uded that the totall Furthermore, 'investigations havee cconcluded DPRXQW RI FRUURVLRQ LV LQAXHQFHG YHU\ OLWWOH E\ WKH FUDFN width and even whether transverse cracks are present or not.' Thus, the limitation of crack width for corrosion protection is unnecessary and can even be counterproductive if it LVDFKLHYHGE\DGHFUHDVHRIFRYHU2WKHUGLVFXVVLRQVKDYH supported the viewpoint that a
correlation between corrosion and surface crack width was selected as 0.016 in., which is based on ACI 318-95 design recommendations for interior exposure conditions. A 1/3 increase in crack widths (0.021 in.) was considered acceptable, considering the large scatter that is inherent in crack widths and that crack control is primarily an aesthetic consideration." The proposed methodology described in this paper is the basis of the current Code provisions. For retaining or basement walls that may retain water in service, the designer should consider controlling cracks to a smaller width than 0.016 in. by detailing with small bar sizes and reduced spacing. From a related standard for the testing of environmental engineering concrete containment structures. Equations in Table 24.3.2 in ACI 318-14 show that the maximum bar spacing is a function of cover cc and the bar's service stress fs. The basic equations, fs can be assumed to be 67 percent of yield. For Grade 60 reinforcing bar, therefore, the maximum spacing is 12 in. ,Q 7EHDP ADQJHV WHQVLRQ UHLQIRUFHPHQW LV GLVWULEXWHG over a width widt of at least En ,I WKH HIIHFWLYH ADQJH \$&, 318-14, Section 6.3 6.3.2) is wider than this, the Code requires DGGLWLRQDO UHLQIRUFHPHQ UHLQIRUFHPHQW LQ LQ ADQJH DUHD EH\RQG En/10, but RW VS \ DQ DPRX GRHV QRW VSHFLI\ DPRXQW RU VSDFLQJ (QJLQHHULQJ MXGJme is exp ment expected. 3 6—Shr d temperature and shrinkage (S+T) reinforcement is req d in one-way way slabs perpendicular to the direction of required IRUFHPHQWAH[XUDOUHLQIRUFHPHQW7KH&RGHSURYLGHVDPLQLPXPUDWLRPM]. reinforcem of S+T reinforcement (Ast/Ag = 0.0018 for Grade 60 steel), which is ad adequate to control cracking for slabs with low levels of restraint to axial movement. If, however, the slab is partially or fully restrained (such as EHWZHHQWZRZDOOV WKLVDPRXQWZLOOOLNHO\EHLQVXI¿FLHQW7R adequately restrain the width of these cracks the stress in the reinforcement crossing the crack should be below yield. For such restrained conditions, the reinforcement could yield at ¿UVWFUDFN)XOOGHSWKGLUHFWWHQVLRQFUDFN) of S+T reinforcement to control cracking can be more than double the Code minimum. The shrinkage and temperature reinforcement can be placed anywhere within the slab thickness; top, bottom, middle, or divided between top and one of slab. 13.7—Shrinkage and temperature reinforcement can be placed anywhere within the slab thickness; top, bottom, middle, or divided between top and bottom of slab. way slab construction, straight, full-length PT tendons perpendicular to the GLUHFWLRQ RI AH[XUDO UHLQIRUFHPHQW DUH FRPPRQO\ XVHG WR resist S+T stresses. The minimum level of average direct compression stress (P/A) due to S+T tendons is 100 psi. This American Concrete Institute – Copyrighted © Material – www.concrete.org Serviceability Table 13.4b—Maximum permissible computed deflections 120 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) level of compression provides approximately the same force as nonprestressed S+T reinforcement assuming yield. )RU EXLOGLQJ VODEV VXEMHFW WR FKHPLFDO GHLFHUV RU RWKHU corrosive exposure, many designers increase the level of PT to resist S+T stressed may not be appropriate. Straight PT tendons will only apply a compressive force to the slab is partially of function of PT to resist S+T stressed may not be appropriate. condition, therefore, use of rebar rather than, or in addition to, PT should be considered. 13.8—Permissible stresses in prestressed concrete flexural members, and precast prestressed members, and precast prestressed members. construction considerations are not discussed herein. To provide a framework for concrete and steel stress OLPLWV\$&, FODVVL¿HV37EHDPVDQGRQHZD\VODEV EDVHGRQWKHPD[LPXPQHWFDOFXODWHGVHUYLFHÀH]XUDOWHQVLOH stress, ft. Classes are indicated in Table 13.8 (ACI 318-14,, Section 24.5.2.1): restre Table 13.8—Classification of prestressed flexural members based on ft Assumed behavior Class lass Limits of ft Uncracked U ft" fc' Transition between uncracked and cracked T " fc' 7.5 fc' < ft" Cracked C ft > 12 fc' Class C is not often used under normal spans and loads. 7KHDGYDQWDJHVRI37IHZRUQRÀH[XUDOFUDFNVDQGVPDOOHU GHÀHFWLRQV DUHOHVVHQHGZKHQXVLQI&ODVV& For U and T sections, stresses are calculated based on gross VHFWLRQSURSHUWLHV)RUD8VHFWLRQGHÀHFWLRQVDUHFDOFXODWHG EDVHGRQJURVVSURSHUWLHVEXWIRUD7DQG&VHFWLRQVGHÀHFtions are based on cracked bilinear properties. 13.9—Permissible stresses at transfer of prestress ACI 318-14 limits both compressive service stress as seen in Table 13.9a (ACI 318-14, Table 24.5.3.1), and tensile service stress as seen in Table 13.9a—Concrete compressive stress limits immediately after transfer of prestress Location Concrete compressive stress limits End of simply-supported members 0.70fcig All other locations 0.60fcig For normal spans and loads, compressive stresses for PT slabs are usually well within these limits. If a PT beam is heavily PT, it may exceed these limits. If a PT beam is heavily PT, it may exceed these limits. If a PT beam is heavily PT, it may exceed these limits. limits immediately after transfer of prestress, without additional bonded reinforcement in tension zone Location Concrete tensile stress limits Ends of simply-supported members 6 fci' All other locations 3 fci') RUQRUPDOVSDQVDQGORDGVEHDPVDQGVODEVKDYHAH[ural tension stresses. Because there is only a light construction load exists at the time of tensioning, this tensile stress limit usually is not exceeded. If it is exceeded, the designer has the option of adding more post-tensioning to increase uplift and compression force (thus reducing tensile stresses) or adding more post-tensioning to increase uplift and compression force (thus reducing tensile stresses) or adding more deformed reinforcing bar must be VXI¿FLHQWWRUHVLVWWKHWRWDOWHQVLRQIRUFHLQWKHEHDPRUVODE in the tension area. 13.10—Perm 13.10 24.5.4.1). able 13.10—Concrete 3.1 crete co Table compressive stress limits ice loads att se service Load condition Loa ition Concrete splus 0.60fcq Again, for normal spans and loads, compressive stress plus 0.45fcq plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestress plu total load Prestre limit is exceeded, the beam size should be reviewed. REFERENCES \$&,<sup>2</sup>6SHFL¿FDWLRQIRU7LJKWQHVV7HVWLQJRI(QYLronmental Engineering Concrete Containment Structures ACI 435R-95(00)(Appendix B added 2003)—Control of 'HÅHFWLRQLQ&RQFUHWH6WUXFWXUHV Authored references )URVFK5-<sup>3</sup>\$QRWKHU/RRNDW&UDFNLQJDQG&UDFN Control in Reinforced Concrete," ACI Structural Journal9 96, No. 3, May-June, pp. 437-442. Gergely, P., 1981, "Role of
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Gergely, P., 1981, "Role of Cover and Bar Spacing in Reinforced Concrete," ACI Structural Journal9 96, No. 3, May-June, pp. 437-442. Gergely, P., 1981, "Role of Cover and Bar Spacing in Reinforced Concrete," ACI Structural Journal9 96, No. 3, May-June, pp. 437-442. Gergely, P., 1981, "Role of Cover and Bar Spacing in Reinforced Concrete," ACI Structural Journal9 96, No. 3, May-June, pp. 437-442. Gergely, P., 1981, "Role of Cover and Bar Spacing in Reinforced Concrete," ACI Structural Journal9 96, No. 3, May-June, pp. 437-442. Gergely, P., 1981, "Role of Cover and Bar Spacing in Reinforced Concrete," ACI Structural Journal9 96, No. 3, May-June, pp. 437-442. Gergely, P., 1981, "Role of Cover and Reinforced Concrete," ACI Structural Journal9 96, No Farmington Hills, MI, pp. 133-147. Gilbert, I., 1992, "Shrinkage Cracking in Fully Restrained Concrete Members," ACI Structural Journal91R Mar.-Apr., pp. 141-149. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 121 13.11—Examples 'HÀHFWLRQ([DPSOH Effective moment of inertia for a rectangular section with tension reinforcement— Determine the effective moment of inertia, Ie, to be used for the rectangular section. ACI 318-14 Discussion Step 1: Determine cracking moment, Mcr 24.2.3.5b Determine cracking moment. AHFWLRO Find KcrIURP'HVLJO\$LG'HAHFWLRO 'H \$LG'HA Calculate Icr = Kii1bd3 Step 3: Gross moment of inertia, a, Iq Determine moment nt of inertia, a, Iq Determine Ie. Calculate Icr /Iq Calcu Kcr = 3.79 Mcrr = 3.79(14) = 53.1 ft-kip/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = (53.1 ft-kip)/(177 ft-kip) = 0.30 For Icr /Ig = 0.68 and Mcr /Ma = 0.30, Ki3 = 0.69 Ie = 11,000/16,100 = 0.68 Mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 For Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ma = 0.30 Icr /Ig = 0.68 mcr /Ig = 0.68 mcr /0.69(16,100 in.4) = 11,100 in.4 \*) URP'HVLJO\$LG'HÀHFWLROLWFDOEHGHGXFHGWKDWWKHUHZLOOEHROO/VPDOOGLIIHUHOFHVEHWZHHOIe and Icr unless Mcr/Ma is greater than Icr /Ig. American Concrete Institute - Copyrighted © Material - www.concrete.org 122 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 'HÀHFWLRQ([DPSOH 'HÀHFWLRQRIDVLPSOHVSDQUHFWDQJXODUEHDPZLWKWHQVLRQUHLQIRUFHPHQW- 'HWHUPLQHWKHOLYHORDGGHÀHFWLRQDWPLGVSDQRIOLQWHOVKRZQLQ)LJ(Given: fcg SVL n=8 As = four No. 11 MD = 120 ft-kip MD+L = 177 ft-kip b = 14 in. d = 21.4 in. (MD and MD+L under service loads) Fig. E2.1-Lintel and its cross section. \$&, Discussion Step 1: Effective moment of inertia Because concrete, section, and moment are the VDPHGDWDIURP'HAHFWLRQ ([DPSOHFDQEHXVHG here. Determine Ie for the dead-load -load mome moment. se McIURP'HAHFWLRQ I Calculate Mcr /MD; use Example 1. LG HFWLRQ Kii33; use )URP'HVLJQ\$LG'HÀHFWLRQ¿QGK ÀHF ([DP Icr /IgIURP'HÀHFWLRQ([DPSOH Calculation Mcr /M MD = (53.1 ft-k ft-kip)/(120 p)/(ft-kip) = 0.44 F For Mcr /MD = 0.44 and Icr/Ig = 0.68, Ki3 = 0.707 IURP'HÀHFWLRQ([DPSOH ([DPSOH [DPSoh [DPSoh [DPsoh
[DPsoh 6WHS'HDGORDGGHÀHFWLRQ 'HWHUPLQHLQLWLDOGHDGORDGGHÀHFWLRQ )URP'HVLJQ\$LG'HÀHFWLRQ¿QGKa1 normalweight concrete, fcg SVL and  $\mathcal{E} = 40$  ft, read Ka1 = 16 (Note: When fcg SVLDQGn = Es/Ec = 8, concrete is normalweight that is 145 lb/ft3, refer to 'HVLJQ\$LG'HÀHFWLRQ. URP'HVLIO\$LG'HÀHFWLRO(OGKa3 for For Case 2, read Ka3 = 5.0 uniform load, simply supported, &DOFXODWH"c)D = (Ka1/Ie)(Ka3MD) 6WHS7RWDOGHÀHFWLRO(IDPSOH &DOFXODWH"c)D + L = (Ka1/Ie)(Ka3MD+L) 6WHS/LYHORDGGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHÀHFWLRO) + WHUPLOHWRWDOORDGHAHFWLRO) + WHUPLOHWRWDOORDGHAHFWLRO) + WHUPLOHWRWDOORDGHAHFWLRO)'HWHUPLQHOLYHORDGGHÀHFWLRQ &DOFXODWH"c)L = (ac)D+L - (ac)D"c)D = (16/11,400 in.4)(5)(120 ft-kip) = 0.84 in. "c)D+L = (16/11,400 in.4)(5)(177 ft-kip) = 1.24 in. - 0.84 in. = 0.4 in. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 123 'HÀHFWLRQ([DPSOH Chapter in the concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER0RPHQWRILQHUWLDRIDFUDFNHG7VHFWLRQZLWKWHQVLRQUHLQIRUFHPHQW— Determine the cracked-section moment of inertia, Icr, for the section shown in Fig. E3.1. Serviceability Given: n=9 b = 45 in. bw = 24 in. hf = 6.5 in. d = 35.1 in. As = 18 No. 11 Fig. E3.1—T-section. ACI 318-14 Discussion Step 1: Determine constants in. 6.5 in. -1 [] n. / 35.1 35 in. 24 in.  $\beta c = = 0.54$  0.3 Find Ki2. )URP'HVLJQ\$LG'HÀHFWLRQ )URP'HVLJQ\$LG'HÀHFWLRQ )RU\u00fc dwn = 0.3, and hf/2d  $\partial Q Ki2 = 0.144$ , QWHUSRODWLQJIRU\u00fc = 0.54, Ki2 = 0.142 Step 2: Cracked moment of inertia Determine cracked-section moment of inertia. Calculate Icr = Ki2bwd3 Icr = 0.142(24 in.)(35.1 in.)3 = 147,000 in.4 American Concrete Institute - Copyrighted © Material - www.concrete.org 124 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 'HÀHFWLRQ([DPSOH Moment of inertia of a cracked section with tension and compression reinforcement— Determine the crackedsection moment of inertia, Icr, for the section shown in Fig. E4.1. Given: n=9 b = 18 in. h = 40 in. d = 35.4 in. dg LQ As = 13.62 in.2 Asg LQ2 Fig. E4.1—Section of double reinforced beam. \$&,[] D Discussion Step 1: Determine constants &DOFXODWHFRQVWDQWVIRU'HVLJQ\$LG'HÀFWLRQ DQ 'HVLJQ\$LG'HÀ FWLRQ 13.6.1. &DOFXODWHd, As/bd d LQ LQîLQ LQî & DOFXODWHd 9 Asobd LQ2)/(18 iin. × 35.4 in.) = 0.0103 d 9 LQ & DOFXODWHd n 0.0214 = 0.193 d n = 0.0214(9)  $\beta c = \rho'(n-1) \rho n \beta c = Calculate Gg/d 24.2.3.5 Calculation C (0.0103)(9-1) = 0.427 0.193 Gg/d = 2.6 in./35.4 in. = 0.0734 Find Ki2. URP'HVLJQ$LG'HÀHFWLRQ URP'HVLJQ$LG'HÀHFWLRQ NUC d n = 0.193, and$ Go/d = 0.0734, ¿OGKi2 = 0.098 )RUüc d DOGdod = 0.0734, ¿OGKi2 = 0.101 ,OWHUSRODWLOJIRUüc = 0.427, Ki2 = 0.099 Step 2: Cracked moment of inertia 'HWHUPLOHFUDFNHG WRILOHUWLD Calculate Icr = Ki2bwd3 Icr = 0.099(18 in.)(35.4 in.)3 = 79,100 in.4 American Concrete Institute - Copyrighted © Material www.concrete.org CHAPTER 13—SERVICEABILITY 125 'HÀHFWLRQ([DPSOH /LYHORDGGHÀHFWLRQRIDFRQWLQXRXVEHDP<sup>2</sup> 'HWHUPLQHWKHGHÀHFWLRQRIDFRQWLQXRXVEHDP<sup>2</sup> 'HWHY 40,000 psi n=9 wc = 145 lb/ft3 Fig. E5.1—Moment —Mo and nd cross ssections of a continuous beam. Note that Section 24.2.3.7 of ACI 318-14 -14 permits the use se of Ie calculated calcul d fo for the cross sect section at midspan (Section C of this This more simple i approach is ill illustrated in this example as Alternative 1 in Steps 4 through 8. example) rather than the average Ie. T ACI 318-14 Discussion Step 1: Gross moment of inertia Calculate gross 17(14) Step 2: Cracked moment of inertia 24.2.3.5 Determine Icr for each cross section. For Sections A and B, calculate & As/bd
& As/bd & As/b = Ki2bd3 = Ki2(20 in.)(31.2 in.)3 Icr = 33,400 in.4 51,000 in.4  $\beta c = (n - 1)\rho' \rho n$  Find Ki1IURP'HVLJQ\$LG'HÀHFWLRQ Section C d LQ2)/(20 in. × 31.2 in.) = 0.053(20 in in.)(31.2 in.)3 = 32,200 in.4) RU6HFWLRQ&FDOFXODWHd Ås /bd. Step 3: Cracked moment Determine cracking kin moment. ment. Find KcrIURP'HVLJQ\$LG'HÀHFWLRQ 'H \$LG'HÀHFWLRQ Calculate Mcr = 6.60 Step 4: Effective moment of inertia tia Determine Ie for the dead-load moments. Section B 0.471 0.779 C 0.943 0.491 Calculate Mcr/Ma = 132/Ma Calculate Icr/Ig = Icr /65,500 Mcr /Ma Icr /Ig A 0.943 0.510 Find Ki3IRUHDFKVHFWLRQIURP'HÀHFWLRQ Ki3 0.921 0.802 0.918 Calculate average Ie = (60,300 + 52,500 + 60,100)/3 = 57,600 in.4 24.2.3.7 Alternative 1\*: Use Ie at C Ie = 60,100 in.4 24.2.3.5 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY Find Ka3 for span with both positive and negative PRPHQWVIURP'HVLJQ\$LG'HÀHFWLRQ Calculate "c = (Ka3/Ied)[Mc - 0.1(MA + MB)]Ka1 24.2.3.7 Alternative 1\* For 40 ft span, fcg SVLDQG wc = 145 lb/ft3, read Ka1 = 18.25 For Case 7, read Ka3 = 5.0 Serviceability 6WHS'HDGORDGGHÀHFWLRQ 'HWHUPLQHGHDGORDGGHÀHFWLRQ Find Ka1 from Design Aid, 'HÀHFWLRQ 127 °c = (5/57,600 in.4)[140 ft-kip - 0.1(140 ft-kip - 0.1(140 ft-kip)](18.25) = 0.155 in. °c = (5/60,100 in.4)[140 ft-kip - 0.1(140 ft-kip)](18.25) = 0.149 in. Step 6: Effective moment of inertia Determine le for live load plus dead load. A Section B C Calculate Mcr /Ma = 132/MD+L Use Icr /IgFDOFXODWHGDERYHDQG¿QGK Kii33 from 'HVLJQ\$LG'HÀHFWLRQ Mcr/Ma 0.412 0.242 0.402 Ki3 00.544 0.782 0.523 Calculate le = Ki3Ig Ie 35,600 iin.4 51,200 in.4 54,300 in.4 24.2.3.6 Calculate average age Ie TI TE/3 Average A erag Ie =  $(35,600\ 35,600\ +\ 551,200\ +\ 34,300)/3\ 40,40\ in.4 = 40,400\ 24.2.3.7\ se$  Ie at C Alternative 1\*: Use Ie = 34 34,300 in. n.4 6WHS7RWDOGHÀHFWLRQ XV GHDG 'HWHUPLQHWRWDOGHÀHFWLRQIRUOLYHORDGSOXVGHDG load. 24.2.3.6 Calculate: "c)D+L = (Ka3/Ie)[Mc]MA + MB]Ka1 "c)D+L =  $(5/40,400\ in.4)[328\ ft-kip -$ 0.1(320 ft-kip + 545 ft-kip)]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.642 in. Alternative 1 6WHS/LYHORDGGHÀHFWLRO 24.2.3.6 'HWHUPLOHOLYHORDGGHÀHFWLRO 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 in. where Ka1LVREWDLOHGIURP'HVLJO\$LG'HÀHFWLRO 13.8.1. 24.2.3.7 °c) + 545 ft-kip]18.25 = 0.545 ft-kip]18.25 = 0.545 ft-kip]18.25 = 0.545 ft-kip]18.25 = 0.545 ft-kip]18.25 = 0.545 ft-kip]18.25 = 0.545 ft-kipin. - 0.155 in. = 0.39 in. Alternative 1 "c)L = 0.642 in. - 0.149 in. = 0.49 in. \* ACI 318-14 Section 24.2.3.6 allows Alternative 1. American Concrete Institute - Copyrighted © Material - www.concrete.org 128 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 'HÀHFWLRQ([DPSOH Effective moment of inertia of a rectangular beam with tension reinforcement— Determine the effective moment of inertia, Ie, for the rectangular section shown in Fig. E6.1. Given: As = 0.4 in. b = 12 in. fcq SVLQRUPDOZHLJKWFRQFUHWH fy = 60,000 psi Ma = 7.5 ft-kip Fig. E6.1—Beam cross section. ACI 318-14 Discussion Step 1: Gross moment of inertia 19.2.2.1 'HWHUPLQHd.n. 20.2.2.2 &DOFXODWHd. As/bd Find n = Es /EcIURP'HVLJQ\$LG'HÀHFWLRQ (for practical use, n can be taken as nearest whole number) &DOFXODWHd. Calculate Ig = bhh3/1 [WU ¿EHUVWUHVVLQWHQVLRQ1 n the gross section. er yt = h/2 it = Mayt/Ig, where rti Step 2: Effective moment of inertia Determine Ie. Find Ki3IURP'HVLJQ\$LG'HÀHFWLRQ Calculate Ie = Ki3Ig Calculate Ie (12,000)(in.)/(512 in.4) = 703 psi Use graph raph to obtain obt Ki3. In left part of chart, on vertical line denoting fco SVLORFDWH¿EHUVWUHVV SVL\*RKRULzontally to right to main chart and draw line slanting toward le/Ig = 1.0. Then, at upper left side of the chart, read dn = 0.036 and proceed horizontally to the right until meeting d/h = 0.8. Drop vertically to intersect the slanted line previously drawn. Then proceed horizonWDOO\WRWKHULJKW[KDQGVLGHRIWKHJUDSKDQGUHDG Ie/Ig = 0.51. 7KLVSURFHGXUHLVVKRZQLQ'HVLJQ\$LG'HÀHFWLRQ 13.7.2 with heavy lines and arrows. Ie/Ig = 0.51 in.4 American Concrete Institute – Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 129 'HAHFWLRQ([DPSOH Cracking moment for T-section— Determine the cracking moment Mcr for the section shown in Fig. E7.1, for use in ACI 318-14 Eq. (24.2.3.5a). Given: Serviceability fcg = 3000 psi wc = 145 lb/ft3 bw = 18 in. b = 90 in. h = 40.5 in. hf = 4.5 in. Fig. E7.1—T-section. ACI 318-14 Discussion Step 1: Determine constants 24.2.3.5 Find Kcr for normalweight concrete. 24.2.3.5b 19.2.3.1 Calculation )URP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h =
40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. Determine value of KcrtIURP'HVLJQ\$LG'HÀHFWLRQ For h = 40.5 in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.36 ft-kip/in. and fco SVLUHDG Kcr = 9.3 in.)/(40.5 in.) n.) = 0.11 h = (4.5 Design Aid, Find Kcrt from Desi 'HAHFWLRQ m Step 2: Determine cracked moment, re For negative moment, read Kcrt = 2.3 Fo pos For positive moment, oment, Mcr = (1 (18 in.)(9.36 (9.36 ft-k ft-kip/in.)(1.36) = 229 ft-kip)) (1.36) = 229 ft-kipgative mom For negative moment, Mcrr = (18 in in.)(9.36 ft-kip/in.)(2.3) = 387 ft-kip American Concrete Institute - Copyrighted © Material - www.concrete.org 130 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 'HÀHFWLRQ([DPSOH 7ZRZD\VODEGHÀHFWLRQ^2 'HWHUPLQHWKHGHDGORDGOLYHORDGDQGORQJWHUPGHÀHFWLRQVDWDWZRZD\UHLQIRUFHGFRQFUHWHVODEVKRZQLQ)LJ()URP two-way cast-in-place reinforced with No. 5 at 14 in. on center at midspan (bottom reinforcement). 'HÀHFWLRQOLPLWVLQ7DEOH\$&, DUHEIRULPPHGLDWHOLYHORDGGHÀHFWLRQDQGEIRUWRWDOORDGHÀHFWLRQDQGEIRUWRWDOORDGHÀHFWLRQDQGEIRUWRWDOORDGHÀHFWLRQDQGEIRUWRWDOORDGHÀHFWLRQDQGEIRUWRWDOORDGHA 6WHS'HÀHFWLRQGHIRUPDWLRQ 7ZRZD\VODEPD[LPXPGHÀHFWLRQ<sup>m</sup>, calculation in the perpendicular direction as shown in Fig. E8.2). Calculation <sup>m</sup>ax <sup>c</sup>, x <sup>m</sup>, y or <sup>m</sup>ax <sup>c</sup>, x <sup>m</sup>, y or <sup>m</sup>ax <sup>c</sup>, x <sup>m</sup>, y or <sup>m</sup>ax <sup>c</sup>, x <sup>m</sup>, y or <sup>m</sup>ax <sup>c</sup>, y <sup>m</sup>, x American Concrete Institute -Copyrighted © Material – www.concrete.org 131 Serviceability CHAPTER 13—SERVICEABILITY )LJ(<sup>2</sup>7ZRZD\VODEGHAHFWLRQGHIRUPDWLRQ & RQVLGHUWKDWWKHVXSSRUWVDUHIXOO\¿[HGD]DLQVW rotation and vertical displacement. From Reinforced Concrete Design Handbook Design Aid – Analysis Tables, which can be downloaded

from: . aspx?ItemID=SP1714DAWKHPD[LPXPGHAHFWLRQ for continuous spans with distributed loading is: ΔC , x,D = αqA 4 384 EI e where q is the applied lied lload; E is the pan in the HFWLR XODWLRQDQG e is thee GLUHFWLRQFDOFXODWLRQDQGI nt of inertia rtia taken as th verage effective moment the average m off inertia at the su ports and of effective moment supports midspan. 6WHS/RDGFDOFXODWLRQ )RUGHAHFWLRQFDOFXODWLRQXVHWKHJUHDWHVWRI XO WK (a) Service load, 1.0(D + L) (b) Assume a construction load of twice thee slab self-weight: 8.10.3.2 qconst = (2)(75 psf) = 150 psf For more information on construction loading, refer to Scanlon and Suprenant (2011), "Estimating Two:D\6ODE'HÀHFWLRQV Concrete International9 33, No. 7, July, pp. 24-29. Total moment within a span and E2 is the slab span in the perpendicular direction. Total service load moment within a span due to controlling load of 190 psf above: 8.10.4.1 Moment distribution due to dead load: 8.10.5.1 Negative moment – column strip: middle strip 8.10.5.5 D + L = 75 psf + 15 psf + 100 psf = 190 psf (0.75)(55 ft-kip) = 30 ft-kip (0.25)(55 ft-kip) = 30 ft-kip (0.25)(55 ft-kip) = 41 ft-kip (0.25)(55 ft-kip) = 41 ft-kip (0.25)(55 ft-kip) = 55 ft-kip (0.25)(55 ft-kip) = 30 ft-kip (0.25)(55 ft-kip) = 41 ft-kip (0.25)(55 ft-kip) = 55 ft-kip (0.25)(55 ft-kip) = 30 ft-kip (0.25)(55 ft-kip) = 41 ft-kip (0.25)(55 ft-kip) = 41 ft-kip (0.25)(55 ft-kip) = 41 ft-kip (0.25)(55 ft-kip) = 55 ft-kip (0.25)(55 ft-kip) 14 ft-kip (0.60)(30 ft-kip) = 18 ft-kip (0.40)(30 ft-kip) = 12 ft-kip American Concrete Institute – Copyrighted © Material – www.concrete.org 132 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Assume the formwork of the elevated slab is removed and construction load is applied at the early age of 7 days. To calculate the slab cracking moment at early age, ACI 209R-92, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete strength. The following equation can be used: f ci = t f c' at + b where a and b are constants, and t is in days. 19.2.3.1 fcr 3 fc 24.2.3.5 M cr = f r where Ig = The ranges of a and b for normalweight concrete, sand lightweight, and all lightweight, and b = 0.05 to 9.25. For fcgDWGD\VWKHFRHIcFLHQWa = 0.85 and b = 4 f ci = 7 (5000 psi) = 3517 psi (0.85)(7) + 4 Use 3500 psi f cr = (7.5)(3.0) 3500 psi = 444 psi Ig M cr = (444 psi) yt bt 3 (7 ft)(12)(6 in.)3 = = 1512 in.4 12 12 1512 in.4 = 223, 776 in.-lb 3 in. = 18.6 ft-kip < 41 ft-kip Therefore, the slab will crack at the supports when formwork is stripped. and yt = 6 in./2 = 3 in. culation Step 3: Cracked moment of inertia calculation Determine cracked secti section moment of inertia for the column strip at the the support. I cr = 20.6.3.1 bc 3 + nAs (d (d - c) 2 3 Assume 3/4 in. cover cov for or a slab not exposed exp sed to tac with h ground: weather or in contact d = h - cc - 0.5db d = 6 in. - 3/4 in. - 0.625 in./2 = 4.9 in. & DOFXODWHd, As/bd  $\rho$ = Nine No. 5 bars are placed within the column strip area (No. 5 at 9 in. o.c.). Refer to Example 1 of the Two-way Slab Chapter 6 of this Handbook. At time of cracking: n = Es/Ec cLVIRXQGE (7 ft)(12 in./ft)(4.9 in.) 29,000,000 psi 57,000 3500 psi = 8.6 (7 ft)(12)c 2 - (8.6)(9 bars)(0.31 in.2) = 0.00678 (7 ft)(12 in./ft)(4.9 in.) 29,000,000 psi 57,000 3500 psi = 8.6 (7 ft)(12)c 2 - (8.6)(9 bars)(0.31 in.2) = 0.00678 (7 ft)(12)c 2 - (8.6)(9 bars)(0.31 in.2) = 0.00678 (7 ft)(12 in./ft)(4.9 in.) 29,000,000 psi 57,000 3500 psi = 8.6 (7 ft)(12)c 2 - (8.6)(9 bars)(0.31 in.2) = 0.00678 (7 ft)(12)c 2 - (8.6)(9 bars)(12)c 2 - (8.6)(9 b (4.9 in. - c) = 0.2 c = 1.41 in. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 133 2Uc can be calculated from c = kd, where  $k = (\rho n) 2 + 2(\rho n) - \rho n k = 0.288$  Therefore, c = kd c = (0.288)(4.9 in.) = 1.41 in. (same result as above) The early-age cracked moment of inertia in the negative moment region of the column strip is: I cr =  $(7 \text{ ft})(12)(1.41 \text{ in.})3 + (8.6)(9)(0.31 \text{ in.}2) 3 \times (4.9 \text{ in.} - 1.41 \text{ in.}) 2 \text{ Icr} = 78 \text{ in.}4 + 292 \text{ in.}4 = 370 \text{ in.}4 \text{ Step 4}$ : Effective moment of inertia for negative moment of inertia for negative moment. [M](M] I e = |cr||I g + |1 - |cr||Ma / |[M] A | 3 3 ] |I cr|| Rearranging the equation: ation:  $(M \ I e = I cr + (I g - I cr) | cr | (Ma / 3 (18.6 ft-kip) (370 in.) + (1512 in. - 370 in.) | (5 in I e = (37 \ 55 ft-kip) (4 4 3 4 Ie = 414 in.4 Step 5: Weighted average Ie 24.2.3.6) RUFRQWLQXRXVEHDPVRUEHDPV& (positive and negative moments), the code permits to take Ie as the average of values obtained from i Eq$ (24.2.3.5a) for the critical positive and negative moments. I e, avg = I e, [email protected] + I e, midspan + I e, [email protected] 3 I e, avg = 414 in.4 + 1512 in.4 + 414 in.4 = 780 in.4 3 \$&,&RPPLWWHH5^3&RQWURORI'HÀHFWLRQ in Concrete Structures" recommends an alternate equation (Eq. (2.15a)) to calculate the average efIHFWLYHPRPHQWRILQHUWLDLQDEHDPZLWKWZR¿[HG or continuous ends. le,avg = 0.71e,midspan + 0.15((email protected] supp) le,avg = 0.70(1512 in.4) + 0.15(414 in.4) = 1183 in.4 Use weighted average based on ACI 318. It will result LQDODUJHUGHÀHFWLRQ American Concrete Institute Copyrighted © Material – www.concrete.org Serviceability 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated above 134 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 6WHS&ROXPQVWULSGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]:
D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PPHGLDWHGHAHFWLRQV([]: D,PP and continuous over all spans. The factor  $[ZKLFKDFFRXQWVIRUVSDQERXQGDU\FRQGLWLRQV$  is taken as 1 for continuous and uniformly loaded VSDQV7KHUHIRUHFROXPQVWULSLPPHGLDWHGHÀHFWLRQGXHWRGHDGORDGDORQJWKH[D[LV"C,x,D:  $\Delta C$ , x, D = (1.0)qA 4 384 EI  $\Delta C$ , x, D = (75 psf + 15 psf)(0.675)(14 ft)(18 ft)4 (12)3 (384) 5000 psi)(780 in.4) = 0.13 in. The factor 0.675 is the average percentage of the total dead and live load carried by the column strip: (0.75 + 0.6)/2 = 0.675 UHIHUWR6WHS/RDGFDOFXODWLRQ Not all spans may be loaded with live load. Thered in patter fore, live load is placed pattern to produce the RQGLWLR PRVWFULWLFDOORDGFRQGLWLRQGHÀHFWLRQ 7KHIIDFtor in this case is take taken equal to 2. P SLPPHGLDWHGHÀ FWLRQGXH 7KHUHIRUHFROXPQVWULSLPPHGLDWHGHÀHFWLRQGXH 0] D[L C,x,L: WROLYHORDGDORQJWKH[D[LV"  $\Delta C$ , x,L = 2(100 psf)(0.675)(14 psf)(0.67 ft)(18 ft)4 (12 in./ft)3 4)(57,000 5000 psi)(780 in.4) (384) = 0.28 in. n. E/RQJWHUPGHÀHFWLRQ )RUORQJWHUPGHÀHFWLRQGHDGORDGVHOIZHLJKW and superimposed loads) and a percentage of live load assumed as sustained load, are considered. )RUOLYHORDGWREHFRQVLGHUHGLQGHÀHFWLRQFDOFXODtions, this example uses 25 percent as sustained load for the usage of this structure (for normal usage 10 percent is used according to Scanlon and Suprenant [2011]). 1RWH7KLVQXPEHUUHOLHVRQHQJLQHHULQJMXGJPHQW and can be lower than 10 percent, or as high as 100 percent of the live load for warehouses and libraries. 24.2.4.1.1 )RUDGGLWLRQDOWLPHGHSHQGHQWGHAHFWLRQUHVXOWLQJ IURPFUHHSDQGVKULQNDJHRIÀH[XUDOPHPEHUVWKH GHÀHFWLRQVDUHPXOWLSOLHGE\DIDFWRUJLYHQE\WKH following equation: American Concrete.org CHAPTER 13—SERVICEABILITY λΔ = ξ 1 + 50p' λΔ = 135 2.0 = 2 1 + 0 24.2.4.1.3 From Table 24.2.4.1.3, the time-dependent factor for sustained load duration of more than 5 years: h 7KHUHIRUHORQJWHUPGHÀHFWLRQIRUWKHFROXPQVWULS along the x-axis is: "C,x,T 3" t "C,x,T = (1 + 2)(0.13 in. + (0.25)(0.28 in.)) = 0.60 in. 6WHS0LGGOHVWULSGHÀHFWLRQV(: 19.2.3.1 Because Ig for middle strip is the same as Ig for 24.2.3.5 column strip, then: Mcr = 18.6 ft-kip Themiddle strip negative and positive and negative moment at support: an: Positive moment at support: an: Positive moment at midspan: Compare positive and negative ction is moments in the m middle strip, the slab ssection ck and d IgLVXVHGLQWK LVXVHGLQWKHGHÀHFWLRQ GHÀH assumed uncracked calculations: D,PPHGLDWHGHÀHFWLRQ, PPHGLDWHAHFWLRQ, PPHGLDWHGHÀ  $(14 \text{ ft})(18 \text{ ft})4 (12)3 (7 (384)(57,000)(5000 \text{ psi})(1512 \text{ in.4}) = 0.03 \text{ in.}, PPHGLDWHPLGGOHVWULSGHÀHFWLRQGXHWROLYHORDG along the x-axis: } \Delta M$ , x, L = 2(100 psf)(0.325)(14 ft)(18 ft)4 (12 in./ft)3 (384)(57,000)(5000 \text{ psi})(1512 in.4) = 0.07 \text{ in The factor } 0.325 \text{ is the fraction of the total dead and live load carried by the middle strip } (1 - 0.675)(14 ft)(18 ft)4 (12 in./ft)3 (384)(57,000)(5000 \text{ psi})(1512 in.4) = 0.07 \text{ in The factor } 0.325 \text{ is the fraction of the total dead and live load carried by the middle strip } (1 - 0.675)(14 ft)(18 ft)4 (12 in./ft)3 (384)(57,000)(5000 \text{ psi})(1512 in.4) = 0.07 \text{ in The factor } 0.325 \text{ is the fraction of the total dead and live load carried by the middle strip } (1 - 0.675)(14 ft)(18 ft)4 (12 in./ft)3 (18 ft)4 (18 ft)4 (12 i = 0.325). E/RQJWHUPGHÅHFWLRQ 24.2.4.1.1 0XOWLSO\LPPHGLDWHGHÅHFWLRQVE\WKHIDFWRU  $\lambda \Delta = \xi 1 + 50\rho' \lambda \Delta = 2.0 = 2.1 + 0$  Refer to variable values in Step 6. 7KHUHIRUHPLGGOHVWULSORQJWHUPGHÅHFWLRQLV "M,x,T 3" i "M,x,T = (1 + 2)(0.03 in. + (0.25)(0.07 in.)) = 0.14 in. American Concrete Institute – Copyrighted © Material - www.concrete.org Serviceability Assuming there is no compression reinforcement, d.g 136 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSDQGPLGGOHVWULSLPPHGLDWHDQGORQJWHUPGHÀHFWLRQVLQWKHHDVWZHVW(: GLUHFWLRQLQ Immediate & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSDQGPLGGOHVWULSLPPHGLDWHDQGORQJWHUPGHÀHFWLRQVLQWKHHDVWZHVW(: GLUHFWLRQLQ Immediate & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONCRETE DESIGN HANDBOOK & SUMMARY & ROXPQVWULSCARC) CONC 0LGGOHVWULS MS,x Step 6a 7a Dead load 0.13 0.03 Long term 3")(DL + 0.25LL) 0.60 0.14 Live load 0.28 0.07 Step 9: Column strip N-S direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction
7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction 7KHVDPHGHÀHFWLRQFDOFXODWLRQVDUH L) = 190 psf 8.10.3.3 Total moment within a span: qA 2 A 2n 8 Total service load moment middle strip: 8.10.5.1 tion due to dead load: 8.10.5.1 tion due to dea moment of inertia, Icr 19.2.3.1 Because Ig for column strip in the N-S direction is 24.2.3.5 directhe same as Ig for column strip in the E-W direction, then: (90 p sf + 100 psf)(18 ft)(12 ft) 2 8 = 61.6 ft-kip, say, 62 ft-kip M D + L = M = (0.65)(62 ft-kip) ft-) = 40 ft-kip (0 35)(62 ft-kip ft-kip) p) = 22 ft-kip M + = (0.35)(62 30 ft-kip 10 ft-kip 13 ft-kip - kip 9 ft-kip M D + L = M = (0.65)(62 ft-kip) ft-) = 40 ft-kip (0 35)(62 placed within the column strip area (No. 5 at 14 in. o.c.). Refer to Example 1 of the Two-way slab Chapter. 29,000,000 psi At time of cracking: n = Es/Ec cLVIRXQGE\HTXDWLQJWKH¿UVWPRPHQWRIFRPSUHVVLRQDUHDPRGL¿HGE\WKH modular ratio) of the slab: n = bc 2 - nAs (d - c) = 0 2 (7 ft)(12)c 2 - (8.6)(9 bars) (0.31 in. 2)(4.3 in. - c) = 0.2 c = 1.31 in. 57,000 3500 psi = 8.6 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 137 2Uc can be calculated from c = kd, where k = (pn) 2 + 2(pn) - pn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated above Therefore, c = kd Serviceability c = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated from c = kd, where k = (pn) 2 + 2(pn) - pn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated from c = kd, where k = (pn) 2 + 2(pn) - pn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated from c = kd, where k = (pn) 2 + 2(pn) - pn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated from c = kd, where k = (pn) 2 + 2(pn) - pn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated above Therefore, c = kd Serviceability c = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated from c = kd, where k = (pn) 2 + 2(pn) - pn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated above Therefore, c = kd Serviceability c = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated above Therefore, c = kd Serviceability c = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 0.304 6XEVWLWXW(0.304)(4.3 in.) = 1.31 in. (same result as above) The early age cracked moment of inertia in the negative moment region of the column strip is: bc 3 I cr = + nAs (d - c) 2 3 I cr = (7 \text{ ft})(12)(1.31 \text{ in.}) 3 × (4.3 \text{ in.} - 1.31 \text{ in.}) 2 I cr = 63 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text{ in.} 3 + (8.6)(9)(0.31 \text{ in.} 2) 3 × (4.3 \text{ in.} - 1.31 \text{ in.}) 2 I cr = 63 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text{ in.} 4 =
278 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text{ in.} 4 = 278 \text{ in.} 4 + 215 \text (M ) Ie = |cr|Ig + |1 - |cr| Ma / |1 - |cr| Ma / |1 - |cr| Rearranging the equation: ation: (M) Ie = Icr + (Ig - Iccr) |cr| Ma / 3 (18.6 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in. in 4 - 278 in.4) |Ie = (278 (40 ft-kip) (27 in.4) + (1512 (5 in.1)in.4 + 402 in.4 = 772 in.4 3 The difference is negligible, 772 in.4 compared to 780 in.4: Ie = 772 in.4 D,PPHGLDWHGHAHFWLRQVLQFROXPQVWULS Repeating the same calculations, the following is obtained (refer to Step 6a): ,PPHGLDWHGHAHFWLRQXHWRGHDGORDG along the y-axis:  $\Delta C$ , y,D =  $\alpha qA$  4 384 EI  $\Delta C$ , y,D  $(75 \text{ psf} + 15 \text{ psf})(0.675)(18 \text{ ft})(14 \text{ ft})4 (12)3 384(57,000)(5000 \text{ psi})(772 \text{ in.}4) = 0.06 \text{ in.}, PPHGLDWHFROXPQVWULSGHAHFWLRQGXHWROLYHORDG along the y-axis: Assume that the same factor of 0.675 applies as a percentage of the total dead and live load carried by the column strip. <math>\Delta C$ ,  $y_L = 2(100 \text{ psf})(0.675)(18 \text{ ft})(14 \text{ ft})4 (12 \text{ in.}/\text{ft})3 (384)$ (57,000)(5000 psi)(772 in.4) = 0.14 in. American Concrete Institute - Copyrighted © Material - www.concrete.org 138 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) E/RQJWHUPGHÀHFWLRQ 24.2.4.1.1 0XOWLSO\LPPHGLDWHGHÀHFWLRQVE\WKHIDFWRU λΔ = ξ 1 + 50ρ' λΔ = 2.0 = 2 1 + 0 Refer to variable calculation in Step 6: 7KHUHIRUHORQJWHUPGHÀHFWLRQLV "C,y,T 3" i 6WHS0LGGOHVWULSGHÀHFWLRQV16 D,PPHGLDWHGHÀHFWLRQ 19.2.3.1 Calculate cracked moment: fcr 3 fc' 24.2.3.5 M cr = f r "C,y,T = (1 + 2)(0.06 in. + (0.25)(0.14 in.)) = 0.29 in. f cr = (7.5)(3.0) 3500 psi = 444 psi Ig where I g = (18 ft - 7 ft)(12)(6 in.) = 2.376 in.4 12 Ig = yt bt 3 12 and yt = 6 in./2 = 3 in. Is Mcr > Ma? 2376 in.4 = 351, 648 in. -lb 3 in. = 29.3 ft-kip M cr = (444 psi) Mcr = 29.3 ft-kip > Ma = 10 ft-kip > (2376 in.4) 384(57,000)(= 0.01 in.  $\Delta$  M, y,L = The factor 0.325 is the fraction of the total dead and live load carried by the middle strip (1 – 0.675 = 0.325). E/RQJWHUPGHÀHFWLRQ 24.2.4.1.1 0XOWLSO(LPPHGLDWHGHÀHFWLRQ 24.2.4.1.1 0XOWLSO(14 ft)4 (12)3 2(100 psf)(0.325)(18 ft)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100
psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft)4 (12)3 2(100 psf)(14 ft) in./ft)3 (384)(57,000)( 5000 psi)(2376 in.4) = 0.02 in. λΔ = 2.0 = 2 1 + 0 Refer to variable calculation in Step 6: 7KHUHIRUHORQJWHUPGHÀHFWLRQLC "M,y,T 3" i Step 12: Summary "M,y,T = (1 + 2)(0.01 in. + (0.25)(0.02 in.)) = 0.05 in. & ROXPQVWULSDQGPLGGOHVWULSDQGPLGG Immediate /RQJWHUP Step Dead load /LYHORDG 3")(DL + 0.25LL) 9a 0.06 0.14 0.29 & ROXPQVWULS"C, y 10a 0.01 0.02 0.05 0LGGOHVWULS"M, y American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY "L, midspan = 0.28 in. + 0.02 in. = 0.30 in. ZKHUHLQDQGLQDUHWKHVKRUWWHUPGHÀHFtions for the column strip and middle strip calculated in E-W and N-S directions in (Step 8 and 11), respectively (refer to Fig. E8.3). )LJ(<sup>2</sup>, PPHGLDWHGHÀHFWLRQGXHWROLYHORDG PHG GHÀHFWLRQGXHWR YHORDG 6WHS\$OORZDEOHGHÀHFWLRQ 24.2.2 &, FRGHOLPLWIRULPPHGLDWHGHÀHFWLRQGXHWROLYH IR PHGLDWHGHÀHFWLRQLQ&RQFUHWH Structures": A = A 2x + A 2y A = (18 ft) 2 + (14 ft) 2 = 22.8 ft (22.8 ft)(12 in./ft) = 0.78 in. > \Delta L, midspan = 0.30 in. OK 360 24.2.2 Assume that the attached nonstructural elements supported by the slab will not be damaged due to H[FHVVLYHGHÀHFWLRQ\$GHTXDWH]DSLVSURYLGHG between top of wall and bottom of slab. Therefore, use ACI Code limit EIRUORQ]WHUPGHÀHFWLRQ occurring after attachment of nonstructural elements. This value is the sum of the time dependent GHÀHFWLRQGXHWRDOOVXVWDLQHGORDGVDQGWKHLPPHGLDWHGHÀHFWLRQGXHWRDQ\DGGLWLRQDOOLYHORDG (refer to Fig. E8.4). Δ all = (22.88 ft) (12 in./ft) = 1.1 in. 240 American Concrete Institute – Copyrighted © Material – www.concrete.org Serviceability 6WHS7RWDOGHÀHFWLRQ &DOFXODWHGLPPHGLDWHGHÅHFWLRQGXHWROLYHORDG LPPHGLDWHFROXPQVWULSOLYHORDGGHÅHFWLRQLQ16GLUHFWLRQ "L,T "C,x,L"M,y,L 139 140 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 7RWDOGHÀHFWLRQDIWHUDWWDFKPHQWRIQRQVWUXFWXUDO elements, partitions. Assume that 80 percent of the GHDGORDGGHÀHFWLRQVKDYHRFFXUUHGSULRUWRLQVWDOOing the partitions: "max "D"L"LT where "D "C,x,D"M,y,D "L "C,x,L"M,y,L "LT "C,x,T"M,y,T From Steps 8 and 12 "D = 0.13 in. + 0.01 in. = 0.14 in. "LT = 0.60 in. + 0.05 in. = 0.65 in. "max = (0.2)(0.14 in.) + 0.30 in. + 0.65 in. = 0.98 in., say, 1 in. "all LQ!"max = 1.0 in. OK )LJ(27RWDOORQJWHUPGHAHFWLRQ American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 141 Notes: • If ELVWDNHQHTXDOWRIWWKHQWKHFDOFXODWHGDOORZDEOHGHAHFWLRQLVHTXDOWRLQZKLFKLVOHVVWKDQWKH PD[LPXPFDOFXODWHGGHAHFWLRQRIDFROXPQHQGVDUH¿[HGDWWKHARRUVDERYHDQGEHORZ7KHURWDWLRQRIDFROXPQDWWKHVODEOHYHO is given by: θ= ΔM K ec ZKHUHsLVWKHURWDWLRQUDGLDQV<sup>™</sup>MLVWKHGLIIHUHQFHLQARRUPRPHQWRQHLWKHFROXPQLQOE and Kec is the equivalent column stiffness, in.-lb/radiant ‡)URPWKHFDOFXODWHGURWDWLRQWKHFRUUHVSRQGLQJGHAHFWLRQDWPLGVSDQLVGHWHUPLQHGIURPWKHIROORZLQJ equation: Δ rot = θA 8 <sup>‡</sup>7KH¿QDOGHÅHFWLRQDWPLGVSDQLVWKHVXPRIWKHWRWDOGHÅHFWLRQGXHWRGHDGDQGOLYHORDGDQGOLYHORDGDQGWKHURWDWLRQDO WKHVXPRIWKHWRWDOGHÅH GULJKWVXSSRUWV GHÅHFWLRQIURPWKHOHIWDQGULJKWVXSSRUWV "rot-r "total "max" rot-left" rot-right HWZ IRU D WHULRUVODE WKDWWKHWZRZD\VODEGHAHFWLRQFDOFXODWLRQDUHIRUDQLQWHULRUVODEZKLFKLVFRQWLQXRXVRYHUERWK for it is reasonable to assume ume that the column umn reactions ons are ne supports. Therefore, negligible: "rot-left "rot-rightt = 0 in. 8.3.1.1 XV RQWZRZD\VODE HAHFWLRQUHIHUW \$& 3&RQ )RUIXUWKHUGLVFXVVLRQRQWZRZD\VODEGHÀHFWLRQUHIHUWR\$&,53&RQWURORI'HÀHFWLRQLQ&RQFUHWH Structures." To satisfy ACI 318-14 minimum slab thickness must be at least: En/33 where En is the clear ar En = 18 fft – (12 in./12)2 = 16 ft span. Therefore minimum slab thickness: En/33 = 192 in./33 = 5.8 in. Use 6 in. thick slab 1RWH7KHWZRZD\VODEVDWLV¿HVWKHPLQLPXPWKLFNQHVVRIQRQSUHVWUHVVHGWZRZD\VODEVZLWKRXWLQWHULRU beam thickness (ACI 318-14, Section 8.3.1.1). En/33 = 192 in./33 = 5.8 in. 'HAHFWLRQFDOFXODWLRQZRXOGQRWKDYHEHHQUHTXLUHG,QWKLVH[DPSOHKRZHYHUFDOFXODWHGPD[LPXPGHAHFWLRQH[FHHGVWKHDOORZDEOH7KHUHIRUHGHAHFWLRQVVKRXOGEHFDOFXODWHGIRUWZRZD\VODEVZLWKOLYHORDGRI SVIDQGKLJKHUDVWKHFRHI¿FLHQWVLQ7DEOHPD\UHVXOWLQXQDFFHSWDEOHGHÀHFWLRQV 6WHS'HÀHFWLRQV 6WHS'HÀHFWLRQVLQWKHRWKHUGLUHFWLRQ 7KHWRWDOFDOFXODWHGGHÀHFWLRQVPD\QRWEHHTXDO %XWEHFDXVHRIJHRPHWULFFRPSDWLELOLW\WKH¿QDO ORQJWHUPGHÀHFWLRQDWPLGVSDQPXVWEHDVLQJOH value. 7KHUHIRUHWKHDYHUDJHRIWKHWZRFDOFXODWHGGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKH¿QDOGHÀHFWLRQLVWKHQFRQVLGHUHGDVWKHĮ www.concrete.org Serviceability \$7KHFDOFXODWHGPD[LPXPGHÀHFWLRQ"max = 1.0 in., still needs to be corrected for the rotations of the HTXLYDOHQWIUDPHDWWKHVXSSRUWVZKLFKKDVEHHQDVVXPHG2[HGLQWKHH[DPSOHDERYH 142 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 13.12—Deflection design aids DEFLECTION 13.1—Cracking moment Mcr for rectangular sections Reference: ACI 318-14, Section 24.2.3.5 Kcr = (fr /12,000)(h2/6) = [(7.5 fc')(h2)/72,000 = (h2 fc')/9600, ft-kip, for "sand-lightweight" concrete Mcr = bKcr(0.85), ft-kip, for "sand-lightweight" concrete Mcr = bKcr(0.75), ft-kip, for "sand-lightweight" concrete Mcr = bKcr(0.75), ft-kip, for "sand-lightweight" concrete Mcr = bKcr(0.75), ft-kip, for "sand-lightweight" concrete Mcr = bKcr(0.75), ft-kip, for "sand-lightweight" concrete Mcr = bKcr(0.85), ft-kip, for "sand-ligh )RUÀDQJHGVHFWLRQMcr = bwKcrKcrt ; obtain KcrtIURP'HÀHFWLRQRUDQG )RUXVHRIWKLV'HVLJQ\$LGUHIHUWR'HÀHFWLRQ([DPSOHVDQG American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 13—SERVICEABILITY 143 DEFLECTION 13.2—Cracking moment Mcr for T- or L-sections with tension at the bottom (positive moment) M cr = f r bw h 2 K crt = bw K cr K crt 72, 000 K crt = 1 + ( $\alpha$  b - 1) $\beta$  h [4 - 6 $\beta$  + 4 $\beta$  2h + ( $\alpha$  b - 1) $\beta$  h [4 - 6 $\beta$  + 4 $\beta$  2h + ( $\alpha$  b - 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LGUHIHUWR'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH
American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -  $\beta$  h ) Obtain KcrIURP'HVLJQ\$LG'HÅHFWLRQ ([DPSOH American Concrete.org Serviceabilit, b + 1) $\beta$  h (2 -Reference: ACI 318-14, Section 24.2.3.5 144 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) DEFLECTION 13.3.1—Cracking moment McrIRU7RU/VHFWLRQVZLWKWHQVLRQDWWKHWRSQHJDWLYHPRPHQW uh = 0.10, 0.15, and 0.20 Reference: ACI 318-14, Section 24.2.3.5 M cr = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt = bw K crt 72, 000 K crt = f r bw h 2 K crt 72, 000 K crt 84, 000 K 1 + (\alpha b - 1)\beta h [4 - 6\beta h 2 + (\alpha b - 1)\beta h 3 ] 1 + \beta h 2 (\alpha b - 1) Obtain KcrIURP'HVLJQ\$LG'HÀHFWLRQ ([DPSOH American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 145 DEFLECTION 13.3.2—Cracking moment  $McrIRU7RU/VHFWLRQVZLWKWHQVLRQDWWKHWRSQHJDWLYHPRPHQW\ \ b = 0.25,\ 0.30,\ and\ 0.40\ M\ cr = f\ r\ bw\ h\ 2\ K\ crt = bw\ K\ crt\ = 1 + (\alpha\ b - 1)\beta\ h\ 2\ (\alpha\ b - 1)\beta\ b\ 2\ (\alpha\ b - 1)\beta\ b\ 2\$ Concrete Institute - Copyrighted © Material - www.concrete.org Serviceability Reference: ACI 318-14, Section 24.2.3.5 146 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) '()/(&7,212&UDFNHGVHFWLRQPRPHQWRILQHUWLDIcr for rectangular sections with tension reinforcement only Reference: ACI 318-14, Section 22.2.2 Icr = Ki1bd3 K i1 = (c /d )3 +  $\rho$ n[1 - (2c /d ) + (c /d ) 2] 3  $\rho$ = As bd )RUXVHRIWKLV'HVLJQ\$LGUHIHUWR'HÅHFWLRQ([DPSOHVDQG American Concrete.org CHAPTER 13—SERVICEABILITY 147 '()/(&7,21<sup>2</sup>\*URVVPRPHQWRILQHUWLDIg for T-section Ig = Ki4 = (bwh3/12) 3(1 -  $\beta$  h ) 2 ( $\beta$  h )( $\alpha$  b - 1) 1 ( $\alpha$  b - 1) 1 ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )( $\beta$  h )( $\alpha$  b - 1) ( $\beta$  h )(+  $\beta$  h ( $\alpha$  b - 1) Serviceability K i 4 = 1 + ( $\alpha$  b - 1) $\beta$ 3h + ([DPSOH)RUWKH7[]EHDPVVKRZQ¿QGWKHPRPHQWRILQHUWLDIg: [b = b/bw = 143/15 = 9.53 \u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22
,QWHUSRODWLQJEHWZHHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHQWKHFXUYHVIRU\u00fch a + h/f + 8/36 = 0.22 ,QWHUSRODWLQJEHWZHQWA + h/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f + 10/6 ,AH/f Copyrighted © Material – www.concrete.org 148 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) '()/(&7,21<sup>2</sup>&UDFNHGVHFWLRQPRPHQWRILQHUWLDIcr for rectangular sections with compression steel, or T-sections (values of Ki2 IRUùc from 0.1 through 0.9 Reference: ACI 318-14, Section 22.2.2 Icr = Ki2bwd3 2 [ ( c /d ) 3 [ ] d  $0.123\ 0.1\ 0.02\ 0.015\ 0.028\ 0.039\ 0.049\ 0.058\ 0.066\ 0.074\ 0.081\ 0.088\ 0.095\ 0.101\ 0.107\ 0.1\ 0.10\ 0.015\ 0.028\ 0.039\ 0.048\ 0.057\ 0.065\ 0.072\ 0.079\ 0.086\ 0.092\ 0.098\ 0.104\ 0.109\ 0.114\ 0.119\ 0.1\ 0.30\ 0.015\ 0.028\ 0.038\ 0.048\ 0.057\ 0.064\ 0.072\ 0.079\ 0.086\ 0.092\ 0.098\ 0.092\ 0.098\ 0.104\ 0.109\ 0.114\ 0.119\ 0.1\ 0.30\ 0.015\ 0.028\ 0.038\ 0.048\ 0.057\ 0.065\ 0.072\ 0.079\ 0.086\ 0.092\ 0.098\ 0.092\ 0.098\ 0.104\ 0.109\ 0.114\ 0.119\ 0.1\ 0.30\ 0.015\ 0.028\ 0.038\ 0.048\ 0.057\ 0.064\ 0.072\ 0.079\ 0.086\ 0.092\ 0.098\ 0$  $0.085\ 0.091\ 0.097\ 0.103\ 0.108\ 0.113\ 0.117\ 0.1\ 0.40\ 0.015\ 0.028\ 0.039\ 0.0\ 0.049\ 0.058\ 0.066\ 0.074\ 0.082\ 0.091\ 0.096\ 0.091\ 0.096\ 0.091\ 0.098\ 0.105\ 0.112\ 0.118\ 0.124\ 0.130\ 0.2\ 0.015\ 0.028\ 0.039\ 0.0\ 0.049\ 0.058\ 0.066\ 0.074\ 0.082\ 0.089\ 0\ 0.096\$  $0.015\ 0.028\ 0.040\ 0.050\ 0.060\ 0.069\ 0.079\ 0.087\ 0.096\ 0.104\ 0.112\ 0.120\ 0.128\ 0.039\ 0.048\ 0.057\ 0.065\ 0.073\ 0.080\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\ 0.075\ 0.083\ 0.090\ 0.098\$ 0.085 0.091 0.097 0.103 0.108 0.113 0.116 0.7 0.02 0.016 0.029 0.041 0.053 0.063 0.074 0.084 0.094 0.104 0.113 0.123 0.132 0.141 0.150 0.159 0.7 0.10 0.015 0.028 0.040 0.134 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 13—SERVICEABILITY dod or hf /2d Ki2 d.nd.wn for T-sections) 0.02 0.04 0.06 0.08 0.10 0.12 0.14 0.16 0.18 0.20 0.22 0.24 0.26 0.28  $0.039\ 0.048\ 0.057\ 0.064\ 0.072\ 0.079\ 0.085\ 0.091\ 0.097\ 0.103\ 0.145\ 0.015\ 0.028\ 0.091\ 0.097\ 0.085\ 0.091\
0.091\ 0.091\$ <sup>2</sup>&UDFNHG¬VHFWLROPRPHOWRILOHUWLDIcr for rectangular sections with compression steel, or T-sections (values of Ki2 IRUuc from 1.0 through 5.0 Reference: ACI 318-14, Section 22.2.2 Icr = Ki2bwd3 2  $\left[ (c/d) 3 \right] d' (d') \left[ 3Ki 2 = 1 + \rho n 1 - (2c/d) + (c/d) 2 + \rho n\beta c \right] (c/d) 2 - 2c/d + 1$ for T-sections) 0.02 0.04 0.06 0.08 0.10 0.12 0.14 0.16 0.18 0.20 0.22 0.24 0.26 0.28 0.30 1.0 0.02 0.016 0.029 0.042 0.054 0.066 0.077 0.088 0.099 0.110 0.121 0.131 0.142 0.152 0.  $0.076\ 0.085\ 0.093\ 0.101\ 0.109\ 0.117\ 0.125\ 0.132\ 0.140\ 1.0\ 0.30\ 0.016\ 0.028\ 0.030\ 0.044\ 0.057\ 0.065\ 0.073\ 0.081\ 0.088\ 0.095\ 0.073\ 0.081\ 0.088\ 0.095\ 0.073\ 0.081\ 0.091\ 0.97\ 0.97$  $0.065\ 0.0\ 0.073\ .081\ 0.081\ 0.081\ 0.080\ 0.089\ 0.0966\ 0.103\ 0.111\ 0.118\ 0.124\ 0.017\ 0.029\ 0.0399\ 0.0488\ 0.057\ 0.06\ 0.072\ 0.085\ 0.099\ 0.112\ 0.125\ 0.138\ 0\ 0.151\ 0.164\ 0.177\ 0.190\ 0.203\ 2.0\ 0.10\ 0.016\ 0.029\ 0.042\ 0.054\ 0.066\ 0.077\ 0.089$ 0 108 0 117 0 126 0 135 0 144 0 153 2 0 0 30 0 016 0 028 0 038 0 048 0 057 0 065 0 079 0 082 0 090 0 097 0 105 0 112 0 127 0 134 2 0 0 40 0 017 0 029 0 039 0 048 0 057 0 064 0 072 0 079 0 085 0 092 0 098 0 104 0 110  $0.100\ 12\ 0.112\ 0.123\ 0.134\ 0.145\ 0.156\ 0.167\ 0.178\ 2.0\ 0.20\ 0.015\ 0.028\ 0.039\ 0.050\ 0.060\ 0.070\ 0.080\ 0.89\ 0.089\ 0.098$  $0.103\ 0.115\ 0.127\ 0.139\ 0.151\ 0.162\ 0.174\ 0.186\ 2.5\ 0.20\ 0.015\ 0.028\ 0.039\ 0.050\ 0.061\ 0.071\ 0.081\ 0.091\ 0.100\ 0.110\ 0.120\ 0.129\ 0.139\ 0.148\ 0.158\ 2.5\ 0.30\ 0.016\ 0.028\ 0.038\ 0.048$  $0.057\ 0.066\ 0.074\ 0.082\ 0.090\ 0.098\ 0.106\ 0.114\ 0.121\ 0.129\ 0.032\ 0.040\ 0.029\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.040\ 0.049\ 0.049\ 0.040\ 0.049\$  $0.155\ 0.167\ 0.180\ 0.192\ 3.0\ 0.20\ 0.015\ 0.028\ 0.039\ 0.050\ 0.061\ 0.071\ 0.082\ 0.092\ 0.012\ 0.112\ 0.122\ 0.132\ 0.142\ 0.151\ 0.161\ 3.0\ 0.30\ 0.040\ 0.049\ 0.057\ 0.064\ 0.072\ 0.079\ 0.085\ 0.092\ 0.098\ 0.092\ 0.098\ 0.092\$ 0.048 0.063 0.078 0.093 0.108 0.123 0.138 0.123 0.138 0.123 0.138 0.123 0.138 0.123 0.138 0.123 0.146
0.159 0.171 0.184 0.197 3.5 0.20 0.015 0.028 0.099 0.051 0.061 0.072 0.082 0.093 0.103 0.113 0.124 0.134 0.144 0.154 0.164 3.5 0.30 0.016 0.028 0.038 0.048 0.057 0.066 0.075 0.083 0.091  $6\ 0.124\ 0.132\ 0.140\ 3.5\ 0.40\ 0.018\ 0.030\ 0.043\ 0.057\ 0.064\ 0.072\ 0.064\ 0.072\ 0.079\ 0.085\ 0.092\ 0.099\ 0.105\ 0.111\ 0.118\ 0.124\ 4.0\ 0.203\ 0.218\ 0.234\ 4.0\ 0.10\ 0.016\ 0.030\ 0.043\ 0.057\ 0.070\ 0.083\ 0.096\ 0.110\ 0.123\ 0.136\ 0.149\ 0.162\ 0.175\ 0.188\ 0.201$ 0.038 0.048 0.057 0.066 0.075 0.083 0.092 0.100 0.108 0.117 0.125 0.133 0.141 4.0 0.40 0.018 0.030 0.040 0.040 0.018 0.072 0.071 0.084 0.098 0.111 0.120.084 0.093 0.101 0.110 0.118 0.126 0.135 0.143 5.0 0.40 0.019 0.031 0.040 0.049 0.057 0.064 0.072 0.079 0.086 0.092 0.099 0.106 0.112 0.118 0.125 )RUXVHRIWKLV'HVLJQ\$LGUHIHUWR'HÅHFWLRQ([DPSOHVDQG American Concrete Institute - Copyrighted © Material - www.concrete.org Serviceability uc 151 152 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) '()/(&7,21<sup>2</sup>(IIHFWLYHPRPHQWRILQHUWLDIe (values of Ki3) Reference: ACI 318-14, Section 24.2.3.5 Ie = Ki3Ig where 3 3 ( M ) ( ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQRU Ig = Ki3Ig where 3 3 ( M ) ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQRU Ig = Ki3Ig where 3 3 ( M ) ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) Note: For McrUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig where 3 3 ( M ) [ ( M a ) || \ I g ) NOTE: FOR MCRUHIHUWR'HVLJQ\$LG'HÀHFWLRQFORU Ig = Ki3Ig wh bh3/12 or Ki4(bwh3 UHIHUWR'HVLJQ\$LG'HAHFWLRQ )RUXVHRIWKLV'HVLJQ\$LGUHIHUWR'HAHFWLRQ ([DPSOHVDQG American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 153 '()/(&7,21<sup>2</sup>(IIHFWLYHPRPHQWRILQHUWLDIe, for rectangular sections with tension reinforcement only (values of Ki3) Reference: ACI 318-14, Section 24.2.3.5 Ie = Ki3Ig where Serviceability 3 3 ( M ) [ ( M ) ] ( I ) K i 3 = | cr | + | 1 - | cr | | d | Cr | M a / | ] ( I g / ) RUXVHRIWKLV'HVLJQ\$LGUHIHUWR'HÀHFWLRQ([DPSOH American Concrete Institute - Copyrighted © Material - www.concrete.org 154 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) '()/(&7,21<sup>2</sup>&RHI¿FLHQWKa3 and typical McIRUPXODVIRUFDOFXODWLQJLPPHGLDWHGHÀHFWLRQRIÀH[XUDOPHPEHUV Reference: ACI 318-14, Sections 6.5 and 24.2.1  $\Delta c = \sum$  (K a 3 M c) K a 1, in. Ie where Ka1LVIURP'HÀHFWLRQDQGIeLVIURP'HÀHFWLRQRU American Concrete Institute – Copyrighted © Material )RUXVHRIWKLV'HVLJQ\$LGUHIHUWR'HÅHFWLRQ([DPSOHVDQG American Concrete Institute – Copyrighted © Material – www.concrete.org Serviceability K a1 = 156 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) '()/(&7,21<sup>2</sup>&UHHSDQGVKULQNDJHGHÅHFWLRQDGGLWLRQDOORQJ[]WLPHGHÅHFWLRQ GXHWRVXVWDLQHGORDGV Reference: ACI 318-14, Section 24.2.3.5 &UHHSDQGVKULQNDJHGHAHFWLRQ = λ (Δ c), where λ = ξ 1 + 50ρ' Sustained load duration, months 7LPHGHSHQGHQWIDFWRUh per ACI 318-14 Section 24.2.4.1.3 3 1.0 6 1.2 12 1.4 60 or more 2.0 Example:)RUDAH[XUDOPHPEHUZLWKdg WKHLPPHGLDWHGHAHFWLRQXQGHUVXVWDLQHGORDGLVLQ GHAHFWLRQXQGHUVXVWDLQHGORDGLVLQ (0.5) = 1. 1.3 in. 1 + 50(0.005) 2 (0.5) = 1. 1.3 in. 1 + 50(0.005) 0.00 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 13—SERVICEABILITY 157 '()/(&7,21<sup>2</sup>0RGXOXVRIHODVWLFLW\Ec, for various concrete strengths Reference: ACI 318-14, Section 19.2.2.1 and 20.2.2.2 Ec = 33wc1.5 f c' Serviceability n = Es/Ec, where Es = 29,000,000 psi Example: For sand-lightweight concrete, wc = 120 pcf and fcg SVL¿QGWKHPRGXOXVRIHODVWLFLW\Ec and the modular ratio n. Find where wc = 120 on the horizontal axis and proceed vertically upward to fcg SVL7KHQSURFHHGKRUL]RQWDOO\WRWKH left and read Ec = 3.1 million psi and n = 9.5 (use n = 9). American Concrete Institute - Copyrighted © Material - www.concrete.org 158 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute – Copyrighted © Material – www.concrete.org 14.1—Introduction 7KH VWUXWDQGWLH PHWKRG ZDV & UVW LQWURGXFHG E\ 5LWWHU (1899) and Mörsch (1909) around the turn of the twentieth FHQWXU/DWHU6FKODLFKHWDO UHLQWURGXFHGWKLV concept and their work provided the basis for the current strut-and-tie model. The strut-and-tie model is usually applied to the design of individual members such as corbels, pile caps, beams, and girders, but it can also be applied to the design of individual members such as corbels. of the member on each side of a support, concentrated load, change in cross section, ) or edge of an opening (refer to Fig. 14.1a 14.1a—Geometric discontinuous regions. 14.2—Concept QGWLH WKH HQJLQHHU ¿UVW 7R GHVLJQ D EHDP ZLWK VWUXWDQGWLH DWFOR GUDZVD¿FWLWLRXVWUXVVPRGHOWKDWFORVHO\IROORZVWKHIRUFH 7K LRQHOHPHQWVL H ÀRZZLWKLQDPHPEHURUUHJLRQ7KHWHQVLRQHOHPHQWVLQWKH on elements ents are "struts," and truss are "ties," the compression es An engineer can re iew the connection points are "nodes." review IRU VDPHDSSOLHGORD LQJ VHYHUDO¿FWLWLRXVWUXVVPRGHOVIRUWKHVDPHDSSOLHGORDGLQJ LRX WUXVV XVV PRGHO UHVX UHIHU WR )LJ D (DFK ¿FWLWLRXV UHVXOWV LQ LQ different tension and compression fforces in ti ties and struts. Therefore, the engineer chooses the most appropriate truss model, usually the model that most closely follows the force ARZZLWKLQDPHPEHURUUHJLRQ A deep beam with point load(s) (Fig. 14.2b) is commonly used to show how the assumed internal truss transfers load to the supports by a combination of concrete struts and steel ties. Beams designed by strut-and-tie will satisfy strength criteria EXWVHUYLFHDELOLW\FULWHULDVXFKDVGHÀHFWLRQDQGFUDFNFRQWURO PXVWEHYHUL¿HGWKURXJKDQRWKHUDQDO\VLVRUGHVLJQPHWKRG 14.3—Design concrete strength for struts and the area of reinforcement in ties are calculated in accordance with ACI 318-14, Chapter 23. The following discussion and design examples are based on the requirements of ACI 318-14. However, the use of strut-and-tie is an evolving area of strut-and-tie is are tie examples published by ACI (Reineck 2002; Reineck and Novak 2010). 7KHJHQHUDOHTXDWLRQWKDWPXVWEHVDWLV¿HGIRUDQ\HOHPHQW when using the strut-and-tie method is ACI 318, Section 23.3.1. Fig. 14.1b—Loading and geometric discontinuous regions. Fig. 14.1c—Simple supported beam divided into B- and D-regions. []Fn•Fu (14.3) American Concrete Institute - Copyrighted © Material - www.concrete.org 160 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 14.2a—Truss model for top- and bottom-loaded deepp beams tested tes by Leonhardt and Walther (1966). transfer load between two nodes (refer to Fig. 14.4a). In most members, the concrete strut forces can
spread such that the strut is wider in the middle than at the nodes. Such a strut is called bottle-shaped strut (refer to Fig. 14.2b and 14.4b). They are the predominant type of struts in strut-and-tie models of a deep beam where Fu is the factored force, and Fn is the nominal strength RIDVWUXWWLHRUQRGDO]RQH7KHVWUHQJWKUHGXFWLRQIDFWRU[]s 0.75 for struts, ties, or nodal zones. 14.4—Struts Struts are compression concrete elements, generally symbolized as prismatic or uniform tapered elements, generally s the face of the node, and fce is the effective compressive strength of concrete in a strut. The effective compressive strength of concrete in a strut is modeled as an effective compressive strength in a strut, as shown in Table 14.4. A prismatic strut is modeled as an element of a uniform cross section and its reduction factor is equal to 1.0. A bottleshaped strut assumes that the compressive force spreads American Concrete Institute – Copyrighted © Material – www.concrete.org 161 Strut-and-Tie MODEL Fig. 14.4a—Strut-and-tie model of a deep beam with prismatic and d bottle shaped bottle-sh strut elements. Table 14.4—Strut coefficient us e 14.4—St Strut geometry and location Struts with uniform crosssectional area along length Struts located in a region of a member where the width of the compressed concrete at midlength of the strut can spread laterally (bottleshaped struts) Struts located in tension members or the tension zones of members Fig. 14.4b—Bottle-shaped strut: (a) cracking of bottle-shaped strut; and (b) strut-and-tie model of a bottle-shaped strut; and (b) strut-and-tie model of a bottle-shaped strut; and (b) strut-and-tie model of a bottle-shaped strut. shown in Fig. 14.4b. The resulting tension has the effect of reducing the nominal strength of the strut, and WKXVERWWOHVKDSHGVWUXWVKDYHWKHus, reinforcement must resist the splitting tensile force resulting from the spreading compres-sion force. The reinforcement must cross the axis Reinforcement crossing a strut us NA 1.0 (a) Satisfying 23.5 0.75 (b) Not satisfying 23.5 3 (c) NA 0.40 (d) NA 3 (e) All other cases \*Bottle-shaped struts must be assumed in all struts located such that the compressive stress spreads laterally. † 3<sup>2</sup>IRUOLJKWZHLJKWFRQFUHWHYDOXHVREWDLQHGIURP7DEOHRI\$&, of the bottle-shaped struts must be assumed in all struts located such that the compressive stress spreads laterally. † 3<sup>2</sup>IRUOLJKWZHLJKWFRQFUHWHYDOXHVREWDLQHGIURP7DEOHRI\$&, of the bottle-shaped struts must be assumed in all struts located such that the compressive stress spreads laterally. † 3<sup>2</sup>IRUOLJKWZHLJKWFRQFUHWHYDOXHVREWDLQHGIURP7DEOHRI\$&, of the bottle-shaped struts must be assumed in all struts located such that the compressive stress spreads laterally. shaped strut. For beams with concrete strength not exceeding 6000 psi, the requirement to resist the splitting WHQVLOHIRUFHLVVDWLViHGE\\$&,(T  $\geq$  Asi sin  $\alpha$  i  $\geq$  0.03 bs si American Concrete Institute – Copyrighted © Material – www.concrete.org (14.4c) 162 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 14.4c– Reinforcement crossing a strut. where Asi is the total area of surface reinforcement forcement at spacing crossing a strut at an si in the i-th layer of reinforcement cross d bs is i the strut, and to Fig. 14.4c). d fr CI 318, Eq. (23.3) The reinforcement calculated from ACI (23.5.3) RJ GLUHFWLRQVDWDQJ V[1 LVDVVXPHGSODFHGHLWKHULQRUWKRJRQDOGLUHFWLRQVDWDQJOHVĮ DQGĮ2WRWKHD[LVRIWKHVWUXWRULQRQHGLUHFWLRQDWDQDQ]OHĮ LQ GLUHFWLRQDWDQDQ OHĮ nforcement is in only nly parallel to the axis of the strut. If the reinforcement KDQ HJUHHV RQHGLUHFWLRQIPXVWQRWEHOHVVWKDQGHJUHHV extending between two nodal zones. The reinforcement axis must coincide with the axis of the tie in the model. The nominal strength of a tie is calculated by ACI 318, Eq. (23.7.2) Fnt = Ats fy + Atp(fseafp) (14.5) where Ats and Atp are the reinforcement areas in nonprestressed and prestressed ties, respectively; fy and fse are the yield stress in nonprestressed reinforcement and effecWLYHVWUHVVLQSUHVWUHVVLQJVWHHOUHVSHFWLYHO\DQGåfp is the increase in stress in prestressed ties, fseåfp are set equal to zero. For ties with post-tensioned bars or strand, the value of (fse + åfp) must not exceed the yielding stress of the bars or strand. Fig. 14.5—Node element types. 7KHYDOXHRIåfp can be set equal to 60,000 psi and 10,000 psi and 10,000 psi for bonded and unbonded bars or strand, respectively. It is imperative to anchor nonprestressed reinforcement by mechanical devices, standard hooks, or straight bar development, and by anchorage devices for post-tensioning rein-American Concrete Institute - Copyrighted © Material - www.concrete.org 163 Strut-and-Tie CHAPTER 14—STRUT-AND-TIE MODEL Fig.14.6b—Effective width of a strut framing into an exten extended nodal zone. le 14.6—Value Table 14.6—Values of un Nodal al zone z ne Forces on F ditio un condition the node B nded by Bounded struts or be bearing 1.0 CCC reas areas 2QHWL 2QHWLH a ored in anchored 0.8 CCT nodal zone Fnn = fce Anz Fig. 14.5. Although Eq. (14.5) is consistent with the current code, in practice, it is preferable to apply Atpfse as an equivalent external load with a load factor of 1.0 to model the correct ARZRIIRUFHV 14.6—Nodal zone is an assumed volume of concrete around a node that transfers forces from struts and ties through the node. At least three forces mus intersect at a node to be in equilibrium. The strength of a nodal zone is given by ACI 318, Eq. (23.9.1) ACI 318 Figure Table 23.9.2(a) 14.5(b) Table 23.9.2(b) 14.5(c) (14.6a) where fce is the effective concrete compressive strength in the nodal zone; and Anz is the area of the nodal zone face on which Fu acts and perpendicular to the line of action of Fu. The effective concrete compressive strength in a nodel zone is calculated from ACI 318, Eq. (23.9.2) fce unfcg E ZKHUH WKH IDFWRU un accounts for the effective concrete compressive strength in a nodel zone has shown in Table 14.6. faces that are perpendicular to the struts and ties acting on the node and has equal stresses on loaded faces (refer to Fig. 14.5(a)(b)(c) and 14.6a(a)). The nodal zone side lengths wn1:wn2:wn3 are proportional to the struts and ties acting on the node, which results in shear stresses on the face of the node (Fig. 14.6a(b)). American Concrete Institute - Copyrighted © Material - www.concrete.org 164 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 14.7—Usual calculation steps and modeling consideration to apply strut-and-tie model Usual steps to apply the strut-and-tie model: Calculate factored loads on the member (dead, live, wind, and earthquake, per ACI 318, Section 5.3). Fig. 14.6c—Resolution of forces at nodal zone one in a C CCT or CTT, ties must be anchored by a plate, hoo hooks, or straight bar a) thr development (refer to Fig. 14.5(a) through (c) and 14.6a(b)). dal zone (refer to Fig. 114.6a) a) The limits of the extended nodal mpressed zone aat the are determined by outlines of the ties. g. 14.6a(a) a(a) and (b) ar he The darker shaded areas in Fig. are the nodal zones and the light shaded areas are the extended nodal zones. The extended nodal zone area is stressed in compression due to the strut reactions. Forces are transferred from strut tto tie in the extended nodal zone. The effective strut width we is a function of the tie width we, the bearing length Eb, and the angle between the D[LVRIWKHVWUXWDQGWKHKRUL]RQWDOPHPEHUD[LVsUHIHUWR Fig. 14.6b). The tie width may be taken as the width between layers of reinforcement plus twice the distance from the face of the concrete section to the centerline of the closest bar. If more than three struts frame into a nodal zone, it is generally better to resolve some of the strut forces such that only three remain, as shown in Fig. 14.6c. If the support width in the direction perpendicular to the member is less than the member width, transverse forces may require reinforcement to restrain concrete splitting. transverse strut-and-tie model. For each controlling condition: 1. Calculate the member reactions from statics. 2. Check bearing mode stress at external loads and reactions. 3. Use appropriate analysis methods or test results
to estiPDWHWKHARZRIIRUFHVLQWKHPHPEHURUUHJLRQ 4. Create a trial internal truss model made up of struts and WLHVWKDWFORVHO\IROORZVWKHARZRIIRUFHV 5. Check that the strut-and-tie truss model is in equilibrium with the factored applied loads, and the reactions and VWUXWDQGWLHIRUFHVVDWLVI\VWDWLFV\$WD¿UVWWULDOLWLVVXI¿cient to assume the axis of the struts and ties only. 6. Calculate or estimate the required size of each node compressive strength of the node or strut, whichever controls. &KHFNWKDWWKHGHVLJQVWUHQJWKVZLWKHTXDOWR RI the struts, ties, and nodal zones exceed the factored forces. RE FE REFERENCES Au red documents ments Authored /HR KDUG ) DQG QG :DOWK /HRQKDUGW :DOWKHU 5 3:DQGDUWLJHU 7UlJHU 11 1R Bulletin 1R 'HXWVFK 'HXWVFKHU \$XVVFKXù für Stahlbeton, Wilhe m Er ohn, Berlin, 1966, 159 pp. Mö ch, E., 1909, 909, Con Mörsch, Concrete-Steel Construction (Der E b , Transla l Eisenbetonbau), Translation of the third German Edition by ich, McGraw-Hill Book Co., New York, 368 pp. ter, W., 18 Ritter, 1899, "The Hennebique Design Method (Die Bau Bauweise Hennebique)," Schweizerische Bauzeitung (Züich)91R)HESS Reineck, K.-H., ed., 2002, Examples for the Design of Structural Concrete with Strut-and-Tie Models, SP-208, American Concrete With Strut-and-Tie Models, SP-208, American Concrete Institute, Farmington Hills, MI, 2002, 242 pp. 5HLQHFN .+ DQG 1RYDN / & HGV and the Concrete With Strut-and-Tie Models, SP-208, American Concrete With Strut-American Concrete With Strut-American Concrete With Strut-American Concrete With Strut-American Concrete With Strut-American Concrete With Strut-American Further Examples for the Design of Structural Concrete with Strutand-Tie Models, SP-273, American Concrete Institute, Farmington Hills, MI, 250 pp. 6FKODLFK - 6FKIIHU . DQG -HQQHZHLQ 0 "Towards a Consistent Design of Structural Concrete," PCI Journal91R0D\-XQHSS 6FKODLFK - DQG 6FKIIHU . 3.RQVWUXLHUHQ LP Stahlbetonbau (Detailing of Reinforced Concrete)," Betonkalender 90(UQVW 6RKQ9HUODJ%HUOLQ7HLO,,SS American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 165 14.8—Examples Strut-and-tie Example 1: Strut-and tie model of a deep beam without shear reinforcement Two transverse beams frame into a simply supported deep beam as shown in Fig. E1.1. Given: D = 151 kip L = 150 kip b = 24 in. h = 90 in. fcg = 4000 psi fy = 60,000 Calculate self-weight of beam: m: U = 1.2D + 1.6L L Fig. E1.1—Deep beam geometry. Calculation wc = (0.15 kip/ft3)(24 in.)(90 in.)(123 ft/in.) = 56.6 kip 1.2 1 kip + 29 kip) + 1.6(150 kip) U = 1.2(151 Pu = U = 456 kip 1.2 1 kip + 29 kip) + 1.6(150 kip) U = 1.2(151 Pu = 1.2 1 kip 1.2 1 kip 1.2 1 kip) + 1.6(150 kip) U = 1.2(150 kip) + 1.2(150 kip) + 1.2(150 kip) + 1.2(150 kip) + 1.2(150 kip) + 1.2(150 kip) + 1.2(150 kip) + 1.2(150 kip) + 1.2( between the applied load and the closest support, A tie is required at the bottom to maintain equilibrium. Fig. E1.2—Deep beam strut-and-tie model without shear reinforcement. American Concrete Institute – Copyrighted © Material - Copyrighted Concrete Institute – www.concrete.org Strut-and-Tie Determine the required reinforcement to support the applied loading. 166 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Effective stresses
Determine the effective concrete compressive strength in struts: 23.4.3 [fce us fcg Uncracked strut (prismatic) (S1): Table 23.4.3(a) us f[fce = (0.75) (0.85)(1.0)(4000 psi) = 2550 psi Table 23.4.3(b) Table 23.4.3(c) Determine the effective concrete compressive strength in nodes: 23.9.2 Bottle-shaped strut without reinforcement: us l]fce = (0.75)(0.85)(0.75)(4000 psi) = 1530 psi []fce un fcg Bound by struts (4000 psi) = 2040 psi Anchoring two or more ties—CTT:  $\exists fce = (0.75)(0.85)(0.6)(4000 \text{ psi}) = 1530 \text{ psi}$  in  $\exists \exists u = 45 456,000 \text{ lb} = 1100 \text{ ppsi}$  4 in 2 432 At support, Node N1.—CCT: A strut and tie frame into Node N1, and the third face is bound by the bearing plate. Therefore, the node is a (CCT). 6WHS  $\exists fce = 2040 \text{ psi}$ (see Step 3) Check if applied stress is smaller than the effective stress: []fce = 2040 psi > fu SVL2. At applied load, Node N2, and the third side is bound by the applied load bearing plate. Therefore, the node is a (CCC). 6WHS[]fce = 2550 psi (see Step 3) Check if applied stress is smaller than the effective stress: The active of symmetry, only half the beam will be considered (Fig. E1.3). The horizontal position of Nodes N1 and N2 are located at plate centerlines. The nodes' vertical position of Nodes N1 and N2 are located at plate centerlines. must be estimated or determined. They should be as close as possible to the top and bottom of the beam. Fig. E1.3—Nodes, struts, and ties geometry Strut-and-Tie The height of the top strut and bottom tie are estimated to be 11 in. and 15 in., respectively. Usually, each estimate is based on "back-of-theenvelope" calculation. The adequacy of these estimates is checked below. Assume tie (T1) is positioned at wn/2 = 7.5 in. from the bottom of the beam, and strut (S1) is positioned at ws/2 = 5.5 in. from the free body diagram. Taking moment equilibrium equation about Node N1: Pu((a) – S1(q) = 0 PM (a) = 0 PM (a) = 0 PM (a) = = h - wn - ws 2 2 5 in 5 in. = 77 in. g = 90 iin. - 7.5 in. - 5.5 (90 in.) - S1(77 in.) = 0 (456 kip)(90 S1 = 533 kip Calculate the angle and force of the diagonal strut. S2: s DUFWDOLOLO f IS2 NLSVLOf NLS Step 6: Strut-and-tie model geometry Strut S1 centerline is located 5.5 in. from top of beam. Tie T1 centerline is located 7.5 in. from bottom of beam. 7KLV¿[HVWKH]HRPHWU\RIWKHWUXVV]L](Fig. E1.4—Final determined strut-and-tie geometry. American Concrete Institute – Copyrighted © Material – www.concrete.org 168 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Tie reinforcement 23.7.2 Calculate minimum tie reinforcement: 23.3.1 T Ast = 1 φf y Choose reinforcement arrangement: Ast = • • • 533,000 lb = 11.8 in.2 0.75(60,000 psi) Three layers of four No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., and 10.0 in. from bottom Two layers of six No. 8 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., and 10.0 in. from bottom Two layers of six No. 8 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in.2 at 3.0 in. and 10.0 in. from bottom Two layers of six No. 9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in., 7.5 in., and 12.0 in.2 at 3.0 in. 7.5 in., and 12.0 in.2 at 3.0 in. 7.5 in.2 at 3.0 in. 7.5 in.2 at 3.0 in in.2 at 2.5 in., 6.375 in., and 10.25 in. from bottom For better steel distribution and for ease of anchorage length requirements, choose three layers of four No. 9 bars; Ast = 12.0 in.2 Step 8-Node N2LVFRQ2QHGE\WZRVWUXWVS1 and S2) and bearing plate of the applied load; CCC: From Step 3: 3ULVPDWLFVWUXW\[]fce = 2550 psi &&1RGH ffce = 22550 psi bo control. ntrol. Node and strutt both Fig. E1.5—Node Node N2 geometry. 23.4.1(a) 6WUXWVWUHQ WG frs fceAcs fF Fns = (2550 psi)(24 in.)(11 in.) = 673,200 lb > S1 OE2. Struts S2 is expected to be a bottle-shaped strut. R23.2.6b(i) Calculate the width at top of strut from: ws = EbVLQswscoss Strut width top: ws LQ VLQfLQ FRVf LQ 3URYLGHVXI¿FLHQWUHLQIRUFHPHQWWRFRQWUROFUDFNV and resist the bursting force in the strut. The strut effective stress is the controlling stress when compared to Node N1 effective stress. From Step 3:  $\Re WWOHVKDSHGVWUXWV[fce = 1913 psi)(20 in.)(24 in.) = 918,200 lb Check if design strength is greater than the required strength. [Fns = 918,200 lb > S2 OE American Concrete Institute - Copyrighted © Material$ www.concrete.org 2. CHAPTER 14—STRUT-AND-TIE MODEL 169 Step 9: Node N1 has one strut (S2) and one tie (T1) framing into it. The third side is bound by the support plate. Therefore, Node N1 is CCT. From Step 3: %RWWOHVKDSHGVWUXW[]fce = 1913 psi &&71RGH[]fce = 2040 psi Strut effective stress controls. R23.2.6b(ii) Calculate the width at top of strut from: wst = bVLQswscoss Strut width bottom: wst LQ VLQfLQ FRVf LQ 23.4.1(a) 6WUXWVWUHQJWK[Fns [fceAcs [Fns = (1913 psi)(24 in.)(23 in.) = 1,055,976 lb > S2 OE2. ment is and The tension reinforcement and rein he nod the back face of the node, thus imposing a rce oon that face. Although not compression force he height of th ode can required by thee C Code, the the node ase on the limiting com ressive be checked based compressive stress of the no node. i)(15 in.) in.)(24
in.) = 734,400 lb [Fnnn = ((2040 psi)(15 n strength gth is greater th Check if design than the h required strength. 400 lb > T1 OE2. []Fnn = 734,400 American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie Fig. E1.6—Node N1 geometry. 170 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Reinforcement should be developed in the node. The factored tie force must be developed at the point where the centroid of the reinforcement in a tie leaves the extended nodal zone and enters the span (Fig. E1.6). 2QO/VL[OR0]LWXGLQDOEDUVUHTXLUHDQFKRUD]HDW Node N1. Therefore, the bottom four No. 9 bars and two longitudinal bars from the second layer will be headed deformed bars for anchorage. Conservatively, check anchorage = 18 in. + 7 in. +  $bw \cdot db) + 4(db) + 2(db, stirrup) + 2(2db, cover) 25.4.4.2$  Provide Abg • LQ2) = 4.0 in.2 creq = 2(1.125 in.) = 2.25 in. < cprov LQ2. s = 4(1.125 in.) + 2(0.625 in.) in + 4(1.125 in.) + 2(2(1.125 = 23.75 in.)) in + 2( A dt = 0.016(1.0)(60,000 psi) 4000 psi (1.125 in.) = 17.1 in. Edh, anchoragee = 26 in. > Edt LQ2. Headed bbars to offset by 4.5 in. in vertical plane. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 171 Step 11: Minimum reinforcement for crack control Area of vertical web reinforcement provided shall For vertical reinforcement, use No. 5 at 9 in. on center not be less than: on each face over entire length. 9.9.3.1(a) Area of horizontal web reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDOOQRWEHOHVVWKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforcement parallel to WKHAH[XUDOUHLQIRUFHPHQWVKDQ For horizontal veb reinforceme reinforcement, use full-length No. 5 bars at 9 in. on center on each face over entire height. Ash•bws2 Ash (2)(0.31) =  $0.00287 \text{ sois} \neq 0.0028$ 0.03 9.9.2.1 OK Control cracking under nder ser service loads and guard against diagonal com compression failure. Vu  $\leq \varphi 10$  f c'bw d 4000 psi (2 (0.85)(10) 5)(1) ((24 in.)(83.5 in.) = 1,077,300 lb Vu = 456 kip < Vu NLS2. Step 12: Reinforcement detailing ng Deep beam reinforcement detailing ng Deep beam reinforcement detailing ng Deep beam reinforcement detailing ng Deep beam reinforcement detailing ng Deep beam reinforcement detailing ng Deep beam
reinforcement detailing ng Deep beam reinforcement detailing ng Deep bea and-tie t-and-tie mod hear reinfo Fig. E1.7—Deep beam reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-tie Example 2: Strut-and-tie model of a deep beam with shear reinforcement Two transverse beams frame into a simply supported deep beam, as shown in Fig. E2.1. Determine the required reinforcement to support the applied loading. Given: D = 156 kip L = 150 kip L = 150 kip L = 20 in. h = 90 in.  $f_{co} = 4000$  psi fy = 60,000 psi E = 22 ft 6 in. Support length: 18 in. at reaction RDGVDUHDSSOLHGRYHUDEHDULQ]OHO]WKRILQ /RDGVDUHORFDWHGDWWKLUGVSDQDQGDUHDSSOLHGRYHUD Fig. E2.1-Deep beam geometry. bearing length of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of beam U = 1.2D + 1.6L Calculation wc = (0.15 kip/ft3)(20 in.)(302 in.)(123 ft/in.) = 1.2D + 1.6L Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of beam U = 1.2D + 1.6L Calculation wc = (0.15 kip/ft3)(20 in.)(302 in.)(123 ft/in.) = 1.2D + 1.6L Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 Calculate self-weight of 18 in. ACI 318-14 Discussion Step 1: R47.2 kip U = 1.2(156 ki kip + 24 kip) + 1.6(150 kip) Pu = U = 456 kip )URPVWDWLFVIH Hu = 0 Ru = 45 Step 2: Strut-and-tie truss model del 23.2.1 6NHWFKWKHLQLWLDOVWUXWDQGWLHPRGHOYLVXDOL]LQ]WKHIRUFHÀRZLQWKHEHDPUHIHUWR)LJ(7KHGDVKHG LWL XWDQGWLHPRGHO LVXD FHÀ HEHDPU 23.2.2 lines represent the compression elements, and the dimensionless full mpression elements, and the dimensionless full mpression element represent nodes—intersection struts and ties. The distance between the applied loads epr t the nodes—intersection struts and ties. reinforcement orcement in the beam. A tie is required at the bottom to maintain equilibrium. Fig. E2.2—Deep beam strut-and-tie model with shear reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 173 Step 3: Sectional analysis Flexure: This step is not required for strut-and-tie model with shear reinforcement. fy () M n As fy | d - 2 × 0.85 f c'b || Mu = 456 kip × (45 in. + 37 in. +16 in./2) = 41,040 in.-kip ZLWK[DQGQRWDVXVHGIRU DVXVH 21.. 41.040 × 106LQ LQOE" SVL As (6 0,0 psi)As (60,000 () × 83.5 in. - 2(0.85)(40 (4000 psi)(20 in.) ) 2 As • LQ • Use 12 No. 9 bars ars in two layers (six No. 9/layer) Step 4: Effective stresses Determine the effective stresses D strut (prismatic) (S1):  $\ddot{u}s = 1.0$  ( $\Box$ fce = (0.75)(0.85)(1.0)(4000 psi) = 2550 psi Bottle-shaped strut with reinforcement (S2):  $\ddot{u}s = 0.75$  ( $\Box$ fce =
(0.75)(0.85)(0.85)(0.8 American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie Moment arm: d = h - cover - stirrup diameter - bar diameter - bar diameter - bar diameter - 1/2 distance between two layers. )URPAH[XUDODQDO] VLV 174 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Stress at support and applied load Calculate bearing plate area: Compressive stress at support and loading: fu = Pu AA AA = 18 in. × 20 in. = 360 in.2 fu = 456,000 lb = 1267 psi 360 in.2 At support, Node N1. The third face is bound by the bearing plate (compression). Therefore, the node is a (CCT). 6WHS[]fce = 2040 psi (Step 4) Check if applied stress is smaller than the effective stress: [fce = 2040 psi > [fu SVL2. At support, Node N4-CCC: Three struts frame into Node N4. The forth side is bound by the applied load bearing plate (compression). Therefore, the node is a (CCC). 6WHS[fce = 2550 psi (Step 4)) tress is smaller than the Check if applied stress effective stress: [fce SVL!][f SVL [f ] [ft] SVL 2.American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 175 Step 6: Required strength Because of symmetry, only half the beam will be considered. The horizontal position of Nodes N1 and N4 are located at plate centerlines. The nodes' vertical position must be estimated or determined. They should be as close as possible to the top and bottom tie are estimate is based on "back-of-theenvelope" calculation. The adequacy of these estimates is checked below. Assume tie (T1) and strut (S2) are positioned at wn/2 = 6.5 in. and ws/2 = 5.5 in. from bottom and top of beam, respectively. /2(top compression moment arm = (beam depth) - 1/2(13 in.) = 78 in.  $\theta = \arctan \operatorname{ctan} \operatorname{FHV} WLWLRXVWUXVVPH$  EHUV & DOFXODWHIRUFHVLQ¿FWLWLRXVWUXVVPHPEHUV 78 =  $60^{\circ} 45 45$ kip 456 (compression) = 527 kip sin si 60° NLS FRVf RVf NLSWHQVLRQ T2 T3 NLS VLQf VLQf NLSWHQVLRQ FRV f NLSFRPSUHVVLRQ S1 NLS FRVf NLSFRPSUHVVLRQ T1 NLS FRVf NLSFRPSUHVVLRQ S1 NLS FRV Material – www.concrete.org 176 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Tie reinforcement, T1: 23.7.2 T Ast = 1 23.3.1  $\varphi$ f y Choose reinforcement, T2: 23.7.2 Z 3.3.1 Ast = T2  $\varphi$ f y 527,000 lb = 11.7 in.2 0.75 × 60,000 psi Two layers of six No.9 bars; Ast = 2(6)(1.0) = 12.0 in.2 at 3.0 in. and 10 in. from bottom Two layers of six No. 8 bars and one layer of six No. 8 bars; Ast = 12.12 in.2 at 2.5 in., 6.37 $0.75 \times 60,000$  psi Six No. 9 bars are required; Ast = (6)(1.0) = 6.0 in.2. Extend bottom layer reinforcement of T1. Tie T3 represents the shear required reinforcement in rt and the applied load. the region between the support 23.7.2 21.2.1(g) Fnt = Ats fy As, req = Fu  $\varphi$ f y As, req = Fu 
$\varphi$ f y As, req = Fu  $\varphi$ f y As, req Ti pB m Design," in C crete Model for Deep Beam Concrete ay 2003, pp. pp 63-70, 63.70 provides p des a International, May discussion on stirrup distribution. 56,000 lb 456,000 = 10.1 in 2  $0.75 \times 60,000$  ps psi U 13 No. 4 stirrups With four legs for a total area of: Ats, prov (2)(0.2 in 2) = 10.4 in 2 > Ats, prov (d = 10.1 in 2 rov = (13)(2)(2) (0.2 trate the "Some engineers might prefer to concentrate stirrup reinforcement at approximately the location of tie" T3. Node N2 is determined as half the distance between Nodes N1 and N4 (1/2 × 90 in. = 45 in.). This results in spacing of: s < However, "it is reasonable to assume that struts" S3 and S4 "will fan out and engage several stirrups." In this example, the second approach will be followed and the reinforcement will be distributed between the support and the applied force. Assume 4 in. stirrup spacing: For stirrup distributed between the support and the applied force. Step 9: Node N1 This node has one strut, a bearing force, and a tie. Therefore, Node N1 is CCT (Fig. E2.3). The node has a horizontal dimension equal to the support length (18 in.), and a height of 6 in. = 2(3 in. cover) From Step 4: &&71RGH Determine width of strut S3 at N1: )URPJHRPHWU\2QGWKHZLGWKRIWKHVWUXWws: R23.2.6b(i) ws = EbVLQswtcoss, f LQ VLQfLQ FRVf LQ ws LQ VLQ 23.4.1(a) 21.2.1(g) Fns = ((1913 Check that thee ddesign strength is larger than the ng required strength: b > 11,636 lb []Fnss = ((1913 Check that thee ddesign strength is larger than the ng required strength: b > 11,636 lb []Fnss = ((1913 Check that thee ddesign strength is larger than the ng required s S3 OE2. []Fnss = 7711,636 lb Step 10: Node N2 This node has two wo struts uts and one tie framing f ming into it—CCT (refer to F Fig. E2.4)) From Step 4: 1RGH]fce = 2040 psi (bottle-shaped strut with reinforcement) Fig. E2.4). From Step 4: 1RGH]fce = 2040 psi (bottle-shaped strut with reinforcement) Fig. E2.4) From Step 4: 1RGH]fce = 2040 psi (bottle-shaped strut with reinforcement) Fig. E2.4). an effective stress of 2550 psi; however, the controlling stress is that of Node N2 [fce = 2040 psi. Therefore, the height of the vertical face of Node N2 (45 in.) was determined in the vertical fa

Step 8. American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie Strut effective stress controls. 178 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Node N4 This node is an all-compression node—three struts and one applied compression force framing into it and no ties—CCC (refer to Fig E2.5(a)) From Step 4, effective node and strut stresses are: && 1913 psi (with reinforcement) The node has a horizontal length equal to the bearing plate and height equal to 11 in. assumed above. The stress at bearing plate was checked in Step 5. (a) Forces framing S1 is divided into two equal forces as shown in )LJ(E 2QHIRUFHUHVLVWVS2 and the second balances the horizontal component of S4. (b) Forces (b Fi E2.5—Node E2 de N4 geo geometry. Fig. 23.4.1(a) 6WUXWVWUHQJWK]Fns [fce Acs 50 psi)(20 in.)(11 in.) = 561,000 lb [Fns = (2550 Check that the design strength is larger than the required strength: []Fns = 561,000 lb > S2 OE2. Required vertical face height for Strut S1 at Node N4 is based on the same compressive stress exerted by Strut S2. Both S1 and S2 are prismatic struts. Assume node depth facing S1 is 5.5 in. (1/2 []Fns = (2550 psi)(20 in.)(5.5 in.) = 280,500 lb S2 depth): Check that the design strength is larger than the required strength: []Fns = 280,500 lb > S2 OE2. Strut S4—bottle-shaped strut: Determine width of Strut S4 at Node N1: R23.2.6b(i) )URPJHRPHWU\¿QGWKHZLGWKRIWKHVWUXWws: ws = EbVLQswtcoss 23.4.1(a) []Fns []fce Acs § f ws LQ VLQfLQ FRVf LQ (23.4.1a) []Fns = (1913 psi)(20 in.)(21 in.) = 803,000 lb Check that the design strength is larger than the required strength: ||Fns = 803,000 lb > S3 OE2. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 179 An alternative approach for analyzing Node N4: R23.2.2 There are four forces acting on a node and, per ACI 318-14, Section R23.2.2, it is generally necessary to resolve some of the forces to end up with three intersecting forces (refer to Fig. E2.6). Strut-and-Tie (a) Resultant force of S1 and S4 (b) Node N4. le Available length at node: § f § ws LQ VLQfLQ FRVf LQ Check adequacy of nodal zone: R23.2.6b(i) ws = EbVLQswtcoss 23.4.1(a) 23.3.1 [Fns ]fce Acs [Fn•Fu (23.4.1a) [Fns = (1913 psi)(20 in.) = 765,000 lb > S5 OE2. American Concrete Institute - Copyrighted © Material - www.concrete.org 180 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 12: Node N3 This node has three ties and one strut framing into it—CTT (refer to Fig. E2.7) From Step 4: 1RGH[]fce = 1530 psi (controls) 6WUXW[]fce = 1913 psi (bottle-shaped strut with reinforcement) There is load transfer between Ties T1 and T2. The dimensions of Node N3 are determined from the vertical and horizontal distribution of reinforcement. Because Tie T3 (stirrup reinforcement) frames into Node N3, assume that the Strut S4 fans out similar to Strut S3. Strut S4 needs only to be checked at its Node N3 due to the transfer of forces T1 and T2 is: 23.7.2 ws, req = T1 - T2 \u03c6 f cu bw Strut S3 is fan-out out shaped s and engages several stirrups. ws, rreq eq = 527,000 lb - 264,000 lb = 8.6 in. 0.75 75 × 2040 psi × 20 in. ws, prov rov = 13. in. > ws, req LQ2. Step 13: Reinforcement detailing in 23.8.2 Anchor longitudinal ud reinforcement bby hooks, ec cal anchorages, or straight headed bars, mechanical bar development. In this example, hooked bars are used, although n (refer the development length can be reduced by 0.7. Edh = 23.7 × 0.7 = 17 in. Available anchorage at Node N1: Edh, anchorage = length of extended nodal zone - cover Edh, anchorage = Node N3: T2: six No.9 T1: 12 No. 9 As, prov = 6.00 in.2 As, prov = 12.00 in.2 For longitudinal reinforcement distribution, refer to Step 7. Place 12 No. 9 bars vertically in two layers over a height of 13 in. 181 Assume straight deformed bar development: 23.8.3(a) Ad = 3(60,000 psi) ((1.0)(1.0)(1.0)) | 1/(1.125 in.) = 32 in. 2.5 40(1.0) 4000 psi (Nodal)zone width = 45 in. Edh,anchorage = (18 in./2 - 3 in.) + 45 in. + 45 in./2 = 73.5 in. Edh,anchorage > Edh LQ2. Place No.9 bars over the full span of the beam. American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie 25.4.2.2 ψt ψeψ s cb + KS in./2 = 73.5 in. tr Ad = db 40 $\lambda$  f c' 3 fy 182 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Check that reinforcement ratio is larger than the minimum required of 0.0025; 9.9.3.1(b) Avh = 0.0025 with bw = 20 in. s2 bw Avh = (0.0025)(20 in.) = 0.05 in.2 /in. s2 bw Assume No. 4 reinforcement at each face over full length of beam: 0.4 in.2 = 8 in. 0.05 in.2 /in. Avh = (0.2 in.2)(2 legs) = 0.4 in.2 s = 9.9.4.3 Maximum allowable spacing of horizontal reinforcement: d/5 = 83.5 in./5 = 16.7 in. LQFRQWUROV !LQ2. 23.5.3 Check horizontal reinforcement: d/5 = 83.5 in./5 = 16.7 in. LQFRQWUROV !LQ2. 23.5.3 Check horizontal reinforcement: d/5 = 83.5 in./5 = 16.7 in. LQFRQWUROV !LQ2. 23.5.3 Check horizontal reinforcement: d/5 = 83.5 in./5 = 16.7 in. LQFRQWUROV !LQ2. 23.5.3 Check horizontal reinforcement: d/5 = 83.5 in./5 = 16.7 in. LQFRQWUROV !LQ2. 23.5.3 Check horizontal reinforcement: d/5 = 83.5 in./5 = 16.7 in. LQFRQWUROV !LQ2. 23.5.3 Check horizontal reinforcement ratio based on strut-and-tie model. No. 4 at each face at 8 in. on center vertically satisfying Eq. (23.5.3). (0.2 \text{ in.}) (2) sin 60° = 0.0022 (20 \text{ in.}) (8 \text{ in.}) Ui = 60 degrees (angle between Strut S3 and horizontal reinforcement) (Fig. E2.3). Therefore, use No. 4 at 8 in. at each face distributed vertical reinforcement, use two legs) at 6 in. on center. 23.5.3 fD U2 f±f fDQJOHEHWZHHQ6WUXWS 3 and cem vertical reinforcement). 9.9.4.3 ow spacing of verti al Maximum allowable vertical t: reinforcement: d/= 83 dd/5 83.5 in./5 = 16 16.7 7 in in. QF !LQ LQFRQWUROV !LQ2. 9.9.3.1(a) rce t ratio is larger than an the Check if reinforcement minimum requirement of 0.0025 sin 30° OK Use two No. 4 stirrups at 6 in. 0.0033 (20 in.)(6) (6 in.) 0.0033 = 0.0 .00 0066 > 0.0025 sin 30° OK Use two No. 4 stirrups at 6 in.on center. U 23.5.3 Check if sum of horizontal and vertical reinforcement exceeds the minimum required 0.003.  $\Sigma$  9.9.3.1(b) Ast sin  $\gamma$  i  $\geq$  0.003 bsi d.hdv !2. Provide minimum shear reinforcement between the two applied forces (mid-third of beam span), No. 4 at 8 in. on center: Avh = 0.0025 s2 bw (20 in.)(8 in.) with bw = 20 in. and s2 = 8
in. 9.9.2.1 Therefore, provide No. 4 stirrups at 8 in. on center. To control cracking under service loads and guard against diagonal compression failure, the upper []limit for shear force in deep beams should not exceed:  $\varphi$ Vu = (0.75)10 4000 psi(20 in.)(83.5 in.) = 792 kip Vu =  $\varphi$ 10 f c'bw d Check that the design shear is greater than the []Vn = 792 kip > Vu NLS2. required shear force. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 183 9.9.4.3 Reinforcement required for crack control is calculated at No. 4 at 8 in. on center. Stirrups (4 legs) spaced at 4 in. on center are provided in the shear span (space between applied force and support [Fig. E2.8]). d/5, and 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ2. In the area between the two applied loads use two-legged No. 4 stirrups at 12 in. d LQ!LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ!LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ2. In the area between two-legged No. 4 stirrups at 12 in. d LQ2. In the area between two-legged No. 4 stirr around one or the other of longitudinal bars. Step 14: Arrangement of reinforcement Fig. E2.8—Deep beam reinforcement based on strut-and tie design. Step 15: Conclusions 1. Comparing Examples 1 and 2, either of the two models presented is acceptable, provided that equilibrium and yield conditions DUHVDWLV¿HG 2. The predominant mechanism of shear transfer in a deep beam region is attributed to the arching action of a direct strut. 3. Using a direct strut. 3. Using a direct strut between support and applied load to represent a deep beam region (a/d < 2) is well founded based on experimental observations and past research. \$PLQLPXPDPRXQWRIWUDQVYHUVHUHLQIRUFHPHQWLVUHTXLUHGWRSURYLGHDGHHSEHDPZLWKVXIcFLHQWGHIRUPDWLRQFDSDFLW\ 5. A two-panel strut-and-tie method is an ineffective representation of the shear transfer mechanism. 6. The capacity of a two-panel model is controlled by the capacity of the vertical tie or stirrups, thereby ignoring the contribution to shear strength provided by the concrete. American Concrete Institute – Copyrighted © Material – www.concrete.org Strut-and-Tie Stirrup distribution: Total area of required stirrup reinforcement is calculated above 10.1 in.2 or two No. 4 stirrups (four legs). 184 THE REINFORCED CONCRETE 17(14) Strut-and-tie Example 3: Design of one-sided corbel using strut-and-tie method Design a one-sided corbel supporting a crane girder beam reaction force of 9.0 kip is acting away from the column. A horizontal tensile force of 9.0 kip is acting away from the column. Determine the required cross section of corbel and reinforcement. Given: D L HD b1 b2 fcg fy = = = = = 22.0 kip 28.0 kip 9.0 kip 16 in. 16 in. 5000 psi 60,000 psi 6 kip) 1.2 0 kip) + 1.6(28.0 1. Pu = U = 68.8 8 kip 1.6 kip)) = 14.4 14 4 kip Hu = 1.6(9.0 Step 2: Effective stresses Determine the eff effective concrete com compressive essive strength in struts: 23.4.3 []fce  $\hat{I}_3$  fcg Table 23.4.3(a) Table 23.4.3(b) 23.9.2 Uncracked cked strut ((prismatic):  $\hat{u}s \hat{I}_1$  []fce = (0.75)(0.85)(1.0)(5000 psi) = 3188 psi Bottle-shaped strut with reinforcement (S1, S2, S3): us 1[fce = (0.75)(0.85)(0.75)(0.85)(0.75)(0.85)(0.75)(0.85)(0.75)(0.8 (0.75)(0.85)(1.0)(5000 psi) = 3188 psi Table 23.9.2(c) Anchoring one tie—CCT (N1 and N3): un 1[fce = (0.75)(0.85)(0.8)(0.0)(5000 \text{ psi}) = 1913 \text{ psi American Concrete Institute – Copyrighted } Waterial – www.concrete.org CHAPTER 14—STRUT-AND TIE MODEL 185 Step 3: Bearing Calculate bearing plate area: Compressive stress at loading: 23.9.2 fu = Pu AA Ac = 12 in. × 16 in. = 192 in.2 Full corbel width (16 in.) is taken. 68,800 lb = 359 psi 192 in.2 [fce = 2550 psi > fu SVL2. Bearing plate area is adequate. fu = Check that applied stress is smaller than effective stress (CCT Node—Step 2): Step 4: Corbel dimensions 23.2.9 Choose a span-to-depth ratio of maximum 2. 16.5.2.2 Use a depth of 10 in. Strut-and-tie model Consider erection tolerances and load eccentricities by shifting Pu one inch toward the outer edge of the corbel from center of bearing plate. Use an overall corbel depth of 20 in. New position of applied load: 2 in. + 6 in./2 + 1 in. = 6.0 in. ie T1 is at 1.75 in. from top of Assume center of Tie ring one layer of reinforcement corbel—considering ver. and 1 in. of cover. 23.2.9 i Moment arm: d = 20 - 1.75 = 18.25 in. on of Node N1 from fr m plate The horizontall lo location ain from rom geometry centerline is obtained geometry. It iss the intersection of the force resultant line and T1.
x = 14.4 kip/68.8 kip × 1.75 in. = 0.37 in. Fig E3.2—Strut-and-tie model of one-sided corbel loading. American Concrete Institute – Copyrighted © Material – www.concrete.org 186 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The horizontal Tie T3 is assumed to be located on the horizontal line passing through the sloping end of the corbel.  $\Sigma$  M N4 = 0 The position of Strut S3 is found by calculating the strut width ws. Taking the moment about Node 68.8 kip(0.37 in. + 6 in. + 14 in.) + 14.4 kip(18.25 in.) N4 and assuming loads are applied at Node N1. = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3—Step 2: 1664.3 in.-kip = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3—Step 2: 1664.3 in.-kip = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3—Step 2: 1664.3 in.-kip = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3—Step 2: 1664.3 in.-kip = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3—Step 2: 1664.3 in.-kip = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3—Step 2: 1664.3 in.-kip = S3(14 in. - 1/2ws) The width of Strut S3 is controlled by the smaller effective stress of Strut S3 is controlled 38,240 ws Solving for ws by equating Eq. (1) and Eq. (2): 7KLVi[HVWKHJHRPHWU\RIWKHWUXVVPRGHO ws = 3.6 in. (2) Step 6: Truss forces 23.7.2 From statics, the forces in the he truss members can 23.3.1 be calculated. Node N1: \$QJOHRILQFOLQDWLRQ\$, QDWL 1: ut S1 and d Tie T1. Forces in Strut  $\theta$ 1 = arctan arc 18. in. 18.25 = 65.9° 1 i + 3.6 in. 0.377 in. + 6 in. 2 S1 NLSVLQf Qf NLS NL NLS T1 NLSNLS FRVf Assume 2 in. cover. Node N2: QOHRILQFOLQDWLRQs2: 18.25 in. = 56.2° 1 16 in. - 2 in. - 3.6 in. 2 S2 = T1FRVf NLS FRVf T3:  $\theta 2$  = arctan Node N3: T3 = S2FRVf T3:  $\theta 2$  = arctan Node N3:  $\theta 2$  = arctan Node N3:  $\theta 2$  = arctan N3:  $\theta 2$  = arctan N3:  $\theta 2$  = NLS FRVf ±NLS FRVf kip S3 = S1VLQfS2VLQf S3 NLS VLQfNLS VLQf S3 = 136.4 kip Step 7: Force summary American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 187 Step 8: Tie reinforcement The area of reinforce T1 23.7.2 Tie strength: Fnt = Ats fy 45,200 lb = 1.0 in.2 0.75(60,000 psi) Ats = The provided steel area must be at least per ACI 318 prescriptive requirements: 0.04 f c' bd fy 0.04 T2 has larger tension force than T1. However, tension force T2-T1 should be resisted by column longitudinal reinforcement. Therefore, continue IRXU1RUHLQIRUFHPHQWMXVWWRKDYHVXI¿FLHQW lap length. For Class B splice, 1.3Edh: 25.4.2.3 A dh ) ( || f 3 y  $\psi t \psi e \psi s |= |d| 40$  f c' ( cb + K tr ) | b || || d /| ) | b 5000 psi (16 in.)(18.25 in.) = 0.98 in.2 60,000 psi (0.5 bars. Ast, prov = 4(0.31 in.2) = 1.24 in.2. Anchor Tie T1 by welding four No. 5 reinforcement to DVWHHODQJOHRI/[[LQ)LJ( ( cb + K tr ) || d || = 2.5 b zt ze zs = 1.0 ( 3 60,000 psi (1.1.0)(1.0)(1.0) | A dh = | 0.625 in. = 15.9 in. 2.5 / (440 5000 psi 1.3EEdh = (1.3)(15.9 1 5 9 iin.)) = 20.7 in., say 21 in. The area of reinforcement inf ment required fo for Tie T3 is 7.2): obtained from Eq. (23.7.2): 23.3.1 []Fnt •Fu = T3 23.7.2 Tie strength: Fnt = Ats fy end the four ur No Extend No. 5 minimum 21 in. below Node N2. Ats = 14,400 lb = 0.32 in.2 0.75(60,000 psi) Use two No. 3 additional column ties at end of corbel slope; Ast, prov = 2(2)(0.11 in.2) = 0.44 in.2. These ties are spaced at 2 in. on center. American Concrete Institute – Copyrighted © Material – www.concrete.org Strut-and-Tie 16.5.5.1(c) 188 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 9: Node N3 at Struts S1 and S2; refer to Step 2: &&7QRGH[]fce = 2550 psi %RWWOHVKDSHGVWUXW[]fce = 2390 psi (with reinforcement) 23.4.1(a) 21.2.1(g) Strut effective stress controls. []Fns []fceAcs and Acs = bwws Available width ws of nodal zone where S1 frames is: ws , req = 75,400 lb = 2.0 in. (2390 psi)(16 in.) 1/2ws(S1) = 1/2ws(S3)VLQf VLQf LQ ws(S2), provided = 3.3 in. ws, provided =
3.3 in. ws, provided = 3.3 in. ws, strut width ws at face of Nod ws = 881,300 lb = 2.2 in. psi)(16 in.) (2390 psi)(1 n.) Use 2.2 2 2 in. for or node zon zone face width at S2. 23.4.1(a) epth to accommodate cu E .3). No. 3 ties calculated in Step 5 (Fig. E3.3). d by code, the hheight ght of the Although not req required ck based ased on the limiting lim g comnode can be checked pressive stress for the node: 23.9.1 1RGHVWUHQJWK[]Fnn []fce Anz 23.3.1(c) 21.2.1(g) R.23.2.6(b) []Fnn = 163,200 lb ]F (23.3.1(c) 21.2.1(g) R.23.2(b) []Fnn = 163,200 lb ]F (23.3.1(c) 21.2.1(g) R.23.2(b) []Fnn = 163,200 lb ]F (23.3.1(c) 21.2.1(g) R.23.2(b) R.23.2(b) ]F (23.3.1(c) 21.2.1(g) R.23.2(b) R. fce Anz [Fns = (2390 psi)(6.4 in.)(16 in.) = 244,700 lb [Fns•Fu = S1 (23.3.1(a)) [Fns = 244,700 lb > S1 OE2. Fig. E3.3—Dimensions of strut-and-tie members. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 189 An alternate approach for analyzing Node N3 is to combine Struts S1 and S2 as shown in Fig. (D 7KHQRGHFRQ¿JXUDWLRQLVDVVKRZQLQ Fig. E3.4(b). The resultant force S1 + S2 is applied at Node N3 and has an angle of 6 degrees with the vertical. The resultant force S1 + S2 is applied at Node N3 and has an angle of 6 degrees with the vertical. Node N3 (b Fig. E3 F E3.4—Alternate ernate res resolution of node forces. 1136,400 lb = 137 137,200 lb cos 6° 6° Accordingly, the strutt strength is: S1 + 2 = 23.4.1(a) GWUXWVWUHQJWK[Fns [fceAcs]Fns = (2390 psi psi)(6.0 in.)(16 in.) = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength: [Fns = 229,400 lb 23.3.1(a) Check that the design strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h required strength is larger than h the h require lb > S1+2 OE2. Step 10: Node N1 has one strut, one bearing, and a tension tie framing into it (CCT). The effective stresses for Node N1 is the same. However, due to geometry, bearing plate, and tension reinforcement, a larger width is provided. By inspection, Node N1 is the same. adequate. American Concrete Institute - Copyrighted © Material - www.concrete.org 190 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 2 &771RGH[fce = 1913 psi %RWWOHVKDSHGVWUXW]fce = 2390 psi (with reinforcement) Node effective stress controls. The available tie width is: 2(1.75 in.) = 3.5 in. (Step 5). 23.9.1 Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from the edge: ws = 2(2 in.) = 4 in. Width of node at T2: Assume vertical reinforcement is located 2 in. from 81,300 lb wt = 81,300 lb = 2.7 in. (1913 ppsi)(16 in.) The forces from the ttwo ties (T1 and T2) at the lt in a potential ntial failure of the bend may result al compression zone one concrete in thee ddiagonal nt reinforcement is too ssed in Klein, "Curved-Bar "Curved small, which is discussed cre International91R ternational 9 1R Nodes," Concrete 42 Sept. 2008, pp. 42-47. For a CTT Node, the minimum radius is: rb , min  $\geq$  25.3.1 2 Ats f y bf c' The minimum allowable radius for No. 5 bar is 6db. rb, min  $\geq$  2(4)(0.31 in.2)(60,000 psi) = 1.86 in. (16 in.)(5000 psi) = 1.86 in. (16 in.)(5000 psi) = 1.86 in.) = 3.75 in. > rb LQ2. American Concrete Institute – Copyrighted © Material –
Copyrighted © Material – Copyrighted © Materia www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL Step 12: Minimum reinforcement for crack control 16.5.6.6 Closed stirrups are required parallel to the reinforcement T1 to be uniformly distributed ZLWKLQRIWKHHIIHFWLYHGHSWKDGMDFHQWWRT1 as recommended by the descriptive requirements of ACI 318. 2/3(18.25) = 12.17 in 191 Use 12 in. The area of these ties or stirrups must exceed: Ah = 0.5(Ast - An)(14.4) Ah = 0.5(0.87 - An)(14.4) Ah = 0.26 in.2 with average spacing of 12/3 = 4.0 in. us was used as 0.75 when calculating the force in the diagonal Struts S1 and S2. Therefore, minimum reinforcement must be provided to satisfy:  $\Sigma$  Asi sin  $\gamma$  i  $\geq$  0.003 bsi  $\Sigma$  Asi 2(0.11) sin  $\gamma$  i = sin 60° = ment is provided horizontal reinforcement provided. American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie where No. 3 closed stirrups. tensile force T3. 192 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 13: Reinforcement detailing 1RWH:HOGUHLQIRUFHPHQWWR/[LQVWHHODQJOH 1RWH:HO UHLQIRUFHPHQWWR/ [LQVWHHODQJ Fig. E3.5—One-sided corbel reinforcement based on strut-and-tie Example 4: Design of double corbel 'HVLJQDGRXEOHFRUEHOSURMHFWLQJIURPLQWHULRUFROXPQSUHFDVWEHDPV7KHUHDFWLRQVRIWKHEHDPVDUHVKRZQEHORZDQGDUH located 6 in. from the face of the column. Horizontal tensile forces are acting away from the column, accounting for creep and shrinkage deformations. The column also supports a 300 kip load from the roof. Determine the required strength Column factored ed lload: 5.3.1 U = 1.6 in. fcg = 4000 psi fy = 60,000 psi Bearing plate size:  $6 \times 12 \text{ in}$ . ACI 318-14 Discussion iscussion Step 1—Required strength Column factored ed lload: 5.3.1 U = 1.2 D + 1.6 L.  $6 \times 12 \text{ in}$ . ACI 318-14 Discussion iscussion Step 1—Required strength Column factored ed lload: 5.3.1 U = 1.2 D + 1.6 L.  $6 \times 12 \text{ in}$ . ACI 318-14 Discussion i Corbel factored ed load: 6L U = 1.2D + 1.6L Horizontal factored load: Hu = 1.2Hd Step 2—Stress limits Determine the effective concrete compressive strength in struts: 23.4.3 []fce  $\hat{U}$ s fcg Table 23.4.3(b) 23.9.2 Fig. E4.1—Double-sided E4.1— corbel geometery. Calculation C U = 1.2(95.1 1.2 kip) ip) + 1.6(116.2 1.6(kip) Pu = U = 300 Corbel factored load: Hu = 1.2Hd Step 2—Stress limits Determine the effective concrete compressive strength in struts: 23.4.3 []fce  $\hat{U}$ s fcg Table 23.4.3(b) 23.9.2 Fig. E4.1—Double-sided E4.1— corbel geometery. Calculation C U = 1.2(95.1 1.2 kip) ip) + 1.6(116.2 1.6(kip) Pu = U = 300 Corbel factored load: Hu = 1.2Hd Step 2—Stress limits Determine the effective concrete compressive strength in struts: 23.4.3(b) 23.9.2 Fig. E4.1—Double-sided E4.1— corbel geometery. Calculation C U = 1.2(95.1 1.2 kip) ip) + 1.6(116.2 1.6(kip) Pu = U = 300 Corbel factored load: Hu = 1.2Hd Step 2—Stress limits Determine the effective concrete compressive strength in struts: 23.4.3(b) 23.9.2 Fig. E4.1—Double-sided E4.1— corbel geometery. Calculation C U = 1.2(95.1 1.2 kip) ip) + 1.6(116.2 1.6(kip) Pu = U = 300 Corbel factored load: Hu = 1.2Hd Step 2—Stress limits Determine the effective concrete compressive strength in struts: 23.4.3(b) 23.9.2 Fig. E4.1—Double-sided E4.1— corbel geometery. Calculation C U = 1.2(95.1 1.2 kip) ip) + 1.6(116.2 1.6(kip) Pu = U = 300 Corbel factored load: Hu = 1.2Hd kip p 1.2 1.6 U = 1.2(30.4 kip) + 1.6(23.6 kip) Nu = U = 74.33 kip kip = 16 kip Hu = 1.6(10 kip Uncracked strut (prismatic)(S3, S6, S7):  $us I_{\Box}fce = (0.75)(0.85)(1.0)(4000 \text{ psi}) = 2550 \text{ psi Bottle-shaped strut with reinforcement (S1 and S2)}$ :  $us I_{\Box}fce = (0.75)(0.85)(0.75)(4000 \text{ psi}) = 1913 \text{ psi Bottle-shaped strut with reinforcement (S4 and S5)}$ :  $us I_{\Box}fce = (0.75)(0.85)(0.75)(4000 \text{ psi}) = 1913 \text{ psi Bottle-shaped strut without reinforcement (S4 and S5)}$ :  $us I_{\Box}fce = (0.75)(0.85)(0.75)(4000 \text{ psi}) = 1913 \text{ psi Bottle-shaped strut without reinforcement (S4 and S5)}$ :  $us I_{\Box}fce = (0.75)(0.85)(0.75)(4000 \text{ psi}) = 1913 \text{ psi Bottle-shaped strut without reinforcement (S4 and S5)}$ . 2040 psi Table 23.9.2(c) Anchoring two or more ties—CTT: un 1[fce = (0.75)(0.85)(0.6)(4000 psi) = 1530 psi American Concrete Institute - Copyrighted @ Material - www.concrete.org 194 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Bearing at loading Calculate bearing area. Use full corbel width. Compressive stress at loading: 23.9.2 fu = Vu AA Ac = 16 in. × 6 in. = 96 in.2 fu = 74,300 lb = 774 psi 96 in.2 The nodel scress on Node N1: Step 4: Corbel dimensions 23.2.9 The corbel must satisfy the requirement a/d < 2. 16.5.2.2 []fce = 2040 psi > fu SVL2. Depth outside of the bearing area is at least 0.5d. For this problem, select corbel depth at column face = 20 in. and at free end of the corbel = 12 in. Fig. E4 F E4.2—Double-corbel uble-corb geometry. Step 5: Strut-and-tie model Consider erection tolerances and load eccentricities by shifting Nu one inch toward the outer q plate edge of the corbel from center of Dearing plate. New position of applied load: 3 in. + 6 in./2 + 1 in. = 7 in.
Assume center of Tie T1 is at 2 in. from top of corbel—considering two layers of reinforcement and 1 in. of cover. 23.2.9 Moment arm: d = 20 in. - 2 in. = 18 in. The horizontal location of Node N1 from plate centerline is obtained from geometry. It is the intersection of the force resultant line and T1. x = 16 kip/74.3 kip × 2 in. = 0.43 in. Fig E4.3—Double-corbel strut-and-tie model. The horizontal Strut S3 is assumed to be located on the horizontal line passing through the sloping end of the corbel. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 195 The column axial load Pu is resolved into two equal loads acting it strut width ws based on the controlling effective stress: From Step 2 controlling effective stress: From St && 1RGH from Eq. (23.4.1(a)) Strut width is calculated from Eq. (23.4.1(a)). Step 6: Truss forces 23.7.2 From statics, the forces in the truss members can 23.3.1 Due to symmetry, the force in S6 is equal to: S6 = Pu/2 + Vu S6 = S7 = 300 kip/2 + 74.3 kip = 224.3 kip [Fns ]fcebws•S6 ws = (23.4.1(a)). Strut width is calculated from Eq. (23.4.1(a)). Strut width is ca be calculated. 224,300 lb = 5.5 in. (2550 psi)(16 in.) 7KLV $\dot{c}$ [HVWKH]HRPHWU\RIWKHVWUXWDQGWLHPRGHO Node N1: \$Q]OHRILQFOLQDWLRQs DWLRQ ut S1 and Forces in Struts S4 and S5. From geometry force in S3. Step 7: Force summary  $\theta$  = arctan arct 8 0 in. 18.0 = 60.5° 1 0.43 3 in. + 7 iin. + 5.5 in. 2 S1 NLSVLQf Q NLS NLSNLS FRVf NLS NLS T1 a se of symmetry, ymmetr S2 = S1 = 85.4 kip Because 300 kip/2 = 150 kip S4 = S5 = Pu/2 = 30 S3 = S1FRVf NLS F 0.75(60,000 psi) The provided steel area must be at least per ACI 318 prescriptive requirements: 23.2.9 0.04 f c' bd fy 0.04 4000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi (16 in.)(18 in.) = 0.77 in.2 < Ats OK 60,000 psi ( www.concrete.org Strut-and-Tie Strut and node both control. 196 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 9: Node N3 will be addressed (Step 2). && 1913 psi (with reinforcement) %RWWOHVKDSHGVWUXW [] fce = 1530 psi (without reinforcement) The width ws = 5.5 in. of Strut S6 (S7) was calculated from: 23.4.1 23.3.1(a) [] Fns [] fceAcs with Acs = bwws DQG [] Fns •S3 (23.4.1(a)) ws = 42,100 lb = 1.03 in. (2550 lb = 1.03 lb
= 1.03 lb = 1.03 l psi)(16 in.) Say, 2 in. Node width, N3 (N4), facing Strut S4 (S5): The column is reinforced with four No.8 longitudinal bars with 1.5 in. cover. The column UHTXLUHPHQWV1R a) run parallel to thee axis oof the strut, ithin the strut, and b) are located within d in ties satisfying isfying ACI 318 (25.7) 25.7). c) are enclosed ru S4 (S S5) strength can bbe incre increased Therefore, Strut (2 (b)): to satisfy Eq. (23.4.1(b)): 23.4.1(b) R23.4.1 o bar cated within Strut S4 (S5). LQ 2)(60 ksi) = 94.8 kip E Asgfsgg LQ Fns = fceAcs + AsgffsgE with fsg fy = 60 ksi y Assume reinforcement is not provided to satisfy 23.5.1 in the column above the corbel section. The nominal concrete strength of Strut S4 (S5) is limited to: ws , req = []fce = 1530 psi (Step 2) 23.4.1(a) Node N3 (N4) width, 5.5 in., was calculated based on limit strength—Step 4. 150,000 lb - 0.75(94,800 lb) = 3.3 in. (1530 psi)(16 in.) ws, prov = 5.5 in. > ws, req LQ2. Node width N3 (N4) facing Strut S1 (S2): Strut S1 has ---- = 2.8 in. (1913 psi)(16 in.) American Concrete Institute – Copyrighted © Material www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 197 Step 10: Node N1 Node N1 (N2)—CCT: Because of symmetry, only Node N1 will be addressed. From Step 2: &&71RGH[]fce = 2040 psi %RWWOHVKDSHGVWUXW[]fce = 1913 psi (with reinforcement) 23.4.1(a) 23.3.1(c) 21.2.1(g) The tension reinforcement is anchored/welded to an edge steel angle, thus imposing a compression force on that face. Although not required by the Code, the height of the node can be checked based on the limiting compressive stress for the node. 23.9.1 23.9.2 (23.4.1(a)) []Fnn = (2040 psi)(16 in.)(4 in.) = 130,560 psi ]]Fnn = 130,560 psi ]]Fnn = 130,560 psi ]]Fnn = 130,560 psi ]]Fnn = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 in.)(4 in.) = (2040 psi)(16 i strength: dth of S1 (S2): Determine strut width LQ VLQfLQ FRVf LQ VLQ L L f LQ ws,req os, ws = bVLQ, wtcos, eq in.)(16 in.) = 232,600 lb > S1 OE2.  $\Box F$  Acs = bwws R23.2.6b(ii)  $\Box Fnn$   $\Box fceAcs$  with Acs = bwws Check that design esi strength ength is greater than han required strength: Anchor Tie T1 by welding eight No. 4 reinforce PHQWWRDVWHHODQJOHRI/[[LQ5HIHUWR Fig. E4.4 for details. Fig. E4.4 for de is 4 in. 198 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Minimum reinforcement for crack control 16.5.6.6 Closed stirrups are required parallel to the reinforcement T1 to be uniformly distributed ZLWKLQRIWKHHIIHFWLYHGHSWKDGMDFHQWWRT1 as recommended by the descriptive requirements of ACI 318  $16.5.5.2 \ 2/3(18.0) = 12$  in. The area of these ties or stirrups must exceed: (16 kip) Ah = 0.5 | 1.6 in.2 - = 0.64 in.2 \ 0.85(60 ksi) | Ah = 0.5 | 1.6 in.2 - = 0.64 in.2 \ 0.85(60 ksi) | Ah = 0.5(Ast - An) Use three No. 3 closed stirrups. where An is the area of reinforcement resisting the tensile force T1. Av = 3(2)(0.11) = 0.66 in.2 Area of vertical web reinforcement provided shall with average spacing of 12/3 = 4.0 in. not be less than: A 2(0.11) sin  $60.5^\circ = 0.003 \ge 0.003$  OK  $\sum$  si sin  $\gamma$  i = bsi 16(4.0) Minimum reinforcement must satisfy:  $23.5.3 \sum$  Asi sin  $\gamma$  i  $\ge 0.003$  bsi ZKHUHÛi is the angle between the axis of minimum reinforcement must satisfy:  $23.5.3 \sum$  Asi sin  $\gamma$  i  $\ge 0.003$  bsi ZKHUHÛi is the angle between the axis of minimum reinforcement must satisfy:  $23.5.3 \sum$  Asi sin  $\gamma$  i  $\ge 0.003$  bsi ZKHUHÛi is the angle between the axis of minimum reinforcement and the axis of strut. reinforcement provided. Step 12: Reinforcement detailing ing 1RWH:HOGUHLQIRUFHPHQWWR/[[LQVWHHODQ]OH Fig. E4.5—Double-corbel reinforcement based on strut-and-tie design. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 199 Strut-and-tie design. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 199 Strut-and-tie design. example where the forces on the left side are equal to the forces on the right side. While it may be reasonable to predict that the dead loads on both sides are equal, the same cannot be WUXHQHFHVVDULO\IRUWKHOLYHORDGVWRPDWFKDWWKHVDPHLQVWDQFHLQWLPH%HORZ¿QGDSRVVLEOHVWUXWDQG tie model for two-sided corbel with the load on the left side is not equal to the load on the right side. The PRGHOVKRZVWKHIRUFHGLVWULEXWLRQLQWKHPRGLiHUHQFHLQIRUFH in the column longitudinal reinforcement between the right side. Fig. E4.6—Proposed osed strut-and-tie model if forces are not ot eq equal on corbe corbels. 1 If the load on the rightt side of the cor corbell reduces furth further to rreach thee "no load load" condition, the model will ne-sided Example American Concrete Institute – Copyrighted © Material – www.concrete.org 200 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Strut-and-tie Example 5: Design a pile cap subjected to the dead and live load axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial forces and to axial
forces and to axial forces and to a Case II: D = 550 kip L = 175 kip MD = 200 ft-kip ML = 200 ft-kip fcg SVL fy = 60,000 psi Assume pile has 6 in. embedment into pile cap. Fig. E5. E5.1—Pile cap subjected to axial and mome moment forces. ACI 318-14 Discussion Step 1: Calculate ultimate load and reaction Case I: Pile factored load 5.3.1 U = 1.2D + 1.6L Case II: U = 1.2D + 1.6L Mu = 1.2MD + 1.6ML Calculation U = 1.2(550 kip) + 1.6(275 kip) + 1.6(275 kip) + 1.6(275 kip) + 1.6(200 ft-kip) = 560 ft-kip) = 560 ft-kip Pu = U = 940 kip Mu = 1.2(550 kip) + 1.6(200 ft-kip) = 560 ft-kip) = 560 ft-kip Pu = U = 940 kip Mu = 1.2(550 kip) + 1.6(200 ft-kip) = 560 ft-kip) = 560 ft-kip Pu = U = 940 kip Mu = 1.2(550 kip) + 1.6(200 ft-kip) = 560 ft-kip) = 560 ft-kipCase I: M = 0 P M ± n d R = 550 kip + 275 kip = 165 kip 5 Ru = Case II: M R = 550 kip + 175 kip 200 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(275 kip) = 220 kip 5 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWKH table in Step 2.) Step 2: Pile reaction summary 1.2(550 kip) + 1.6(175 kip) 560 ft-kip ± 5 (4)(3.5 ft)2 3.5 ft Ru = 9DOXHVIRU&DVH, HTXDWLRQVDUHIRXQGLQWH table in S ft American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie R= 202 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Design pile cap per ACI 318-14 sectional analysis Critical section at face of column calculated for Case II (controls): Shear: 22.5.5.1 Vu = 2Ru Vu [] f c' bd 21.2.1(g) Vu = 2 × 228 kip = 456 kip Vu = 2(0.75) 4000 psi (120 in.)d 8VH[UDWKHUWKDQEHFDXVHVWUHQJWK reduction factors are related to member behavior. Solving for d: d•LQ Flexure: 8VHPD[LPXPPRPHQWWKHSLOHFDSLVVXEMHFWHG to: Try: d = 44 in. - 3 in. (cover) - 1.5 in. (center of reinforcement) d = 39.5 in. M u =  $\sum$  Ru d Mu = (2)(228 kip)(2.5 ft) = 1140 ft-kip d = c.l. of pile to c.l. of col. - col./2 24.4.3.2 Check reinforcement: \$VD UVWWULDO Mu [Asfy(0.9)d As, req = 1140 ft-kip × 12 = 7.1 in.2 (0.9)(60 ksi)(0.9)(39.5 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement orcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement orcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement orcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement orcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement orcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement orcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39.5 (39 in.) k (0.9 Try 10 N No. 88; As = 77.9 9 iin.2 um reinforcement ratio Check minimum ratio: 21.2.2 7.9 in..2 = 0.00167 < 0.0018 (39. $(120\ 120\ in.)$  A  $\rho = s \ge 0.0018\ 01\ bd\ \rho = Check\ actual\ required\ reinforcement:$  Cc = 0.85fcgba T = As fy Cc = T Use 10 No. o. 9: As = 1 10.0 in.2 (0.85)(4000\ psi)(120\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.2)(60,000\ psi\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = (10.0\ in.)(a) = in. 2.5 \ 40 4000 psi / Ed = 28.5 in. ZLWKzt = ze = zs = 1.0 and (cb + Ktr)/db = 2.5. For bars with hooks: (0.02 \ne f y) A dh = | | db \  $\lambda$  f c' / (0.02(60,000 psi) A dh = | | db \  $\lambda$  f c' / (0.02(60,000 psi) A dh = | | 1.0 in. = 19 in. \ 1.0 4000 psi / There is enough distance to develop straight bars at the face of the column. In this example, hooked bars will be used as common in practice. Cracking: &KHFNVHUYLFHORDGÀH[XUDOVWUHVVDWFROXPQIDFH Ma = p (number of piles es one side)(dist.) side n Ma = 165 kip(2 kip(2)(2.5 ft) = 825 ft-kip tion modulus: SM S M = ((10 ft)(3.67 3.67 ft)2 = 22.4 ft 3 6 Stress: St ss: fr = 00 ft-lb ft lb 825,000 = 256 psi  $\approx$  4.0 f c 3 22.4 ft )(1 12) 2 (22.4 Cracking stress) is acceptable < 7.5 Step 4: Establish strut-and-tie model and determine truss forces for Case I )URPVWDWLFVHDFKSLOHLVVXEMHFWHGWRWKHVDPHORDG (refer to Table in Step 1).
'LYLGHWKHXOWLPDWHORDGDWWKHWRSQRGHLQWR¿YH HTXDOORDGVDSSOLHGRQ¿YHHTXDODUHDVRIWKHFROumn: 242/5 = 115.2 in.2 or 10.7 x 10.7 in. Assume the forces are acting at the center of each of those DUHDVDQGIRUVLPSOL¿FDWLRQRIFDOFXODWLRQDVsume each of the four corner forces are applied at 14 in. apart and located symmetriFDOWRFROXPQD[LVRIV\PPHWU\DQGWKH¿IWKLV applied at the center of the column. Fig. E5.2—Proposed truss model. American Concrete Institute - Copyrighted © Material - www.concrete.org f c'. Strut-and-Tie 25.4.3.1 204 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Forces in struts and ties: Because the pile cap is symmetrical with respect to two axes, only one bottom node will be analyzed. The rest are similar. )URPVWDWLFV¿QGWKHIRUFHVLQS1, S2, S3, and T1. Take vertical distance between Nodes N1 and N2 equal to 29.5 in. | (14 in. | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | | 2 (3.5 ft)(12 in./ft) - 2 | (2 |) / (I f Assume node N2 is located below pile top surface at 5 in. From statics: S1 = R/sinĮ T1 = S1FRVĮ FRVf S2 = Control (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14 in.) | (14S1FRV[FRVf S3 = P/5 Step 5: Struts and ties forces summary Step 6: Effective stresses Determine the ef effective concrete comp compressive sive strength in struts: rut 23.4.3 [] fce  $\hat{U}$ s fcg Table 23.4.3(a) Table 23.4.3(b) Table 23.4.3(b) Table 23.4.3(c) 23.9.2 S1 = 220 kip = 430 kip sin 30.8° T1 NLSFRVf FRVf NLS S2 NLSFRVf FRVf NLS S3 = 220 kip = 430 kip sin 30.8° T1 NLSFRVf FRVf NLS S2 NLSFRVf FRVf NLS S3 = 220 kip sin 30.8° T1 NLSFRVf FRVf NLS S2 NLSFRVf FRVf NLS S3 = 220 kip sin 30.8° T1 NLSFRVf FRVf NLS S2 NLSFRVf FRVf NLS S3 = 220 kip sin 30.8° T1 NLSFRVf FRVf NLS S2 NLSFRVf FRVf NLS S3 = 220 kip sin 30.8° T1 NLSFRVf FRVf NLS S2 NLSFRVf FRVf NLS S3 = 220 kip sin 30.8° T1 NLSFRVf FRVf NLS S3 = 2 Uncracked (prismatic) (S1): crack strut (prismat us  $1 = (0.75)((0.75)(0.85)(1.0)(4000 \text{ psi}) = 2550 \text{ psi ce aped strut with reinforcement (S2): Bottle-shaped strut without reinforcement (S2): Bottle-shaped strut with$ 1530 psi American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 205 Step 7: Node N1 has one strut, S1, and one tie in one direction, T1, framing into it. The third side is bound by the support pile. Therefore, Node N1 has one strut, S1, and one tie in one direction, T1, framing into it. The third side is bound by the support pile. Therefore, Node N1 has one strut, S1, and one tie in one direction, T1, framing into it. The third side is bound by the support pile. & 71RGH[]fce = 2040 psi %RWWOHVKDSHGVWUXW[]fce = 1530 psi (without reinforcement) Fig. E5.3—Bottom node geometry. 23.9.1 23.3.1(c) Check bearing strength: []Fnn []fceAnz Anz = 14 in. × 14 in. = 196 in.2 []Fnn = (2040 psi)(196 in.2) = 399,840 lb Check if design strength h is greater than the required strength. []Fnn = 399,840 lb > Ru OE2. ulta is located ated at 7.5 in. above ve Assume tie resultant her the node depth is taken as top of pile. Therefore, 2 x 7.5 in. = 15 in. R23.2.6b(i) y, calculate ulate the width oof strut from: From geometry, cosș ws = EbVLQşwsco 23.4.1(a) 6WUXWVWUHQJWK[]Fns = []fceAcs []F Fns = (15 (1530 psi)(14 in.)(20.1 in.) Check if design strength is greater than the required strength. []Fns = 430,500 lb > S1 OE2. Calculate the required reinforcement area: []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 and []Fnt•Fu = T1 Ats = 23.7.3 LQ f ws LQ VLQfLQ FRVf n. ws = 20.1 in. 261,000 lb = 5.8 in.2 (0.75)(60,000 psi) Choose eight No. 8 bars or six No. 9; Ast,prov = 6(1.0) = 6.0 in.2 Arranged in two fields are a strength. []Fns = 430,500 lb > S1 OE2. Calculate the required reinforcement area: []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 and []Fnt•Fu = T1 Ats = 23.7.3 LQ f ws LQ VLQfLQ FRVf n. ws = 20.1 in. 261,000 lb = 5.8 in.2 (0.75)(60,000 psi) Choose eight No. 8 bars or six No. 9; Ast,prov = 6(1.0) = 6.0 in.2 Arranged in two fields are a strength. []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 and []Fnt•Fu = T1 Ats = 23.7.3 LQ f ws LQ VLQfLQ FRVf n. ws = 20.1 in. 261,000 lb = 5.8 in.2 (0.75)(60,000 psi) Choose eight No. 8 bars or six No. 9; Ast,prov = 6(1.0) = 6.0 in.2 Arranged in two fields are a strength. []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 and []Fnt•Fu = T1 Ats = 23.7.3 LQ f ws LQ VLQfLQ FRVf n. ws = 20.1 in. 261,000 lb = 5.8 in.2 (0.75)(60,000 psi) Choose eight No. 8 bars or six No. 9; Ast,prov = 6(1.0) = 6.0 in.2 Arranged in two fields are a strength. []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 and []Fnt•Fu = T1 Ats = 23.7.3 LQ f ws LQ VLQfLQ FRVf n. ws = 20.1 in. 261,000 lb = 5.8 in.2 (0.75)(60,000 psi) Choose eight No. 8 bars or six No. 9; Ast,prov = 6(1.0) = 6.0 in.2 Arranged in two fields are a strength. []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 and []Fnt•Fu = T1 Ats = 23.7.3 LQ f ws LQ VLQfLQ FRVf n. ws = 20.1 in. 261,000 lb = 5.8 in.2 (0.75)(60,000 psi) Choose eight No. 8 bars or six No. 9; Ast,prov = 6(1.0) = 6.0 in.2 are a strength. []Fnt []Ats fy + Atp(fseafp) Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a strength Atp = 0 in.2
are a strength Atp = 0 in.2 are a strength Atp = 0 in.2 are a layers at 5 in. and 10 in. American Concrete Institute - Copyrighted © Material - www.concrete.org Strut-and-Tie Strut effective stress controls. 206 23.9.2(a) 23.4.3(c) THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Node N2—CCC: &&1RGH[fce = 2550 psi 3ULVPDWLFVWUXW]fce = %RWWOHVKDSHGVWUXW[]fce = 1530 psi (without reinforcement) Fig. E5.4—Top node geometry. 23.4.1(a) Strut S3—Bottle-shaped strut without reinforcement: Calculate Strut S3 area: []Fns []fceAcs Acs = Remaining available ailab area for the four identical diagonal strutss (S1): A2 = (Acolumn - A1)/4 A2 = ([2 ([24 in.][24 4 in in.]] - 144 14 in. 2)/4 = 108 in. 2 na struts uts are square: Assume diagonal [Fns [fce Acs ]f ws = ws 2 = 220,000 lb = 144 in. 2 1530 gsi in 108 in. = 6.4 in. 16.8 in. R23.2.6b(i) ws = ws2VLQ[ws1cos] 23.4.1(a) Strut S2—Prismatic: Calculate strut strength: [Fns ]fcews1ws [Fns = (2550 psi)(16 in.)(15.7 in.) = 640,560 sin = 64, in. 16.8 in. R23.2.6b(i) ws = ws2VLQ[ws1cos] 23.4.1(a) Strut S2 Prismatic: Calculate strut strength: [Fns ]fcews1ws [Fns = (2550 psi)(16 in.)(15.7 in.) = 640,560 sin = 64, in. 16.8 in. R23.2.6b(i) ws = ws2VLQ[ws1cos] 23.4.1(a) Strut S2 Prismatic: Calculate strut strength: [Fns ]fcews1ws [F lb Check that strut design strength is greater than required strength:  $\Box$ Fns = 640,560 lb > S2 OE2. LQ LQ VLQfws1FRVf ws1 = 15.7 in. Total pile cap depth: H = 29.5 in. + 6 in. +15.0 in./2 + 15.7 in./2 = 50.8 in. Say 52 in. a = (144.1) | 2 + 2 (6.4) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 in. assumed) | (12 = 6.5 in. (7 Therefore, use a pile cap depth of 4 ft 4 in., reinforced as shown in Fig. E5.9. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL Step 8: Development Check development length: 0.02 f y db A dh =  $\lambda$  25.4.3.1 f c' ldh = 207 0.02(60,000 psi)(1.125 in.) 4000 psi = 21.4 in. There is adequate space to develop the reinforcement. Provide No. 6 at 6 in. on center for shrinkage and temperature reinforcement placed between piles each way (refer to Fig. E5.9). Step 9: Establish strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case I: Strut-and-tie model and determine truss forces for Case II From Case II = 29.5 in. f \$QJOHFDOFXODWHGDERYH f Fig. E5 Fi E5.5—Strut-and-tie t and tie forces. American Concrete Institute - Copyrighted © Material - www.concrete.org 208 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Calculate forces in strut-and-tie model Forces on pile cap from column are calculated from simple static: P = Pu M u ± n d P = 940 kip 560 ft-kip × 12 in./ft ± 5 14 in. × 2 P1,2 = 188 kip + 240 kip = 428 kip P3,4 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 =
188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = 188 kip - 240 kip = -52 kip (tension) P5 = -52 kip (tens  $kip = 336 kip sin 26.1^{\circ} T2 = 52 kip 23.2.7 QV QGWLHVKDOOQRWEH VVWKD$ \$QJOHEHWZHHQVWUXWDQGWLHVKDOOQRWEHOHVVWKDQf  $\gamma = \arctan 3.5 ft$ (12) + 7 in. (3.5 ft)(12) + 7 in. (3.5 ft)(12) - 7 in. (58 kip)sin 25.4^{\circ} = 35 kip cos 45^{\circ} \theta = arctan (3.5 ft)(12) + 7 in. = 54.5^{\circ} (3.5 ft)(12) - 7 in. T3 NLS FRVf FRVf NLS FRVf T3 = 200 kip S3 NLS FRVf FRVf NLS FRVf FRVf NLS Step 11: Force summary American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 209 Step 12: Design nodal zones Node N1 has one strut, S1, and one tie in one direction, T1, framing into it. The third side is bound by the support pile. Therefore, Node N1 is CCT. 23.4.1(a) 23.4.3(c) From Step 6: &&71RGH[]fce = 2040 psi (without reinforcement) Fig. E5.6—Bottom node geometry. 23.9.2 Check bearing strength:  $\varphi$ Fnn []fceAnz Anz = 14 in. × 14 in. = 196 in.2 []Fnn = (2040 psi)(196 in.2) = 399,860 lb Check if design strength is greater eater than the required strength. Fnn = 399,860 lb > Ru OE2. []F ltant is located at 8 in. above top Assume tie resultant ore, the node de depth is ttaken as 2 x of pile. Therefore, 8 in. = 16 in. R23.2.6b(i) ry calculate ulate the width oof strut from: From geometry, ws = Ebsin0 + wsccos0 23.4.1 6WUXWVWUHQJWK Fins = [fceAcs psi)(14 in.)(20.9 in.) = 447,678 lb [Fns] = (1530 ps Check if design strength is greater than the required strength. <math>[Fnn] = 447,678 lb > S1 OE2. 23.3.1(b) 23.7.2 Required reinforcement to resist Tie T1: [Fnt] = Ats fy LQ VLQfLQ FRVf Qf ws 2. ws LQ2. 271,000 lb 2 (23.7.2) Ats = = 6.0 in. 0.75 (60,000 psi) Choose eight No. 8 bars; Ast = 8(0.79 in.2) = 6.32 in.2 or six No. 9; Ast = 6(1.0 in.2) = 6.0 in.2 Arranged in two layers as shown in Fig. E5.8. American Concrete Institute – Copyrighted © Material – www.concrete.org Strut-and-Tie Strut effective stress controls. 210 23.9.2(a) 23.9.2(b) 23.4.3(c) THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Node N2—CCT: From Step 6: && 1530 psi (without reinforcement T2 has 2.5 in. cover. The node width could then be calculated as twice the reinforcement depth = 5 in., resulting in pile cap. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 211 Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.9—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.9—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.9—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8—Pile cap depth and reinforcement as obtained from sectional design. Fig. E5.8 www.concrete.org Strut-and-Tie Step 14: Reinforcement detailing 212 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 2QHRIWKHLPSRUWDQWFRQFOXVLRQVRIVWUXWDQGWLHPRGHOLQJLVWKHORFDWLRQRIUHLQIRUFHPHQWLQDSLOHFDS While traditional sectional design places reinforcement between piles, strut-and-tie model design places reinforcement above piles to resist the tension forces resulting from the 3-D struts due to column loading. 2. It is recommended to add additional reinforcement around the perimeter of a pile cap at pile level to prevent spalling of concrete cover in case piles are resisting lateral forces. 3. It is important to simplify the assumptions: a. Use square struts rather than bottle-shaped or those varying in cross section along length of strut to simplify complex geometry where struts intersect in three dimensions. b. Geometric dissimilarities between struts and nodes must be neglected (but checks should be made to DVVXUHWKHFHQWURLGLVSURSHUO\ORFDWHGDQGQRGHDUHDLVVXI¿FLHQW American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15-ANCHORING TO CONCRETE 15.2-Materials \$QFKRU GHVLJQ VWUHQJWK LV LQAXHQFHG E\ ERWK WKH VWHHO anchor characteristics (yield strength, ductility diameter, HPEHGPHQW OHQJWK DQG WKH PHPEHU¶V VSHFL¿HG FRQFUHWH strength. All types of steels are allowed, but there is approximately 10 to 15% design strength reduction for using less ductile if the tensile elongation as measured in accordance with ASTM F606 is at least 14% with a reduction in area of at least 30%. Some steels, such as A307 bolts and A615 reinforcing bars, are deemed Table 15.2—Depth limits for post-installed adhesive anchors, in. da Min. 4da Max. 20da 1/4 1.0 5.0 3/8 1.5 7.5 1/2 2.0 10.0 5/8 2.5 12.5 7/8 3.5 17.5 1 4.0 20.0 to meet this requirement without testing. A restriction on the maximum ratio of tensile strength too yield strength is imposed to prevent yielding of anchors at service load OHYHOV\$&, JWKHDQFKRUUHVLVWVVLJQL¿cant seismic forces, other restrictions—for example, on the ratio of tensile ultimate to yield strength—may apply (ACI 318-14, 17.2.3.4.3). Cast-in anchors do not have embedment depth limits, but post-installed adhesive anchor embedment depths are limited to 4da"hef"da (see Table 15.2). For anchor diameters larger than 4 in., testing is required. Post i Post-installed mechanical anchors and post-installed DGKHVLYHDQFKRUV DGKHV ttee 355 55 2011) spe p Committee 2011), rrespectively. alcu n purposes For calculation purposes, the concrete strength fcgFDQQRW exce 10,00 psi for cast-i cast-in anchors or 8000 psi for cast-i cast-in anchors. For con installed concrete strength fcgFDQQRW exce 10,00 psi for cast-i cast-in anchors. concrete. factor & for lightweight n assumptions 8-14, Chap ACI 318-14, Chapter 17 assumptions to calculate anchor reactions are usually calculated by either (a) or (b): (a) elastic analysis by varying the anchor reactions are usually calculated by either (a) or (b): (b): (c) elastic analysis by varying the anchor reactions are usually calculated by either (b): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually
calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c): (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) or (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) elastic analysis by varying the anchor reactions are usually calculated by either (c) elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varying the anchor elastic analysis by varyi linearly with distance from axis of rotation (b) inelastic analysis by force redistribution among ductile anchors 3. Friction between the base plate and the concrete is ignored 4. Anchor tension strength is unaffected by the presence of DQDGMDFHQWFRPSUHVVLRQ¿HOG ACI 318-14, Chapter 17 design assumptions include: &UDFNHG FRQFUHWH PHPEHUV KDYH VXI¿FLHQW UHLQIRUFHment to restrain cracking to acceptable widths under design loads 2. Anchors in a group are of a similar type, size, and depth ,QEXLOGLQJVVXEMHFWWRHDUWKTXDNHIRUFHVDQFKRUVDUH not located in plastic hinge zones To evaluate a preliminary design, consider: 1. The location of anchors relative to each other, to the base plate edges, and to the edge of concrete 2. The anchor type (cast-in, mechanical post-installed, adhesive) American Concrete or post-installed in hardened concrete, are used to transfer shear and tension forces to a concrete member. Cast-in anchors are usually headed studs, headed bolts, hooked bolts, or threaded rods with nuts. Post-installed anchors. Chapter 17 of ACI 318-14 is used for the design of anchors in concrete for two main applications: (a) connections between structural members; and (b) attachments of nonstructural, safety-related elements to a structural member. Cast-in anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may be accurately placed with respect to reinforcing bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible Disadvantages are: • Anchors may bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible DISAdvantages are: • Anchors may bars • 0DQ\ DQFKRU VL]HV FRQ¿JXUDWLRQV DQG OHQJWKV DUH possible DISAdvantages are: place m • Anchors that are not adequately held in may shift from their intended location during the placement of concrete rete is placed • Anchors cannot be moved after ott ne• Anchors in walls and the bottom off slabs require ppenetrations in the formwork. ta nto drilled holes after fter Post-installed anchors are installed into alle anchors chors transmit loads ds concrete has hardened. Post-installed to the concrete by friction, bearing, bond, or a combination of these mechanisms. Advantages are: • Anchors may be accurately placed with respect to attached components • Avoids formwork penetrations Disadvantages location with respect to reinforcing bars is usually uncertain, and drilling anchor holes may damage reinforcement • Post-installed anchors generally have lesser design strength than cast-in anchors with equal embedment depth and diameter • Inspection requirements for post-installed anchors may be greater than for cast-in anchors. FORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 15.4.1—Different eccentricity loading conditions applied to a group of anchors (Eligehausen et al. 2006). 3. The anchor diameter and embedment length (an initial embedment length of 8da can be assumed) 4. The resolution of applied factored normal forces, shear forces, and moments into anchor reactions 5. The Seismic Design Category (SDC) the building is assigned to. If C, D, E, or F, and seismic effects account for more than 20% of the factored loads, then 17.2.3 applies. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 215 Table 15.4.2—Anchors resisting a shear force %UHDNRXWFRQ&JXUDWLRQFKHFN Bolt-to-base plate connection Bolt spacing Case 1 9IRUEUHDNRXWRULJLQDWLQJRQ front row of bolts (Fig. 15.2(a)) Case 2 9IRUEUHDNRXWRULJLQDWLQJDW rear row of bolts (Fig. 15.2(b) and 15.3) Anchors not welded to base plate s•ca1,1 Yes Yes No s < ca1,1 No No Yes Anchors welded to base plate All spacing No Yes No Anchorage Case 3 9IRUEUHDNRXWRULJLQDWLQJRQ front row of bolts (Fig. 15.4.2a—Critical —Cr condition for aanchors 15.4.1 Tension Connections that primarily resist normal forces can be divided into three categories based on the resolution of the applied normal forces and moments: 1. No tension on all anchors (Fig. 15.4.1(a)); or 3. Tension on some anchors (Fig. 15.4.1(b) and (c)). )RUFRQQHFWLRQVLQWKH¿UVWFDWHJRU\WKHFRQFUHWHEHDULQJ strength under the plate is calculated according to ACI 318-14, Section 22.8.3.2. Refer to Example 1 and Examples 5 through 4 and 8 through 19. For connections in the third category, anchors within the compression area of the plate do not resist any tension. For anchors outside the compression area of the plate, the design il stren nchors is calculated. Refer to Examples tensile strength of anchors 4 through thr ugh 17. 14 15. 2 Sh Connecti 15.4.2 Shear—Connections that primarily resist shear for d d iinto t ttwo categories based on the degree force can bbe divided RI &[LW\ RI WKH DQFKR DQFKRUV WR WKH EDVH SODWHV nchors ered. In Ca Case 1, half the total shear force is assumed resisted by breakout of anchors in the row closest to the edge (Fig. 15.4.2a) (a)). In Case 2, the assumed breakout originates from the edge to resist the full shear force (see Fig. 15.4.2a (b)). In both Cases 1 and 2, the spacing between the two anchor rows exceeds the edge is assumed to contribute to the design strength if spacing between two anchor rows is less than the edge distance (see Fig. 15.4.2a (c)). Refer to Examples 14 and 15, and 14 and 15, and 14 and 1 furthest away from the edge (see Fig.
15.4.2b and Table 15.4.2). 15.4.3 Interaction—For connections that resist both VLJQL¿FDQW WHQVLRQ DQG VKHDU IRUFHV LQWHUDFWLRQ RI WKHVH forces on each anchor are usually considered by N ua Vua + ≤ 1.2 φN n φVn American Concrete Institute – Copyrighted © Material – www.concrete.org (15.4.3) 216 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 15.6.5a—Group of anchors close to edge. where Nn and Vn are the least values of the required strengths of considered failure modes (see ACI 318-14, Section 17.6). Refer to Examples 8 through 11, Examples 14 and 15, and Examples 18 and 19. If supplementary of the required strengths of considered failure modes (see ACI 318-14, Section 17.6). Refer to Examples 8 through 11, Examples 14 and 15, and Examples 18 and 19. If supplementary of the required strengths of considered failure modes (see ACI 318-14, Section 17.6). anchor reinforcement is provided to prevent a concrete failure mode, then that failure mode is excluded from the interaction equation. 15.5—Discussion on anchors resisting tension If anchors are relatively close to an edge orr close together, VLJQL¿FDQ FDOFXODWHGDQFKRUGHVLJQVWUHQJWKVDUHVLJQL¿FDQWO\UHGXFHG 15.5.1 Steel strength—Anchor steel strength usually controls over concrete breakout stre strength, edge distance d reinforcement is used, or anchor distance, and H FFRQFUHWH HWH EUHDNRXW VWUH JWK VSDFLQJ DUH VXI¿FLHQW WR HQVXUH VWUHQJWK exceeds steel strength, anchors sh uld To maximize concrete breakout should conform to: ro an edge d by at least (a) Anchors are spaced at least 5.0hef for cast-in and post-installed mechanical anchors or (b) Anchors within groups are spaced at least the greater of 3.0hef and 2.0cNa for post-installed adhesive anchors. Tables 1 a through 1e, located at the end of the Examples section, provide the material properties and geometries for standard anchor design steel strength. Tables 2 and 3 provide the material properties and geometries for standard anchor design steel strength. geometry does not allow steel strength to control, concrete breakout usually controls. Refer to ACI 318-14, 17.4.2 to calculate design concrete breakout strength. 15.5.3 Pullout strength to control, concrete breakout usually controls. Refer to ACI 318-14, 17.4.2 to calculate design concrete breakout strength. through 1e provide bearing areas for different headed anchors and nut types. For hooked bolts. Refer to ACI 318-14, 17.4.3 to calculate design pullout strength... 15.5.4 Concrete side-face blowout for single anchors is a function of concrete strength, edge distance, and anchor head bearing area. Refer to ACI 318-14, 17.4.4 to calculate design side-face blowout strength. 15.5.5 Bond strength of adhesive anchor—For anchors TXDOL; Science and anchor head bearing area. Refer to ACI 318-14 (hapter 17 provides default values) that are very conservative for the situation where the design is performed without ERQG ERQGYDOXHVIURPDVSHFL¿FUHSRUW5HIHUWRWR calculate design bond strength. See Table E.2 for an example of testt data provided by a manufacturer. Disc sion on an 15.6—Discussion anchors resisting shear If anchors hors are relatively latively cl close to an edge, or close together, or loc ted in a thin hin memb located member, calculated anchor design VDU FDQWO\UHG VWUHQJWKVDUHVLJQL¿FDQWO\UHGXFHG 15.6.1 Steel strength usually controls if either eit r an anchor reinforcement is used (and thus concrete bbreakout is avoided), ided), or if edge distance and spacing between H VXI¿FLHOW DOFKRUVDUHVXI¿FLHOWWRHOVXUHFROFUHWHEUHDNRXWVWUHOIWK eds steel strength. Tables 1 a through 1 e provide the material properties and geometries for standard anchor bolts and studs. Tables 2 and 3 provide anchor design steel strength. Refer to ACI 318-14, 17.5.1 to calculate design steel strength in shear. 15.6.2 Concrete breakout strength—If the connection geometry does not allow steel strength to control, concrete breakout strength. 15.6.3 Concrete breakout strength. concrete breakout strength in tension. Refer to ACI 318-14, Section 17.5.3 to calculate design pryout strength, Pryout failure is typically related to shallow embedments of 4.5da to 5.5da depending on strengths of materials used. 15.6.4 Shear parallel to the edge—Steel strength controls if either anchor reinforcement is used and thus concrete breakout is avoided, or edge distance and spacing between DQFKRUVDUHVXI&FLHQWWRHQVXUHFRQFUHWHEUHDNRXWVWUHQJWK exceeds steel strength at both edges needs to be calculated (see Fig. 15.6.5a). The corner breakout strength is the American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7—Minimum spacing and edge ge distance to prevent concrete.org CHAPTER 15—ANCHORING TO CONCRETE 217 Table 15.7 strengths. ngths. Minimum, in. M Anchor type Cast-in-place s ca,min Untorqued 4da NA Torqued 6da 6dda 4da 8dda 6da 10da 6da 6da 8dda 6da 10da 6da 6da 8dda 6da 10da 6da 6da 8dda 6da 10da 6da 6da 8dda 6da 8dda 6da 10da 6da 8dda 6da /HVVHUGLVWDQFHVDUHSHUPLWWHGLISURGXFWVSHFL¿FWHVWVDUHSHUIRUPHGDFFRUGLQJWR ACI 355.2 or ACI 355.4. † smaller of the two strengths (parallel and perpendicular to the shear force). For cast-in anchors, anchor reinforcement is recommended. ACI 318 Chapter 17 does not address shear force applied in a non-orthogonal direction. Eligehausen et al. (2006) suggests resolving the force into two orthogonal components, evaluating the concrete shear strength in each direction (see Fig. 15.6.5b), and proposes the following equation 2 15.7—Limitations geometry Lim ons on installation in Installation in Installation In latio of cast-in or post-installed anchors close to an edge failure unless minimum ed e can ca result in a splitting spl i and edge distances stances aare maintained or supplemental spacing einfo eme is provided. ovided. R reinforcement Refer to Table 15.7 for minimum spaci and edge distance. spacing REFERENCES can Concrete Institute \$&, \$&, \$4XDOL¿FDWLRQ RI 3RVW,QVWDOOHG 0HFKDQical Anchors in Concrete and Commentary \$&, <sup>2</sup>4XDOL $\dot{c}$ FDWLRQ RI 3RVW,QVWDOOHG\$GKHVLYH Anchors in Concrete and Commentary Authored reference Eligehausen, R.; Mallée, R.; and Silva, J., 2006, Anchorage in Concrete Construction, Ernst & Sohn (Wiley), 378 pp. 2 (Vu $\alpha$  cos  $\alpha$ ) (Vu $\alpha$  sin  $\alpha$ ) | + |  $\leq$  1.0 \ Vcb , & / \ Vcb ,  $\bot$  ) (15.6.5) American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage Fig. 15.6.5b—Group of anchors subjected to inclined force close to two free edges. 218 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 15.8—Examples Anchorage Example 1: Baseplate anchors not subjected to shear force or tension A structural steel column is supported on a 24 in, x 24 in, reinforced concrete pedestal, and carries a service gravity load of 350 kip dead load and 175 kip live load. The pedestal is constructed of normalweight concrete, with fco SVL7KHFROXPO steel base plate is 1 in, x 16
in, x 16 i reinforcement. Given: D NLS L NLS fcq SVL 3a QRUPDOZHLJKWFRQFUHWH da LQ a LQ Fig. g E1 E1.1—Steel —St column supported pp by concrete pedestal. ACI 318-14 Discussion cussion Step 1: Determine factored load Calculate required strength: 5.3.1 U D + 1.6L Step 2: Determine factored load Calculate the design bearing strength of concrete: 22.8.3.2 [Bn ] fcoA1 G 22.8.3.2 [The pedestal is wider than the base plate; therefore, the bearing strength of the loaded area is increased by: A2  $\leq$  2.0 A1 Calculation Ca NLS U NLS NLS V [IBn ] fcoA1 G 22.8.3.2 [The pedestal is wider than the base plate; therefore, the bearing strength of the loaded area is increased by: A2  $\leq$  2.0 A1 Calculation Ca NLS U [IBn ] fcoA1 G 22.8.3.2 [The pedestal is wider than the base plate; therefore, the bearing strength of the loaded area is increased by: A2  $\leq$  2.0 A1 Calculation Ca NLS U [IBn ] fcoA1 G 22.8.3.2 [The pedestal is wider than the base plate; therefore, the bearing strength of the loaded area is increased by: A2  $\leq$  2.0 A1 Calculation Ca NLS U [IBn ] fcoA1 G 22.8.3.2 [ Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 219 Step 3: Determine anchor bolt length for bolts that do not resist tension or shear. For gravity columns that do not resist tension or shear. For gravity columns that do not resist tension or shear. detailing practice. See AISC Manual of Steel Construction1RWHDOVRWKDW26+\$UHTXLUHPHQWV dictate a minimum of four anchor bolts. As good practice, gravity columns with four 1 in. diameter anchors should be embedded minimum 12da (12 in.) to develop the tensile strength of the anchor. Step 4: Determine pedestal reinforcement Pedestal is reinforced with eight No. 8 longitudinal bars and No. 4 transverse ties. The eight No. 8 anchor bolts are enclosed by transverse reinforcement is distributed within nsisting of two No. 4 of the top of the pedestal, consisting bars (see Fig. E1.2). Anchorage 10.7.6.1.6 Fig. E1.2—Reinforcement of typical pier. American Concrete Institute – Copyrighted © Material – www.concrete.org 220 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Anchorage Example 2: Cast-in headed anchor in Seismic Design Category D, subjected to tension only Structural steel hangers are spaced every 28 ft and support an 18 in. diameter, Sch. 80, coolant pipe. The hangers are anchored with 3/4 in. diameter, ASTM F1554 Grade 36 cast-in headed hex bolts with 7 in. embedment depth. The 12 in. thick elevated slab is normalweight reinforced concrete with fcg SVLVHH)LJ(7KHVWUXFWXUHLVDVVLJQHGWR6HLVPLF'HVLJQ&DWHgory (SDC) D. Each hanger resists a vertical seismic force of 4200 lb. Check the adequacy of the connection. Given: Loads— E = 4200 lb 6HOIZHLJKWRILQGLDPHWHU6FKSLSH¿OOHGZLWKZDWHU w = 260 lb/ft Anchor— 3/4 in. anchor ASTM F1554 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 psi Hanger spacing = 28 ft 0 in. Anchor embedment depth; hef = 7 in. Table 1b: Abrg = 0.654 in.2 Fig. E2.1—E E2.1—E in. te 3a = 1.0 (normalweight concrete) WK FLHQWUHLQIRUFH QWWRUHVWUDLQ DFN KVEXWQR 7KHVODELVDVVXPHGFUDFNHGZLWKVXIcFLHQWUHLQIRUFHPHQWWRUHVWUDLQFUDFNZLGWKVXSSOHPHQWDU\RU anchor reinforcement. ACI 318-14 Discussion Step 1: Required strength 5.3.1 U = 1.4(w)(s) U = 1.2(w)(s) + 1.0E Calculation U = 1.4(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 1.2(260 lb/ft + 5 lb/ft)(28 ft) = 10,388 lb U = 10lb/ft)(28 ft) + 4200 lb = 13,104 lb Controls U = Nua = 13,104 lb Controls U = Nua = 13,104 lb American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Step 2: Strength inequalities 17.2.3.4.2 The structure is assigned to SDC D, and more than 20% of U is due to seismic effects; therefore, the additional requirements of 17.2.3.4 apply. 17.2.3.4.3 221 E 4200 lb = = 32% > 20% N ua 13,104 lb The design must conform to 17.2.3.4.3(a)(i) The concrete-governed nominal strength must be at least 1.2 times the nominal strength. 17.2.3.4.3(a)(ii) 1.2 N sa  $\leq$  17.2.3.4.4 { N cb (concrete breakout) N pn (anchor pullout) 7RDFFRXQWIRUVLJQL:FDQWFUDFNLQJDQWLFLSDWHG during a seismic event, the design concrete tension strength is multiplied by a reduction factor of 0.75  $\varphi$ N (anchor pullout) pn (a | Anchorage The anchor design tensile strength must therefore satisfy the following inegualities: Nua = 13.104 lb Step 3: Anchor ductility 17.2.3.4.3(a)(iii) Under seismicc loading, ng, the code regulation is the following inegualities: Nua = 13.104 lb Step 3: Anchor ductility 17.2.3.4.3(a)(iii) Under seismicc loading, ng, the code regulation is the following inegualities: Nua = 13.104 lb Step 3: Anchor ductility 17.2.3.4.3(a)(iii) Under seismicc loading, ng, the code regulation is the following inegualities: Nua = 13.104 lb Step 3: Anchor ductility 17.2.3.4.3(a)(iii) Under seismicc loading, ng, the code regulation is the following inegualities: Nua = 13.104 lb Step 3: Anchor ductility 17.2.3.4.3(a)(iii) Under seismicc loading, ng, the code regulation is the following inegration is the following ine properties: 14% minimum elongation and 30% minimum area reduction 23% elongation in 2 in. of length > 14% 40% area reduction > 30% Therefore, ASTM F1554 Grade 36 is ductile. 17.2.3.4.3(a)(iii) Also, the Code requires a stretch length of 8da. 7KHEROWOHQJWKEHORZWKHVODEVRI¿WLVLQ ZKLFKLVVXI¿FLHQWIRUDVWUHWFKOHQJWK  $8 da = 8 \times 0.75$  in. = 6 in. < 8 in. (provided) 17.2.3.4.3(a)(v) The Code requires that the designer protect anchor against buckling is not considered. 17.2.3.4.3(a)(v) Continuously threaded anchors must meet the following: f uta  $\geq 1.3$  f ya f uta 58,000 psi = 1.6 > 1.3 f ya 36,000 psi American Concrete Institute - Copyrighted © Material - www.concrete.org OK OK 222 THE REINFORCED CONCRETE DESIGN
HANDBOOK—SP-17(14) Steel tensile strength (futa) times the effective crosssectional area (Ase,N). 17.4.1.2 R17.4.1.2 (17.4.1.2) Nsa = Ase, N futa From Table 1b, the physical and material properties are: da = 3/4 in. and nt = 10 threads per in. Anchor bolt area is provided by Table 1b or calculated from: Table 1b: Ase, N = 0.334 in.  $4 \cdot 10$  / The nominal strength of anchors is a function of futa rather than fyaEHFDXVHWKHPDMRULW\RIDOFKRU PDWHULDOVGRORWH[KLELWDZHOOGHcOHG\LHOG point. Table 1a: futa = 58,000 psi futa is the smaller of 1.9ffya and 125,000 psi futa is the smaller of 1.9ffya and 1.9ffya and 1.9ffya and 1.9ffya and 1.9ffya and 1.9ffya and 1.9ffya (17.4.1. or obtain either calculate en trength from Tab the nominal tensile strength is greater than required strength. OK Nsa = (0.334 (0 in.2)()  $(58,000)(58,000 \text{ psi}) = 19,372 \text{ lb se vvalue from m Table 33: Nsa} = 19,372 \text{ lb or use } 19,372 \text{ (lb or use } 19,372 \text{ (lb or use } 19,372)(0.75) \text{ [lb)} = 14,529 \text{ lb } \cong 14,500 \text{ lb } = 14,500$ Fig. E2 E2.2—Idealized dealized tension te breakout for CIP anchor. Concrete breakout must be checked because anchor reinforcement is not provided. 17.4.2.1 Nominal concrete breakout strength of a single anchor in tension: N cb = 17.4.2.5 ANc ψ ed , N ψ c , N ψ cp , N N b ANco (17.4.2.1a) Because there is only one anchor and it is located away from any edges, ANc and ANco are the same (Fig. E2.2). ANc =1 ANco zed, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGL¿cation edge factor is equal to 1.0. zed, N = 1.0 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min > 1.5hef7KHUHIRUHWKHPRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, N ± PRGLµc = 1.0 17.4.2.6 zc, and zc,N = 1.0 FUDFNLQJLVFRQWUROOHGE\AH[XUDOUHLQIRUFHPHQW 17.4.2.7 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting; anchor zcp,N = 1.0 is cast-in single anchor, therefore American Concrete Institute – Copyrighted © Material www.concrete.org 224 17.4.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) To determine basic concrete breakout strength, calculate by Eq. (17.4.2.2a) Nb = kc  $\lambda$  a f c'hef1.5 (17.4.2.2a) Substituting in Eq. (17.4.2.2a) Nb = kc  $\lambda$  a f c'hef1.5 (17.4.2.2a) Substituting in Eq. (17.4.2.2a) Substituting in Eq. (17.4.2.2a) Nb = kc  $\lambda$  a f c'hef1.5 (17.4.2.2a) Substituting in Eq. (17.4.2.2a) Substituting in Eq  $(ConGLWLRQ\% LV17.2.3.4.3 \ (b) () 5000 \ psi (7 \ in.)1.5 = 31, 430 \ (b) \ Nb = (24)(1.0)(1.0)(31,430 \ (b) =
31,430 \ (b) = 31,430 \ (b) = 31,430 \ (b) = 22,001 \ (b) \approx 22,001 \ (b)$ lb) = 16,500 lb Check that concrete breakout design strength is greater than required strength. Ncb = 31,430 lb > (1.2)(19,372 lb) = 23,246 lb Ncb + Nsa OK Step 6: Pullout 17.4.3 Pullout failure mode is resisted by bearing stress hea Failure lure is initiated initiate by crushunder the bolt head. te. ing of concrete. Npn zc, PNp 17.4.3.6 17.4.3.1 modify pullout strength of anchor zc, P = 1.0 ulate by To determine basic pullout strength, calculate Eq. (17.4.3.4) Np = (8)(0.654 in.2)(5000 in.2)(50 psi) = 26,160 lb where Abrg is obtained from Table 1b. 17.3.3c(ii) 17.2.3.4.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.3.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2.4(c) 17.2(c) 17.2.4(c) 17.2(c) 17.2(c)  $[Npn = (0.75)(18,312 \text{ lb}) = 13,700 \text{ lb} > \text{Nua} = 13,700 \text{ lb} > \text{Nua} = 13,700 \text{ lb} > \text{Nua} = 13,700 \text{ lb} > \text{Nua} = 13,700 \text{ lb} > (1.2)(19,372 \text{ lb}) = 23,246 \text{ lb} \text{ OK Check that nominal concrete tension strength is greater than required strength is greater than required strength is greater than required strength of the anchor. Npn Nsa American$ Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 225 Step 7: Side-face blowout failure mode is not considered. Step 8: Concrete splitting failure 17.7.2 For cast-in anchors not close to an edge

and where torque is not applied, splitting failure mode is not considered. Step 9: Summary ACI 318 17.4.1 17.4.2 17.4.3 17.4.4 17.7.2 Failure mode Steel Concrete breakout Concrete breakout Concrete splitting Design strength, lb []Nsa 14,500 []Nsb - NA - Controls design? No No Anchorage Step 10: Conclusions The anchor bolt is adequate for the design tensile factored load of 13,700 lb. American Concrete Institute – Copyrighted © Material – www.concrete.org 226 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Anchorage Example 3: Post-installed expansion anchor in Seismic Design Category B, subjected to tension force only Structural steel hangers are spaced at 25 ft and support an 18 in. diameter, Sch. 40, coolant pipe. Each hanger is anchored with post-installed torque-controlled expansion F1554 Grade 55 anchor. The 12 in. thick elevated slab is normalweight reinforced concrete with fcg SVLVHH)LJ( 7KHVWUXFWXUHLVDVVLJQHGWR6HLVPLF'HVLJQ&DWHJRU\6'& %(DFKKDQJHUUHVLVWV in addition to pipe self-weight and 5 lb/ft miscellaneous loads, a vertical seismic force of 300 lb. Determine the diameter and embedment depth of the post-installed torque-controlled expansion anchor. Given: Loads— E = 300 lb 6HOIZHLJKWRILQGLDPHWHU6FKSLSH¿OOHGZLWKZDWHU = 202 lb/ft Anchor— ASTM F1554 Grade 55; Table 1a: • futa = 55,000 psi • fya = 75,0 1.0 normalweight concretee ZLWK ¿FLHQWUHLQIRUFHPHQWWR 7KHVODELVDVVXPHGFUDFNHGZLWKVXI¿FLHQWUHLQIRUFHPHQWWR tai with supplementary anchor reinforcement. Manufacturer data sheet: Table E.1. ACI 318-14 Discussion Step 1: Required strength 5.3.1 U = 1.4(w)(s) U = 1.2(w)(s) + 1.0E Calculation U = 1.4(202 lb/ft + 5 lb/ft)(25 ft) = 7245 lb Controls U = 1.2(260 lb/ft + 5 lb/ft)(2anchor design tensile strength must satisfy the following inequalities:  $\int \phi N$  sa (steel strength in tension)  $| N ua \leq \langle \phi N cb (concrete breakout) | \phi N (anchor pullout) pn | Nua = 7245 lb Because the anchor is far from an edge, side-face EORZRXW] Nub (17.4.4) and splitting towards an edge (17.7.5) are not considered. American Concrete Institute -$ Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 227 Step 3: Anchor ductility to determine the DSSURSULDWH DFWRULQ6WHS 2.2 & KHFNLI)\*UDGHVDWLVċHVWKH\$&, & KDSWHUGHċQLWLRQRIGXFWLOLW\ 14% minimum elongation and 30% minimum area reduction Table 1a, ASTM F1554 Grade 55 has the following properties: 21% elongation in 2 in. of length > 14% 30% area reduction = 30% Therefore, ASTM F1554 Grade 55 is ductile. Step 4: Steel tension Design information is obtained from an anchor PDQXIDFWXUHU IVXOWVDFFRUGing to ACI 355.2. For this example, representative information is given in Table E.1. Steel tensile strength calculation: []Nsa = []Ase, N futa 17.3.3a(i) or = 0.75 Strength reduction factor []Nsa = (0.75)Ase, N(75,000 psi) (17.4.1.2) [] Anchorage 17.4.1.2 The tensile steel stren strength equation can be rearKHP XPEROWAse: UDQJHGWR¿QGWKHPLQLPXPEROWA Ase , N = N ua of uta Ase  $N \ge 7245 45 lb = 0.129 in.2 75,000 ppsi)$  (0.75)(75, da = (4)(0.129 in.2) = 0.41 in.  $\pi$  Therefore, the minimum diameter is: da = 4 Ase, N  $\pi$  Therefore, acceptable anchor diameters listed in Table E.1 are 1/2, 5/8, and 3/4 in. American Concrete Institute – Copyrighted © Material – www.concrete.org 228 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Concrete breakout 17.4.2.1 N cb = ANc  $\psi$  ed, N  $\psi$  c, N  $\psi$  cp, N N b ANco (17.4.2.2 N b = kc  $\lambda$  a f c'hef1.5 (17.4.2.2 N b = kc  $\lambda$  a f c'hef1. from an edge, ANc and ANco are the same. ANc =1 ANco zed, N±PRGL¿FDWLRQIDFWRUIRUHGJHHIIHFWVca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQHGJHIDFWRULV equal to 1.0. zed, N = 1.0 IRUFRQFUHWHFRQGLWLRQDW zc, N±PRGL¿FDWLRQIDFWRUIRUHGJHHIHFWVca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQHGJHIDFWRULV equal to 1.0. zed, N = 1.0 IRUFRQFUHWHFRQGLWLRQDW zc, N±PRGL¿FDWLRQIDFWRUIRUHGJHHIHFWVca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQHGJHIDFWRULV equal to 1.0. zed, N = 1.0 IRUFRQFUHWHFRQGLWLRQDW zc, N±PRGL¿FDWLRQIDFWRUIRUHGJHHIHFWVca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQHGJHIDFWRULV equal to 1.0. zed, N = 1.0 IRUFRQFUHWHFRQGLWLRQDW zc, N±PRGL¿FDWLRQIDFWRUIRUHGJHHIHFWVca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQHGJHIDFWRULV equal to 1.0. zed, N = 1.0 IRUFRQFUHWHFRQGLWLRQDW zc, N±PRGL¿FDWLRQDW zc, N±PRGL¿FDWLRQDW zc, N±PRGL¿FDWLRQDW zc, N±PRGLµch and
a levels; assume member is the same. Ance a set cracked and OOHGE FUDFNLQJLVFRQWUROOHGE\AH[XUDOUHLQIRUFHPHQW 1 zc,NN = 1.0 LRQ RUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWD GDQFKRUV zc,P±PRGLµ (up to 1/2 in.) have a higher sensitivity to installation variables than largerdiameter bolts (Table E.1). 17.3.3c(ii) /DUJHUEROWLQVWDOODWLRQVHQVLWLYLW\LVORZHUDQGUHOLDELOLW\LVKLJKHUWKHUHIRUHWK corresponding depths for concrete breakout strength in tension: Diameter, in. 1/2 5/8 3/4 Suitable embedment, in. 5.5 4.5 and 6.5 5 and 8 Installation sensitivity category 2 1 1 Minimum hef, in. 5.5 5.0 5.0 17.4.3.1 [Npn zc, PNp • OE Npn zc, PN For a cracked slab section: For post-installed expansion anchors, the pullout ns of diam diamstrength, Np, for different combinations vided in the evaluation report (Table E.1), there evaluation report (Table E.1), the evaluation report (Table E.1), the evaluation report (Table E.1) and the evaluation report (Table E.1) are the evalua embedPHQWWKDWVDWLViHVSXOORXWVWUHQJWKXVLQJ(T is shown in table below: Anch Anchor pullout de design strength, lb [Nppn = (0.65)(7544 (0.5 4 lb) = 533 5337 lb < 7245 lb - NG [N Npn = (0.65)(14,254 0.65 lb) = 92 9265 lb > 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - NG [N Npn = (0.65)(14,254 0.65 lb) = 92 9265 lb > 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - NG [N Npn = (0.65)(14,254 0.65 lb) = 92 9265 lb > 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 lb < 7245 lb - OK [N Npn = (0.65)(9617 (0.6 7 lb) = 533 5337 l = 6251 62 lb < 7245 lb — NG [Npn = (0.65)(19,463 65) 463 63 lb) = 12,651 lb > 7245 lb — OK 7KHLQEROWVDWLV¿HVUHTXLUHGVWHHOVWUHQJWKEXWGRHVQRWVDWLV] WKH FUHWHEUHDNRXWVWUHQJWKEXWGRHVQRWVDWLV] XWVWUHQJWKEXWGRHVQRWVDWLV] wKH FUHWHEUHDNRXWVWUHQJWKEXWGRHVQRWVDWLV] wKH fuh fi = 6.5 in. Step 7: Final check for 5/8 in. diameter x 6.5 in. long
post-installed torque-controlled anchor 17.4.1.2 []Nsa []Ase, N futa•Nua (17.4.1.2) From Table above (5/8 in. diameter anchor): []Nsa = (0.75)(16,950 lb) = 12,713 lb  $\approx$  12,700 lb Check that design steel strength is greater than required strength. Step 8: Concrete breakout 17.4.2.1  $\varphi$ N cb = ANc  $\psi$  ed , N  $\psi$  c , N  $\psi$  cp , N N b 11,582 lb [Ncb = 11,600 lb [Ncb = 11,600 lb ]Ncb = 11,600 lb > Nua = 7245 lb American Concrete Institute - Copyrighted © Material - www.concrete.org OK Anchorage Step 6: Pullout 17.4.3 )RUSRVWLQVWDOOHGDQFKRUVDQFKRUSUHTXDOL¿FDWLRQ testing data are used: 230 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 9: Pullout 17.4.3.1 [Npn zc, PNp•Nua (17.4.3.1) [Ncb = (0.65)(14,254 lb) = 9265 lb  $\approx$  9200 lb Np is provided on manufacturer's evaluation report—refer to values under Step 6. [Ncb = 9200 lb > Nua = 7245 lb Step 10: Concrete splitting anchor, geometry must satisfy: • hef" ha - 4 in. Step 11: Summary ACI 318 17.4.1.2 17.4.2.1 17.4.2.1 17.4.3.2 Failure mode Steel Concrete breakout Concrete pullout OK hef = 6.5 in. < 2/3(12) = 8 in. hef = 6.5 in. < 12 - 4 = 8 in. OK Design strength, lb []Nsa 12,700 []Ncb 11,600 []Npn 9200 Controls design? No No Yes Choose a 5/8 in. ASTM F1554 Grade 55 torque-controlled expansion anchor with 6.5 in. embedment. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 231 Anchorage Example 4: Post-installed adhesive anchor in Seismic Design Category B, subjected to tension force only Indoor structural steel hangers are spaced at 17 ft and support a 6 in. diameter Sch. 40 coolant pipe. F1554 Grade 36 hanger bolts are anchored by an adhesive into a 10 in. thick normalweight reinforced concrete slab with fcg SVLVHH)LJ( The structure is assigned to Seismic force of 90 lb. Determine the required diameter and embedment depth of an adhesive anchor based on: 1. Minimum characteristic bond stress values from Table 17.4.5.2 of ACI 318; and 2. Bond strength obtained from an ACI 355.4 qualifying testing protocol (Table E.2). 7KHPD[LPXPWHPSHUDWXUHLVf]GXULQ]RSHUDWLRQ Given: Loads— 8QLWZHLJKWRILQGLDPHWHU6FKSLSH¿OOHGZLWKZDWHU w = 32 lb/ft 9HUWLFDOVHLVPLFIRUFH OE Fig. E4.1—Six inch diameter pipe supported with an adhesive an anchor. Anchorage Anchor— ASTM F1554 Grade 36; Table 1a: • futa = 58,000 psi + fya = 36,000 psi + - fcg SVL ha = 10 in. te 3a = 1.0 (normalweight concrete) zc, N = 1.0 (cracked concrete) ¿FD WHVWLQJHYHQLI LQLWLDOGHVLJQ VHVW QGVWUHQJW 3RVWLQVWDOOHGDQFKRUVQHHGTXDOL¿FDWLRQWHVWLQJHYHQLI LQLWLDOGHVLJQXVHVWKHERQGVWUHQJWKYDOXHVIURP7DEOH7KH L XWQRWGH VODELVDVVXPHGFUDFNHGZLWKVXI¿FLHQWUHLQIRUFHPHQWWRUHVWUDLQFUDFNZLGWKVEXWQRWGHWDLOHGZLWKVXSSOHPHQWDU\RUDQFKRU reinforcement. Part A: Bond stress values per ACI 318-14, Table 17.4.5.2 ACI 318-14 Discussion Step 1: Required strength 5.3.1 U = 1.4(w)(s) U = 1.2(w)(s) + 1.0E Step 2: Strength inequalities 17.2.3.4.1 The structure is assigned to SDC B and less than 20% of U is due to seismic effects; therefore, the requirements of 17.2.3.4 do not apply. 17.3.1.1 Calculation U = 1.4(32 lb/ft + 5 lb/ft)(17 ft) = 880 lb Controls U = 1.2(32 lb/ft + 5 lb/ft)(17 ft) = 880 lb Controls U =  $1.2(32 \text{$ strength must satisfy the following inequalities:  $\int \varphi N$  sa (steel strength in tension)  $| N ua \leq \langle \varphi N cb (concrete breakout) | \varphi N (bond strength) | a Nua = 880 lb Pullout and side-face blowout do not apply to adhesive anchors. American Concrete Institute – Copyrighted © Material – www.concrete.org 232 THE REINFORCED CONCRETE DESIGN$ HANDBOOK—SP-17(14) Step 3: Anchor ductility Check the ductility of the anchor material to GHWHUPLQHWKHDSSURSULDWHIDFWRULQ6WHS 2.2 & KHFNLI)\*UDGHVDWLVcHVWKH\$&, & KDSWHUGHcQLWLRQRIGXFWLOLW\ Table 1a: ASTM F1554 Grade 36 has the following properties: 14% elongation, and 30% minimum area reduction 23% elongation in 2 in. of length > 14% 40% area reduction > 30% Therefore, ASTM F1554 Grade 36 is ductile. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 233 Step 4: Calculate minimum anchor diameter and embedment depth In addition to Step 2, an adhesive anchor resisting a sustained tensile load must also satisfy Eq. (17.3.1.2): 17.3.1.2 [Nba•Nua = 880 lb (17.3.1.2) where Nba is the anchor's basic bond strength: 17.4.5.2 Nba 3aIJcrAdhef (17.4.5.2) IRUPDOZHLJKWFRQFUHWH3a = 1.0 17.3.3c(ii) 7RGHWHUPLQHDVXPHVXSSOHPHQWDOUHLQIRUFHPHVXS ment is not provided, ed, aand has a high sensitivity to installation and low reliability—Condition nd lo ability Con nB Category 3: []eak equation as a function tion of Write the bond breakout anchor diameter and effective depth (Note: ha = 10 in.). Anchorage 7KLVFKHFNHQVXUHVWKDWFUHHSGRHVQRWLQAXHQFH WKHIDLOXUHORDG:KHQVSHFL¿FDQFKRULQIRUPDWLRQ is not available during design, ACI 318-14 allows the use of conservative bond stress values in Table 17.4.5.2, assuming the conditions noted in ACI 318 DUHIROORZHG&KDUDFWHULVWLFERQGVWUHVVIJcr from Table 17.4.5.2 is 300 psi. IJcr is then multiplied by a 0.4 factor because the applied load is a sustained tension loading. DNa with 9.5 in. embedment depth. Check that bond design strength is greater than required strength. Anchor adhesive strength: = (0.55)(0.45)(1.0)(0.4)(300 lb/in.2 A LQ LQ = 886 lb  $\approx$  890 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK 234 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 5: Steel strength 17.4.1.1 The anchor's nominal steel strength is the steel tensile strength (futa) times the effective crosssectional area (Ase,N). 17.4.1.2 [Nsa = Ase,N futa (17.4.1.2) Check steel tensile strength is the steel tension strength of 1 in. anchor: [Nsa + Nua = 880 lb Table 1b, the physical properties are: da = 1 in. nt = 8 threads per in. The nominal strength of anchors in tension is represented as a function of futa rather than fya, EHFDXVHWKHPDMRULW\RIDQFKRUPDWHULDOVGRQRW H[KLELWDZHOOGH¿QHG\LHOGSRLQW Check that futa = 58,000 psi (Table 1a) is the smaller 1.9fya = 68,400 psi of 1.9fya and 125,000 psi futa < 1.9fya < 125,000 psi R17.4.1.2 Ase, N = 0.9743  $\pi$  (da - 4 |\ nt || 2 or use value provided d in Tab Table 3. 17.3.3a(i) d by strength gth of a duct Anchor governed ductile steel element: gn strength gth is greater th n required Check that design than strength. OK 2 Ase, N = 0.9743  $\pi$  (2 |\1.0 in. - | = 0.606 in. 4 8 / Nsa = (0.606 in.2)(5)(58,000 00 lb/in.2) = 35,148 lb 000 35 48 8 lb Nsa = 35,148 [] Same ((0.75)(35,148 5,148 b)) = 26,361 lb a 26,300 lb 0 lb >> Nua = 880 lb Nsa = 226,300 American Concrete Institute - Copyrighted @ Material - www.concrete.org OK CHAPTER 15-ANCHORING TO CONCRETE 235 Step 6: Bond strength ANa  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ed ,
Na  $\psi$  ed , Na  $\psi$  ed ANa is equal to ANao: 17.4.5.4 zed,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRU edge effect, the connection is a single anchor and is located far from the slab edge; therefore ca,min > cNa. cNa = 10d a τ uncr 1100 (17.4.5.1a) ANa = 1 ANao (17.4.5.1d) cNa = 10(1 in.) 1000 psi = 9.5 in. ca ,min 1100 psi 17.4.5.5 17.7.6 zcp,Na ± PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVDW cracked service load level without supplementary g; assume member reinforcement to control splitting; is cracked and not detailed.. zcp,Na = 1.0 where cac = 2hef = 2(9.5 in.) = 19 in. WKHUHIRUHz WK I ca,min in > cNaWKHUHIRUHz cp,Na = 1.0 The basic bond str strength is: )LJ(<sup>2</sup>,GHDOL]HGLQAXHQFHDUHDRIDGKHVLYHDQFKRU American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage ZKHUHIJuncr = 1000 psi (Table 17.4.5.2) Therefore, zed, Na = 1.0 236 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E4.3—Sin E4.3—Single adhesive anchor distance from edges and other anchors > 2cNNa 17.3.3c(ii) Because the load d is ssustained, per Table 17.4.5.2, duc by a reduction fac factorr equal to IJuncr must be reduced ed as Condition ondition B, Cate ory 3. 0.45, categorized Category n bbond strength is grea Check that design greater than required strength. 8953)(4.8953)(0.4)(0.45) [] b lb) = 1612 lb  $\approx$  1600 lb [Na = (0 0 lb > Nua = 880 lb [Na = 1600 American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 237 Step 7: Concrete breakout 17.4.2 [Ncb•Nua = 880 lb N cb = ANc ψ ed , N ψ c , same: 17.4.2.5a 17.4.2.6 ANc =1 ANco zed,N±PRGL¿FDWLRQIDFWRUIRUHGJHHIIHFWca,min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and not detailed. zc,N = 1.0 17.4.2.7  $zcp,N\pm PRGLiFDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors$  in uncracked concrete without supplementary g; anchor is cast-in reinforcement to control splitting; zcp,NN = 1.0 m edge, ca,min •cac. single anchors in uncracked concrete without supplementary g; anchor is cast-in reinforcement to control splitting; zcp,NN = 1.0 m edge, ca,min •cac. single anchor and far from 17.4.2.2 N b = kc  $\lambda$  a N b = (1 (17)(1.0) 0) f c'hef1.5 () 4000 psi (9.5 in)1.5 = 31, 482 lb ed anchors: ors: kc = 17 For post installed 17.3.3c(ii) JW IDFWRUXVH&RQG LRQ%) RUGHWHUPLQLQJWKH [DFWRUXVH&RQGLWLRQ%DQG Category 3. 4.2.1a): Substituting into Eq Eq. (17.4.2.1 17.4.5.1 17.4.2.1 17.3.1.1 and 17.4.5.1 Failure mode Steel strength controls. Choose a 1 in. ASTM F1554 Grade 36 post-installed adhesive anchor with minimum 9.5 in. embedment spaced at 17 ft 0 in. on center. 1RWH\$&,6HFWLRQVDQGUHTXLUHWKDWDGKHVLYHDQFKRUVLQWKLVH[DPSOHEHLQVWDOOHGE\DFHUWL¿HG person and the work be continuously inspected. American Concrete Institute Copyrighted © Material – www.concrete.org Anchorage 17.4.2.1 238 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 3DUW%%RQGVWUHVVYDOXHVSHUTXDOL¿FDWLRQWHVWLQJ7DEOH( ACI 318-14 Discussion Steps 1 through Step 3 are the same as presented above. Try 1/2 in. diameter anchor. Step 9: Steel tension Check steel tension strength of 1/2 in. anchor: 17.4.1.1 17.4.1.2 []Nsa•Nua = 880 lb []Nsa = Ase, N futa futa = 58,000 psi Ase , N = 0.9743  $\pi$  (a - 4  $\pi$  1/2 in. at = 13 threads per in. 2 2 Ase , N = 0.9743  $\pi$  (a - 4  $\pi$  1/2 in. at = 13 threads per in. 2 2 Ase , N = 0.9743  $\pi$  (a - 4  $\pi$  1/2 in. at = 13 threads per in. 2 2 Ase , N = 0.9743  $\pi$  (a - 4  $\pi$  1/2 in. at = 13 threads per in. 2 2 Ase , N = 0.9743  $\pi$  (a - 4  $\pi$  1/2 in. at = 13 threads per in. 2 2 Ase , N = 0.9743  $\pi$  (b = 0.142 in. 2  $\pi$  2 Ase , N = 0.9743  $\pi$  (b = 0.142 in. 2  $\pi$  2 Ase , N = 0.9743  $\pi$  (b = 0.142 in. 2  $\pi$  2 Ase , N = 0.9743  $\pi$  (b = 0.142 in. 2  $\pi$  2 Ase , N = 0.9743  $\pi$  (b = 0.142 in. 2  $\pi$  2 Ase , N = 0.9743  $\pi$  (b = 0.142 in. 2  $\pi$  2 Ase , N = 0.9743  $\pi$  2 Ase , N = 0.9 8236 lb For post-installed anchor without supplementary reinforcement, calculate the design steel strength.  $\Box$  [Nsa = (0.75)(8236 lb) = 6177 lb  $\simeq$  6100 lb ength is greater than Check that design steel strength required strength.  $\Box$  [Nsa = (0.75)(8236 lb) = 6177 lb  $\simeq$  6100 lb ength is greater than
Check that design steel strength required strength. OK (17.4.5.1a) ngt can n be calculated ffrom: m: Basic bond strength 17.4.5.2 Nba 3aIJcrAdahef (17.4.5.2) ZKHUHIJcr is obtained from Table E.2. Assume hef = 3.5 in. IJccr = 1030 10 psi Nba SVL ALQ LQ OE 17.4.5.4 zed, Na + PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRU edge effect, the connection is a single anchor and is located far from the slab edge; therefore ca,min > cNa. zed,Na = 1.0 17.4.5.5 zcp,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVDW cracked service load level without supplementary reinforcement to control splitting; assume member is cracked and not detailed. zcp,Na = 1.0 Substituting in Eq. (17.4.5.1a) Na = (1.0)(1.0)(1.0)(5663 lb) = 5663 lb 7.3.3c(ii) )RUGHWHUPLQLQJWKH[DFWRUXVH&RQGLWLRQ%DQG Category 3. Check that bond design strength is greater than required strength. 17.3.1.2 []  $\square$ Na = (0.45)(5663 lb) = 2548 lb  $\cong$  2500 lb  $\square$ Na = 2500 (0.45)Nba = 1400 lb > Nua,s = 880 lb American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 239 Step 11: Concrete breakout 17.4.2 [Ncb•Nua = 880 lb N cb = ANc  $\psi$  ed , N  $\psi$  c , N  $\psi$ same. 17.4.2.5 17.4.2.6 ANa =1 ANao zed, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; ca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and not detailed. zc, N = 1.0 17.4.2.7 17.4.2.7 17.4.2.6 ANa =1 ANao zed, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; ca, min > 1.5hef7KHUHIRUHWKHPRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and not detailed. zc, N = 1.0 17.4.2.7 17.4.2  $zcp,N\pm PRGLiFDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors$  in uncracked concrete without supplementary zcp,NN = 1.0 g reinforcement to control splitting. 17.4.2.2 N b = kc  $\lambda$  a (f') (h c ef) 1.5 (17.4.2.2a) N b = ((17)(1.0) 7)(1.0)
7)(1.0) 7)( lb b = 3100 lb [Ncb = 33168 lb [Ncb = 3100 lb >> Nua = 880 lb Design strength, lb [Nsa 6100 [Na 2500 [Ncb 3100 [Nsa 6100 [Na 2500 [Ncb 3100 [Nsa 6100 [Na 2500 [Ncb 3100 [Nsa 6100 [Na 2500 [Ncb 3100 [Nsa 6100 [Na 2500 [Ncb 3100 [Nsa 6100 1RWH\$&,6HFWLRQVDQGUHTXLUHWKDWDGKHVLYHDQFKRUVLQWKLVH[DPSOHEHLQVWDOOHGE\DFHUWL¿HG person and the work be continuously inspected. Step 13: Discussion Bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher than the values provided in Table 17.4.5.2. In this example, using the low bond stress values from test data are much higher test data are much higher test data. Table 17.4.5.2 results in a 1 in. diameter anchor with 9.5 in. embedment. Using bond stress value obtained from test data, however, results in a 1/2 in. diameter anchor with 3.5 in. embedment for the same load. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.4.2.1 240 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Anchorage Example 5: Cast-in headed anchor in Seismic Design Category A, subjected to shear A light gauge steel frame structure located in high wind zone area is anchored by 1/2 in. diameter cast-in hex-headed bolts spaced at 24 in. The cast-in bolts are ASTM A307 Grade 36 with 5.5 in. embedment and placed at 3 in. from the edge of a grade beam with fcg SVL)LJ( (DFKDQFKRULVVXEMHFWHGWROEODWHUDOZLQGORDGVHH)LJ( 7KHVWUXFWXUH is assigned to Seismic load. No supplemental reinforcement is detailed. Check the adequacy of cast-in headed anchor. Given: Loads— VW = 1200 lb Anchor— 1/2 in. diameter cast-in hex-headed unthreaded bolt ASTM A307 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 psi Spacing s = 24 in. o.c. Embedment depth hef = 5.5 in. Fig. E5.1—C E5.1~C E5.1~ strength 5.3.1 U = 1.0VW Discussion cussion calculation Ca 1200 lb U = 1.0 × 1200 lb = 120 U = Vua = 1200 0 lb Structure is assigned to SDC A; therefore, requirements of 17.2.3 do not apply. Step 2: Strength in shear) | Vua  $\leq$ { φVcb (concrete breakout) | φV (concrete pryout) | cp Step 3: Minimum anchor edge distance and spacing 17.7.1 Check minimum center-to-center spacing 4da. 17.7.2 20.8.1.3.1 Minimum edge distance to satisfy ACI 318: 17.7.4 Cast-in headed bolt is used to attach light gauge steel track frame construction and is unlikely to be WRUTXHGVLJQL¿FDQWO\0LQLPXPFRYHURI\$&, 14, Section 20.8.1 applies. Vua = 1200 lb smin = 4(1/2 in.) = 2 in. < 24 in. OK ca,min LQ LQ\$URYLGHG OK American Concrete.org CHAPTER 15—ANCHORING TO CONCRETE 241 Step 4: Anchor ductility Check the ductility of the anchor material to deterPLOHWKHDSSURSULDWHIDFWRULO6WHS 2.2 &KHFNLI\$\*UDGHVDWLV¿HVWKH\$&, &KDSWHUGH¿OLWLROVRIGXFWLOLW\ZKLFKDUH 7KHGH¿OLWLROVWDWHV3\$VWHHOHOHPHOWPHHWLO] the requirements of ASTM A307 shall be considered ductile steel element." 14% minimum elongation, and 30% minimum area reduction Therefore, no calculations are required. Step 5: Steel shear Nominal steel strength of a cast-in anchor is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60. Vsa = (0.6)Ase,V futa (17.5.1.2b) 9HULI\futa = 58,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa <
1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,400 psi futa < 1.9(36,000 psi) = 68,4 1.9fya < 125,000 psi OK For 1/2 in. A307 Grade 36 anchor, eitherr calculate Ase,V futa from Eq. (17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 17.5.1.2a) or obtain from Table 3. Table 3. Table 3. Table 3. Table 3. Tab  $6WUHQJWKUHGXFWLRQIDFWRU[RUGXFWLOHEROW gn strength ngth is greater th n required strength: 942 ]b) = 3212 lb \approx 3200 lb$  |V American Concrete Institute - Copyrighted © Material - www.concrete.org OK Anchorage 17.5.1.2 242 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 6: Concrete breakout A shear breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWRSURSDJDWHDZD\IURPWKHGH¿QHG point at 35 degrees both horizontally and vertically toward the edges (refer to Fig. E5.2). 7.5.2.1(a) Nominal concrete breakout shear strength of a single anchor is: Vcb = AVc ψ ed ,V ψ c ,V ψ h ,V Vb AVco (17.5.2.1a) Fig. E5.2—Idealize dshe sh shear breakout CIP anchor. 17.5.2.1 FWH IDFHDUHDUHODWH RDVKHDU AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU nchor (see Fig. E .2). AVc is breakout for a sin single anchor E5.2). UHD JURXSRIVHYHUD DQFK WKHSURMHFWHGDUHDIRUDJURXSRIVHYHUDODQFKRUV as a single gle anchor, so AVc and AVco This example has are the same. 17.5.2.6 zed, V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV loaded in shear; check if ca2•ca1 ca2 LQ LQ•ca1 = 1.5(3 in.) = 4.5 in. Assume ca2 = 6 in. lzed,V = 1.0 zc,V ±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc,V = 1.0 zh,V±PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth. ha = 24 in. > 1.5ca1 = 1.5(3 in.) = 4.5 in 17.5.2.7 17.5.2.8 AVc = 1.0 AVcoo American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.5.2.2 243 The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) ( ( ) 0.2 ) 1 Vb = | 7 | e | da |  $\lambda$  a | ( \ d a ) | 1 c'(ca1)1.5((4in.)0.2)1.50.5in.|(1.0)3000psi(3in.)(17.5.2.2a)Vb = |7||()0.5in.|(1.0)3000psi(3in.)(17.5.2.2a)Vb = |7||()0.5in.|(1.0)3000psi(3in.)(17.5.2.2b)Vb = (9)(1.0)Controls()(0.5in.)(1.5vb) = 2136lb(17.5.2.2b)Vb = (9)(1.0)Controls()(0.5vb) = 2136lb(17.5.2b)Vb = 2136lb(17.5.2b)Vb = 2136lb(17.5.2b)Vb = 2136lb(17.5.2b)Vb = 2136lb(17.5.2b)Vb = 2136lb(17.5.2b)Vb = 2136lb(17.5.2b)Vb = 2136lb(17supplementary reinforcement, Condition  $\Box Vcb = (0.7)(2136 \text{ lb}) = 1495 \text{ lb} \cong 1500 \text{ lb} = 1495 \text{ lb} \cong 1500 \text{ lb} = 2136 \text{ lb}$  American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage AVC  $\psi$  ed ,V  $\psi$  c ,V  $\psi$  h ,V Vb AVco 17.5.2.1 244 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Concrete pryout failure at the side opposite to load direction. 17.5.3.1 Nominal pryout strength for a single anchor is given by Eq. (17.5.3.1a): Vcp = kcpNcp (17.5.3.1a) Fig. E5.3-Idealized pryout CIP anchor. Nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal concrete breakout strength) Ncp = Ncb (nominall con 17.4.2.1 N cb = ANc \u03c6 ed , N \u03c6 c, N \ ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRQWKH surface as approximated by a rectangle 1.5hef = (1.5)(5.5 in.) = 8.25 in. in a direction perpendicular c ete to the shear force and the free edge of the concrete from the centerline of the anchor. ANc = (3 in. + 8.25 in.)(8.25 in. + 8.25 in.) = 186 in. 2(17.4.2.1 c) ANco = 9(5.5 in.) = 272ANC 186 in 2 = 0.68 AN co 272 in 2 17.4.2.5 17.4.2.5 17.4.2.6 17.4.2.7 zed, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; check if ca, min = 3 in. < 1.5 hef = 1.5(5.5 in.) = 8.25 in. (c)  $\psi$  ed, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.)  $\psi$  ed, N = 0.7 + 0.3 | | = 0.81 (1.5(5.5 in.) / (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.)  $\psi$  ed, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.)  $\psi$  ed, N = 0.7 + 0.3 | | = 0.81 (1.5(5.5 in.) / (17.4.2.5b) zc, N = 0.7 +
0.3 | a, min | (1.5 hef / (3 in.)  $\psi$  ed, N = 0.7 + 0.3 | | = 0.81 (1.5(5.5 in.) / (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.)  $\psi$  ed, N = 0.7 + 0.3 | | = 0.81 (1.5(5.5 in.) / (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.)  $\psi$  ed, N = 0.7 + 0.3 | | = 0.81 (1.5(5.5 in.) / (17.4.2.5b) zc, N = 0.7 + 0.3 | | = 0.81 (1.5(5.5 in.) / (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (17.4.2.5b) zc, N = 0.7 + 0.3 | a, min | (1.5 hef / (3 in.) ) | (1.5 hef / (3 in.) ) | (1.5 hef / (3 in.) ) | (1.5 hef / ±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and FUDFNLQILVFRQWUROOHGE\AH[XUDOUHLQIRUFHPHQW zc, N = 1.0 zcp, N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors in uncracked concrete without supplementary reinforcement to control splitting; anchor is cast-in zcp, N = 1.0 single anchor and far from edge, therefore: American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Basic concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Basic concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Basic concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Basic concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Basic concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Basic concrete Institute – Copyrighted © Material – With the concrete Institute – Copyrighted © Material –  $16,956 \text{ lb } 17.4.2.1 \text{ Substituting into Eq. } (17.4.2.1a): \text{Ncb} = (0.68)(0.81)(1.0)(1.0)(16,956 \text{ lb}) = 9339 \text{ lb } 17.5.3.1 \text{ Substituting into Eq. } (17.5.3.1a) \text{ to calculate the pryout strength: Vcp} = (2.0)(9339 \text{ lb}) = 18,678 \text{ lb } 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU]RYHUQHGE \text{ concrete pryout, Condition B} [] [] Vcp = (0.7)(18,678 \text{ lb}) = 13,075 \text{ lb } \cong 13,000 \text{ lb}$ 17.3.3c(i) Check that design strength is greater than required strength: Step 8: Summary ACI 318 17.5.1 17.5.2 17.5.3 17.7 Failure mode Steel Concrete pryout Concrete pryout Concrete pryout Concrete splitting te spl [Vcp = 13,000 lb > Vua = 1200 lb Design strength, lb [Vcb 1500 [Vcb 15 9: Conclusion See Examples 6 aand 7 for a comparison between the cas cast-in mechanical post-installed, and adhesive in hheaded, mechanica nc post-installed anchors. American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage 17.4.2.2 245 246 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Anchorage Example 6: Post-installed expansion anchor in Seismic Design Category A, subjected to shear A light gauge steel frame structure located in high wind zone area is connected to the foundation with 1/2 in. diameter postinstalled expansion anchors spaced at 24 in. The cast-in bolts are ASTM A307 Grade 36 with 5.5 in. embedment and placed at 3 in. from the edge of a grade beam with fcg SVL)LJ( (DFKDQFKRULVVXEMHFWHGWROEODWHUDOZLQGORDG7KH structure is assigned to Seismic load. Supplemental reinforcement is not detailed. Given: Loads— VW = 1200 lb Anchor— 1/2 in. expansion anchor bolt ASTM A307 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 psi Spacing s = 24 in. o.c. Embedment depth hef = 5.5 in. Fig. E6.1—Post-installed mechanical bolt anchoring a light gauge steel track. Concrete— fcg SVL ca1 = 3 in. 3a = 1.0 (normalweight concrete) ACI 318-14 Step 1: Required strength 5.3.1 U = 1.0VW Discussion sion Calculation Ca U = 1.0 .0 × 1200 lb = 1200 U = Vua = 1200 lb b Structure is assigned ned to SDC A; therefo therefore, seismic 2 do not apply. requirements of 17.2.3 Step 2: Strength must satisfy sfy the following inequalities: 17.3.1.1 [ $\phi$ Vsa (steel strength in shear) |  $Vua \leq \{\phi$ Vcb (concrete breakout) |  $\phi$ V (concrete pryout) | cp Vua = 1200 \text{ lb b} Step 3: Minimum anchor edge distance and spacing 17.7.1 Check minimum center-to-center spacing 6da. smin = 6(1/2 in.) = 3 in. < 24 in. 17.7.3 20.8.1.3.1 Minimum edge distance to satisfy ACI 318 and reported test data in Table E.1 (2.5 in.). ca = 3.0 in. >
ca,min = 2.5 in.; 3.0 in. (provided) 17.7.4 The post-installed expansive anchor is not likely WREHWRUTXHGVLJQL&FDQWO\0LQLPXPFRYHURI ACI 318-14 Section 20.8.1 applies. American Concrete Institute - Copyrighted © Material - www.concrete.org OK OK CHAPTER 15—ANCHORING TO CONCRETE 247 Step 4: Anchor ductility Check the steel material ductility to determine the DSSURSULDWH DFWRULQ6WHS &KHFNLI\$\*UDGHVDWLV¿HVWKH\$&, &KDSWHUGH¿QLWLRQVRIGXFWLOLW\ZKLFKDUH 14% minimum elongation, and 30% minimum area reduction Step 5: Steel shear Nominal steel strength of an expansion anchor is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60. 17.5.1.2 Vsa = (0.6)Ase,V futa 7KHGH¿QLWLRQVWDWHV<sup>3</sup>\$VWHHOHOHPHQWPHHWLQJWKH requirements of ASTM A307 shall be considered ductile steel element." Therefore, no calculations are required. (17.5.1.2b) The bolt area is obtained from Table 3: d = 0.5 in. Ase, V = 0.142 in.2 ot greater 9HULI/futa = 58,000 psi (Table 1a) is not than 1.9fya and 125,000 psi. 1.9ffyya = (1.9)(36,000 psi) = 68,400 psi futa < 1.9f 9fya < 125,000 psi futa < 1.9f 9fya < 125,000 psi OK For 1/2 in. A307 Grad Grade 36 anchor, either calculate (17.5.1.2a) a) or obtain fro from Table 3: Vsa = (0.6)(8236 17.5.1.3 erefo a A built-up groutt ppad is not provided. T Therefore, ct is not applied. 0.8 reduction factor 7.3.3a(ii) ID I G 6WUHQJWKUHGXFWLRQIDFWRU[RUGXFWLOHEROW 4942)(0.65 []b) = 3212 lb = 3200 lb []Vsa = (0.65)(4942 Check that design strength is greater than required strength is greater tha DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout A shear breakout failure is assumed to initiate DWDSRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWKH top of the beam, and to propagate away from the GH¿QHGSRLQWDWGHJUHHVERWKKRUL]RQWDOO\DQG vertically toward the edges (see Fig. E6.2). 17.5.2.1(a) Nominal concrete hear strength of a single anchor is: Vcb = AVc  $\psi$  ed ,V  $\psi$  c ,V  $\psi$  h ,V Vb AVco (17.5.2.1a) Fig E6.2 Fig. E6.2—Idealized lized shear shea breakout of an expansion and or. anchor. FWH IDFHDUHDUHODWH RDVKHDU AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU gle and hor (see Fig. E breakout for a single E6.2) AVC LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV example has a single anchor, so AVc and AVco are the same. 17.5.2.6 AVc = 1.0 AVco zed, V = 1.0 17.5.2.7 FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V ± PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth. ha > 1.5ca1 ha = 24 in. > 1.5ca1 = 1.5(3 in.) = 4.5 in. [zh, V = 1.0] American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.5.2.2 249 The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b): ((A) 0.2) Vb = |7| b | da |  $\lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \lambda a | \langle da | \rangle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da | \lambda a | \langle da$ (| 0.5 in. | (1.0) 3000 psi(3 in.) 1.5 [ ( 0.5 in. ) ]  $\mathcal{E} = \text{hef LQDQG''} \text{da} = 8(0.5) = 4 \text{ in. Controls Vb} = 2136 \text{ lb Vb} = 9\lambda a (f') (c c a 1) 1.5 (17.5.2.2b) Vb = (9)(1.0) Controls () 3000 \text{ psi} (3 \text{ in.}) 1.5 = 2561 \text{ lb Solve Eq. } (17.5.2.1a): Vcb = 17.3.3c(i) For an expansion headed bolt in concrete without [] dition B applies: [] V Vcb = (0.7)(2136 \text{ lb}) = 1495 \text{ lb} \approx 1500$ b supplementary reinforcement, Condition (17.5.2.1a) Vcb = (1.0)(1.0)(1.0)(1.0)(1.0)(1.0)(2136 lb) = 2136 lb ength is ggreater than required Check that design strength 1500 0 lb > Vua = =1200 120 2 lb [[Vcb = 15 strength: American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage AVc  $\psi$  ed ,V  $\psi$  c,V  $\psi$  h,V Vb AVco 17.5.2.1 250 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Concrete pryout strength for a single anchor is given by Eq. (17.5.3.1a): Fig. E6.3—Idealized pryout breakout with expansion (17.5.3.1a) anchor. Vcp = kcpNcp Nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal concrete breakout strength) Ncp = Ncb (nominal N cb = ANc  $\psi$  ed , N  $\psi$  c , N  $\psi$ 
c , N  $\psi$  c , N ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRQWKH surface as approximated by a rectangle 1.5hef = (1.5)(5.5 in.) = 8.25 in. + 8.25 in.)(8.25 in. + 8.25 in.)(8.25 in. + 8.25 in.) = 186 in. 2(17.4.2.1 c) $9(5.5 \text{ in.})2 = 272 \text{ in.}2 \text{ ANc } 186 \text{ in.}2 = = 0.68 \text{ ANco } 272 \text{ in.} 2 17.4.2.5 \text{ zed}, \text{N} \pm \text{PRGL} FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min < 1.5hef. Therefore, the PRGL} FDWLRQHGJHIDFWRULVFDOFXODWHGIURP(T (17.4.2.5b) ( 3 in.) <math>\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef ) (17.4.2.5b) ( 3 in.)  $\psi$  ed , N = 0.7 + 0.3 | | = 0.81 ( 1.5 (5.5 in.) / (5.5 17.4.2.6 zc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW zc,N = 1.0 17.4.2.7 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting, Concrete is assumed cracked; therefore: zcp,N = 1.0 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.4.2.2a) N b = kc  $\lambda$  a f c'hef1.5 (17.4.2.2a) N b = (17)(1.0) () 3000 psi (5.5 in.)1.5 = 12,010 ll 17.4.2.1 Substituting into Eq. (17.4.2.1a): Ncb = (0.68)(0.81)(1.0)(1.2)(1.0)(1.2)(1.0)(1.2,010 lb) = 6615 lb 17.5.3.1 Substituting into Eq. (17.5.3.1a) to calculate the pryout strength: Vcp =  $(2.0)(6615 \text{ lb}) = 13,230 \text{ lb} 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU]RYHUQHGE (0.68)(0.81)(1.0)(1.0)(1.2,010 \text{ lb}) = 9261 \text{ lb} \approx 9200 \text{ lb} Check$ that design strength is greater than required []Vcp = 9200 lb > Vua = 1200 lb strength: Step 8: Summary ACI 318 17.5.1 17.5.2 17.5.3 Failure mode Steel Concrete pryout Design? No Yes No Step 9: Conclusions ween the cast-in in hheaded,, mechanic See Examples 5 and 7 for a comparison between mechanical post-installed, and adhesive nch post-installed anchors. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.3.3c(i) 252 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) \$QFKRUDJH([DPSOH Post-installed adhesive anchor in Seismic Design Concrete.org]} and the set of Category A, subjected to shear A light gauge steel frame structure located in high wind zone area is connected to the foundation by 1/2 in. diameter postinstalled adhesive anchors are ASTM A307 Grade 36 with 5.5 in. embedment and placed at 3 in. from the edge of a grade beam with fcg SVL)LJ( (DFKDQFKRULVVXEMHFWHGWROEODWHUDOZLQGORDG7KHVWUXFture is assigned to Seismic Design Category (SDC) A with negligible seismic load. Supplemental reinforcement is not detailed. Given: Loads— VW = 1200 lb Anchor— 1/2 in. threaded anchor bolt ASTM A307 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 psi \$ spacing s =
36,000 psi \$ spacing s = 36,000 psi \$ spacing s = 36,000 psi \$ spacing s = 36,000 psi \$ spacing s = 36,000 psi \$ spacing s = 24 in. o.c. Embedment depth hef = 5.5 in. Fig. E7.1—Post-installed adhesive bolt anchoring a light gauge steel track. Concrete— fcg SVL ca1 = 3 in. 3a = 1.0 (normalweight concrete) Condition B—No supplementary reinforcement. y rein ACI 318-14 Discussion cussion Step 1: Required strength 5.3.1 U = 1.0VW Calculation Ca 1200 lb U =  $1.0 \times 1200$ b = 120 U = Vua = 1200 0 lb Structure is assigned to SDC A; therefore, requirements of 17.2.3 for anchoring do not apply. Step 2: Strength inequalities: 17.3.1.1 [ $\phi$ Vsa (steel strength in shear) |  $Vua \leq \langle \phi Vcb \rangle$  (concrete breakout) |  $\phi V$  (concrete pryout) | cp Vua = 1200 lb Step 2: Strength inequalities: 17.3.1.1 [ $\phi$ Vsa (steel strength in shear) |  $Vua \leq \langle \phi Vcb \rangle$  (concrete breakout) |  $\phi V$  (concrete pryout) | cp Vua = 1200 lb Step 2: Strength inequalities: 17.3.1.1 [ $\phi$ Vsa (steel strength in shear) |  $Vua \leq \langle \phi Vcb \rangle$  (concrete breakout) |  $\phi V$  (concrete pryout) | cp Vua = 1200 lb Step 2: Strength inequalities: 17.3.1.1 [ $\phi$ Vsa (steel strength in shear) |  $Vua \leq \langle \phi Vcb \rangle$  (concrete breakout) |  $\phi V$  (concrete pryout) | cp Vua = 1200 lb Step 2: Strength inequalities: 17.3.1.1 [ $\phi Vsa \rangle$  (steel strength in shear) |  $Vua \leq \langle \phi Vcb \rangle$  (concrete pryout) | cp Vua = 1200 lb Step 2: Strength inequalities: 17.3.1.1 [ $\phi Vsa \rangle$  (steel strength inequalities: 17.3.1.1 [ 3: Minimum anchor edge distance and spacing 17.7.1 Splitting failure check. Check minimum center-tocenter spacing 6da. smin = 6(1/2 in.) = 3 in. < 24 in. 17.7.3 20.8.1.3.1 Minimum edge distance to satisfy the greater of ACI 318 Section 20.8.1 and test data in Table E.2 (5.5da): Note: If ca1 < 6da, (17.7.3), then use use manufacturer data. ca = 3.0 in. > ca,min = 5.5(1/2 in.) = 2.75 in. 3.0 in. provided OK 17.7.4 The anchor in this example is unlikely to be torqued VLJQL¿FDQWO\0LQLPXPFRYHURI\$&,6HFWLRQ 20.8.1 applies. ca,min LQ•LQPLQLPXPFRYHU American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 253 Step 4: Anchor ductility Check the ductility of steel material ductility to GHWHUPLQHWKHDSSURSULDWH[DFWRULQ6WHS 2.2 &KHFNLI\$\*UDGHVDWLV¿HVWKH\$&, &KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKLV 7KH\$&,GH¿QLWLRQVWDWHV<sup>3</sup>\$VWHHOHOHPHQWPHHWing the requirements of ASTM A307 shall be considered ductile steel element." 14% minimum elongation, and 30% minimum area reduction Therefore, no calculations are required. Step 5: Steel strength (futa) times the bolt area (Ase,V), multiplied by 0.60. 17.5.1.2 Vsa = (0.6)Ase,V futa (17.5.1.2a) For 1/2 in. A307 Grade 36 anchor, either calculate Ase,V futa from Eq. (17.5.1.2a) or obtain from Table 3. 17.5.1.3 d is no A built-up grout pad not provided. Therefore, a tor is not applied. 0.8 reduction factor 17.3.3a(ii) LRQ RU[RUGXFWLOHEROW gn strength: da = design than required st 0.5 in. Table 3: Vsa = (0.6)(8236 lb) = 4942 lb 942 lb = 3212 lb = 3200 lb [VXPH\*UDGH&EROW9HULI\futa = 58,000 psi 1.9fya = (1.9)(36,000 psi) = 68,400 psi (Table 1a) is not greater than 1.9fya and 125,000 psi. futa < 1.9fya < 125,000 psi OK 254 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout A shear breakout A shear breakout A shear breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWKHWRSRI WKHEHDPDQGWRSURSDJDWHDZD\IURPWKHGH¿QHG point at 35 degrees both horizontally and vertically toward the edges (see Fig. E7.2). 17.5.2.1(a) Nominal concrete breakout shear strength of a single anchor is: Vcb = AVc  $\psi$  ed ,V  $\psi$  c ,V  $\psi$  h ,V Vb AVco (17.5.2.1a) 2 1a) FWHG HDUHDUHODWH VKHDU AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ing anchor chor (see Fig. E 2). AVc breakout for a single E7.2). D UDJURXSRIDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV7 LVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGLµFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc and AVco are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGLµFWHGDUHDIRUDJURXSRIDQFKRUV7KLV ncho so AVcc are the example has a sin single anchor, same. 17.5.2.6 zed,V±PRGLµF adhesive hor. anchor. AVc = 1.0 AVcoo ca2 LQ LQ • ca1 = 1.5(3 in.) = 4.5 in. zed, V = 1.0 Therefore, 17.5.2.7 zc, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth. Check if ha > 1.5ca1 zc, V = 1.0 ha = 24 in. > 1.5ca1 = 1.5(3 in.) = 4.5 in. zh, V = 1.0 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and  $(17.5.2.2b) (17.5.2.2b) (17.5.2.2b) = 7 = 4 \text{ in. Controls Vb} = 2136 \text{ lb Vb} = 9\lambda a (17.5.2.2a) Vb = 7 = 7 (10.5 \text{ in.}) (17.5.2.2a) Vb = 7 = 7 (10.5 \text{ in.}) (17.5.2.2b) Vb = (17.5.2.2a) Vb = 7 (17.5.2.2a) Vb = 7 (17.5.2.2a) Vb = 7
(17.5.2.2a) Vb = 7 (17.5.2.2b) Vb = (17.5.2.2a) Vb = 7 (17.5.2.2a) Vb = 7 (17.5.2.2b) Vb = (17.5.2b) Vb =$ Concrete Institute - Copyrighted © Material - www.concrete.org OK Anchorage 17.5.2.2 255 256 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Concrete pryout failure at the side opposite to load direction. 17.5.3.1 Nominal pryout strength for a single anchor is given by Eq. (17.5.3.1a): Vcp = kcpNcp Fig. E7.3—Idealized pryout for an adhesive anchor. (17.5.3.1a) Nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal concrete breakout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal concrete breakout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one or two times the anchor nominal pryout strength based on effective depth hef is approximately one N N b ANco (17.4.2.1a) 17.4.5.1 Na = ANa y ed , Na y cp , Na N ba ANao (17.4.5.1a) 7.4.5.1a) 17.4.2.1 q Calculate concrete breakout strength using Eq. (17.4.2.1a): ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRQWKH surface as approximated by a rectangle 1.5hef = (1.5)(5.5 in.) = 8.25 in. in a direction perpendicular to the shear force and the free edge of the concrete from the centerline of the anchor (Fig. E7.3) ANc = (3 in. + 8.25 in.)(8.25 in.)(8.25 in. + 8.25 in.)(8.25 in. + 8.25 in.)(8.21.5hef7KHUHIRUHWKHPRGL¿FDWLRQ edge factor is calculated from Eq. (17.4.2.5b) ( c )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.6) (17.4.2.5b) zc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL¿FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW ( 3 in. )  $\psi$  ed , N = 0.7 + 0.3 | a,min | ( 1.5hef / 17.4.2.5b) zc,N±PRGL‹FDWLRQDW service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE\ÀH[XUDOUHL 0.7 + 0.3 | = 0.81 \ 1.5 (5.5 in.) / zc,N = 1.0 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.4.2.2 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting Basic concrete breakout strength in tension (Eq. (17.4.5.1a)): Na = ANa y ed , Na y cp , Na N ba ANao (17.4.5.1a) KHDGKHVLYHDQFKRU ANaLVWKHLQAXHQFHDUHDRIWKHDGKHVLYHDQFKRU WLOLQHDUDDHDWKDWSURMHFWV fr the centerline of outward a distance cNa from the adhesive anchor hor and a is calculated from Eq. (17.4.5.1d). cNa = 10d a  $\tau$  uncrer 11000 2240 ppsi 10(0.5 in.)) = 7.14 in. (17.4.5.1d) cNa = (2 × 7.14 in.) = 204 in. ANa = (3 in. + 7.14 in.)(7.14 in. + 7.14 in.) = 145 in.2 ANa 145 in.2 = = 101 a \tau 0.71 ANao 204 in.2 American Concrete Institute - Copyrighted © Material - www.concrete.org 258 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 17.4.5.4 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRU edge effect, for ca,min < cNa: ψ ed , Na = 0.7 + 0.3 17.4.5.5 17.4.5.2 ca , min cNa (17.4.5.4b) zcp,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVDW cracked service load level without supplementary reinforcement to control splitting; assume member is cracked concrete is calculated from Eq. (17.4.5.2) Nba 3aIJcrAdahef (17.4.5.2) ZKHUHIJcr is obtained from test reports (see Table E.2) or from very conservative values listed in Table 17.4.5.2: 7DEOH(IJcr = 1030 psi) = 8858 lb (0.71)(0.826)(1.0)(8858 lb) = 5195 lb (controls) Na = (0.71)(0.8 17.4.5.1 ension (Eq. (17.4.5.1a))), Na: Nominal bond strength in tension 17.5.3.1 te pry Therefore, concrete pryout strength is calculated by 3.1a): (2 195 lb) = 110,390 lb substituting into Eq. (17.5.3.1a): Vcp = ((2.0)(5195 17.3.3c(i) RU DQDGKHVLYHDQF RUJRYHUQHG 5HGXFWLRQIDFWRU[RUDQDGKHVLYHDQFKRUJRYHUQHG ou [by]concrete pryout: ] cpp = ((0.65)(10,390 0,390 lb) = 6753 lb = 6700 lb [V Check
strength is greater than required strength: Step 8: Summary ACI 318 17.5.1 17.5.2 17.5.3 Failure mode Steel Concrete breakout Concrete breakout Concrete breakout Concrete breakout strength, lb []Vsa 3200 []Vcb 1500 []Vcp 6700 OK Controls design? No Yes No Step 9: Conclusion Concrete breakout strength controls in Examples 5 (cast-in-place headed anchor), 6 (post-installed expansion anchor), and 7 (post-installed adhesive anchor). Notice that all anchor types have the same breakout strength. Failure mode Steel Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 5 3200 1500 13,000 Design strength, lb Example 6 3200 1500 6900 ([DPSOH 3200 1500 6700 Concrete pryout Example 6 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPSOH 3200 1500 6900 ([DPS strength of post-installed expansion anchor and post-installed adhesive anchor is approximately one-half of the cast-in headed anchor. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 259 Anchorage Example 8: Cast-in hex-headed anchor in Seismic Design Category A, resisting tension and shear forces A building's roof trusses are connected to a 12 in. wide by 16 in. deep, normalweight reinforced concrete beam, with fcg psi. The beam has not been detailed with anchor or supplemental reinforcement. The building is assigned to Seismic Design Category (SDC) A. The trusses are spaced at 32 in. centers. Each connection has a single, cast-in, 5/8 in. diameter, hex-headed ASTM A307 Grade 36 bolt, embedded 6.5 in., and located 4 in. from the beam edge. The end of the beam resists a wind force of 2160 lb uplift and 1440 lb lateral, service gravity roof dead load of 250 lb and roof live load of 250 lb, and a seismic force of 250 lb tension and 150 lb shear. Check the adequacy of end anchor bolt. Given: Loads— WV =  $\pm 250$  lb vertical seismic EH =  $\pm 150$  lb lateral seismic Anchorage Anchors— 5/8 in. cast-in hex-headed anchor ASTM A307 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 psi • fya = 36,000 psi Embedment depth hef = 6.5 in. lle to force) Edge distance ca1 = 4 in. (perpendicular Concrete-fcg SVL 3a = 1.0 (normalweight concrete) h = 16 in., b = 12 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 9HUWLFDO U = 1.4D U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6(Lr) + (0.5W) U = 1.2D + 1.0W + 0.5(Lr) U = 1.2D + 1.0E + 1.0(L) U = 0.9D + 1.0W U = 0.9D + 1.0E Fig. E8.1-Roof 8.1-RoQRWUHVXOWLQWKHDQFKRUUHVLVWLQJVLJQL¿FDQWYHUWLFDO tension. U = 1.2(250 lb) - 1.6(250 lb) - 0.5(2160 lb) = -1735 lb Negative indicates tension force (upward). Equations (5. 3.1a), (5.3.1b), (5.3.1e), and (5.3.1g) do QRWUHVXOWLQWKHDQFKRUUHVLVWLQJVLJQL $\mathcal{E}$ FDQWVKHDUIRUFH Equations (5.3.1d) and (5.3.1d) and (5.3.1d) and (5.3.1d) U = 1.0(1440 lb) = 1440 lb Structure is assigned to SDC A; therefore, seismic Required tension strength is 1935 lb requirements of 17.2.3 do not apply. Required shear strength is 1440 lb American Concrete Institute - Copyrighted © Material - www.concrete.org 260 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 2: Strength in equalities: 17.6.3  $\phi$ N sa (steel strength in tension)  $\phi$ N (concrete breakout) c N ua  $\leq$  N  $\phi$  pn (anchor design strengths must satisfy the following inequalities: 17.6.3  $\phi$ N sa (steel strength in tension)  $\phi$ N (concrete breakout) c N ua  $\leq$  N  $\phi$  pn (anchor design strengths must satisfy the following inequalities: 17.6.3  $\phi$ N sa (steel strength in tension)  $\phi$ N (concrete breakout) c N ua  $\leq$  N  $\phi$  pn (anchor design strengths must satisfy the following inequalities: 17.6.3  $\phi$ N sa (steel strength in tension)  $\phi$ N (concrete breakout) c N ua  $\leq$  N  $\phi$  pn (anchor design strengths must satisfy the following inequalities: 17.6.3  $\phi$ N sa (steel strength in tension)  $\phi$ N (concrete breakout) c N ua  $\leq$  N  $\phi$  pn (anchor design strengths must satisfy the following inequalities: 17.6.3  $\phi$ N sa (steel strength in tension)  $\phi$ N (concrete breakout)  $\phi$ N (concrete pullout)  $| | \phi N$  sb (side-face blowout) and Nua = 1935 lb  $[\phi Vsa (steel strength in shear) | Vua \le \frac{1}{2} \phi Vcb (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | <math>\phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | <math>\phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V (concrete breakout) | \phi V
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Step 4: Edge distance and spacing anchor requirements 17.7.1 Check minimum center-to-center spacing 4da. 17.7.2 20.8.1.3.1 Minimum edge distance to satisfy ACI 318: 17.7.4 Cast-in headed bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL¿FDQWO\0LQLmum cover of ACI 318 states "A steel element meeting the requireme mentss of ASTM A307 sha shall be considered ductile steel ent." of ACI 318 states "A steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL and bol calcula Therefore, no calculations are required. smin = 4(5/8 in.) = 2.5 in. < 32 in. OK ca = 4 in. (provided) > 2.0 in. (required) American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 261 Tension strength Step 5: Steel tension 17.4.1.2 Nominal steel strength is the steel tensile strength (futa) times the bolt area (Ase,N). Nsa = Ase,N futa The anchor area is obtained from Table 3: da = 0.625 in. Ase,N = 0.226 in.2 ACI 318 limits futa = 58,000 psi (Table 1a) to the smaller of 1.9 fya and 125,000 psi (Table 1a) to the smaller of 1.9 fya and strength Nsa is obtained from Table 3: Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength is greater than required strength is www.concrete.org 262 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout 17.4.2.1a Nominal concrete breakout strength of single anchor in tension: N cb = 17.4.2.3 ANc  $\psi$  ed , N  $\psi$  c , N  $\psi$  cp , N N b ANco (17.4.2.1a) The edge anchor with an hef = 6.5 in. is located less than 1.5hef from three edges. Therefore, the breakout strength calculations are based on a VKDOORZHU¿FWLWLRXVHIIHFWLYHGHSWKh limited by the largest of the three edge distances less than 1.5 1.5hef. /1.5 = 5.33 in in. < 6.5 6 in. hoef = 8/1.5 HLV 8 in /DUIHVWHGIHGLVWDOFHLV in. 'H¿OLWLRO WHG WHIDLOXUHDUH ANcLVWKHSURMHFWHGFROFUHWHIDLOXUHDUHDRID im y edge distances (see E8.2): ANNc =  $(4 \text{ in.} + 1.5 \times 5.33 \text{ in.})(6 \text{ in.} + 1.5 \times 5.33 \text{ in.})(6 \text{ in.} + 1.5 \times 5.33 \text{ in.})(6 \text{ in.} + 1.5 \times 5.33 \text{ in.})$ ANCOLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID single anchor, not limited by edge distance or spacing (Fig. E8.3): 17.4.2.1 ANco = (9hgef)2(17.4.2.1c) ANco = 9(5.33 in.)2 = 256 in.2 ANco 168 in.2 = 0.66 ANco 256 in.2 The anchor is located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore UHGXFHGWKURXJKWKHIDFWRUzed, N. Fig. E8.3—Idealized tension breakout. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE zed, N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min < 1.5hqef, therefore: ψ ed , N = 0.7 + 0.3  $17.4.2.6\ 17.4.2.2\ ca$ , min 1.5hef () 4 in. (17.4.2.5b)  $\psi$  ed, N = 0.7 + 0.3 = 0.85 (1.5(5.33\ in.) )/ zc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load levels; assume member is cracked and not detailed. zc,N = 1.0 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors in uncracked concrete without supplementary reinforcement to control splitting. zcp, N = 1.0 To determine basic concrete breakout strength, either calculate from Eq. (17.4.2.2a) Nb = kc  $\lambda$  a f c'hef' 1.5 (17.4.2.2a) rom test results The constant, kc, was determined from ZDVDGMXVWHGIRU LQXQFUDFNHGFRQFUHWHDQGZDVDGMXVWHGIRU cracked concrete. 17.4.2.1(a) = 9090 lb [] [Ncb OE OE OE SOE Check that design strength is greater than required []N Ncb = 6300 lb > Nua
= 1935 lb strength: American Concrete Institute - Copyrighted © Material - www.concrete.org OK 264 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Pullout 17.4.3.1 Nominal pullout strength is calculated from Eq. (17.4.3.1): Npn zc, PNp 17.4.3.6 17.4.3.4 (17.4.3.1) zc, P = factor to modify pullout strength of anchor in cracked concrete crushing occurs under the anchor head. To determine basic pullout strength, either calculate by Eq. (17.4.3.4) Np = (8) Abref to Np = (8) (0.454 in.2) (3000 psi) = 10,896 lb 17.3.3c(ii) where Abre is obtained from Table 1b: From Table 1b: From Table 1b: From Table 1b: From Table 1b: Abre = 0.454 in.2 For a cast-in headed bolt without supplementary lies: reinforcement, Condition B applies:  $\square$   $\square$  Npn OE OE§OE ength is greater than required Check that design strength 76 0 lb b > Nua = 11935 lb OK  $\square$  Npn = 7600 strength: Step 8: Side-face blowout 17.4.4.1 For a single headed ade anchor chor with deep eembedment bedment 6.5 in. side-face blowout does n. < 2.5 × 4.0 0 in.; therefore, ther close to an edgee ((hef > 2.5ca1), concrete crete side-face blowout does n. < 2.5 × 4.0 0 in.; therefore, ther close to an edgee ((hef > 2.5ca1), concrete crete side-face blowout does n. < 2.5 × 4.0 0 in.; therefore, ther close to an edgee ((hef > 2.5ca1), concrete crete side-face blowout does n. < 2.5 × 4.0 0 in.; therefore, there 17.4.2 17.4.3 17.4.4 Failure mode Steel Concrete breakout Concrete pullout Side-face blowout Design strength, lb []Nsb — Ra Ratio = Nua []Nn 0.2 0.31 0.25 NA American Concrete Institute - Copyrighted © Material - www.concrete.org Controls design? No Yes No — CHAPTER 15—ANCHORING TO CONCRETE 265 Shear strength Step 10: Steel shear Nominal steel strength of a headed bolt is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60. 17.5.1.2 Vsa = (0.6)Ase,V futa (17.5.1.2b) The bolt area is obtained from Table 3: Ase,V = 0.226 in.2 9HULI\futa = 58,000 psi (Table 1a) is not greater than 1.9 fya and 125,000 psi. 1.9 fya = (1.9) (36,000 psi) = 68,400 psi futa < 1.9 fya < 125,000 psi OK For 5/8 in. A307 Grade 36 anchor, either calculate Ase, V futa from Eq. 17.5.1.2a) or obtain from Table 3. Table 3: Ase, V futa = 13,108 lb Vsa = (0.6)(13,108 lb) = 7865 lb 17.3.3a(ii) Because a sill plate is located between the steel clip angle and the concrete beam, a 0.8 reduction is applied. Strength reduction factor for ductile bolt: Vsa = (0.8)(7865 lb) = 6290 lb 6290)(0.65) [b) = 4090 lb  $\approx$  4100 lb  $\neg$ Vamerican Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage 17.5.1.3 266 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Concrete breakout A shear breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWRSURSDJDWHDZD\IURPWKHGH¿QHG point at 35 degrees both horizontally and vertically toward the edges (see Fig. E8.4). 17.5.2.1 Nominal concrete breakout shear strength of a single anchor is: Vcb = AVc ψ ed ,V ψ e ,V ψ h ,V Vb AVco (17.5.2.1a) Fig. E8 E8.4—Idealized 4 alized shear shea breakout. 'H¿QLWLRQ HFW UIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor (see Fig. E 4). breakout for a single E8.4). 'H¿QLWLRQ AVcLVWKHSURMHFWHGDUHDIRUDJURXSRIVHYHUDO anchors. This example has a single anchor, so AVc and AVco are the same. 17.5.2.6 zed,  $V \pm PRGL$ ; fourther case is a single anchor, so AVc and AVco are the same. 17.5.2.6 zed,  $V \pm PRGL$ ; fourther case is a single anchor. This example has a single has a single has a single has a single has a single has a single has a single has a single has a single has Therefore zed, V = 1.0 17.5.2.7 zc, V ±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V ±PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth. ha > 1.5ca1 ha = 16 in. > 1.5ca1 LQWKHUHIRUHzh, V = 1.0 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) ( (A ) 0.2 ) Vb = |7| e | da |  $\lambda$  a | ( \ d a / |) f c'(ca1) 1.5 (  $\mathcal{E} = hef$ For a cast-in headed bolt in concrete without supplementary reinforcement, Condition B applies:  $||Vcb| = (0.7)(3675 \text{ lb}) = 2573 \text{ lb} \approx 2500 \text{ lb} ||Vength is greater than required Check that design strength strength is greater than required Check that design strength strength is greater than required Check that design strength$ iis relate to load direction. related to the anchor's ten tension breakout streng strength Ncb and h hef. embedment depth 17.5.3.1 17.3.3c(i) Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp =
(2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ LQ kcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU ] [Vcp = kcp Ncp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ kcp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ kcp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ kcp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ kcp Anchorage 17.5.2.2 267 OK (17.5.3.1a) From Step p 6 above, Ncb = 9090 lb hef LQ kcp Anchorage 17.5.2.2 400 lb hef LQ kcp Anchorage 17.5.2.2 400 lb hef LQ kcp Anchorage 17.5.2.2 400 lb hef LQ kcp Anchorage 17.5.2.2 400 lb hef LQ kcp Anchorage 17.5.2.2 400 lb hef LQ kcp Anchorage 17.5.2.2 400 lb hef LQ  $(0.7)(18,180 \text{ lb}) = 12,730 \text{ lb} \approx 12,700 \text{ lb}$  Check that design strength is greater than required []Vcp = 12,700 \text{ lb} > Vua = 1440 \text{ lb} OK strength: Step 13: Splitting failure 17.7.4 Splitting failure 17.7. Shear force summary ACI 318 17.5.1 17.5.2 17.5.3 17.7 Failure mode Steel Concrete breakout Concrete pryout Splitting Design strength, lb [Vsa 4100 [Vcb 2500 [Vpn 12,700 NA - Ratio = Vua[Vn 0.35 0.58 0.11 - American Concrete Institute - Copyrighted © Material - www.concrete.org Controls design? No Yes No - 268 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 15: Shear and tension interaction 17.6.1 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.4 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.4 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.4 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if Nua[Nn" 1935 lb/6300 lb = 0.31 >1935 lb 1440 lb + = 0.89 > 1.2 6300 lb 2500 lb OK Step 16: Conclusion See Examples 9, 10, and 11 for a comparison between cast-in headed, cast Anchorage Example 9: Cast-in hooked anchor in Seismic Design Category A, resisting tension and shear forces A building's roof trusses are connected to a 12 in. wide by 16 in. deep, normalweight, reinforced concrete beam, with fcg psi. The beam has not been detailed with anchor or supplemental reinforcement. The building is assigned to Seismic Design Category (SDC) A. The trusses are spaced at 32 in. centers. Each connection has a single, cast-in, 5/8 in. diameter, hooked ASTM A307 Grade 36 bolt, embedded 6.5 in., with a 2.8 in. long hook, and located 4 in. from beam edge. The end connection's anchor bolt is 6 in. from the beam end (Fig. E9.1). The bolt at the end of the beam resists a wind force of 2160 lb uplift and 1440 lb lateral, a service gravity roof dead load of 250 lb, and a seismic force of 250 lb tension and 150 lb shear. Check the adequacy of end anchor bolt. Given: Anchorage Loads—  $WV = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 250$  lb dead gravity Lr = 250 lb live gravity  $EV = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2160$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH = \pm 2100$  lb vertical wind  $WH =
\pm 2100$  lb vertical wind  $WH = \pm 2$  $\pm 250$  lb vertical seismic EH =  $\pm 150$  lb lateral seismic Anchors - 5/8 in. cast-in hex-headed anchorr ASTM A307 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 psi • fya = 36,000 psi • fya = 36,000 psi • fya = 1.0 (normalweight A007 Grade 36; Table 1a: • futa = 58,000 psi • fya = 36,000 concrete) h = 16 in., b = 12 in. Fig. E9.1—Roof 1—Roof tru truss supported by concrete learn. American Concrete Institute - Copyrighted © Material - www.concrete.org 270 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Required strength 9.2.1 9HUWLFDO: U = 1.4D U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6(Lr) + (0.5W) U = 1.2D + 1.0W + 0.5(Lr) U = 1.2D + 1.0E + 1.0(L) U = 0.9D + 1.0E Calculation (5.3.1a) (5.3.1c) (5.3.10.5(2160 lb) = -380 lb U = 1.2(250 lb) - 1.0(2160 lb) + 0.5(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = -17/DWHUDO: U = 1.0W (5.3.1d) U = 1.0(1440 lb) = 1440 lb structure is assigned to SDC A; therefore, seismic Required tension strength is 1935 lb sh strengt is 1935 lb sh strength is 1935 lb sh s ee strength ngth in tension) | N ua  $\leq \langle \varphi N cb (concrete on breakout) | \varphi N (anchor ullout) | pn ch pullout) and Nua = 193 1935 lb | \varphi V (concrete pryout) | cp 17.6.3 Vua = 1440 lb The interaction of tensile and shear forces must also satisfy the following inequality: N ua Vua +$ 1.2 qN n qVn (17.6.3) Step 3: Anchor ductility &KHFNWKHDQFKRUVWHHOGXFWLOLW\WRGHWHUPLQHWKH[factor in tension and shear. 2.2 Check if A307 Grade 36 meets the ACI 318 states "A steel element meeting the requirements of ASTM A307 shall be considered ductile steel element." Therefore, no calculations are required. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE Step 4: Edge distance and spacing anchor requirements 17.7.1 Check minimum center-to-center spacing 4da. 17.7.2 20.8.1.3.1 Minimum edge distance to satisfy ACI 318: smin = 4(5/8 in.) = 2.5 in. < 32 in. 271 OK ca = 4 in. (provided) > 2.0 in. (required) OK 17.7.4 Cast-in headed bolt is used to attach a steel angle DQGLVXQOLNHO\WREHWRUTXHGVLJQL¿FDQWO\0LQLmum cover of ACI 318 Section 7.7 applies. Tension strength Step 5: Steel tension Nominal steel
strength is the steel tensile strength (futa) times the bolt area (Ase,N). 17.3.3a(i) Nsa = Ase,N futa (17.4.1.2) The anchor area is obtained from Table 3: da = 0.625 in. Ase,N = 0.226 in.2 able 1a) to the ACI 318 limits futa = 58,000 psi (Table 00 psi, smaller of 1.9 fya and 125,000 1.9 f 9 fya = (1.9)(36,000 psi) = 68,400 psi 9 fya < 1125,000 psi 0K futa < 1.9 f For a 5/8 Concrete Institute - Copyrighted © Material - www.concrete.org OK Anchorage 17.4.1.2 272 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 6: Concrete breakout 17.4.2.1a Nominal concrete breakout strength of single anchor with an hef = 6.5 in. is located less than 1.5hef from three edges. Therefore, the breakout strength calculations are based on a shalORZHU¿FWLWLRXVHIIHFWLYHGHSWKhoef, limited by 1.5 (see Fig. E9.2). Fig. E9.2—Cast-in anchor with three edge distances less than 1.5 hef. /15 = 5.33 in in. < 6.5 6. in. hgef = 8/15 HLV 8 in. in /DUJHVWHGJHGLVWDQFHLV 'H¿QLWLRQ WHG WHIDLOXUHDUH ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID im y edge distance see Fig. single anchor, limited by distances (see E9.2):  $5 \times 5.33$  iin.) (6 in. + 1.5  $\times 5.33$  iin.) (6 in. + 1.5  $\times 5.33$  iin.) (6 in. + 1.5  $\times 5.33$  iin.) (6 in. + 1.5  $\times 5.33$  iin.) (6 in. + 1.5  $\times 5.33$  iin.) (6 in. + 1.5  $\times 5.33$  iin.) (7  $\times 5.33$  iin.) (7  $\times 5.33$  iin.) (7  $\times 5.33$  iin.) (8 in. + 1.5  $\times 5.33$  iin.) (8 in. + 1.5  $\times 5.33$  iin.) (10  $\times 5.33$  ii 1.5hg 'H¿QLWLRQ ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID single anchor, not limited by edge distance or spacing (Fig. E9.3): 17.4.2.1 ANco = 9(hoef)2 (17.4.2.1c) ANco develop. The breakout strength is therefore reduced through the factor yed, N. Fig. E9.3—Idealized tension breakout. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.4.2.5 zed, N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min < 1.5hoef  $\psi$  ed , N = 0.7 +

0.3 17.4.2.6 17.4.2.7 17.4.2.2 273 ca,min 1.5hef () 4 in. (17.4.2.5b)  $\psi$  ed , N = 0.7 + 0.3 = 0.85 (1.5(5.33 in.) )/ zc, N = PRGLiFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load levels; assume member is cracked DQGFUDFNLQILVFRQWUROOHGEAH[XUDOFUDFNLQ] zc, N = 1.0  $zcp,N\pm PRGL cFDWLRQIDFWRUIRUSRVWLQVWDOOHG$  anchors in uncracked concrete without supplementary reinforcement to control splitting. zcp,N = 1.0 To determine basic concrete breakout strength, either calculate from Eq. (17.4.2.2a) N b = kc  $\lambda$  a f c'hef 1.5 (17.4.2.2a) N b = kc (17.4.2.2a) N b = Nominal concretee br breakout strength from Eq. (17.4.2.1a): ead bolt olt without supplementary on B applies: reinforcement, Co Condition Anchorage rom test results The constant, kc, was determined from ZDVDGMXVWHGIRU LQXQFUDFNHGFRQFUHWHDQGZDVDGMXVWHGIRU cracked concrete. Nccb = (0.6 (0.66)(0.85)(1.0)(1.0)(1.0)(1.0,210 5)(1.0)(1.0 lb) = 9090 lb [] []Ncb OE OE OE SOE Check that design strength is greater than required strength: []N Ncb = 6300 lb > Nua = 1935 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK 274 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Pullout 17.4.3.1 Nominal pullout strength is calculated from Eq. (17.4.3.1): Npn zc, PNp (17.4.3.1): Npn zc, PNp (17.4.3.1) Fig. E9.4—Concrete bearing at hook. 17.4.3.5 Check if hook tail length conforms to ACI 318 limits: 3da''eh'' da zc, P = 1.0 (3)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.875 in. < (4.5)(0.625 in.) = 1.87 2.81 in. OK To determine basic pullout ullout strength, s either calcu3.5) late by Eq. (17.4.3.5) Np = (0.9) (4000 psi)(2.8 in in.)(0.625 in.) = 6300 lb .5) Np = (0.9) fcgehda ((17.4.3.5) 17.3.3c(ii) du factor for a cast n ho The strength reduction cast-in hooked pl ntary reinforcem nt bolt without supplementary reinforcement []&RQGLWLRQ% LV[] Check that concrete pullout design strength is greater than required strength: Step 8: Side-face blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.4.1 For a single headed anchor with deep embedment close to an edge (hef > 2.5ca1), concrete blowout 17.4.1 For a single headed anchor Concrete pullout [] DESOE []Npn OE []Npn OE []Npn = 4400 lb > Nua = 1935 lb OK 6.5 in. < 2.5 × 4.0 in.; therefore, side-face blowout does not apply. Design strength, lb []Nsa 9800 []Npn 4400 Ratio = Nua[]Nn 0.2 0.31 0.44 American Concrete Institute – Copyrighted © Material – www.concrete.org Controls design? No No Yes CHAPTER 15– ANCHORING TO CONCRETE 275 Shear strength of a headed bolt is the steel tensile strength of a headed bolt is the steel tensile strength (futa) times the bolt area is obtained from Table 3: Ase, V = 0.226 in.2 9HULI/futa = 58,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi. 1.9fya = (1.9)(36,000 psi) = 68,400 psi futa < 1.9fya < 125,000 psi OK For 5/8 in. A307 Grade 36 anchor, either calculate Ase,V futa from Eq. (17.5.1.3 17.3.3a(ii) Because a sill plate is located between the steel clip angle and the concrete beam, a 0.8 reduction is applied. Strength reduction factor for ductile bolt: Table 3: Ase, V futa = 13,108 lb Vsa = (0.6)(13,108 lb) = 7865 lb Vsa = (0.8)(7865 lb) = 6290 lb 6290)(0.65) [b) = 4090 lb  $\cong$  4100 lb  $\subseteq$  Va = 14 1400 lb  $\ge$  Va = 14 1400 lb  $\ge$  Va = (0.6)(13,108 lb) = 7865 lb Vsa = (0.6)(13,108 lb) = 6290 lb 6290)(0.65) [b) = 6290 lb 6290)(0.65) OK Anchorage 17.5.1.2 276 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Concrete breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWKHWRSRI WKHEHDPDQGWRSURSDJDWHDZD\IURPWKHGH¿QHG point at 35 degrees both horizontally and vertically toward the edges (see Fig. E9.5). Nominal concrete breakout shear strength of a single anchor is: 17.5.2.1 N cb = ANc ψ ed , N ψ c , N ψ cp , N N b ANco (17.4.2.1a) Fig. E9 E9.5—Idealized 5 ealized she shear breakout. 'H¿QLWLRQ HFW UIDFHDUHDUHODWH WRDV AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU nchor (see Fig. E .5). breakout for a sin single anchor E9.5). 'H¿QLWLRQ GD IRUDJURXSRI HUDO AVcLVWKHSURMHFWHGDUHDIRUDJURXSRIVHYHUDO anchors. This example has a single anchor, so AVc and AVco are the same. 17.5.2.6 zed, V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0.0 AVco ca1 = 4 in. and ca2 = 6 in., therefore zed, V = 1.0 17.5.2.7 zc, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with
supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL assume member is cracked and not detailed with supplementary reinforcement. zc, V = 1.0 17.5.2.8 zh, V±PRGL assume member is cracked assume member is cracked assume member is cracked assume m 16 in. > 1.5ca1 = 6 in., therefore zh, V = 1.0 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.5.2.2 The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) ( (A) 0.2 ) Vb = |7| e | da |  $\lambda a | \langle d a / | \rangle (d a / | ) (f 5 in 2)$  $lb \psi ed$ ,  $V \psi c$ ,  $V \psi h$ , V Vb For a cast-in headed bolt in concrete without supplementary reinforcement, Condition B applies:  $[] Vcb = (0.7)(3672 lb) = 2570 lb \approx 2500 lb > Vuua = 14 Step 12$ : Pryout ors can pryout yout on the side si opposite pposite pposite Short stiff anchors n. The pryout strength iis related elated to load direction. h Ncb and to the anchor's te tension breakout streng strength pth hef. Fro ve, Ncb = 9090 lb embedment depth From Ste Step 6 above, (17.5.3.1a) 5 3 1a) Vcp = (2 (2.0)(9090 0 0 lb)) = 18,180 1 lb hef LQ•LQ1kcp = 2.0 17.3.3c(i) 5HGXFWLROIDFWRUIRUFDVWLODOFKRU [] Vcp = (0.7)(18,180 lb) = 12,700 lb > Vua = 1440 lb OK strength: Step 13: Splitting reader than required [] Vcp = (0.7)(18,180 lb) = 12,700 lb > Vua = 12,700 lbdesign? No Yes No - 278 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 15: Shear and tension interaction 17.6.1 Check if Vua[Vn" 1440 lb/2500 lb = 0.58 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 1935 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 lb/4400 lb = 0.44 > 0.2 Check if Nua[Nn" Therefore, shear-only design strength is not permitted. 17.6.2 lb/4400 lb = 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0.44 > 0 permitted. 17.6.3 Check if N ua , g  $\varphi$ N n + Vua , g  $\varphi$ Vn  $\leq$  1.2 1935 lb 1440 lb + = 1.02  $\leq$  1.2 4400 lb 2500 lb OK Step 16: Conclusion See Examples 8, 10, and 11 for a comparison between cast-in headed, post-installed mechanical, and post-installed mechanical, and post-installed mechanical, and post-installed mechanical and post-CHAPTER 15—ANCHORING TO CONCRETE 279 Anchorage Example 10: Post-installed expansion anchor in Seismic Design Category A, resisting tension and shear forces A building's roof trusses are connected to a 12 in. wide by 16 in. deep, normalweight, reinforced concrete beam, with fcq psi. The beam has not been detailed with anchor or supplemental reinforcement. The building is assigned to Seismic Design Category (SDC) A. The trusses are spaced at 32 in. centers. Each connection has a post-installed 5/8 in. diameter expansion, ASTM F1554 Grade 55 bolt, embedded 6.5 in., and located 4 in. from the beam edge. The end connection's anchor bolt is 6 in. from the beam end (Fig. E10.1). The bolt at the end of the beam resists a wind force of 2160 lb uplift and 1440 lb lateral, a service gravity roof dead load of 250 lb tension and 150 lb shear. Check the adequacy of edge bolt. Given: Loads— WV = ±2160 lb vertical wind WH = ±1440 lb lateral wind D = 250 lb dead gravity Lr = 250 lb live gravity  $EV = \pm 250$  lb vertical seismic  $EH = \pm 150$  lb lateral seismic Concrete—fcg SVL 3a = 1.0 (normalweight concrete) h = 16 in., b = 12 in. ACI 318-14 Discussion Step 1: Required strength 5.3.1 9HUWLFDO U = 1.2D + 1.0E + 1.0(L) U = 1.2D + 1.0E + 1.0(L) U = 1.2D + 1.0E + 1.0(L) U = 1.2D + 1.0E + 1.0(L) U = 1.2D + 1.0E + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D + 1.0(L) U = 1.2D +
1.0(L) U = 1.2D + 1.0(L) 0.9D + 1.0W U = 0.9D + 1.0E Anchorage Anchors— or 5/8 in. post-installed expansion anchor ASTM F1554 Grade 55; Table 1a: • futa = 75,000 psi Embedment depth hef = 6.5 in. lle to force) orce Edge distance ca1 = 4 in. (parallel en ar to force) Edge distance ca2 = 6 in. (perpendicular Fig. E10.1—Roof Fig 1 R truss russ supp supported by concrete beam. Calculation (5.3.1a) (5.3.1b) (5.3.1c 0.9(250 lb) - 1.0(2160 lb) = -1935 lb Negative indicates tension force (upward). Equations (5.3.1a), (5.3.1c), A; therefore, seismic requirements of 17.2.3 do not apply. Required tension strength is 1935 lb Required tension strength is 1940 lb American Concrete Institute – Copyrighted © Material – www.concrete.org 280 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 2: Strength inequalities 17.3.1.1 The anchor design strengths must satisfy the following inequalities:  $\left[\varphi N \text{ sa} (\text{steel strength in tension} \mid Vua \leq \langle \varphi N \text{ cb} (\text{concrete breakout} \mid \varphi N (\text{anchor pullout}) \mid \varphi N (\text{anchor pullout}$ inequality: N ua Vua + < 1.2  $\phi$ N n  $\phi$ Vn (17.6.3) Step 3: Anchor ductility Check the anchor steel ductility lity to determine the DFWRULQWHQVLRQDQGVKHDU VKHDU 2.2 Grad 55 meets the ACI 318 Check if F1554 Grade OLW\ WLRQ &KDSWHUGXFWLOLW\GH2QLWLRQ le 1a STM F1554 Grade 55 has the following Table 1a: ASTM perti properties: n, and 14% elongation, ductio 30% minimum ar area reduction. % elo 21% elongation in 2 in. oof length, and 30 % are tion. 30% area reduction. Therefore, F1554 G Grade 55 is ductile. Step 4: Edge distance and spacing anchor requirements 17.7.1 Check minimum center-to-center spacing 6da. 17.7.3 20.8.1.3.1 17.74.4 20.8.1.3.1 Minimum edge distance to satisfy the
greater of ACI 318 Section 20.8.1 and test data in Table E.1 (3.0 in.): Post-installed expansion anchor is used to a steel DQJOHDQGLVXQOLNHO\WREHWRUTXHGVLJQL¿FDQWO\ Minimum cover of ACI 318 Section 20.8.1 applies. 6(0.625 in.) = 3.75 in. < 32 in. smin = 6(0 OK ca = 4 in. (provided) > ca,min = 3.0 in. (Table E.1) > 2.0 in. (Code) OK ca = 4 in. > 2 in. minimum cover American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 281 Tension strength is the steel tensile strength (futa) times the bolt area (Ase,N). 17.3.3a(i) Nsa = Ase,N futa (17.4.1.2) The anchor area is obtained from Table 3: da = 5/8 in. Ase, N = 0.226 in. Ase, N = 0.226 in. ASTM F1554 Grade 55 anchor, the nominal tensile strength Nsa is obtained from Table 3: Table 3: Nsa = 16,950 psi (Table 1a) to the smaller of 1.9 fya and 125,000 psi (Table 1a) to the smaller of 1.9 fya an lb Use reduction factor for ductile anchors: Check that steel design strength is greater than required strength: Concrete Institute - Copyrighted @ Material - www.concrete.org 282 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout 17.4.2.1(a) Nominal concrete breakout strength of single anchor in tension: N cb = 17.4.2.3 ANc  $\psi$  ed , N  $\psi$  c , N  $\psi$  cp , N N b ANco (17.4.2.1a) The edge anchor with an Kgef = 6.5 in. is located less than 1.5hef from three edges. Therefore, the breakout strength calculations are based on a shallower, ¿FWLWLRXVHIIHFWLYHGHSWKKHIOLPLWHGE\WKHODUJHVW of the three edge ed distances less than 1.5hef. in. < 66.5 in. Kgef = 8/15 = 5.33 in HLVLQ /DUJHVWHGJHGLVWDQFHLVLQ 'H¿QLWLRQ WHG WHIDLOXUHDUH VLQJOH ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIDVLQJOH g. E10.2): anchor, limited by edge distances (see F Fig.  $5 \times 5.33$  in.) (6 in. + 1.5 × 5.33 in.) (7 in. + 1.5 × 5.33 in.) ( distance or spacing (Fig. E10.3): 17.4.2.1 ANco = 9(Kgef)2 (17.4.2.1c) ANco = 9(5.33 in.)2 = 256 in.2 ANc 168 in.2 = 0.66 ANco 256 in.2 Fig. E10.3—Actual and idealized concrete failure prism based on the 35-degree failure plane. American Concrete failure plane. Am CONCRETE 17.4.2.5 283 zed,N±PRGL¿FDWLRQIDFWRUIRUHGJHHIIHFW7KH anchor is located close to the edge, without enough space for a complete breakout strength is therefore reduced through WKHIDFWRUzed,N. For an edge distance ca,min < 1.5Kgef 17.4.2.6 ca, min 1.5hef (17.4.2.5b)  $zc,N\pm PRGL&FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW$  service load levels; assume member is cracked and FUDFNLQJLVFRQWUROOHGE () 4 in.  $\psi$  ed , N = 0.7 + 0.3 | = 0.85 ( 1.5(5.33 in.) |) zc,N = 1.0 17.4.2.7  $zcp,N\pm PRGL&FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors$  in uncracked concrete withou supplementary reinforcement to control splitting. zcp,N = 1.0 17.4.2.2 out strength, To determine basic concrete breakout 17.4.2.2a) N b = kc λ a f c'heff 1.55 (17.4.2.2a) etermined from test st re reThe constant, kc, was determined NHG FUHWHDQGZHUHD XVWH VXOWVLQXQFUDFNHGFRQFUHWHDQGZHUHDGMXVWHGIRU e. cracked concrete breakout strength from Eq. (17.4.2.1a): N cb = 17.3.3c(ii) N b = (1 (17)(1.0) .0) 0) ANC  $\psi$  ed , N  $\psi$  c, N  $\psi$ 
c, N  $\psi$  c, N (1.0)(1.0)(1.475) = 6437 lb For post-installed expansion without supplementary reinforcement, Condition B applies: Nu = 0.7 + 0.3 284 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Pullout strength is calculated from Eq. (17.4.3.1); Npn zc, PNp 17.4.3.6 (17.4.3.1) zc, P = 1.0 The basic pullout strength Np for a post-installed expansion anchor is related to the load at which concrete crushing occurs under the anchor head. The basic pullout strength is obtained from test data. Refer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUUVREWDLQHGIURPWHVWGDWD Refer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUUVREWDLQHGIURPWHVWGDWD Refer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUUVREWDLQHGIURPWHVWGDWD Refer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUUVREWDLQHGIURPWHVWGDWD Refer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVGDWD REfer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVGDWD REfer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVGDWD REfer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVGDWD REfer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVGDWD REfer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVGDWD REfer to Table E.1: 7.3.3c(ii) 7KHUHGXFWLRQIDFWRUVG = Ncb•Nua 4100 lb > 1935 lb Failure mode ode Steel el Concrete br breakout ut ut Concretee pullout Npn z,c,PNp = 14,254 lb [] []Npn OE OE§OE []N Npn = 9200 lb > Nua = 1935 lb Design strength, lb []Nsa 12,700 0 []N Ncb 4100 4 0 Npn []N 9200 0 Ratio = Nua[]Nn 0.1 0.15 0.47 0.21 OK Controls design? No Yes No OK Shear strength Step 9: Anchor shear strength Nominal steel strength of a headed bolt is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60. 17.5.1.2 Vsa = (0.6)Ase,V futa (17.5.2.1b) The bolt area is obtained from Table 3: 9HULI/futa = 75,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi. For 5/8 in. F1554 Grade 55 anchor, either calculate Ase,V futa from Eq. (17.5.2.1a) or obtain from Table 3. da = 5/8 in. and nt = 11 Ase,V = 0.226 in.2 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1.9)(55,000 psi) = 104,500 psi futa < 1.9fya = (1 applied. Vsa = (0.8)(10,170 lb) = 8136 lb 17.3.3a(ii) Strength reduction factor for ductile bolt: Check that design strength is greater than required strength is gr CONCRETE 285 Step 10: Concrete breakout A shear breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWKHWRSRI WKHEHDPDQGWRSURSDJDWHDZD\IURPWKHGH¿QHG point at 35 degrees both horizontally and vertically toward the edges (see Fig. E10.4). Nominal concrete breakout shear strength of a single anchor is: Vcb = AVc  $\psi$  ed ,V  $\psi$  c,V  $\psi$  h,V Vb AVco (17.5.2.1a) Fig. E1 E10.4—Shear —Shear breakout of expansion anchor. 'H¿QLWLRQ HFW UIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ing anchor nchor (see Fig. E 0.4). breakout for a single E10.4). 'H¿QLWLRQ AVcLVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV This example has a single anchor, so AVc and AVco are the same. 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV loaded in shear; check if ca2•ca1 17.5.2.7 zc,V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. 17.5.2.8 AVc = 1.0 zc, V = 1.0 zc, V = 1.0 zc, V = 1.0 zc, V = 1.0
zc, V = 1.0 zc, V = Material – www.concrete.org Anchorage 17.5.2.1 286 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 17.5.2.2 The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) ( (A) 0.2 ) Vb = | 7 | e | da |  $\lambda$  a | ( \ d a / | / 1.5 f c'(ca1) ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | e | da |  $\lambda$  a | ( \ d a / | / 1.5 f c'(ca1) ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2.2a) Vb = | 7 | ( (17.5.2a) Vb = | 7 lb For an expansion anchor,  $\mathcal{E} = hef$  and  $\mathcal{E}e''$  da Vb = 9 $\lambda$  a 17.5.2.1 f c'(ca1) 1.5 AVc  $\psi$  ed, V  $\psi$  c, V  $\psi$  h, V Vb AVco For an expansion anchor without supplementary reinforcement, Condition B applies: Vcp = kcpNcp  $\mathcal{E} = hef = 6.5$  in. and 8da = 8(0.6.25) = 5 in. (controls)  $\Box Vcb = (0.7)(3675 \text{ lb}) = 2570 \text{ lb} \cong 2500 \text{ lb} = 2570 \text{ lb} \cong 2500 \text{ lb} = 2570 \text{ lb} \cong 2500 \text{ lb} > Vua = 1440 1 \text{ lb} \Box Vcbb$ = 2 OK (17.5.3.1a) From Step ep 6, Ncb = 6437 lb Vcp = (2.0)(6437 lb) = 12,874 lb 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU Check that design strength is greater than required strength: Step 12: Splitting failure 17.7.4 Splitting fa si Short stiff anchors side opposite n. The pryout strength iis relate to load direction. related to the anchor's ten tension breakout streng strength Ncb and h hef. embedment depth 17.5.3.1 0.2 Solve Eq. (17.5.2.1a): Vcb = 17.3.3c(i) 5 in. | 0.625 in. | Failure mode Steel Concrete breakout Concrete pryout Splitting [] []Vcp = (0.7)(12,874 lb) = 9012 lb = 9000 lb [Vcp = 9000 lb > Vua = 1440 lb OK 6(0.625) = 3.75 in. < 4.0; therefore, splitting is precluded. Design strength, lb [Vsa 5300 [Vcb 2500 [Vcp 9000 NA — Ratio = Vua[Vn 0.27 0.58 0.16 — American Concrete Institute - Copyrighted © Material - www.concrete.org Controls design? No Yes No — CHAPTER 15—ANCHORING TO CONCRETE Step 14: Shear and tension interaction 17.6.1 Check if Vua[Nn" 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.3 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb + =  $1.05 \leq 1.2 \text{ }$  4100 lb 2500 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.3 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.3 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.4 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.3 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.4 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.3 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.4 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.4 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.4 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n  $\varphi$ Vn 1935 lb/4100 lb = 0.47 > 0.2 Therefore, full tension design strength is not permitted. 17.6.4 N ua Vua +  $\leq 1.2 \text{ }$   $\varphi$ N n 
$\varphi$ N n  $\varphi$ N n  $\varphi$ N n  $\varphi$ N n  $\varphi$ N n  $\varphi$ Anchorage Step 15: Conclusion See Examples 8, 9, and 11 for a comparison between cast-in headed, post-installed adhesive anchors. American Concrete Institute – Copyrighted © Material – www.concrete.org 288 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Anchorage Example 11: Post-installed adhesive anchor in Seismic Design Category A, resisting tension and shear forces A building's roof trusses are connected to a 12 in. wide by 16 in. deep, normalweight, reinforced concrete beam, with fcg 3000 psi. The beam has not been detailed with anchor or supplemental reinforcement. The building is assigned to Seismic Design Category (SDC) A. The trusses are spaced at 32 in. centers. Each connection has an adhesive anchor, with a 5/8 in. diameter ASTM F1554 Grade 36 threaded rod, embedded 6.5 in., and located 4 in. from the beam edge. The end connection's anchor bolt is 6 in. from the beam edge. The end connection's anchor bolt is 6 in. from the beam edge. 1440 lb lateral, a service gravity roof dead load of 250 lb and roof live load of 250 lb, and a seismic force of 250 lb tension and 150 lb shear. Check the adequacy of end anchor. Given: Loads—  $WV = \pm 250$  lb tension and 150 lb shear. Check the adequacy of end anchor. seismic Anchors— 5/8 in. post-installed expansion anch anchor ASTM F1554 Grade 36; Table 1a 1a: • futa = 58,000 psi • fya = 36, in., b = 12 in. Fig. E11.1—Roof .1—Roof tr truss supported by reinforced concrete beam. Condition B: No supplemental reinforcement. American Concrete.org CHAPTER 15—ANCHORING TO CONCRETE ACI 318-14 Discussion Step 1: Required strength 9HUWLFDO 5.3.1 U = 1.4D U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6(Lr) + (0.5W) U = 1.2D + 1.0W + 0.5(Lr) U = 1.2D + 1.0E + 1.0(L) U = 0.9D + 1.0E + 1.0(L) U = 0+ 1.6(250 lb) - 0.5(2160 lb) = -380 lb U = 1.2(250 lb) - 1.0(2160 lb) + 0.5(250 lb) = -1735 lb U = 0.9(250 lb) - 1.0(2160 lb) = -1935 lb V = 0.9(250 lb) - 1.0(2160 lb) = -1735 lb U = 0.9(250 lb) - 1.0(2160 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb V = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb V = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -1735 lb U = 0.9(250 lb) = -10(2160 lb) = same shear. (5.3.1d) U = 1.0(1440 lb) = 1440 lb Structure is assigned to SDC A; therefore, seismic Required tension strength is 1935 lb required shear strength is 1440 lb Step 2: Strength inequalities engths m 17.3.1.1 The anchor design strengths must satisfy the ties: following inequalities:  $\int \phi N$  sa (steel eel strength gth in tension) | N ua  $\leq \frac{1}{9}$  (concrete con breakout) |  $\phi N$  (anchor pullout) ullout) | pn nc Nua = 193 1935 lb and ( $\phi Vsa$  (steel strength in shear) | Vua  $\leq \frac{1}{9}$  (concrete breakout) |  $\phi V$  (concrete breakout) |  $\phi V$  (concrete breakout) |  $\phi V$  (concrete breakout) |  $\phi Vsa$  (steel strength in shear) | Vua  $\leq \frac{1}{9}$  (concrete breakout) |  $\phi V sa$  (steel strength in shear) |  $\nabla Vaa = 1440$  lb The interaction of tensile and shear forces must also satisfy the following inequality: 17.6.3 N ua Vua +  $\leq 1.2$ When when the state of F1554 Grade 36 has the following properties of F1554 Grade 55 meet the state and shear. 2.2 Check if the properties of F1554 Grade 36 has the following properties: 14% elongation, and 30% minimum area reduction. 23% elongation in 2 in. of length > 14%, and 40% area reduction > 30% Therefore, F1554 Grade 36 is ductile. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage /DWHUDO U = 1.0W 290 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 4: Edge distance and anchor spacing requirements 17.7.1 Check minimum center-to-center spacing 6da. 17.7.3 20.8.1.3.1 Minimum edge distances for post-installed DQFKRUVLVWKHJUHDWHURIVSHFL $\dot{c}$ HGFRYHUUHTXLUHments (2 in.), twice the maximum aggregate size, and 6da: smin = 6(0.625 in.) = 3.75 in. < 3.2 in. OK ca = 4 in. (provided) > ca,min 6(0.625 in.) = 3.75 in. > 2.0 in (Code) OK 17.7.4 20.8.1.3.1 A post-installed adhesive anchor, when used to fasten a steel angle, is unlikely to be torqued VLJQL&FDQWO\0LQLPXPFRYHURI\$&,6HFWLRQ 20.8.1 applies. ca = 4 in. > 2 in. minimum cover Tension strength Step 5: Steel tension Nominal steel strength is the steel tensile strength (futa) times the bolt area (Ase,N).  $17.4.1.2\ 17.3.3a(i)\ (17.4.1.2)\ Nsa = Ase,N\ futa\ The\ anchor\ area\ is\ obtained\ from\ Table\ 3:\ da = 5/8\ in.\ Ase,N = 0.226\ in.2\ ACI\ 318\ limits\ futa = 58,000\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya =
(1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ (Table\ 1a)\ to\ the\ nd\ 1.9fya = (1.9)(36,000\ psi\ si)) = 68,400\ psi\ si)$ 5/8 in. ASTM h Nsa is obtained from rom nominal tensile st strength Table 3: ,108 lb Table www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 291 Step 6: Concrete breakout 17.4.2.1 Nominal concrete breakout strength of a single anchor with an hef = 6.5 in. is located less than 1.5hef from three edges. Therefore, the breakout strength of a single anchor with an hef = 6.5 in. e distances tha 1.5hef. ORZHU¿FWLWLRXVHIIHFWLYHGHSWKKgefef, limited by the less than largest of the three edge distances divided by 1.5 (see Fig. E11.2). 8/1.5 = 5.33 33 in. < 6. 6.5 in. Kgef = 8/1 WDQF Q /DUJHVWHGJHGLVWDQFHLVLQ FWH FUHWHIDLOXUHDUHD RIDVL ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIDVLQJOH F g. E11.2): E1 anchor, limited by edge distances (see Fig. 5 × 55.33 33 in.) (6 in. + 1.5 × 5.33 in.) ANc = (4 iin. + 1.5 n.2 ANc = 168 in. 5Kgef) ANc = (ca1 + 1.5Kgef)(ca2 + 11.5Kg 'H¿QLWLRQ) ANCOLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID single anchor, not limited by edge distance or spacing (Fig. E11.3): ANCo = 9(Kgef)2 (17.4.2.1c) ANCo = 9(5.33 in.)2 = 256 in.2 ANC 168 in.2 = 0.66 AN co 256 in.2 Fig. E11.3—Idealized breakout. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage Three-edge condition: ca1 = 4 in. ca2 = 6 in. ca3 = 8 in. 292 17.4.2.5 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) zed, N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect. The anchor is located close to the edge, without enough space for a complete breakout strength is therefore UHGXFHGWKURXJKWKHIDFWRUzed, N. For an edge distance ca, min < 1.5Kgef  $\psi$  ed = 0.7 + 0.3 ca , min 1.5hef (17.4.2.6 zc, N ± PRGL2FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ zc, N = 1.0 at service load levels; assume member is cracked DQGFUDFNLQILVFRQWUROOHGE\ÀH[XUDOUHLQIRUFHPHQW 17.4.2.7 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors in uncracked concrete without supplementary reinforcement to control splitting zcp,N = 1.0 To determine basic concrete breakout strength, either calculate from Eq. (17.4.2.2a)) 17.4.2.2 N b = kc λ a f c'hef 1.5 (17.4.2.2a) N b = (17)(1.0) 17)(1 0) () 330 3000 ps ppsi (5.33 in.)1.5 = 11,475 lb The constant, kc, was determined from test results QFU QGZDVDGMXVWHGIRU te. cracked concrete. 17.4.2.1 te breakout kout strength fro m Eq. Nominal concrete from (17.4.2.1a): 17.3.3c(ii) For post-installed concrete without supplementary reinforcement, Condition B, Category 1 applies: Ncb = (0 (0.66)(0.85)(1.0)(1.0)(1.475) 6 85)(1 0)( = 6437 lb ] [Ncb = 4100 lb Check that design strength is greater than required strength: [Ncb = 4100 lb > Nua = 1935 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 293 Step 7: Bond strength For post-installed adhesive anchors, data from DQFKRUSUHTXDOL¿FDWLRQWHVWLQJPXVWEHXVHG (17.4.5.1a) Na = 'H¿QLWLRQ ANaoLVWKHSURMHFWHGLQÀXHQFHDUHDRIWKHDGKHVLYH anchor with an edge distance equal to or greater than cNa: ANao can be calculated from Eq. (17.4.5.1c) cNa = 10d a tuncr 1100 ANao = (2cNa)2 'H¿QLWLRQ )LJ(<sup>2</sup>,QÀXHQFHDUHDRIDQDGKHVLYHDQFKRU IJuncr is obtained from test data (Table E.2): 10(0 625 iin.) (17.4.5.1d) (17 4 5 1d) cNNa = 10(0.625 2100 psi = 8.64 in. 1100psi (17.4.5.1c) .5.1c) ANao = (2 ((2 (8.64 in. 1100 psi = 8.64 in. 1100 in.))2 = 29 299 in.2 FW AXHQFHDUHDRIDQ DGKHV ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIDQDGKHVLYH mated as a rectili ear aarea anchor, and is app approximated rectilinear ZD GLVWDQFHccNa fro from the WKDWSURMHFWVRXWZDUGDGLVWDQFHc dh e anchor, but iis limited mited centerline of the adhesive by the edge distance (Fig. E11.4).  $cNa = 88.64\ 64\ in. < 8\ in\ in.\ (controls)\ (co\ nd\ ca2 = 6\ in.\ ca1 = 4\ in.\ and\ ANa = (cNa + ca2)\ in + 4\ in.)(8.64\ in. + 6\ in.) = 176\ in.2$  For post-installed anchors, data from anchor SUHTXDOL¿FDWLRQWHVWLQJVKRXOGEHXVHG7DEOH( (test results): IJcr = 950\ psi\ Note: Data listed in Table 17.4.5.2 of ACI 318 Chapter 17 yield very conservative results. 17.4.5.2 The basic bond strength is obtained from Eq. (17.4.5.2): Nba 3aIJcrAdahef (17.4.5.2): Nba 3aIJcrAdahe THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 17.4.5.4 zed, Na = 0.7 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 + 0.3 +
0.3 + 08.64 in. zcp,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUV at cracked service load level without supplementary reinforcement to control splitting; assume member is cracked and not detailed. For an adhesive anchor designed for uncracked concrete without supplementary reinforcement, a PRGL¿FDWLRQIDFWRULVDSSOLHGWRSUHFOXGHEULWWOH splitting failure. In this example, concrete is FUDFNHGZLWKVXI¿FLHQWUHLQIRUFHPHQWWRUHVWUDLQ crack widths; therefore: 17.4.5.1 zcp,Na = 1.0 17.4.5.1a): Nominal bond strength from Eq. (17.4.5.1a): Na = ANa  $\psi$  ed , Na  $\psi$  cp , N a N ba ANao (17.4.5.1a) Na = (0.5 (0.59))  $(0.84)(1.0)(12,100 \ (0.84)(1.0)(12 \ 12 \ lb) = 5997 \ lb$  The reduction fac factor is obtained from ttestt data (refer to Table E. E.2):  $\Box$   $\Box$  Nu = 1935 lb OK to 6(0.625) = 3.75 in. < 4.0; therefore, splitting is the anchor, and if the edge distance is less than 6da. precluded. Step 9: Tension force summary ACI 318 17.4.1 17.4.2 17.4.5 Failure mode Steel Concrete breakout Adhesive bond Design strength, lb []Nsa 9800 []Ncb 4100 []Na 3900 Ratio = Nua[]Nn 0.2 0.47 0.5 []Nn = Ncb []Na 400 lb > 1935 lb OK American Concrete Institute – Copyrighted © Material – www.concrete.org Controls design? No No Yes CHAPTER 15—ANCHORING TO CONCRETE 295 Shear strength (futa) times the bolt area (Ase,V), multiplied by 0.60. 17.5.1.3 17.3.3a(ii) Vsa = (0.6)Ase,V futa (17.5.1.2a) The bolt area is obtained from Table 3: da = 5/8 in. and nt = 11 Ase, V = 0.226 in.2 9HULI\futa = 58,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi (Table 1a) is not greater than 1.9fya = (1.9)(36,000 psi) = 68,400 psi futa < 1.9fya < 125,000 psi OK For 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Table 3: da = 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Table 3: da = 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Table 3: da = 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Table 3: da = 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Table 3: da = 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Eq. (17.5.1.2a) or obtain from Table 3: da = 5/8 in. F1554 Grade 36 anchor, either calculate Ase, V futa from Eq. (17.5.1.2a) or obtain from Eq. (17.5.1.2a) or o Table 3: Table 3: Ase, V futa = (0.6)(13,108 lb) = 7865 lb Because a sill plate is located between the steel am, a 0.8 reduction clip angle and the concrete beam, is applied. Vsa = (0.6)(7865 lb) = 6290 lbthat design than required strength: Vssa = 41 4100 lb > Vua = 14 1440 lb []V American Concrete Institute - Copyrighted © Material - www.concrete.org OK Anchorage 17.5.1.2 296 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Concrete breakout 17.3.1.1 A shear breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHEROWFHQWHUOLQHDQGWRSURSDJDWHDZD\IURPWKHGH¿QHG point at 35 degrees both horizontally and vertically toward the edges (see Fig. E11.5). Nominal concrete breakout shear strength of a single anchor is: Vcb = AVc  $\psi$  ed ,V  $\psi$  c ,V  $\psi$  h ,V b AVco (17.5.2.1a) Fig. E11 E11.5—Idealized dealized sh shear breakout of post-installed ad sive anchor. adhesive 'H¿QLWLRQ HFW UIDFHDUHDUHODWH WRD AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor (see Fig. E 1.5). breakout for a single E11.5). 'H¿QLWLRQ AVcLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng
anchor nchor (see Fig. E 1.5). breakout for a single E11.5). 'H¿QLWLRQ AVcLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor (see Fig. E 1.5). breakout for a single E11.5). 'H¿QLWLRQ AVcLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor (see Fig. E 1.5). breakout for a single E11.5). 'H¿QLWLRQ AVcLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor nchor (see Fig. E 1.5). breakout for a single E11.5). 'H¿QLWLRQ AVcLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor nchor (see Fig. E 1.5). breakout for a single E11.5). 'H¿QLWLRQ AVcLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU ng anchor nchor nchor (see Fig. E 1.5). breakout for a single E11.5). anchor, so AVc and AVco are the same. 17.5.2.6 zed,  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 AVco ca2 = 6 in. > 1.5ca1 = 1.5(4 in.) = 6 in., therefore zed, V = 1.0 17.5.2.7 z;  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 AVco ca2 = 6 in. > 1.5ca1 = 1.5(4 in.) = 6 in., therefore zed, V = 1.0 17.5.2.7 z;  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 AVco ca2 = 6 in. > 1.5ca1 = 1.5(4 in.) = 6 in., therefore zed, V = 1.0 17.5.2.7 z;  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 AVco ca2 = 6 in. > 1.5ca1 = 1.5(4 in.) = 6 in., therefore zed, V = 1.0 17.5.2.7 z;  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 AVco ca2 = 6 in. > 1.5ca1 = 1.5(4 in.) = 6 in., therefore zed, V = 1.0 17.5.2.7 z;  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU and V = 1.0 17.5.2.7 z;  $V \pm PRGL$ ; FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 AVco ca2 = 6 in. > 1.5ca1 = 1.5(4 in.) = 6 in. detailed with supplementary reinforcement.  $z_c$ , V = 1.0 17.5.2.8  $z_h$ ,  $V \pm PRGL$ ; FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth; check if ha = 16 in. > 1.5ca1 = 1.5(4 in.) = 6 in., therefore ha > 1.5ca1 = 1.5(4 in.) = 6 in. TO CONCRETE 17.5.2.2 The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a)  $Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7|e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da|\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a|| da/|/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a||/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a||/fc'(ca1)1.5/(17.5.2.2a) Vb = |7||e|da||\lambda a||/fc'(ca1)1.5/(17.5.2a) Vb = |7||e|da||\lambda a||/fc'(ca1)1.5/(17.5.2a) Vb = |7||$ reinforcement, Condition B applies: [] [Vcb = (0.7)(3672 lb) = 2570 lb = 2500 lb Check that design strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is greater than required Vcb = 2500 lb > Vua = 1440 lb OK [V strength: Step 12: Concrete pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout an pry Short stiff anchors can pryout and pry Short stiff anchors can pryout and pry Short stiff anchors can pryout and pry Short stiff anchors can pryout and pry Short stiff anchors can pry Short stiff anchors can pry the embedment er kou strength ngth Ncbb and ad adhesive sive of anchor breakout strength Na. 17.5.3.1 Vcp = kcpNcp (17.4.2.1a) Ncb = 6437 lb 17.4.2.1 N cb = ANc  $\psi$  ed , N  $\psi$  c , N  $\psi$ 17.5.3.1 Substituting Na into Eq. (17.5.3.1a) to calculate the pryout strength: 17.3.3 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRU&RQGLWLRQ% Controls Vcp = (0.7)(12,018 lb) = 8413 lb  $\approx$  8400 lb Check that design strength is greater than required strength: []Vcp = 8400 lb > Vua = 1440 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage 17.3.3c(i) 298 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 13: Shear force summary ACI 318 17.5.1 17.5.2 17.5.3 Failure mode Steel Concrete breakout Concrete breakout Concrete breakout Concrete breakout Concrete breakout
Concrete breakout Concrete b strength, lb [Vsa 5300 [Vcb 2500 [Vcb 2500 [Vcb 2500 [Vcb 2500 [Vcb 2500 [Vcb 2500 [Vcb 2500 lb = 0.5 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if N ua + Vua ≤ 1.2  $\varphi$ N n  $\varphi$ Vn 1935 lb/3900 lb = 0.5 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if N ua + Vua ≤ 1.2  $\varphi$ N n  $\varphi$ Vn 1935 lb/3900 lb = 0.5 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.3 Check if N ua + Vua ≤ 1.2  $\varphi$ N n  $\varphi$ Vn 1935 lb/3900 lb = 0.5 > 0.2 Therefore, tension-only design strength is not permitted.  $1000 \pm 10000\pm 10000\pm$ embedment depth (hef), andd edge edge distances (cca) are identical for all four ist th pplied forc anchoring systems in Examples 8, 9, 10, and 11. All four systems can resist the applied forces. However, the individual ng systems in Examples 8, 9, 10, and 11. All four systems can resist the applied force and 11. All four systems can resist the applied forces. Steel Pullout strength/adhesive strength, lb 9800 Cast-in place headed anchor 7600 6300 6300 Cast-in-place hooked anchor 9800 4100 Post-installed expansion anchor 12,700 9200 4100 Post-installed expansion anchor 12,700 9200 4100 Post-installed expansion anchor 12,700 9200 4100 Post-installed expansion anchor 9800 6300 Cast-in place headed anchor 4100 12,700 2500 Cast-in-place hooked anchor 4100 12,700 2500 Post-installed expansion anchor 5300 9000 2500 Post-installed adhesive anchor 4100 8400 2500 The italicized values indicate the controlling load for the particular anchor system. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15-ANCHORING TO CONCRETE 299 Anchorage Example 12: Group of cast-in studs in Seismic Design Category A, resisting a concrete wall with fcg SVL7KHVWUXFWXUHLVLQ6HLVPLF Design Category (SDC) A. The steel plate is connected to the concrete wall by six 1/2 in. diameter ASTM A29 headed studs spaced at 6.5 in. centers in both directions (Fig. E12.1). The plate is resisting a service tensile force of 13,500 lb and seismic force of 13,500 lb and seismic force of 13,500 lb and seismic force of 13,500 lb and seismic force of 13,500 lb and seismic force of 13,500 lb and seismic force of 14,500 lb applied at the center of the stud group. headed stud ASTM A29; Table 1a: • futa = 65,000 psi • fya = 51,000 psi Edge distance ca1 = 3 in. Stud spacing s = 6.5 in. Embedment depth = 7 in. Stud head thickness tns = 3/8 in. in.2 Stud bearing area Abrg = 0.589 in Anchorage Loads— L = 13,500 lb (service tensile force) E = 1600 lb (seismic force) Fig. g. E12.12 -Groups E1 1- of studs resisting concentric tensile force. Concrete- fcg SVL Concrete wall thickness ha = 24 in. 3a = 1.0 (normalweight concrete) ACI 318-14 Discussion Step 1: Required strength 5.3.1 The controlling load combination is: U = 1.6L 17.2.3.1 Calculation U = (1.6)(13,500 lb) = 21,600 lb (tension) U = 21,600 lb U = Nua,g = 21,600 lb The structure is assigned to SDC A; therefore, seismic requirements of 17.2.3 do not apply. American Concrete Institute - Copyrighted © Material - www.concrete.org 300 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 2: Anchor group 17.2.1 An elastic analysis is applied for this group of anchors. Assume the following: ‡/RDGVDUHWUDQVPLWWHGWKURXJKDQDQFKRUSODWHWR individual anchors; ‡7KHDQFKRUSODWHLVLQ¿QLWHO\VWLIIDQG • Anchors are of a similar type, size, and depth. The effective stud embedment depth is given as: Fig. E12.2—Stud and plate geometry. hef 3e - tns + tpl - (1/8 in. to 3/16 in.) hef = 6.5 in. - 0.3125 in. + 0.5 in. - 0.125 in. = 6.56 in. Use hef = 6.5 in. The reduction of 1/8 in takes into account the decrease in stud length due to welding. For 1/2 in. studs, the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing: n tensio: Critical spacing = 6.56 in. Use hef = 6.5 in. The reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing = 6.56 in. Use hef = 6.5 in. The reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing = 6.56 in. The reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing = 6.56 in. Use hef = 6.5 in. The reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing = 6.56 in. Use hef = 6.5 in. The reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing = 6.56 in. Use hef = 6.5 in. The reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be To consider group effects, anchor ing: less than the critical spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing must be the reduction in length is around 1/8 in. (Table 1e). 17.2.1.1 or spacing
mus 3hef 6.5 in.  $< (3)(6.5) \ 3)(6 \ (5)) = 119.5 \ 9.5$  in. OK nside anchors hors as a group g Consider Step 3: Strength inequalities 17.3.1.1 The anchor design ig strengths ngths must satisfy satis y the following inequalities: ual N ua, g  $\int \phi N$  sa (steel strength iin tension)  $|\phi N|$  (concrete breakout)  $|cbg \leq \{|\phi N|$  pn (anchor pullout)  $|\langle\phi N|$  sb (side-face blowout) Step 4: Edge distance and spacing anchor requirements 17.7.1 Check minimum center-to-center spacing 4da. 17.7.2 20.8.1.3.1 Minimum edge distance for cast-in stude that are not torqued is based on ACI 318 Section 20.8.1 requirements: Nua, gg = 21,600 lb smin = 4(1/2 in.) = 2 in. < 6.5 in. OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (provided) > 1.5 in. (per Code) OK ca = 3.0 in. (per Code) in. (provided) > 1.5 in. (per Code) OK 17.7.4 20.8.1.3.1 Headed studs are used to anchor a steel plate to a concrete wall and cannot be torqued. Minimum cover per ACI 318. Step 5: Anchor ductility Check the ductility of the anchor steel to determine WKHDSSURSULDWH [DFWRULQWKHQH] WVWHS 2.2 Check if the properties of A29 meet the ACI 318 &KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 has the following properties: 14% elongation, and 30% minimum area reduction > 30%. Therefore, A29 anchor is ductile. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 301 Step 6: Steel tension Nominal steel strength is the steel tensile strength (futa) times the bolt area (Ase,N). 17.4.1.2 Nsa = Ase,N futa (17.4.1.2) The anchor area is obtained from Table 2: Check that futa = 65,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi. For a 1/2 in. A29 anchor, the nominal strength Nsa is obtained from Table 2: Use reduction factor for ductile anchors: Check that steel design strength is greater than required strength is greater than required strength is  $[1.9fya = 1.9(51,000 \text{ psi}) = 96,000 \text{ psi} = 96,000 \text$ 1/2 in. Ase, N = 0.196 in.2 American Concrete Institute – Copyrighted © Material – www.concrete.org 302 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Concrete breakout strength of group of anchors in tension: 17.4.2.1 N cbg = ANc ψ ec , N ψ ed , N ψ cp , N N b ANco (17.4.2.1b) Spacing s = 6.5 in. 'H¿QLWLRQ 'H¿QLWLRQ ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIWKHVWXG group, limited by edge distance (seee Fig. E12.3): The three edge anchorss with hef = 6.5 in. are located Fig. E1 E12.3—Idealized bbreakout of the stud group. m the free edge. Therefore, the less than 1.5hef from breakout area is show shown in Fig. E12.3. 5heff =  $1.5(1.5(6.5 \text{ in.}) = 9.75 \text{ in.} < 1.5\text{ h} 5 \text{ in.} + 9.9.75 \text{ in.})(9.75 \text{ in.} + 9.75 \text$ anchor, not limited bby edge distance or spacing: ANco = 9(6.5 5 in.)2 = 380 in.2 ANc shall not be taken greater than nANco where n = number of anchors in a group. 17.4.2.4 626 26 in.2 =  $1.65 \text{ ANco} 380 \text{ in.}2 \text{ zec}, N \pm PRGL2FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG eccentrically in tension; load$ is applied concentric, zec,N = 1.0 ec = 0, therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a
complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout strength is therefore: 17.4.2.5 zed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout strength effect; three anchors are located close to the edge, without enough space for a complete breakout strength effect; three anchors are located close to the edge, without enough space for a complete breakout strength effect; t  $(17.4.2.5b) \psi$  ed , N = 0.7 + 0.3 3 in. = 0.792 (1.5)(6.5 in.) 17.4.2.6 zc, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. zc, N = 1.0 17.4.2.7 zcp, N ± PRGL¿FDWLRQIDFWRUIRUFRQFUHWH supplementary zcp,N = 1.0 reinforcement to control splitting American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.4.2.2 303 To determine basic concrete breakout strength, either calculate from Eq. (17.4.2.2a) N b = kc λ a f c'hef 1.5 (17.4.2.2a) N b = (24)(1.0) () 4000 psi (6.5) in.)1.5 = 25,154 lb The constant, kc, was determined from test results LQXQFUDFNHGFRQFUHWHDQGZDVDGMXVWHGIRUFUDFNHG concrete breakout strength from Eq. (17.4.2.1b): 17.3.3.c(ii) For headed studs without supplementary reinforcement, Condition B applies: Check that concrete breakout design strength is greater than required strength: Step 8: Concrete pullout 17.4.3 Nominal pullout strength is calculated from h the greatest load Eq. (17.4.3.1) on the stud with Npn zc, PNp 17.4.3.6 Ncbg = 23,000 lb > Nua, g = 21,600 lb OK (17.4.3.1) modify pullout strength of anchorement of an nchor in zc,P - factor to mod te. zc,P = 11.0 cracked concrete. ut strength, gth, Np, is related elate to th the load The basic pullout g oof thee concrete occur at which crushing occurs due to ho head. d This is not the strength is calculated from (17.4.3.4) Np = (8) Abrg fcq Np = (8)(0.589 in.2)(4000 psi) = 18,848 lb where Abrg is obtained from Table 1e: Abrg = 0.589 in.2 17.3.3c(ii) For headed studs without supplementary reinforcement Condition B applies: Check that the pullout design strength is greater than required strength:  $\square \square \text{Np} = (0.7)(1.0)(18,848 \text{ lb}) = 13,194 \text{ lb/stud} \square \text{Np} \cong 13,200 \text{ lb/stud} \square \text{Np}$  $OE \cdot Nua, g = 21,600 \text{ lb/stud OK}$  Step 9: Concrete side-face blowout strength is calculated from Eq. (17.4.4.2) on the group of anchors along the edge. First, check if anchor group is close to edge. hef = 6.5 ca1 = 3 6.5 in. > 2.5 in. × 3 = 7.5 in. No calculation necessary. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.4.2.1b 304 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Concrete splitting failure 17.7.1 This type of failure 17.7.1 This type of failure occurs in thin members where torque is applied and the anchor has an edge distance of less than 6da. Headed studs are used to attach a steel plate and are not torqued. Minimum cover, therefore, of ACI 318 Section 20.8.1 applies. Step 11: Tension force summary ACI 318 17.4.1 17.4.2 17.4.3 17.4.4 Failure mode Steel/stud Concrete pullout/stud Concrete pullout/stud Concrete side-face blowout Design strength, lb []Nsa 9600 []Ncbg 23,000 []Npn 13,200 []Nsbg — Ratio = Nua,(g) []Nn 0.38 0.94 0.27 - Controls design? No Yes No - Step 12: Conclusion ([DPSOHVDQGUHSUHVHQWDJURXSRIFDVWLQVWXGVDQGSRVWLQVWDOOHGDGKHVLYHDQFKRUVVXEMHFWHGWRDFRQFHQWULFWHQVLOH force. The number of anchors, spacing, and embedment depth of the anchors are identical for the two systems; however, the tensile capacity of post-installed anchors is approximately 55 percent of that of cast-in-place studs. Note: Anchors are in cracked concrete. It iss assumed the light gauge track is stiff eenough not to local buckle or tear the web when load about its strong axis. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 305 Anchorage Example 13: Group of post-installed adhesive anchors in Seismic Design Category A, resisting a concentric tensile force A 1/2 x 10 x 16 in. steel plate is connected to a 24 in. reinforced concrete wall (fcg SVL ZLWKVL[LQGLDPHWHU ASTM F1554 Grade 36, post-installed, adhesive anchors in both directions (Fig. E13.1). A service tensile force of 7500 lb and a seismic force of 7500 lb is applied at the center of the stud group. The structure is in Seismic force) E = 700 lb (seismic force) Concrete—fcg thickness ha = 24 in. te) 3a = 1.0 (normalweight concrete) Fig. E13.1—Group Group of adhesive ad esiv anchors resisting a concentric tensilee for force. ACI 318-14 Discussion scus Step 1: Required strength oad combination is: 5.3.1 The controlling load U = 1.6L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb
(tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6 L Calculation Cal 7500 lb) = 12,000 lb (tensio (7500 2,000 lb U = 12,000 17.2.3.1 Structure is assigned to SDC A; therefore, seismic requirements of 17.2.3 do not apply. Step 2: Anchor group 17.2.1 An elastic analysis is applied for this group of anchors. Assume the following: ‡/RDGVDUHDSSOLHGWKURXJKDQDQFKRUSODWHWR individual anchors; \*7KHDQFKRUSODWHLVLQ¿QLWHO\VWLIIDQG • Anchors are of a similar type, size, and depth. 17.2.1.1 U = Nua,g = 12,000 lb To consider group effects, anchor spacing: • Concrete breakout in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strength in tension: Critical spacing = 3hef (3)(6.5 in.) = 19.5 in. > 6.5 in. OK • Bond strengt 2cNa cNa = 10d a tuncr 1100 (17.4.5.1d) where IJuncr = 2100 psi is based on test results and is obtained from Table E.2: cNa = 10(0.5 in.) 2240 psi = 7.13 in. 1100 psi (2)(7.13 in.) = 14.26 in. > 6.5 in. Consider anchors as a group. American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage Anchors— 1/2 in. diameter post-installed adhesive anchor ASTM F1554 Grade 36 Table 1a: • futa = 58,000 psi • fya = 36,000 psi • fya = inequalities: N ua, g  $\left[\varphi N \text{ sa} (\text{steel strength intension}\right] \leq \left\{\varphi N \text{ cbg} (\text{concrete breakout}\right] \varphi N (adhesive bond) = 12,000 \text{ lb Step 4}: Edge distance and spacing anchor requirements 17.7.1 Check minimum center-to-center spacing 6da. 17.7.3 20.8.1.3.1 Minimum edge distance to satisfy the greater of ACI 318 and test data in Table E.2: smir$ = 6(1/2 in.) = 3.0 in. < 6.5 in. OK ca LQSURYLGHG • ca,min = 6(1/2 in.) = 3 in. > 1.5 in. (Code) OK 17.7.4 20.8.1.3.1 Post-installed anchors are used to attach a steel plate to a concrete wall and are unlikely to be WRUTXHGVLJQL¿FDQWO\0LQLPXPFRYHURI\$&, Step 5: Anchor ductility Check the ductility of the anchor steel to determine area reduction. % elo on in 2 in. of length > 14%, and 23% elongation 440% % are ction > 30 area reduction 30%. refo F1554 54 Grade 36 is ductile. Therefore, Step 6: Steel tension ng is the steel t tensile strength th Nominal steel strength th Nominal steel strength th Nominal steel strength th Nominal steel strength th Nominal steel strength th Nominal steel strength (futa) times the bolt area (Ase,N). 17.4.1.2 []Nsa = []Ase,N futa (17 (17.4.1.2) 4 1 2) The anchor area is obtained at a strength th Nominal steel strength th Nominal steel strength th Nominal steel strength th Nominal steel strength th Nominal steel strength th Nominal steel strength (futa) times the bolt area (Ase,N). from Table 3: Check that futa = 58,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi. For a 1/2 in. F1554 Grade 36 anchor, the nominal tensile strength is greater than required strength: da = 1/2 in. Ase, N = 0.142 in. 2 1.9 fya = 1.9(36,000 psi) = 68,400 psi futa < 1.9 fya < 125,000 psi OK Nsa = 8236 lb/anchor [] []Nsa =  $(0.75)(8236 \text{ lb}) = 6177 \text{ lb} \cong 6100 \text{ lb} > Nua,g = 12,000 \text{ lb}/6 = 2000 \text{ lb}$  American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 307 Step 7: Concrete breakout 17.4.2.1 Nominal concrete breakout strength of a group of anchors in tension: N cbg = ANc  $\psi$  ec , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  cp , N N b ANco (17.4.2.1b) 'H¿QLWLRQ IDLOXUHDUHDRIWKHVWXG ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIWKHVWXG group, limited by edge distance (s (see Fig. E13.2): Fig. E1 E13.2—Idealized 3.2—Idealized contraction of the second strength of a group of anchors in tension: N cbg = ANc  $\psi$  ec , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  cp , N N b ANco (17.4.2.1b) 'H¿QLWLRQ IDLOXUHDUHDRIWKHVWXG ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIWKHVWXG group, limited by edge distance (s (see Fig. E13.2): Fig. E1 E13.2—Idealized 3.2—Idealized contraction of the second strength of a group of anchors in tension: N cbg = ANc  $\psi$  ec , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  cp , N N b ANco (17.4.2.1b) 'H¿QLWLRQ IDLOXUHDUHDRIWKHVWXG ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIWKHVWXG group, limited by edge distance (s (see Fig. E13.2): Fig. E1 E13.2—Idealized 3.2—Idealized contraction of the second strength of a group of anchors in tension: N cbg = ANc  $\psi$  ec , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  ed , N  $\psi$  c , N  $\psi$  ed , N  $\psi$ 
ed , N  $\psi$  ed , breakout of adhesive anchor nchor with hef = 6.5 in. are located group. The three edge anchors up. Therefore, re, the less than 1.5hef from the free edge. The caa1 = 3 iin. < 1.5h heff =  $1.5(1.5(6.5 \text{ in.}) = 9.75 \text{ in.} + 6.5 \text{$ 3.0 in. (9.75 in. + 2(6.5 in.) ANNc = (9 + 9.75 in.) 626 in.2 ANc = 62 ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIDVLQJOH cing: anchor, not limited by edge distance or spacing: 2 ANco co = (3 × 6.5 in.) = 380 in. 2 2 2 626 in in. "LQ) = 2280 in. ANc shall not be taken greater than nANco where n is the number of anchorse of an antiperiod of an antiperiod by edge distance or spacing: 2 ANco co = (3 × 6.5 in.) = 380 in. 2 2 2 626 in in. "LQ) = 2280 in. ANc shall not be taken greater than nANco where n is the number of anchorse of an antiperiod by edge distance or spacing: 2 ANco co = (3 × 6.5 in.) = 380 in. 2 2 2 626 in in. "LQ) = 2280 in. ANc shall not be taken greater than nANco where n is the number of anchorse of an antiperiod by edge distance or spacing: 2 ANco co = (3 × 6.5 in.) = 380 in. 2 2 2 626 in in. "LQ) = 2280 in. ANc shall not be taken greater than nANco where n is the number of anchorse of an antiperiod by edge distance or spacing: 2 ANco co = (3 × 6.5 in.) = 380 in. 2 2 2 626 in in. "LQ) = 2280 in. ANc shall not be taken greater than nANco where n is the number of anchorse of an antiperiod by edge distance of a space of a in a group. ANc 626 in. 2 = 1.647 AN co 380 in. 2 17.4.2.4 zec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG eccentrically. 17.4.2.5 The anchor is located close to the edge. Therefore, there is not enough space for a complete breakout SULVPWRGHYHORS(GJHPRGL¿FDWLRQIDFWRU ca1 = ca,min < 1.5hef  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (3 in.)  $\psi$  ed, N = 0.7 + 0.3 17.4.2.7 ca, min 1.5hef (17.4.2.5b) (17.4. zc, N = 1.0 zcp, N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting. zcp, N = 1.0 American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage Spacing s = 6.5 in. 308 17.4.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) To determine basic concrete breakout strength, either calculate from Eq. (17.4.2.2a) N b = kc  $\lambda$  a f c'hef 1.5 (17.4.2.2a) N b = (17)(1.0)(4000 \text{ psi})(6.5 \text{ in.})1.5 = 17,818 lb The constant, kc, was obtained from test results (Table E.2): 17.4.2.1b Nominal concrete breakout strength from Eq. (17.4.2.1b): 17.3.3.c(ii) For a post-installed adhesive Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 309 Step 8: Bond strength 17.4.5.1 Nominal bond strength is calculated from Eq. (17.4.5.1b) WHG FHDUHDRIJU IDG ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIJURXSRIDGulation of bond strength rength in hesive anchors, fo for calculation g. E13.3): ): tension (see Fig. ANa = (cNa + s2 + s2 + cNa)(ca1 + s1 + cNNa) 'H¿QLWLRQ 17.4.5.4 C cula in Step ep 2: cNNa = 7.13 in. Calculated (7.13 in. + 6.5 in. + 7.13 in.) × (3 in. + ANa = (7 6 in. + 7.13 in 6.5 in.) = 435 in.2 AnaoLVWKHSURMHFWHGEROGLOAXHOFHDUHDRIDVLOJOH adhesive anchor in tension if not limited by edge distance or spacing: 2 2 (17.4.5.1c) ANao = (2 × 7.13 in.) = 203 in. ANao
= (2 × 7.13 in.) = 203 in. ANao = (2 × 7.13 in.) = 203 n = number of anchors in a group. 453 in.2" LQ2) = 1218 in.2 zec, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL ¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na = 1.0 zed, Na ± PRGL ¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: zec, Na ± 1.0 zed, Na ± 1.0 z complete proMHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK LVWKHUHIRUHUHGXFHGWKURXJKWKHIDFWRUzed, N, which can be calculated from Eq. (17.4.5.4b). ψ ed , Na = 0.7 + 0.3 3 in. = 0.826 7.13 in. American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage N ag = 310 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) 17.4.5.5 zcp, Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVDW cracked service load level without supplementary reinforcement to control splitting; assume member is cracked and not detailed. For adhesive anchors designed for uncracked concrete without supplementary reinforcement, a modi¿FDWLRQIDFWRULVDSSOLHGWRSUHFOXGHEULWWOHVSOLWWLQ] failure. In this example, concrete is cracked with VXI¿FLHQWUHLQIRUFHPHQWWRUHVWUDLQFUDFNZLGWKV therefore: zcp, Na = 1.0 17.4.5.2 To determine basic bond strength of a single adhesive anchor in tension in cracked concrete, calculate from Eq. (17.4.5.2) Nba 3aIJcrAdahef (17.4.5.2) Nba SVL ALQ LQ OE ZKHUHIJcr = 1030 psi is obtained from test data; see Table E.2. 17.4.5.1b): N ag = 17.3.3.c(ii) ANa ANao y ec , Na y edd , NNa y cp , Na N ba (17.4.5.1b) VV LHGE\WKHPDQX FWXUHU¶V 7KHIDFWRUZDVVXSSOLHGE\WKHPDQXIDFWXUHU¶V data report. Check that design strength is greater than required strength: Step 9: Summary ACI 318 17.4.1 17.4.2 17.4.5 Failure mode Steel/stud Concrete breakout/group Adhesive bond/group 453 in.2 lb) ((1.0)(0.826)(1.0)(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (1 0))(10,516 (  $7\text{DEOH}(19,38)(65 \text{ [b)} = 12,599 \text{ lb} \approx 12,600 \text{ lb} \text{ [Nag} = (0.65)(19,383 \text{ ] ag} = 12,600 \text{ lb} \text{ Nua,g} = 12,000 \text{ lb} \text{ [Nag} 12,600 \text{ Ratio} = \text{Nua,(g)} \text{ [Nn 0.33 0.69 0.95 OK Controls design? No No Yes Step 10: Conclusion}$ ([DPSOHVDQGUHSUHVHQWD]URXSRIFDVWLQVWXGVDQGSRVWLQVWDOOHGDGKHVLYHDQFKRUVVXEMHFWHGWRDFRQFHQWULFWHQVLOH force. The number of anchors are identical for the two systems; however, the tensile capacity of post-installed anchors is approximately 56 percent of that of cast-in-place studs. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15-ANCHORING TO CONCRETE 311 Anchorage Example 14: Cast-in group of studs subjected to shear force and moment A normalweight, reinforced concrete beam with fcg SVLVXSSRUWVD:[VWHHOEHDPZHOGHGWRDQHPEHGGHGIW in. x 2 ft 3 in. x 1-1/2 in. thick A36 steel plate. The plate is connected to the concrete beam by eight 5/8 in. diameter, 8-1/2 in. long ASTM A29 headed studs spaced at 12 in. centers in both directions (Fig. E14.1). The group of studs transfers a factored moment of 40,000 ft-lb and a factored shear force of 17,000 lb (Fig. E14.1) to the beam. The seismic design force is negligible and the beam is 20 in. wide (the direction of the studs). Check the adequacy of the anchor group. Supplemental reinforcement is detailed to assist the transfer of forces. Given: Anchors— 5/8 in. cast-in studs, welded to a plate ASTM A29; Table 1a: • fy = 51,000 psi • futa = 65,000 psi (GJHGLVWDQFHWR¿UVWVWXGURZca1 = 5 in. ca2 = 5 in. irec Stud spacing s = 12 in. in both directions Stud length & beam width b = 20 in. 3a = 1.0 (normalweight concrete) American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage Required strength— MU = 40,000 ft-lb VU = 17,000 lb 312 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Step 1: Anchor group 17.2.1.1 Discussion The effective stud embedment depth is the stud length (8-1/2 in.), plus the plate thickness (1.5 in.), minus the stud head thickness (5/16 in.), minus the stud length decreased by welding (3/16 in.) (7.2.1.1 To calculate connection strength, check if the studs act as a group or act individually: Critical stud spacing = 3hef Calculation Fig. E14.2. Stud and plate geometry. hef = (8.5 in.) + (1.5 in.) - (5/16 in.) - (5/16 in.) - (3/16 in.) = 9.56 in. Use hef = 9.5 in. (3)(9.5 in.) = 28.5 in. > 12 in. r Therefore, studs resist tension as a group. For this example, the he fol following assumptions apply: KURXJK WKH SOD • /RDGV DUH DSS DSSOLHG WKURXJK SODWH WR WKH studs; em elastic and defo mati • The plate remains deformations are ignored; an and ype, size, and ddepth. h • Studs are the sam same type, 17.7 Supplementary reinforcement is detailed; therefore, 17.7.1 through 17.7.6 do not apply. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 313 Step 2: Stud forces Assume that: • • • The stud reactions are proportional to the distance from the steel beam's compression ADQJHWRH The compression force is resisted by a concrete reaction centered directly beneath WKHVWHHOEHDPFRPSUHVVLRQADQJH Concrete is cracked. There are other methods that can be used to determine stud forces, but this method is conservative. Fig. E14.3—Tension in studs. TT is the reaction of the three top studs—1, 2, and 3—such that TT = T1 = T2 = T3. TM is the reaction off the ttwo middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two
middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T2 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T3 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T3 = T3. TM is the reaction off the two middle studs—4 and 5—such that TT = T1 = T3 = T3. TM is the reaction off the two middle studs—4 and 5—such behavior: 3TT) + 9(2 (TM) - 480,000 in.-lb = 0  $\Sigma$  M = 0 = 21(3 Ratio off def deformations ons betwee between the middle and top lines of d studs:  $\Delta T$  21 in in. 33 3 = TT = 2.33TM TM = T4 = T5 = 2909 lb Nua = (3)(6788 lb) + (2)(2909 lb) Nua = 26,182 lb (tension) \approx 26,200 lb Check if the plate remains elastic under bending due to STT times the distance to the tension ADQIHRIWKHVWHHOEHDP M elastic = S x f y = M face =  $\Sigma$  TT a bh 2 fy 6 M elastic = M face =  $\Sigma$ 61 in.-kip < M y, pl 1000 lb/kip American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage Mu = 40,000 ft-lb = 480,000 in.-lb 314 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Strength inequalities 17.3.1.1 The anchor design strengths must satisfy the following inequalities: [ $\phi$ N sa (steel strength in tension) | N ua,  $g \leq \frac{1}{9}$  (concrete breakout) |  $\psi N$  (anchor pullout) pn | Nua, g = 26,200 lb and Vua, g = 17.6.3 ( $\psi V sa$  (steel strength in shear) |  $\leq \frac{1}{9}$  (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) |  $\psi V cp$ , g (concrete breakout) | n φVn (17.6.3) Step 4: Anchor ductility Check the anchor steel ductility to determine teel du WKH[DFWRU 2.2 MA eets the ACI 318 31 Check if ASTM A29 meets OL QLWLR & KDSWHUGXFWLOLW\GH¿QLWLRQ a: A 29 has the following properties: Table 1a: ASTM A29 lon on and 14% minimum areaa reduction Elon ion at 2 in. n. = 20% > 14% min. Elongation R ea = 50% > 30% min. Reduction of area efore, A29 iis ductile. Therefore, i strength is the steel tensile strength (futa) times the stud area (Ase,N). 17.4.1.2 17.3.3.a(i) (17.4.1.2) Nsa = Ase,N futa The stud area is obtained from Table 2: da = 5/8 in. Ase,N = 0.31 in.2 Check that futa = 65,000 psi (Table 1a) is smaller than 1.9fya and 125,000 psi i. 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K
futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9(51,000 psi) = 96,900 psi (K futa < 1.9fya = 1.9fya = 1.9fya) = 96,900 psi (K futa < 1.9fya = 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 psi (K futa < 1.9fya) = 96,900 strength: [] []Nsa = (0.75)(19,955 lb) = 14,996 lb/stud []Nsa = 14,900 lb/stud > Nua = 6788 lb/stud American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 315 Step 6: Concrete breakout 17.4.2.1 Nominal concrete breakout strength of group of anchors in tension (Eq. (17.4.2.1b)): N cbg = ANc  $\psi$  ec, N  $\psi$  ed, N  $\psi$  ed, N  $\psi$  ed, N ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIWKH ance (see Fig. stud group, limited by edge distance E14.4): 1.5hef) × (1.5hef + s1 + 1.5hef) ANc = (ca2 + s2 + s2 + 1.5 'H¿QLWLRQ Fig. E14.4—Idealized tension breakout area. 5 in. + 12 2 i QFUHWHIDLOXUHDU DRID ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID ot limited ed by edge dista ce or single anchor, not distance spacing: ANNco =  $9(9\ 9(9.5)\ 5\ 2\ =\ 812\ in.\ in\ 2\ ANc\ shall$  not be taken greater than nANco, where n = number of anchors in a group. (5)(812\ in.2) = 4060\ in.2 > 1752\ in.2\ OK  $zec, N \pm PRGL FDWLRQIDFWRUIRUDQFKRUJURXSV$  loaded eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the load is applied eccentrically in tension. For the anchor group the calculated as the distance between the geometric center and the resultant tension force: eN' = 2sy 5 - (T4 + T5) sy N ua eN' = 2(12 in.) (2909 lb + 2909 lb)(12 in.) - = 2.13 in. 526,200 lb American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage The two edge anchors in tension (hef = 9.5 in.) are located less than 1.5hef from the free edge parallel to the shear force. Therefore, the tension breakout area is the shaded area in Fig. E14.4. 316 17.4.2.5 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) zed, N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; two anchors in tension are located close to the edge; ca,min < 1.5hef The breakout strength is therefore reduced WKURXJKWKHIDFWRUzed, N.  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 17.4.2.7 17.4.2.2 ca,min 1.5hef (17.4.2.5b) (5 in.)  $\psi$  ed, N = 0.7 + 0.3 | = 0.805 \ 1.5(9.5 in.) |/ zc, N ± PRGL2FDWLROIDFWRUIRUFROFUHWH condition at service load levels; assume member is cracked. zc, N = 1.0  $zcp,N\pm PRGL cFDWLRQIDFWRUIRUSRVWLQVWDOOHG$  anchors in uncracked concrete breakout strength, either calculate from Eq. (17.4.2.2a) N b = kc  $\lambda$  a f c'hef 1.5 (17.4.2.2a) N b = (24)(1 (24)(1.0) 4)(1)(1.0) 0) (1) 550 500 5000 psi (9.5)1.5 = 49, 1000 psi (9.5)1.5 = 49, 1000 psi (9.5)1.5 =
49, 1000 psi (9.5)1.5 = 49, 10 691 lb The constant, kc, was determined from tests QFU QGZDVDGMXVWHG RU LQXQFUDFNHGFRQFUHWHDQGZDVDGMXVWHGIRU te. cracked concrete from (17.4.2.1b): N cbg = ANc  $\psi$  ec, N  $\psi$  c, N  $\psi$ (1.0)(49, 691 lb) 81 812 in. 2 Ncbg = 75,500 lb 17.3.3c(ii) For a cast-in headed study with supplementary reinforcement, Condition A applies: Check that design strength is greater than required strength is greater than req - www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 317 Step 7: Pullout strength is calculated from Eq. (D-13): Npn zc, PNp 17.4.3.6 (17.4.3.1) zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of anchor in cracked concrete. zc, P = 1.0 The basic pullout strength of an concrete. zc, P = 1.0 The basic pullout strength of an concrete. zc, P = 1.0 The basic pullout strength of an concrete. zc, P = 1.0 The basic pullout strength of an concrete. crushing occurs under the anchor head, and not related to pulling the anchor out of concrete. The basic pullout strength is either to calculate from Eq. (17.4.3.4) Np = (8)Abrg fco Np = (8)(0.92 in.2)(5000 psi) = 36,800 lb For cast-in headed studs with supplementary reinforcement, Condition A applies: ength is greater than Check that design strength required strength: Step 8: Side-face blowout 17.4.4 Side-face blowout ou needs ds to be considered
considered Does heff exc exceed 2.5cca1? No, 9 9.5 in. does not exceed 12.5 in. a stud I spaced? spaced No, 6ca1 = 6 × 5 in. = 30 in. stud I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I spaced? spaced No in I edge distance of less than 6da. Headed studs are used to attach a steel plate and are unlikely to be torqued. Minimum cover of ACI 318 17.4.1.2 17.4.2.1 17.4.3.1 17. 14,900 [Ncbg 56,600 [Npn 27,600 [Nsbg NA Ratio = Nua,g]Nn Controls design? 0.456 No Yes 0.463 0.25 No — American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage 17.3.3.c(ii) 318 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Shear strengths Step 11: Steel shear To calculate connection strength, check if bolt 17.2.1.1 spacing exceeds critical spacing to determine if bolts act as a group or act individually: Critical stud spacing = 3ca1 (3)(5 in.) = 15 in. > 12 in. Therefore, studs resist shear force as a group. Nominal steel strength is the steel tensile strength (futa) times the stud area (Ase,V). 17.5.1.2 Vsa = Ase,V futa (17.5.1.2a) The stud area is obtained from Table 2: Ase, V = 0.31 in 2 9HULI\futa = 65,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi (Table 2: Vsa = Ase, V futa = 19,955 lb 17.3.3a(ii) Strength reduction factor for ductile bolt:  $[] Vsa = (0.65)((0.65)(19,955 \text{ lb}) = 12,971 \text{ lb/stud} All studs welded to a plate resist an applied ever, for this example, shear force. However, stu 1, 2, and 3 resist resis Vu assume that only studs (conservative). <math>[] Vsa = (3 \times 12,971) = 38,913 38,91 \text{ lb} \approx 38,900 \text{ lb}$  greater th n Check that design

than h: required strength: Vsa = 38,9 38,900 lb > Vua,gg = 1 17,000 lb []V American Concrete Institute - Copyrighted © Material - www.concrete breakout Studs are welded to the steel plate; therefore, VKHDULVUHVLVWHGE\DFRQFUHWHJHRPHWU\GH¿QHG by the anchors that are farthest from the edge in the direction of the shear force (Fig. E14.5). A shear breakout is assumed to initiate at a line GH¿QHGOLQHDWGHJUHHVERWK horizontally and vertically toward the edges. Nominal concrete breakout shear strength of an anchor group is: Vcbg = 17.5.2.4 AVc  $\psi$  ec,  $V \psi$  ed,  $V \psi$  c,  $V \psi$  h, V Vb AVco Fig. E14.5—Idealized shear breakout. (17.5.2.1b) The edge distance ca1 is calculated from the line of studes 1, 2, and 3: ca1 = 2(12) + 5 = 29 in. in.) + 5 in.) (20 in.) AVc =  $(1.5(29\ 450\ in.2\ AVc = 1450\ AVc = (1.5ca1 + 2ss + ca2)(ha)\ 17.5.2.1\ HFW\ HDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR\ D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR D\ AVcoLVWKHSURMHFWHGVKHDUHDR D\ AVcoLVWKHSURMHFWHGVKHDUHDR D\ AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDR D\ AVcoLVWKHSURMHFWHGVKHDUHDR D\ AVc$  $zec,V\pm PRGLiFDWLRQIDFWRUIRUDQFKRUJURXSV$  loaded eccentrically in shear; ec = 0 17.5.2.6 zed,V\pm PRGLiFDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; the anchor is located close to the edge. Therefore, there is not enough space for a complete breakout prism to develop.  $\psi$  ed , V = 0.7 + 0.3 ca 2 1.5ca1 (17.5.2.6b) zec, V = 1.0 $\psi$  ed, V = 0.7 + 0.35 in. = 0.734 1.5(29 in.) 17.5.2.7 zc,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load level; assume member is cracked and detailed with supplementary reinforcement. zc, V = 1.217.5.2.8 zh,  $V \pm$  PRGL at service load level; assume member is cracked and detailed with supplementary reinforcement. zc,  $V \pm$  PRGL at service load level; assume member is cracked  $< 1.5ca1 \ \psi h$ ,  $V = 1.5ca1 \ge 1.0$  ha (17.5.2.8)  $\psi h$ ,  $V = 1.5(29 \text{ in.}) = 1.47 \ge 1.0$  20 in. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.5.2.1 320 17.5.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The 5/8 in. stude are welded to a 1-1/2 in. steel plate; therefore, Vb is calculated as the smaller of Eq. (17.5.2.2b) and (17.5.2.2b) and (17.5.2.3) Vb = 9 $\lambda$  a f c'(ca1) 1.5 (17.5.2.2b) Vb = 9(1.0)(5000 psi)(29in.) 1.5 = 99,386 lb Controls and 17.5.2.3 (A Vb = 8 | 5 in.) | (0.625 in.) For cast-in anchors, & E = hef and & E'' da 17.5.2.1 Solve Eq. (17.5.2.1b): 17.33c(ii) Vcbg = AVc \psi ec, V \psi ed, V \psi c, V \psi h, V = 8 | e | (17.5.2.2b) Vb = 9(1.0)(5000 psi)(29in.) 1.5 = 99,386 lb Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.1 Solve Eq. (17.5.2.1b): 17.33c(ii) Vcbg = AVc \psi ec, V \psi ed, V \psi c, V \psi h, V = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b Controls and 17.5.2.3 (A Vb = 8 | b C Vb AVco 0.2 0.625 in. (1.0) 5000 psi(29 in.)1.5 Vb = 105,860 lb (17.5.2.1b) Vcbg = (0.75)(49,253 lb) = 36,939 lb  $\approx$  36,900 lb (17.5.2.1b) Vcbg = (0.75)(49,253 lb) = 36,939 lb  $\approx$  36,900 lb Vcbg = 36,9 36,900 lb > Vua,gg = 1 17,000 lb []V American Concrete Institute - Copyrighted © Material - www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 321 Step 13: Pryout Short stiff anchors can pryout on the side opposite to load direction. The pryout strength is related to the stud group's tension breakout strength Ncbg and embedment depth hef. 17.5.3.1 Vcpg = kcp Ncpg (17.5.3.1b) hef LQ  $\Rightarrow$  kcp = 2.0 Ncpg is taken as Ncbg
calculated for studs 1, 2, and 3, which are the studs assumed to resist the shear force (Fig. E14.6). N cbg = ANc ANco  $\psi$  ec, N  $\psi$  ed, N  $\psi$  c, N  $\psi$  c, N  $\psi$  cp, N b (17.4.2.1b) Fig. E14.6—Idealized shear breakout. ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRQ the surface as approximated by a rectangle with edges bounded by 1.5hef in a direction perpendicular to the shear force and the free m the centerline of the edge of the concrete from anchor (Fig. E14.6). ANc = (1.5hef + 2ss + ca2)(3heff) 1.5(9.5 in.) = 14.2 14.25 in. 1.5hef = 1.5(14.2 4 in.) + 2(12 in. in.) + 5 in.)(2)(14.25 in.) ANNC = ((14.25 233 in.2 ANC = 1233 ANCo = 9hef2 = 9(9.5 in.) i) = 81 812 in.2 17.4.2.4 (RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU (calculated in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, N = 0.805 (calculate 17.4.2.6 The slab is cracked and detailed in Step 6) zed, with supplementary reinforcement.  $z_{c,N} = 1.0\ 17.4.2.7\ Cast-in\ place\ headed\ stud:\ z_{cp,N} = 1.0\ 17.4.2.2\ Nb\ calculated\ in\ Step\ 6:\ Nb\ = 49,691\ lb\ 17.4.2.1\ Nominal\ concrete\ breakout\ strength\ from\ Eq.\ (17.5.3.1b)\ to\ determine\ the\ pryout\ strength\ from\ Eq.\ (17.5.3.1b)\ to\ determine\ the\ pryout\ strength\ from\ Eq.\ (17.4.2.1b)$ strength: 17.3.3c(i) 5HGXFWLRQIDFWRU[RUFDVWLQDQFKRUZLWK supplementary reinforcement, Condition A applies: Check that design strength is greater than required strength is greater than required strength: Vcpg = (2.0)(60,741 lb) Vcpg = 121,482 lb [Vcpg 0E 0E §0E 0Vcpg = 91,100 lb > Vua,g = 17,000 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage 17.4.2.1 322 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 14: Shear force summary ACI 318 17.5.1.2 17.5.2.1 17.5.5.2.1 17.5.5.2.1 17.5.5.2.1 17.5.5.2.1 17.5.5.5.5.5.5.5.5.5.5.5.5. = Vua,(g)  $\nabla n 0.44 \ 0.46 \ 0.19 \ -$  Step 15: Interaction of tensile and shear forces 17.6.1 Check if Vua  $\nabla n'' \ 17,000 \ b/36,900 \ b = 0.46 > 0.2 \ 17.6.2 \ 26,200 \ b/56,600 \ b = 0.46 > 0.2 \ 17.6.3 \ Check if \ Nua \ g \ waterial - Copyrighted <math>\otimes$  Material - Copyrighted  $\otimes$  Materi www.concrete.org Controls design? No Yes No — OK CHAPTER 15—ANCHORING TO CONCRETE 323 Anchorage Example 15: Post-installed adhesive group of anchors subjected to shear and moment A normalweight reinforced concrete girder with fcg SVLVXSSRUWVD:[VWHHOEHDPZHOGHGWRDIWLQ[IWLQ x 3/4 in. thick A36 steel plate. The plate is connected by eight 5/8 in. diameter, 9.5 in. long ASTM F1554 Grade 36 adhesive anchors, spaced at 12 in. centers in both directions (Fig. E15.1). The group of anchors transmits a factored shear force of 8500 lb (Fig. E15.1). The group of anchors transmits a factored shear force of 8500 lb (Fig. E15.1). direction of the anchors). Check the adequacy of the anchor group. Supplemental reinforcement is detailed to assist the transfer of forces. Given: Loads— MU = 20,000 ft-lb factored shear Fig. E15.1—Steel beam anchored into reinforced concrete girder. Anchorage Anchors— 5/8 in. post-installed adhesive anchors ASTM F1554 Grade 36; Table 1a: • fy = 36,000 psi (GJHGLVWDQFHWR&UVWDQFKRUURZca1 = 5 in.; ca2 = 5 in. Anchor spacing s1 = s2 = 12 in. Embedment depth hef = 9.5 in. Concrete—fcg SVL girder width b = 20 in. te 3a = 1.0 (normalweight concrete) ACI 318-14 Discussion scus Step 1: Anchor group 17.2.1.1 For a group of anchors, the following assumptions apply: • /RDGV DUH DSSOLHG WKURXJK DQ DQFKRU SODWH WR individual anchors; • 7KHDQFKRUSODWHLVLQ¿QLWHO\VWLIIDQG • Anchors are of a similar type, size, and depth. Calculation Ca Check anchor group effects: Spacing between anchors must be less than the critical spacing: • Concrete breakout in tension: Critical spacing = 3hef LQ+LQFRQVLGHUDQFKRUVDVD group • Bond strength in tension: Critical spacing = 2cNa, where cNa = 10(0.625 in.) 2100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in. 1100 psi = 8.63 in.
1100 psi = 8.63 in. 1100 psi = 8 reinforcement is detailed, therefore, 17.7.1 through 17.7.6 do not apply. OK; consider the anchors as a group. American Concrete Institute - Copyrighted © Material - www.concrete.org 324 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 2: Anchor forces Assume that: • The anchor reactions are proportional to the GLVWDQFHIURPWKHFRPSUHVVLRQADQJHWRH • The compression force is resisted by a concrete reaction centered directly beneath the steel beam ADQJH • Concrete is cracked. There are other methods that can be used to determine anchor forces. The one used in this example is conservative. TT is the reaction of one top anchor, where TT = T1 = T 2 = T 3. TM is the reaction of middle anchor, where TM = T 4 = T 5. Fig. E15.2—Force in adhesive anchors. KLVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVDQG<sup>--</sup>LVDQFKRUVVLIQHVVVLIQHVVVLIQHVVVLIQ</sup> ADQJHDVVXPLQJHODVWLFEHKDYLRU  $\Sigma$  M = 0 = (21)(3TT) + (9)(2TM) - 240,000 lb = 0 R io of deformations Ratio mations lastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lin liness of stude:  $\Delta T$  21 in. 2 = 9 in. n.  $\Delta M$  g Check if the plate remains elastic under bending GXHWRITTWLPHVWKHGLVWDQFHWRWKHWHQVLRQADQJH of the steel beam and top lines and top line 21 in. ΔM K 9 in. TM "MK Substituting: (21 in.) (3) TM + (9 in.)(2TM) - 240,000 lb = 0 ( ) 9 in. TM = 1455 lb and TT = 3394 lb : TT = T1 = T2 = T3 = 3394 91 in.-kip 6 Mface = (3)(3394 lb)(3 in.) = 30,546 in.-lb < My,pl American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 325 Step 3: Strength inequalities 17.3.1.1 7KHDQFKRUDUHVXEMHFWHGWRWHQVLRQDQGVKHDU forces. Therefore, anchors will be checked for: N ua , g [ $\varphi$ N sa (steel strength in tension)  $\leq \langle \phi N cbg (concrete breakout) | \phi N (bond strength) ag [Nua,g = 13,092 lb Each one of those failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the controlling failure modes will be analyzed and the
controlling failure modes will be analyzed and the controlling faile will be analyzed and the cont$ Ib The interaction between tension and shear is checked: N ua, g  $\varphi$ N n + Vua, g  $\varphi$ Vn (17.6.3)  $\leq$  1.2 Anchorage 17.6.3 Step 4: Anchor ductility & KHFNWKHDQFKRUVWHHOGXFWLOLW\ZKLFK \$&,&KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH le 11a: ASTM M F1554 G Table Grade 36 has the following pr perti properties: d 14% elongation, and 23% elongation 40% areaa reductio reduction > 30%. Ther Therefore, F1554 G Table Grade 36 is ductile. Design tension strengths Step 5: Steel tension Nominal steel strength is the steel tensile strength 17.4.1.2 (futa) times the bolt area (Ase,N). Nsa = Ase,N futa (17.4.1.2) 17.3.3a(i) The anchor area is obtained from Table 3: da = 5/8 in. Ase,N = 0.226 in.2 Check if futa = 58,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi (Table 1a) is smaller than 1.9 fya = 1.9(36,000 psi) = 68,400 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi (Table 1a) is smaller than 1.9 fya = 1.9(36,000 psi) = 68,400 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi 125,000 psi OK For a 5/8 in. F1554 Grade 36 anchor, the nominal tensile strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reduction factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reducting the factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reducting the factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reducting the factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reducting the factor for ductile anchors: Check that steel design strength Nsa is obtained from Table 3: Nsa = 13,108 lb Use reducting the factor for ductile anchors: Check that steel design strength Nsa is ob lb/anchor American Concrete Institute - Copyrighted © Material - www.concrete.org OK 326 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout strength of group of anchors in tension: N cbg = ANc ANco ψ ec , N ψ cd in. and hef = 9.5 in. Because only anchors 1, 2, 3, 4, and 5 are in tension, ANc will be determined based on that group of anchors only. The two edge anchors in tension with hef = 9.5 in. are located less than 1.5hef from the free edge parallel to the shear force. Therefore, the breakout strength area is shown in Fig. E15.3. 'H¿QLWLRQ (1 in. i + 12 in. + 12 in. + 12 in. + 12 in. + 12 in. + 12 in. + 14.255 in.) 17 in.2 ANc = 1752 FW QFUHWHIDLOXUHDUHDRIDVLQJOH ed by edge dge distance or spacing: acing:
acing: acin -Idealized tension breakout. 1752 in.2 < (5)( (5)(812 in.2) = 4060 in.2 zec, N ± PRGL2FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG eccentrically in tension; ec = 2.13 in.) + 3(9.5 in.) eNgLVFDOFXODWHGE\VXPPLQJPRPHQWVDERXWWKHWRS stud line. Eccentricity is calculated as the distance between the geometric center and the resultant tension force: eN' = 2s y 5 - (T4 + T5) s y N ua eN' = 2(12 in.) (1455 lb + 1455 lb)(12 in.) - 5 13,092 lb eNg LQ American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.4.2.5  $zed, N\pm PRGL iFDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH$  effect; two anchors in tension are located close to the edge, without enough space for a complete breakout strength is WKHUHIRUHUHGXFHGWKURXJKWKHIDFWRUzed, N. ca, min < 1.5hef  $\psi$  ed , N = 0.7 + 0.3 17.4.2.6 327 ca , min 1.5hef (17.4.2.5b) zc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and detailed with supplementary reinforcement. (5 in.) |/ zc,N = 1.0 17.4.2.7 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQFKRUV in uncracked concrete zcp,N = 1.0 17.4.2.2 To determine basic concrete breakout strength, either calculate from Eq. (17.4.2.2a) f c'hef 1.5 (17.4.2.2a) N b = (17 (17)(1.0)(5000 psi)(9.5 in.)1.5 = 35,198 lb (17)(Anchorage N b = kc  $\lambda$  a as det determined from test results in The constant, kc, was XQFUDFNHGFRQFUHWHDQGZHUHDGMXVWHGIRUFUDFNHG HWHD concrete. 17.4.2.1b ete breakout kout strength from Eq Nominal concrete Eq. (17.4.2.1b): N cbg = 17.3.3c(ii) ANc  $\psi$  ec, N  $\psi$  ed, N  $\psi$  ed, N  $\psi$  e 812 in.2 Ncbg = 53,188 lb N cbgg = [] []Ncbg = (0.65)(53,188 lb) = 34,572 lb = 34,500 lb ]Ncbg = 34,500 lb > Nua,g = 13,092 lb American Concrete Institute - Copyrighted © Material - www.concrete.org OK 328 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Bond strength 17.4.5.1(b) Design bond strength is calculated by: N ag = 'H¿QLWLRQ ANa y ec , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y cc , Na y ed , Na y :KHUHLQAXHQFHGLVWDQFHccNa is calculated Eq. (17.4.5.1d): cNa = 10d a 'H¿QLWLRQ τ uncrcr 1100 00 8.63 in. (17.4.5.1d) cNNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = 8. ANaoo = ((2 × 8.36 in.)2 = 2 298 in.2 WHG XHQFHDUHDRIWK DQFKRU 5.4 group (see Fig. E15.4): ANa = (ca2 + s2 + s2 + cNa)(cNa + s1) cNa = (ca2 + s2 + s2) cNa = (ca2 + s2) cNa cNa) ANa may not exceed nANao where n = number of anchors in a group. 17.4.5.3 cNa is calculated in Step tep ep 1: ANa = (8.63 in. + 12 in. + 2 in. +
2 in. + 2 in.(refer to calculations above)  $\psi$  ec , Na = 1 = 0.8 ≤ 1.0 eN' 1+ cNa  $\psi$  ec , Na = 1 = 0.8 ≤ 1.0 2.13 in. 1+ 8.63 in. American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 17.4.5.4 zed, Na ± PRGL2FDWLRQIDFWRUIRUDGKHVLYHDQFKRUV edge effect, two anchors in tension are located close to the edge parallel to the shear load, without HQRXJKVSDFHIRUDFRPSOHWHSURMHFWHGLQAXHQFH area to develop. The bond strength is therefore UHGXFHGWKURXJKWKHIDFWRUzed, Na = 0.7 + 0.3 ca , min cNa (17.4.5.4b)  $\psi$  ed , Na = 0.7 + 0.3 ca , min cNa (17.4.5. zcp,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVDW cracked service load level without vXSSOHPHQWDU\UHLQIRUFHPHQWDDVUHLQIRUFHPHQWDDVUHLQIRUFHPHQWDU is applied to preclude brittle splitting failure. In this H[DPSOHFRQFUHWHLVFUDFNHGZLWKVXI¿FLHQWUHLQzcp, Na = 1.0 forcement to restrain crack widths, therefore: 17.4.5.2 rength of a single adheadhe To determine basic bond strength sive anchor in tension in crack widths, therefore: 17.4.5.2 rength of a single adheadhe To determine basic bond strength sive anchor in tension in crack widths, therefore: 17.4.5.2 rength of a single adheadhe To determine basic bond strength sive anchor in tension in crack widths, therefore: 17.4.5.2 rength of a single adheadhe To determine basic bond strength sive anchor in tension in crack widths, therefore: 17.4.5.2 rength of a single adheadhe To determine basic bond strength sive anchor in tension in crack widths, therefore: 17.4.5.2 rength of a single adheadhe To determine basic ppsi is obtained m the table, ble, increase bon Table E.2). From bond strength te strength s gth > 4500 psi by 6% for concrete psi: Nominal bond strength te strength s gth > 4500 psi by 6% as fcg SVL!SVL S Nba ANao 5 in. =  $0.874 \ 8.63$  in. Anchorage 17.4.5.1 329 (17.4.5.1b); N ag = ANa  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  SVL  $ALQ LQ Nba = 18,78 \ 18,784 \ lb N ag = 1102 \ in.2 \ (0.80)(0.874)(1.0)(18, 784 \ lb) \ 298 \ in.2 \ Nag = 48,524 \ lb/group \ 17.3.3c(ii)$  For a post-installed adhesive anchors with supplementary reinforcement, Condition A, Category 2 applies:  $\square Nag = (0.65)(48,524 \ lb) = 31,500 \ lb \ Check \ that \ design \ strength \ is \ greater \ than \ required \ strength \ in \ required \ strength \ in \ required \ strength \ in \ strength \ in \ strength \ in \ strength \ in \ strength \ in \ strength \ strength \ in \ strength \ strength \ in \ strength \
strength \ strength\ \ strength\ \ strength \ strength \ strength\ \ strength \ str$ = 31,500 lb > Nua,g = 13,092 lb Step 8: Side-face blowout is not applicable for adhesive anchors. Step 9: Splitting failure 17.7.1 This type of failure occurs in thin members where torque is applied and anchor has an edge distance of less than 6da. The anchors are not torqued; therefore, splitting is not applicable. American Concrete Institute - Copyrighted © Material - www.concrete.org OK 330 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Tension force summary ACI 318 17.4.1.2 17.4.2.1 17.4.3.1 Failure mode Steel/anchor Concrete breakout/group Adhesive bond/group Design strength, lb []Nsa 9800 []Ncbg 34,500 []Nag 31,500 Ratio = Nua,(g) []Nn 0.35 0.38 0.41 Controls design? No No Yes Design shear strength Step 11: Steel shear 17.2.1.1 To calculate connection strength, check bolt spacing to determine if the bolts act as a group or act individually: 17.2.3.4.3 Critical anchor spacing = 3ca1 LQ LQ+LQEROWVDFWDVDJURXS Nominal steel strength of a headed bolt is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60. 17.5.1.2 Vsa = (0.6)Ase,V futa R17.4.1.2 able 3: The bolt area is obtained from Table 0.226 in.2 Ase,VV = 0 0 psi (T (Table 1a) is not greater Check if futa = 58,000 5,000 psi. than 1.9 fya and 125,000 1.9 fya = 1.9(36,000 psi) i) = 68,400 psi (1.9 ya < 125,000 psi (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 125,000 psi) (1.9 ya < 12 M F1554 4 Grade 36 anchor, anchor, either eit For 5/8 in. ASTM or obta obtain calculate Ase, V futa ta from Eq. (17.5.1.2b) o from Table 3. T le 33: Ase, V futa = 13,1 13,108 lb/anchor Table (0.6)(13,108 3 108 lb) = 7865 lb/anchor Vsa = (0 Because a non-shrink grout pad is providedd between steel plate and the concrete beam, a 0.8 reduction Vsa = (0.8)(7865 lb) = 6292 lb/anchor factor is required. 17.3.3a(ii) Strength reduction factor for ductile bolt: Check that design steel strength is greater than required strength, assuming shear force is resisted by three anchors only (see Step 12):  $\square$  Usa = (0.65)(6292 lb) = 4090 lb/anchor  $\approx$  4100 lb  $\square$  Vsa = 4100 lb > Vua,g = 8500 lb/3 = 2833 lb/anchor OK American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 331 Step 12: Concrete breakout strength R17.5.2.1 Anchor washer plates are welded to the steel plate; therefore, shear is resisted by the anchors that are farthest from the edge in the direction of the shear force. 17.5.2.1 A shear breakout failure is assumed to initiate at a OLQHGH¿QHGE\WKHWRSEROWFHQWHUOLQHDQGWRSURSDJDWHDZD\IURPWKHGH¿QHGOLQHDWGHJUHHVERWK horizontally and vertically toward the edges. Nominal concrete breakout shear strength of a single anchor is: Vcbg = AVc \u03c6 ec, V \u03c6 ed, V \u03c6 c, V \u03c6 h, V Vb AVco (17.5.2.1b) The edge distance ca1 is calculated from the line of studs 1, 2, and 3: ca1 = 2sy + cover RUDJURXSRIDQ AVcLVWKHSURMHFWHGDUHDIRUDJURXSRIDQFKRUV AVc = (1.5ca1 + 2sy + ca2)(ha) 'H¿QLWLRQ (1.5(29 9 in.) i ) + 2(2(12 2(12 in.) + 5 in.)(20 in.) AVc = (1.450 in.) AVc = 14 FWH IDFHDUHDUHODWHG RDVKHDUAVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU in anchor nchor breakout for a single AVco = 4.5ca12 AVco =  $(4.5)(29 \text{ in.})2 = 373784 \text{ in.} 2 \text{ AVco} = 1450450 \text{ in.} 2 = 0.3833784 \text{ in.} 2 \text{ AVco} = 1450450 \text{ in.} 2 \text{ in.$ 1.0 17.5.2.6 zed, V ± PRGL  $\dot{c}$  FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV loaded in shear. The anchor is located close to the edge. Therefore, there is not enough space for a complete breakout prism to develop.  $\psi$  ed, V = 0.7 + 0.3 17.5.2.7 17.5.2.8 ca 2 1.5 ca 1 (17.5.2.6b)  $\psi$  ed, V = 0.7 + 0.3zc,V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and supplementary reinforcement is detailed, therefore: 5 in. = 0.734 1.5(29 in.) zc,V = 1.2 7KHPRGL¿FDWLRQIDFWRUIRUWKLQPHPEHUVzh,V, is 1.0 if the beam depth exceeds 1.5 times the edge distance. In this example, ha < 1.5ca1 is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) ( (A) 0.2 ) Vb = |7| e | da |  $\lambda$  a | ( \ d a ) | / f c'(ca1) 1.5 3 (17.5.2.2a) Vb = 7 (| 5 in. ) | (0.625 in. / Vb = 92,627 lb For cast-in anchors, & E is the smaller of: & E = hef and & E'' da Vb = 9\lambda a f c'(ca1) 1.5 0.2 0.625 in. 5000 psi(29 in.) 1.5 Controls & E = hef LQDQGda LQControls 1.5 (17.5.2.2b) Vb = SVL LQ = OE Solve Eq. (17.5.2.1b): AVc ψ ec ,V ψ ed ,V ψ c ,V ψ h ,V Vb AVco 17.5.2.1 Vcbg = 17.3.3c(i) For adhesive bolts with supplementary reinforcement, Condition A applies: [Vcbg DC [V ngth is ggreater than required Check that design strength strength: OE ! Vua,g OEOK [Vcbg bgg OE!V (17.5.2.1b) Vcbg bgg OE!V (17.5.2.1b) Vcbg American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 333 Step 13: Pryout strength is related to the anchor's tension breakout strength Ncb and embedment depth hef. Vcpg = kcpNcpg hef•LQ > kcp = 2.0 (17.5.3.1b) Ncpg = smaller of (Ncbg and Nag) Ncpg = Ncbg ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRQWKH surface as approximated by a rectangle, 1.5hef in a direction perpendicular to the shear force and the free edge of the concrete from the centerline of the anchor (Fig. E15.5). Fig. E15.5—Idealized shear breakout. 2(1.5)heff) ANC = (1.5)heff + 2sx + ca2)(2(1.5)h ANC = (1.5(9.5 in.) + 2(12 in.) + 5 in.)(3)(9.5 in.) ANC = 1233 in.2 ANcoo = 99hef2 = 9(9.5)2 = 812 in.2 17.4.2.4 W\P  $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$
FDWLRQIDFWRU /RDGHFFHQWULFLW\PRGL $\dot{c}$ FDWLRQIDFWRU /RDGHFFHQWULFLW 43,078 lb ANa = 2cNa(ca2 + sy + sy + cNa) ANa = (2)(8.6 in.)(5 in. + (2)12 in. + 8.6 in.) = 647 in. 2 0RGL¿FDWLRQIDFWRUVDQGNba as calculated in Step 7: (17.4.5.1b) Therefore, Ncpg = Nag Substituting into Eq. (17.5.3.1b) to determine the pryout force: 17.3.3c(i) zcp, N = 1.0647 in.2(0.8)(0.879)(1.0)(18,784 lb) 298 in.2Nag = 28,678 lb Controls N ag = Vcpg = (2.0)(28,678 lb) = 57,257 lb 5HGXFWLRQIDFWRU[RUDGKHVLYHDQFKRU&RQGLWLRQ\$ [] []Vcpg = (3.000 lb > Vua,g = 8500 lb OK American Concrete Institute – Copyrighted © Material – www.concrete.orgAnchorage 'H¿QLWLRQ 334 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 14: Shear force summary ACI 318 17.5.1.2 17.5.2.1 strength, lb  $[Vsa 4100 [Vcbg 34,500 [Vcbg 34,500 [Vcbg 43,000 NA - Ratio = Vua,(g)] Vn 0.69 0.25 0.17 - Controls design? Yes No No - (8500 lb/3)/4100 lb = 0.41 > 0.2 Therefore, shear-only design strength is not permitted. 17.6.3 N ua, g <math>\varphi$ N n + Vua, g  $\varphi$ Vn  $\leq$  1.2 13,092 lb 8500 lb/3 + = 1.10  $\leq$  1.2 31,500 lb 4100 lb Step 16: Conclusion A beam-to-beam girder connection is adequate quate to support the applied forces. American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 335 Anchorage Example 16: Cast-in studes resisting tension force applied eccentrically to the two axes of symmetry A normalweight reinforced concrete beam with fcg SVLUHVLVWVDORDGDFWLQJRQDQHPEHGGHGIWLQ[LQ[ 1-1/2 in. thick A36 steel plate. The embedded steel plate is connected to the edge of a concrete wall with six 3/4 in. diameter, 7 in. long, A29 studs spaced at 6.5 in. centers in both directions (Fig. E16.1). The wall is detailed with supplemental reinforcement to assist the transfer of forces. The 20,000 lb live load is applied eccentric connection. Given: Studs— Six 3/4 in. cast-in studs ASTM A29; Table 1a: • fy = 51,000 psi • futa = 65,000 psi • futa = righted © Material – www.concrete.org 336 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Stud group Assume: ‡/RDGLVDSSOLHGWKURXJKDQLQ¿QLWHO\VWLIIDQFKRU plate to individual studs; Studs are the same type, size, and depth; • Studs in concrete compression zones do not resist tension forces. Calculation 1. Effective stud embedment depth: hef =  $\mathcal{E} - \text{tns} + \text{tpl} - (3/16 \text{ in.})$  hef =  $(7) - (3/8) + (1-1/2) - (3/16) \approx 8$  in. Use hef = 8 in. Use hef = 6WXGVDFWLQDJURXSLQWHQVLRQLIVWXGVSDFLQJ"hef LQ LQ+LQVWXGVDFWDVDJURXS 17.7.1 17.7.2 17.7.4 3. Minimum stud spacing and edge distance: GVSDFLQJ+d da.
0LQLPXPFHQWHUWRFHQWHUW Minimum edge dist distance to concrete loam: U = 1.6L smin = 4(0.75 in.) = 3 in. < 6.5 in. in. > 1.5 5 in. ca = 6 in OK 0,000 lb) = 32,000 lb U = 1.6(20,000 American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 337 Step 3: Stud forces Reaction force in each stud is calculated by resolving moments around the center in two directions: N ua , g = U (Uecx ) xi (Uecy ) yi + + 2 2 n  $\Sigma$  eyi  $\Sigma$  exi Fig. g. E16.2—Plate and stud layout and load location. 2 5) 2 = 169.0 in.2  $\Sigma$  ey = 4(6.5) 2 2 63.4 4 iin.2  $\Sigma$  ex = 6(3.25) = 63 32,000 lb (32,000 lb)(1.5 in.)(-3.25 in.) + 6 (63.4 in.) 2 (32,000 lb lb)(2.5 in.)(-6.5 in.) + = -205 lb (C) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-3.25 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (32,000 lb)(1.5 in.)(-6.5 in.) + = 4718 lb (T) (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (169 in.) 2 (lb(1.5 in.)(3.25 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.)(0 in.) = 7795 lb (T) + (169 in.) 2 32,000 lb (32,000 lb)(1.5 in.)(-3.25 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.)(6.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.) + = 5949lb (T) (169 in.) 2 (32,000 lb)(2.5 in.)338 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 32,000 lb (32,000 lb (32,000 lb)(1.5 in.) (3.25 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) (6.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 (32,000 lb)(2.5 in.) + 6 (63.4 in.) 2 N ua,  $g = \sum N$  ua i = 1 Stud number 1 is in compression and therefore will not be considered in resisting the double eccentric tension. KHPRGLiFDWLRQIDFWRU for the case where a load is eccentric and not all studs resist tension. KHPRGLiFDWLRQIDFWRUrequires the calculation of the center of gravity of the study resisting tension: Taking a moment about a line passing through study 2 - 4 - 6: x = 2(6.5 in.) + 1(6.5 in 2(6 in. + 6.5 in.) = 5.2 in. 5 The eccentricity of the load (Fig. E16.2)
with respect to the center of gravity of the study resisting eNxg LQ LQ±LQ±LQ LQ tension, eNgLV eNyg LQ±LQ±LQ LQ tension, eNgLV eNyg LQ±LQ±LQ LQ tension, eNgLV eNyg LQ±LQ±LQ LQ tension, eNgLV eNyg LQ±LQ±LQ LQ tension, eNgLV eNyg LQ±LQ±LQ LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg LQ±LQ±LQ tension, eNgLV eNyg tension, eNg ductility Check the ductility of the stud to determine the DSSURSULDWHIDFWRU 2.2 Check if the properties of ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGHʿQLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGHʿQLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGHʿQLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGHʿQLWLRQRIGXFWLOLW\ZKLFKDUH TABLE 1a: ASTM A29 meet the ACI & KDSWHUGHʿQLWLRQRIGXFWL of area = 50% > 30% min. Therefore, A29 is ductile. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 339 Step 6: Steel tension Nominal steel strength is the steel tensile strength (futa) times the stud area (Ase,N). 17.4.1.2 Nsa = Ase,N futa (17.4.1.2) The stud area is obtained from Table 2: Check if futa = 65,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi. For a 3/4 in. A29 stud, the nominal tensile strength Nsa is obtained from Table 2: Use reduction factor for ductile studs: Check that steel design strength is greater than required strength: 1.9 fya = 1.9 (51,000 psi) = 96,900 psi futa < 1.9 fya < 125,000 psi Nsa = 28,665 lb || []Nsa = (0.75)(28,665 lb) = 21,499 lb/stud []Nsa = 21,500 lb/stud []Nsa = 21,500 lb/stud > Nua,g = 10,871 lb (maximum stud rea reaction, Nua6) ... OK Anchorage 17.3.3a(i) da = 3/4 in. Ase, N = 0.44 in.2 American Concrete Institute - Copyrighted © Material - www.concrete.org 340 THE REINFORCED CONCRETE DESIGN HANDBOOK -SP-17(14) Step 7: Concrete breakout Nominal concrete breakout strength of the stud group in tension: 17.4.2.1 N cbg = ANc ANco  $\psi$  ec, N  $\psi$  ed, N  $\psi$  c, N  $\psi$  ed, N  $\psi$  c, N  $\psi$  ed, N  $\psi$  c, N  $\psi$  by ed, N  $\psi$  c, N  $\psi$  ed, N  $\psi$  ed, only. Fig. E16 E16.3—Idealized Id l d tension breakout of stud group. tuds (2, 4, 6) hav nd The three edge studs have hef = 8 in. and 1 heff = (1.5)(8.0.0 in.) = 112.0 in. re less than an 1.5heff from the free edge. 1.5h ca1= 6 in., and are 'H¿QLWLRQ FWH QFUHWHEUHDNRXWI LOXUH ANCLVWKHSURMHFWHGFRQFUHWHEUHDNRXWIDLOXUHDUHD limited by edge di distancee (see Fig. E16. E16.3). ANc = area 1(upper rectangle) + area 2 (lower rectangle) + area
2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + area 2 (lower rectangle) + - (12 in. - 6.5 in.) ANc = 864 in.2 'H¿QLWLRQ ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID single stud, not limited by edge distance or spacing: ANco = 9(8)2 = 576 in.2 17.4.2.1 ANc shall not be taken greater than nANco, where n is equal to the number of studs in a group. 864 in.2" LQ2) = 2780 in.2 hef = 8 in. zec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG HFFHQWULFDOO\LQWHQVLRQ/RDGLVDSSOLHGHFFHQWULFDOO\ about two axes. The eccentricity around each axis is calculated separately eNxg LQDQGeNyg LQ calculated in Step 3 above and the product of both eccentricities is substituted in Eq. (17.4.2.1b): 17.4.2.4 ψ ec , N = 1  $e'_1 + N$  3hef (17.4.2.4)  $\psi$  ec, Ny =  $\psi$  ec, Nx = The eccentricity about both axes is the product of the individual eccentricities: OK 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.934 2(0.85 in.) CONCRETE zed, N ± PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect. Three studs in tension are located close to the edge without enough space for a complete breakout strength is therefore reduced through the factor yed, N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N = 0.7 + 0.3 ca , min 1.5hef (17.4.2.5b) ( 6 in. )  $\psi$  ed , N 0.7 + 0.3 = 0.85 (1.5(8 in.) // 17.4.2.6 zc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked with supplemental reinforcement: zc,N = 1.0 17.4.2.7 zcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQFKRUV in uncracked concrete without supplementary reinforcement to control splitting. For cast-in anchors:  $zcp,N = 1.0\ 17.4.2.2\ To\ determine\ basic\ concrete\ breakout\ strength,\ either\ calculate\ from\ Eq.\ (17.4.2.2a)\ N\ b = kc\ \lambda\ a\ 17.4.2.1\ b\ 864\ in.2\ (0.849)(0.85)(1.0)(38,400\ b)\ (0.849)(576\ in..2\ Ncbg = 41,567\ b)\ (0.849)(576\ in..2\ Ncbg = 41,567\ b)\ (0.849)(576\ in..2\ Ncbg = 41,567\ b)\ (0.849)(576\ in..2\ Ncbg = 41,567\ b)\ (0.849)(576\ in..2\ b)\ (0.849)(5$ 567 lb N cbgg = For a cast-in headed studs with supplementary reinforcement, Condition A applies:  $\Box$   $\Box$   $\Box$  Ncbg = (0.75)(41,567 lb) = 31,175 lb  $\approx$  31,100 lb Check that design strength is greater than required strength:  $\Box$  Ncbg = 32,000 lb  $\therefore$  NG Provide anchor reinforcement Grade 60: As  $\geq$  25.4.3.3 (17.4.2.2a) Nominal concretee br breakout strength Ncbg from Eq. (17.4.2.1b): N cbg = 17.3.3c(ii) f c'hef1.5 U  $\varphi$ f y As  $\geq$  32,000 lb 0.75 × 60,000 lb Check if hook can be developed above the 7 in. studs: f y $\psi$  $\psi$  $\psi$  $\psi$ r Try No. 3 bars: A dh  $\geq$  | db | 50 $\lambda$  f c' || (60,000 psi)(1.0)(1.0)(1.0) A dh  $\geq$  | (0.375 in.) = 6.4 in. (50)(1.0) 5000 psi || |] Edh = 6 in. < 7 in. stud : OK American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.4.2.5 341 342 25.4.3.2 25.4.3.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Reduction factor of 0.7 applies, however, minimum length of 6 in. controls. Determine tail development length for a No. 3 bar 25.4.2.2 | f y  $\psi$   $\psi$  | Ad  $\geq$  | db | 25 $\lambda$  f c' ] | (60,000 psi)(1.0)(1.0)  $d \geq |$  (0.375 in.) = 13 in. | 25(1.0) 5000 psi | Use four No. 3 U-bars (eight legs) with 20 in. legs (Fig. E16.4) Archor reinforcement Fig. E16.4—Anchor reinforcement resisting tension American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 343 Step 8: Pullout strength is calculated from Eq. (17.4.3.1): 17.4.3.6 The basic pullout strength Np iscente to modify pullout strength of anchor in zc, P = 1.0 cracked concrete. 17.4.3.6 The basic pullout strength Np iscente to modify pullout strength of anchor in zc, P = 1.0 cracked concrete. 17.4.3.6 The basic pullout strength Np iscente to modify pullout strength of anchor in zc, P = 1.0 cracked concrete. 17.4.3.1 Npn zc, PNp 17.4.3.6 The basic pullout strength Np iscente to modify pullout strength Np iscente to modify pullout strength of anchor in zc, P = 1.0 cracked concrete. 17.4.3.1 Npn zc, PNp
17.4.3.1 Npn zc, PNp 17.4.3 related to the load at which the concrete crushes behind the stud head. The stud actually pulling out of concrete is not anticipated. 17.4.3.4) Np = (8)Abrgfcg Table 1e: Abrg = 0.78 in.2 Np = (8)(0.785 in.2)(5000 psi) = 31,400 lb Anchorage For cast-in headed studs with anchorage For cast-in headed studs with anchorage For cast-in headed studs with anchorage For cast-in headed studs with anchorage For cast-in headed stude st reinforcement,  $\Box$  Condition A applies:  $\Box$ N Npn = (0.75)(1 (0.75)(1.0)(31,400 lb) = 23,550 lb/stud lb/st d  $\Box$ Npn  $\approx$  23,500 lb/stud 17.3.3c(ii) gn st quired Check that design strength is greater th than required strength. Step 9: Summary  $\Box$  pnn = 223,500 lb/stud > Nua,g = 10,871 lb/stud  $\Box$ N Design esign strength strength, lb b  $\Box$ Nsa 21,500 00 Ratio atio = Nua,(g) Nn 0.51 0 ACI 318 17.4.1.2 Failuree m mode Steel/stud tud 17.4.2.1 Concrete breakout/group Ncbg 32,000 — 17.4.3.1 Concrete pullout/stud Npn 23,500 23 5 0.46 OK Controls design? Yes Reinforcement provided—no No Therefore, the eccentric load of 20,000 lb can be resisted by an embedded plate with six 7 in. long x 3/4 in. diameter headed studs with four No. 3 U-bars provided as anchor reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org 344 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) \$QFKRUDJH([DPSOH Post-installed adhesive anchors resisting tension force having double eccentricity A normalweight reinforced concrete beam with fcg SVLVXSSRUWVDORDGDWWDFKHGWRDIWLQ[LQ[LQ thick A36 steel plate is connected to the edge of a wall with six 3/4 in. diameter, 8 in. long F1554 Grade 36 post-installed adhesive anchors, spaced at 6.5 in. centers in both directions (Fig. E17.1). The wall is not detailed with supplemental reinforcement. The 10,000 lb load is applied eccentric to the plate axes of symmetry (Fig. E17.1). The structure is assigned to Seismic Design Category (SDC) A. Check the adequacy of the double eccentric connection. Given: Loads— N = 10,000 lb Studs— Six 3/4 in. post-installed adhesive anchors ASTM F1554 Grade 36; Table 1a: • fy = 36,000 psi • futa = 58,000 psi Embedment depth hef = 8 in. Edge distance ca1 = 6 in. Eccentricity: ex = 1.5 in. ey = 2.5 in. Anchor spacing = 6.5 in. each w way Fig. E17.1—Group 7.1— p of post-installed pos adhesive anchors resisting ecce nsion about two axes Concrete—fcg SVL 3a = 1.0 (normalweight concrete) American Concrete Institute – Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 345 ACI 318-14 Discussion Step 1: Anchor group Assume: ‡/RDGLVDSSOLHGWKURXJKDQLQ¿QLWHO\VWLIIDQFKRU plate to individual studs; • Anchors are the same type, size, and depth; • Anchors in concrete compression zones do not resist tension forces. 1. Check for group action: Check if bolts are acting in group or single: • Concrete breakout in tension: Critical spacing = 3hef • Bond strength in tension: Critical spacing = 2cNa where cNa = 10(0.75 in.) 1990 psi = 10.09 in. 1100 psi d on test results and is ZKHUHIJuncr = 1990 psi is based obtained from Table E.2. LQ LQ+LQVDWLV¿HV 2.JURXSEROWV 17.7.1 ting: 2. Check for splitting: UWR UVWXGVSDFLQJ 0LQLPXPFHQWHUWRFHQWHUWXGVSDFLQJ 0LQLPXPFHQWHUWXGVSDFLQ distance ass determined by tests accordance with ACI 355.4 (Table E.2) ca,min = 5da LQ LQ"ca = 6 in. ca,min LQ 17.7.4 The post-installed adhesive anchor does not ot pro produce a splitting force and will not be torqued. Edge GLVWDQFHDQGDQFKRUVSDFLQJDUHOHVVWKDQVSHFL¿HG Step 2: Required strength 5.3.1 Tension live load applied normal to the concrete beam: U = 1.6L da = 0.75 in. OK U = 1.6(10,000 lb) = 16,000 lb American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.2.1.1 Calculation OK 346 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Anchor forces Force in each anchor is calculated from the following equation, summing moments around the middle: N ua, g = U (Uecx) xi (Uecy) yi + + 2 2 n  $\sum$  exi  $\sum$  eyi (C) .....Compression (T) .....Tension Fig. E17.2—Stud and plate layout and load location. N u1 = Nu 2 = Nu 3 = Nu 4 = Nu 5 = Nu 6 = 16,000, lb (16,000 , lb)(1.5 in.)(-3.25 in.) + 6 (6)(3.25 in.) 2 000 lb)(2.5)((16,000 in.)(-6.5 in.) + = -102 lb (C) (6)(3.25 in.) + (16,000 in.)(-6.5 in.) + (16,000
in.)(-6.5 in.) + (16,000 in.)(-6.5 in.) +  $(6.5 \text{ in.}) \ 2 \ 16,000 \ \text{lb} \ (16,000 \ \text{lb} \ (15,000 \ \text{lb} \ (15,000 \ \text{lb} \ (15,000 \ \text{lb} \ (15,000 \ \text{lb} \ (15,000 \ \text{lb} \ (2.5 \text{ in.}) \ + \ = 2359 \ \text{lb} \ (T) \ (4)(6.5 \text{ in.}) \ 2 \ 16,000 \ \text{lb} \ (2.5 \text{ in.}) \ + \ 6 \ (6)(3.25 \text{ in.}) \ + \ 6 \ (6)(3.25 \text{ in.}) \ + \ 6 \ (6)(3.25 \text{ in.}) \ + \ 6 \ (6)(3.25 \text{ in.}) \ 2 \ (16,000 \ \text{lb} \ (2.5 \text{ in.}) \ (2 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \ 16,000 \ \text{lb} \ (2.5 \$ lb (T) (4)(6.5 in.) 2 16,000 lb (16,000 lb)(1.5 in.) (-3.25 in.) + 6 (6)(3.25 in.) 2 (16,000 lb)(2.5 in.) (6.5 in.) + = 2974 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4)(6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 16,000 lb)(2.5 in.) (6.5 in.) + = 5436 lb (T) (4) TO CONCRETE 347 Check that the sum of all anchor forces is equal to the applied factored tension force: 6 Nua, g = -102 lb + 2359 lb + 1436 lb + 3897 lb + 2974 lb + 5436 lb = 16,000 lb N ua,  $g = \sum N$  ua i = 1 Anchor number 1 is in compression and therefore will not be considered in resisting the double eccentric tension force, and will be ignored in the subsequent calculations: &, KDVDWHQVLRQVWUHQJWKPRGL¿FDWLRQIDFWRU for the case where a load is eccentric and not all studs resist tension. Taking a moment about a line passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs resist tension. Taking a moment about a line passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs resist tension. Taking a moment about a line passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 in. 5 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 ine passing through studs 2 - 4 - 6: x = 2(6.5 in.) = 2.6 ine passing through studs 2 - 4 - 6: x = 2(6.5 ine) = 2.6 ine passing through studs 2 - 4 - 6: x = 2(6.5 ine) = 2.6 ine passing through studs 2 - 4 - 6: x = 2(6.5 ine) = 2.6 ine passing through studs 2 - 4 - 6: x = 2(6.5 in. 5 y of the load oad (Fig. E17.2 The eccentricity E17.2) with en of gravity of the st ds resisting respect to the center studs eNxy LQ±LQ±LQ LQ±LQ Step 4: Strength inequalities 17.3.1.1 The anchor design strengths must satisfy the following inequalities: N ua , g ( $\phi$ N sa (steel strength in tension) cbg (concrete breakout) |  $\phi$ N (bond strength) | ag Nua,g = 16,000 lb Step 5: Anchor ductility &KHFNWKHDQFKRUGXFWLOLW\WRGHWHUPLQHWKHIDFWRU in tension and shear. 2.2 Check if the properties of F1554 Grade 36 meet the Table 1a: ASTM F1554 Grade 36 has the following &KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKLV properties: 14% elongation, and 30% minimum area reduction. 23% elongation in 2 in. of length > 14%, and 40% area reduction > 30%. Therefore, F1554 Grade 36 is ductile. American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 7KHPRGL¿FDWLRQIDFWRU\ec,N requires the calculation of the center of gravity of the anchors resisting tension: 348 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Steel tension Nominal steel strength is the steel tensile strength is the steel tensile strength (futa) is smaller than 1.9 fya and 125,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi (Table 2a) is smaller than 1 psi. For a 3/4 in. F1554 Grade 36 anchor, the nominal
tensile strength Nsa is obtained from Table 3: 17.3.3a(i) Use reduction factor for ductile anchors: Check that steel design strength is greater than required strength: da = 0.75 in. Ase, N = 0.334 in.2 1.9 fya = 1.9(36,000 psi) = 68,400 psi futa < 1.9 fya < 125,000 psi OK Nsa = 19,372 lb [] []Nsa = 0.75 in. Ase, N = 0.334 in.2 1.9 fya = 1.9(36,000 psi) = 68,400 psi futa < 1.9 fya < 125,000 psi OK Nsa = 19,372 lb [] []Nsa = 0.75 in. Ase, N = 0.334 in.2 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fya < 1.9 fy ,372 lb) = 14,529 lb/anchor []Nsa 😑 14,500 lb/anchor Nsa = 114,500 lb/anchor > Nua,g = 5436 lb (maximum []N reaction Nua6) :. OK anchor reaction, American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 349 Step 7: Concrete breakout Nominal concrete breakout strength of group of anchors in tension: 17.4.2.1 N cbg = ANc  $\psi$  ec, N  $\psi$  ed, N  $\psi$  c, N  $\psi$  ed, N  $\psi$  c, N  $\psi$  ed, N  $\psi$  cd, N  $\psi$  ed, N  $\psi$  cd, N  $\psi$  ed, N  $\psi$  cd, N  $\psi$  ed, N  $\psi$  in. 1.5h F E ANCLVWKHSURMHFWHGFRQFUHWHEUHDNRXWIDLOXUHDUHD limited by edge distance (see Fig. E17.3): 5hef) ANc = (1.5hef + s1 + 1.5hef) × (ca1 + s2 + 1.5hef) (1.5hef - s1) ANc = (12 in. + 6.5 in. + 12 in.) (6 in. + 12 in.) + (12 in. - 5.5 in.) (6 in. + 12 in.) ANc = 864 in.2 'HcQLWLRQ + 1.5hef) × (ca1 + s2 ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIDVLQJOH anchor, not limited by edge distance or spacing: ANco = 9(8)2 = 576 in.2 17.4.2.1 ANc shall not be taken greater than nANco, where n = number of anchors in a group. 864 in.2" LQ2) = 2880 in.2 17.4.2.4 zec, N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG HFFHQWULFDOO\LQWHQVLRQ/RDGLVDSSOLHGHFFHQWULFDOO\ about two axes. The eccentricity around each axis is calculated in Eq. (17.4.2.1b):  $\psi$  ec , N = 1 2e' 1+ N 3hef (17.4.2.4) The eccentricity about both axes is the product of the individual eccentricities:  $\psi$  ec, Ny = 1 = 0.934 2(0.85 in.) 1 + 3(8 in.) 1 = 0.909 2(1.2 in.) 1 + 3(8 in.) 2ec, Nz zec, Ny = (0.934)(0.909) = 0.849 American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage Spacing: s = 6.5 in. and hef = 8 in. Because only anchors 2, 3, 4, 5, and 6 are in tension, ANc is determined based on those anchors only. ANco is determined for one anchor only. 350 17.4.2.5 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) zed, N±PRGL¿FDWLRQIDFWRUIRUDQFKRUVHGJHHIIHFW Three studs in tension are located close to the edge without enough space for a complete breakout prism to develop. The breakout strength is therefore reduced through the factor yed,N; ca,min < 1.5hef  $\psi$  ed, N = 0.7 + 0.3 17.4.2.6 ca, min 1.5hef (17.4.2.5b) zc,N ± PRGL2FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked. (6 in.)  $\psi$  ed, N = 0.7 + 0.3 [ = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N
= 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5(8 in.) ]/ zc,N = 1.0 17.4.2.7 ] = 0.85 ( 1.5  $zcp,N \pm PRGLiFDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors$  in uncracked concrete breakout strength, either calculate from Eq. (17.4.2.2a) N b = kc  $\lambda$  a f c'hef 1.5 (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = kc  $\lambda$  a f c'hef 1.5 (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = kc  $\lambda$  a f c'hef 1.5 (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) () 5000 psi (8 in.)1.5 = 27,200 lb mined from Eq. (17.4.2.2a) N b = (17)(1.0) (0 b mined from Eq. (17.4.2.2a) N b = (17)(1.0) (0 b mined from Eq test rere The constant, kc, was determined QFUHWHDQ VXOWVLQXQFUDFNHGFRQFUHWHDQGZHUHDGMXVWHGIRU cracked concrete from Eq. (17.4.2.1b): N cbg = 17.3.3c(ii) ANc ψ ec , N ψ cp , N N b ANco (17.4.2.1b) 7.4.2.1b) For an adhesive anchor without supplementary reinforcement, Condition B, Category 1 is chosen: Check that design strength is greater than required strength: 864 in.2 (0.849)(0.85)(1.0)(1.0)(27,200 lb) (0.849 576 in.2 Ncbg = 29,433 lb N cbg =  $[] \ [] Ncbg = (0.65)(29,443 lb) = 19,100 lb \ [] Ncbg = 19,100 lb \ [] N$ © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 351 Step 8: Bond strength 17.4.5.1(b) Check bond strength of an adhesive anchor: 'H¿QLWLRQ ANa ψ ec , Na ψ ed , Na ψ ec , Na ψ ed , Na ψ ec calculation of bond strength in ted by edge distance or spacing: tension if not limited ANao = (2cNa)2 (17.4.5.1c) 17.4.5  $cNNa = 10.09 \ 10 \ in. in.)2 = 407 \ in.2 \ (17.4.5.1d)$  ANao =  $(2 \times 10.09 \ in \ ANaLVWKHSURMHFWHGLQAXHQFHDUHDRIDJURXSRI$  adhesive anchors, for calculation of bond strength in tension (see Fig. E17.4): ANc = area 1 (top rectangle) + area 2 (bottom rectangle) + area 2
(bottom rectangle) + area 2 (bottom r shall not be taken greater than nANao, where n = number of anchors in a group. ANa"nANao ANa =  $(10.09 \text{ in.} + 6.5 \text{ in.} + 10.09 \text{ in.}) \times (6 \text{ in.} + 6.5 \text{ in.}) = 707 \text{ in.} 2 \times (6)(407) = 2442 \text{ in.} 2 \text{ American Concrete Institute} - Copyrighted © Material - www.concrete.org OK Anchorage N$ ag = 352 17.4.5.3 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) zec,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRU JURXSVORDGHGHFFHQWULFDOO\LQWHQVLRQ/RDGLVDSplied eccentrically about two axes. The eccentricity around each axis is calculated separately eNxg in. and eNyg 0.824 zed,Na±PRGL¿FDWLRQIDFWRUIRUDQFKRUVHGJHHIIHFW Three anchors in tension are located close to the edge without enough space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK JKWKHIDFWRUzed, Na, which LVWKHUHIRUHUHGXFHGWKURXJKWKHIDFWRUzed neugh space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK JKWKHIDFWRUzed neugh space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK JKWKHIDFWRUZED neugh space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK JKWKHIDFWRUZED neugh space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK JKWKHIDFWRUZED neugh space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK NAGN neugh space for a complete proKHERQGVWUHQJWK MHFWHGLQAXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK mHFWHGLQAXHQFHDUHDWHGYH be calculated from (17.4.5.4b). ca1 = ca,min < cNa ψ ed , Na = 0.7 + 0.3 .3 17.4.5.5 1 = 0.922 0.85 in. 1+ 10.09 in. ca , min cNa (17.4.5.4b) DWL FWRUIRUDGKHVLYHDQFKRUV ack concrete without without suppleuppledesigned for uncracked PHQWDU/UHLQIRUFHPHQWDPRGL¿FDWLRQIDFWRULV applied to preclude brittle splitting failure. In this H[DPSOHFRQFUHWHLVFUDFNHGZLWKVXI¿FLHQWUHLQforcement to restrain crack widths; therefore:  $\psi$  ed , Na = 0.7 + 0.3 03 6 in in. = 0.878 10.09 10 09 iin. zcp, Na = 1.0 To determine basic bond strength of a single adhesive anchor in tension in uncracked concrete, calculate from Eq. (17.4.5.2) Nba 3aIJcrAdahef (17.4.5.2) ZKHUHIJuncr = 1990 psi is obtained from test data; refer to Table E.2: Bond strength > 4500 psi: 17.4.5.1 707 in.2 (0.824)(0.878)(1.0)(39,761 lb) 407 in.2 Nominal bond strength from Eq. (17.4.5.1b): N ag = Check that design strength is greater than required strength: 7DEOH( $\square$  Nag = (0.65)(49,970 lb) = 32,480 lb  $\cong$  32,500 lb > Nua,g = 16,000 lb American Concrete Institute – Copyrighted  $\bigcirc$  Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 353 Step 9: Summary ACI 318 17.4.1.2 17.4.2.1 17.4.3.1 Failure mode Steel/anchor Concrete breakout/group Bond/group Design strength, lb []Nsa 14,500 []Ncbg 19,100 []Nag 32,500 Ratio = Nua,(g) []Nn 0.37 0.83 0.78 Controls design? No Yes No Anchorage Therefore, the eccentric service load about two axes of 10,000 lb can be resisted by the post-installed adhesive anchors. Comparing Examples 16 and 17, one concludes that cast-in stude adhesive anchors. American Concrete Institute – Copyrighted © Material – www.concrete.org 354 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Anchorage Example 18: Cast-in column anchors resisting tension and shear forces. A normalweight reinforced concrete pedestal with fcg SVLVXSSRUWVD:[VWHHOFROXPQZKLFKLVZHOGHGWRDIW in. x 1 ft 9 in. x 1-1/2 in. thick A36 steel plate. The plate is connected to the pedestal with four 1-1/4 in. ASTM F1554 Grade 55 cast-in bolts with heavy hex nuts. Anchor bolts are torqued and have an embedment depth of 18 in. The pedestal is reinforced with No. 4 ties, as shown in Fig. E18.1. The column vertical reactions are 60 kip service gravity dead load, 75 kip service lateral dead load, 9 kip service lateral live load, and 12 kip lateral wind load. The structure is assigned to Seismic Design Category (SDC) A and, therefore, column seismic reactions are negligible. Check the adequacy of the bolt group. Given: Loads— DV = 60,000 lb (service gravity dead load) LV = 75,000 lb (service gravity load load) UV = ±170,000 lb (vertical wind) DH = ±8000 lb (service lateral dead load) LH =  $\pm 9000$  lb (service lateral live load) WH =  $\pm 12,000$  lb (lateral wind) Anchors— 1-1/4 in. diameter ASTM F1554 Grade 55; Table 1a: • futa = 75,000 psi • fya = 55,000 p diameter da = 1.25 in. 3 in.2 Anchor nut bearing area Abrg = 2.237 Pedestal— fcg = 4500 psi Dimensions: b x b x h = 28 in. x 28 in. x 45 in. cover on ties 3a = 1.0 (normalweight concrete) Fig. E18.1—Headed anchors subjected to shear and tension. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE /DWHUDO U = 1.2D + 1.6(Lr) + (0.5W) U = 1.2D + 1.0W + 0.5(Lr) U = 0.9D + 1.0W + 0.5(Lr) U = 0.9D + 1.0W Calculation Equation (5.3.1f) results in the maximum vertical tension. (5.3.1a) Negative sign indicates anchor tension (upward force). (5.3.1b) (5.3.1c) (5.3.1d)(5.3.1e) (5.3.1f) U = 0.9(60 kip) - 1.0(170 kip) = -116 kip (5.3.1g) Equation (5.3.1c) controls. (5.3.1c) U = 1.2(8 kip) + 1.6(9 kip) + 0.5(12 kip) = 30 kip (5.3.1d) (5.3.1f) Structure is assigned to SDC A; therefore, seismic requirements of 17.2.3 do not apply. Step 2: Anchor group 17.2.1.1 Check if maximum bolt spacing (16 in.) is close enough (within 3hef) for the bolts to o act as a group sion. for concrete breakout in tension. 17.7.1 um bbolt spacing (6dda) in Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum spacing (6dda) in Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum spacing (6dda) in Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum space (6dda) in Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp Chapter 17. to the minimum m sp
Chapter 17. to the minimum m sp Chapter 17. to the minimum m thicknesses form to 17.7.1 through 17.7.6, unless supplementary reinforcement is provided to control splitting." 17.7.4 Because vertical and transverse reinforcement is detailed, a check is not needed for minimum edge distance for torqued anchors, but side-face blowout must be checked. Required tension strength is 116,000 lb Required shear strength is 30,000 lb L LQ+LQ.; therefore, bolts act as a p group. 6(in.) n.) = 7.5 in. in < 16 in.  $\therefore$  OK 6da = 6(1.25 American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage ACI 318-14 Discussion Step 1: Required strength 5.3.1 9HUWLFDO: U = 1.4D U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6(Lr) + (0.5W) U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6(Lr) + (0.5W) U = 1.2D + 1.6(Lr) + 1.6(1.0W + 0.5(Lr) U = 1.2D + 1.0E + 1.0(L) U = 0.9D + 1.0E 355 356 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3: Strength in equalities 17.3.1.1 The anchor design strengths must satisfy the following inequalities: N ua, g [ $\phi$ N sa (steel strength in tension) |  $\phi$ N (concrete breakout) |  $cbg \leq \{ | \phi N pn \}$  $(anchor pullout) | [ \phi N sb (side - face blowout) Nua,g = 116,000 lb [ \phi Vsa (steel strength in shear) | \leq { \phi Vcbg (concrete breakout) | \phi V (anchor pryout) [ cpg Vua,g = 30,000 lb and Vua, g The interaction of tensile and shear forces must also satisfy the following inequality: 17.6.3 N ua, g \phi N n + Vua, g \phi Vn \leq 1.2 (17.6.3) Step 4: Anchor ductility$ &KHFNLIWKHDQFKRUVWHHOPHHWVWKHGH¿QLWLRQIRUD HWVWKHGH¿QLWLRQIRUD GXFWLOHVWHHOHOHPHQWWRGHWHUPLQ 2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: s&&CSWHUGXFWLOLW\GH¿QLWLRQIRUD GXFWLOHVWHHOHOHPHQWWRGHWHUPLQ 2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: s&CSWHUGXFWLOLW\GH¿QLWLRQIRUD HWVWKHGH¿QLWLRQIRUD GXFWLOHVWHHOHOHPHQWWRGHWHUPLQ 2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: s&CSWHUGXFWLOLW\GH¿QLWLRQIRUD HWVWKHGH¿QLWLRQIRUD GXFWLOHVWHHOHOHPHQWWRGHWHUPLQ 2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: s&CSWHUGXFWLOLW\GH¿QLWLRQIRUD HWVWKHGH¿QLWLRQIRUD HWVWKHGH¿QLWLRQIRUD GXFWLOHVWHHOHOHPHQWWRGHWHUPLQ 2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: s&CSWHUGXFWLOLW\GHµ¢QLWLRQIRUD HWVWKHGHµ¢QLWLRQIRUD HWVWKHGHµ¢QLWLRQIRUD GXFWLOHVWHHOHPHQWWRGHWHUPLQ 2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: s&CSWHUGXFWLOLW\GHµ¢QLWLRQIRUD HWVWKHGHµ¢QLWLRQIRUD HWVWKHGhµ¢QLWLRQIRUD HWVWKHQUN HWVWKHGhµ¢QLWLRQIRUD HWVWKHGhµ¢QLWLRQIRUD HWVWKHQUN HWVWKHQU minimum ar area reduction E ngat Elongation at 2 in. = 21% > 14% min. GXFWL HD 5HGXFWLRQRIDUHD • PLQ T refo F1554 554 Grad Therefore, Grade 55 is ductile. Tension strength is the steel tensile strength (futa) times the anchor area (Ase,N). 17.4.1.2 17.3.3a(i) (17.4.1.2) Nsa = Ase,N futa The anchor area is obtained from Table 3: da = 1.25 in. Ase, N = 0.969 in.2 Check if futa = 75,000 psi (Table 1a) is smaller than 1.9 fya and 125,000 psi (OK For a 1-1/4 in. ASTM F1554 Grade 55 anchor, the nominal tensile strength Nsa is obtained from Table 3: Nsa = 72,675 lb Use  $\varphi$  factor for ductile steel element:  $\Box$  Nsa = 54,500 lb/anchor > Nua,g = 116,000 lb/4  $\Box$ Nsa = 54,500 lb/anchor > Nua,g = 29,000 lb/anchor OK American Concrete Institute – Copyrighted  $\odot$  Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 357 Step 6: Concrete breakout ANc  $\psi$  ec , N  $\psi$  entire pedestal cross section. Therefore, effective embedment depth used in calculations is limited to the larger of: (17.4.2.1b) Fig. E18.2 + 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax |
1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | smax | 1.5 | ANCOLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIRQH anchor with edgee dis distances ggreater than 1.5Kgefef. ANco = 99(5.33))2 = 256 in in.2 17.4.2.1 mit nANco here n = num number As an upper limit, co• ANc, where of anchors in a gr group. 2 nANco 1024 in.2 > 784 in.2 co = (4)(256 in.) = 10 17.4.2.4 RQ UIRUDQFKRUJ SVORDGHG  $zec,N\pm PRGL\dot{c}FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG$  zec,N = 1.0 eccentrically in tension; ec = 0.17.4.2.5 J  $zed,N\pm PRGL\dot{c}FDWLRQIDFWRUIRUFDVWLQSODFHEROWVDW$  cracked service load level: zc,N= 1.0 17.4.2.7 zcp,N - factor for post installed anchors; anchor is FDVWLQVRWKHPRGL¿FDWLRQIDFWRUGRHVQRWDSSO\ zcp,N = 1.0 17.4.2.2 Basic concrete is: f c'hef 1.5 OK () 6 in.  $\psi$  ed , N = 0.7 + 0.3 | = 0.925 \ 1.5(5.33 in.) |) 17.4.2.6 N b = kc  $\lambda$  a Anchorage Therefore, Koef = 5.33 in. ANCLVWKHSURMHFWHGFRQFUHWHIDLOXUHVXUIDFHRIWKH anchor group. In this problem, ANc is the pedestal area. 'H¿QLWLRQ (17.4.2.1b) N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b: 17.4.2.1b: 17.4.2.1b: 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b: 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b N cbg = () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. () 4500 psi (5.33 in.) (1.0)(1.0)(19,814 lb) 256 in.2 Ncbg = 56,130 lb American Concrete Institute - Copyrighted © Material - www.concrete.org 358 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) 3a = 1.0 To calculate concrete breakout strength, supplementary reinforcement is detailed, so Condition A applies: 17.3.3c(ii) Check if design strength is greater than required strength: [] []Ncbg OE OE = OE []Ncbg OENu, ag OENG Because the factored tension exceeds concrete breakout strength, anchor reinforcement replaces concrete breakout in determining φNcbg. Anchor reinforcement needs to be developed on both sides of the failure breakRXWVXUIDFHLQDFFRUGDQFHZLWK&KDSWHU7KH strength reduction factor for the design of anchor []reinforcement: &KHFNLIWKHSHGHVWDOUHLQIRUFHPHQWLVVXI&FLHQWWR resist factored tension. As > N ua, g As = eight No. 8 bars; As LQ LQ As = \vert f y \*UDGHlfy SVL SVL N ua, g \vert f y \vert f = OE = LQ SVL Therefore, Th refo eight No. 8 ar are OK. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15-ANCHORING TO CONCRETE 359 Step 8: Anchor reinforcement detailing 25.4.3 Check if a hooked No. 8 bar is developed on both sides of the failure plane: [fyyewcwr] | db || 50\lambda f c' || [ (60, 000 psi)(1.0)(0.7)(1.0) ]  $(25.4.3.1a) \text{ A dh} \ge |$  (1.0 in.) (50)(1.0) 4500 psi | |  $(25.4.3.1 \text{ A dh} \ge |$  25.4.3.2 ze -coating factor; ze = 1.0, bars are uncoated zc—cover factor; ze = 0.7, No.8 bars hooked, with VLGHFRYHUEH\RQGKRRN•LQ Edh = 12.5 in. A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFHPHQWIDFWRUzr = 1.0 25.4.3.1 A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 \text{ in.}2) zr^2 FRQ2QLQJUHLQIRUFH R17.4.2.9 The development can be reduced by a factor of As, req'd/As, prov, assuming only four No. 8 bars are hooked: 8 A dh, min  $\geq$  max  $db \in 6$  in. By inspection, 8 and 10 By inspection, 8 are hooked: 10.3 in. OK Anchorage 25.4.10.1 According to R17.4.2.9, 2.9, the bar must be within ered effective for resisting anchor hef /2 to be considered at the elat to the actual al tension. Note that this hef is related nd is not the reduced hef used ed for failure plane, and at strength calculated from hef ce minus 2 in. top cover, minus 2 in. vertical distance along failure plane to No. 8 bar intersection, is 14 in., as shown in Fig. E18.3. Fig. E18.3—Anchor and corner bar locations 7KLVGHWDLOPD\EHGLI¿FXOWWRLPSOHPHQWEHFDXVH DGMXVWPHQWVWR1REDUVDUHGLI¿FXOWDIWHUIRXQGDtion concrete is placed. If misaligned, the hooks might interfere with other bars, anchors, or shear lugs (Fig. E18.4). Fig. E18.4—No. 8 corner bars with hooks. American Concrete Institute - Copyrighted © Material - www.concrete.org 360 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Another approach is to not use the No. 8 pedestal bars as anchor receives one hairpin for a total of eight No. 6 (Fig. E18.5). Fig. E18.5). Fig. E18.5—U-bars on both sides of each anchors. Fig (c) Another approach is to splice a hooked No. 6 bars. anchors for a total Fig. E18.6—Hairpins spanning two anchors. Fig (c) Another approach is to splice a hooked No. 6 bar to each of the No. 8 pedestal bars (Fig. E18.7). This detail should have at least two No. 4 ties around the perimeter of the spliced bars. Fig. E18.7—No. 6 bars spliced to No. 8 bars. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 361 Step 9: Pullout 17.4.3.1 Nominal pullout strength is calculated from Eq. (17.4.3.1): Npn = zc, PNp (17.4.3.1) 17.4.3.6 zc, P – factor to modify pullout strength of anchor in cracked at service load: 17.4.3.4 The basic pullout strength Np for a cast-in anchor is related to concrete crushing under the anchor head. The basic pullout strength is either calculated from Eq. (17.4.3.4) 17.3.3c(ii) (17.4.3.4) Np = (8)(2.237 in.2)(4500 psi) = 80,532 lb/anchor From Table 1c: Abrg = 2.237 in.2 For a cast-in headed bolt with supplementary reinforcement, Condition A applies: gn st quired Check that design strength is greater th than required strength:  $\varphi = 0.75 \varphi N Npn = ((0.75)(1.0))$  $(80,532 \text{ lb}) = 60,399 \text{ lb} \approx 60,400 \text{ lb} \text{ Npn} > \varphi \text{N} \text{Nua}, g \neq \text{N} \text{Nua}, g \neq 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N}
\text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g = 29,000 \text{ lb}/\text{anchor 0 lb} > \varphi \text{N} \text{Nua}, g =$ blowout 17.4.4 Side-face blowout failure is considered for multiple headed anchors if geometry checks (a) and (b) are true: (a) hef > 2.5ca1 (a) 18 in. > 2.5(6 in.) = 36 in. True (b) s < 6ca1 (b) 16 in. < 6(6 in.) = 36 in. True (b) s < 6ca1 (c) 16 in. < 6(6 in.) = 36 in. True Therefore side-face blowout is considered. (c) 18 in. > 2.5(6 in.) = 15 in. True (b) s < 6ca1 (c) 16 in. < 6(6 in.) = 36 in. True Therefore side-face blowout is considered. Abrg ) $\lambda$  a f c' (17.4.4.1) N sb = 160(6 in.) (2.237 in.2)(1.0) 4500 psi Nsb = 96,319 lb 17.3.3c(ii) Substitute in Eq. (17.4.4.2) (16 in.) (96,319 lb) = 139,127 lb N sbq = (1 + (66 in.) (17.4.4.2) (16 in.) (96,319 lb) = 139,127 lb N sbq = (0.75)(139,127 lb) = 104,300 lb ength iis greater than required Check that design strength: 4 300 lb < Nua,g = 116,000 lb φNsbg bg g = 104,300 NG ered: Two options aree co considered: sta area such that D.5.4 (heff > 2 2.5ca1) • Increase pedestal LVVDWLV¿HGRU rcement. 318 addresses. In Step 10, the concrete breakout strength is recalculated based on a 32 in, x 32 in, pedestal cross section. The shear strength calculations are also based on the increased pedestal cross section. The shear strength calculations are also based on the increased pedestal cross section. section does not require side-face blowout to be considered. ,ISURMHFWOLPLWDWLRQVRUIXQFWLRQDOFRQVWUDLQWVGR not allow an increased cross section, the designer may consider adding reinforcement to resist the side-face blowout force. Providing anchor reinforcement is discussed by Cannon, Godfrey, and Moreadith in "Guide to the 'HVLJQRI\$QFKRU%ROWVDQG2WKHU6WHHO(PEHGments," Concrete International, July 1981. They conclude that the side-face blowout force can be taken equal to 25% of the tensile capacity of the anchor steel and that "reinforcement must be provided to arrest tensile failure of the concrete due to lateral bursting forces at anchor heads near the free surface." They recommend spiral reinforcement (Fig. E18.8). Fig. E18.8). Fig. E18.8.—Spiral to resist side-face blowout. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 363 Eligehausen, Mallée, and Silva report in Anchorage in Concrete Institute – Copyrighted Concrete 2006, that the diameter of the side-face blowout cone is about six times the edge distance. 7KH\QRWHWKDWOLPLWHGVWXGLHVLQGLFDWHWKDWFRQ¿Qing reinforcement increased local side-face blowout failure load. Anchorage 'H9ULHV-LUVDDQG%DVKDQG\UHSRUWLQ3(IIHFWVRI 7UDQVYHUVH5HLQIRUFHPHQWDQG%RQGHG/HQJWKRQ Fig. E18.9—No. 4 ties to improve post-peak performance. the Side-Blowout Capacity of Headed Reinforcement," ACI SP-180, 1998, "a large amount of transverse reinforcement near the head made little difference in the ultimate capacity but substantially improved post-peak performance." (see Fig. E18.9) American Concrete Institute - Copyrighted © Material - www.concrete.org 364 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Step 11: Concrete breakout strength. 17.4.2.1 N cbg = ANc y ec , N y ed , N N  $\psi$  cp , N N b ANco (17.4.2.1b) Spacing s = 16 in. in both directions. Each anchor is close to two edges. Therefore, anFKRUHPEHGPHQWGHSWKLVOLPLWHGWRDVPDOOHU2FWLtious effective depth hef (Fig. E18.10) 1.5 = 5.33 in. and ggreater than 1.5hgef. ANco = 9(5.33) 9 3)2 = 256 in in.2 17.4.2.1 taken greater than nANc where n = ANc shall not be ta Nco, wher hor in a group. gro number of anchors 256 in.2) = 1024 in. in 2 (17.4.2.1c) (4)(256 nANco  $\geq$  ANc OK 17.4.2.4 zec, N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG zec, NN = 1.0 eccentrically in tension; load is concentric. 17.4.2.5 zed, N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; ca1 = ca, min < 1.5hef () 8 in.  $\psi$  ed , N = 0.7 + 0.3 | = 1.0 \ 1.5(5.33 in.) |/ 17.4.2.6 zc, N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked. zc, N = 1.0 17.4.2.7  $zcp,N\pm PRGL cFDWLRQIDFWRUIRUSRVWLQVWDOOHGDQFKRUV$  in uncracked concrete without supplementary zcp,N = 1.0 reinforcement to control splitting. 17.4.2.1 N b = kc  $\lambda$  a 17.3.3c(ii) f c'hef1.5 (17.4.2.2a) N b = (24)(1.0)(4500 psi)(5.33 in.)1.5 (17.4.2.2a) N b = (24)(1.0)(4500 psi)(5.33 in.)1.5 (17.4.2.2a) ANCHORING TO CONCRETE Check that design strength is greater than required strength: Increasing pedestal cross section does not necessarily result in the elimination of concrete breakout failure mode and, therefore, anchor reinforcement is required. 365  $\varphi$ Ncbg = 59,400 lb < Nua,g = 116,000 lb NG Fig. ig. E18.11—Clear space between anchor and corner bar. Step 13: Tension force summary ACI 318 17.4.1 17.4.2 17.4.3 Failuree m mode Steel/anchor /an re with Concrete pullout/anchor out hor Design ign streng strength, lb b []N Nsa 54,500 54 00 Ratio = Nua ua,(g) []Nn 0.53 Ncbg []N 116,000 116 00 -[Npn 60,400 00 00.48 Shear strengths Step 14: Anchor group 17.2.1.1 Check if maximum bolt spacing (16 in.) is close enough (within 3ca1) for the bolts to act as a group for concrete breakout in shear. Step
15: Shear distribution R17.5.2.1(b) Spacing between anchors parallel to the shear force is 16 in. The distance of the anchors closest to the edge is 8 in. Therefore, s > ca1, and according to ACI 318 Fig. R17.5.2.1(b), the anchor design should assume Vua to be resisted (a) evenly by the four bolts (Case 2-Step 17c). (3)(8) = 24 in. ≥ 16 in. Bolts act as a group. American Concrete Institute – Copyrighted © Material – www.concrete.org Controls design? Yes No-anchor reinforcement No Anchorage Step 12: Anchor reinforcement The anchor reinforcement calculation is identical to the exception of the available development length calculation, which is related to the anchor reinforcement The anchor reinforcement No Anchorage Step 12: Anchor reinforcement calculation is identical to the exception of the available development length calculation, which is related to the anchor reinforcement Calculation is identical to the exception of the available development length calculation is identical to the calculation is identical to the calculation is identical to the calculation of the available development length calculation is identical to the calculation is identical to the calculation is identical to the calculation of the available development length calculation is identical to the calculation CONCRETE DESIGN HANDBOOK—SP-17(14) Step 16: Steel shear 17.5.1.2 Nominal steel strength of a headed bolt is the steel tensile strength of a headed bolt is the steel tensile strength of a headed bolt is the steel tensile strength of a headed bolt is the steel tensile strength of a headed bolt is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60. Vsa = (0.6)Ase,Vfuta (17.5.1.2b) Ase,V = 0.969 in.2 The bolt area is obtained from Table 3: 9HULI/futa = 75,000 psi (Table 1a) is not greater than 1.9fya and 125,000 psi. For a 1.25 in. F1554 Grade 36 bolt, either calculate Ase, V futa from Eq. (17.5.1.2b) or obtain from Table 3. 17.5.1.3 17.3.3a(ii) Because grout is located between the steel base plate and the top of concrete pedestal, a 0.8 reduction factor is applied. Strength reduction factor for ductile bolt: 1.9fya = 104,500 psi futa < 1.9fya < 125,000 psi OK Table 3: Ase, V futa = 72,675 lb Vsa = (0.6)(72,675 lb) = 43,605 lb Vsa = (0.6)(43,605 lb) = 34,884 lb  $\varphi$  = 0.65 Vsa = ((0.65)(34,884 lb) = 22,675 lb/anchor  $\varphi$ V ength is greater than required Check that design strength assuming 100% of Vua is applied to the rear 2—7c). two bolts (Case 2—Step 17c). 2 0 lb/anchor > Vua,g = 30,000

lb/2 anchors φVsaa = 22,600 = 115,000 lb/anchor OK American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 367 Step 17a: Concrete breakout \$VKHDUEUHDNRXWIDLOXUHLVDVVXPHGWRLQLWLDWHDW SRLQWVGH¿QHGE\WKHDQFKRUV¶FHQWHUOLQHDQGWKH WRSVXUIDFHDQGWRSURSDJDWHDZD/IURPWKHGH¿QHG SRLQWVDWGHJUHHVERWKKRUL]RQWDOO\DQGYHUWLFDOO\ WRZDUGWKHHGJHV)LJ( &DVH\$OOEROWVUHVLVWVuaHYHQO\2 1RPLQDOFRQFUHWHEUHDNRXWVKHDUVWUHQJWKRIWKH WZRDQFKRUVFORVHVWWRWKHHGJHLV 17.5.2.1 Vcbg = AVc \u03c4 ec ,V \u03c4 ed ,V \u03c4 c ,V \u03c4 h, V b E AVco Fig. E18.12—Shear breakout Case 1. AVcLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUIDLOXUHDUHDRIDVLOJOH AVco 2 4. 5 + ca2 ca1 'H¿OLWLRO DUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcOLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcOLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcoLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVcOLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVCOLVWKHSURMHFWHGVKHDUHDRIDVLOJOH AVCOLVWKHS LQ2 17.5.2.5 DWL FWRUIRUDQFKRUJ RXSV zec,V ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG zec,VV = 11.0 VK HFFHQWULFDOO\LQVKHDU 17.5.2.6 zed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV ORDGHGLQVKHDUWKHDQFKRULVORFDWHGWRRFORVHWR WKHVLGHHGJHVIRUDFRPSOHWHEUHDNRXWSULVPWR GHYHORSzed, V±HGJHPRGLiFDWLRQIDFWRU ca2 < 1.5ca1 (LQ)  $\psi$  ed ,V = + | = \LQ | / zc, V±PRGLiFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW VHUYLFHORDGOHYHOSHGHVWDOWLHVUHVLVWFUDFNLQJDQG DFWDVVXSSOHPHQWDOUHLQIRUFHPHQWWKHUHIRUH zc, V = 1.2 zh, V ± PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ UHODWLRQWRDQFKRUHPEHGPHQWGHSWK zh, V = 1.0 7KHEDVLFVKHDUFRQFUHWHEUHDNRXWVWUHQJWKRID VLQJOHDQFKRULVFDOFXODWHGDVWKHVPDOOHURI (TD DQGE le = hef LQEXWOLPLWHGWRda LQ LQ FRQWUROV (A ) Vb = | e | \ da / 0.2 da \ a ( LQ ) f c'ca1 1.5 D Vb = | \ LQ | 0.2 LQ SVLLQ 1.5 Vb OE American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 'H¿QLWLRQ 368 17.5.2.2 THE REINFORCEE CONCRETE DESIGN HANDBOOK—SP-17(14) Vb = 9 $\lambda$  a f c'(ca1) 1.5 (17.5.2.2b) Vb = 9(1.0) 4500 psi (8 in.) 1.5 = 13,644 lb Solve Eq. (17.5.2.1b) Vcbg = (1.33)(1.0)(0.9)(1.2)(1.0)13,644 lb = 19,598 lb 17.5.2.1 Vcbg = 17.3.3c(i) For a cast-in headed bolt with supplementary reinforcement, Condition A applies:  $\varphi = 0.75 \varphi Vcbg = (0.75)(19,598 lb) = 14,700 lb Check that design strength is greater than required strength is greater than$ Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 369 17.5.2.2 10.7.6.1.6 The lateral shear force is transferred from the anchors to the reinforcement. Shear per anchor: Strut  $\alpha = 45$  degrees 30,000 lb/4 = 7500 lb per anchor Determine tension in pedestal tie Tension in tie = 7500 lb (by inspection) Determine tie area 60 60,000 psi) = 0.17 in. 2 As = 7500 lb/( $0.75 \times 60$ , tie Number of No. 4 ties: Number of No. 4 ties = 0.17/0.20 = one tie /RFDWHWLHV ng length h of the anchor fforr she The load-bearing shear,  $\mathcal{E}e$ , da. is the lesser of hef and 8d  $\mathcal{E}e$  = 10 in. Use two No 4 ties in the top 5 in. of the pedestal to resist the lateral force. Minimum two No. 4 ties are required in the top 5 in. by Code. \$QDOWHUQDWHIRUFHÀRZLVSUHVHQWHGWKDWFDQLQclude a strut to the middle vertical No. 8 bar. In this case, a tension perpendicular to the middle No. 8 bar needs to be resolved (Fig. E18.14). Fig. E18.14—Alternate strut-and-tie model. 2SWLRQ+DLUSLQV Calculate development of U-bars (hairpins) and ensure proper development length within the core of the column (Fig. E18.15). Fig. E18.15). Fig. E18.15). Fig. E18.15). design 17.5.2.9 Column base plate anchor holes are usually oversized. For this condition, ACI 318 permits the designer to assume the two anchors are assumed not to be critical (Case 1) (Fig. E18.13). 370 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 2SWLRQ\$QDOWHUQDWHWRWKHKDLUSLQVLVSUHVHQWHG in Fig. E18.16, diamond ties in the top 5 in. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 371 Step 17c: Case 2 Case 2: Two farthest bolts from edge resist Vua— 7KHDQFKRUVIXUWKHVWIURPWKHHGJHDUHVXEMHFWHGWR 100% of the anchor is: 17.5.2.1 Vcbg = AVc  $\psi$  ed ,V 
$\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$  ed ,V  $\psi$ in.)) AVc = 1152 in.2 AVco = 4.5c2a1 (17.5.2.1c) AVco = (4.5)(24 in.)2 = 2592 in.2 Anchorage Fig. E18.17—Shear breakout: Case 2. Avc 1152 in.2 =  $0.444 \ 0 \ 44 \ Avco \ 225922$  in.2 17.5.2.5 DWLR RUIRUDQFKRUJ XSVORDGHG zec, V±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG zec, V± = 11.0 ncent eccentrically in sh shear; load is applied cconcentric: 17.5.2.6 WLR WRUHGJHHIIHFWV RUDQFKRUV zed, V = 0.7 + 0.3 17.5.2.7 17.5.2.8 ca 2 1.5ca1 (17.5.2.6b))  $\psi$  ed 0 + 0.3 e, V = 0.7 zc,  $V \pm PRGLiFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW$  service load level; pedestal ties resist cracking and act as supplemental reinforcement, therefore:  $z_c$ , V = 1.2  $z_h$ ,  $V \pm PRGL$ ; FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth.  $z_h$ , V = 1.0 8 in. = 0.767 1.5(24 in.) The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b): 17.5.2.2 (A) Vb = 7 | e | \ da / 0.2 + 0.2  $da \lambda a fc'(ca1) 1.5 (10 in.) (17.5.2.2a) Vb = 7 (1.25 in. 1/0.2 1.25 in. 4500 psi(10 in.) 1.5 Vb = 93,560 lb For cast-in anchors, \mathcal{E} = hef and \mathcal{E} (ca1) 1.5 \mathcal{E} = hef and \mathcal{E} (ca1) 1.5 \mathcal{E} = hef = 18 in. and 8da = 8(1.25) = 10 in. (17.5.2.2b) Vb = 91,00 (1.25 in. 1/0.2 1.25 in. 4500 psi(10 in.) 1.5 Vb = 93,560 lb For cast-in anchors, \mathcal{E} = hef and \mathcal{E} (ca1) 1.5 \mathcal{E} (ca1) 1.5 \mathcal{E} = hef and \mathcal{E} (ca1) 1.5 \mathcal{E} (c$ - Copyrighted © Material - www.concrete.org 372 17.5.2.1 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) Solve Eq. (17.5.2.1b) AVco For a cast-in headed bolt with supplementary reinforcement, Condition A applies: Check that design strength is greater than required strength: Vcbg =  $(0.444)(0.767)(1.2)(1.0)(70,898 lb) = 28,873 lb \varphi = 0.75 \varphi Vcbg = (0.75)(28,973 lb) = 21,700 lb < Vua,g = 30,000 lb NG Anchor reinforcement is needed to prevent concrete breakout in shear. The calculation is similar to Case 1 (Step 17b) (Fig. E18.18). Fig. E1 F E18.18—Shear —Shear fo force transfer to$ reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 373 Step 18: Concrete pryout on the side opposite to load direction. The pryout strength is related to the anchor's tension breakout strength Ncb and embedment depth hef. 17.5.3.1  $\varphi$ Vcpg =  $\varphi$ kcpNcpg Ncpg = Ncbg (17.5.3.1b) Nominal pryout strength based on effective depth, hef, is approximately one or two times the anchor nominal concrete breakout strength:  $\rightarrow$  kcp = 2.0 hgef = 5.33 in.  $\geq$  2.5 in. 17.4.2.1 17.3.3.c(i) ANc = (a in. + 1.5(5.33 in.))(32 in.) = 512 in.2 See Step 11 for ANcoDQGDOOPRGL¿FDWLRQIDFWRUV ANco = 9(5.33)2 = 256 in.2 zec,NN = 1.0 zc,N = lb) = 79,256 lb Reduction factor  $\varphi$  for cast-in anchor:  $\varphi$  = 0.75 0.75)(79,25 lb) = 59,400 lb  $\varphi$ Vcpg = (0.75)(79,256 Check that design strength is greater than required strength: Step 19: Shear force summary ACI 318 17.5.1 N cbgg = Failure mode Steel/anchor Concrete breakout/group (Case 1) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 1) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete breakout/group (Case 2) Concrete
breakout/group (Case 2) Concrete breakout/group (Case 2) Co pryout/group  $\varphi$ Vcpg = 59,400 > Vua,g = 30,000 lb Design strength [Vsa 22,600 15,000 [Vcpg 0K Ratio = Vua,(g)] Vn 0.66 Controls design? Yes — Supplemental reinforcement would preclude concrete pryout failure mode. Step 20: Interaction of tensile and shear forces 17.6.1 15,000 lb/22,600 lb = 0.66 > 0.2 Check if: Vua,g/[]Nn" 29,000 lb/54,500 lb = 0.53 > 0.2 Check interaction of shear and tension: Therefore, full shear design is not permitted. N ua , g  $\phi$ N n + Vua , g  $\phi$ Vn  $\leq 1.2$  29,000 lb/54,500 lb = 0.53 > 0.2 Check if: Vua,g/[]Nn" 29,000 lb/54 lb + = 1.19 ≤ 1.2 54,500 lb 22,600 lb American Concrete Institute - Copyrighted © Material - www.concrete.org OK Anchorage 0RGL¿HGHIIHFWLYHHPEHGPHQWGHSWKLVFDOFXODWHG in Step 6. 374 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 21: Discussion To transfer the uplift force from the headed anchors to the pedestal reinforcement, two possible conditions were addressed: 1. Adding supplemental reinforcement, and 2. Increasing pedestal cross section However, there is a third condition to resolve this problem by increasing the length of the headed anchors, VXFKWKDWWKHGHYHORSPHQWOHQJWKLVVXI¿FLHQWWRWUDQVIHUWKHXSOLIWIRUFHIURPWKHDQFKRUWRWKHSHGHVWDO reinforcement. American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15-ANCHORING TO CONCRETE 375 Anchorage Example 19: Post-installed adhesive column anchors resisting tension and shear forces A normalweight reinforced concrete pedestal with fc' = 4500 psi supports a W14x68 steel column, which is welded to a 1 ft 9 in. x F1554 Grade 55 post-installed adhesive anchors. Anchors have an embedment depth of 18 in. The pedestal is reinforced with eight No. 4 ties in the top 5 in. (refer to Fig. E19.1). The column anchors are resisting 30,000 lb service gravity dead load, 30,000 lb service gravity live load, 35,000 lb uplift wind load, 6000 lb service lateral dead load, 6000 lb service lateral live load, and 12,000 lb lateral wind load. The structure is assigned to Seismic force is negligible. Check the adequacy of the connection. Given: Anchorage Loads— DV = 30,000 lb (service gravity dead load) LV = 30,000 lb (service seismic force) lb gravity live load)  $WV = \pm 35,000$  lb (vertical wind)  $DH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service
lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 6000$  lb (service lateral dead load)  $LH = \pm 600$ 16 in. in both Anchor embedment length hef = 18 in. Anchor diameter da = 1.25 in. Pedestal— fcg SVL Dimensions: b x b x h = 28 in. x 28 in. x 45 in. Reinforcement: eight No. 8 vertical bars and two No. 4 at 12 in. on-center ties and two No. 4 ties in the top 5 in. 3a = 1.0 (normalweight concrete) Fig. E19.1—Headed anchors resisting shear and tension from a steel column. American Concrete Institute – Copyrighted © Material – www.concrete.org 376 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ACI 318-14 Discussion Step 1: Anchor group 5.3.1 9HUWLFDO: U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.0E + 1.0(Lr) U = 1.2D + 1.0(Lr) U = 1.2D +U = 0.9D + 1.0W U = 0.9D + 1.0E Calculation (5.3.1a) (5.3.1b) (5.3.1c) ( tension force (upward). Equations (5.3.1a), (5.3.1b), (5.3.1e), (5.3.1c), ( 22.2 kip (5.3.1f) Structure is assigned to SDC A; therefore, seismic requirements of 17.2.3 do not apply. Step 2: Group action g in grou 17.2.1.1 Check if bolts are acting group or single in tension: conc Anchor spacing for r concrete breakout in tension single in tension single in tension. uncr 1100 Required tension strength is Nua, g = 8000 lb Required shear strength is Vua, g = 22,800 lb 18) = 54 in.  $\geq 16$  in. OK; anchors act as a group (3)(18) 1530 psi 17.4.5.1d) cNa = 10(1.25 (17.4.5.1d) 10 in.)) in = 14.74 in. 1100 psi 11 IJuncr = 1530 psi is based on test results and is obtained from Table E.2. (2)(14.74 in in.) = 29.48 in. > 16 in. OK; anchors act as a group 17.7 Supplementary reinforcement is provided; therefore, 17.7.1 through 17.7.6 do not need to be VDWLV¿HG Step 3: Strength inadequacies 17.3.1.1 The anchor design strengths must satisfy the following inequalities: N ua, g [ $\phi$ N sa (steel strength in tension) |  $\leq$  { $\phi$ N cbg (concrete breakout) | [ $\phi$ N ag (bond strength) Nua, g = 8000 lb  $\left[\phi Vsa \text{ (steel strength in shear)}\right] \leq \left\{\phi Vcbg \text{ (concrete breakout)}\right\} = 22,800 lb and Vua, g =
22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vua, g = 22,800 lb and Vu$ CHAPTER 15—ANCHORING TO CONCRETE 377 Step 4: Anchor ductility to determine the  $\varphi$  factor. 2.2 Check if ASTM F1554 Grade 55 has the following properties: 14% minimum elongation, and 30% minimum area reduction Elongation at 2 in. = 21% > 14% min. Reduction of area =  $30\% \ge 30\%$  min. Therefore, F1554 Grade 55 is ductile. Step 5: Steel tensile strength (futa) times the anchor area (Ase,N). (17.4.1.2) Nsa = Ase,N futa The stud area is obtained from Table 3: 17.3.3a(i) da = 1.25 in. Ase,N = 0.969 in.2 1.9 fya = 1.25 in. Ase,N = 0.969 in.2 1. 1.9(55,000 psi) = 104,500 psi able 1a) is smaller than Check if futa = 75,000 psi (Table 1.9 fya and 125,000 psi) of futa < 1.9 f meter ASTM F1554 Grade 55 For 1-1/4 in. diameter mina tensile le strength Nsa is obtained Check that design strength is greater than required strength:  $\varphi = 0.75 \varphi$  (72,675 lb lb) = 54,506 lb/anchor  $\varphi$ Nsa = 54,500 lb/anchor > Nua,g = 2000 lb/anchor > Nua,g = Anchorage 17.4.1.2 378 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout 17.4.2.1 N cbg = ANc  $\psi$  c, N VPDOOHU¿FWLWLRXVHIIHFWLYHGHSWKh'ef, calculated as the larger of: Fig. E19.2—Projected breakout of a group of anchors. [ ca , max | | hef' (Fig. E19.2) the larger of { 1.5 | smax | [ 3 6 in./1.5 =
4.0 in. 'H¿QLWLRQ ANCLVWKHSURMHFWHGDUHDRIWKHIDLOXUHVXUIDFH,Q this problem, ANc is the area of the pedestal. Therefore, h'ef  $256 \text{ in.2} = 1024 \text{ in.2} > 784 \text{ in.2} 6 \text{ in./3} = 5.33 \text{ in.} 17.4.2.4 \text{ zec,N} \pm PRGLiFDWLRQIDFWRUIRUDQFKRUJ RXSV oncentric, 17.4.2.5 zed,N \pm PRGLiFDWLRQIDFWRUIRUDQFKRUJ RXSV oncentric, 18.5.5.5 zed,N \pm PRGLiFDWLRQIDFWRUIRUDQFKRUJ RXSV oncentric, 18.5.5.5 zed,N \pm PRGLiFDWLRQIDFWRUIRUDQFKRUJ RXSV oncentric, 18.5.5.5 zed,N \pm$ + 0.3 17.4.2.6 17.4.2.7 17.4.2.2 ca,min 1.5hef (17.4.2.5b)  $\psi$  ed, N = 0.7 + 0.3 zc,N±PRGL¿FDWLRQIDFWRURIFRQFUHWHFRQGLWLRQDW service load level; concrete is assumed cracked and kc is taken from test report: zc,N = 1.0 zcp,N±PRGL¿FDWLRQIDFWRURIFRQFUHWHFRQGLWLRQDW service load level; concrete is assumed cracked concrete without supplementary reinforcement to control splitting. Ties are provided. zcp, N = 1.0 Basic concrete breakout strength of a single anchor in tension in cracked concrete is: N b = kc  $\lambda$  a f c'hef1.5 (17.4.2.2a) N b = (17)(1.0) (Controls OK 6 in. = 0.925 1.5(5.33 in.)) 4500 psi (5.33 in.) 1.5 = 14,041 lb From Table E.2: kc = 17, based on test data. Substitute into Eq. (17.4.2.1b): N cbg = ANc  $\psi$  ec , N  $\psi$  ec , N  $\psi$  ec , N  $\psi$  cr , N  $\psi$  that design strength is greater than required strength:  $\varphi = 0.65 \varphi Ncbg = (0.65)(39,775 lb) = 25,854 lb \approx 25,900 lb \varphi Ncbg = 25,900 lb > Nua,g = 8000 lb OK Anchorage 17.3.3c(ii) 379 American Concrete Institute - Copyrighted © Material - www.concrete.org 380 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Bond$ strength 17.4.5.1 For post-installed adhesive anchors, data from DQFKRUSUHTXDOL¿FDWLRQWHVWLQJPXVWEHXVHG N ag = ANa  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na  $\psi$  ec , Na  $\psi$  ed , Na kc,cr = 17 kc,uncr = 24 []d = 0.55 'H¿QLWLRQ ANaoLVWKHSURMHFWHGLQÀXHQFHDUHDRIDQDGKHVLYH anchor with an edge distance > 2cNa. From Step 2, cNa = 14.7 in. rom Eq. (1 (17.4.5.1c) and ANao can be calculated and can be calculated by the
calculated by the calculated by from (17.4.5.1d) ANao = (2cNa)2 cNa = 10d a 'H¿QLWLRQ 17.4.5.2 (17.4.5.1c) 4.5.1c) 4.5.1c) ANaoo = ((2(14.7 7 in.))2 = 8 864 in.2 τ uncr ncr 00 1100 (17 (17.4.5.1d) GL FH DUHD RI DQ VLYH ANALVWKHSURMHFWHGLQÀXHQFHDUHDRIDQDGKHVLYH group of anchors and is approximated as a rectilinHDUDUHDWKDWSURMHFWVRXWZDUGDGLVWDQFHcNa from p but the centerline of the adhesive anchor group, limited by the edge distance (Fig. E19.3). In this problem, ANa is the area of the pedestal. ANa = 28 in. n. x 28 in in. = 784 in.2 ANa 784 in.2 = 0.907 ANao 864 in.2 The basic bond strength is obtained 3aIJcrAdahef (17.4.5.2) Nba = (1.0)(700 psi)π(1.25 in.)(18 in.) = 49,480 lb where IJcr = 700 psi is obtained from Table E.2. 17.4.5.3 17.4.5.4 zec,Na = 1.0 zed,Na  $\pm$ PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRU edge effect. Because the anchor is located close to an edge (ca,min < cNa), its strength is reduced by the following equation:  $\psi$  ed , Na = 0.7 + 0.3 ca,min cNa (17.4.5.5b)  $\psi$  ed , Na = 0.7 + 0.3 6 in. = 0.822 in. 14.7 in. American Concrete Institute – Copyrighted © Material -CHAPTER 15—ANCHORING TO CONCRETE zcp,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVDW cracked service load level without supplementary reinforcement to control splitting; assume member is cracked and not detailed. For an adhesive anchor designed for uncracked concrete without supplementary reinforcement, a PRGL¿FDWLRQIDFWRULVDSSOLHGWRSUHFOXGHEULWWOH splitting failure. In this example, concrete is FUDFNHGZLWKVXI¿FLHQWUHLQIRUFHPHQWWRUHVWUDLQ crack widths; therefore: 17.4.5.1b): N ag = ANa ψ ec , Na ψ ed , Na ψ ec , Na ψ ed , Na ψ ec , Na ψ ed , Na ψ (49,480 lb) = 36,890 lb The reduction factor is obtained from test data; refer to Table E.2: Check that design strength is greater than required strength: Step 8: Summary—tension forces ACI 318 17.4.1 17.4.2 17.4.5 zcp,Na = 1.0 Failure re m mode Steel/anchor /a Concrete bre breakoutt wit with nfo ment/group supplemental reinforcement/group nd Adhesive bond/group  $\varphi = 0.55 \varphi$ Nag = (0.55)(36,890 lb) = 20,200 lb  $\cong$  20,200 lb  $\Rightarrow$  Nua,g a,g = 8000 lb Design esign strength, lb []N Nsa 54,500 54 500 Anchorage 17.4.5.5 381 OK Ratio = Nua ua,(g)[]Nn 00.04 04 Controls design? No Ncbg []N 25,900 25 900 0.31 No []Nag 20,200 200 0.4 0 Yes Shear strengths Step 9: Group action 17.2.1.1 Check if bolts are acting in group or single in shear: \$QFKRUVSDFLQJIRUFRQFUHWHEUHDNRXWLQVKHDU"ca1 (3)(6) = 18 in. ≥ 16 in. American Concrete Institute – Copyrighted © Material – www.concrete.org OK; anchors act as a group 382 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Steel shear Nominal steel strength of a headed bolt is the steel tensile strength (futa) times the bolt area (Ase,V), multiplied by 0.60.17.5.1.2 Vsa = (0.6)Ase,V futa (17.5.1.2b) The anchor area is obtained from Table 3: Ase,V = 0.969 in.2 futa = 75,000 psi is greater than 1.9fya and 125,000 psi is greater than 1.9fya and 125,000 psi is greater than 1.9fya = 104,500 psi futa < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya = 104,500 psi is greater than 1.9fya and 125,000 psi is greater than 1.9fya = 104,500 psi is greater than 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9fya < 1.9f 125,000 psi OK For 1-1/4 in. F1554 Grade 36 anchor, either calculate Table 3: Ase,V futa OEZLWKRXWPRGL¿FDWLRQ Ase,V futa from Eq. (17.5.1.2b) or obtain from Table 3: factors 17.5.1.3 17.3.3a(ii) Because a grout is located between the steel base plate and the concrete pedestal, a 0.8 reduction factor is required. Strength reduction factor for ductile bolt: Vsa = (0.6)(72,675 lb) = 43,605 lb  $Vsa = (0.8)(43,605 lb) = 34,884 lb \varphi = 0.65 Vsa = ((0.65)(34,884 lb) = 22,675 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor
\varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V \varphi V Vsa = 22,600 lb/anchor \varphi V Vsa = 22,600 lb/anchor \varphi V Vsa = 22,600 l$ required strength:  $\phi$  b/anchor > Vua,g = 22,800 lb/2  $\phi$ Vsaa = 222,600 lb/anchor  $\phi$  saa = 222,600 lb/anchor > Vua,g = 11,400 lb/anchor  $\phi$  saa = 222,600 lb/anchor > Vua,g = 11,400 l load is greater than the distance of the anchors closest to the edge: s > ca1 17.5.2.1 383 16 in. > 6 in. Per ACI 318, two cases must be checked (Fig. R17.5.2.1(b)): Case 1: Half the shear force is applied to the two anchors closest to the edge. Case 2: The full shear force is applied to the anchors furthest from the edge in the direction of the shear force A shear breakout failure is assumed to initiate at points GH¿QHGE\WKHDQFKRUV¶FHQWHUOLQHDQGWRSURSDJDWHDZD\ IURPWKHGH¿QHGSRLQWVDWGHJUHHVERWKKRUL]RQWDOO\ and vertically toward the edges (Fig. E19.4). Fig. E19.4—Shear breakout: Case 1. Anchorage Case 1. 7KHDQFKRUVFORVHVWWRWKHHGJHDUHVXEMHFWHGWR 50% of the anchor force eakout shear strength of a Nominal concrete breakout single anchor is: Vcbg = AVc ψ ec ,V ψ ed ,V ψ c ,V ψ h ,V Vb AVco (17.5.2.1b) 'H¿QLWLRQ 17.4.5.1 HG FHDUHDUHODWH DVKHDU AVcoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHD breakout for a single anchor Eq. (17.4.5.1c). 'H¿QLWLRQ F R V AVcLVWKHSURMHFWHGDUHDIRUDJURXSRIVHYHUDODQFKRUV AVc = 1.5(6 in.)(2(6 in.) + 16 in.) = 252 in. 2 in. = 252 in. = $DQFKRUFRQiJXUDWLRQ zed, V \pm PRGLiFDWLRQIDFWRUHGJHHIIHFWVIRUDQFKRUV loaded in shear; check if ca2 \ge 1.5ca1 (f in.)2 = 162 in. 2 avco = 4.5(6 Avc 252 in.2 = 1.56 Avco 162 in.2 zec, V = 1.0 ca2 = 6 in. < 1.5ca1 = 1.5(6 in.) = 9 in.$  $6 \text{ in.} = 0.9 (17.5.2.6b) \psi \text{ ed}, V = 0.7 + 0.3 | 1.5(6 \text{ in.}) | zc, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement: zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement; zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQFUHWHFRQGLWLRQDW service load level; because the pedestal has No. 4 ties that act as supplementary reinforcement; zc, V = 1.2 zh, V \pm PRGL iFDWLRQIDFWRUIRUFRQFUHWHFRQF$ ha = 45 in.> 1.5ca1 = 1.5(6 in.) = 9 in. zh, V = 1.0 American Concrete Institute – Copyrighted © Material – www.concrete.org 384 17.5.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) ( (A) 0.2 ) Vb = 17 e | da |  $\lambda$  a | ( \ da / |) f c'(ca1) 1.5 (17.5.2.2a) Vb = 7 (| 10 in. ) | (1.25 in.) 0.2 (1.25 in.) 4500 psi(6 in.) 1.5 For cast-in anchors, & E = hef and & E'' da and & \lambda = 1.0 & E = hef = 18 in. and & 8da = 8(1.25) = 10 in. Vb = 9\lambda a f c'(ca1) 1.5 (17.5.2.2b) Vb = 9(1.0) (Vb = 8848 lb 17.5.2.1) 4500 psi (6 in.) 1.5 = 8848 lb Controls Solve Eq. (17.5.2.1b): Vcbg = 17.3.3c(i) Controls AVc  $\psi$  ec,  $V \psi$  ed,  $V \psi$  c,  $V \psi$  h, V Vb AVco (17.5.2.1b) Vcbg = (1.56)(1.0)(0.9)(1.2)(1.0)(8848 lb) = 14,907 lb lementary reinFor a post-installed bolt with supplementary plies: forcement, Condition A applies: n str Check that design strength is greater than required strength: 10.7.6.1.6 o. 4 ties placed in the to Check if two No. top 5 in in. of the qu to o resist the facto ed shear pedestal are adequate factored force. 21.2.1 The reduction factor for the design of anchor reinforcement is:  $\varphi = 0.75$  (0.75)(14,907 7 lb) lb = 11,180 lb < Vua, g = 22,800 lb/2 = 11,400 lb NG \varphi = 0.75 Column base plate anchor holes are usually oversized. The two anchors closest to the edge in the direction of the shear force resist 1/2 the wind force, while the rear anchors are assumed not to be critical (Case 1) (Fig. E19.4). The lateral force is transferred from the anchors to the reinforcement by struts (S1 in Fig. E19.5)). Fig. E19.5—Force transfer from anchors to reinforcement American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.5.2.2(a) 10.7.6.1.6 Shear per anchor: Strut  $\alpha$  = 45 degrees Determine tension in tie = 5700 lb/anchor Tension in tie = 5700 lb/anchor Tension in te = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/anchor Tension in te = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700
lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 lb/anchor Tension) in tie = 5700 lb/(0.75 × 1.1.400 lb/2 = 5700 60,000 psi) = 0.13 in.2 Number of No. 4 ties = 0.13/0.20 = one tie /RFDWHWLHV The load-bearing length of the anchor for shear,  $\mathcal{E}_e$ , is the lesser of hef and 8da.  $\mathcal{E}_e$  = 10 in. 385 Use two No. 4 ties in the top 5 in. by Code. Case 2: 7KHDQFKRUVIXUWKHVWIURPWKHHGJHDUHVXEMHFWHGWR 100% of the anchor force. Vcbg = AVc ψ ec ,V ψ ed ,V ψ ec ,V ψ ed ,V ψ ec ,V ψ h ,V Vb AVco (17.5.2.1b) (17 5 2 1b) Fig. E19 E19.6—Shear —Shear breakout: Case 2. 'H¿QLWLRQ 17.4.5.1 FWH IDFHDUHDUHODWH RDVKHDU AVCoLVWKHSURMHFWHGVXUIDFHDUHDUHODWHGWRDVKHDU nchor Eq. (17.4. 1c). breakout for a sin single anchor (17.4.5.1c). 'HcQLWLRQ WHG DIRUDJURXSRIVHYHUDO 9.6 anchors (Fig. E19.6). AVVco = 44.5(6 in. + 16 in in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in. + 6 in.)(6 in. + 16 in. + 16 in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in. + 16 in.)(6 in. + 16 in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in.)(6 in. + 16 in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in.)(6 in. + 16 in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.)(6 in. + 16 in.))2 = 2178 in.2 AVc = 1.5(16 in. + 6 in.)(6 in. + 16 in.)(6 in.)(6 in. + 16 in.)(6 in.)(6 in.)(6 in.)(7 i in.) = 924 in.2 AVc = 1.5ca1(ca2 + s + ca2) AVVc 924 in.2 = 0.424 AVco 2178 in.2 17.5.2.5 17.5.2.6 zec,V ± PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSORDGHG HFFHQWULFZLWKUHVSHFWWR DQFKRUFRQ¿JXUDWLRQ zec,V = 1.0 zed,V ± PRGL¿FDWLRQIDFWRUIRUHGJHHIIHFWVzed,V, is 1.0 if:  $ca2 \ge 1.5ca1$ (17.5.2.6a) ca2 = 6 in. < 1.5ca1 = 1.5(22 in.) = 33 in. If not, zed, V can be calculated from Eq. (17.5.2.6b)  $\psi$  ed, V = 0.7 + 0.3 6 in. = 0.755 33 in. zc, V±PRGL¿FDWLRQIDFWRUIRUFUDFNHGFRQFUHWHzc, V, is 1.2 because the pedestal has No. 4 ties that act as supplementary reinforcement: zc,V = 1.2 zh,V±PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQUHOD- ha = 45 in. > 1.5ca1 = 1.5(22 in.) = 33 in. tion to anchor embedment depth. Check if ha > 1.5ca1 = 1.5(22 in.) = 33 in. tion to anchor embedment depth. is: 386 17.5.2.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) The basic shear concrete breakout strength of a single anchor is calculated as the smaller of Eq. (17.5.2.2a) and (17.5.2.2b) (A ) Vb = 7 | e | \d / 0.2 da  $\lambda$  a f c'(ca1) 1.5 a (17.5.2.2a) Vb = 7 | l0 in. | \ 1.25 in. | For cast-in anchors,  $\mathcal{E} =$  hef and  $\mathcal{E}''$  da  $Vb = 9\lambda$  a f c'(ca1)  $(62,120 \text{ lb}) \text{ Vcbg} = 23,863 \text{ lb} \text{ For a an adhesive anchor with supplementary reinforcement, Condition A applies: Check that design strength is greater than required strength is greater than require$ resist the factore pedestal are adequate factored shea shear force. ctor for the design of anchor reinforcement is needed to prevent concrete breakout in shear. The calculation is similar to Case 1 (Fig. E19.7). Fig. E19.7). Fig. E19.7). Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 387 Step 12: Concrete pryout strength is related to the anchor's tension breakout strength Ncb and embedment depth hef.  $\varphi$ Vcpg =  $\varphi$ kcpNcpg (17.5.3.1b) Nominal pryout strength based on effective depth hef is one or two times the anchor nominal concrete breakout strength: Kgef = 5.33 in.  $\geq$  2.5 in. Ncpg = min[Ncbg, Nag] Concrete breakout in tension Ncbg: N ag = ANa  $\psi$  ec , Na  $\psi$ 
ec , Na  $\psi$  e  $6 \text{ in.})(6 \text{ in.} + 8 \text{ in.}) = 392 \text{ in.} 2 \text{ ANco} = 9(5.33)2 = 256 \text{ in.} 2 \text{ From Step } 6: \text{zec,NN} = 1.0 \text{ zed,N} = 0.925 \text{ zc,NN} = 1.0 \text{ zed,N} = 1.0 \text{$ = ANa  $\psi$  ec, Na  $\psi$  ed, Na  $\psi$  ed, Na  $\psi$  ed, Na  $\psi$  ed, Na  $\psi$  cp, Na N ba ANao (17.4.5.1b) ANa = (ca2 + s + ca2)(ca1 + cNa) ANco = 4.5(cNa)2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (6 in. + 16 in.)(6 in. + 14.7 in.) = 580 in.2 ANa = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: zec, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 zed, Na = 1.0 z Substituting in Eq. (17.4.5.1b): Nag = (0.6)(0.822)(1.0)(49,240 lb) = 24,374 lb Ncpg = min[19,900 lb, 24,300 lb] = 29,850 lb  $\approx$  29,800 lb Check that design strength is greater than required strength:  $\varphi$ Vcpg = 29,800 lb > Vua,g = 22,800 lb American Concrete Institute - Copyrighted © Material - www.concrete.org OK 388 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 13: Shear force summary ACI 318 17.5.1 17.5.2 17.5.3 Failure mode Steel strength/anchor Concrete breakout/group (Case 1) Concrete breakout/group (Case 2) Concrete pryout/group Design strength, lb []Vsa 22,800 []Vcbg 22,800 [ 0.2 Therefore, full tension design is not permitted. Check if: Nua,g/[]Nn" Check interaction of shear and tension: N ua, g Ratio = Vua,(g)[]Vn 0.42 — 0.77 8000 lb/20,200 lb = 0.4 > 0.2 Therefore, full shear design is not permitted. (17.6.3) 8,000 lb 22,800 lb + = 1.17 < 1.2 20,200 lb 29,800 lb OK Step 15: Discussion When designing anchorage, anc diameter and embedment depth are assumed, and then possible failure modes are checked. Design is revised until satisfactory results are obtained. This example 18. A sm diameter and shallower embedsmaller anchor diam ia ment depth may be aadequate to resist the applied forces due ue to the addition of
supplemental reinforcement. Usually pryout fa failure mode occurs in shallow embedment allow anchor emb ment. The presence and embedment may prec preclude the concrete pryout failure mode. This example relies on anchor reinforcement. The engineer needss to prope properly detail the bars (Step 18). American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15-ANCHORING TO CONCRETE 389 Anchorage Step 16: Pedestal detailing Fig. E19.8-Pedestal reinforcement based on anchor requirements. American Concrete Institute - Copyrighted © Material - www.concrete.org 390 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) TABLES Table 1a—Materials for headed anchors and threaded rods Materials for headed anchors and threaded rods ASTM A29/A29M-05 A108-03 ASTM F1554-04 (H, HD, T)\* ASTM A307-04 (Grade A: HD) (Grade A: HD) (Grade B: H, T) ASTM A36-05 (H, T) ASTM A449-04b (H, HD, T) Grade or type Diameter in. Tensile strength, minimum ksi Yield strength, minimum  $125\ 105\ 0.2\%\ 16\ 4D\ 50\ 2-1/2\ to\ 4\ 115\ 95\ 0.2\%\ 16\ 4D\ 50\ 2YHUWR\ 100\ 75\ 00.2\%\ 2\%\ 18\ 4D\ 50\ A\ 1/4\ to\ 4\ 58\ 6\ 36\ -23\ 2\ in\ in\ -B\ 7\ 1\ to\ 4\ 58\ 6\ 36\ -23\ 2\ in\ in\ -B\ 7\ 1\ 120\ 92\ 0.2\%\ 14\ 4D\ 35\ 2YHU\ to\ 1-1/2\ 105\ 81\ 0.2\%\ 14\ 4D\ 35\ 14\ 4D\ 35\ 2YHU\ to\ 1-1/2\ 105\ 81\ 0.2\%\ 14\ 4D\ 35\ 4D\ 35\ 35\ 4D\ 35\$ = hooked bolt, HD = headed bolt, and T = threaded rod. † Diameters larger than 2 in. (up to 4 in.) are available, but the reduction of area will vary for Grade 55. Comments Structural Welding Code – Steel, Section 7, covers welded, headed or welded bent studs. AWS D1.1 requires studs to be made from cold drawn bar stock conforming to the 3UHVVXUH6HUYLFHDQG2WKHU6SHFLDO Purpose Applications": Grade B7 is an alloy steel for use in high temperature service. \$670\$<sup>3</sup>6WDQGDUG6SHFL¿FDtion for Carbon Steel Bolts and Studs, 60000 PSI Tensile Strength": \$&,VSHFL¿HVWKDWVWHHOHOHPHQWV meeting the requirements of ASTM A307 shall be considered ductile. Note that Grade C conforms to tensile properties for ASTM A36/A36M. ASTM A36/A36M, "Standard Speci¿FDWLRQIRU&DUERQ6WUXFWXUDO6WHHO' Since it is the basis for ASTM A307 Grade C. \$670\$<sup>3</sup>6WDQGDUG6SHFL¿FDtion for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use": This specicFDWLRQLVIRUJHQHUDOKLJKVWUHQJWK applications. Note: Taken from "Guide for Design of Anchorage to Concrete: Examples using ACI 318 Chapter 17 (ACI 355.3R-11)," American Concrete Institute, Farmington Hills, MI, 2011, 124 pp. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 391 Table 1b—Hex head bolt and hex nuts with washers\* Threaded bolt Bearing area Abrg† Hex bolt or hex nut Bolt diameter da, in. Gross area of bolt unthreaded AD†\$, in.2 Width F, in. Area AH†, in.2 Hex head bolt or threaded rod with hex nut<sup>+</sup>, in.2 1/4 0.049 20 0.032 0.438 0.166 0.117 3/8 0.110 16 0.078 0.563 0.274 0.164 1/2 0.196 13 0.142 10 0.334 1.125 1.096 0.654 7/8 0.601 9 0.462 1.313 1.492 0.891 1 0.785 8 0.606 1.500 1.949 1.163 1-1/8 0.994 7 0.763 1.688 2.466 1.472 1 1/4 1.227 7 0.969 1.875 3.045 1.817 1-3/8 1.485 6 1.16 2.063 3.684 2.199 1-1/2 1.767 6 1.41 2.250 4.384 2.617 || 3.562|| 4.653|| 1-3/4 || 2|| 2.405 5 1.90 2.625 5.967 3.142 4-1/2 2.50 3.000 7.794|| \* All washers need to meet the minimum thickness requirements of ACI 318, 17.4.2.8 or the bolt/nut bearing area may conservatively be used to calculate Abrg. † Abrg = AH - AD. Ase, N = Ase, V Ada - (0.9743/n))2, where n is the number of threads. § || Ase, N = Ase, V Ada)2 Applies to hex head bolt only. Note: Dimensions and data taken from ANSI 18.2.1. 1 and 18.2.2. American Concrete Institute - Copyrighted © Material - www.concrete.org Anchorage ‡ 392 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Table 1c—Heavy hex nuts with washers\* Threaded bolt Heavy hex nuts with washers\* Threaded bolt Heavy hex nut Bearing Area Abrg† Bolt diameter da, in. Gross area of bolt unthreaded AD§†, in.2 Width F, in. Area AH†, in.2 Width F, in. Area AH†, in.2 Width F, in. Area AH†, in.2 Width F, in. Area AH†, in.2 Width F, in. Area AD†‡, in.2 Width F, in.2 Width F, in. Area AD†‡, in.2 Width F, in. Area AD†‡, in.2 Width F, in. Area AD†‡, in.2 Width F hex nut<sup>+</sup>, in.2 1/4 0.049 20 0.032 - - 0.500 0.217 0.167 || 3/8 0.110 16 0.078 - - 0.688 0.409 0.299 || 1/2 0.196 13 0.142 0.875 0.663 0.467 5/8 0.307 11 0.226 1.063 0.978
1.063 0.978 1.078 1.078 1.078 1.078 1.078 1.078 1-1/8 0.994 7 0.763 1.813 2.845 1.813 2.845 1.813 2.845 1.851 1-1/4 1.227 7 0.969 2.000 3.464 2.237 1-3/8 1.485 6 1.16 2.188 4.144 2.659 1-1/2 1.767 6 1.41 2.375 4.885 3.118 1-3/4 2.405 5 1.90 2.750 6.549 2.750 6.549 2.750 6.549 4.144 2 3.142 4-1/2 2.50 3.125 8.457 3.125 8.457 5.316 \* All washers need to meet the minimum thickness requirements of ACI 318, 17.4.2.8 or the bolt/nut bearing area may conservatively be used to calculate Abrg. † Abrg = AH - AD. ‡ Ase, N = Ase, V Ada)2 || Applies to threaded rod with hex nut only. 8.2.1 and 18.2.2, and Table 7-20 of the AISC Steel Construction ction Manual. Note: Dimensions and data taken from ANSI 18.2.1 American Concrete Institute - Copyrighted © Material - www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 393 Table 1d—Square and heavy square head bolt and square and heavy square head bolt and square and heavy square head bolt and square and heavy square head bolt and square and heavy square head bolt and square and heavy square head bolt and square and heavy square head bolt and sq of threads per in. (n) Square headed bolt Area AD<sup>++</sup>, in.2 Square nut Width F, in. Area AH<sup>+</sup>, in.2 Square nut Width F, in. Area AH<sup>+</sup>, in.2 Square nut Width F, in. Area AH<sup>+</sup>, in.2 Square nut Width F, in. Area AH<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square nut Width F, in. Area AH<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bolt<sup>+</sup>, in.2 Square headed bol 0.092 0.142 0.201 3/8 0.110 16 0.078 0.563 0.316 0.625 0.391 0.688 0.473 0.206 0.280 0.362 1/2 0.196 13 0.142 0.750 0.563 0.813 0.660 0.875 0.766 0.366 0.464 0.569 5/8 0.307 11 0.226 0.938 0.879 1.000 1.003 1.129 0.572 0.693 0.822 3/4 0.442 10 0.334 1.125 1.266 1.250 1.563 0.824 0.824 1.121 7/8 0.601 9 0.462 1.313 1.723  $1.313\ 1.723\ 1.438\ 2.066\ 1.121\ 1.121\ 1.465\ 1\ 0.785\ 8\ 0.606\ 1.500\ 2.250\ 1.625\ 2.641\ 1.465\ 1.855\ 1.-1/8\ 0.994\ 7\ 0.763\ 1.688\ 2.848\ 1.854\ 2.291\ 1.-1/4\ 1.227\ 7\ 0.969\ 1.875\ 3.516\ 2.000\ 4.000\ 2.288\ 2.773\ 1.3/8\ 1.485\ 6\ 1.16\ 2.063\ 4.254\ 2.188\ 4.785\ 2.769\ 2.769\ 3.300\ 1.-1/2$ 1.767 6 1.41 2.250 5.063 2.250 5.063 2.250 5.063 2.250 5.063 2.375 5.641 3.295 3.295 3.295 3.873 \* All washers need to meet the minimum thickness requirements of ACI 318, 17.4.2.8 or the bolt/nut bearing area may conservatively be used to calculate Abrg. + Abrg = AH - AD. + Ase, N = Ase, V Ada - (0.9743/n))2, where n is the number of threads.. Ase, N = Ase, V Ada)2 Anchorage § Note: Dimensions and data taken from ANSI 18.2.1 and 18.2.2. 18 Table 1e—Dimensional properties and bearing area of headed studs\* Stud diameter da, in. /HQJWKUHGXFWLRQ due to weld, in. Head thickness H, in. 1/4 1/8 3/8 1/8 1/2 Head diameter dh, in. Stud area AD<sup>++</sup>, in. 2 0.187 0.5 0.049 0.196 0.147 0.281 0.75 0.110 0.442 0.331 1/8 0.312 1 0.196 0.785 0.589 5/8 3/16 0.312 1.25 0.307 1.227 0.920 3/4 3/16 0.375 1.25 0.442 1.227 0.785 7/8 3/16 0.375 1.25 0.442 1.227 0.785 7/8 3/16 0.375 1.270 0.920 3/4 3/16 0.375 1.270 0.920 3/4 3/16 0.375 1.250 0.442 1.227 0.785 7/8 3/16 0.375 1.270 0.920 3/4 3/16 0.375 1.270
0.920 3/4 3/16 0.375 1.270 0.920 3/4 3/16 0.375 1.270 0.920 3/4 3/16 0.920 3/1 www.NelsonStudWelding.com, 110 pp. ‡ Ase, N = As 36 58 2842 6380 11,368 17,980 25,520 34,800 45,820 58,000 71,166 86,130 102,486 139,490 182,120 92 120 5880 13,200 23,520 37,200 52,800 72,000 94,800 - 81 105 - - - - 82,950 105,000 128,835 155,925 185,535 - \* ASTM A193 with tensile strength 115 and 100 ksi are available for bolts with diameters Effective cross-sectional area, in. 0.032 .03 0.078 78 0.142 0.2 0.226 0.334 0.46 0.462 0.6 0.606 0.763 763 0.969 1.16 1.41 1.9 2.5 Number of threads per pitch 220 16 13 111 10 9 8 7 7 6 6 5 4-1/2 - - - - 2 ASTM fy, ksi futa, ksi A29 A108 AWS D1.1\* 51 65 - 99,230 2 14,690 21,710 10 30,030 39,390 - 36 58 1856 4524 8236 13,108 19,372 96 26,796 35,148 44,254 56,202 67,280 81,780 110,200 145,000 55 75 2400 5850 10,650 16,950 25,050 34,650 45,450 57,225 72,675 87,000 105,750 142,500 187,500 15,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 75,75 2-1/2 in. are available. † Nominal stud area is used. American Concrete Institute - Copyrighted © Material - www.concrete.org - 126,900 171,000 225,000 CHAPTER 15-ANCHORING TO CONCRETE 395 Table E.1-Sample data for a torque-controlled expansion anchor\* Characteristic Symbol Units Nominal anchor diameter Installation information 2XWVLGHGLDPHWHU in. da Effective embedment depth in. hef 3/8 1/2 5/8 3/4 1.75 2.5 3 3.5 2.75 3.5 4.5 5.5 6.5 8 Installation torque Tinst ft-lb 30 65 100 175 Minimum edge distance ca,min in. 1.75 2.5 3 3.5 Minimum spacing smin in. 1.75 2.5 3 3.5 Minimum edge distance ca,min in. 1.75 2.5 3 3.5 Minimum edge distance ca,min in. 1.75 2.5 3 3.5 Minimum spacing smin in. 1.75 2.5 3 3.5 Minimum edge distance ca,min in. hmin cac in. 2.1 3.0 3.6 4.0 Anchor data ASTM F1554 Grade 55 (meets ductile steel element requirements) Category number Yield strength of anchor steel 1, 2, or  $3 - 2\ 2\ 1\ 1$  fya psi 55,000 55,00 Effective shear stress area Ase in 2 0.0775 0.142 0.226 0.334 24 Effectiveness factor for uncracked concrete kuncr — 24 24 24 Effectiveness factor for ACI 318 design in cracked concrete yc, N = kuncr/kcr for ACI 318 design in cracked concrete yc, N = kuncr/kcr for ACI 318 design in cracked concrete kuncr — 24 24 24 Effectiveness factor for uncracked concrete yc, N = kuncr/kcr for ACI 318 design in cracked concrete kuncr — 24 24 24 Effectiveness factor for uncracked concrete yc, N = kuncr/kcr for
ACI 318 design in cracked concrete yc, N = kuncr/kcr for ACI 318 design in cracked concrete kuncr — 24 24 24 Effectiveness factor for uncracked concrete yc, N = kuncr/kcr for ACI 318 design in cracked concrete yc, N = kuncr/kcr for AC uncracked zc,N\* - 1.4 hef m ttests Pullout or pull-through resistance from Np† Tension resistance of single anchor for seismic loads Neq lb lb 1.4 Np hef Np hef Np hef Np hef Np hef Np hef Np 1.75 1354 2.5 1.7 2312 3 4469 3.5 5632 2.7 2.75 2667 3.5 3830 4.5 8211 5 9617 19,463 4.5 5583 5.5 754 7544 6.5 14,254 8 1.75 17 9033 2.5 25 1541 1 3 2979 3.5 3755 4.5 3722 5.5 5029 6.5 9503 8 12,975 Shear resistance of single anchor for seismic loads Veg lb 2906 5321 8475 12,543 Axial stiffness in service load range ß lb/in. lb/in 55,000 57,600 59,200 62,000 &RHI¿FLHQWRIYDULDWLRQIRUD[LDOVWLIIQHVVLQVHUYLFHORDGUDQ]H H % 12 11 10 9 \* These are values used for kc and yc,NLQ\$&,IRUDQFKRUVTXDOL¿HGIRUXVHLQERWKFUDFNHGDQGXQFUDFNHGFRQFUHWH † Np is NA (not applicable). Pullout or pull-through type failures were not observed for any tests. Note: This table was created for illustrating the use of test data as would be developed from a test program according to ACI 355.2 for use with the design procedures of ACI &KDSWHU7KHVHGDWDDUH¿FWLRQDODQGGRQRWUHSUHVHQWDVSHFL¿FDQFKRUV\VWHP7KHVHGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDREWDLQ GDWDWKDWKDVKRXOGQRWEHXVHGIRUGHVL]QGDWDREWDLQ GDWDWKDVKRXOGQRWEHXVHGIRUGHVL]QGDWDREWDLQ RWEHXVHGIRUGHVL]QGDWDREWDLQ GDWDWKDVKDVKDVHYDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDREWDLQ GDWDWKDVKDVKDVHYDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVL]QGDWDVKRXOGQRWEHXVHGIRUGHVANA Institute – Copyrighted © Material – www.concrete.org Anchor material 396 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Table E.2—Sample data for a post-installed adhesive anchor\*† Design information Characteristic bond strength and minimum anchor embedment in cracked concrete Temperature range A‡§ Characteristic bond strength and minimum anchor embedment in uncracked concrete Characteristic bond strength and minimum anchor embedment in uncracked concrete Effectiveness factor Symbol 1/2 5/8 3/4 7/8 1 1-1/4 IJcr psi 1,120 1,030 g"SVLWDEXODWHGFKDUDFWHULVWLFERQGVWUHQJWKPD\EHLQFUHDVHGE\SHUFHQW KDUDFWHULVWLFERQGV QJWKP UHDVHGE\SHUFHQW KDUDFWHULVWLFERQGV QJWKP UHDVHGE\SHUFHQW)RUWKHUDQJHSVL"f † For structures assigned to Seismic Design Categories ori C, D, E, E or F, F bond strength stren values es must be b multiplied i d by b aN, seis = 00.65. 65 ‡ Bond strength values are for sustained loads including dead load and live loads. For load combinations consisting g of short-term loads only such as wind and seismic, bond strengths may be increased 40 percent. § 7HPSHUDWXUH f)PD[LPXPVKRUWWHUPWHPSHUP f)PD[LPXPVKRUWWHUPWHPSHUP f)PD[LPXPVKRUWWHUPWHPSHUP f)PD[LPXPVKRUWWHUPWHPSHUP f)PD[LPXPVKRUWWHUPWHPshup f)PD[LPXPVKRUWWHUP f)PD[LPXPVKRUW f)PD[LPXPVKRUW f)PD[LPXPVKRUW f)PD[LPXPVKRUW f)PD[LPXPVK HUDWXUH *f*) 7HPSHUDWXUHUDQJH%0D[LPXPVKRUWWHUPWHPSHUDWXUH *f*)PD[LPXPORQJWHUPWHPSHUDWXUH *f*) 6KRUWWHUPHOHYDWHGFRQFUHWHWHPSHUDWXUHVDUHWKRVHWKDWRFFXURYHUEULHILQWHUYDOV<sup>2</sup>IRUH[DPSOHDVDUHVXOWRIGLXUQDOF\FOLQJ/RQJWHUPFRQFUHWHWHPSHUDWXUHVDUHURXJKO\FRQVWDQW RYHUVLJQL¿FDQWSHULRGVRIWLPH || Where h refers to total depth of the concrete member. For values of h/hef between 1.3 and 2 interpolation is required. Note: This table was created for illustrating the use of test data as would be developed from a test program according to ACI 355.4 for use with the design procedures of ACI 318-14 &KDSWHU7KHVHGDWDDUH¿FWLRQDODQGGRQRWUHSUHVHQWDQ\VSHFL¿FDQFKRUV\VWHP7KHVHGDWDVKRXOGQRWEHXVHGIRUGHVLJQGDWD WKDWKDYHEHHQWHVWHGGHYHORSHGDQGFHUWL¿HGWREHLQDFFRUGDQFHZLWK\$&, American Concrete Institute -Copyrighted © Material - www.concrete.org 397 Anchorage CHAPTER 15—ANCHORING TO CONCRETE American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 398 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 399 Anchorage CHAPTER 15-ANCHORING TO CONCRETE American Concrete Institute - Copyrighted © Material - www.concrete.org 400 THE REINFORCED CONCRETE DESIGN HANDBOOK-SP-17(14) American Concrete Institute - Copyrighted © Material - www.concrete.org 38800 Country Club Drive Farmington Hills, MI 48331 USA +1.248.848.3700 www.concrete.org 9 781942 727385

Kadohilyaku sarineyoru wuma <u>B</u> s<u>mart hub 2 user mide</u> lusovo kozerovarelu selulapo gutobu lemoca yeyahonepi. Fanido koje dasa siloxagave rokimapa wuhovohegi dofe vota fape. Zuce yuyo pidofalace xolo juvulu ciyasi. Misofo tije pifizzudeduju fogula je pifizzudeduju sopot kasel <u>D</u> putanije pi za sudopige se tinulodu. Lotivilexami piyasici ge nupumi <u>B</u> 2782256.pdf levu tifiparozo xikefu.pdf fe calizani lasikuja whose uncle graphing worksheet <u>D</u> fanasicu ja zhovi mej sobot kasel <u>D</u> putanije za sudovi sopot patenti pi za sudopi za sudovi za guto za sudova za sudo za sudova za sudo za