Sp-017(14) the reinforced concrete design handbook pdf

An ACI Handbook The Reinforced Concrete Design Handbook A Companion to ACI 318-14 Volume 1: Member Design SP-17(14) ACI 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 SLABS BEAMS DIAPHRAGMS COLUMNS STRUCTURAL REINFORCED CONCRETE WALLS FOUNDATIONS 38800 Country Club Drive Farmington Hills, MI 48331 USA +1.248.848.3700 www.concrete.org 9 781942 727385 ACI SP-17(14) Volume 1 THE REINFORCED CONCRETE DESIGN HANDBOOK A Companion to ACI 318-14 Editors: Andrew Taylor Trey Hamilton III Antonio Nanni First Printing September 2015 ISBN: 978-1-942727-37-8 Errata as of September 2, 2016 THE REINFORCED CONCRETE DESIGN HANDBOOK Volume 1 ~ Ninth Edition Copyright by the American Concrete Institute, Farmington Hills, MI. All rights reserved. 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Senecal, Gregory M. Zeisler, and Jerzy Z. Zemajtis Technical Editors: Shannon B. Banchero, Emily H. Bush, and Cherrie L. 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$&,'DQKHOGYDULRXVHQJLQHHULQJSRVLWLRQVLQWKHSULYDWHVHFWRU.96&RUS%HIRUH WKDWKHZDV3URMHFW(QJLQHHUIRU6NLGPRUH2ZLQJV0HUULOOLQ&KLFDJR,'&+H received his BS in civil engineering from the University of Buffalo, Buffalo, NY and his 06LQFLYLODQGVWUXFWXUDOHQJLQHHULQJIURP/HKLJK8QLYHUVLW\%HWKOHKHP3$+HZDVD licensed professional engineer in several states. ,QKLVSHUVRQDOOLIH'DQZDVDQDYLGIROORZHUPDQ\LQJRXWVZLWKKLVWKUHHURWKHUVRQHYHUSRVVLEOH+HZDVDOVRDQDFWLYH PHPEHURI2XU6DYLRU/XWKHUDQ&KXUFKLQ+DUWODQG0,DQGDQDGHGLFDWHGIDQRIWKHSSVSORHDPVKHUHKHZDVFRDFKLQJ6WDWH6SDUtans 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 EXLOGLQJVDQGUHODWHGVWUXFWXUHV7KLVHGLWLRQLVDPDMRUUHYLVLRQWKDWEULQJVLWXSWRGDWHZLWKWKHDSSURDFKDQGSURYLVLRQVRI 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 UHIHUHQFHVKRZLQJVWHSE\VWHSFDOFXODWLRQV²IROORZHGE\DGGLWLRQDOLQIRUPDWLRQFRQ¿UPLQJWKHDSSOLFDWLRQRIWKHGHVLJQ¿QGLQJVE\VHOHFWLQJ 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 appreciates 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 $QLQGD\'XWWD &KDUOHV'RODQ0DWWKHZ+XOLJ5RQDOG.OHPHQFLF-DPHV/DL6WHYHQ0F&DEH0LNH0DUWLQ+DQL1LFRODV 'DYLG 3DUNHU-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 HandbookLVSEXOLVKHGLQWZRYROXPHV&KDSWHUVWKURXJKLQFOXGHGLQ9ROXPH'HVLJQLDLGVDGDPRUHQWHUDFWLRQGLDJUDPV([FHOVSUHDGVKHHWVDUHDYDLOD available for free download from the following ACI webpage links: Keywords: DQFKRULQJWRFRQFUHWHEHDPVFROXPQVFUDFNLQJGHÀHFWLRQGLDSKUDJPGXUDELOLW\H[DPSOHVUHQJWKIRRWLQJV frames; piles; pile caps; post-tensioning; punching shear; retaining wall; shear strength; seismic; slabs; splicing; stiffness; structural analysis; structural systems; strut-and-tie; walls. Khaled Nahlawi Managing Editor

 1.1—Introduction The building depicted in this chapter was developed to show how, by various examples in this Handbook, to design and detail a common concrete 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 A-A American Concrete Institute – Copyrighted © Material – www.concrete.org 12 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 1.3—Loads The following 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 = 100 lb/ft2 • Typical Floor: Private rooms and corridors serving them = 65 lb/ft2 Roof Live Load: • Unoccupied = 20 lb/ft2 Snow Load: • Ground load, Pg = 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 directionality factor, Kd = 0.85 0 • Topographic factor, Kst = 1.0 5 ((rigid) • Gust-effect factor, Gf = 0.85 QW +/-0.18 • ,QWHUQDOSUHVVXUHFRHI¿FLHQWGC pi = +/ • Directional Procedure Seismic Load: • 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 Þ R=5 Þ Cs = 0.046 • Moment-resting frame system; ordinary reinforced concrete moment frame in the east-west direction Þ R=3 Þ Cs = 0.032 The fcgIRUFROXPQVDQGZDOOVLQWKLVVWRU\EXLOGLQJVPD\ be different than the fcgXVHGIRUWKHÀRRUVWHP&RQFUHWH SODFHPHQWXVXDOO\SURFHHGVLQVXFKDZD\WKDWGXUDELOLW\UHTXLUHPHQWVDUHVDWLVILHGDWHDFKVWRU\¿UVW WKHYHUWLFDOPHPEHUVVXFKDVFROXPQVDQGVKHDUZDOOVWKHQWKHÀRRU members, such as beams and slabs. It is desirable to keep the concrete strengths of the vertical members within a ratio RIRIWKHÀRRUFRQFUHWHVWUHQJWK6HFWLRQLQ$&, VWDWHVWKDWWKLVLVWUDWLRLVH[FHHGHGWKHÀRRUFRQFUHWH in the area immediately around the vertical members must be "puddled" with higher strength concrete. Usually this situation only becomes an issue for taller buildings. For this example, the building height is moderate and the loads are typical. The locally available aggregate is a durable dolomitic limestone. Thus, the concrete can readily have a higher fcgWKDQWKHLQLWLDOO\DVVXPHGSVL$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 satis¿HG7KHIROORZLQJFRQFUHWHPDWHULDOSURSHUWLHVDUHFKRVHQ • fcg SVL • Normalweight, wc = 150 lb/ft3 • Ec = 4,030 4,030,000 psi • Ĥ 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 JUHDWO\ 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\VLVDQDFFRXQWIRUWKH ect. The P Poisson effect. Poisson ratio can vary due to material erties, but an average value for concrete is 0.2. Recomproperties, PHQGDWLR PHQGHLQRUVRWKHWKHUDPDWHUDOFRHI¿FLHQWRIH[SDQVLRQe th, of concrete can be found in ACI 209R. The most common and most available nonprestressed reinforcement is Grade 60. Higher grades are 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 • Es = 29,000,000 psi 1.4—Material properties The material properties for any building should have a reasonable knowledge of locally available concrete and steel materials. As a preliminary value for this example, a speci¿HGFRQFUHWHFRPSUHVVLYHVWUHQJWKfcgRISVLLVXVHG SURYLGHV IRU D VDWLVIDFWRU\ ÀRRU GHVLJQ ,Q WKH 86 UHLQIRUFLQJVWHHOIRUÀRRUGHVLJQLVXVXDOO\VSHFL¿HGDV psi.

REFERENCES American Concrete Institute ACI 209R-92—Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 2—STRUCTURAL SYSTEMS 2.2—Materials The concrete mixture proportion needs to satisfy the design properties and limits in ACI 318-14, Chapter 19, and the reinforcing steel needs to satisfy the design properties and limits in Chapter 20 (ACI 318-14). 2.3—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 llloading g (elastic analysis, a 1. Wind ACI 318-14, Chapter 6) 2. E rthqu ading (AC Earthquake loading (ACI 318-14, Chapter 18) Wi quake lo Wind an and earthquake loads are dynamic in nature; owev r, th however, they differr in the m manner in which these 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 lo surface. Earthquake 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 potential energy when it is resisted by an obstruction. Wind pressure is related to the wind velocity, building height, building surface, the surrounding 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 quasi-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, wind tunnel testing is not unusual. 2.3.2 Earthquake loading²7KHPDLQREMHFWLYHRIVWUXFtural design is life safety; that is, preserving the lives of occupants and passersby. Serviceability and minimizing HFRQRPLFDOORVVHVZKHQVXEMHFWHGWRVHLVPLFHYHQWVDUHDOVRLPSRUWDQWREMHFWLYHV%\ studying the results of previous earthquakes on various structural systems, improvements to code provisions and design practices have been achieved. These improvements have led to a reduction in damage of reinforced concrete structures that experience an earthquake. Some code improvements for PHPEHUVWKDWUHVLVWVLJQL¿FDQWVHLVPLFDFFHOHUDWLRQVDUH American Concrete Institute – Copyrighted © Material – www.concrete.org Structural Systems 2.1—Introduction A chapter on 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, were seldom explicitly 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. Structural systems and their component members must WLIIQHVV VR WKDW SURYLGH 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 PHPEHUVDQGFRQQHFWLRQ,GHQWL¿HVWKHVWUXFWXUDOPHPEHUVDQGFRQQHFWLRQ KFed 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 DQG URRI PHPEHU RZHD\ 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 (Chapter 14) (g) Joints and connections (Chapters 15 and 16) In Table 2.1, code chapters are correlated with the chapters LQ9ROXPHVDQGRIWKLV+DQGERRN 14 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) contains provisions meant to ensure inelastic deformation 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 and potentially critical structures—for 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 representing seismic forces "ASCE 7-10." 7KHUHTXLUHPHQWWKDWFROXPQVLQDIUDPHGÀH[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 stiffnes dissipation capacity (with less de deterioration stiffness and strength) UV WR HQVXUH ÀH[XUDO 'HVLJQLQJ DQG GHWDLOLQJ PH PEHUV ÀH[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\VWHPSLUUHJXODULWLHV 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 roughly approximates the 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 speci¿HGE\PRVWFRGHV7KLVLVEHFDXVHFRGHVJHQHUDOO\DFFRXQW for force reductions due to inelastic response. For example, ASCE 7-10 applies an R IDFWRU UHVSRQVH PRGL¿FDWLRQ factor), which accounts for the ductility of a building, system over-strength, and energy dissipation through the soil-founGDWLRQVWHPV³$&(´,WVLPSOL¿HVWKHVHLVPLFGHVLJQ process such that linear static elastic analysis can be used for building designs. The R factor reduces the calculated lateral loads 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 structure. R-values range from 1-1/2 for 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—Structural systems All structures must have a continuous load path that can be traced from all load sources or load application to WKH IRXQGDWLRQ 7KH PRVW EHWZHHQ WKH YHUWLFDO PHPEHUV (columns and walls) and the horizontal members (beams, slabs, diaphragms, and foundations) are crucial to this concept. Properly detailed cast-in-place (CIP) reinforced FRQFUHWHMRLQWVWUDQVIHUPRPHQWVDQGVKHDUVIURPWKHÀRRU into columns and walls, thus creating a continuous load path. 7KH PRVW GHVLJQ VWUHQJWK $&, &KDSWHU PXVW \ of course, adequately resist the factored forces applied to WKH MRLQW 5HIHU DOVR WR$&, 5 IRU MRLQW GHVLJQ DQG detailing inform information. ngineers commo Engineers commonly refer to a structure's gravity-loadJ V\ VWHP */56 UHVLVWLQJ V\VWHP */56 DDQ DQG ODWHUDOIRUFHUHVLVWLQJ V\VWHP $OO PEHUVRID /56 $OOPHPEHUVRID&,3UHLQIRUFHGFRQFUHWHVWUXFWXUH FRQ XWHUWKH */56DQGP FRQWULEXWHWRWKH*/56DQGPRVWFRQWULEXWHWRERWKV\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 ZLWKRXW DQ\ VWRWKHG ZLWKRXWDQFKDQJHVWRWKHGHVLJQRUGHWDLOLQJRIWKH*/56 mem rs. A uilding in members. As the building increases in height, the importance L DQG QG GGHWDLO LOL WKH /56) 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/56 LVXVXDOO\LQÀXHQFHGE\DUFKLWHFWXUDO FRQVLGHUations 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. Frames²/DWHUDO GHIRUPDWLRQ RI IUDPHV DUH SULPDULO\ GXH WR VWRU\VKHDU7KHUHODWLYHVWRU\GHÀHFWLRQVWKHUHIRUHGHSHQG on the horizontal shear applied at each story level. 2. Walls²/DWHUDOGHIRUPDWLRQVDUHGXHWRERWKVKHDUDQG 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 system²/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 system. At such a height, stiffness, controls the building design. sions that determine the ASCE 7 (2010) provides provisions C A tthrough F). As a buildSeismic Design Categories (SDC y increases, in es, from A thr ing's Seismic Design Category throughh pro sively more rigorous rigo ous F, ASCE 7 (2010) requires a progressively 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 FRQWURORWKHGHVLJQRIWKH/)56 For buildings in SDC C, seismic loads are likely to control GHVLJQIRUFHVDQGVLPSOLFLWFGHWDLOLQJLVUHTXLUHG/)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 KHLJKW OLPLWDWLRQ based on assumed structural performance level behavior. Figure 2.4 shows approximate height limits for different structural systems. Table 2.4 provides ASCE limits for choosing a structural system for a particular building. The ranges of applicability VKRZQ DUH LQÀXHQFHG E\ RFFXSDQF\ UHTXLUHPHQWV DUFKLWHFWXUDO FRQVLGHUDWLRQVLQWHUQDOWUDI¿FÀRZSDUWLFXODUO\LQ WKHORZHUÀRRUV WKHVWUXFWXUH¶VKHLJKWDQGDVSHFWUDWLRDQG load intensity and types (live, wind, and earthquake). 2.4.1 Gravity-resisting systems—A gravity-load-resisting V\VWHP */56 LV WKH FRPSRVLWH RI KRUL]RQWDO ÀRRU PHPEHUV and vertical members that support the horizontal members. Gravity loads ar are resisted by reinforced concrete members JK D[LDO ÀH[XUH VKHDU DQG WRUVLRQ ÀH[XUDO VKHDU DQG WRUVLRQDO VWLIIQHVV DQG rma m 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-forceUHVLVWLQJ V\ )56 PX UHVLVWLQJ V\VWHP/56 PXVWKDYHDGHTXDWHVWUHQJWKWRUHVLVW 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 DQGVHUVLRQ DQGPRPHQW DQGVHUYLFHDELOLW\ DQG DUH FRQWUROOHGFRQFUHSXWW red for tall buildings). be considered CI 318-14, Section 18.2.1, lists the relevant code ACI VHFWLRQV VHFWLRQV IRU HDFK 6'& DV LW DSSOLHV WR D VSHFL¿F VHLVPLF force-resisting system. The following lateral-force-resisting systems are addressed as follows. 2.4.2.1 Moment-resisting frames—Cast-in-place momentresisting 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, EHDPVVODEVDQGMRLQWGHIRUPDWLRQV GHIRUPDWLRQGXHWR VKHDULQFROXPQVDQGMRLQWVDQG GHIRUPDWLRQVGXHWRD[LDO force in columns. Yielding in the frame members or the foundation can VLJQL¿FDQWO\LQFUHDVHWKHODWHUDOGLVSODFHPHQW7KHHIIHFWRI secondary moments caused by column axial forces multiSOLHGE\ODWHUDOGHÀHFWLRQV³ ¨HIIHFW ZLOOIXUWKHULQFUHDVH WKHODWHUDOGHÀHFWLRQ In buildings, moment-resisting frames are usually arranged parallel to the principal orthogonal axes of the structure and WKH IUDPHV DUH LQWHUFRQQHFWHG E\ ÀRRU GLDSKUDJPV &$&, 318, Chapter 12). Moment-resisting frames usually allow WKH PD[LPXP ÀH[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\PRPHQWIUDPH(20) 1/ NP NP NP Intermediate moment frame (IMF) 1/ 1/ NP NP NP Special Moment frame (SMF) 1/ 1/ 1/ 1/ 1/ Structural walls (only): %XLOGLQJIUDPHV\VWHPVVWUXFWXUDOZDOOVDUHWKHSULPDU\ODWHUDOORDGUHVLVWLQJ\VWHP)\/56 2UGLQDU\VWUXFWXUDOZDOO26: 1/ NP NP NP Special structural wall (SSW)* 1/ 1/ 160 ft 160 ft 100 ft Bearing wall systems (structural walls are the primary lateral- and gravity-load-resisting system): 26: 1/ NP NP NP SSW* 1/ 1/ 160 ft 160 ft 100 ft 'XDOV\VWHPVVWUXFWXUDOZDOOVDUHWKHSULPDU\)56DQGWKHPRPHQWUHVLVWLQJIUDPHVDFFRXQW\DWOHDVW of the lateral load): 26:ZLWK20) 1/ NP NP NP 26:ZLWK,0) 1/ 1/ NP NP NP 26:ZLWK60) 1/ 1/ NP NP NP 66:ZLWK20) NP NP NP NP SSW with IMF 1/ 1/ 160 ft 100 ft 100 ft SSW with SMF 1/ 1/ 1/ 1/ 1/ F * Height limits can be increasedd per ASCE 7 (2010), Section 12.2.5.4. 13 QRWS 1RWHV1/ QROLPLW13 QRWSHUPLWWHG)LJ²ÀHFWLRQVXQGHUJUDYLW\ORDG 2.4.2.2 Shear walls—Reinforced concrete shear walls are often introduced 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)RUEXLOGLQJVZLWKRXWVLJQL¿FDQWPRPHQWUHVLVWLQJ 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 IUDPH ODWHUDO GHÀHFWLRQ RI the system. Reinforced concrete shear walls are often used around HOHYDWRUDQGVWDLUVKDIWVWRDFKLHYHWKHUHTXLUHGODWHUDOVWLIIQHVV For shear wall types and functions, refer to Table 2.4.2.2. A shear wall building usually consists of a series of parallel shear walls in orthogonal directions that resists lateral loads and supports vertical loads. ,Q PXOWLVWRU\ EHDULQJ ZDOO EXLOGLQJV VLJQL¿FDQW stiffness method is relatively easy to program and evaluate. A more sophisticated use of the direct stiffness method is the ¿QLWHHOHPHQWPHWKRG7KHVWUXFWXUHLVPRGHOHGDVDLVFUHWH elements connected at nodes. Each member of the struc- American Concrete Institute – Copyrighted © Material – www.concrete.org ture consists of multiple rectangular elements, which more accurately determine the behavior of the member. The act of modeling 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.)RUVWUDLJKWIRUZDUGGHVLJQVDQGHVLJQHUPRUHHI¿FLHQWO\ analyze the structure by dividing it into parts and using less complicated programs to analyze each part. 3.4.2 Two-dimensional frame modeling—A building can be divided into parts that are analyzed separately. For example, 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[SHFWHGRYHUDOOODWHUDOGHÀHFWLRQRI 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 GHÀHFWLRQV metry of beams at crossing points is not required. The geometry HP)RU VO FDQ YDU\ GHSHQGLQJ RQ WKH ÀRRU 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. mes, it is permitted to m el For beam-column moment frames, model om iven in Section 6.3.2 3.2 T-beams, with the limits on geometry given en simpler ler to ignore the slab of ACI 318; however, it is 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 section may not be conservative; refer 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 ÀRRU 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 Section 12.8.4.2 in ASCE 7-10. For wind loads, a torsional moment should be applied 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 ma m of other tim time-consuming mathematical tasks. GXFH FRPSXWDWLRQ PSXWDWLRQ WLPH FRQFUHWH ÀRRUV DUH VRPH7R UHGXFH time times mode modeled as rigid dia diaphragms, reducing the number RIG\Q PLF VRIIUHHG RIG\QDPLFDGHJUHHVRIIUHHGRPWKUHHSHUÀRRU WZR i tal translations 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 rat span-to-depth are 3 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 UHDFWLRQVLQWKHZDOOVDQRYHDQGEHORZWKHÀRRU$VHPLULJLG diaphragm requires more computation power but provides a distribution of lateral forces and calculates slab stresses. A semi-rigid 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 only to a portion of a 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." 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\ÀRRUV\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 ÀRRU PHPEHUV 7KLV PHWKRG LV SUREDEO\ XVHG more often to estimate initial member sizes for computer LQSXWRUIRULQLWLDOFRVWHVWLPDWHVWKDQIRU¿QDOGHVLJQ 3.5.3 First-order analysis²&RGH UHTXLUHPHQWV IRU ¿UVW order analysis are provided in Section 6.6 in ACI 318. 3.5.3.1 Section properties—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 of the structure for strength design. For service level load 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- LQHUWLDHTXDWLRQVDOVRKDYHWKHVWLIIQHVVUHGXFWLRQ k already applied. Section 6.6.3.1.2 of ACI 318 also allows a VLPSOL¿FDWLRQRIXVLQJ Ig 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 effects²$ ¿UVWRUGHU DQDO\VLV LQ ACI 318 assumes that only primary stresses are calculated. 6HFRQGDU\ VWUHVVHV FDXVHG E\ WKH ODWHUDO GHÀHFWLRQ FDXVHG by the design loads are not calculated. First-order analysis is typical when hand-calculation methods are used or basic matrix analysis computer programs are used that are not programmed for iterative analysis. This method ignores the P¨ HIIHFWV ZKLFK DUH WKH second-order effects caused by vertical loads acting on the EXLOGLQJ¶VODWHUDOO\GHIRUPHGFRQ¿JXUDWLRQ)L 7R approximately account for these secondary effects, a moment PDJQL¿HULVDSSOLHGWR¿UVWRUGHUFROXPQDQGZDOO design moments American Concrete Institute – Copyrighted © Material – www.concrete.org)LJ²3 ¨ HIIHFWV 7KHGHVLJQHUPXVWDFFRXQWIRUVOHQGHUQHVVLQD¿UVWRUGHU DQDO\VLV)LJXUH 5 LQ $&, SURYLGHV D ÀRZ 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 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¿HGIRUPj HIIHFWVDORQJWKHPHPEHU,IWKHVWRU\LV RPHQW DUH PDJQL¿HG IRU VZD\ WKH FROXPQ RU ZDOO HQG PRPHQWV Pj P¨ HIIHFWVDORQJWKHPHPEHUPj DQGDWWKHHQGVDUHWR story drift (P¨ DU VLVDOORZDVVRUV SHU3.5.3.3 Superposition²/LQHDUDQDO\VLVDOORZVVXSHUUmb loads. This is very position 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 WRRORV 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 VODE DQG EHDP PRPHQWV DQG shears by taking advantage of the ductility provided through the code detailing requirements. Ductile detailing is required IRU FRQWLQXRXV ¿EHUV DUH 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 spans. If column spacing is not uniform, some beam design moments may be slightly lower than the EHDPUHTXLUHGPRPHQWV2QFHWKHVHOLEHOGLQJKDGEHHQYHORSHG at factored loads, however, the moments will 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¨HIIHFWVRQWKHPHPEHUGXHWRVWRU\ drift are computed. A computer algorithm then automatically FDUULHVRXWDVHULHVRILWHUDWLYHDQDO\VHVXQWLOWKHQHZGHÀHFWLRQYDOXHVXQWLOWKHVROXWLRQFRQYHUJHVWRWKH¿QDOVHFRQGDU\ 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¨ HIIHFWV6RIWZDUHFDQHDOZLWKWKHDGGLWLRQDO moment due to building lateral deformation but some software does not calculate the secondary moments along the member length. The designer may have to model the column as at least two segments to capture this effect. Even though a column 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 secondary moment along the length of the column. 3.5.5 Inelastic second-order analysis—The consideration of the no nonlinear behavior of structures arising from inearity of the st nonlinearity stress-strain curve for concrete and steel element, particula rly ly under large deformations, can reinforcement, 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) or geometry (for xamp e, ssecond-order order mo example, 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 superposition 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 analysis²7KH¿QLWHHOHPHQWPHWKRGDQDO\VLV of concrete structures is permitted by ACI 318 and can be XVHGWRVWDVVHVVLVHDFK¿UVWRUGHUHODVWLFVHFRQGRUGHUHODVWLF inelastic second-order 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\VLVLVZLGHO\XVHGDQGDFFHSWDEOHWRRO IRU DQDO\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(/)SURFHGXUHLVFRPPRQO\XVHGGXHWRLWVVLPSOLFLW\ 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) or Linear or Nonlinear or Seismic Response History procedure, nonlinear (Ch. 16 of ASCE 7), may be required. RegardRFH VDFFHSWDEOHI OO OHVVRIL UUHJXODULWLHVWKH(/)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 W\SHV RI VWUXFWXUDO LUUHJXODULWLHV GHVFULEHV ¿YH ¿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 7 and ACI 318 have additional requirements to account for WKHDQWLFLSDWHGEHKDYLRUDWHUDOIRUFHUHVLVWLQJV\VWHPVIRU FRQFUHWH VWUXFWXUHV DUH GH¿QHG LQ$6&(DQG &KDSWHU RI$&,(DFK/)56KDVUHVSRQVHPRGL¿FDWLRQFRHI¿FLHQWRYHUVWUHQJWKIDFWRUÖDQGGHÀHFWLRQDPSOL¿FDWLRQ REFERENCES American Concrete Institute ACI Committee 442-77—Response of Buildings to /DWHUDO)RUFHV American Society of Civil Engineers $6&(²0LQLPXP'HVLJQ/RDGVIRU%XLOGLQJVDQG 2WKHU6WUXFWXU 2WKHU6WUXFWXUHV 6&(²6H $6&(²6HLVPLF(YDOXDWLRQDQG5HWURW5HKDELOLListing Buildi ngs gs tation off Ex Existing 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 LOGLQJVDQG 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 Technical Brief No. 4 (NIST 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($2& 6HLVPRORJ\ &RPPLWWHH ³$ %UHI *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 VHOHFWLRQRIPDWHULDOVRUWKHFRQFUHWHPL[WXUHGHVLJQ2WKHU DVSHFWVLQÀXHQFLQJGXUDELOLW\LQFOXGHUHLQIRUFLQJEDUVHOHFWion, 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 depend on transport of water through concrete. For this reason, it is essential to understand the mechanisms themselves and how different concrete-making mateULDOVLQÀXHQFLQJDGPL[WXUHVDQGWKHLUSURSRUWLRQVLQÀXHQFH concrete's resistance to these mechanisms. 4.1.1 Permeability²3HUPHDELOLW\LVFDQHGH¿QHGDV³WKH HDVHZLWKZKLFKDÀXLGFDQÀRZWKURXJKDVROLG´RUDV³WKH ability of concrete to resist penetration by water or other substances (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\RIWKHVHSRUHV ores in cemen capillary ppores cement pa paste are relevant to concrete durare responsi bility, ass the they are responsible for the transport properties of FRQ $& 5.RVP FRQFUHWH$&,5.RVPDWNDDQG:LOVRQ 7KHLQÀXence 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). curing on permeab permeability is presented in Fig. 4.1.1b. 4.1.2 F Freezing and thawing—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 IUHHLQJ ZDWHU¿OOHG capillary cavities, osmotic pressure is produced by water Fig. 4.1.1b—(left) Effect of w/cDQGLQLWLDOFXULQJRQKDUGUDXOLFZDWHU 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 to freezing and thawing of concrete (Kosmatka et al. 2008). entering partly frozen capillary cavities (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 2VPRWLFSUHVVXUHVGHYHORSGXHWR concentrations of alkali solutions in the paste. When pressure in concrete due to freezing exceeds the tensile strength of concrete, some damage occurs, especially if concrete is saturated with water and exposed to repeated cycles of freezing and thawing. The resulting damage is cumulative as it increases with additional repetitions of freezing and 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 freezin prior to freezing-and-thawing exposure, which is demoned by data presented prese strated in Fig. 4.1.2. fates—Sulfates ppresent in soil and water can react 4.1.3 Sulf Sulfates—Sulfates ydrat compounds in the hardened cement paste with hydrated DQG XFH VXI¿FLHQW LQWHUQDO SUHVVXUH WR GLVLQWHJUDWH WKH FRQFUHWH Howe er, formation f n of new crystalline substances due to However, eacti artly res those reactions is partly 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 are most susceptible to sulfate attack, the use of sulfate-resistant cements will not stop the sulfate attack, either (Fig. 4.1.3). Resistance to sulfate attack can be greatly increased by decreasing the

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 'HÀHFWLRQV—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 IRU37VODEVDVZHOOWKHGHVLJQHUFDOFXODWHVGHÀHFWLRQV7KH FDOFXODWHG GHÀHFWLRQV PXVW QRW H[FHHG WKH OLPLWV JLYHQ LQ 6HFWLRQ$&,'HÀHFWLRQVFDQEHFDOFXODWHGE\ 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 GHÀHFWLRQ 7KHVH YDULDEOHV VKRXOG EH FRQVLGHUHG ZKHQ DVVHVVLQJ WKH DFFXUDF\ RI GHÀHFWLRQ calculat calculations. In addition, creep over time will increase the LPPHGLDWHGHÀH LPPHGLDWHGHÀHFWLRQV LFDOO\ ZLWK D 3 7\SLFDOO\ 37 VODE VODE GHÀHFWLRQV DUH XVXDOO\ im m the maximum net concrete small. If th thee designer limits crac tensile stres stress to below cracking stress under service loads, GHÀH 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 SVHXGRVHUYLFHÀH[XUDOVWUHVVOLPLW)R DQDO\ )RU 37 VVODEV WKH DQDO\VLV RI FRQFUHWH ÀH[XUDO WHQVLRQ itical part of the design. In Section 8.3.4.1 of stresses is a critical WKHFRQF $&,WKHFRQFUHWHÀH[XUDOWHQVLOHVWUHVVLQQHJDWLYH 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 ÀH[XUDO VWUHVV OLPLWV are below the concrete cracking stress of 7.5 f c′ , thus KDYLQJWKHHIIHFWRIOLPLWLQJGHÀHFWLRQV,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. %HIRUHWKHVODEÀH[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 midspan and over supports; this maximizes the parabolic drape. Anchors are typically placed 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 stresses. American Concrete Institute – Copyrighted © Material – www.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 or unbonded 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. load considerations, such as wheel loads in parking garages. These result in local shear forces on the slab,, requiring veriU VWUHQJWK ¿FDWLRQ RI WKH VODE¶V 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 necessa y to as well as additional reinforcement, 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 ÀH[XUDO VWUHVVHV DUH FDOFXODWHG DQG WKH GHVLJQHU 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 concrete compresVLYHVWUHVVGXHWR37EXWPDMRULW\RIHQJLQHHUVXVHSVLDV a general minimum for cast-in-place slabs. For slabs exposed to aggressive environments, engineers usually design slabs with an increased minimum concrete compressive stress. 5.4—Required strength The design of one-way slabs are typically controlled by moment strength, not concrete stress or shear strength. Assuming a uniform load, the designer calculates the unit (usually a 1 ft width) factored slab moments. The required DUHDRIÀH[XUDOUHLQIRUFHPHQWRYHUDXQLWVODEZLGWKLVFDOFXlated with the same assumptions as a beam. ment stre 5.4.1 Calculation of required moment strength—For nonprestressed reinforced slabs, a quick way to calculate a slab's gravity design moments (iff the slab meets the speciRQV WKHPRPHQW ¿HGJHRPHWULFDQGORDGFRQGLWLRQV LVE\WKHPRPHQWFRHI &KDSWHU RI RI $&, ¿FLHQWV 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 rather than the 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 UHTXLUHGDUHDRIÀH[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 < 60,000 0.0020Ag 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 layers. For nonprestressed reinforced two-way slabs without beams, Fig. 6.9.1 (ACI 318-14, Commentary Fig. R8.6.1.1) provides a typical layout of column strip and middle strip top and bottom bars. ACI 318-14, Fig. 8.7.4.1.3(a) 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 ÀH[XUDO PRPHQWV $GGLWLRQDO UHLQforcement per ACI 318-14, Section 8.7.3, is required for this condition. 6.9.3 Post-tensioned slab – Reinforcing bar area and placing²2YHU HDFK FROXPQ UHJLRQ WKH &RGH UHTXLUHV DQ DUHDRIÀH[XUDOUHLQIRUFLQJEDUIRUWZRZD\ DUHDVWDcf in each direction, placed within 1.5h of the outside of the column. The Code also requires reinforcing bar in positive moment VLWKHFDOFXODWHG DUHDVLWKHFDOFXODWHGVWUHVVLQWKHSUHFRPSUHVVHG WHQVLOHIFRQFUHWHWKHDW sua ly midspan att th areas (usually the bottom) exceeds 2 f c′ or if d for strength. ngth. Botto required 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 at the re PT tendons, required design strength. The top EDUVP VSHF[WOHLDVWE EDUVPXVWH[WHQGWRDWOHDVWEQRQWZRPHDFKVLGHRIWKHFROXPQ7KH E E QHHGHG G G EHUWRPPEDUVLQHHGHG PXVWEHSDUDOOHOVWDEQGEHHFWHHQHUHRI 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 WKH 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 is required in a PT two-way slab. Post-tensioned tendons are XVXDOO\SODFHGLQDQWZRRUWKRJRQDOGLUHFWLRQV,QWKLVFRQ¿Juration, the Code allows banding of tendons in one direction and in the 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 between peripheral lines of shear studs 6SDFLQJEHWZHHQDGMDFHQWVKHDUVWXGVRQ peripheral line nearest to column face Condition Maximum distance or spacing, in. All d/2 Nonprestressed slab with vu"Ì f c′ 3d/4 Nonprestressed slab with vu!Ì f c′ d/2 Prestressed 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 Deformed bars or welded wire reinforcement As,min, in.2 0.0020Ag fy, psi < 60,000 • Greater of: 0.0018 × 60, 000 Ag fy 0.0014Ag 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) $&,52²&RQWURORI'HÀHFWLRQLQ&RQFUHWH6WUXFtures (Reapproved 2000) Authored references Corley, W. G.; Sozen, M. A.; and Siess, C. P., 1961,"Equivalent-Frame Analysis for Reinforced Concrete Slabs," Structural Research Series No. 218, Civil Engineering Studies, University of Illinois, June, 166 pp. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 87 &RUOH*DQG-LUVD-²(TXLYDOHQW)UDPH Analysis for Slab Design," ACI Journal Proceedings9 No. 11, Nov., pp 875-884. -LUVD-26RJHQ0$DQG6LHVV&3³(IIHFWV RI 3DWWHUQ /RDGLQJ RQ 5HLQIRUFHG &RQFUHWH)ORRU 6ODEV´ Structural Research Series No. 269, Civil Engineering 6WXGLHV8QLYHUVLW\RI,OOLQRLV8UEDQD,/-XO\ -LUVD-26RJHQ0$DQG6LHVV&3³3DWWHUQ /RDGLQJ RQ 5HLQIRUFHG &RQFUHWH)ORRU 6ODEV´ Proceedings$6&(91ER67-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%0DWHULDOSURSHUWLHVDUHVHOHFWHGEDVHGRQWKHGHVLJQHUH[SHULHQFHLQHIRUMXGJPHQW ORDDOO\DVVIRUGDWLDODQGLVDOORZHGFXWHQVLYHVKHDU ... Lateral load 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)LJ(²)LUVWÀRRUSODQ American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—TWO-WAY SLABS 89 ACI 318-14 Discussion Step 1: Geometry 8.2.1 This slab is designed using the Direct Design 8.10 Method (DDM) in Section 8.10. 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 DDM. 8.10.2.5 8.10.2.6 The uniform design loads satisfy the limits of Sec8.10.2.7 tions 8.10.2.5 and 8.10.2.6 to allow use of DDM. 8.2.4 8.2.5 89 Calculation There are at least three continuous 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 Columns are not offset through the slab so Section LVVDWLV¿HG All design loads are distributed uniformly and due to JUDYLW\RQO\VR6HFWLRQLVVDWLV¿HG Ratio of unfactored live load to unfactored dead load is approxi approximately 100/102.5 = 0.98. This ratio is less WKDQVR6HFWLRQ WKDQVR6HFWLRQLVVDWLV¿HG re are ar no o supportin There supporting beams so Section 8.10.2.7 is e not app applicable. 8.3.1.1 KLF VIRUGHÀHFWLRQ QWURO &KHFNWKHVODEWKLFNQHVVIRUGHÀHFWLRQFRQWURO 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 as that for the external panel. The remainder of the building is using a slab thickness of 7 in.; therefore, use 7 in. The slightly thicker than QHFHVVDU\VODEDLVZLKHWKDWHÀHFWLRQVDQGVKHDU strength. 8.3.1.3 1RFDOFXODWHGÀHFWLRQVLVSHUPLWWHGVLQFHWKHVODE ZLWKWKHVODEWKLFNQHVVDQGZKLFKLVWKHPLQLPXPWKHPLQLPXPVODEWKLFNQHVV ... 8.3.2 &DOFXODWHGGHÀHFWLRQVDUHQRWUHTXLUHGEHFDXVHWKH VODEWKLFNQHVVLVVDWLV¿HV6HFWLRQ of ACI 318-14. 6WHS/RDGVDQGORDGSDWWHUQV 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¿HGLQ6HSDWWHUQ 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[LPDWHSXQFKLQJ shear check. This check should reduce the probability of having to repeat the calculations shown in this example. This 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 (φvn/vug): For interior columns: φvn/vug• For edge columns: φvn/vug• For corner columns: φvn/vug• If these ratios are not exceeded, it is possible that the slab will not satisfy two-way shear strength requirements. The design slab could be thickened, drop panels added, or other options for adding twoway shear strength may be considered. For φvn, the calculations here are discussed in Step 10 more fully: vn = 4 f c′ = 4 5000 ksi = 0.283 ksi 1000 (4) vn = | 2 + | (β) (α d) vn = | 2 + s | bo) (f c′ = 6 5000 ksi = 0.424 ksi 1000 f c′ = 3.89 5000 ksi = 0.275 ksi* 1000 *controls φvn = 0.75 × 0.275 ksi = 0.206 ksi For vug, the calculations here are discussed in Step 7 more fully: vug = Vu bo d 29.6 in. × 29.6 in.) 283 kip (Vu = |14 ft × 18 ft − |) × (144 1000 ft 2 Vu = 70 kip vug = 70 ki kip = 0.106 ksi 118.4 in. 1 n. × 5.6 in. φvn/vvugg •∴ proceed. Notee th that due to the bbasement wall supporting the exterior perimeter oof the slab, the punching shear for the edge an and corner columns will not need to be checked in this example. Step 4: Analysis – Direct design method moment determination 8.4.1.3 The geometry of the design is shown in Fig. E1.2. 8.4.1.4 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 l VKHDUZDOOVWKHVODE¶VÀH[XUDODQDO\VLVLVRQWKHFRPEZDOOVWKH bined with the lateral load analysis. Section 8.10 provides the DDM to determine the slab's factored moments. Mo = qu A 2 A 2n 8 /RQJVSDQ ℓn = 16 ft ℓ2 = 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¿QLWLRQVIRSDQHOVQHVVDQGVWULSV VRI HOVDQGVWULSV 8.4.1.9 The lateral loads in this building are assumed to be resisted by shear walls with the slabs only acting as diaphragms between the shear walls. The slab LVDVVXPHGQRWWRFRQWULEXWHWRÀH[XUDOUHVLVWDQFHWR lateral loads. 8.4.2, The slab's factored 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 8.10.2 Step 1. 8.10.3.1, Refer to Fig. E1.2 for slab span lengths in both 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 The DDM calculates a total panel Mo and then uses FRHI¿FLHQWVWRGHWHUPLQHPD[LPXPSRVLWLYHDQG negative design moments. In this example, all spans have ℓn = 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 8.10.4.4 8.10.4.5 8.10.5 8.10.5.1 8.10.5.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Table 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 two interior QHJDWLYHPRPHQWVDWWKH¿UVWLQWHULRUFROXPQ controls the design of the slab. Proportion the 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 negative and positive Mu as described earlier in Section 8.10.4, Table 8.10.5.1 then proportions the interior negative Mu assumed to be resisted by the colum column strip. After distributing the ttotal 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 8.10.5.1, ℓ2/ℓ1 DQG[f1 = 0. Therefore, the top line of the table controls: Mu,int.neg, cs = 0.75 × 83 ft-kip = 63 ft-kip In Table 8.10.5.2, 0.5.2, ℓ2/ℓ / 1 DQG[f1 = 0. umi thee wall beha Assuming behaves as a beam, C is calculated WRGHWHUPLQHù WR HWR Eq (8.10.5.2(a) and (b)). t using Eq. Ecb C βt = 2 Ecs I s (x) x3 y C = |1 − 0.63 | (y) 3 x = 10 in. y = 120 in. C = 37,900 in.4 Ecb = Ecs bh3 168 in. × (7 in.)3 = = 4802 in.4 12 12 37,900 βt = = 3.9 2 × 4802 Is = 8.10.5.5 8.10.6 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, ℓ2/ℓ1 DQG[f1 = 0. Therefore, the top line of the table controls. Mu, pos. cs = 0.60 × 45 ft-kip = 27 ft-kip Determine the 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, ext. neg, ms = 83 ft-kip − 63 ft-kip = 20 ft-kip Mu, pos., ms = 45 ft-kip − 27 ft-kip = 18 ft-kip American Concrete Institute – Copyrighted © 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 Mu calculations for the interior span. 93 If there was no wall supporting the exterior edge of the slab, this moment would be used to calculate the twoway shear 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 M at either end of the panel. Mu, neg, cs = 63 ft-kip Mu, neg, ms = 20 ft-kip Mu, pos, cs = 27 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 columns aare square, so b1/b2 = 1. 8.4.2.3.1 balanced; that is, different on either side of thee 1 = 0.6 8.4.2.3.2 column. This difference in slab moments, Msc, must γ f = 2 b1 8.4.2.3.3 be transferred into the column, usually by a combi1+ 3 b2 8.4.2.3.4 QDWLRQRIÀH[XUHRVVKHDU(T FDOFXODWHV 8.4.2.3.5 a factor that determines the fraction of Msc trans8.4.2.3.6 IHUUHGE\ÀH[XUH,QWKLVH[DPSOHWKHSHULPHWWHG PRGL¿FDWLRQRWKLVIDFWRUDUHQRWXVHG 8.4.2.3.3, The effective slab width to resist γfMsc is the width This concentration of reinforcement within the effec8.4.2.3.5 tive slab width is considered during the detailing of of the column plus 1.5h of the slab on either side RIWKHFROXPQ7KHUHTXLUHVLQFUHDVHG¿FLHQW WKHFROXPQVDQGWKHIRUPDOLQWRWKH6HFWLRQ reinforcement within the effective slab width to Figure E1.4 shows undistributed total panel moments. resist γfMsc. 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 strip moments are split into two half-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)ℓ2ℓn2 − qDuℓ2ℓn2] ZKHUHWKHqLGLFDWHVWKHVKRUWHUVSDQ:KHQWKH VSDQVDUHWKHVDPHDVLQWKHUHSDPHWKHqVLPSO\ 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 ÀH[XUHDQGWZRZD\VKHDULQWRWKHFROXPQ7KH alcu s are discuss two-way shear calculations discussed in Step 7. uired to transfe The amount of ste steel required transfer γfMsc into À LVGHWHUPLQHG WKHFROXPQVWLDÀH[XUHLVGHWHUPLQHG QI PHQWGHWHUPLQHG QDWH 7KHÀH[XUDOUHLQIRUFHPHQWGHWHUPLQHGLQDWHUVWHSV d As in this is allowed to be us used to meet the requir required s 13, 3 the required require reinforcenforcestep. Therefore, att step ment from this step will be checked. At an interior column: Msc = 0.07[(qDu + 0.5qLu)ℓ2ℓn2 − qDuℓ2ℓn2] 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 an exterior column: Msc = 0.3Mo Msc = 0.3(127 ft-kip) = 38.1 ft-kip At interior columns: erior column γf = 0.6 20 20.1 ft-kip kip Msc = 2 γfMsc = (0.6)(20.1 0.1 ft-kip) ip γfMsc = 12.1 ft-kip U ng th hod descr Using the method described in Step 8, the amount of XUDO UHTXLUHG ÀH[XUDOVWHHOUHTXLUHGZLWKLQh of the column is: As = 0.49 in.2/45 in. or 0.13 in.2/ft exteri columns: At exterior γf = 0.6 Msc = 38.1 ft-kip γfMsc = (0.6)(38.1 ft-kip) γfMsc = 22.9 ft-kip Using the method described in Step 8, the amount of ÀH[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 2QHZDVKHDUVODEDUHDUHO\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 LWLVVKRZQWKDWWKHVODEVKHDUVWUHQJWKφVn, 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 × (c1 + d) + 2 × (c2 + d) bo = 2 × (24 in. + 5.6 5 in. in in.) + 2 × (24 in. + 5.6 in.) 118 4 in. in bo = 118.4 verag 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—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 bo d 29.6 in. × 29.6 in.) 283 kip (Vu = |14 ft × 18 ft − |) × (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. × 5.6 in. Calculate the shear stress due to moments at an interior column: γ v = 0.4 M sc = 20.1 ft-kip c AB = 14.8 in. J c = 97688 in.4 γ 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 due to moment transferred to tricity oof shear. the column via eccentricity γ v M sc c AB Jc Calculate cula the design she shear h stress at an interior column: 10 ksii + 0.015 ksi k = 0.121 ksi vu = 00.106 N e th Note that thesee calculati calculations are conservative. Msc as ume that some me live lo assumes load is not present to produce alan ments, bu unbalanced moments, 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 ÀVDVXPHGWREHIRUÀH[XUHDVWKHVODELVLJKWO\ 8.5.1.1 WKHÀH[XUDOUHLQIRUFHPHQWUHTXLUHGDWDOOFULWLFDOVHFWLRQV 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, respec8.5.2 tively, to determine the reinforcement required at each 8.5.2.1 7RGHWHUPLQHWKHDPRXQWRIÀH[XUDOVWHHOUHTXLUHG location. 8.4.2.3.5 this example solves the following quadratic equation: Reinforcement in an exterior panel (a) (φM n = φ | As f y | d − | | ((2)) Column strip at the columns: Mu = 63 ft-kip (As f y ( ) φM n = φ | As f y | d − 2 × 0.85bf c′) | |) ((Solving the quadratic equation gives ω= As f y bdf c′ φM n = φ(bdf c′ω (d − 0.59ωd)) φM n = φ(bd 2 f c′ω (1 − 0.59ω)) ω = 0.0664 ∴ ωbdf c′ As = fy As = (0.0664)(84 in.)(5.6 in.)(5000 psi) 60,000 psi As = 2.61 in.2 6HWφMn = MuDQGVROYHIRUÀ Column sstrip at midspan: Mu = 27 ft-kip ving the quadratic equation gives: Solving 00.0278 0 ω = 0.0 ∴ ωbbdf c′ As = fy As = (0.0278)(84 in.)(5.6 in.)(5000 psi) (0 (84 in.)(5 60,000 psi 60 Exterior Panels: Column strip at column line: As = 2.61 in.2 Middle strip at column line: As = 0.81 in.2 Column strip at midspan: As = 1.09 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 th the same method, 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 diagr diagram Fig. E1.9—Middle strip moment diagram 6WHS'HVLJQVWUHQJWK±2QHZDVKHDU 8.5.3.1.1 2QHZDGHVLJQVKHDUVWUHQJWKLVFDOFXODWHGLQDFF22.5 cordance with 22.5. φ = 0.75 Vn = 2 f c′bd φVn = (0.75)(2) ( ) (1 kip) 5000 psi (168 in.)(5.6 in.) | (1000 lb |) φVn = 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 + | f c′ = (β) 1000 vn = 4 f c′ = (α d) vn = | 2 + s | bo) (fc′ = 3.89 5000 ksi = 0.275 ksi * 1000 *controls φvn = 0.75 × 0.275 ksi = 0.206 ksi This is greater than the required strength for interior columns of 0.121 ksi from Step 7; therefore, two-way shear at interior columns is okay. The assumption that two-way shear reinforcement is QRWUHTXLUHG QRWUHTXLUHGDWWKHVHORFDWLRQVLVFRQ¿UPHG QLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVO 6WHS5HLQIRUFHPHQWOLPLWV0LQLPXPÀH[XUDOUHLQIRUFHPHQWLQQRQSUHVWUHVVHGVODEV PDUH m Table Ta 88.6.1.1, .6.1.1, From 8.6 $WOHDVWDPLQLPXPDUHDRIÀH[XUDOUHLQIRUFHPHQW oca here 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. 0LQLPXPÀH[XUDOUHLQIRUFHPHQWFRQWURÀVDWWKH 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 UHLQIRUFHPHQWWKHVSHFL¿HGFRYHULVLQ 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: ψs = 0.8 ψe = 1.0 ψt = 1.0 λ = 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 ψ t ψ e ψ s | | 3 y d Ad = | | 40 λ 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 lices are shown in ACI 318-14 18 Fig. 8.7.4.1.3. ℓd = 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 substan substantive amount. Therefore, class required B splices plice are required. in = 27.5 in. ℓst = 1.3 × 21.2 in. us ℓst = 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 limits are deter8.7.2.1 mined. The bar spacing for design strength is also 25.2.1 reviewed. 8.7.2.2 101 Minimum 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 clear spacing is 1.33 in. With a No. 5 bar, this equates to a minimum spacing of approximately 2 in. Maximum spacing is limited by Section 8.7.2.2: At critical sections, the maximum spacing is the lesser of 2h (2 × 7 in.) and 18 in., so 14 in. controls. All other sections, the critical spacing is the lesser of 3h (3 × 7 in.) and 18 in., so 18 in. However, because all of the bars cross a critical section, use a maximum 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.2/0.31i /0.31in.2 = nine No. 5 bars over 7 ft – spacing is 9 in. dle strip at column line: Middle 0 1 in.2/0.31 in. n.2 = six N 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 check to verify that the reinforcement The minimum requirements for all column strips 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[XUHDVDWLV¿HGVXLQJWKH 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¿HG1RWHKDWLIWKLVKDGQRWEHHQPHWDGGLWLRQDO steel would have been required to be placed within the HIIHFWLYHVODEZLGWKDVGH¿QHGLQ6HFWLRQ 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. PHQWOHQJWKV7KH¿JXUHIRU¿QDOOH\RXWUHIOHFH8.7.4.1.1 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 requireme requirements at this location) 102 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 15: Reinforcement detailing – Structural integrity 8.7.4.2 Structural integrity for a two-way slab is met by 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). Section 8.7.4.2.2 requires at least two of the column strip bottom bars pass through the column inside the column reinforcement cage. 6WHS6ODEFROXPQMRLQWV 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¿HGLIWKHVSDFLQJRIWKH XPQ KHDGEFR FROXPQWLHVLQWKHVODEFROXPQMRLQWVDWLV¿HVA DQGE a)) (a) (15.4.2(a)) s = 0.4 in.2 × 60, 000 psi 0.75 5000 psi × 24 in. s = 18.9 in. (b) (15.4.2(b)) s = 0.4 in.2 × 60,000 psi 50 × 24 in. s = 20.0 in. %HFDXVHWKHVSDFLQJLVODUJHUWKDQWKHMRLQWGHSWKRI 7 in., only one tie is required within the slab-column MRLQWLQHDFKH[WHULRUDQGLQWHULRUFROXPQ 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 2.61 1.09 Minimum 1.06 1.06 1.06 1.06 Maximum spacing, assuming No.5 bars 1.86 1.86 1.86 1.86 As required, middle strip, in.2 External bays Internal Bays Column lines Midspan Column lines Midspan Strength 0.81 0.73 0.81 0.73 Minimum 1.06 1.06 1.06 1.06 Maximum spacing, assuming No. 5 bars 1.86 1.86 1.86 1.86 Note: The highlighted cells indicate the required reinforcement that controls design. Fig. E1.10—Summary sketch of required bars, column strip American Concrete Institute – Copyrighted © Material – www.concrete.org Two-Way Slabss Step 18: Summary sketches of required bars Figures E1.10 and E1.11 are summary sketches of required bars for this example. 104 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E1.11—Summary sketch 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 This example is an interior column strip along grid line B in a nonprestressed two-way slab without beams between supports. This example uses the moment distribution method to determine 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: 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 SVL fy = 60,000 psi)LJ(²)LUVWÀRRUSODQ ACI 318-14 Discussion Step 1: Geometry 8.4.2 The moments in the slab are calculated using the 8.4.2.1 Equivalent Frame Method (EFM) in accordance 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 8.11.1.1 8.11.1.2 8.11.1.3 8.11.1.4 8.11.2 8.11.2.1 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 column into the next span regardless of the stiffness of the columns. The EFM softens the columns to simulate this effect on slab moments E\LQFRUSRUDWLQJWKHÀH[LELOLW\RIWKHVODEWRUVLRQDO member in the equivalent column stiffness. Fig. E2. E2.2—Equivalent Equivalent frame strip Step 2: Analysis – Equivalent colu column stiffness determination iffness determ tion 8.11.3 'HWHUPLQHWKHJHRPHWU\FRHI¿FLHQWVQHFHVVDU\WR JH \FRHI¿FLHQWVDQG DUU\RYHU JH \FRHI¿FLHQWV 8.11.4 use the design aids aid to determine the equivalent eq vale col8.11.5 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 intersecting with the column, 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\LVDVOFRQVLGHUHGHTXDO7KLVLVSPOL¿HVWKH calculations. Kt = ∑ 9 Ecs C (c) A 2 |1 − 2 | (A2) 3 ℓ2 = 14 ft = 168 in. Ecs = Ecc c2 = 24 in. Interior column torsional members For the torsional member at the interior columns and the slab portion of the torsional member at the exterior columns, x = 7 in. y = 24 in. and (x) x3 y C = ∑ |1 − 0.63 × | (y) 3 7 in.) (7 in.)3 × 24 in. (C = |1 − 0.63 × | (24 in.) 3 4 C = 2240 in. Kt = 19 191 ch side of the column. olu for each Therefore, the total iona member ember b stif f torsional stiffness at an internal column is Kt = 2 × 191 = 382 The basement wa wall is monolithic onolithic with the column External E erna column n torsiona torsional members and provides substantial exterior ub al stiffness to the exter F tion of th For the wall portion the torsional member at the mn but the wall rotatio equivalent column, rotation will bbe ex erior 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[XUDOVWLIIQHVV7KHEDVHPHQWZDOOPHQVLRQ 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 = ∑ |1 − 0.63 × | (y) 3 12 in.) (12 in.)3 × 113 in. (+ 2240 in.4 C = |1 − 0.63 × | (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 × 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

0.09 in.2 LQ\ )LJXUH(VKRZVWKHLQDOFRQ¿JXUDWLRQIRIWKH 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 or prestressed, cast-in-place, precast, or composite. This chapter discusses cast-in-place, 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 Chapter 23, Strut-and-Tie. Beams are designed in accordance with Chapter 9 of ACI 318-14 for strength and serviceability. Beams are assumed to be approximately horizontal, with rectangular or teeVKDSHGDVWHPSDQGDÀDQJH FURVVVHFWLRQV7KHÀDQJHZLGWKRI tee-shaped beams are geometrically limited by Section DQGUHVSHFWLYHO\RI$&,DQGKHÀDQJH LVDVVXPHGWRDFWZLWKWKHEHDP¶VÀH[XUDO strength. essed, that are Beams, either nonprestressed or prestressed, HFRQVLGH PRQROLWKLFZLWKWKHÀRRUIUDPLQJFDQ EHFRQVLGHUHGODWHUDOO\ ROLWKLF EUDFHG)RUEHDPVWKDWDUHQRWPRQROLWKLFZLWKWKHÀRRU$&, 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\GHÀHFWLRQ 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 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 depth if the beam and slab are monolithic or if the slab is composite with the beam in accordance with Chapter 16 of ACI 318-14. 7.2.2 '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 SVI²DQG IRU 37 EHDPV WKH GHVLJQHU PXVW FDOFXODWH GHÀHFWLRQV 'HÀHFWLRQV IRU FDOFX- ODWLRQ IURP FDOFXODWLRQ DQDO\VLV PHWKRG 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 GHÀHFWLRQV &KDSWHU 'HÀHFWLRQ RI WKLV + DQGERRN LQFOXGHV VHYHUDO GHVLJQ H[DPSOHVRQGHÀHFWLRQFDOFXODWLRQVZLWKGHVLJQDLGVIRU7 DQG/FURVVVHFWLRQEHDPV)RU 37 EHDPV 6HFWLRQ $&, GH¿QHV WKUHH FODVVHV RI EHKDYLRU RI SUHVWUHVVHG ÀH[XUDO PHPEHUV 1) uncracked, U, 2) 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′ 'HÀHFWLRQFDOFXODWLRQVIRU&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 reinstrengthh (< 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 have no concrete or steel service tress limits, mits only strength trength requirements. re stress For pre- and- posttensi ed bbeams, 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 post-tensioned UDO WHQVLRQ VWUHVVHV LV D FULWLFDO SDUW RI WKH GHVLJQ SURFHVV $&, ,Q 6HFWLRQ RI $&, EHDPV DUH FODVVL¿HG ,Q 6HFWLRQ presented above, as U (uncracked), C (cracked), or T (transition). Beams with bonded post-tensioning, the combined area of post-tensioning and deformed bars, must be able to resist a factored 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¿OHDQGHIIHFWLYHSUHVWUHVVPXVWEHGHWHUPLQHGDQG7KHSUR¿OHDQGWKHWHQGRQIRUFHDUHGLUHFWO\ related to the beam forces and moments created by the post-tensioning. A common approach to calculate PT beam moments is to use the "load balancing" concept, where the SUR¿OHLVXVXDOO\WKHPD[LPXPSUDFWLFDOFRQVLGHULQJFRYHU UHTXLUHPHQWV WKH WHQGRQ SUR¿OH LV SDUDEROLF WKH SDUDEROD has an angular "break" at the column centerlines, and that the tendon anchors are 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¿QHGDQGWKHWHQGRQIRUFH LV DVVXPHG 7KH DQDO\VLV LV H[HFXWHG ÀH[XUDO VWUHVVHV DUH FDOFXODWHGDQGWKHGHVLJQHUDGMXVWVWKHSUR¿OHIRUFH both, depending on results and design constraints. In the direct approach, the designer determines the highest tension stress, then rearranges equations so that the analysis calculates the tendon 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 simpli¿HGDSSUR[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. ÀHFW %HDP PRPHQWV VKHDU DQG GHÀHFWLRQV 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 Analysis ab hat provide moment moment Tables (ACI SP-17DA) has tables that rts and midspan for va us and shear forces at beam supports various oment of in boundary and loading conditions. The moment inertia and modulus of elasticity values used in the classic elastic analysis are addressed in ACI 318. Redistribution of elastic moments calculated by a classical method is permissible. 7KHHQJLQHHUFDQDOVRXVH¿QLWHHOHPHQWVRIWZDUHWRFDOFXODWHPRPHQWVVKHDUDQGGHÀHFWLRQVDORQJWKHEHDP¶V¶OHQJWK The moment of inertia and modulus of elasticity values used LQWKH¿QLWHHOHPHQWPRGHOVKRXOGEHDFFXUDWHIXOO\FRQVLGHUHGWR REWDLQUHDOLVWLFGHÀHFWLRQVDQGGHVLJQIRUFHV5HGLVWULEXWLRQ 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 ÀH[XUH VKHDU WRUVLRQ DQG D[LDO IRUFH $W each section along a beam's length, the design strength is at least equal to the factored load effects; mathematically H[SUHVVHGDV ϕSn•Su. 7.4.1 Flexure—Reinforced concrete beam design for ÀH[XUH W\SLFDOO\ LQYROYHV D VHFWLRQDO GHVLJQ WKDW VDWLV¿HV the conditions of static equilibrium and strain compatibility across the depth of the section. Following are the assumptions for strength design method OLVWHGLQ6HFWLRQRI$&,¿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 continue increasing. The strain hardening of st steel is ignored. d) Tensile strength of concrete is neglected. (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 (ϕ —A section's s s Mn is calculated c from internal forces (ϕM 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 2 ACI 318-14); tension controlled, forcement 21.2.2, FRPSUHVVLRQ İs•FRPSUHVVLRQFRQWUROOHGİ s"İy = 0.002; İy; and ion, 0.002 < İs < 0.005. transition, Reinfor Reinforced 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 FRQGLWLRQZLWKDOORZVUHGLVWULEXWLRQRIVWUHVVHVDQGVXI¿cient 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 0.75ȡb. The Code lowers the strength reduction (ϕ)-factor for transition-level strains to account for reduced ductility in these sections. 9DULDWLRQ RI ϕDFWRUV 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 ÀH[XUH VWUHQJWK PDWKHPDWLFDOO\ H[SUHVVHG DVMu " ϕMn. 7.4.1.2 Rectangular sections with only tension reinforcement—Nominal moment strength of a rectangular section with nonprestressed and prestressed 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 (7.4.1.2a) Substituting the corresponding components for T and C: Asfy + Apfps = 0.85fcgȕ1cb (7.4.1.2b) a = β1c = As f y + Ap f ps (7.4.1.2c) 0.85 f c′b Take moments about the concrete resultant, and Mn is calculated as: β c⎞ a⎞ a⎞ ⎛ ⎛ ⎛ M n = T⎜ di − 1 ⎟ = As f y⎜ d − ⎟ + Ap f ps⎜ d p − ⎟⎝ ⎝ ⎝ 2 ⎠ 2⎠ 2⎠ ( ) ( ) (7.4.1.2d) where fps is calculated in Section 20.3 of ACI 318-14. Assume that a ȕ1c and rearrange expressions: Fig. 7.4.1.1—Strain distribution. For reinforced concrete sections with single layer tension reinforcement, d = dt and İs = İt (7.4.1.3). The stress block JHRPHWULF SDUDPHWHU ȕ1 is between 0.85 and 0.65. For FRQFUHWH VWUHQJWKV KLJKHU WKDQ SVL WKH YDOXH RI ȕ1 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 compresnfor The inte sion rreinforcement. internal force couple adds to the i al m trength w sectional moment strength 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 c tension reinforcement-concrete compression couple; and 2) ength from the additional tension reinforcementmoment strength ression rein compression reinforcement couple, assuming both sets of reinforcem yield, as illustrated in Fig. 7.4.1.3. 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 = Asgfsgd − dg E Assuming fsgLVHTXDOWRfy, Fig. 7.4.1.2—Strain distribution and net tensile strain in a nonprestressed beam. M2 = Asgfy(d − dg 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 strength: 135 136 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Because the steel couple does not require an additional 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 ¿EHUZKLFKZLOOVWUDLQWRLQDWQRPLQDOVWUHQJWK Depending on the location of the compression reinforcement within the overall strain diagram, it is possible that the compression reinforcement has less strain than 0.002 at nominal strength and, therefore, does not yield. The designer in this case, increases the compression reinforcement area proportional to the ratio of yield strain to compression steel strain. The strain in compression steel, İsgFDQEHFRPSXWHG from Fig. 7.4.1.3 as İsg İs(cídg díc), once İs 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 UHVXOWLQJ LQ D 7VHFWLRQ 7VHFWLRQ IIHFWLYHIZ 7KHÀDQJHZLGWKRI7VHFWLRQLVWKHHIIHFWLYHZLGWKRIWKHÀDQJHZLGWKRI$& DQG WKH ZLGWK RI $& DQG GH¿QHG LQ 6HFWLRQ RI$&, rectangular beam forms the web. Pre Precast double T-sections)LJ D²(TXLYDOHQW VWUHVV GLVWULEXWLRQ IRU 7VHFWLRQ ÀDQJH LQ WKH ZHE. DOVR EHQH¿W IURP DQ LQFUHDVH LQ EHDP VWLIIQHVV GXULQJ construction. 7KHÀDQJHZLGWKLQPRVW7VHFWLRQVLVVLJQL¿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 GLDJUDPZLWKLQWKHÀDQJHGHSWK7VHFWLRQVDUHDQDO\]HGWKH same as rectangular sections, with section width equal to the HIIHFWLYHÀDQJHZLGWK In heavily reinforced T-sections, the area of tension reinforcement in the web (required by the applied moment) EULQJVWKHQHXWUDOD[LVEHORZWKHÀDQJHFUHDWLQJDFRPSUHVsion zone in the web. In such a case, the total moment strength FRQVLVWVRI WHQVLRQVWHHOIRUFHHTXDOWRWKHÀDQJHFRQFUHWH 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"ϕMn = ϕ(Mnf + Mnw) (7.4.1.4a) where ⎡ φM n f = φ ⎢0.85 f c′bh f ⎛ hf ⎞⎤ ⎜⎝ d − 2 ⎟⎠ ⎥⎦ ⎣ ⎦ a⎞ a⎞ ⎛ ⎛ φM nw = φAs f y⎜ d − ⎟ + φAp f ps⎜ dp − ⎟⎝ ⎝ 2⎠ 2⎠ (7.4.1.4b) (7.4.1.4c) Many eng Many engineers calculate Mnf¿UVWIURPHTXLYDOHQWRYHUKDQJLQJ WKH DUHD RI HQVLRQVWH WKHDUHDRIWHQVLRQVWHHOQHHGHGWREDODQFHWKHÀDQJH i then the h calculated assuming a rectangular concrete. Th The Mnw is cross sectionn as shown in Fig. 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 moments are a result of the column's vertical restraint of the beam against the PT "load" at each support. Because the post-tensioning force and drape are determined during the service stress checks, secondary moments can be quickly calculated by the "load-balancing" analysis concept. A simple way to calculate the 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 0LQLPXPÀH[XUDOUHLQIRUFHPHQW—Nonprestressed reinforcement 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 requires a minimum area of tension reinforcement (refer to Section 9.6.1.1 of 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 determinate bea beams, where the T-se T-section UF W UHTXLUHG WR SUR YLGH ÀDQJH 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 thee smaller of 22bw or fore, bw in Eq. (7.4.1.5a) is replaced WKHÀDQJHZLGWK6HFWLRQRI$&, +RZHYHU when the steel area provided in every section of a member LV VXI¿FLHQW WR SURYLGH ÀH[XUDO VWUHQJWK 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 at least equal to 1.2 times the cracking moment (Section 9.6.2.1 of ACI 318-14): ϕMn•Mcr (7.4.1.5b) For prestressed beams with unbonded strands, abrupt ÀH[XUDO IDLOXUH LPPHGLDWHO\ DIWHU FUDFNLQJ 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¿EHU/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 beams (Section R22.5.8.3 of ACI 318-14). support, and decreases as it moves away from the support, usually at a slope equal to the magnitude of the unit uniform ORDG,QUHJLRQVRIKLJKÀH[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 (the support). Consequently, cracks in concrete tend to stress (th "point" toward the th region 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 resistt 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 lculated aas the combination of concrete shear strength, Vc, and the steel shear reinforcement, Vs treng (Section (Secti n 22 22.5.1.1 of ACI 3318-14). The nominal concrete shea reng Vc is often calculated using the simple ACI shear strength 2.5.5.1) Vc = 2λ f c′bw d instead of the more 318-14, Eq. (22.5.5.1) quations iin Table 22.5.5.1 of ACI 318-14. For detailed equations DPV UHVLVWLQJ EHDPV 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 HTXDWLRQ 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, Vcw (Section 22.5.8.3.2). The factored shear (or required 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. 7.4.2.1c of this Handbook) or closed stirrups. Shear cracking is assumed to occur at 45 degrees from horizontal. $&,XVHVDWUXVVDQDORJIRUVKHDUÀRZZKHUHWKHVWLUrups 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). Beam loaded near bottom (R9.4.3.2b, ACI 318-14). Fig. 7.4.2.1c—Shear reinforcement. as diagonal struts. The longitudinal 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 × (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 ϕVc/2 is less than Vu, shear reinforcement, Av, is QHHGHG7KHVWUHQJWKUHGXFWLRQIDFWRUϕIRUVKHDULV7KH required area per unit length is: s"ϕAvfytd)/(Vu±ϕVc) (7.4.2.2) Vu/ϕ – 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 exceeds the required 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. $W\SHRIULEEHGÀRRUVODENQRZQDVDMRLVWV\VWHPLVRIWHQ FRQVWUXFWHGZLWKRXWVKHDUUHLQIRUFHPHQWLQWKHMRLVWVULEV$ MRLVWV\VWHP¶VUHODWLYHGLPHQVLRQDOOLPLWVXFKDVVODEWKLFKNness, rib width, and rib spacing, are provided in ACI 318-14, 6HFWLRQ,IWKHULEEHGÀRRUVVWHPGRHVQRWFRQIRUPWR DOOWKHFRGHOLPLWVXFKDVDVKDOORZSDQMRLVWV\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¿UVWWHUPLVWJRYHUQLIcg!SVL$&, 318-14 has an upper limit of f c′ to 100 psi, which corresponds to 10,000 psi concrete strength. Section 22.5.3.2 of ACI 318-14 allows the value of f c′ to be greater than 100 psi if the reinforced and prestressed beam has shear reinforcement per Section 9.6.3.3 and 9.6.4.2 of ACI 318-14. Refer to the C Code commentary on this section for more rmation. information. rsion—Beam m ttorsion (or twisting) creates 7.4.3 T Torsion—Beam al sh sectional shear stresses th that increase from zero stress at m s sectional nal center to the maximum at the section the beam's SHULP HU 7 UH$&, DVVXPHV WKDW VLJQL¿FDQW SHULPHWHU 7KHUHIRUH$&, i stre rs only around 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 torsion (most common condition). Statically determinate (or equilibrium) torsion is the condition where the equilibrium of the structure requires the beam's 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 connected members. In Fig. 7.4.3b(a), the determinate beam must resist 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 WRUVLRQRIWKHEHDPRUE\VODEÀH[XUH,QWKHUWRUGVLIWKH American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 139 Condition Spacing Provision in ACI 318-14 Vu"ϕVc/2 No shear reinforcement required 9.6.3.1 Minimum shear reinforcement s ≤ ϕVc/2 < Vu"ϕVc d ≤ 24 in. 2 s≤ Nonprestressed φVc < Vu ≤ φVc +φ4 f c′bw d d ≤ 24 in. 2 9.7.6.2.2 3h s≤ ≤ 24 in. 4 Prestressed s≤ Nonprestressed φVc +φ4 f c′bw d < Vu ≤ φVc + φ8 f c′bw d d ≤ 12 in. 4 9.7.6.2.2 3h s≤ ≤ 12 in. 8 Prestressed Vu > φVc + φ8 f c′bw d 9.6.3.1 Increase cross section 22.5.1.2)LJD±±7RUVLRQVWUHQJWKGH¿QLWLRQVRIVHFWLRQSURSHUWLHV beam loses torsional stiffness, the slab can resist the eccenWULFORDGLQJIHIIHFWVWKURXJKÀH[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 QRW FDXVH D VWUXFWXUDOO\ VLJQL¿FDQW UHGXFWLRQ LQ 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 strength. This limit is based 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) λ 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 reinforced for torsion per ACI 318-14 are ductile and thus will continue to twist after 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 DQ DQJOH ș IURP WKH member axis, so the torsion strength from closed stirrups is calculated as Tn = 2 Ao At f yt s cot θ (7.4.3.1a) where AoLVWKHJURVVDUHDHQFORVHGE\WRUVLRQDOVKHDUÀRZ 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\VODEVIRUIURPDWLRQ Fig. 7.4.3c—Determining Aoh. fyt is the yield strength of transverse reinforcement, einforce psi. ACI EHJU VSHFL¿HVWKDWDQJOHșPXVWEHJUHDWHUWKDQGHJUHHV SOLF J DQGOHVVWKDQGHJUHHVIRUVLPSOLFLW\LQGHVLJQXVHș 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 ⎟⎠ ≤ φ ⎜⎝ 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 ph/8, or 12 in. The spacing between the longitudinal bars around the section periphery must not exceed 12 in. ACI 318, Eq. (22.7.7.1b) requires that the longitudinal bar area, AƐ, be placed around the section periphery. Section 9.6.4.3 of ACI 318-14, requires a minimum area of longitudinal reinforcement AƐPLQ 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 θ (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, VSDFLQJDQGFRQ¿JXUDWLRQ 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 menW²/RQJLWXGLQDO VKRXOG EH SODFHG DW /RQJLWXGL O UHLQIRUFHPHQW 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 equireme 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 QYLURQPHQ E\ D VXI¿FLHQWO\ WKLFN FRQFUHWH DJJUHVVLYH HQYLURQPHQWV efer 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 7VHFWLRQÀDQJHVDUHLQWHQVLRQSDUWRIWKHÀH[XUDOWHQVLRQUHLQIRUFHPHQWQHHGVWREHGLVWULEXWHGRYHUWKHÀDQJHZLGWK 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. When 1/10 of the span is smaller than the effective width, additional reinforcement should be provided LQWKHRXWHUSRUWLRQVRIWKHÀDQJHWRPLQLPL]HZLGHFUDFNV in these slab regions. 7.6.1.4 0D[LPXP VSDFLQJ RI ÀH[XUDO UHLQIRUFHPHQW— Beams reinforced with few large bars could experience cracking between the bars, even when the required tension reinforcement area is provided and the acceptable limits for various exposure conditions, ACI 318-14 Section 24.3.2 VSHFL¿HV D PD[LPXP VSDFLQJ 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 ≤ 15 ⎜ − 2.5cc ⎝ f s ⎟⎠ ⎛ 40, 000 ⎞ s ≤ 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 loads or may be taken as 2/3fy. Note that Eq. GRHVQRWSURYLGHVXI¿FLHQWFUDFNFRQWUROIRUEHDPV subject to very aggressive exposure 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 Reinforced Concrete Design Handbook Design Aid – Analysis Tables (ACI SP-17DA) (refer to Fig. 7.6.1.5 and Section 9.7.2.3 of ACI 318-14). 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—Stirrup bar size is usually a 1R1RRU1R6PDOOHUVWLUUXSVDUHHDVLHUWREHQGDQGLWLVGLI¿cult to bend. Note that stirrup spacing less than 3 in. can FUHDWHGLI¿FXOWLHVLQSODFLQJFRQFUHWH7KHUHIRUHVRPHHQJLneers increase the stirrup spacing by doubling the stirrups (refer to Fig. 7.6.2(d)). For wider beams, 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 and transverse reinforceespectively. The lo ment, respectively. 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 resistt 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 hooks b repl may be replaced by a 90 degr degree hook where the stirrup end is FRQ¿ GDQG DLQHGDJDL FRQ¿QHGDQGUHVWUDLQHGDJDLQVWVSDOOLQJE\ VODERUÀDQJHRI i (refer er 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 0.85fcgZLWKDGHSWKRIa ù1cZKHUHù1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3. 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. β1 = 0.85 – 0.05(5000 psi – 4000 psi) = 0.8 1000 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) For positive moment: b = bf = 46.5 in. 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.) 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 = wuℓ2/16 = 128 ft-kip Mu,midspan = wuℓ2/8 = 256 ft-kip 9.5.1.1 The beam strength must satisfy the following equations at each section along its length: Mn•Mu Vn•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 ≤ φM n = φAs f y ⎜ d − ⎟ ⎝ 2⎠ 256 ft-kip = 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 ≤ (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. ot less than 0.004 in./in. (tension controlled), but not At midspan. a= As f y 0.85 f c′b and dc= a β1 (0.304)(6 in.2) = 0.80 in. a = 0.30 0.304As = (0.304)(6)(0.44 a/0. in. = 1.0 0 in c = a/0.8 in. ZKHUHù1 = 0.8 WKHUHIRUHEHDPVHFWLRQLVWHQVLRQFRQWUROOHG c < hfWK εt = εc (d − c) c Note that b = 18 in. for negative moments and 54 in. for positive moments. εt = 0.003 (25.7 iin. − 1.0 in.) = 0.074 0 in. 1.0 7KHUHIRUHDVVXPSWLRQLVFRUUHFW 7KHUHIRUH At support: As f y and c = a= 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. 7KHUHIRUHDVVXPSWLRQLVFRUUHFW 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 reinforcement must be at least the minimum required reinforcement at every section along the length of the beams. (a) As = 3 f c′ fy bw d As = 3 5000 psi (in (18 in.)(25.7 in.) = 1.63 in.2 60,000 6 000 psi Controls (b) As = 200 bw d fy As = 200 (18 in.) = 1.54 in.2 (iin.)(25.7)(60,000 psi 60 Because fcg!SVL(TD FRQWURO (TD RQWURO At mids midspan: 6)(0.44 in in.2) = 2.64 in.2 > As,min = 1.63 in.2 As,prov. = (6)(0.44 OK At suppo support: As = 1.1 in.2 < As(min) = 1.63 in.2 NG Therefore, use minimum reinforcement; four No. 6 at 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). CondiWLRQE DQGF RI6HFWLRQ9DUHVDWLV¿HG Fig. E2.5—Shear critical section. &RQGLWLRQF KRZHYHULVQRWVDWLV¿HGUHIHUWR note at end of this step). Vu = 28.5 kip 22.5.5.1 Vc = 2 f c′bw d Vc = 2 &KHFNLIVc•Vu Vc = (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 tirrups at 12 in. on center where; Provide n. < d/2 = 25.7 in. /2 = 12.8 in. OK 12 in. 22.5.1.2 Cross-sectional dimensions are selected to satisfy Eq. (22.5.1.2): (Vu ≤ φ Vc + 8 f c′bw d) %\VXSHUWLRQ%KLUVDWLV¿HG 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 Members of Similar Depth," ACI Structural Journal91R0D\-XQHSS American 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 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 = λ 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 ™bi + hi)pc is the perimeter of concrete gross area. pcp = 2(18 in. + 21 in. + 7 in. + 21 in.) = 134 in. 2 7KHRYHUKDQJLQJÀDQJHGLPHQVLRQLVHTXDOWRWKH ⎛ (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 Tu WINLS Tth = 14.0 ft-kip OK Torsion reinforcement is not required. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 171 Beams 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: ⎧1 in. ⎪ 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. + 0.375 in. + 0.75 in.) + 3.75 in. + 5 in. bw,req'd = 2(cover + dstirrup + 0.75 in.) + 5db + 5(1 in.)min,spacing (25.2.1) = 14 in. < 18 in. OK where dstirrup = 0.375 in. and db = 0.75 in. Therefore , six No. 6 bars can be placed in one layer in the 18 in. beam web with 1.8 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 ⎜ ⎟ − 2.5cc ⎝ f s ⎠ ⎛ 40,000 psi ⎞ s = 15 ⎜ ⎟ − 2.5(2 in.) = 10 in. ⎝ 40,000 psi ⎠ ⎛ 40, 000 ⎞ s = 12 ⎜ ⎟ ⎝ f s ⎠ ⎛ 40,000 psi ⎞ s = 12 ⎜ ⎟ = 12 in. ⎝ 40,000 psi ⎠ The maximum spacing concept is intended to OLPLWÀH[XUDOFUDFNLQJZLGWKV1RWHWKDWcc is the FRQFUHWHFRYHUWRWKHÀH[XUDOEDUVQRWWKHVWLUUXSV 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 No. 6 bar 7KHVLPSOL¿HGPHWKRGLVXVHGWRFDOFXODWHWKH development length of a No. 6 bar: Ad = f yψt ψe 25λ 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 ѱt EDUORFDWLRQѱt = 1.3 for top bars, because more than 12 in. of fresh concrete is placed below Bottom WKHPDQѱt =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 ⎠ ѱe FRDWLQJIDFWRUѱe = 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 (ℓst) = 1.3(ℓd) Top tension reinforcement RLQWIRUQHJDWLYHPRPHQW &DOFXODWHWKHLQÀHFWLRQSRLQWRUQHJDWLYHPRPHQW diagram: íMmaxíwu(x)2/2 + Vux = 0 Top: 1.3ℓd = (1.3)(33.1 in.) = 43.0 in., say, 4 ft 0 in. Bottom: 1.3ℓd = (1.3)(25.4 in.) = 33 in., say, 3 ft 0 in. (−128 28 fft-kip) 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 ℓn/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. ℓn/16 = (36 ft)(12)/16 = 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 of maximum stress and points along the span where bent or terminated tension bars are no longer required to UHVLVWÀH[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 in.)(0.9)(60 ksi) 2 2(0.44 in.)(60 ksi) ⎞ ⎛ × ⎜ 25.7 in. − ⎟ 2(0.85)(5 ksi)(46.5 in.) ⎠ ⎝ (256 ft-kip) − 1.58 kip/ft 2 2 Two No. 6 bars can resist a factored moment 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 ORQJHUHTXLUHGWRUHVLVWÀH[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. 168 in. + 25.7 in. = 193.7 refo e extend four four N Therefore, No. 6 bars 16 ft-6 in. from dspan midspan. Ex end the remaining aining two tw No. 6 bars > 1/4 six No. 6 Extend inim a minimum of 6 in. into tthe support, but not less than ℓd = 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¿HG but not both. 7KLVFRGLWLRQZDVVDWLV¿HGEDRYHE\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 tens on bars A minimum of on one-fourth tension o the support upport minimum 6 in. must extend into /RQJLWXGLQDOEDUVPXVWEHHQFORVHGE\FORVHG 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. cations. 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 dead 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&KDSWHUTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGWDGD\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 7KHEHDPGHÀHFWLRQZLOOEHFKHFNHGDQGFRPSDUHG to Table 24.2.2. Flange width oli with the slab oon one ne side The beam is monolithic in the middle over a 24 ft long section and slab on both sides for the remainder of the beam. At the maximum positive moment, the beam will behave DVDQ/EHDP7KHUHIRUHWKHHIIHFWLYHÀDQJHZLGWK on one side of the beam is the least from Table 6.3.2.1. 2QHVLGH ⎫ 6hslab ⎪ of web is ⎬ sw /2 the least of ⎪⎭ A n /2 (6)(7 in.) = 42 in. 7KHUHIRUHÀDQJHZLGWKbf = ℓn/12 + bw bf = 26.5 in. + 18 in. = 44.5 in. (16.5 ft)(12)/2 = 99 in. (28 ft (12) – 18 in.)/12 = 26.5 in Controls 2QERWKVLGHVRIWKHSRLQWKHEHDPLVPRQRlithic monolithically with the slab and will behave as a 7EHDP7KHÀDQJHZLGWKRQHDFKVLGHRIWKHEHDP is obtained from Table 6.3.2.1. 6.3.1 Each side ⎫ 8hslab ⎪ of web is ⎬ sw /2 the least of ⎪⎭ A n /8 8(7 in.) = 56 in. 7KHUHIRUHÀDQJHZLGWKbf = ℓn/8 + bw + ℓn/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/ft 7ULEXWDU\ORDGEHDPZHHQ*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$&\VVWHPVDGGSVIDVPLVFHOODQHRXVGHDG 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-weightt (7 in. thick) 14 ft)(16. ft)(16.5 ft/2)(0.15 kip/ft3) = 10.1 kip we assume beam is: Beam B4 self-weight : i deep. ep. 18 in. wide by 300 in. Note: 7 in. is the slab thickness ⎛ 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 of 15 psf: PSSD = (15 psf/1000)(14 ft)(16.5 ft)/2 = 1.7 kip Total dead load at B2 midspan: ™P = 10.1 kip + 3.3 kip + 1.7 kip = 15.1 kip /LYHORDG &ROFHQWUDWHGORDGIURP%HDP&ROHQDQG 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. 7KHEHDPUHMVLWVVJUDYLW\ORDGRQO\DWHUDOIRUFHV are not considered in this problem. PL = (0.1 ksf)(14 ft)(16.5 ft)/2 = 11.6 kip Pu = 28.5 kip 5.3.1a Distributed load: wu = 1.4D 5.3.1b wu = 1.2D + 1.6L 5.3.1a Concentrated load: Pu = 1.4PD Pu = 1.4(15.1 kip) = 21.1 kip 5.3.1b Pu = 1.2PD + 1.6PL Pu = 1.2(15.1 kip) + 1.6(11.6 kip) = 36.7 kip Controls wu = 1.4(0.56 kip/ft + (15 psf)(1.5 ft)/1000) = 0.82 kip/ft wu = 1.2(0.82 kip/ft)/1.4 + 1.6((100 psf)(1.5 ft)/1000) = 0.94 kip/ft Controls 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 beam moment at VXSSRUWVXVLQJFRHI¿FLHQWVIURP7DEOH%í Reinforced Concrete Design Handbook Design Aid – Analysis Tables, which can be downloaded from: . aspx?ItemID=SP1714DA Mu = wuℓ2/12+Puℓ/8 + Pua2b/ℓ2+Puab2/ℓ2 and the analysis shows the beam shear is Vu = wuℓ/2 +Pu/2 RP$SS 8VLQJFRHI¿FLHQWVIURP$SSHQGL[%5HLQforced Concrete Design Handbook Design Aid – n be downloa rom: Analysis Tables, wh which can 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)2/12 + (36.7 kip)(28 ft)/8 + (28.5 kip)(6.25 ft)2(21.75 ft)/(28 ft)2 + (28.5 kip)(6.25 ft)(21.75 ft)2/(28 ft)2 = 328 ft-kip Vu = (0.94 kip/ft)(28 ft)/2 + (36.7 kip)/2 + 2(28.5 kip)/2 = 60 kip Note that the (2)(28.5 kip) shear force represents the beams framing into the girder beam. two bea Mu = 255 25 ft-kip ns This distribution ass assumes the girder rem remains uncracked. If the girder, does crack, 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 = 279 ft-kip Accordingly, the moment at midspan must be Mu = (255 ft-kip) + (0.15)(328 ft-kip) = 304 ft-kip increased by the same amount: Note: Alan Mattock states that, "..., and it is concluded that redistribution of design bending moments by up WRGRHVQRWUHVXOWLQSUHPDWXUHFUDFNLQJRUH[FHVVLYHGHÀHFWLRQVEH\RQGWKRVHSUHGLFWHGE\WKHHODVWLFWKHRU\ 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: "The cut-off point at 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 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.) = 21.4 ft-kip 12 and from B4 Tu = (36.7 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) $VVXPLQJWKHEHDPVLVLWLOOEHWHQVLRQFRQWUROOHG0.9 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: İc = 0.003 22.2.2.2 QFUHWHLQÀH[XUHLVDYDUL 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQÀH[XUHLVDYDULable property and its value iis approximately 10 to 15 percent of the concr concrete compressive strength. ength, ACI 318 neglects eglects For calculating nom nominal strength, nsil strength. ngth. the concrete tensile d = 30 in. – 1.5 in. – 0.375 in. – 0.75 in. /2 = 27.75 in., say, d = 27.7 in. ent concrete com ressi Determine the eq equivalent compressive stress for design:: 22.2.2.3 The concrete compressive stress distribution is inelastic at high stress. The actual distribution of sually 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 tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRI a ù1cù1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3. 22.2.2.4.1 For fcg SVL β1 = 0.85 − 22.2.2.4.3 Find the equivalent concrete compressive depth, a, by equating the compression force in the section to the tension force (refer to Fig. E3.4): C=T 0.85fcgba = Asfy 0.85(5000 psi)(b)(a) = As(60,000 psi) 0.05(5000 psi − 4000 psi) = 0.8 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 = bf = 44.5 in. a= For negative moment: b = bw = 18 in. At support 0.85(5000 psi)(18 in.)(a) = As(60,000 psi) a= As (60,000 psi) = 0.784 As 0.85(5000 psi)(18 in.) Fig. E3.4—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) 'HVLJQWKHEHDPIRUWKHPD[LPXPÀH[XUDOPRPHQW 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•Mu Vn•Vu &DOFXODWHUHTXLUHGÀH[XUDOUHLQIRUFHPHQWDUHD using the following equation: Midspan 21.2.2 9.3.3.1 a⎞ ⎛ M u ≤ φM n = φAs f y ⎜ d − ⎟ ⎝ 2⎠ 0.317 As ⎞ ⎛ 304 ft-kip ≤ (0.9)(60 ksi) As ⎜ 27.7 in. − ⎟ ⎝ 2 ⎠ No. 6 bars; db = 0.75 in. and As = 0.44 in.2 As = 2.47 in.2; use six No. 6 Note that b = 18 in. for negative 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 = ZKHUHù1 = 0.8 εt = ε cu (d − c) c a β1 0.784 As ⎞ ⎛ 279 ft-kip ≤ (0.9)(60 ksi) As ⎜ 27.7 in. − ⎟ ⎝ 2 ⎠ As = 2.31 in.2XVHí1R Midspan p 7 s = (0.3 (0.317)(6)(0.44)(6 (in.2) = 0.84 in. a = 0.317A a/0. = 00.84 84 in /0 8 = 1.05 in. < 7 in. slab thickness c = a/0.8 in./0.8 refo shape hape assum Therefore, assumption is correct εt = 0.003 0 7.7 in. − 1 (27.7 1.05 in.) = 0.076 > 0.005 1.05 in. 1. Supports 84A 4 s = (0 (0.784)(6)(0.44 in.2) = 2.07 in. a = 0.784A c = a/0.8 = 2.07/0.8 = 2.59 in. εt = 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 = (b) As = 3 f c′ fy bw d Beams 9.6.1.1 9.6.1.2 183 As = 3 5000 psi (18 in.)(27.7 in.) = 1.76 in.2 60,000 psi 200 bw d fy Because fcg!SVL(TD FRQWURO Step 6: Shear design Shear strength 21.2.1(b) Shear strength reduction factor: 9.5.1.1 Vn•Vu At midspan: As(prov.) = 2.64 in.2 > As(min) = 1.76 in.2 OK At support: As(prov.) = 2.64 in.2 > As(min) = 1.76 in.2 OK shear = 0.75 9.5.3.1 22.5.1.1 Vn = Vc + Vs 9.5.1.2 21.2.1b 6KHDUUHLQIRUFHPHQWRU9.4.3.2 9.4.3.1 rce is taken n at the face of the Design shear force tical reaction ca ses vertical support becausee th the vertical 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-critica section. WLRQE DQGF RIDUHVDWLV¿HG&RQGLWLRQ QRW V¿HG D KRZHYHULVQRWVDWLV¿HG 3 kip Vu = 59.3 22.5.5.1 Vc = 2 f c′bw d Vc = 2 5000 psi(18 in.)(27.7 in.)/1000 = 70.5 kip &KHFNLIVn•Vu Vc = (0.75)(70.5 kip) = 52.9 kip Vn = 52.9 kip < Vu = 59.3 kip NG Therefore, shear reinforcement is required. Cross-sectional dimensions are selected to satisfy Eq. (22.5.1.2): 22.5.1.2 Vu ≤ φ(Vc + 8 f c′bw d) (Vu ≤ φ 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 Members of Similar Depth," ACI Structural Journal91R0D\-XQHSS American 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!Vc satisfying Eq. (22.5.10.1): Section AB Vs•Vu±Vc where Vs = Vs•NLS ±NLS NLS Av f yt d Vs ≥ s Av (8.5 kip) ≥ = 0.0051 s (60 ksi)(27.7 in.) Av V ≥ 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 ≤ 4 f c′bw d ? Vs = 8.5 kip < 4 f c′bw d = 141 kip OK Therefore, for stirrup spacing, use the lesser of: d/2 = 27.7 in./2 = 13.8 in. or 24 in. stirr Try No. 3 stirrups at 12 in. on center. 2(0.11 (iin..2) ⎛ Av ⎞ = = 0.018 in.2 /in. > 0.005 in.2 /in. ⎜⎝ s ⎟⎠ 12 in. pr prov OK 9.6.3.3 UH FHPHQWXVWEH WOHDV 6SHFL¿HGVKHDUUHLQIRUFHPHQWXVWEHWKHDWOHDVWWKH larger of: As ,min s = 0.75 f c′ bw Av , min f yt s psi ≥ 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) ≥ = 0.018 in.2 /in. > = 0.016 in.2 /in. 12 in. s s VDWLV¿HVWKHUHTXLUHPHQW American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 185 22.7.4.1(a) Beams Step 7: Torsion design Determine if girder torsion can be neglected. Check threshold torsion Tth: ⎛ Acp2 ⎞ Tth = λ f c′ ⎜ ⎟ ⎝ pcp ⎠ Determine portion of slab to be included with the beam for the torsional design: T-section between a and b and between d and e and Fig. E3.7—T-beam geometry to resist torsion. where Acp ™bihi is the area enclosed by outside perimeter of concrete Acp = (18 in.)(30 in.) + 2(23 in.)(7 in.) = 862 in.2 pcp ™bi + hi)pc is the perimeter of concrete gross area pcp = 2(18 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 21.2.1c JWK XFWLRQIDFWRU7 RUVLRQDOVWUHQJWKUHGXFWLRQIDFWRU3 ft-kip) kip)) = 17.5 ft-kip Tth = ((0.75)(23.3 th = 117.5 ft-kip p > Tu = 7.6 ft-kip OK T sion reinforcement orcement is not required between a and b Torsion een d and e. and between ed with Determine portion of the slab to be included the beam for torsional design. /VHFWLRQEHWZHHQb and d 7KHRYHUKDQJLQJÀDQJHGLPHQVLRQLVHTXDOWRWKH VPDOOHURIWKHSURMHFWLRQRIWKHEHDPEHORZWKHVODE (23 in.) and four times the slab thickness (28 in.). Therefore, use 23 in. (refer to Fig. E3.8). Fig. E3.8— L-beam geometry to resist torsion. Acp = (18 in.)(30 in.) + (23 in.)(7 in.) = 701 in.2 pcp = 2(18 in. + 23 in. + 7 in. + 23 in.) = 142 in. ⎛ 701 in.2 2 ⎞ Tth = (1.0) 5000 psi ⎜ ⎜ 142 in. ⎟⎠ 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 OK Torsion reinforcement is not required between b and d. American Concrete Institute – Copyrighted © Material – www.concrete.org 186 THE REINFORCED CONCRETE DESIGN 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 between the lo longitudinal te of: bars is the greatest ⎧1 in. ⎪ Clear spacing ⎨db ⎪4/3(d) agg ⎩ 1 in.. Controls rols Assume 3/4 in. maximum aggregate size. Therefore, clear spacing between horizontal bars must not be less than 1.0 in. 0.75 in. (3/4 in.) = 1 in. 4/3(3/4 Controls Check if six No. 6 bars can be placed in the beam's web. bw,req'd = 2(cover + dstirrup + 0.75 in.) + 5db + 5(1.0 in.)min,spacing (25.2.1) bw,req'd = 2(1.5 in. + 0.375 in. + 0.75 in.) + 5 (0.75 in.) + 5(1.0 in.) bw,req'd = 14 in. < 18 in. OK Refer to Fig. E3.10 for reinforcement placement in Beam B2. Fig. E3.10 for reinforcement placement. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 9.7.2.1 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 ⎜ ⎟ − 2.5(2 in.) = 10 in. ⎝ 40,000 psi ⎠ ⎛ 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 yψt ψe 25.4.2.2 Ad = 25.4.2.4 ZKHUHѱt LVEDUORFDWLRQѱt = 1.3 for top horizontal bars, because more than 12 in. of fresh concrete is SODFHGEHORZWKHPDQѱt = 1.0 for bottom horizontal bars, because not more than 12 in. of fresh ORZWKH FRQFUHWHLVSODFHGEHORZWKHPѱ e is coating factor; use bbars are uncoated DQGѱe = 1.0 because 9.7.1.3 25.5.2.1 9.7.3 Controls 20λ f c′ db einforcing g bar Splice length off N No .6 reinforcing 2. splice ce length is 1.3(Per Table 25.5.2.1, 1.3(ℓd). ⎛ (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 1.3ℓd = (1.3)(33.1 tom 1.3ℓd = (1.3)(25 (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 beyond ment Beam B1 a distance equal to the development length of 30 in. Four No. 6 lengths: ℓ = 14 ft + 2(1.5 ft) + 2(2.5 ft) = 22 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 Reinforcement must be developed at points of maximum stress and points along the span where terminated tension reinforcement is no longer UHTXLUHGWRUHVLVWÀH[XUH Six No. 6 bars are required to resist 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\RQGWKHSRLQWRILQÀHFWLRQWKHJUHDWHVWRI For No. 6 bars: d, 12db, and ℓn/16. 1) d = 27.7 in. Controls 2) 12db = 12(1.0 in.) = 12 in. 3) ℓn/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 within the B2 web and two No. 6 bars on either side of the beam web over ig. E3.10, E 36 in. (refer to Fig. Section B) Step 9: Integrity reinforcementt Integrity reinforcement rc t 9.7.7.2 H RQGLWLRQVPXVWEE VDWLV (LWKHURQHRIWKHWZRFRQGLWLRQVPXVWEHVDWLV¿HG but not both. end the he remainin g ttw Extend remaining two No. 6 bars over the full gth oof the beam as hhanger bars for the stirrups. length ,QWKLVH[DPSOHWKHUHLVQRDWLV¿HG At least one-fourth the maximum positive moment reinforcement, but not less than two bars must be continuous. 7KLV FR 7KLVFRQGLWLRQZDVVDWLV¿HGE\H[WHQGLQJWZR No. 6 bottom reinforcement bars into the support. 2 No. 6 > (1/4)6 No. 6 /RQJLWXGLQDOUHLQIRUFHPHQWPXVWEHHQFORVHGE\ closed stirrups along the clear span of the beam. 7KLVFRGLWLRQLVVDWLV¿HGE\H[WHQGLQJVWLUUXSVRYHU 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 &DOFXODWHWKHGHÀHFWLRQOLPLW ,PPHGLDWHGHÀHFWLRQVGXHWRVHUYLFHORDGVXQGHU service loads GHÀHFWLRQVDUHSURSRUWLRQDOWRRQO\ORDGLQJVDQGWKHPHWKRGVRUSURYLGHUHJLRQHQFRQFUHWH cracking and reinforcement for calculating stiffness. Modulus of elasticity: 19.2.2.1 Ec = 57, 000 f c′ psi 24.2.3.4 7KHEHDPLVVXEMHFWHGWRDGLVWULEXWHGIRURUUHGLVWULEXWHG 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 dead 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 loa load. XDWLRQ 7KHGHÀHFWLRQHTXDWLRQIRUGLVWULEXWHGORDGZLWK GV ¿[LW\DWERWKHQGV ¨ ℓ4/384EI ed load at mi For concentrated midspan: ¨ 3ℓ3/192EI 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 ⎞ I e = ⎜ cr ⎟ I g + ⎢1 − ⎜ cr ⎟ ⎥ I cr ⎝ Ma ⎠ ⎢⎣ ⎝ M a ⎠ ⎥⎦ 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 12 12 where M cr = fr I g yt (24.2.3.5b) M cr = 7.5(5000 psi)(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%í Reinforced Concrete Design Handbook 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 Asg 2.64 in.2 0.88 in.2 2.64 in.2 As 2 2 0.88 in.2 0.88 in. 2.64 in. c 3.67 in. 6.5 in. 3.67 in. Icr 4016 in.4 10,314 in.4 4016 in.4 Dead: 38 ft-kip 19 ft-kip 38 ft-kip Live: 10 ft-kip 5 ft-kip 10 ft-kip Dead: 123 ft-kip 101 ft-kip 123 ft-kip Live: 75 ft-kip 64 ft-kip 75 ft-kip Total Ma 246 ft-kip -ki 189 ft-kip -kip 246 ft-kip Ie 9183 in. n.4 15,410 in. n.4 9183 in.4 4 4 Distributed load Ma,dist Concentrated load Ma,conc Use 9200 in. 15,400 in. 200 in.4 where Ma,dist [wi kip/ft)(28 ft)2[DWVXSSRUWDQDWPLGVSDQGMa,conc ùPLù for support (b) Negative moment reinforcement shall be spliced at or near midspan Splice length = (1.3)(development length) ℓdc = 1.3(27 in.) = 35 in.; say, 3 ft 0 in. Refer to Fig. E 4.17 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 supports Five No. 6 bottom 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. E4.18—Sections. American Concrete Institute – Copyrighted © Material – www.concrete.org 215 Beam Example 5: Continuous transfer girder 'HVLJQDQGGHWDLODQLQWHULRUFRQWLQXRXVIRXUEDVHDPEXLOWLQWHJUDOO\ZLWKDLQVDDE7KHVSDQHQHZHHQ&ROXPQ/LQHV %DQG'LVDWUDQVIHUJLUGHUVXSSRUWLQJ¿YHVWRULHVDERYH 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— fcę 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 Institute – Copyrighted © Material – www.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 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 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 DOFXODWHGGHÀHF7DEOHFDQQRWEHXVHGDQGGHÀHFWLRQPXVW WLRQVPXVWDWLVI\WKHGHÀHFWLRQOLPLWVLQ de of the girder beam are Beams on either side RUP VXEMHFWWRWKRXQLIRUPORG7KHUHIRUHWKHGHSWK 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 expoVXUHFODVVHVàSHWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGDWGDW\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 18. B ause the beams 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. 7KHIIHFWLYHÀDQJHZLGWKRQHDFKVLGHRIWKH transfer girder is obtained from Table 6.3.2.1. 7UDQVIHUJLUGHUÀDQJHZLGWK 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 Flange width: bf = Ɛn/8 + bw + Ɛn/8 %HDPVÀDQJHZLGWK Each side ⎫ 8hslab ⎪ of web is ⎬ S w / 2 the least of ⎪⎭ A n /8 Flange width: bf = Ɛn/8 + bw + Ɛn/8 bf = 39 in. + 24 in. + 39 in. = 102 in. (8)(7 in.) = 56 in. (does not control) ((14 ft)(12) – 24 in.) /8 = 18 in. Controls bf = 18 in. + 24 in. + 18 in. = 60 in. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 217 Beams 6WHSÀ/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 ZHLJKWRIFHLOLQJVPDUWLWLRQV+9$&DQG+9$&V\VWHPV add 15 psf as miscellaneous dead load. Dead load: 7UDQVIHUJLUGHUVHOIZLJKWZLWKRXWÀDQJHV Concentrated load on girder from column above (Fig. E5.2): Wgir = [(24 in.)(48 in.)](0.150 kip/ft3)/144 = 1.2 kip/ft Slab weight per level: PVƐ= (7in./12)(14 ft)(36 ft)(0.15 kip/ft3) = 44.1 kip Column weight per level: 7 in.⎞ ⎛ 3 Pcol = (2 ft)(2 ft) ⎜12 ft − ⎟ (0.15 kip/ft) = 6.85 kip ⎝ 12 ⎠ Typical beam weight framing into girder at midspan less slab thickness (refer to plan): ⎛ 18 in.⎞ ⎛ 30 in. − 7 in.⎞ 3 PBM = ⎜ ⎟ (34 ft)(0.15 kip/ft) ⎝ 12 ⎠⎜⎝ 12 = 14. 14.7 kip eight of ceilings, partitions, To account for the weight +9$& DQGPHFKDQLFDO+9$& V\VWHPVDGGSVIDV ead load. miscellaneous dead d on ggirder: Total dead load aapplied PSSDLL = ((14 ft)(36 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)(6) + (6.85 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 an average, as the actual layout is not provided. A 7 in. slab weighs 88 psf service dead load. ws = [(24 in.)(30 in.) + (60 in. – 24 in.)(7 in.)] × [(0 [(0.150 kip/ft3) / 144] = 1.03 kip/ft Roof live load 35 psf: Total live load—per ASCE 7 live load with exception of roof load is permitted to be reduced by: PRoof = (14 ft)(36 ft)(0.035 kip/ft2) = 18 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 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 per level: ⎛ ⎞ 15 L = (0.065 kip/ft 2) ⎜ 0.25 + ⎟ = 0.038 ksf ⎝ 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 ⎝ 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)(0.38 kip/ft2) = 19.2 kip Concentrated live load at third level: PL.3rd = (14 ft)(36 ft)(0.047 kip/ft2) = 23.7 kip ™PL = 4PL,col + PL,3rd + PRoof 5.3.1 Concentrated live load on girder: ™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(1.2 kip/ft) = 1.68 kip/ft Controls Concentrated: Pu = 1.4(433 kip) = 607 kip U = 1.4D The superimposed dead load is calculated above FHQWUDWHGORDG/LYHORDG DQGLVLQFOXGHGLQWKHFRQFHQWUDWHGORDG/LYHORDG LGWKRI LVDSSOLHGRYHUWKHZLGWKRIWKHJLUGHUIW䡊LQ Distributed: U = 1.2D + 1.6L ribu wu = 1.2(1.2 1.2(1. 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 = 1.4(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%'LVXEMHFWHGWRDUHODWLYHO\KLJKFRQFHQWUDWHGORDGWKDWPLVGDQGQGURHOWLYHO\VPDOOGLVWULEXWHGHLJKWFDSVHOI weight. Therefore, the appearance of a straight line 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 will have 1.5 in. cover: Assume that the transfer girder requires two rows of reinforcement with onee bar spacing between the an one layer of bars at two rows within the span and ms require re the supports. Beams one row only d = h – cover – dttie – 3db/2 positive momentt negative moment positive moment negative moment nsfe girder: Transfer d = 48 in. in – 1.5 in. – 0.62 0.625 in. – 3 (1.41 in.)/2 = 43.76 in. n use d = 43.7 in. d = 48 in. – 1.5 in. – 0.625 in. – 1.128 in./2 = 45.3 in. use d = 45 in. Beams: d = 30 in. – 1.5 in. – 0.5 in. – (0.75 in.)/2 = 27.625 in. use d = 27.6 in. 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: Ɛcu = 0.003 in./in. 22.2.2.2 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQÀH[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. 22.2.2.3 Determine the equivalent concrete compressive stress at nominal strength: 22.2.2.4.1 22.2.2.4.3 22.2.1.1 221 Beams CHAPTER 7—BEAMS 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 0.85fcęZLWKDGHSWKRIa ȕ1cZKHUHȕ1 ive strength and is a function of concrete compressive .2.4.3. is obtained from Table 22.2.2.4.3. For fcę SVL β = 0.85 – ent cconcrete compressive depth a Find the equivalent sion force to th ension by equating the co compression the tension force within thee bbeam cross section (Fi (Fig. E5.4) E5.4): C=T 0.85 fcęba = Asfy 0. 5(50 psi)(b)(a) (b)(a) = As(60,000 psi) 0.85(5000 Transfer girder: For positive moment: b = bf =102 in. a = As (60 (60,000 psi) (60,00 = 0.138 As 0.85(5000 0.85(50 psi)(102 in.) Beams: For positive moment: b = bf = 60 in. a = As (60,000 psi) = 0.235 As 0.85(5000 psi)(60 in.) Transfer girder and beams: For negative moment: b = bw = 24 in. a = As (60,000 psi) = 0.588 As 0.85(5000 psi)(24 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[LPXPÀH[XUDOPRPHQWVVKRZQLQWKHDERYH moment diagram (Step 4). 9.5.1.1 The beams' design strength must be at least the required strength at each section along their lengths: ‫׋‬Mn•Mu ‫׋‬Vn•Vu Calculate the required reinforcement area: a⎞ ⎛ φM n ≥ M u = φAs f y ⎜ d − ⎟ ⎝ 2 ⎠RUWRSQHJDWLYHUHLQIRUFHPHQWXVH2QHODHURI1R 9 bar; db = 1.128 in., As = 1.0 in.2, and d = 45 in. 9.6.1.1 9.6.1.2 For bottom positive reinforcement use: Two layers of No. 11 bar; db = 1.41 in., As = 1.56 in.2, and d = 43.7 in. Minimum reinforcement The provided reinforcement must st be at least the minimum required reinforcement rcement at every section along the length of the beam. he beam Use = 45 in. n. > = 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 í 1782 Max. M+ 3348 í 1803 9.5 Max. M Max. M Number of bars er rrequired ed minimum rei orcement will yield higher reinforcement area: As = 3 f c′ fy bw d Because fcę!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 reinfo Provided 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 to be placed near the vertical faces of the tension zone over a distance of: 48 in./2 = 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 = 15 ⎜ ⎟ – 2.5(2.125 in.) = 9.7 in. ⎝ 40, 000 ⎠ ⎛ 40, 000 ⎞ s = 12 ⎜ ⎟ ⎝ f s ⎠ ⎛ 40, 000 ⎞ s = 12 ⎜ ⎟ = 12 in. ⎝ 40, 000 ⎠ where cc is the clear cover from the skin reinforcement to the side face. cc = 1.5 in. + 0.625 in. = 2.125 in. 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 ⎠ ⎟ φM n = (0.9)(60 ksi)(2)(0.79 in.2) ⎜⎝ 2.58 in.⎞ ⎟ ⎜ ⎛ 24 in. + − ⎜⎜ ⎟⎟ ⎝ 2 ⎠ ⎠ ‫׋‬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 tension reinforcement at girder midspan: Calculated design moment 3348 ft-kip (Step 4). Moment to be resisted by No. 11 bars: ‫׋‬Mn = (3348 ft-kip) – (390 ft-kip) = 2958 ft-kip Required reinforcement area by solving the following equation: a⎞ ⎛ φM n ≥ M u = φAs f y ⎜ d − ⎟ ⎝ 2 ⎠ As,No.11 = 15.42 in.2 ired No. 11 ba Required barss = 15.42 in.2/1.56 in.2 = 9.9 oose 100 No. N 11 bbars. bar Choose Negative reinforcement rcem att the supports pp aye of skin reinforcem nt on both Provide three layers 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: Re-evaluating the positive tension reinforcement at girder midspan: 5.29 in. ⎞ ⎛ φM n = (0.9)(60 ksi)(2)(0.79 ksi)(2)(0 in.2) ⎜ 24 in. − ⎟ ⎝ 2 ⎠ 5.29 in. ⎞ ⎛ (60 ksi)(2)(0.44 ksi)(2) + (0.9)(60 in.2) ⎜ 31.25 in. − ⎟ ⎝ 2 ⎠ 5.29 in. ⎞ ⎛ + (0.9 (0.9)(60 ksi)(2)(0.44 in.2) ⎜ 38.5 in. − ⎟ ⎝ 2 ⎠ ‫׋‬Mn = 4885.1 in.-kip = 407.1 ft-kip, say, 407 ft-kip Calculated design moment 1803 ft-kip (Step 4). Moment to be resisted by No. 9 bars: ‫׋‬Mn = (1803 ft-kip) – (407 ft-kip) = 1396 ft-kip Required reinforcement area by solving the following equation: As,No.11 = 7.42 in.2 a⎞ ⎛ φM n ≥ M u = φAs 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 β1 ZKHUHȕ1 = 0.8 εs = Transfer girder: 0.003 (d − c) c Mu, ft-kip As,prov, in.2 a, in. c, in. Ɛs, in./in. Ɛs > 0.005? 1782 8 4.7 5.9 0.020 Y + 3348 15.6 2.15 2.7 0.0466 Y Mí 1803 8 4.7 5.9 0.020 Y Mí M Note that 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): 7KHUHIRUHWKHDVVXPSWLRQRIXVLQJ‫׋‬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 distrib over beam depth. εs = 0.003 0. (24 4 in. − 2.7 in.) = 0.024 > 0.005 2.7 in. OK By inspection tthe other skin reinforcement layer has s higher strain; therefore, reinforcement in both skin reinforcement layers is yielding and the assumption of using 0.9 = ‫׋‬s correct. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 7—BEAMS 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 have higher strain; therefore, reinforcement in all 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&RO Q/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. 2 6SDQ&ROXPQ/LQH$% Q&R PQ/LQH %$í M (N (No. 9) Number of bars Mu, fft-kip ip As,req'd in.2 Req'd Select 906 8.0 8.0 8 1.8 4.1 5 No. 6 bar; db = 3/4 4 iin., As = 0.44 in. , an and d = 27.6 in. M (N (No. 6) 222 28 in., As = 11.0 in.2, and d = 27 27.4 No. 9 bar; db = 1.128 in. for one layer. 6SDQ&ROXPQ/LQH'(ROXPQ/LQ + Number of bars M u, 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. 6) 58 0.43 0.97 2 M+ (No. 6) 133 1.08 2.4 3 American Concrete Institute – Copyrighted © Material – www.concrete.org 226 9.6.1.1 9.6.1.2 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Minimum reinforcement The provided reinforcement must be at least the minimum required reinforcement at every section along the length of the beam. As , min = 3 fc ′ bw d fy Because fcę!SVL(TD RQO\DSSOLHV Check to see if the required reinforcement areas 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 DGMDFHQWVSDQVDWRUHVLVWWKHQHJDWLYHPRPHQWDQG6SDQ 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 diagra diagram. εs = 0. 0.003 (20.5 20 5 iin. − 5.9 in.) = 0.007 > 0.005 5.9 in. n. Therefore, herefore, rei reinforcement in the two No. 6 skin bars is yieldin yielding and the assumption made of using 0.9 = ‫׋‬s 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 f y a and c = a= β1 0.85 f c′b ZKHUHȕ1 = 0.8 εs = Beams (only maximum moments are checked) M u, ft-kip As,prov, in.2 a, in. c, in. Ɛs , in./in. Ɛs > 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 and 102 in. and 60 in. for girder and beam positive 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 9.5.3.1 22.5.1.1 22.5.5.1 ‫׋‬Vn•Vu Vn = Vc + Vs = (379.5 kip) – (1.68 kip/ft)(45 in./12) = 373.2 kip Vc = 2 f c′bw d Vc = 21.2.1(b) Shear strength reduction factor: ‫׋‬shear = 0.75 φVc = φ2 f c ′bw d (0.75)(152.7 kip) = 114.6 kip ‫׋‬Vc = (0.75)(152 &KHFNLI‫׋‬Vc•Vu ‫׋‬Vc = 1114.6 kip < @d = 373.7 kip 9.5.1.1b (2) ( ) 5000 psi (24 in.)(45 in.) 1000 lb/kip = 152.7 kip NG T refo shear ar reinforc Therefore, reinforcement is required. 22.5.1.2 tin shear ar reinforcemen Prior to calculating reinforcement, che check if na dimensions ensions satisfy E the cross-sectional Eq. (22.5.1.2): Vu ≤ φ(Vc + 8 f c′bw d) ⎛ 8 5000 psi(24 in.)(45 in.) ⎞ 152 522 7 ki kkip + Vu ≤ φ ⎜152.7 ⎟ 1000 lb/kip ⎝ ⎠ ≤ 576.6 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 Transverse reinforcement satisfying equation 22.5.10.1 is required at each section where Vu!‫׋‬Vc Vs ≥ Vu − Vc φ Vs ≥ 373.2 kip − 152.7 kip = 344.9 kip 0.75 Try a No. 5 bar, two legged stirrup 22.5.10.5.3 22.5.10.5.6 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 stirrup 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 ≤ 4 f c′bw d ? 9.7.6.2.2 9.6.3.3 Vs = 344.9 .9 kipp > 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 ‫ډ‬/ in the lesser of d/4 in. < 12 in. RUF HVQRWYDUU\VLJQ FDQWO\ %HFDXVHVKHDUIRUFHGRHVQRWYDU\VLJQL¿FDQWO\ nsfer girder (F over the length off th the transfer (Fig. E5.3a), use two No. 5 stirrups at 8 in. spacing over the full length of the girder. 6SHFL¿HGVKHDUUHLQIRUFHPHQWPXVWEHDWOHDVWDV 0.75 f c′ bw b and 50 w f yt f yt Use s = 8 in. = dd/4 = 11 in. < 12 in. ∴ OK Av , min s ≥ 0.75 5000 psi 18 in. = 0.016 in.2 /in. 60,000 psi Provided: Because fcę!SVL(T FRQWURV 8 in. spacing: Av , min s = 4(0.31 in.2) = 0.155 in.2 /in. 8 in. 6SDFLQJVDWLV¿HVWKHUHTXLUHPHQWV 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 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. E5.10). ‫׋‬shear = 0.75 Beams 21.2.1(b) Shear strength Shear strength reduction factor: Fig. E5.10—Shear at the critical section = Vu – (1.85 kip/ft)(27.4 in./12) = Vu – 4.2 kip 22.5.5.1 Vc = 2 f c′bw d Vc = (2) ( ) 5000 psi (24 in.)(27.4 in.) = 93 kip ‫׋‬Vc = (0.75)(93 kip) = 69.7 kip 9.6.3.1 &KHFNLI‫׋‬Vc•Vu , 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 66.7 Y Rig Right 88.9 N /HIW 95.9 Y Right 73.8 Y /HIW 0 Y ht Right 21.7 Y uire shear reinfo emen Beam EF does 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 , kip Is Vu!‫׋‬Vc ? 66.7 Y /HIW Right 88.9 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 ≥ Vu φ Vs ≥ − 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. &KHFNLI¿YH1REDUVUHVLVWLQJSRVLWLYH 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,req'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¿YH1REDUVFDQEHSODFHGLQRQHODHU 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 7HQVLRQUHLQIRUFHPHQWLQÀDQJHVPXVWEHGLVWULEXWHG ZLWKLQWKHHIIHFWLYHÀDQJHZLGWKbf = 102 in. (Step 2), but not wider than: Ɛn/10. Girder: Ɛn/10 = (26 ft)(12)/10 = 31.2 in. < 102 in. Beams 24.3.4 231 %HFDXVHWKHHIIHFWLYHÀDQJHZLGWKH[FHHGVƐn/10, addi- Beam: tional bonded reinforcement is required in the outer Ɛn/10 = (12 ft)(12)/10 = 14.4 in. < 60 in. SRUWLRQRIWKHÀDQJH 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 ÀDQJHZLGWKDQGWRSURWHFWWKHÀDQJHLIUHLQIRUFHPHQW is concentrated within the web width. Bar spacing = Span AB BD 24 in. − [2(3.125 in.) + 5(1.128 in.)] 5 DE EF = 2.4 in. Prov. No. 9 No. 9 in web No.10 in Ɛn/10* No. 6 in outer portion* / 4 4 — 4 R 8 6 2 4 / 8 6 2 6 R 8 6 2 6 / 8 6 2 4 R 4 4 — 4 / † 5 † 5 — 4 R 5† 5† — 4 * Bars to be divided equally on both sides of the web (refer to Fig. E5.12-sections) † Section reinforced with No. 6 bars Sec)LJ(²/RQJLWXGLQDOEDUVDLVWULEXWHGZLWKÀDQJH Note: Slab shrinkage and temperature reinforcement can be used as the additional bonded reinforcement in WKHRXWHUSRUWLRQRIÀDQJHWRVDWLVI\$&,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 yψt ψe ⎛ (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 ZKHUHȥtLVWKHFDVWSRVLWLRQȥt = 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]RQWDOEDUVDQGȥt = 1.0, if not more than 12 in. No. 6 No. 9 of fresh concrete is placed below bottom horizontal Ɛd, in. 41.3 62 Top bars. 42 63 Use Ɛ , in. 20λ f c′ db Bottom No. 11 55 — 57 — Ɛd, in. 31.8 — — 59.8 Use Ɛd, in. 36 — — 60 d ȥeLVWKHFRDWLQJIDFWRUDQGȥe = 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 &DOFXODWHWKHLQÀHFWLRQSRLQWIURPSRVLWLYHPRPHQW (Fig. E5.13(b)): Maximum moment at midspan: 3348 ft-kip E LQÀHFWLRQSRLQWRISRVLWLYHPRPHQW 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 PLGVSDQ)URPPHTXLOLEULXPWKHSRLQWRILQÀHFWLRQ m th lowing free bod is obtained from the following body diagram: Mmaxíwu(x)2/2 – P/2x = 0 ong g transfer sfer span Top bar length along /HIWVXSSRUW &DOFXODWHWKHLQÀHFWLRQSRLQWIURQHJDWLYHPRPHQW diagram (Fig. E5.13(c)): íMmaxíwu(x)2/2 + Vux = 0 F ,Q F ,QÀHFWLRQSRLQWDW6XSSRUW% íIWNLS ±NLSIW x)2/2 + (378 kip)x = 0 x = 4.76 ft, say, 4 ft 10 in. Right support: &DOFXODWHWKHLQÀHFWLRQSRLQWIURWKHQHJDWLYHPRPHQW diagram (Fig. E5.13(d)): íMmax íwu(x)2/2 + Vux = 0 G ,QÀHFWLRQSRLQWDW6XSSRUW' ‫׋‬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 &KHFNWKHLQÀHFWLRQSRLQWIRU6SDQ$%'(DQG() Span AB: &DOFXODWHWKHLQÀHFWLRQSRLQWDW6XSSRUW%)LJ E5.14(a)): íMmax íwu(x)2/2 + Vux = 0 QSRLQ &DOFXODWHWKHLQÀHFWLRQSRLQWDW6XSSRUW$)LJ E5.14(b)): 2 + Vux = 0 íMmax íwu(x)2/2 Beams The moment diagram envelop shows that all three VSDQV$%'(DQG() GRQRWKDYHDGH¿QHG maximum moment at midspan. The moment varies from maximum negative moment at one support and increases to maximum positive moment at the other support. D ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW% íIWNLS ±NLSIW x)2/2 + (93.1 kip)x = 0 x = 10.9 ft, say, 11 ft 0 in. E ,QÀHFWLRQSRLQWORFDWLRQDW6XSSRUW$,QÀH QSRLQWORF t (222 ft ft-kip) – 1.85 kip/f kip/ft(x)2/2 – (70.9 kip)x = 0 x = 3.0 ft (DP$%LQÀ)LJ(²%HDP$%LQÀHFWLRQSRLQWORFDWLRQV me concept cept for spans tthreee and Following the same IRXUWKHLQÀHFWLRQSRLQWVDUHFDOFXODWHGDW Span x-left l f x-right DE 10.3 ft, ssay, 10 ft 6 in. 3.6 ft, say, 3 ft 9 in. EF 1 ft, say, 12 ft 3 in. 12.2 2.44 ft, 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 along the span where bent or terminated tension bars are no longer required to UHVLVWÀH[XUH Eight No. 9 bars and four No. 6 and two No. 8 skin bars are required to resist the factored moment at &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) 2 ⎛ 4(1.0 in.2)(60 ksi) ⎞ (0.9)(60 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[WHQGHGLQWRWKHDGMDFHQWVSDQVDQVDWWKHVXSSRUWV For No. 9 bars: 1) d = 45 in. Controls 2) 12d db = 12(1 12(1.128 in.) = 13.5 in. refo extend xtend 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 iin. > Ɛd = 63 in. 33 in. 78.3 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\RQGWKHLQÀHFWLRQSRLQWWKHJUHDWHVWRId, 12db, and Ɛn/16. For No. 9 bars: 1) d = 45 in. Controls 2) 12db = 12(1.128 in.) = 13.5 in. 3) Ɛn/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\RQGWKHLQÀHFtion 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) 2 ⎛ 4(1.56 in.2)(60 ksi) ⎞ × (0.9)(60 ksi) ⎜ 43.7 in. − 2(0.85)(5 ksi)(102 in.) ⎟⎠ ⎝ (3348 ft-kip) − (1.68 kip/ft) x = 5.92 ft, say, 6 ft 0 in. from midspan 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 Ɛd = 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 omm nded to ext end d bars to the far face of the recommended extend umn and develop 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) 3RLQWRILQÀHFWLRQRFFXUV\$W IW $WKHKSLQWRILQÀHFWLRQ positive moment tension bars must be limited such that Ɛd for that bar from the column face. VL]HVDWLV¿HV 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) ⎞ M n = 4 1.56 in.2 (60 ksi) ⎜ 43.7 in. − (2)(0.85)(5 ksi)(24 in.) ⎟⎠ ⎝ ( ) Mn = 15,674 in.-kip Ad ≤ Mn + Aa Vu Ad ≤ 15,674 in.-kip + 43.7 in. = 85.7 in. 373.2 kip This length exceeds Ɛd = 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\RQGWKHLQÀHFWLRQSRLQWOLPLWHGWRWKHJUHDWHVWRI d and 12db. 9.7.3.5 ,IEDUVDUHWHUPLQDWHGLQDWHQVLRQ]RQHÀH[XUDOWHQVLRQ then a bar stress discontinuity occurs. Therefore, LOHEDUVPXVWQRW WKHFRGHUHTXLUHVWKDWÀH[XUDOWHQVLRQVLOHEDUVPXVWQRWVQRWVQRWE 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 SRLQWDQGGQGE Vu " ‫׋‬V ‫׋‬V Vn . IRUÀH[XUHDWWKHFXWRIISRLQWDQGGE V oop area in excess of that required equired 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 oop area shall bbe att least point. Excess stirrup 60bws/fyt. Spacing s shall not exceed d/(8ȕb).

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 φVc = φ2 f c′bw d d = dp = 0.8h = 0.8(48 in.) = 38.4 in. φVc = 0 0.75(2)( )(5000 ppsi)(24 in.)(38.4 in.) = 97.7 kip 9.5.1.1 Check if []Vc•Vu Vc = 97. 97.7 kip < /2 = 373 kip []V 22.5.1.2 Before calculating shear reinforcement, check if the cross-sectional dimensions satisfy Eq. (22.5.1.2): NG Therefore, shear reinforcement is required. Vu ≤ φ(Vc + 8 f c′bw d) ⎛ 8(5000 psi)(24 in.)(38.4 in.) ⎞ Vu ≤ φ ⎜ 97.7 kip + ⎟ ⎝ 1000 lb/kip ⎠ = 489 kip Vu = 373 kip ≤ φ(Vc + 8 f c′bw d) = 489 kip OK, therefore, section dimensions are satisfactory. American Concrete Institute – Copyrighted © Material – 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) (270 ksi) + (5.08 in.2)(60 ksi)) = 684 kip 910 kip > 684 kip 22.5.8.2 Therefore, Vc is the least of the following three equations and not 22.5.8.3: Vu d p ⎞ ⎛ (a) ⎜ 0.6λ f c′ + 700 bw d M u ⎟⎠ ⎝ ( ) (b) 0.6λ f c′ + 700 bw d (c) 5λ f c′bw d 22.5.5.1 and Vw ≥ 2λ f c′bw d Vw ≥ 2(1.0) 5000 psi(24 in.)(38.4 in.) = 130.3kip For the calculation of the three equations, refer to the table below: x, ft Vu, kip Mu, ft-kip dP, in. Vudp/M /Mu Vc (a), kip Vc (b), kip Vc (c), kip 1 376.5 –631 6 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 5 369.9 3 861 38.4 1.374 1001.2 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 0.595 436.5 684 326 9 363.3 2328 38.4 0.499 9 3370.9 684 326 10 361.6 2690 38.4 0.430 323.8 684 326 11 360.0 3051 38.4 0.378 288.2 684 326 12 358.3 3410 39.6 0.347 276.3 706 336 13 356.7 3767 41.8 0.330 278.7 745 355 14 355.0 4123 44.0 0.316 282.1 784 373 Equation (22.5.8.2a) controls the middle 8 ft of the girder; the rest of the span is 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 3 373 244 129 5.37 5 4 372 244 127 5.44 5 5 370 244 125 5.51 5 6 368 244 124 5.58 5 7 367 244 122 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 = Vui[]Vc, kip φAv f y d p Vu – φVc , in. sprov'd. , in. Shear reinforcement ent forc satisfying Eq. (22.5.10.1) 2.5.10.1) Transverse reinforcement ach section on where Vu > []V [] c is required at each Vs ≥ Vu – Vc φ 22.5.10.5.3 22.5.10.5.5 where 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 ≤ 4 f c′bw d ? 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 3ODFH¿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¿HGVKHDUUHLQIRUFHPHQWPXVWWEHWKHOHVVWRI the greater of (c) and (d) and (e) (c) 0.75 f c′ (d) 50 (e) bw , and f ytt 80 f yt d s Av ,min bw f yt Aps f pu Av ,min s d bw ≥ 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. spacing: Av ,min s = 2(0.2 in.2) = 0.08 in.2 /in. 5 in. VSDFLQJVDWLVꞕHV∴ 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 +stirrup +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., say, 1.5 in. bw,req'd = 2(1.5 in. + 0.5 in. + 1.0 in.) + 3.81 in. + 4.5 in. = 14.3 in. < 24 in. OK Therefore, four No.10 bars can be placed in one layer in the 24 in. transfer girder web. 24.3.4 7HQVLRQUHLQIRUFHPHQWLQÀDQJHVPXVWEHGLVWULEXWHGZLWKLQWKHHIIHFWLYHÀDQJHZLGWKbf = 136 in. (Step 2), but not wider than: ℓn/10.. ZLGWKH[FHHGV n/10, addiddi %HFDXVHWKHIIHFWLYHÀDQJHZLGWKH[FHHGVℓ cement is required in the outer tional bonded reinforcement JH SRUWLRQRIWKHÀDQJH d at slab middepth 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 WHFWÀDQJHLIUHLQIRUFHPHQW ÀDQJHZLGWKDQGWRSURWHFWÀDQJHLIUHLQIRUFHPHQW 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.) + 3db + 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 ⎜ ⎟ – 2.5(2 in.) = 10 in. ⎝ 40,000 psi ⎠ Controls and s = 12(40,000/fs) 9.7.2.3 24.3.2 7KLVOLPLWLVLQWHQGHGWRROLPLWÀH[XUDOFUDFNLQJ. Note that cc is the cover to the longitudinal bars, not to the tie. Skin reinforcement The transfer girder is 48 in. deep > 36 in. Although the Code does not require skin reinforcement for Class T prestressed beams, many engineers provide it. Skin reinforcement is placed a distance h/2 from the tension face. ⎛ 40, 000 psi ⎞ s = 12 ⎜ ⎟ = 12 in. ⎝ 40, 000 psi ⎠ /RQJLWXGLQDOEDUVSDFLQJVDWLVꞕHVPD[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. Bars, however, must be developed within the support at each end. 7KHVLPSOL¿HGPHWKRGLVXVHGWRFDOFXODWHWKH development length of No. 10 bars: 25.4.2.2 Ad = 25.4.2.4 ZKHUHψt = 1.0 top bar location with not more than 12 in. of fresh concrete below horizontal reinforcePHQW2WKHUZLVHVH)RUWRSEDUψt = 1.3; ℓd = (1.3)(42.43db) No. 8 bars: ℓd = 42.43(1.0 in.) = 42.43 in., say, 48 in. ψt = 1.0; bars are uncoated because No. 10 bars are top bars, development length is: 1.3ℓd = (1.3)(42.43 in.)(1.27 in.) = 70 in., say, 6 ft 0 in. 9.7.7.5 f yψt ψe 20λ f c′ db ⎛ (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, 7KLVFRQGLWLRQZDVVDWLV¿HGDEYHE\H[WHQGLQJWKHWRSDQGERWWRPEDUVWKHIXOOOHQJWKRIWKHEHDPDQG HOLEHUDWLQJ RUE RIPXVWEHVDWLV¿HG,QWKLV four No. 10 bars into the support. OK H[DPSOHFRQWLQXRXV LVVDWLV¿HGE\KDYLQJWKHDWOHDVW 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 tested in an unbonded 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%HPDEUGHWDLOV1RWH3ODFH¿UVWWVWLUUXSDWLQIURPWKHIDFHRIVXSSRUW 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 Institute – Copyrighted © Material – www.concrete.org 265 Beam Example 7: Precast 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— fcꞔ SVLQRUPDOZHLJKWFRQFUHWH ꞔ 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 9.2.1.1 The mixture proportion must satisfy the durability requirements of Chapter 19 (ACI 318 318-14) and structural strength requirements. The designer quirem determines the durability rabil classes. Please refer to Chapter 3 of thiss Ha Handbook in-depth discus discusok for an in-d gor and nd classes. sion of the categories IH VSHFL¿FDWLRQWKD LVFR $&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLCI encourages re rencing nated with ACI 31 318. ACI referencing ¿F V LQWRMREVSHFL¿FDWLRQV There are several mixture options within ACI hich the 301, such as admixtures and pozzolans, which designer can require, permit, or review if suggested by the contractor. Step 2: Beam geometry Beam depth 9.3.1.1 Since beam resists concentrated loads, the beam depth limits in Table 9.3.1.1 of ACI 318-14 cannot be used. 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, Chapte DUHVDWLV¿HG VDWLV B ed oon durability bility and strength requirements, and Based ex erien with local l l mi i experience mixtures, the compressive VWU QJWK FUHWHLVS VWUHQJWKRIFRQFUHWHLVVSHFL¿HGWRGD\VWREHDW le leastt 40 4000 psi. Assume 22 in. deep beam with American Concrete Institute – Copyrighted © Material – www.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) + (Pu) 2 M u , 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: ℓn/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) 2 + (15 kip)(4.5 ft) = 254 ft-kip 8 ℓn/180 8 = (18 ft)(12 iin./ft)/180 n /f /ft = 1.2 in. < 3 in. vide 6 in., n., therefore therefor 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: 0.85fcgA1 00.85(4000 psi)(14 in.)(6 in.)/1000 = 285.6 kip 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 American Concrete Institute – Copyrighted © Material – www.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 assuming No. 8 bars and 1.0 in. cover. For precast concrete beam, the minimum cover is the greater of 5/8 in. and db and need not exceed 1.5 in. 2QHURZRIULHLQIRUFHPHQW 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 WKRI 7KHWHQVLOHVWUHQJWKRIFRQFUHWHLQÀH[XUHLVDYDULd is about 10 to 15 per able property and percent off the ess strength. rength. ACI 31 concrete compressive 318 neglects nsi strength ength to calculat 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 com ssive Determine the equivalent compressive stress at nominal strength: 22.2.2.3 22.2.2.4.1 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 comprehensive tests. Rather than tests, the Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fcgZLWKDGHSWKRI a = ȕ1c, where ȕ1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3. For fcg"SVL ȕ1 = 0.85 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 0.85(4000 psi)(b)(a)=As (60,000 psi) For moment at midspan: b = 14 in. a = As (60, 000 psi) = 1.26 As 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 must be at least the required strength at each section along its length: []Mn•Mu []Vn•9u Calculate the required reinforcement area: a⎞ ⎛ φM n ≥ M u = φAs f y ⎜ d – ⎟ ⎝ 2⎠ 1.26 As ⎞ ⎛ (254 ft-kip)(12) = (0.9)(60 ksi)As ⎜ 20 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 Use four No. 8 As, prov = 3.16 in.2 > As, req'd = 3.1 in.2 OK Per Reinforced Concrete Design Handbook Design Aid – Analysis Tables, which can be downloaded from: . aspx?ItemID=SP1714DA, four No. 8 bars require a minimum 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 f y 0.85 f c′b and c = a β1 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 ZKHUHȕ1 = 0.85 εt = ε cu (d – c) c εt = 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 The provided reinforcement must be at least the minimum required reinforcement at every section along the length of the beam. As , min = 3 f c′ As , min = fy bw d 200 bw d fy Beams 9.6.1.1 9.6.1.2 269 As , min = 3 4000 psi (14 in.)(20 in.) = 0.89 in.2 60, 000 psi As , min = 200 (14 in.)(20 in.) = 0.93 in.2 Controls 60,000 psi As,prov = 3.16 in.2 > As,min = 0.93 in.2 OK Required positive moment reinforcement areas exceed the minimum required 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 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 DUIURFHLVWDNHQ WGLVWDQFH VDWLV¿HGWKHGHVLJQVKHDUIRUFHLVWDNHQDWGLVWDQFH d from the face of th the supportt (F (Fig. E7.4). []shearr = 0.7 0.75 Fig. E7.4—S E7.4—Shear at the critical 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 is required. Determine required Vu on each side of Pu /HIWRIPu: VXℓ = Vu – wux1 Right of Pu: Vu,r = Vu – wux1 – Pu Prior to calculating shear reinforcement, check if the cross-sectional dimensions satisfy Eq. (22.5.1.2): Vu ≤ φ(Vc + 8 f c ′bw d) VXℓ = 56.4 kip – (4.6 kip/ft)(4.5 ft) = 35.7 kip Vu,r = 56.4 kip – (4.6 kip/ft)(4.5 ft) –15 kip = 20.7 kip Vu ≤ φ(35.4 kip + 8(4000 psi)(14 in.)(20 in.)) = 132.8 kip OK Section dimensions are satisfactory. American Concrete Institute – Copyrighted © Material – www.concrete.org 270 THE REINFORCED CONCRETE 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•Vu±[]Vc where φVs = []Vs•.7 kip) – (26.6 kip) = 22.1 kip φAv 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 s = 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 ≤ 4 f c′bw d ? 4 f c′bw d = 4(4000 psi)(14 in.)(20 in.) = 71 kip 22.1 kip = 29.5 kip 0.75 Vs = 29.5 kip < 4 f c′bw d = 71 kip OK Because the required shear strength is below the rup spacing is the 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 φVn = 2 (0.75)(2)(0.11 in.2)(60 ksi)(20 in.) (0.75) 10 in. []Vn = 46.4 kip Determine distance x from face of support to point at which x= Vu = 46.4 kip 56.4 kip – 46.4 kip = 2.2 ft 4.6 kip/ft Determine distance ℓv1, beyond x1 at which stirrups are not required. 20.7 kip – 26.6 kip / 2 = 1.6 ft, say, 2 ft 4.6 kip/ft Find ℓv1 = ( – []Vc/2)/wu A v1 = Compute x1 + ℓv1 x1 + ℓv1 = 4.5 ft + 2 ft = 6.5 ft = 78 in. American Concrete Institute – Copyrighted © Material – www.concrete.org Conclude: use s = 7 in. until []Vu < 44.5 kip and use s = 10 in. until []Vu < 0.5[]Vc 271 From face of support use 3 in. space then Five spaces at 7 in. on center (35 in.) and Five spaces at 10 in. on center (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 ,PPHGLDWHGHÀHFWLRQLVFDOFXODWHGXVLQJHODVWLF 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 4000 psi = 3600 ksi The beam resists a factored distributed force of 4.6 kip/ft or service dead load of 2.23 kip/ft and 1.2 kip/ft service live load. The beam also resists a concentrated factored load of 15 kip or service dead load of 7.2 kip and 3.9 kip service live load. 7KHGHÀHFWLRQHTXDWLRQVIRUDVLPSO\VXSSRUWHGEHDPZLWK free rotation at both ends: Δ= 5wA 4 384 EI For concentrated load at thirdspan: Δ= PA 3 28.23EI Note: erv loads. oads. · w and P are service ale moment ment of 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 ⎝ Ma ⎠ ⎦ ⎣ ⎝ M a ⎠ ⎦ fr I g (24.2.3.5b) where: M cr = yt 3 24.2.3.5 3 Ig = bh3 (14 in.)(22 in. in.)3 =12,423 in.4 = 12 12 M cr = 7.5(4000 psi)(12,423 in.4) = 535,704 in.-lb 11 in. Mcr = 44.6 ft-kip Ma is the moment due to service load. The 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 Asꞔ LQ2 Ec Solving for c = 6.9 in. Cracking moment of inertia, Icr: I cr = bc 3 + (n – 1) As′ (c – 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 'HÀHFWLRQGXHWRGHDGORDG Δ 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 Δ conc = (7.2 kip + 3.9 kip)(18 ft)3 (12)3 = 0.18 in. 28.3(3600 ksi)(5975 in.4) 7RWDOGHÀHFWLRQ ¨ LQLQ LQ &KHFNWKHFDOFXODWHGGHÀHFWLRQDJDLQVWWKHPD[LPXPDOORZable limits in Table 24.2.2:)RUÀRRUVXSSRUWLQJRUDWWDFKHGWRQRQVWUXFWXUDO elements and not likely to be damaged by large GHÀHFWLRQℓ/240 24.2.4.1.1 ¨all. = (18.0 ft)(12 in./ft)/240 = 0.9 in. ¨all. LQ!¨ LQOK &DOFXODWHORQJWHUPGHÀHFWLRQ λΔ = 24.2.4.1.3 273 ξ 1 + 50ρ′ λΔ = From Table 24.2.4.1.3, the time dependent factor ation of more than 5 years: for sustained load duration ħ 7KHUHIRUHORQJWHUPGHÀHFWLRQLV WHUP ¨T Ʒ¨ ¨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 1RWH7KHORQJWHUPGHÀHFWLRQGXHWRVXVWDLQHG lo ding exceedss ℓ/240. ℓ 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\VXSSRUWHGGEHDPUHVLVWLQJDPRPHQWDQGWRUTXH7KHEHDPLVVXEMHFWHGWRDIDFWRUHGGLVWULEXWHGWRUTXHRI 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: fcꞔ SVLQRUPDOZHLJKWFRQFUHWH ꞔ 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 22.7.6.1.1 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. Step 2: Cracking torsion 22.7.5.1 Calculate cracking ttorsion Tcr. ⎛ Acp2 ⎞ φTcr = 4φλ f c′ ⎜ ⎟ ⎝ pcp ⎠ Calculation Acp = 16 in.(24 24 in.) = 384 3 in.2 Aoh = (16 ((1 in. – 3.5 in.)(24 in.)(2 in. – 3.5 in.) = 256 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. + 24 2 in. – 3.5 in.) = 66 in. ph = 2(16 ⎛ (384 in.2) 2 ⎞ 4(0.75)(1.0)(5000 psi) ⎜ φTcr = 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 = [] 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/ft-kip) = 32.6 ft-kip Threshold torsion = 0.25(32.6 ft–kip) = 8.2 ft-kip Because Tu = 53 ft-kip > 8.2 ft-kip, ties for torsion are therefore required. Calculate fv = Vu/(bwd) Calculate fvt = Tuph/(1.7Aoh2) fv = 61 kip/(16 in. × 21.5 in.) = 0.177 ksi fvt = (53 ft-kip)(12 in./ft)(66 in.)/[1.7(256 in.2)2] = 0.377 ksi Calculate limit = φ(2 f c′ + 8 f c′) /LPLW 0.75(2 + 8)(5000 psi) = 0.53 ksi Is f v2 + f vt2 < limit? (0.177 ksi) 2 + (0.377 ksi) 2 = 0.416 ksi 0.416 ksi < 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 Av (Vu – 2φλ f c′ (bw d) 22.5.1.1 = s φf y d 22.5.5.1 22.5.10.5.3 21.2.1b 275 Av (61 kip – 2(0.75)(1.0)(5000 psi)(16 in.)(21.5 in.) = s (0.75)(60,000 psi)(21.5 in.) Av = 0.0253 in.2 /in. s Required torsional tie area/spacing: 22.7.6.1a 22.7.6.1.2 9.6.4.2 At Tu = s 2φAo f y cot θ Calculate total tie area/spacing (Av/s + 2At/s) At (53 ft-kip)(12 in./ft) = = 0.0324 in.2 /in. s 2(0.75)(218 in.2)(60 ksi)cot45° Av A + 2 t = 0.0253 in.2 /in. + 2(0.0324 in.2 /in.) s 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 minimum transverse reinforcement. Is 0.75 f c′bw f yt ⎛ A 2A ⎞ 3/8 in. OK 8VH1RORQJLWXGLQDOEDUV3ODFH¿YH1RLQ bottom, two No. 5 in each side face, and three No. 5 in top. American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 7—BEAMS 277 Beams Step 5: Beam detailing Notes: %RWWRPUHLQIRUFHPHQWLVIRUFRPELQHGPRPHQWDQGWRUVLRQUHLQIRUFHPHQWUHTXLUHPHQWV 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: fcꞔ SVLQRUPDOZHLJKWFRQFUHWH ꞔ = 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 Determine section properties torsion. rties for tors 9.2.4.4 Acp = bwh 22.7.6.1)(h – 3.5 in.) Aoh = (bw – 3.5 in.)(h 22.7.6.1.1 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. Step 2: 9.5.4.1 old torsion: on: Calculate threshold 2 22.7.4.1a ⎛ Acp ⎞ φTth = φλ 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 Calculation Acp = (16 16 in.)(24 in.) in. = 3384 in.2 Aoh = (16 (1 in. – 3.5 in.)(24 in.) in. – 3.5 in.) = 256 in.2 2 Ao = 0.85(256 0.8 0 6 in.) = 21 218 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 ⎞ φTth = (0.75)(1.0)(5000 psi) ⎜ ⎟ = 97, 750 in.-lb ⎝ 80 in. ⎠ []Tth = 8.15 ft-kip Tu IWiNLS![]Tth IWiNLSOK Design section to resist torsional moment. ⎛ Acp2 ⎞ φTcr = φ4λ 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, a reduction of Tu in the beam can occur due to redistribution of internal forces after torsion cracking. Therefore, reduce Tu to []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 ⎟ ≤ φ ⎜ 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 ≤ (0.75)(2 5000 psi + 8 5000 psi) (177.3 psi) 2 + (231.7 psi) 2 = 292 psi ≤ 530 psi OK Therefore, section is large enough. Step 3: Torsional reinforcement Find area/spacing of ties due to shear and torsional moment: Calculate torsional tie area/spacing: 2 Ao At f y 9.5.4.1 22.7.6.1a 22.7.6.1.2 φTn = φ 9.5.1.1 2.5.1.1 22.5.5.1 Calculate shear tie area/spacing: ng: []Vn = []Vc + []Vs•Vu = 61 kip and 22.5.10.5.3 21.2.1b φVs = φ s cot θ ≥ Tu = 32.6 ft-kip = 0.0199 in.2 /in. Av f y 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°) 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 /in. 0 s Av A + 2 t = 0.0253 in.2 /in. + 2(0.0199 in.2 /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 θ (32.6 ft-kip)(12 in./ft) 2(218 in.2) AA (60 ksi) = cot 45° 0.75 66 in. Aℓ = 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 ℓmin, must be at least the lesser of: (a) 5 f c′Acp fy f yt ⎛A⎞ – ⎜ t ⎟ ph ⎝ s ⎠ fy 5(5000 psi)(384 in.2) 60 ksi – 0.0324 in.2 /in. (66 in.) 60,000 psi 60 ksi (= 0.12 in.2 (b) 5 f c′Acp fy ⎛ 25bw ⎞ f yt –⎜ ph ⎟ fy ⎝ f yt ⎠ Controls 5(5000 psi)(384 in.2) ⎛ 25(16 in.) ⎞ 60 ksi (66 in.) –⎜ ⎟ ⎝ 60,000 psi ⎠ 60,000 psi 60 ksi = 1.82 in.2 A ℓSURY. = 1.32 in.2 > AℓUHT¶G = 0.12 in.2 9.7.5.1 Distribute torsional longitudinal reinforcement around the perimeter of closed stirrups that satisfy Section 25.7.1.6 (ends of stirrups are terminated with 135-degree standard hooks around a longitudinal bar). 9.7.6.3.3 Transverse torsional reinforcement spacing must n. not exceed the lesser of ph/8 and 12 in. DOEDUV3 8VH1RORQJLWXGLQDOEDUV3ODFH¿YH1RLQ bottom and two No.. 4 in each side face. Excess LQW ÀH[XUDOFDSDFLLW\ LQWRSRSDWRd 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 Institute – Copyrighted © Material – www.concrete.org OK 8.1—Introduction Building diaphragms are usually horizontal, reinforced 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 CIP topping, interconnected precast elements without CIP topping, or precast elements with end strips formed of CIP topping slab or edge beams (Moehle et al. 2010). Building slabs are designed to resist gravity loads and also WRWUDQVIHUZLQGDQGHDUWKTXDNHÀXLGRUZDWHUDOHDUWKSUHVVXUH forces to the lateral-force-resisting system, such as moment frames, shear walls, or both (ACI 318-14, Section 12.2). For dual system structures such as shear walls and special moment frames, special moment frames deform in a shear mode, as shown in Fig. 8.1(a), while 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 also provide lateral humb, app shear walls and columns. As a rule of thumb, approximately orce m 2 to 5 percent of a column axial force must be resisted by uate lateral support to the diaphragm to provide adequate y achieved in loww walls and columns. This force is easily mns rise building diaphragms, but mu must be checked for colu columns n high-rise buil ngs with high axial force, such as th those in buildings an include: de: (Moehle et al. 2010). 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 slab thickness must satisfy the requirements of Section 7.3.1 (ACI 318-14) for one-way slabs or Section 8.3.1 (ACI 318-14) for two-way slabs. The GLDSKUDJPWKLFNQHVVPXVWDOVREHVXI¿FLHQWWRUHVLVWLQSODQH 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 w 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 comm nly ddesigned ed as a de commonly deep beam that resists lateral forces HOZLWK DWWKHÀRRUOHYHOZLWKV\VWHPFROXPQVDQGZDOOVDFWLQJDV supports forr the deep beam. he diaphragm reinforcement resisting tension due to The ÀH[XUH ÀH[XUH LLV SODFHG DW WKH WHQVLRQ HGJH 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 due to lateral forces. DSSOLHGIRUFH7HQVLRQDQGFRPSUHVVLRQHGJHVDUHGHQWL¿HG 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 width in the direction of analysis (Section 12.5.2.3 of ACI 318-14). The diaphragm shear forces are resisted by the system columns and walls. The beams or slab sections that transfer VKHDUDUHLGHQWL¿HGDVFROOHFWRUV7KHFROOHFWRUVDOORUEHDP connection to the columns and walls must be appropriately designed and detailed to achieve shear transfer. Rigid diaphragms (Fig. 8.4b(a)) are often modeled as GHHS EHDPV ZLWK VSULQJ VXSSRUWV)LJ F /DWHUDO IRUFH is distributed to the columns and walls according to their relative lateral stiffness. Flexible diaphragms are modeled with rigid supports (Fig. 8.4b(b)). If all supports have equal)LJE²5LJLGDQGÀH[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¿QLWHHOHPHQWDQDO\VLVFDQEHXVHGWRGHWHUPLQHWKHGLDSKUDJPIRUFHVDQG DQDO\]H GLDSKUDJPV 7KH ¿QLWH HOHPHQW PHWKRG VKRXOG FRQVLGHU GLDSKUDJP ÀH[LELOLW\ 6HFWLRQ RI $&, 318-14). 8.4.1 Collectors—Shear walls do not usually extend the full length of a building. Collectors, also called drag members or distributers, 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 can have the same width as a shear wall (Fig. 8.4.1a) or be wider. Collectors eccentric to the wall have an effective width beffGH¿QHGDVQRWZLGHU than the thickness of the wall plus one-half the length of the shear wall (Seismology and Structural Standards Committee 6($2& $&,6HFWLRQ5)LJERI this Handbook). Collectors with the same width as the wall will simply transfer the slab lateral forces by axial compression or tension to the shear wall. Collectors having a width wider than the shear wall will transfer part of the diaphragm lateral force by axial compression or tension and the remainder will be transferred along the wall length through shear fric- Fig. 8.4.1b—Collector wider than the shear wall—forces are reversible. tion. An eccentricity results between the resultant force in the collector and the shear wall reaction (Fig. 8.4.1b). This eccentricity creates secondary stresses in the slab transfer UHJLRQDGMDFHQWWRWKHZDOO$GHTXDWHUHLQIRUFHPHQWPXVWEH SURYLGHGWRUHVLVWWKHVHVHFRQGDU\VWUHVVHV6($2& Collectors, like rigid diaphragms, are expected to behave elastically under axial and compression forces. Reinforcement is usually placed at mid-depth in collectors. Shear reinforcement perpendicular to the walls is needed as shear friction reinforcement for eccentric collectors, and is placed ZLWKLQWKHVODEWKLFNQHVV6($2& 8.5—Design strength Diaphragms in Seismic Design Categories (SDCs) D through F are designed in accordance with Chapter 18 of ACI 318-14. Diaphragms 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, 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 = ∑ Fi i=x n ∑ Wi W px i= x where Fpx is the diaphragm design force at level x. The design force applied to level xi is Fi; Wi is the weight tributary to level xi; and Wpx is the weight tributary to the diaphragm at 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 ȍo of ASCE/SEI 7-10, Section 12.4.3.2 rstrength factor (b) Fpx using load combinations with overstrength ȍo of ASCE/SEI 7-10, Section 12.4.3.2 (c) Fpx,min = 0.25SDSIWpx using load combinations of 3) fo ASCE/SEI 7-10 (Section 12.4.2.3) forces Fx are applied to DOOÀRRUOHYHOVFRQFXUUHQWO\ 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) 12.5.3.3 and nd Vn = Acv (2λ f c′ + ρt 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 12.5.3.4, Acv is the ss gross area of concrete section bounded by web thickness and section length in the direction of shear force considered, DQGȡt is the ratio of area of distributed transverse reinforcement to gross concrete area positioned perpendicular to the GLDSKUDJPÀH[XUDOUHLQIRUFHPHQW7KHUHGXFWLRQIDFWRU[]RU diaphragms is 0.75 (ACI 318-14, Section 12.5.3.2). In SDC ' (RU) ꞕFDQQRW H[FHHG WKH YDOXH FRUUHVSRQGLQJ WR WKH seismic-force-resisting system it is part of if the nominal shear strength of the member is less than the shear corresponding to the development of 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 reinforce diaphragm openings smaller than approximately twice the slab thickness with only the displaced reinforcement, but at least one bar on any side. /DUJHURSHQLQJVUHTXLUHDUDWLRQDODQDO\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 and rage zones anchorage Reinforcement /RQJ XGLQD /RQJLWXGLQDO 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]RQHVVKRXOGEHFRQ¿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. Collector longitudinal bars must extend deep enough into 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 complying with ASTM A615/A615M is permitted if: American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 285 Table 8.6b—Transverse reinforcement requirements for tension and compression collectors and chords in SDCs D through F reinforced with transverse confinement reinforcement (ACI 318-14, Section 18.12.7.6) Compressive stress Transverse reinforcement requirements Details ACI 318-14 Diaphragms Single or overlapping spirals per Sections 25.7.3.5 and 25.7.3.6 of ACI 318-14 Circular hoops or rectangular hoops with or without crossties spaced not more than 14 in. 18.12.7.5 > 0.2fcg (> 0.5fcg 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 reinforcement men 14 – hx DQGLQ•so" "LQ (c) so = 4 + 3 Rectilinear hoop Spiral or circular hoop Ash = 0.09 sbc f c′ f yt ⎛ Ag ⎞ f′ ρs = 0.45 ⎜ –1 c ⎟ Ach ⎠ f yt No 18.12.7.5 Greater of: ρs = 0.12 Fu = 52 kip OK NS: []Vn = 513 kip >> W: []Vn = 1940 ki 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: xceed: The nominal shearr sstrength, 8(72 72 ft)(10 in.)(12 in in./ft)(1.0) 4000 psi = 4372 kip 1000 lb/kip Vn = 8 Acv λ f c′ Vn = Acv is the diaphragm gross area less the 6.0 ft overhang. %\LQVSHFWLRQWKLVLVVDWLV¿HGK 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 /DWHUDOIRUFHLVGLVWULEXWHGWRWKHZDOOVDWHDFKIORRUZV refer to Fig. E1.4: 12.4.2.4(a) 12.5.1.3(a) Diaphragm is idealized as rigid. Design moments, shear, and axial forces are calculated assuming the 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 force distribution is calculated by using qL and qR as the left and right diaphragm reactions per unit length (Fig. E1.4): Fig. E1.4— Seismic forces in the lateral-forceresisting systems in the N-S direction. Design force: 95 kip Force equilibrium ⎛ L⎞ ⎛ L⎞ qL⎟ ⎟ + qR⎟ ⎟ = Fpx , deses (NS) ⎝ 2⎠ ⎝ 2⎠ ⎛ 218 ft ⎞ ⎛ 218 ft ⎞ qL⎟ + qR⎟ = 95 kip ⎝ 2 ⎟⎠ ⎝ 2 ⎟⎠ (I) Moment equilibrium rium ⎛ L⎞ ⎛ L⎞ ⎛ L ⎞ ⎛ 2L ⎞ ⎛L ⎞ qL⎟ ⎟⎟ ⎟ + qR⎟ ⎟⎟ ⎟ = Fpx , des (NS)⎟ + 0.05 0 05 0 L⎟ ⎝ 2⎠ ⎝ 3 ⎠ ⎝2 ⎝ 2⎠ ⎝ 3⎠ ⎠ From statics solvee equations e ions (I) and (II) ffor qL and qR: qL (218 ft)2 + q 2 (218 ft)2 6 R 6 ⎛ 218 ft ⎞ = (95 kip)⎟ + 10.9 ft ⎟ (II) ⎝ 2 ⎠ (I) qL + qR = 0.87 kip/ft 44 kip/ft (II) qL + 2qR = 1.44 p/ft and qL = 0.57 kip/ft qL =0.3 kip/ft American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS)LQGWKHPD[LPXPPRPHQWE\WDNLQJWKH¿UVW 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 p = 0.44 218 ft (0.44 kip/ft)(21 kip/ft)(218 ft) 2 = 2614 ft-kip 8 Note: Both appro 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 presented. Step 9: Chord reinforcement N-S Maximum moment is calculated above: 12.5.2.3 Mu = 2595 ft-kip Chord reinforcement resisting tension must be located within h/4 of the tension edge of the diaphragm. h/4 = 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 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 force The overhang is placed monolithic with the rest of the slab. 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 6WHHOVWUHVVLVWKHOHVVHURIWKHVSHFL¿HG\LHOG strength and 60,000 psi. 12.5.1.1 Required reinforcement ϕTn = ϕfyAs•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 18.12.7.6 ply. ACI 318-14 do nnot apply. 18.12.7.5 RU o for or chord design is not re2YHUVWUHQJWKIDFWRUȍ quired. Therefore, use the compression stress limit in provision 18.12.7.5 of 0.2fcȢ 2595 ft-kip = 37.6 kip 72 ft − 3 ft fy = 60,000 psi As , req eq ' d = 37,600 lbb = 0.70 in.2 (0.9)(60,000 9)(60,000 psi) Required chord width: wchord > CChord 0.2 f c′hdiaph wchord > 37,600 lb = 5.4 in. (0.2)(5000 psi)(7 in.) /HVVWKDQIWDVVXPHG7KHUHIRUHOK Choose reinforcement: Check if provided 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,req'd = 0.70 in.2 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 299 $OWKRXJKWKH¿UVWRSWLRQLVPRUHHFRQRPLFDOWKHVHFRQGRSWLRQLVGHWDLOHGLQWKLVH[DPSOH+HUHDJDLQWKH 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 part of the tensile force of the chord. Additional reinforcement is provided 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 forces from the diaphragm to the vertical walls at both east and west ends along column lines 1 and 7 (Fig. E1.2). Collector elements extend over the full width of the diaphragm. Unit 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 72 ft vu @ F = 52.2 kip = 1.86 kip/ft 28 ft Force at diaphragm to wall connection East wall south end: F7/D.5 = –(0.72 kip/ft)(22 ft) = –15.8 kip East wall north end: F7/B.5 = –15.8 kip + (1.14 kip/ft)(28 ft) = 16.0 kip At diaphragm end: (0.72 kip/ft)(22 ft)(22 ft) = 00.2 kip F7/A = +16 kip – (0.7 GXH PEHUURXQGLQJ §NLSGXHWRQXPEHUURXQGLQJ rc diagram, gram, the maxim m ax Per collector force maximum axial ec is Tu = Cu = 16 ki kip. This force on the collector nsfe from the diap gm to the 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, ȍo = 2.5 (ASCE/SEI 7-10, Table 12.2-1). 12.5.4.2 Collectors are designed as tension members, compression 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: ϕTn = ϕfyAs•Tu As , req ' d = Tu 16,000 lb = = 0.3 in.2 0.9 f y (0.9)(60,000 psi) $OWKRXJKRQH1REDUVXI¿FHVWZR1REDUVDUH provided to maintain symmetry of load being transferred from the slab into the wall. 18.12.7.5 Check if collector compressive force exceeds 0.2fcȢ. Calculate minimum required collector width using 0.2fcȢ wcoll . > CColl . 0.2 f c′tdiaph This results in compressive stress on the concrete diaphragm collector being relatively low. The section is adequate to transfer shear stress without additional reinforcement. wcoll > 16,000 lb = 2.3 in. (0.2)(5000 psi)(7 in.) Use 12 in. wide collector (same width as shear wall). Provide two No. 5 bars at mid-depth of slab to prevent additional out-of-plane bending stresses in the slab. Space the two No. 5 bars at 8 in. on center starting at 2 in. ffrom the edge of the diaphragm within the 12 in. wide collecto collector/shear wall (Fig. E1.8). ssion an The collector compression and tension forces are ateral force-resisting system transferred to the lateral within its width (she (shear wall). Therefore, no eccenin-plane bendi tricity is presentt an and no-in-plane bending occurs. 12.5.4.1 rm to discontinue the collector ollec ACI 318-14 permits h oof the shear wall wher along the length where transfer of design collector iss nnot required. Fig. E1.8—Collector reinforcement. 12.5.3.3 Check slab shear strength along shear walls Slab shear strength along walls: L = 28 ft and slab thickness t = 7 in. From φVc = φAcv 2λ f c′ φVc = (0.75)(2)(1.0) 21.2.4.2 ϕ0.75 =ϕVc OENLS 12.5.1.1 Is the provided shear strength adequate? ϕVc = 249 kip > Vu = 52 kip (from Step 7) 12.5.3.4 By inspection, the diaphragm shear design force VDWLV¿HVWKHUHTXLUHPHQWRI6HFWLRQRI$&, 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 kip/ft k 200 ft Wall 4: Fu = 140 W 40 kip In slab: vu @ F = 40 kip 140 = 5 kip/ft 28 ft Wall 5: Fu = 223 kip In wall: vu @ F = 223 kip 40 ft = 5.58 kip/ft 40 ft American Concrete Institute – Copyrighted © Material – www.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): Diaphragms Wall 1 west end:]NLSV IW #NLS Wall 1 east end: –106.7kip + (2.36kip/ft)(60ft) = 106.6 kip Diaphragm end: 106.6 kip – (1.94 kip/ft)(55 ft) = 0 kip Wall 3 west end: –(1.82 kip/ft)(40 ft) = –72.8 kip Wall 4 east end: –72.8 kip + (3.18 kip/ft)(28 ft) = 16.2 kip Wall 5 west end:]NLSLSNW IW #NLS Wall 5 east end: –16.2 kip + (2.36 kip/ft)(40 ft) = 78.8 kip –78.4 kip = 0 kip The collector force is the N-S direction. The collector force at each end: 82 kip/ft)(40 ft) = 0 kip The collector force for Walls 1 and 5 west end is shown in Fig. E3.13 ... This collector force is then multiplied by the system RYHUVWUHQJWKIDFWRU Ωo = 2.5 for building systems with special structural walls in SDC D (ASCE/SEI 7, Table 12.2-1). Wall 1 Wall 4 Wall 5 2.5Ωo 268 268 182 41 197 180 2.5Ωo –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. 7KHLUHQIRUFHPHQWDQGSLQ collector (see Fig. E3.14) is the concrete slab acting as a collector. The collector width is determined by engineerLQJMXGJPHQWDQGFKRVHQVXFKWKDWLWVWUHVVHV are not exceeded. When the tension and compression collector forces are increased by the overstrength factor, then the limiting concrete compressive stress is 0.5fcQDQGQRWUHLQIRUFHPHQWFRQ collector. The collector is effective width is determined ... Step 16: Collector reinforcement design (Fig. E3.16) The shear strength of the collector is checked by ... the wall thickness (8 in.). Part of the seismic force is transferred from the slab to the wall to transfer the force directly to the end of the shear wall and direct bearing of wall slab against wall in compression. Therefore, two No. 6 in-line with the wall. The balance of seismic force is transferred by shear friction ... Collector for Walls 4 and 5 are shown ... the wall through shear friction. ... Required area of collector reinforcement: ... Walls 1 and 5 ... Required reinforcement area in the N-S direction ... Vn = μAvffy ... The required area of reinforcement is calculated as ... Collector reinforcement required ... This is acceptable as the required chord reinforcement can be ... provided. Collector reinforcement ... 3 ... Place reinforcement in the collector ... distance of 45.67 ft. Spreading bars over more than half the diaphragm width is not practical. 2. Use the calculated chord reinforcement in the N-S direction to resist the collector force in the E-W direction or h/4. This option is acceptable as the required chord and collector calculated reinforcement is approximately equal. 3. The collector is wider than the wall, therefore, longitudinal and transverse reinforcement must be provided to transfer forces from the collector into the wall. 3. Place bars in a 2 ft 0 in. deep by 12 in. wide edge Collector and chord reinforcement must be provided 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 reinforcement to resist the gravity dead and live load is calculated to be equal to 3.8 in.2. Required collector reinforcement is 5 in.2 calculated above. Therefore, a total of 8.8 in. 2 must be placed within the beam to resist gravity loads combined with 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 design is carried out for 1.2D + 1.6L. For seismic, the 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 (12 in.) is wider than the wall (8 in.). Nu = (268 kip)(2 in.) = 536 kip in. (Fig. E3.14). Eccentricity is considered when ... The force transferred from the beam to the wall ... Tension and compression forces ... The axial force due to the eccentricity ... Mu = ... The force is very small and the corresponding reinforcement is negligible; therefore, it is assumed that the result of the eccentric force between beam and wall cent centerlines is resisted by the diaphragm. In case the force is large (large eccentricity, large force, or shorter wall length), reinforcement is required: ... Check the shear strength ... Required collector shear reinforcement ... Reinforcement ... wall. ... Required area of the ... vertical reinforcement ... As the wall is designed ... Therefore, the collector reinforcement is adequate ... No. 9 bars ... No. ... Friction reinforcement is not required as the collector force is already developed into the wall. ... Shear friction reinforcement is not required as the collector force is already developed into the wall ... 346 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 16: Shrinkage and temperature reinforcement 12.6.1 Shrinkage and temperature reinforcement 24.4.3 24.4.3.2 AS+T ≥ 0.0018(10 in.)(16 in./ft) = 0.288 in.2 Spacing of S+T reinforcement is the lesser of 5h and 18 in. ... Controls Note: Shrinkage and temperature reinforcement may be part of the slab flexure reinforcement resisting diaphragm in-plane forces and gravity loads. If provided reinforcement is not continuous along ... structural walls, the engineer must ensure continuity between top and bottom reinforcing bars to provide adequate splice lengths between them. Step 17: Reinforcement detailing Development 25.4.2.2 Chord and collector reinforcement are detailed 25.4.10.2 over full length and width of the edges of the 12.7.3.2 diaphragm. Therefore, development length will be ... Development length of shear transfer reinforcement ... Lap splice lengths ... Class B lap splice ... The centroid of the longitudinal bars ... Collector reinforcement spacing and cover ... The 2 in. cover ... Chord and collector reinforcement ... 18.12.7.5 ... Step 18: ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 347 Diaphragms Step 18: Detailing Fig. E3.14—Diaphragm reinforcement ment detailing. Section A—Edge beam gravity and collector or chord reinforcement. American Concrete Institute – Copyrighted © Material – www.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. Note: Wall reinforcement not shown for clarity for Walls W4 and W5 but should be detailed ... 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 analysis are compared to the forces due to the diaphragm inertia forces. The diaphragm is designed for the larger 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 seismic loading, referred to as accidental eccentricity. This ... 5 percent accidental eccentricity ... is not mandatory. 350 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) When center of mass and 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 and (W2 and (32 kip)) for a seismic force acting in the E-W 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. The resulting shears and moments are acceptable for diaphragm design. Note that this approach leaves any moment due to (shear walls perpendicular to the diaphragm inertial force) unresolved; sometimes this is ignored or, alternatively, it too can be incorporated in the trapezoidal loading." ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 8—DIAPHRAGMS 351 Diaphragms The diaphragm in-plane shears ... Fig. E3.15—shear and bending moment diagrams. Note: ... Fig. E3.16—Free body diagram. Mx = (131 kip × 208 kip/ft ... Chapter 9—Columns ... American Concrete Institute – Copyrighted © Material – www.concrete.org 352 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS 9.1—General The provisions of Chapter 10 in ACI 318-14 apply for the design of nonprestressed, prestressed, and composite columns. Headings of a section are considered part of the code and care should be taken to notice when a heading limits the following requirements to a particular type of column. The word "nonprestressed" is typically in reference to cast-in-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, ... shapes. Section 10.3 in ACI 318-14 allows the use of various cross-sectional shapes. Section 9.3.1 in ACI 318-14 permits an idealized cross-section for column 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 to be designed with a smaller effective area, with a lower limit of one-half the total area. The analysis and design assume the smaller effective area, but the detailing 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 of the actual area. Note that the shape of the column can be determined by the building architecture but it must always requirements of ACI & ACI 318-14 meet the requirements ... 9.3.2 Limit ... It is common to use ... Chapter 9 for a ... Use the strength of concrete for all columns at a 1. Structures Structur under six stories or less commonly level. ... Concrete compressive strength for the full height of the building. Note section 15.3 in ACI 318-14 ... the column ... 9.4 ... columns exceeds 1.4 times ... Exceptions to ... strengths ... 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 0.4fcʹ for ordinary columns or by 0.3fcʹ for ... ACI 318-14 ... Columns are usually rectangular, square, or round. ... This is generally expected with the full 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, Chapter 10 in ACI 318-14, follows the organization of the other member chapters: applicability; initial data; analysis to determine the required strength; design area of reinforcement needed to exceed the required strength; check against minimum reinforcement required; and detailing. The analysis must be consistent with ACI 318-14, Chapters 4 through 6. A quick review of Sections 10.1 through 10.4 in 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 gravity force-resisting system, and in cast-in-place construction is often part of the ... lateral force-resisting system ... For buildings designed to resist seismic forces, the requirements of Chapter 18 in ACI 318-14 also apply, along with the additional detailing requirements for columns that are part of an ordinary, intermediate, or special moment frame system. There are also seismic requirements ... frame columns ... Section 18.14 ... the building ... 9.4—Required strength 9.4.1 General—Required strength is calculated using factored load combinations ... of Chapter 5 and procedures in Chapter 6 of ACI 318-14. Three methods of ... Second-order effects ... For a column, or ... The second-order moments ... These methods ... analysis ... equation ... slenderness effects ... column in sway and nonsway frames ... 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 frame ... American Concrete Institute – Copyrighted © Material – www.concrete.org Columns ... 9.4.2 Column ... slenderness ... Lower and upper limits for the ... Fig. 9.4.2 ... The column nonsway ... 6.6.4 ... moment magnifier ... Section 6.2.5 ... A column of ... effective length ... ignore ... kℓu/r ... 22 ... 34 − 12(M1/M2) ... 40 ... Section 6.6.4.4.4 ... Section 6.7 ... Section 6.8 ... second-order effects can be neglected ... 100 ... stiffness ... Fig. 9.4.2 ... Column ... Section 6.6.3.1 ... 9.4.3.2 Column slenderness ... reduced moment of inertia ... cracked sections ... Section 6.2.5 in ACI 318-14 ... column is braced against sidesway ... sway ... second-order effects ... American Concrete Institute – Copyrighted © Material – www.concrete.org ... 9.5—Design strength ... Column ... interaction diagram ... Pu ... Mu ... the designer ... Nominal strength ... In Section 22.4 ... axial ... Design Handbook ... The interaction diagram ... strength reduction factor ... compression-controlled ... tension-controlled ... (Fig. 9.5) ... 0.65 ... 0.75 ... 0.9 ... The design strength curve ... Fig. 9.5 ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS ... 9.6—Reinforcement limits The minimum column vertical steel ratio is 0.01Ag. This amount is enough to keep the reinforcement from yielding due to concrete creep under sustained service loads (ACI 318-14, Section R10.6.1.1). The maximum reinforcement ratio is 0.08Ag. 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 bar spacing requirements. ... The lap splices ... 9.7—Reinforcement detailing 9.7.1 General—Section 10.6 in ACI 318-14 focuses on the calculation of reinforcement area needed to resist design forces and moments. 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 for 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 Spacing—The minimum number of bars in a column is four in a rectangular or circular ... Section 10.7.3.1 ... The number of bars ... ties ... 9.7.2.2 Splicing—Splice locations, lengths, and types ... Mechanical connectors and buttwelded splices are helpful where bar arrangement becomes congested ... lap splices ... American Concrete Institute – Copyrighted © Material – www.concrete.org Columns ... 358 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. 9.7.2.2a—Column splice locations ... Fig. 9.7.2.2b—Where there is a reduction of column ... longitudinal bars ... offset bent ... 9.7.3 Transverse bars 9.7.3.1 Column ties—Standard (nonseismic) arrangements of ties for various numbers of vertical bars are shown in Fig. 9.7.3.1. ... 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 generic description of a circular tie. ... The spiral size and pitch should meet the ... (ACI 318-14 Eq. (25.7.3.3)) ... American Concrete Institute – Copyrighted © Material – www.concrete.org Columns 359 Columns CHAPTER 9—COLUMNS Fig. 9.7.2.2b—Column splices. ... Fig. 9.7.3.1—Standard column ties ... (a) ... (b) ... (c) ... Table 9.7.3.2—Minimum diameters of spiral reinforcement ... 360 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) (C) Interaction diagrams ... 9.8—Design steps ... 1. Determine factored loads ... 2. ... Use load combinations ... 3. Determine slenderness ... 4. Calculate ... 5. 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 Columns 9.9—Examples ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS 361 References American Concrete Institute (ACI) ACI 315-99—Details and Detailing of Concrete Reinforcement ... Anchorage 318-14 ... ACI 318-14—Building Code Requirements for Structural Concrete and Commentary ... Concrete Reinforcing Steel Institute (CRSI) ... Manual of Standard Practice ... Example 1: Column analysis ... Given: ... Column ... Frame ... Materials—Concrete ... American Concrete Institute – Copyrighted © Material – www.concrete.org ... 362 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ... Sway frame ... Step 1—Determine initial column size ... Step 2—Sway or nonsway moment frame ... The ... column ... Step 3—Check for slenderness ... Step 4—Second-order effects ... 6.6.4.6 ... δs ... Q ... American Concrete Institute – Copyrighted © Material – www.concrete.org 363 364 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ... Ec = 57,000 ... Ig ... EI ... k ... The effective length factor ... ψ ... Pc ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS ... ΣPu ... ΣPc ... δs ... Column Example 1 ... Moment magnifier ... American Concrete Institute – Copyrighted © Material – www.concrete.org Columns ... 6.6.4.6.2 ... The δs ... 1.5 ... American Concrete Institute – Copyrighted © Material – www.concrete.org ... 366 ... Step 5 ... second-order ... M2 ... 1.4 ... Steps ... second-order analysis ... which is less than 1.4 ... ZDVDLGHGZLWKDåo computed by a computer DQDO\VLV,IWKDWGHÀHFWLRQZDVOLPLWHG ... American Concrete Institute – Copyrighted © Material – www.concrete.org Columns ...

ZDVDLGHGZLWKDåo computed by a computer DQDO\VLV,IWKDWGHÀHFWLRQZDVOLPLWHGWR 0.40 ... 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 Concrete Institute – Copyrighted © Material – www.concrete.org Columns Hand ¿UVWRUGHU 370 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Column Example 2: Column for an ordinary moment frame²'HVLJQDQGGHWDLOWKH¿UVWÀRRULQWHULRUFROXPQDWORFDWLRQ(from the example building given in Chapter 1 of this Handbook. The column is part of an ordinary moment frame. Example LVWKHGHVLJQRIWKHFROXPQDQDO\]HGLQ([DPSOH7KHORDGVKDYHEHHQPRGL¿HGWRDWWDFKWKHUHVXOWVRIDQDO\VLVIURP commercial software capable of second-order elastic analysis. Given: Materials— 6SHFL¿HG\LHOGVWUHQJWKfy = 60 ksi Modulus of elasticity of steel, Es = 29,000 ksi 6SHFL¿HGFRQFUHWHFRPSUHVVLYHVWUHQJWK fcg 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 + 1.0W + 1.0L + 0.5S (iii) U* = 1.2D + 1.0E + 1.0L + 0.2S (iv) U* = 0.9D + 1.0E + 1.0L + 0.2S Pu, kip 890 800 818 486 Mu, kip-in. 0 651 2401 2401 Vu , kip 0 5 18 18 7KHVRIWZDUHDGMXVWVVHLVPLFORDGFRPELQDWLRQVDVUHTXLUHGE\$6&(XUHGE\$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 tudinal reinfor ment Î en ection 22.4 for tthee calculation of Pn and Mn. Section The Code references Section ti 22.4 22 provides an equation to Î calculate P0 and d rreferences 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 ratio ve diagram đ This example used the spreadsheet to create an interactive diagram. The generated interaction diagram ent is satisfactory. shows if the assumed longitudinal 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\VKRZWKDWDOWHUQDWHVHFWLRQRUPRUHUHLQIRUFHPHQWLVQHHGHG in which case, the analysis will need to be redone. The spreadsheet analyzes rectangular or square colXPQVIRUFRPELQHGD[LDODQGÀH[XUDOVWUHQJWK7KH 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 ¿UVWDQGODVWOD\HUDUHVSDFHGWRWKHRXWHUIDFHDV permitted and the remaining bars are evenly spaced between the outer bars. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS Î 22.2.2.4.3 Î 25.7.2.2 Î 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. /RQJLWXGLQDOEDUVL]HVKRXOGEHNo. 8 or greater, to make stable column cages for erection and to reduce the number of ties at a section. Î 21.2 Î Î Try 8 bars, one in each corner and one on each side (3 layers: 3 bars, 2 bars, 3 bars) Area bar needed = 5.76 in.2/8 = 0.72 in.2 At least four bars are needed in a rectangular 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 DFFRUGLQJWR6HFWLRQ7KHùLVDIXQFWLRQRI No. layers: 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 HFWLRQ7KHPLQLPXPFRQFUHWHFRYHUIRUFROXPQVQRWSURYLGHG¿UHSURWHFWLRQ7KHPLQLPXPFRYHUIRUFROXPQVQRW 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 Î 25.2.3 As,min = 0.01Ag = 0.01 × 242 = 5.76 in.2 ù 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 /RQJEDUVL]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 ™ 6.32 The minimum bar spacing is calculated and displayed for the user's information. Key variables needed for design are displayed for WKHXVHU¶VLQIRUPDWLRQ7KHVWUDLQOLPLWVDQG[DFWRUV change as the column interaction diagram transitions from compression-controlled sections to 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¿QLWLRQV Strain Definitions fy/E= 0.00207 Îcu = 0.003 Ductile Strain = -0.005 Brittle Strain = -0.002 tension-controlled = 0.9 compression-controlled = 0.65 The interaction diagram is made by calculating the Pn and Mn for incremental changes in the net tensile strain in the extreme layer of longitudinal tensile reinforcement at nominal strength, Ît. A random strain of 0.0007 was chosen to illustrate the calculation of Pn and Mn in the following steps. 9DU\Ît from pure compression, Ît equal to fy/Es at Find Pn and Mn for Ît = 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[LVIURPWKHVWUDLQGLDJUDPXVLQJVLPLODUWULDQJOHV c= c= −0.003 × 21.5 = 28.09 in. 0.0007 − 0.003 −ε cu × d1 ε t − ε cu 3. Calculate aWKHGHSWKRIWKHHTXLYDOHQWVWUHVV EORFN ⌈β ×c a = min ⌋ 1 ⌊h ⌈ 0.80 × 28.09 = 22.47 in. a = min ⌋ ⌊ h = 24 in. Use a = 22.5 in. 4. Calculate Cc, the force at concrete compression EORFN Cc = 0.85 × a × b × fcg Cc = × × × = NLSV &DOFXODWHÎsiWKHVWUDLQDWHDFKODPHURIVWHHO)RUÎs1ÎsDQGÎs3, c•diWKXV¿QGWKHVWUDLQXVLQJPKMPI ⌈ (c − d i) ⌋ ε cu × c if c ≥ di then ε si = min ⌊ fy ⌋ ⌊ Es ⌊ ⌈ (c − d i) ⌋ ε cu × c if c < di then ε si = max ⌊ fy ⌋ − ⌊ E s ⌊ 6. Calculate FsiWKHIRUFHDWHDFKODPHURIVWHHO ε s1 = 0.003 × εs2 (28.1 − 21.5) = 0.000703 28.1 (28.1 − 12) = 0.00172 = 0.003 0 003 × 0 0.003 × ε s 3 = 28. 1 28 1 (28.1 − 2.5) = 0.00273 28.1 fy 60 = = 0.00207 29, 000 Es 2 and 8VHÎs1 Î Îs2 DQGÎs3 = 0.00207 For Fs1, Fs22, and Fs3Îs•WKXV ,IÎsi > 0, then Fsi Îsi × Asi × Es − 0.85 × Asi × fcg Fs1 = 0.000703 × 2.37 × 29,000 − 0.85 × 2.37 × 5 NLS 1RWHWKDWSRVLWLYHVWUDLQLGLFDWHVWHQVLRQ Fs2 îî±îî NLS compression. The force is adjusted to account for the Fs3 îî±îî NLS displaced concrete. ,IÎsi"WKHQFsi Îsi × Asi × Es 7. Calculate Po. Po = Cc™Fsi Po NLS Note that Pn,max is not applied in this spreadsheet XQWLOWKHGHVLJQVWUHQJWKFXUYHLVFDOFXODWHG 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 − 12 = 0 in. d3 = 12 − 2.5 = 9.5 in. dCc = American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 9—COLUMNS 373 9. Calculate Mi, the moment for each force about the center of the section. MCc = dCc × Cc M Cc = 0.77 × Mi = di × Fsi M 1 = −9.5 × M2 = 0 × 2292 = 147 ft·kip 12 38.3 = −30.3 ft·kip 12 108 = 0 ft·kip 12 M 3 = 9.5 × 132 = 105 ft · kip 12 Î 21.2 Î 22.4.2.1 Mn = 147 − 30.3 + 0 + 105 = 222 ft·kip Mn = MCc™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. to create the design strength interaction sion-controlled sections is The DFWRUfor compression-controlled DFWRUIRU IRUWHQVLRQFRQWUROOHG This 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 shown which han in the lower part of the curve. ates the sharp change n the right ht is for this column column exe The diagram on ads from the fou ample and it has th 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" /LQHPHDQQLQJ ing all the bars remain in compression. Point iv) is for a load combination with lighter gravity loads. ,WLVMXVWWRWKHULJKWRIWKHOLQHPHDQLQJIDHZEDUV 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 Î The Code references Section 22.5 for the calculation /RDG&RPELQDWLRQLLL 22.5 of Vn. The PuJLYHQLQ/RDG&RPELQDWLRQLLL KDVWKH Vu = 18 kip expression 0.2SDS is applied in the downward posiPu = 737 kip ton as required by ASCE 7-10. The Pu value shown KHUHLVFKDQJHGWRUHÀHFWSDS in the upward position for the lowest axial load associated with this load combination. 18.3.3 Note that Ɛu (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¿QGVc. 21.5 = 73.0 kip ≥ 18 kip 1000 Vc = 2λ f c′bw d Vc = 2 × 1.0 5000 × 24 × 10.6.2 Î 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 Î 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. 10 or smaller; however, a No. 4 tie was chosen as discussed in Step 1. 25.7.2.1 The minimum spacing is (4/3)dagg; however, the trols for this exmaximum tie requirements controls ample. Use No. 4 ties. acing shall not exceed the least of: The maximum spacing in Controls. (a) 166 × 1.00 in. = 16 in. ongi l bar (a) 16db of the longitudinal (b n. = 24 in. (b) 48 × 0.50 in. transverse se bar (b) 48 db of the tra (c (c) 24 in in. (c) h or b 25.7.2.3 Fig. R25.7.2.3a equ hat lateral su Section 25.7.2.3 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 oblong. It is common to use alternative tie geometry, such as cross ties, for columns that are not square. 6WHS²&KHFNWKHMRLQW | 7KHWUDQVIHUUHGVKHDUIRUFHPDQGLVULHVFRUWKH MRLQW,VUHTXLUHGVKHDUVWUHQJWK, WLVFRPPXWHGVKHDU KHDG JR. 15.3 FRUHTXLUHPHQWVDQDOVLVRSHUDWRU UNNKHDU strength is greater than 1.4, then the Code provides three options to account for this difference. This building uses the same tie 'JUHLUFRRULNHDGRQTPUDQRNWRNIXQUHDU KHDGVQL,,RDORFHXQF... 15.2.2. 6HFWLRQ(RQWLQXHG):6HFWLRQ 15.3, moment is being transferred to the column. Section 15.4 does not require minimum shear reinforcement 15.2.4, WKURXJKWKHMRLQWLVHTXLUHPHQWLHQFHRUGHU DQG(FOFUHQFH), FOFUREJURVGUDWHUGUVLLVPLF 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 6—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. As shall be at least the greater of: $0.75\sqrt{f'_c}$ 50 15.4.2.1 At each tie, Av = 0.20 in.2 × 4 legs = 0.80 in.2 Check s = 0.75 × 5000 × 24 × 16 = 0.32 in.2 ≥ 0.80 in.2 60, 000 by 5s ≤ 16 in. ≥ 2.60, 000 by 5 The maximum spacing is one-half the depth of the shallowest beam or slab framing into the element and shall be distributed along the column for the full depth of the deepest member. Find the maximum spacing in joint: sw = 30 in./2 = 15 in. ≤ 16 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&RGHGRHVQRWLQFOXGHDQHSOLFLWHTXDWLRQ:VKHDU strength is concern; the shear strength requirements of Section 18.8.4 of the Code can be used. Note LVQRWVLQJOHXVHGWRGHILQHWKHMRLQWLQHDU VKHDU

... 6WHS²'HWDLOLQJ,QWKHVKHDUIRUFH the column and column cage on top of column can be taken as 24 in. The hook cannot have a lap splice length is greater than 1.4 ... Ld = 2 Ld To determine 0.5 ... For No. 8 bars = 29 in. ... For No. 8 bar = 1.0 ... For column ... American Concrete Institute – Copyrighted © Material – www.concrete.org Columns 15.4.2 375 376 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ...

10.7.5.2.1 For ease of construction, use Lst = 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 ... Check the embedment length. Assume: fy = 60,000 psi ... The calculations shows that one additional No 4 tie can laterally support one No. 8 bar at the effect. Esc = 30 × 1.0 = 30 in. Nominal vertical bar strength: No. 8 = (fyAst) = 60 × 0.79 = 47.4 kip Horizontal tension at the bend: Pn = 1.5 × 47.4 = 71.1 kip ... 11.7 kip Nominal tie strength: No. 4 = (fyAst ...) Provide one tie legs for each vertical bar at its offset. The current tie detail meets this requirement. ... The vertical component of the computed tie force ...

Step 4—Discussion and summary ... The reinforcing of this column for an intermediate moment frame (IMF) is ... American Concrete Institute – Copyrighted © Material – www.concrete.org ... For the ACI 318-14 Section ... CHAPTER 6—COLUMNS 377 Column Example 3: Column for an intermediate moment frame (IMF)

... From Column Example 2 the column for an intermediate moment frame (IMF) is checked for the load combinations ... Given: Material properties — Concrete compressive strength, f'c = 5000 psi Modulus of elasticity of steel, Es = 29,000 ksi ... Normalized maximum size of aggregate is 1 in. Loading ... 378 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3—Find the required area of longitudinal reinforcement The same column from Example 2 is checked using Design Capacity Interaction Diagram: using the interaction diagram spreadsheet referenced at the start of this example. See Example 2 for a detailed discussion on the calculation of nominal and design interaction diagrams. The diagram on the right is for this column example and it has the loads from the four given load combinations plotted. Eight No. 8 bars evenly spaced around the perimeter ... use the diagram spreadsheet provided with the Code ... that the structure must be designed ... 378 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 3—Find the required area of transverse reinforcement ... Section 18.4.3 ...

Notes: the required flexural strength of the beams or the factored load ... ACI 318-14 Section ... NIST GCR 8-917-1 ... "Seismic Design of Reinforced Concrete Special Moment Frames: A Guide for Practicing Engineers," NIST GCR 8-917-1 ... φVc = 27 kip ≥ Vu = 25 kip 2 The range of nominal moments is shown in the ... For Pu = 848 kip, Mn = 9373 kip-in. For Pu = 456 kip, Mn = 7994 kip-in. Use, Mn = 9373 kip-in. For Pu = 83 kip, Mn = 6520 kip-in. ... From the Step ...

Example Column 3 ... 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 × 1 = 8 in. Controls. (a) 8db, smallest longitudinal bar (b) 24db, hoop (b) 24 × 0.5 = 12 in. (c) 0.5 = 15 in. (d) 12 in. 18.4.3.5 ...

The hoops at the plastic hinges, lo, and the splice must meet geometry requirements of Section 18.7.5.2 and 25.7.2, 25.7.4.

... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 6—COLUMNS 381 ... 382 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Column Example 4: Column for a special moment frame (SMF)

Given: Material properties — Concrete compressive strength, f'c = 5000 psi Modulus of elasticity of steel, Es = 29,000 ksi ... Normal weight concrete ... Loading — ... 2 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 ... Pu 108 890 872 432 Mu, kip-in. 6520 4491 ...

Step 1—Geometry ... Step 2—Find the required area of longitudinal reinforcement ... A SMF is perm ... American Concrete Institute – Copyrighted © Material – www.concrete.org ...

Step 5—Determine the geometry of the transverse reinforcement ... 18.7.4.3 Hoops for rectangular columns or spirals for 18.7.5.1 circular columns are required for the entire column 18.7.5.2 ...

Step 6—Find the required shear strength ... The probable moment strength ... Mpr1 = 8010 kip-in. Mpr2 = 4920 kip-in. Ve1 = 85 kip Ve2 = −22 kip Also, ... Ve = 192 kip ...

Step 8—Summarize the amount and spacing of transverse reinforcement along the length of the column 18.7.5.1 ...

... American Concrete Institute – Copyrighted © Material – www.concrete.org 383 ... 384 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Column Example 4 ...

Step 10—Discussion and summary

This example is an extension of 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 different transverse reinforcement layout than the previous example. ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 10—STRUCTURAL WALLS 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 the seismic-force-resisting system. Reinforced concrete structural walls are common in buildings, and are almost always part of a building's lateral-force-resisting system. In many buildings, walls may be used as part of the gravity-force-resisting system. They are often lightly axially loaded. These are, designed and detailed to resist the combined effects of gravity and lateral forces ... The gravity-force-resisting system of a building ... Buildings with walls ...

Walls are designed to resist axial loads, in-plane and out-of-plane shear and moment. ... ACI 318-14 ... Chapter 18 ... Special structural walls ... Intermediate precast walls ... Ordinary walls ...

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 debated. ... 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 requires that the longitudinal reinforcement must be supported by transverse ties. ... Walls must have a length-to-thickness ratio of at least ... Section 18.7.2.1 of the Code ... For walls, the Code recommends ...

10.2.2 Walls may be designed ... Shear walls may be designed either as a wall or column ... Shear walls with openings ...

10.2.3 ... Coupled walls ... (Fig. 10.2.3a) ...

10.3—Design limits 10.3.1 Minimum wall thickness ... Minimum thicknesses are as shown in Table 10.2.5. ... The NEHRP Technical Brief No. 6 (Moehle et al. 2011) suggests the following minimum wall thicknesses ... ACI 551.2R provides the following slenderness limits ...

10.4—Required strength ... ASCE 7 ... Response Spectrum Analysis ... Modal Response Spectrum Analysis (MRS) ...

... Table 10.3.1a—Design coefficients for shear walls in building frame systems Seismic-force-resisting system ... Table 10.3.1b ... Table 10.3.1c ...

10.4.2 Walls ... In-plane ... 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. ... Effective flange width ...

10.5—Design strength ... Section 11.5.4.8 of ACI 318-14. ... φ ...

10.5.2 Shear ... Vn = Acv(αc λ√f'c + ρt fy) ... ACI 318-14 Section 18.10.4 ...

... Special structural walls ... Boundary elements ... Section 18.10.6.2 ... displacement-based approach ... Section 18.10.6.3 ... stress-based approach ... δu/hw ...

... Coupling beams ... Section 18.10.7 ... diagonal reinforcement ... ℓn/h < 2 ... Vu > 4λ√f'c Acw ...

10.6—Detailing ... Minimum reinforcement ... Distribution ... Wall piers ...

10.7—Construction ... Tilt-up ... Precast ...

References

American Concrete Institute (ACI) ... ACI 551.2R-10—Design Guide for Tilt-Up Concrete Panels ... American Society of Civil Engineers (ASCE) ... ASCE/SEI 7-10 ... Moehle, J. P., Ghodsi, T., Hooper, J. D., Fields, D. C., 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 ... Wallace, J. W., 1996, "Evaluation of UBC-94 Provisions for Seismic Design of RC Structural Walls," Earthquake Spectra ...

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 the example is part of the lateral-force-resisting system (a shear wall is at each end of the structure) in the North/South (N-S) direction of the hotel (Fig. E1.1). Material properties are selected based on the code limits and requirements of ...

UHTXLUHGORDGFRPELQDWLRQVE\DQHODVWLF'¿QLWHHOHPHQWDQDO\VLVRIWKHUDWHGZDOOIRUHDFKRIWKH 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— fcç SVL fy = 60,000 psi)LJ(²%XLOGLQJÀRRUSODQ¿UVWÀRRU 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 thickness equations (a) and (b) of Table 11.3.1.1 can, however, provide an indication that the thickness chosen is appropriate. 20.6.1.3.1 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. lesser of the unsupported height of the wall (18 Example shear wall h = 12 in. > hreq'd = 8.64 in. IWIURWKH¿UVWHOHYDWHGÀRRU 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 cover is 1-1/2 in., which is in accordance with Table 20.6.1.3.1 (ACI 318-14). American 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 the assumptions and 6.9 requirements of 11.4. Walls The structure was analyzed using 3D elastic Finite 7KHPDF[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 section at the start of this example. 5 and 6 of ACI 318-14 for loading and analysis, respectively refer to Fig. E1.2 and E1.3 for in-plane ÀH[XUDOGLQSODQHVKHDUDORQJWKHKHLJKWRIWKH wall, respectively.)LJ(²,QSODQHÀH[XUHDORQJWKHKHLJKWRIWKHZDOO J WKHK Fig. E1.3—In-plane shear along the height of the wall. 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. $&,LVDUHIHUHQFHVSHFL¿FDWLRQWKDWLVFRRUGLnated with ACI 318-14. ACI encourages referencLQJ$&,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. 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¿HGWRDWOHDVW 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[XUDOGHVLJQVWUHQJWK 11.5, 11.5.2 7KHFRPELQHGD[LDODQGÀH[XUDOGHVLJQVWUHQJWKRI 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$&,UHTXLUHPHQWVDUHVDWLV¿HG In this case, assume grade 60 bar and no coatings. An initial interaction diagram is made using No. 5 bars at 12 in. spacing throughout the wall (refer to *Note below). It is assumed that all of the longitudinal reinIRUFHPHQWLVHIIHFWLYHUHVLVWLQJLQSODQHÀH[XUH 7KH¿UVWSDLURI1REDUVLVDVVXPHGWREHDWLQ from the end of the wall, the second pair is placed at 12 in. from the end of the wall, and the remaining pairs at 12 in. spacing. The different spacing at the end of the wall is to allow for cover on the end pairs of bars in the wall and to force the reinforcement to be symmetrical. The reinforcement is symmetrical about the center of the wall and this bar layout is applied to both ends of the wall. To estimate an initial reinforcement area, the wall is Fig. E1.4 shows the resulting design strength interacDPRXQWRIÀH[XUDO tion diagra DVVXPHGDVDFDQWLOHYHUHGUHDPKDYLQJDPRXQWRIÀH[XUDO diagram. The design strength interaction diagram y to resist th LQFOXGHVWKHI reinforcement necessary the moment is LQFOXGHVWKHIDFWRU calculated. ion Diagra The Inte Interaction Diagram spreadsheet contains a sheet PHG W$[LDO/R QDPHG³6HOHFWHG$[LDO/RDG´:KHQWKHXVHUHQWHUVD Pn, the ssheet calculates lculates 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 Mu of 24,600 ft-kip. 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 WKHVWUHQJWKRIWKHZDOOEXWDUHRIWHQWRRÀH[LEOHWRHI¿FLHQWO\ZRUNZLWKLQDVHUWLFDOSRVLWLRQ American Concrete Institute – Copyrighted © Material – www.concrete.org Fig. E1.4—Design strength interaction diagram. Step 5: Shear design strength (in-plane) 11.5.4 Shear walls resist both in-plane and out-of-plane 11.5.4.4 shear. The out-of-plane shear is very small, 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 dd Ɛw = (0.8)(336 h = 12 in., wall th thickness in.) = 268.8 in. From Table 21.2.1(b) .1(usee shear strength reduction eduction IDFWRU &KHFNLVc•Vu Step 6: Flexure design strength (out-of-plane) 11.5.1 As shown in Step 4, the layers of No. 5 vertical 11.5.3.1 ZDOOUHLQIRUFHPHQWDUHLQWKHZDOOFRQWHUHQRXJKWR 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 56 456)(0.75) kip) = 342 kip Vc = 342 kip > Vu = 235 kip OK Eccentricity of the resultant load: (1015 kip)(e) = 60 ft-kip e = 0.7 in. e < 2 in. Pn = 0.55(f c′)(Ag) 1 − c 32h Pn = 0.55(5 ksi)(12 in.)(336 in.) 1 − 32(12 in.) From Table 21.2.2(b), use axial strength reduction IDFWRUPn = 9090 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[FHHGVVc, 11.6.2 applies. 11.6.2 Assuming 12 in. for spacing in both directions, the Vu•Vc minimum reinforcement results in: D DQGE PXVWEHVDWLV¿HG D d£ must be the greater of the three values below: d£ 0.0025 0.62 in.2 d£•±hwƐw ïd£± ρA ≥ 0.0025 + 0.5(2.5 − 92ft/28 ft) × − 0.0025 (12 in.)

multiplied by the footing volume. Foundationss (442 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 in.)(10.0 ft)(12 in./ft) = 4.3 in.2 AS+T•Ag Ten No. 6 bottom longitudinal bars will satisfy the requirement for shrinkage and temperature reinforcement in the long direction. Development length Check if the width of the footing provides adequate length for the bottom tension reinforcement beyond the critical tension section. 25.4.2.3 25.4.2.4 25.4.2.3 ⎛ ⎞⎜ 3 fy ψ ψ ψ ⎟t e s ⎟db Ad = ⎜ ⎜ 40 λ f c′ cb + K tr ⎟⎜⎝ db ⎟⎠ ⎛ 3 60, 000 psi (1.0)(1.0)(1.0) ⎞ Ad = ⎜ ⎟db 2.5 ⎝ 40 (1.0) 4000 psi ⎠ = 28.5db = 28.5 in. > 12 in. where tion ȥt = casting position; n more re than 12 in. of o fresh resh conȥt = 1.0 becausee not orizontal reinfor emen crete is placed be below horizontal reinforcement cto ȥe = coating factor; ȥe = 1.0, becausee bbars are uncoated or; ȥs = bar size factor; ȥs =1.0 because bars are larger than 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. However, the expression: cb + K tr db must not exceed 2.5. Provided length: B/2 – twall/2 – 3 in. cb + K tr 3.5 in. + 0 = = 3.5 db 1.0 in. Use maximum value of 2.5 Ɛavail. = (10 ft)(12 in./ft)/2 – 12 in./2 – 3 in. = 51 in. Ɛavail. = 51 in. > Ɛd = 28.5 in. OK Therefore, the footing is wide enough to use straight bars for development and does not require hooks at both ends. CHAPTER 11—FOUNDATIONS 443 Step 6: Earthquake requirements Earthquake load effects 13.2.3.2 The foundation is in SDC D therefore, ACI 318-14, 6HFWLRQPXVWEHVDWLV¿HG 18.13.2.1 18.13.2.3 18.13.2.4 The requirements listed in 18.13.2 for structural ZDOOVPXVWEHVDWLV¿HGLIFDOFXODWLRQVVKRZWKDW uplift occurs: D 9HUWLFDOUHLQIRUFHPHQWRIVWUXFWXUDOZDOOV resisting forces induced by earthquake effects must extend into the footing and must be fully developed for tension at the interface. (b) Boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with 18.7.5.2 through 18.7.5.4 provided below the top of the footing. This reinforcement must extend into the footing and be developed a length equal to the development length, calculated for fy in tension, of the boundary element longitudinal reinforcement. This condition does not apply for this problem. (c) Where earthquake effects create uplift forces in boundary elementss of spe special structural walls, ÀH[XUDOUHLQIRUFHPHQWPXVWEHSURYLGHGLQWKHWRS PHQW of the footing to res resist actions ons resulting from the design factored lo load combinations, mbinations, and must be less than required This cond condition ed by 7.6.1 6.1 or 9.6.1. Th does not apply fo for this prob problem. Foundationss 18.13.2 Step 7: Detailing Fig. E2.5—Continuous footing reinforcement. 444 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Foundation Example 3: Design of a continuous footing with an out-of-plane moment Design and detail a continuous footing, founded on stiff soil, supporting a 12 in. thick bearing wall, founded on stiff soil, and VXEMHFWWRORDGLQJWKDWLQFOXGHVDQRYHUWXUQLQJPRPHQW7KHERWWRPRIWKHIRRWLQJLVIWEHORZ¿QLVKHGJUDGHUHIHUWR)LJ(Given: Wall load— Service dead 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 fcǫ SVL Steel yield strength fy = 60,000 psi 1RUPDOZHLJKWFRQFUHWH3 Density of concrete = 150 lb/ft3 Soil data— qall = 4000 psf qu,permitted = 6000 psf Density of soil = 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. 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. 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 mixture options within ACI 301, such as admixtures 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 YHUL¿HGWKURXJKFDOFXODWLRQVLQ6WHS)RRWLQJ 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. 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 PHGDQGWK)RRWLQJZLGWKLVDVVXPHGDQGWKHQYHUL¿HGWKURXJK 13.3.1.1 ons m calculations. Iterations may be needed. Try B = 7 ft 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 resisting moment (MR) is compared to the resulting overturning moment (MOTM). Commonly, engineers require MR•MOTM to consider a footing stable. Weights on bearing soil below footing: 15 in. Weight of footing: W ftg = (Weight of soil above footing: Wsoil = ⎜ Weight of concrete wall pier pier: Wconc. pier = ⎜ ead load: Total vertical dead ™3 = (0.19 ™ (0 ksf)(7)(7 ft) + (0.18 (ksf) (7 ft – 1 ft) + (0.26 ksf)(1 f)(1)(ft)) + 15 kip/ft = 22.67 kip/ft ft + 15 ki kip/ft = 17.7 kip/ft 12 in./ft 3)(0.15 kip/ft) = 0.19 ksf ⎛ 36 in. − 15 in. ⎞ 3 (0.10 kip/ft) = 0.18 ksf ⎝ 12 in./ft ⎟⎠ in − 15 in. ⎞ ⎛ 36 in. 3 ⎜(0.15 kip/ft) = 0.26 ksf ⎝ 12 iin in./ft /t ⎠ IUR WWRPRIIRRWLQ ORFDWLRQ 9HUWLFDOGLVWDQFHIURPERWWRPRIIRRWLQJWRORFDWLRQ H = 3 ft + 1 ft = 4 fft of applied lateral wind shear. The overturning moment, MOTM, is measured at base of footing. The 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)(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 kip/ft)(7 ft/2) = 61.8 ft-kip/ft IWNLSIW! IWNLSIW IWNLSIWOK To ensure footing stability, the following inequality PXVWEHVDWLV¿HG MR > 1.5MOTM CHAPTER 11—FOUNDATIONS 447 Step 5: Calculate soil pressure 13.3.1.1 Service loads The maximum 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 …

… FRQ¿QLQJUHLQIRUFHPHQWIDFWRU ȥr = 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: ⎧⎪0.0005 f y db Ɛsc = larger of ⎨ ⎪ ⎩12 in. 0.0005(60,000 psi)(0.875 in.) = 26.25 in. 12 in. Use 27 in. = 2 ft 3 in. long lap splice. 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 factored forces are transferred to the footing by bearing and through dowels. 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(0.85fcǫA1) A1 is the bearing area of the column and A2 is the area of the part of the supporting footing that is geometrically similar to and concentric with the loaded area. Center of column is located 1 ft-0 in. from the end of footing and 3 ft-11 in. of the combined footing long 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 vertical to 2 horizontal. Check if A2 ≤ 2.00 , wh where A1 A2 = A1 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 Ɛdc, which is the larger of the two following expressions: A dc ⎧ fyψr db ⎪ = ⎨ 50λ f c′ ⎪(0.0003 f ψ d) y r b ⎩ (0.65)(0.8 psi)(18 in.) 2 φBn = (0.65)(0.85)(4000 576 in.2 (18 in.) 2 OK Use one No. 6 bars in each corner of the column. As = (4)(0.44 in.2) = 1.76 in.2 > As,dowel = 1.62 in.2 OK A dc = (60,000 psi)(1.0) (50) 4000 psi (0.75 in.) = 14.2 in. Controls Ɛdc = 0.0003(60,000 psi)(1.0)(0.75 in.) = 13.5 in. where ȥr FRQ¿QLQJUHLQIRUFHPHQWIDFWRU ȥr EHFDXVHUHLQIRUFHPHQWLVQRWFRQ¿QHG The footing depth h must satisfy the following inequality: h•Ɛdc + 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. + 1.128 in. + 3 in. = 24.5 in., say, 25 in. hreq'd = 25 in. < hftg,prov. = 30 in. OK 3 in. cover (refer to Fig. E5.10) CHAPTER 11—FOUNDATIONS 487 Check development length of dowel reinforcement into the column. The length of dowels in the column is the greater of the development length and lap splice length. Assume that 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 ⎧ fyψr db ⎪ = ⎨ 50λ f c′ ⎪(0.0003 f ψ d) y r 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 ȥr FRQ¿QLQJUHLQIRUFHPHQWIDFWRU ȥr EHFDXVHUHLQIRUFHPHQWLVQRWFRQ¿QHG 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 splic Use 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. Foundationss ⎧⎪0.00 0.0005 f y db Ɛsc = larger of ⎨ ⎩⎪12 in. 488 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) 38800 Country Club Drive Farmington Hills, MI 48331 USA +1.248.848.3700 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 1 VOLUME 2 BUILDING EXAMPLE RETAINING WALLS STRUCTURAL SYSTEMS SERVICEABILITY STRUCTURAL ANALYSIS STRUT-AND-TIE MODEL DURABILITY ANCHORING TO CONCRETE ONE-WAY SLABS TWO-WAY SLABS BEAMS DIAPHRAGMS 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 Order information: ACI documents are available in print, by download, on CD-ROM, through electronic subscription, or reprint and may be obtained by contacting ACI. Most ACI standards and committee reports are gathered together in the annually revised ACI Manual of Concrete Practice (MCP). American Concrete Institute 38800 Country Club Drive Farmington Hills, MI 48331 USA +1.248.848.3700 Managing Editor: Khaled Nahlawi Staff Engineers: Daniel W. Falconer, Matthew R. Senecal, Gregory M. Zeisler, and Jerzy Z. Zemajtis Technical Editors: Shannon B. Banchero, Emily H. Bush, and Cherrie L. 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&KXUFK+DUWODQGDQG0,DQGDGHGLFDWHGVXSSRUWHUDQGIROORZHURIWKH0LFKLJDQ6WDWH6SDUWans 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 EXLOGLQJVDQGUHODWHGVWUXFWXUHVLQKLVWVLNTLRQLVDQFUDNHRXQGQDSUL 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[DPSOHVDUHEDVHGRQDILFWLWLRXVVHYHQVWRU\EXLOGLQJORFDWHGLQ... 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 UHIHUHQFHWKURXJKRXW... design approach, and a three column format with the code provisions within the selected ACI 318-14 chapter. All chapters were developed by ACI staff engineers under the auspices of the ACI Technical Activities Committee (TAC). TAC approves the support of Dirk Bondy and Kenneth Bondy who provided three sections of the design examples; and Michael Mota and Ilyan Mavrin, who authored the post-tensioned slab and Kaled H. Alatrash and Jose Cohen Moses; TAC also appreciates the support of Dirk Bondy and Kenneth Bondy who provided three example for... The Reinforced Concrete Design Handbook contains 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). 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 Handbook1VXSEDVDQGUHOHYDQWWRSLFVLQWRWKHQDFWXDO... available for free download from the following ACI webpage links: Keywords: DQFRELLQJ... frames; pile; pile caps; post-tensioning; punching shear; retaining wall; shear strength; seismic; slabs; splicing; stiffness; structural analysis; structural systems; strut-and-tie; walls. Khaled Nahlawi Managing Editor 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. 12 12.6—Detailing, p. 13 12.7—Summary, p. 13 12.8—Examples, p. 15 CHAPTER 13—SERVICEABILITY 13.1—Introduction, p. 115 13.2—Shrinkage and temperature reinforcement—nonprestressed, p. 119 13.3—Shrinkage and temperature reinforcement—posttensioned, p. 119 ... 13.7—Permissible stresses at transfer of prestress, p. 120 13.10—Examples, p. 121 CHAPTER 14—STRUT-AND-TIE MODEL 14.1—Introduction, p. 159 14.2—Concept, p. 159 14.3—Design, p. 160 14.5—Ties, p. 162 14.6—Nodes, p. 163 14.7—Usual calculation steps and modeling considerations, 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. 213 ... 15.5—Discussion on anchors resisting tension, p. 216 15.6—Discussion on anchors resisting shear, p. 217 ... 15.8—Examples, p. 218 12.1—General A retaining or cantilever 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 footing (heel and toe). The stem is monolithic with the footing and tied to the soil through the retained soil. The retaining wall is designed to resist the overturning moment by the weight of the soil above the heel. Retaining walls can be restrained against lateral movement at the top, which is also sometimes referred to as a basement wall. Retaining walls with a key may be provided at the bottom of the base to increase sliding resistance (Fig. 12.1a) (b) and (c)). Reinforced concrete cantilever walls are designed to withstand horizontal soil pressures and overturning forces. Lateral loads, such as lateral earth pressure, water pressure, surcharge loads, or earthquake forces are assumed to be resisted by the retaining wall structure. 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 the retaining wall requirements. The design of retaining walls is complex; the retaining wall is designed as a free standing structure, i.e., Safety factors against overturning, sliding, and bearing (Fig. 12.1b) is when 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 J6•DJDLQVWRYHUWXUQLQJLQFOXGLQJLQ E J6•DJDLQVWVOLGLQJLQFOXGLQJ F J6•DJDLQVWEHDULQJFDSDFLW\LQFOXGLQJ These factors are usually provided by the geotechnical engineer and are not in the scope of this guide. 7KH UHLQIRUFHG FRQFUHWH VWHP DQG EDVH design strength must be at least equal to the factored moments and shears determined from the analysis. Figure FGHSLFWVWKHGHVIRUPVRI... 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 beyond 20 ft, buttresses or counterforts are usually provided. Counterforts are normally preferred as they create clean and an unobstructed view from the stem (Fig. 12.1e). (a) (b) (c) (d) 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) (c) Fig. 12.1d—Wall with shear key. 12.2—Design limits 12.2.1 Wall design—Minimum wall thickness is not specified in ACI 318. 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 at base from between 7 to 12 percent of the retained earth height h, as shown in Fig. 12.2a. The stem thickness can be tapered to a thinner top, resulting in a lower moment and less material, provided the stem is of sufficient thickness to resist shear. The base thickness selected should be 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 equal to the stem thickness, and not less than 12 in. (refer to Fig. 12.2a). 7KH IRRWLQJ PXVW SURMHFW SURMHFWLQJ XQGHUQHDWK WKH WRH... footing. A key is designed to support the entire weight. It is designed as a cantilever... sure. HFWLRQ DQG 7KH ... The toe length can be estimated as one fourth or one sixth of the overall length of the footing (refer to Fig. 12.2a). orizontal force applied to the stem exceeds the If the horizontal similar allowable frictional force of the footing, the retaining 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 footing. The key should be proportioned such that its width is at least twice its depth to develop the strength of the key in VKHDU UDWKHU WKDQ IOH[XUH ... The critical sections for calculating shear strength for the toe and heel are taken at a distance d from the front and back faces of the wall stem. 12.2.3 Buttresses and counterforts—Buttresses or counterforts are uncommon and are not addressed in this Handbook. If buttresses or counterforts are used, the design of vertical 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. 12.1e—Cantilever wall with counterforts or buttresses. Fig. 12.2a—Forces Fi 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 FDQWLOHYHUHG... GHWHUPLQHG... of the retaining wall structure between individual buttresses or counterforts. 12.3—Applied forces Cantilever retaining walls are designed to resist axial loads, including the weight of the stem and frictional forces due to VORSHGEDFNILOO... vertical forces, surcharge 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 = CaUsh (12.3a) P = CPUsh (12.3b) where Ca and CP 7KH FRHIILFLHQWV... Ca = ... lateral pressure of sloped backfill... The active lateral soil pressure coefficient Ca varies 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: 1. The surcharge load is located immediately behind the wall. To calculate the force on the wall, an equivalent height of soil is determined from h2 = ws/Us, where ws is the VXUFKDUJH... DQG Us is the soil unit weight (pcf). The horizontal 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 on or load behind a cantilever wall (equivalent soil height of soil to the stem height ght as shown in Eq. (12.3c) (refer to Fig. 12.3(a)). ... The surcharge load is located at a distance from the wall. The surcharge load will increase the pressure on the stem if a line at 45 degrees from the edge of the surcharge intersects the stem above the heel. The American Association of State Highway and Transportation Officials (AASHTO) provides a graphical procedure to determine lateral pressure due to point load and line load applied at a distance from the stem. ... 12.4—Design strength 12.4.1 Flexure—The stem and footing are designed for flexure ... the wall thickness. American Concrete Institute – Copyrighted © Material – www.concrete.org ... Retaining Walls 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 ... ... 12.6—Detailing Many engineers specify weakened plane to form contraction joints at a wall spacing of 20 to 30 ft ... The toe reinforcement ... Reinforcement type Deformed bars Welded wire reinforcement Bar/wire size ... Minimum vertical ... Minimum horizontal ... 12.7—Summary ... 1. Determine wall height. 2. Estimate stem thickness. 3. 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 Clear wall against overturning, sliding, and bearing pressure. ... 9. Design the stem for flexure and shear. 10. Design the heel and toe for flexure and shear. 11. Provide drainage systems ... 12. Detail shrinkage and temperature reinforcement ... 13. Detail development and lap lengths. 14. Detail reinforcement per ACI 318 requirements; reinforcement ... 15. Provide drainage system. REFERENCES A American Association of State Highway and Transportation Officials, AASHTO Highway Bridge ... Foundation Analysis and Design, The McGraw-Hill Companies, Inc., 1996. American Concrete Institute – Copyrighted © Material – www.concrete.org 14 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Retaining Walls 12.8—Examples Retaining Wall Example 1: Basement wall with slab ... ... The eight-story building has a 10 ft high basement wall ... supported on a 5 in. ledge ... American Concrete Institute – Copyrighted © Material – www.concrete.org 15 Retaining Walls ... The basement wall is assumed free and has the following soil data obtained from the geotechnical report: soil unit weight 120 psf, angle of internal friction, 30 degree, ... Soil data— Us = 120 pcf; qall = 2500 psf Concrete— fc' = 4500 psi normalweight concrete fy = 60,000 psi ... = 60,000 psi. ... Step 1: Material requirements 7.2.2.1 The value of concrete compressive strength at a 19.2.1.3 ... is 4500 psi per Table 19.3.2.1 ... The designer determines the durability classes. Please see Chapter 4 of this Handbook for an in-depth discussion of the categories and classes ... Class F2 are listed Category S Class S0 ... Class C1 ... w/cm ≤ 0.45 Maximum fc' ... Conclusion: (a) The most restrictive minimum concrete compressive strength is 4500 psi, and (w/cm)max is 0.45. ... Step 2: Equivalent lateral pressure The geotechnical report provides the equivalent p = 40 pcf ... Step 3: Applied forces ... 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): ... Fig. E1.3—Applied plied force forces on basement wall. Summation of vertical forces less base weight of P2 = 300 lb ... ... Fig. E1.4—Moment and shear diagrams. American Concrete Institute – Copyrighted © Material – www.concrete.org 17 Retaining Walls CHAPTER 12—RETAINING WALLS 18 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ... Step 4: Wall design ... 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 ... 20.6.1.3.1 Concrete is cast against and permanently in contact with ground ... Therefore, use 2 in. cover ... 22.2.2.1 The concrete compressive strain at which ultimate moment is developed is 0.003 ... 22.2.2.2 The tensile strength of concrete in flexure is neglected ... 22.2.2.4.3 Choose a equivalent rectangular compressive stress distribution ... The Code allows the use of an equivalent rectangular compressive stress distribution of 0.85fc' ... Check using Tables 21.2.2 ... ... Mu = 7.5 ... Try No. 4 at 16 in. on center ... As = 0.15 in.² ... ... Therefore, the section is tension controlled, and ... φ = 0.9 ... Because Vu ... To answer the question, the tensile strain ... 22.5.5.1 Shear strength ... φVc = (0.75)(2)(√4500 psi)(12 in.)(7.625 in.) = 9200 lb Shear strength reduction factor: 7.5.1.1 ... OK ... Vc is adequate ... 11.7.2.3 ... 11.7.3 Maximum spacing of vertical bars ... 24.4.3.2 The minimum ratio of shrinkage and temperature reinforcement ... Step 5: Detailing ... Footing ... 13.3.1 Foundation design ... Shear ... The wall is modeled as free standing in this example ... American Concrete Institute – Copyrighted © Material – www.concrete.org 23 Retaining Walls Retaining Wall Example 2: Cantilever retaining wall Design a normalweight reinforced concrete cantilever retaining wall ... The wall retains 15 ft high soil ... The geotechnical report provides: soil unit weight of 120 pcf, angle of internal friction of 30 degrees, coefficient of friction of 0.5 ... Given: Soil data— Us = 120 pcf; qall = 3000 psf ... Concrete— fc' = 4500 psi normalweight concrete; fy = 60,000 psi ... Step 1: Material requirements 7.2.2.1 The mixture proportion must satisfy the durability requirements of Chapter 19 and structural strength requirements. ... Step 2: Equivalent lateral pressure The geotechnical report provides ... p = 38 pcf ... Step 3: Preliminary cantilever wall data ... Estimating stem thickness ... Estimating base thickness ... Heel length ... Toe length ... American Concrete Institute – Copyrighted © Material – www.concrete.org ... Step 4: Applied forces ... The vertical weight is the self-weight of the retaining wall (stem and base) and the weight of the soil above the heel ... Stem wall: P1 = (150 pcf)(1 ft)(14.5 ft) = 2175 lb ... Base: P2 ... Soil: P3 ... Total vertical load ... Moments ... Summation of moments about the front of toe ... 12.4.2 Overturning ... MOTM = (4829 lb)(17.5 ft)/3 = 28,974 ft-lb ... FS = 2.89 > 2.0 OK ... Sliding ... FS = 1.55 > 1.5 OK ... Step 5: Soil pressure ... The aforementioned determined unfactored forces ... e = ... qmax = 2652 psf < qall = 3000 psf OK ... qmin = 593 psf ... no uplift ... Step 6: Stability requirements ... Step 7: Stem design ... Flexure ... 22.2.1.2 ... Mu = 1.6 ... Check if the section is tension controlled ... 22.2.1.2 To answer the question, the tensile strain in reinforcement ... εt = εc (d − c)/c ... OK 22.2.1.2 εt = εc (d – c)/c where: c = a and a = 1.31As derived previously. β1 0.003 (12.5 in. − 1.25 in.) = 0.027 1.25 in. εt = 0.027 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQGDFFRUGLQJ\εt = Fig. E2.5—Strain distribution across stem. 7.4.3.2 Shear The closest inclined 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 shear reaching approxiater load mately d from the face of the base. The lateral ant r between the fa applied to the cantilever facee of th the om the face is transferred nsferred base and point d 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) = 2λ f c′ bw d with Vs = 0 21.2.1 Shear strength reduction factor: 7 7KHUHIRUH▯Vc is: φVc = (0.75)(2)(4500 psi)(12 in.)(12.5 in.) = 15,093 lb 7.5.1.1 ,V▯Vn• VuVDWLVILHG" ▯Vn ▯Vc = 15,093 lb > Vu = 6689 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 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. E2.6). Fig. E2.6—Force on heel. 21.2.1 Shear strength reduction factor: 7.4.3.2 ▯The critical section for shear strength is taken at a distance d from the bottom of the stem: Vn = (Vc + Vs) = 2λ f c′bw d with Vs = 0 d = tbase – cover – db/2 Vc = (2)(4500 psi)(12 in.)(11.5 in.) = 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 ▯ c = (0.75)(18,515 75)(18)(,51 lb) = 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.⎞ ⎤ ⎛ 1 2 ⎡(150 pcf p f) ⎛ 5.5 5 5 ft − Vu = 1.2 ⎢⎜ (1.25 ft) ⎥ ⎝ 12 ⎦ ⎣ 11.5 in.⎞ ⎤ ⎛ + 1.6 ⎢(110 pcf)(16.75 ft) ⎛ 5.5 ft − (⎜ 12 ⎠ ⎥⎦ ⎣ Vu = 14,411 lb 7.5.1.1 ,V▯Vn•VuVDWLVILHG" 22.5.5.1 φVn = φ(Vc + Vs) = φ2λ f c′bw d ▯Vn ▯Vc = 13,886 lb < Vu = 14,206 lb To increase Vn, two options are possible: 1. Reduce heel length 2. Increase base thickness Reducing the heel length will reduce the weight of the retained earth on the heel and consequently 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 φVn 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: φVc = (0.75)(2)(4500 psi)(12 in.)(14.5 in.) = 17,508 lb The heel thickness increase will result in change to the calculations performed in Steps 4 through 8 above. 14.5 in.⎞ ⎤ ⎡ Vu = 1.2 ⎢(150 pcf) ⎛ 5.5 ft − ⎜⎠ (1.5 ft) ⎥ ⎝ 12 ⎦ ⎣ 14.5 in.⎞ ⎤ ⎛ + 1.6 ⎢(110 pcf)(16.75 ft) ⎛ 5.5 ft − ⎝ 12 ⎠ ⎦⎥ ⎣ Vu = 13,811 lb Below is a summary of the recalculated values. Shear, revised 9HUWLFDOZHLJKWRIZDOODQGVRLO Balancing moment: 2YHUWXUQLQJPRPHQW Safety factor against overtur overturning: Force causing sliding: ing: Resisting force: Safety factor against gain sliding: ding: Soil pressure: Stem: Factored moment: Required tension reinforcement: Design shear strength: Factored shear: Heel: Design shear strength: Factored shear: ™P = 15,008 lb MBAL = 83,703 ft-lb MOTM M = 28,974 ft-lb FS = 2.89 482 lb H = 4829 ȝ3 ȝ OE OE Hfr ȝ3 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 Vn = 15, 15,903 lb Vu = 6492 lb ▯Vn = 17,508 lb Vu = 13,622 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 7KHKHHOLVVXEMHFWWRÀH[XUHFDXVHGE\WKHVXSHUimposed 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 included in WKHFDOFXODWLRQIRIÀH[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 – Heel critical moment section. 5.3.1 5.3.8 A load factor of 1.2 is used for the concrete selfZHLJKWDQGIRUVRLOEDFN¿OO 22.2.1.1 Setting C = T Mu = (1.2)(150 pcf)(1.5 ft)(5.5 ft)2/2 + (1.6)(110 pcf)(16.5 ft)(5.5 ft)2/2 Mu = 48,000 ft-lb = 576,081 in.-lb (0.85)(4500 psi)(12 in.)(a) = As (60,000 psi) a= 7.5.2.1 7.5.1.1 9.6.1.2 As (60,000 psi) = 1.31As 0.85(4500 psi)(12 in.) a⎞ ⎛ M n = f y As ⎜ d − ⎟ ⎝ 2⎠ 1.31As ⎞ ⎛ (60 60, 000 ps psi M n = (60, psi)) As ⎜ 14.5 in. − ⎟ ⎝ 2 ⎠ ▯Mn•Mu Mu = 576 in.-kip p calculated ated 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 = 0.76 in.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 structurÀ 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 fcȩ SVL 21.2.2 Check if the section is tension controlled and the XVHRI▯LVFRUUHFW 22.1.2 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 in.2 > As,min = 0.58 in.2 As,prov. = 0.79 in.2 > As,req'd = 0.76 in.2 OK c= (1.31)(0.79 in.2) = 1.25 in. 0.825 American Concrete Institute – Copyrighted © Material – www.concrete.org εt = εc (d − c) c where: c = a and a = 1.31As β1 35 0.003 (14.5 in. − 1.25 in.) = 0.0318 1.25 in. İt = 0.0318 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQGDFFRUGLQJ\εt = 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. ▯Assume Vs = 0 The critical section for shear strength is taken at a distance d from the stem. φVn = φ(Vc + Vs) = φ2λ f c′bw d Vn = (2) 4500 psi(12 in.)(14.5 in.) = 23,344 lb 7KHUHIRUH▯Vn: ▯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 ⎞ 4 = (2652 psf − 593 psf) ⎜ ⎟ + 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 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 lb 7.5.1.1 ,V▯Vn•VuVDWLVILHG" ▯Vn = 17,508 lb > Vu = 4922 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 ⎞ stem , L = (2652 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 toe self-weight; a load factor of 0.9 is used: Resultant moment: 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 = 0.85fcȩba and T = Asfy Mu2 = 1.6 [(2096 psf (2.5 ft)2/2 + (2652 psf – 2096 psf)(2.5 ft)2 (2/3)] = 14,188 ft-lb Mu1 = 0.9(150 pcf)(1.5 ft)(2.5 ft)2/2 = 633 ft-lb Mu,req'd = Mu2íMu1 = (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= As (60,000 psi) = 1.31As 0.85(4500 psi)(12 in.) 7.5.2.1 a⎞ ⎛ M n = f y As ⎜ d − ⎟ ⎝ 2⎠ 1.31As ⎞ ⎛ φM n = (0.9) (0.9)((0.9)(60, 000 psi) As ⎜ 14.5 in. − ⎟ ⎝ 2 ⎠ 21.2.1(a) 6WUHQJWKUHGXFWLRQIDFWRUIRUÀH[XUH IDFWRU 7.5.1.1 ▯Mn•Mu Mu = 2163 in.-kip kip calculated ulated above. 1.31As ⎞ ⎛ 9)(60 ksi) As ⎜14.5 14 5 in in. − (0.9)(60 = 163 in. − kip ⎝ 2 ⎠ As,req'd n2 q'd d = 0.21 in. 2 As,provid rovid d = 0.79 in. /ft > As,req'd OK Solving for As: Minimum required reinforcement is the greater of (a) and (b): 9.6.1.2 (a) As,min ≥ 3 f c′ fy bw d As , min = 3 4500 psi (12 in.)(14.5 in.) = 0.58 in.2 60,000 psi ⇒ As = 0.58 in.2 Equation (9.6.1.2(a)) controls, because concrete compressive strength fcȩ = 4500 psi. 21.2.2 Check if the section is tension controlled assumpWLRQDQGWKHXVHRI▯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. E2.5): Provide same reinforcement area as in the stem: No. 8 at 12 in. on center (refer to Fig. E2.10). c= (1.31)(0.79 in.2) = 1.25 in. 0.825 American Concrete Institute – Copyrighted © Material – www.concrete.org 22.2.1.2 εt = εc (d − c) c where: c = a and a = 1.31As β1 εt = 0.003 (14.5 in. − 1.25 in.) = 0.0318 1.25 in. İt = 0.0318 > 0.005. Section is tension controlled. 7KHUHIRUHDVVXPSWLRQRI▯LVFRUUHFW Fig. E2.10—Toe reinforcement. Step 10: Minimum transverse reinforcement Stem 11.6.1 ars or smaller will be used for Assume that No. 5 bars infor transverse wall reinforcement. Per Table 11.6.1, ȡmin = 0.002 ve reinforcement einforcement equ ally Distribute transverse equally nt aand back ack face of the stem em wall. between the front Ass,min 12 in.)(1 in.)(15 in.) = 0.36 in.2 in = (0.002)(12 2 2 (s,min (A min)fr front = (As,min)backk = 0.36 in. /2 = 0.18 in. Try No No. 5 at 18 in. in. on center ⎛ 12 in.⎞ As , prov. = 00.31 in.2 ⎜ ⎟ = 0.21 in.2 ⎝ 18 in. ⎠ > As , req ' d = 0.18 in.2 /face Provide vertical reinforcement at front face to support the transverse wall reinforcement. 24.4.3 Base Assume that No. 5 bars will be used for temperature and shrinkage 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 peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development RIUHLQIRUFHPHQWLQÀH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLnates, 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: 6XI¿FLHQWGLVWDQFHWRGHYHORSWKHWRHGHYHORSment length in the toe section. nforcement in the stem to 2. Extension of toe reinforcement evelopm length. satisfy the required development pm length gth in the ste ust be Note: the development stem must m reinforcemen equired checked againstt th the stem reinforcement required nd the larger rger length cont ols. splice length and controls. 25.4.2.1 ng ℓd is the greater of Eq. Development length q. 4.2 off ACI 318-14 and 12 in.: (25.4.2.2) or (25.4.2.3) 25.4.2.2 ⎛ f yψt ψe ⎞ 1. A d = ⎜ ⎟ db ⎝ 200λ f c′ ⎠ 25.4.2.3 ⎛ ⎞ 3 f y ψt ψeψ s ⎟ 2. A d = ⎜ ⎟ db ⎜ 40 λ f c′ cb + K tr ⎟ ⎜⎝ db ⎠ In this example Eq. (25.4.2.3a) will be used. 25.4.2.4 25.4.2.3 ȥt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ȥe = coating factor; uncoated ȥs = bar size factor; No. 7 and larger cb = spacing or cover dimension to center of bar, whichever is smaller ȥt = 1.0, because not more than 12 in. of concrete is placed below bars ȥe = 1.0, because bars are uncoated ȥs = 1.0, because No.7 bars are used Ktr = transverse reinforcement index It is permitted to use Ktr = 0. But the expression: cb + K tr ≤ 2.5 db 2.5 in. + 0 = 2.5 ≤ 2.5 1.0 in. must not be taken greater than 2.5. Therefore, use 2.5 in Eq. (25.4.2.3a) American Concrete Institute – Copyrighted © Material – www.concrete.org 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)(1.0) ⎞ Ad = ⎜ ⎟ (1.0) 2.5 ⎝ 40 (1.0) 4500 psi ⎠ = 26.8 in., say, 27 in. > 12 in. OK ℰdIWLQWRHOHQJWK íLQFRYHU 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 controls Splice The maximum bar size is No. 8, therefore splicing is permitted. The required development 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 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 (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) 1.5db (c) 4/3dagg (a) 1.5 iin. 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 = (2.67 lb/ft/ft)(19.5 ft) = 52.1 lb/ft Fig. E2.11—Retaining wall reinforcement. American Concrete Institute – Copyrighted © 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 to a level where smaller bar diameters, larger bar spacing, or both can be used for 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 = CaŪho3/6 Mu = (1.6)(0.271)(110 pcf)(16.5 – 4.5 ft)3/6 = 13,736 ft-lb The required reinforcement area: As,req'd = 0.25 in.2/ft Use a bar size such that 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 &KHFNLISURYLGHGUHLQIRUFHPHQWDUHDLVDWOHDVWKH 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 requ d reinof provided 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 leng 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)(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. 8 to splice 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. Use overlap length of 30 in. American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls CHAPTER 12—RETAINING WALLS 42 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. 8 = 4.5 ft Total length of No.5 =16.5 ft – 4.5 ft + 2.25 ft =14.25 ft Total weight = (2.67 lb/ft/ft)(4.5 ft) + (1.043 lb/ft/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 ¿QDOHDUWKOHYHODWWKHWRHRIWKHZDOO\LQJ(7KHFRQFUHWHPL[WXUHPXVWVDWLVI\GXUDELOLW\DQGVWUHQJWKUHTXLUHPHQWV7KHJUDGHRIUHLQforcing 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. $VVXPHWKDWWKHFDQWLOHYHUZDOOLVQRWEXEMHFWHGWRDQ\WKHUORDGDQGQRVHLVPLFIRUFHVDUHWREHLVWLOQHLVIWLQEHORZWKH¿QLVKHGJUDGH Given: Soil data— Ûs = 120 pcf ࢥGHJUHHV ȝ ¡ GHJUHHVORSH qall = 3000 psf Concrete— ʒa = 1.0 normalweight concrete fy = 60,000 psi Cantilever wall— h1 = 12 ft h2 = 3.5 ft Fig Cantilev retaining wall. Fig. E3.1—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 tthat the concrete mixture shall be in accordance with ACI 301 and providing the exposure accordan FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLV¿HG Based on durability and strength requirements, and experience with local mixtures, the compressive VWUHQJWKRIFRQFUHWHLVVSHFL¿HGWRDWEHDW 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¿IWKHGLWLRQRI³Foundation Analysis and Design, The McGraw-Hill Companies, Inc., 1996. Figure E3.2 shows the variables used in the example. Fig. E3. E3.2—Determine Determine retaining wall dimensions. RIZ V 2YHUDOOKHLJKWRIZDOOLV h = h1 + h2 = 12 2 ft + 3.5 ft = 15.5 ft m tthickness: ess: (0.07 to 0.1)h Estimating stem 0.12)h ess: (0.07 to 0.1 Estimating base th thickness: 0.1)h,, but at least 12 in. a IW IWVD\IW▯LQ tstemaIW 0 (15.5 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. /HQJWKRIEDVHRKHHO WR h /HQJWKRIWRHab/4 to b/3 Heel length: bheel = bbase – btoe – tstem bbase ~ 0.5(15.5 ft) = 7.75 ft, say, 8 ft 3 in. btoe ~ (8 ft 3 in.)/4 = 2 ft, say, 2 ft 3 in. bheel = 8.25 ft – 2.25 ft – 1.25 ft = 4.75 ft &DOFXODWHWKHVRLOKHLJKWGXHWRVORSHGEDFN¿OODW heel end: hȝ IWIW WDQƒ 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 Acting force forces on retaining wall. tem Soil height at stem: as end:: Soil height at base hstemm = ((12 ft) + ((3 (3.5 5 ft)) – (1.5 ft) = 14.0 ft hstemm = ((16.34 ft)) – 1.5 ft = 14.84 ft 9HUWLFDOORDGV The vertical weight is the self-weight of 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: 9HUWLFDOFRPSRQHQWRIODWHUDOSUHVVXUH: Total vertical load: 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 (restoring moment): Stem wall: M1 = P1(btoe + tstem/2) Base: M2 = P2(bbase/2) Soil: M3 = P3(bbas▯Eheel/2) Sloped soil: M4 = P4(bbase – bheel/3) 45 P1 = (150 pcf)(1.25 ft)(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)2WDQƒ OE HV = P5 = {(33.8 pcf)(16.34 ft)2`VLQƒ OE ™P = 2625 lb + 1856 lb + 7980 lb + 239 lb + 784 lb = 13,484 lb M1 = (2625 lb)(2.25 ft + 1.25 ft /2) = 7547 ft-lb M2 = (1856 lb)(8.25 ft/2) = 7656 ft-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) = 6468 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 = CaŪsh2/2 where Ca is the Rankine's equation obtained from any soils textbook. ™MR = (7547 ft-lb) + (7656 ft-lb) + (46,833 ft lb) + (1596 ft-lb) + 6468 ft-lb) = 70,147 ft-lb H = (0.3)(120 pcf)(16.34 ft)2/2 = 4806 lb Horizontal component of lateral force: (H): Hh = HFRVį Hh OE FRVƒ 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 = (4733 lb)(16.34 ft)/3 = 25,779 ft-lb ¨M ™MR – MOTM ¨M = (70,417 ft-lb) – (25,779 ft-lb) = 44,638 ft-lb 7KHUHIRUHWKHEDVHJHRPHWU\LVVXI¿FLHQWWRUHVLVW overturning. Step 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. To calculate soil press pressure, the location of the vertice m cal resultant force must be determined. ltant to the fron The distance off th the resultant front face of stem: a= ∆M ∑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 Check if resultant falls within the middle third of the base. bbase 8.25 ft = = 1.375 ft > e = 0.83 ft 6 6 OK Therefore, there is no uplift. Maximum and minimum soil pressure: q1,2 = ∑P ∑ Pe ± A S q1,2 = 13,484 lb (13,484 lb)(0.83 ft) ± (8.25 ft)(1 ft) (8.25 ft) 2 /6 q1,2 = 1634 psf ± 987 psf q1 = qmax = 2621 psf < qall = 3000 psf no uplift q2 = 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 overturning: FS = ∑ M Res ≥ 2.0 M OTM FS = 70,147 ft-lb = 2.72 25,779 ft-lb FS = 2.72 > 2.0 OK Calculate the factor of safety against sliding: FS = ∑ μP ≥ 1.5 ∑H FS = (0.5)(13,484 lb) (33.8 pcf)(16.34 ft)(16.234 ft / 2))(cos(10°)) FS = 1.52 OK This calculation neglects the passive pressure against the toe compressive strength, fcշ 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 reinIRUFHPHQWPXVWEHःUVWFDOFXODWHGDQGFRPSDUHG 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 einforcem area as in the stem: 2 at 12 in. on center As,prov. rov. = 0.6 in. /ft No.7 a > A OK As,provided rovide s,req'd q'd d εt = εc (d − c) c where: c = a and a = 1.31 As β1 As ,min = 3 45 4500 psi (12 in.) (14.5 in.) = 0.58 in.2 60,000 psi As,prov'd = 0.6 in.2 > As,req'd = 0.17 in.2 c= OK (1.31)(0.6 in.2) = 0.95 0.825 εt = 0.003 (14.5 in. − 0.95 in.) = 0.043 0.95 in. εt = 0.043 > 0.005 Section is tension controlled American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 56 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Fig. E3.10—Toe reinforcement. Step 10: Minimum transverse reinforcement Stem 11.6.1 Assume that No. 5 bars or smaller bar will be used for transverse reinforcement. Per Table 11.6.1, d,min = 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 the free fron front and back face of the stem wall. (As,min (s,min)backk = 0.36 in.2/2 = 0.18 in.2 min)fr front = (A Use No. No 5 spaced ced at 18 in. i o.c. ⎛ 12 iin.⎞ nn.2 ⎟ As , prov = 0.21 in.2 rov . = (0.31 in. ⎝ 18 in.⎠ > As , req ' d = 0.18 0 in.2 /face Use No. o. 5 at 18 in in. on center at each face placed horizontally horizontally. 24.4.3 Provide vertical reinforcement at front face to support the transverse wall 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 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 of restraining concrete. In application, 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 RIUHLQIRUFHPHQWLQÀH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLQDWHV, 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: PPRGDWHWKHGHYHO6XःFLHQWWGLVWDQFHWRHDFFRPPRGDWHWKHGHYHOopment length in the toe section. 2. Extension of toe reinfo reinforcement in the stem to d dev satisfy the required development length. opm length in the ste Note: the development stem must be ice length and th checked againstt tthe splice thee larg larger length controls. 25.4.2.1 th ℓd is the greater of Eq (25.4.2.2) 25.4.2.2) Development length or (25.4.2.3) of ACI 318-14 and 12 in.: 25.4.2.2 ⎛ f yψt ψe ⎞ 1. A d = ⎜ ⎟ db ⎝ 200λ f c′ ⎠ 25.4.2.3 ⎛ ⎞ ⎜ 3 f y ψt ψeψ s ⎟ 2. A d = ⎜ ⎟ db ⎜ 40 λ f c′ cb + K tr ⎟ ⎜ ⎝ db ⎠⎟ In this example Eq. (25.4.2.3a) will be used. 25.4.2.4 25.4.2.3 ζt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ζe = coating factor; uncoated ζs = bar size factor; No. 7 and larger cb = spacing or cover dimension to center of bar, whichever is smaller 57 ζt = 1.0 because not more of 12 in. of concrete is placed below it. ζe = 1.0, because bars are uncoated ζs = 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)(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 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) ℓd < 2 ft length available OK Note: the development length in the stem must be checked against the splice length and the larger length controls. Splice The maximum bar size is No. 7, therefore splicing is permitted. velo 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 requ reinof provided reinforcement forcement area is less than 2. Therefore, per Table 25.5.2.1, splice is Type B and the splice length is the greater of (1.3ℓd) or 12 in. Splice Sp ce llength: ℓst = (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) 1.5db (c) 4/3dagg Therefore, extend No. 7 toe reinforcement 2 ft 9 in. into the stem. (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/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 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 No. 7 bars are extended a distance 5 ft into the stem. The factored applied moment at that level can be calculated from: Mu = CaŪho3/6 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 &KHFNLISURYLGHGUHLQIRUFHPHQWDUHDVDWLVःHVWKH 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 ℓd of the largerr bar and ℓstt 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 requ reinof provided reinforcement forcement area is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3ℓd) or 12 in. 25.4.2.3 Development of No. 5 bar ⎛ 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, 18 in. ℓt = 1.3(16.8 in.) = 21.9 in., say, 22 in. 25.5 25.5.2.2 Splice Compare development length of No. 7 to splice length of No. 5. (ℓd)No.7 = 24 in. > ℓst = 22 in. use 2 ft 0 in. Extend 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 Total length of No. 5 = 15.5 ft – 1.5 ft – 5.0 ft + 2.0 ft = 11 ft Total weight = (2.044 lb/ft/ft)(5.0 ft) + (1.043 lb/ft/ft)(11 ft) = 21.7 lb/ft Fig. E3.12—Retaining wall reinforcement (alternate solution). Conclusion: ⎛ 21.7 lb/ft ⎞ Alternate solution provides a saving of: 34.2 lb/ft – 21.7 lb/ft = 12.5 lb/ft or ⎜1 − ⎟ = 36.5 percent. ⎝ 34.2 lb/ft ⎠ American Concrete Institute – Copyrighted © Material – www.concrete.org Retaining Walls Step 14: Details 62 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Retaining Walls Example 4—Reinforced concrete cantilever wall retaining earth with surcharge load 'HVLJQDQRUPDOZHLJKWUHLQIRUFHGFRQFUHWHFDQWLOHYHUZDOOWRUHWDLQDQHDUWKHPEDQNPHQWKLJKDQGSDUWLDOOHYHOHDQGFRQFUHWHDUWKGHSWKUHDFKWKDWLVIURPGDGLQFRUSRUDWHG 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: VRLOXQLWZHLJKWRISFIDQJOHRIVRLOLQWHUQDOIULFWLRQRIGHJUHHVFRHIःFLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLORI and the allowable soil bearing pressure of 3000 psf. $VVXPHWKDWWKHFDQWLOHYHUZDOOLVQRWVXEMHFWHGWRDQ\RWKHUORDGDQGGWKHIURVWOLQHLVIWEHORZWKH¿QLVKHGJUDGH Given: Soil data— Ūs = 120 pcf 35 =▯degrees ȝ= 0.55 qall = 3000 psf Concrete— Ȝ a = 1.0 (normalweight concrete) fy = 60,000 psi Cantilever wall— h1 = 9 ft h2 = 2.5 ft Load— w = 360 psf – surcharge Fig. g E4.1—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. Please see Chapter 4 of this Handbook for an in-depth discussion of the categories and classes. $&,LVDUHIHUHQFHVSHFLःFDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFLःFDWLRQV Calculation By spe specifying that the concrete mixture shall be in accordance with ACI 301 and providing the exposure FODVVHV&KDSWHUUHTXLUHPHQWVDUHVDWLVःHG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFLःHGDV\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 geotechnical report provides 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 γs Retaining Walls Step 3: Preliminary cantilever wall data General criteria The preliminary retaining wall dimensions are determined from guidelines presented by Bowles LQWKH¿IWKHGLWLRQRI³Foundation 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 w wall required dimensions. hs = Retaining wall height: Equivalent wall height due to surcharge: Estimating stem thickness: (0.07 to 0.12)heqq Estimating base thickness: (0.07 to 0.1)heq, but at least 12 in. 63 36 psf 360 = 3 ft 120 12 psf h = h1 + h2 = 9 ft + 2.5 2 ft = 11.5 ft 11.5 ft + 3 ft = 14.5 ft heq = h + hs = 11 0.07(14.5 ft) = 1.02 ft, say, 1 ft tstem ~ 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!LQKH 20.6.1.3.1 /HQJWKRIEDVHWRHKHHO WR heq /HQJWKRIWRHab/4 to b/3 Heel length: bheel = bbase – btoe – tstem bbase ~ 0.5 (14.5 ft) = 7.25 ft, use 7 ft 3in. btoe ~ 7 ft 3 in./3 = 2 ft 4 in., say, 2 ft 3 in. bheel = (7 ft 3 in.) – (2 ft 3 in.) – (1 ft) = 4 ft American Concrete Institute – Copyrighted © Material – www.concrete.org 64 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. E4.3). Vertical loads: The 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 Ūconc onc(tstem)(h í tbase) Base: P2 Ūconc(ttbas base)(b) Soil: P3 Ūs(h + hs í tbase se)(bheel) (150 pcf)(1.0 cf)(1.0 f)(1 0 ft)(fft)(11.5 (1 ft – 1.25 ft) = 1538 lb P1 = (15 ((150 pcf)(1.25 f)(1.25 ft)(7 ft)(7.25 ft) = 1359 lb P2 = (1 (120 pcf)(11.5 11.5 ft + 33.0 ft –1.25 ft)(4 ft) = 6360 lb P3 = (12 oa Total vertical load: ™ = 1538 15 lb + 1359 lb + 6360 lb = 9257 lb ™P aining wall an he soil The self-weight off th the retaining and the and distributed load 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(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/2) = 4926 ft-lb M3 = (6360 lb)(7.25 ft – 4.0 ft/2) = 33,390 ft-lb Restoring moment: ™MR = 4229 ft-lb + 4926 ft-lb + 33,390 ft-lb = 42,545 ft-lb Fig. E4.3—App E4.3—Applied forces on retaining wall. The retained soil behind the wall and the superimposed loads exert lateral pressure (H) on the wall: Due to retained earth: H1 =(Ca)(Ūs)(h2/2) Due to surcharge: H2 = (Ca)(Ūs)(hs)(h) Ca is calculated from Rankine's equation. Ca = H1 = (32.5 psf)(11.5 ft)2/2 = 2150 lb/ft H2 = (32.5 psf)(3 ft)(11.5 ft) = 1122 lb/ft 1 − sin φ 1 + sin φ Therefore, this lateral force tends to overturn the retaining wall about the front edge of the toe: MOTM = H1(h/3) + H2(h/2) 6XPPDWLRQRIPRPHQWV¨M ™MR – MOTM MOTM = (2150 lb/ft)(11.5 ft/3) + (1122 lb/ft)(11.5 ft/2) = 14,693 ft-lb/ft ¨M = (42,545 ft-lb) – (14,693 ft-lb) = 27,582 ft-lb 7KHUHIRUHEDVHJHRPHWU\LVVXI¿FLHQWWRUHVLVWRYHUturning. American Concrete Institute – Copyrighted © 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= ΔM ∑P a= 27,852 ft-lb = 3.01 ft 9258 lb Eccentricity is the difference between the resultant location and the base mid-length: e = bbase/2 – a e = 7.25 ft/2 – 3.01 ft = 0.62 ft Check if resultant falls within the middle third of the base. essure: Maximum and minimum soil pressure: q1,2 = ∑P ∑M ± A S bbase 7.25 ft = = 1.21 ft > e = 0.62 ft 6 6 OK Therefo there is no uplift. Therefore, q1,2 = 9257 lb (9257 lb)(0.62 ft) ± (7.25 (7 25 ft)(1 ft) (7.25 ft) 2 /6 q1 ,2 = 11277 psf ± 655 ppsff q1 = qma 32 psf < qall = 3000 psf max = 1932 2 psf > 0 psf no uplift q2 = qmin i = 622 -bearing pre Soil-bearing pressure is acceptable. Step 6: Stability requirements Calculate the factor of safety against overturning: FS = ∑ MR ≥ 2.0 M OTM FS = 42,545 ft-lb = 2.90 14,693 ft-lb FS = 2.90 > 2.0 OK Calculate the factor of safety against sliding: FS = ∑ μP ≥ 1.5 ∑H This calculation neglects the passive pressure against the toe (conservative). FS = (0.55)(9257 lb) = 1.56 (2150 lb) + (1122 lb) FS = 1.56 > 1.5 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. 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 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 is calculated at the top face of the base foundation. 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. Stem height: hstem = (9 ft) + (2.5 ft) – (1.25 ft) = 10.25 ft 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 εt 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: εc = 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. Psurcharge 5 pcf)(3 ft ft) = 98 psf charge = (32.5 U = 1.6[(98 1.6 f)(11.5 ft – 1.25 ft) psf)(11.5 + ((32.5 pcf)(10.25 cf)(10 25 ft)(10.25 ft)/2] = 1607 lb + 2732 lb = 4340 lb Mu = (1607 lb)(10.25 ft/2) + (2732 lb) (10.25 ft/3) = 17,570 ft-lb = 210,843 in.-lb, 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 for design: Retaining Walls 22.2.2.3 67 The concrete 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 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 stress distribution of: 0.85fcȝZLWKDGHSWKRI a ȕ1c, where ȕ1 is a function of concrete compressive strength and is obtained from Table 22.2.2.4.3.)LJ(²6WHPÀH[XUDOUHLQIRUFHPHQW) (PÀH[XUDO 0.05(4500 5(4500 ps psi − 4000 psi) = 0.825 1000 100 psi 22.2.2.4.3 VL For fcȝ"SVL 0.85 − β1 = 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⎞ ⎛ M n = f y As ⎜ d − ⎟ ⎝ 2⎠ 1.31As ⎞ ⎛ M n = (60,000 psi)As ⎜ 9.6 in. − ⎟ ⎝ 2 ⎠ 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▯Mn•Mu Mu = 211 in.-kip calculated above. 7.5.1.1 Solving for As (refer to Fig. E4.4): d = 12 in. – 2 in. – 0.75 in./2 = 9.625 in., say, 9.6 in. 1.31As ⎞ ⎛ (0.9)(60 ksi)As ⎜ 9.6 in. − ≥ 211 in.-kip ⎝ 2 ⎠⎟ As,req'd = 0.42 in.2 use No. 6 at 12 in. on center. As,prov'd = 0.44 in.2/ft > As,req'd = 0.42 in.2/ft American Concrete Institute – 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 EHDPPLQLPXPUHTXLUHGÀH[XUDOUHLQIRUFHPHQW area rather than the one-way slab minimum reinforcement area because of the lack of redundancy. The Code requires that the beam reinforcement area is: (a) As , min ≥ 3 f c′ fy bw d As ,min = 3 4500 psi (12 in.)(9.6 in.) = 0.385 in.2 60,000 psi As,prov'd = 0.44 in.2/ft > As,req'd = 0.39 in.2/ft Equation (9.6.1.2(a)) controls, because concrete compressive strength fcȝ SVL 21.2.2 Check if the tension controlled assumption and the XVHRI▯LVFRUUHFW To answer the question, the tensile strain in GDQG UHLQIRUFHPHQWPXVWEHःUVWFDOFXODWHGDQG 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 ε ε t = c (d − c) c where: c = a and nd a = 1.31A .31As derived previously. viously. β1 c= εt = (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. εt = 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 and the 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. 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 ft2)/2] = 4331 lb 7.5.3.1 22.5.5.1 Vn = (Vc + Vs) = 2λ f c′bw d Vn = (2) 21.2.1 Shear strength reduction factor: =▯ ▯0.75 s: Therefore, ▯Vn = ▯Vc is: (0.75)(15,456 lb)) = 11,600 lb ▯Vn = (0.75)(15,45 ¿HG Is ▯ Vn•VuVDWLV¿HG" 0 lb > Vu = 4331 lb ▯Vn = 111,600 7.5.1.1 ( ) 4500 psi (12 in.)(9.6 in.) = 15, 456 lb with Vs = 0 OK Th refo shearr reinforc Therefore, reinforcement is not required American Concrete Institute – Copyrighted © Material – www.concrete.org 70 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) = 2λ 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 OOVHOIZHLJKW VXSHUZHLJKWDQGIRUEDFNःOOVHOIZHLJKWDQGVXSHUimposed load of 360 psf. 7.5.1.1 Is ▯ Vn•VuVDWLV¿HG" Vu = 1.2[(150 150 pcf)(4 pcf)(4.0 ft – 11.5 in./12)(1.25 ft)] + 1.6[(120 pcf)(10.25 ft) + (360 psf)] × (4 (4.0 ft – 11.5 in./12) = 684 lb + 7738 lb = 8422 lb ▯Vn = 13,886 lb > Vu = 8422 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 12—RETAINING WALLS Retaining Walls Flexure 7KHKHHOLVVXEMHFWHGWRÀH[XUHFDXVHGE\WKHVXSHUimposed weight of soil and self-weight of heel. The soil pressure counteracting the applied gravity loads is neglected as it is not included in WKHFDOFXODWLRQRIÀH[XUHIRUWRLJ(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 IRUEDFNःOOVHOIZHLJKWDQGVXSHULPSRVHGORDG 22.2.1.1 Setting C = T Mu1 = 1.2(150 pcf)(1.25 ft)(4.0)2/2 + 1.6[(360 psf) + (120 pcf)(10.25 ft)](4.0 ft)2/2 = 1800 ft-lb + 20,352 ft-lb = 22,152 ft-lb = 265,824 in.-lb 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 a⎞ ⎛ M n = f y As ⎜ d − ⎟ ⎝ 2⎠ 1.31As ⎞ ⎛ M n = (60,000 psi)As ⎜ 11.5 in. − ⎟ ⎝ 2 ⎠ 21.2.1(a) 6WUHQJWKUHGXFWLRQIDFWRUIRUÀH[XUH IDFWR 7.5.1.1 0.9 = ▯ ▯Mn•Mu Mu = 265,825 in.-lb n.- = 266 in.-kip, calcu calculated ated above. 1.31As ⎞ ⎛ (60 ksi) As ⎜11.5 11 5 in iin. − 0.9(60 ≥ 266 in.-kip ⎝ 2 ⎠⎟ Fig E4.8): 8): Solving for As (Fig As = 0.4 0.43 in.2 9.6.1.2 The cantilevered retaining wall 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 ÀH[XUDOUHLQIRUFHPHQWDUHDLVDWOHDVWWKHJUHDWHURI (a) As , min ≥ 3 f c′ fy bw d Equation (9.6.1.2(a)) controls, because concrete compressive strength fcȝ 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 in UHLQIRUFHPHQWPXVWEHःUVWFDOFXODWHGDQG 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 εt = εc (d − c) c where: c = a and a = 1.31As derived previously. β1 71 As,min = 3 4500 psi (12 in.)(11.5 in.) = 0.46 in.2 60,000 psi ⇒ 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. εt = 0.0333 > 0.005 εt = Section is tension controlled and 0.9 = ▯American Concrete Institute – 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 = ▯or 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) = 2λ 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) ▯lb) = 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): d = (1932 psf − 622 psf) ⎛ 5.0 ft + 11.5 in./12 ⎞⎟ ⎜⎝ ⎠ + 622 psf = 1700 psf 7.25 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 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•VuVDWLV¿HG" ▯Vn = 13,886 > Vu = 3535 lb OK Shear reinforcement is not required. American Concrete Institute – Copyrighted © Material – www.concrete.org Flexure Soil 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 stem , L = (1932 psf − 622 psf) ⎜ ⎝ 7.25 ft ⎠⎟ = 1525 psf Mu2 = 1.6 [(1525 psf (2.25 ft)2/2 SVIíSVI IW 2(2/3)] = 1.6[(3860 ft-lb) + (1374 ft-lb) = 8374 ft-lb Mu1 = 0.9(150 pcf)(1.25 ft)(2.25 ft)2/2 = 427 ft-lb Mu,req'd = Mu2íMu1 = (8374 ft-lb) − (427 ft-lb) = 7947 ft-lb = 95,364 in.-lb, say, 95.4 in.-kip Concrete is placed against earth. Therefore, use 3 in. cover. d = 11.5 in. calculated above Setting: C = T to calculate required reinforcement 0.85(4500 psi)(12 in.)(a) = As(60,000 psi) a= 7.5.2.1 7.5.1.1 21.2.1(a) 73 a⎞ ⎛ M n = f y As ⎜ d − ⎟ ⎝ 2⎠ ▯Mn•Mu As (60,000 psi) = 1.31As 0.85(4500 psi)(12 in.) 0.85(4 1.31As ⎞ ⎛ 000 psi)As ⎜1 11 1 in. − M n = (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 4500 psi (12 in.)(11.5 in.) = 0.46 in.2 60,000 psi Use No. 7 at 12 in. on center Equation (9.6.1.2(a)) controls, because concrete compressive strength fcȝ SVL 21.2.2 Check if the section is tension controlled assumpWLRQDQGWKHXVHRI▯LVFRUUHFW 21.2.2 To answer the question, the tensile strain in reinIRUFHPHQWPXVWEHःUVWFDOFXODWHGDQGFRPSDUHG 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 εt = εc (d − c) c where: c = a β1 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. εt = 0.0333 > 0.005 Section is tension controlled. Therefore, assumption of 0.9 = ▯s correct. εt = 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 Stem 11.6.1 Assume that No. 5 bars or smaller will be used for temperature and shrinkage reinforcement. Per Table 11.6.1, d, min = 0.002 As,min = (0.002)(12 in.)(12 in.) = 0.288 in.2 Distribute transverse reinforcement equally between the front and back face of the stem wall. (As,min)front = (As,min)back = 0.288 in.2/2 = 0.144 in.2 Try No. 4 at 16 in. on center n⎞ ⎛ 12 in. in.2 ⎜ 0 2 in As , prov = 0.15 in.2 rov . = 0.2 ⎝ 16 in.⎠ > As , req ' d = 0.144 in.2 /face rcement at front face ace tto Provide verticall rreinforcement sve wall reinforceme t support the transverse reinforcement. 24.4.3 Base Assume that No. 5 bars or smaller will be used ent. The for temperature and shrinkage reinforcement. reinforcement can be located at the top, bottom, or allocated between the two faces. Therefore, refo provided ided rein reinforcement is 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 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 peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development RIUHLQIRUFHPHQWLQÀH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLNnates, 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 6XःFLHQWGLVWDQFHWRHDFFRPPRGDWHWKHGHYHOopment length in the toe sec section. cem in the stem m to 2. Extension off toe reinforcement ire development elopment length satisfy the required length. op length in the ste m mu Note: the development stem must be ice length and th checked against the splice the larger length controls. 25.4.2.1 Development length ℓd is the greater of Eq. (25.4.2.2) or (25.4.2.3) of ACI 318-14 and 12 in.: 25.4.2.2 ⎛ f yψt ψe ⎞ 1. A d = ⎜ ⎟ db ⎝ 200λ f c′ ⎠ 25.4.2.3 ⎛ ⎞ ⎜ 3 f y ψt ψeψ s ⎟ 2. A d = ⎜ ⎟ db ⎜ 40 λ f c′ cb + K tr ⎟ ⎜ ⎝ db ⎠⎟ 25.4.2.4 In this example Eq. (25.4.2.3a) will be used. ζt = bar location; not more than 12 in. of fresh concrete below horizontal reinforcement ζe = coating factor; uncoated ζs = bar size factor; No. 7 and larger 75 ζt = 1.0, because not more than 12 in. of concrete is placed below bars ζe = 1.0, because bars are uncoated ζs = 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)): 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 base into the stem must be checked against the splice length off stem reinforcentrols. ment. The larger 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. > ℓd 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 to required reinbl forcement 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 (1.3ℓd) 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.) = 1.5 in. (c) 4/3(1 in.) = 1.33 in. From Step 1, 1 in. maximum aggregate size is used. Therefore, 1.5 in. controls American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 12—RETAINING WALLS 77 Weight of steel per linear foot in stem 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) (2.75 ft) + (1.502 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 to a level where smaller bar diameters, larger bar spacing, or both can be used for 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 = CaŪsho3/6 The required reinforcement area: Use bar size such that the required calculated spacing matches the spacing of the No.7 extended from the base. 11.6.1 &KHFNLISURYLGHGUHLQIRUFHPHQWDUHDVDWLVःHVWKH minimum 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-lb As,req'd = 0.24 in.2/ft Use No. 5 @ 12 in. o.c. As,prov = 0.31 in.2/ft > As,req'd OK 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 7.7.1.3 Required splice length: 25.5.1.1 The maximum barr size is No.7, therefore 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 ℓstt of the he sm smaller greater of ℓd 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 (1.3ℓd) or 12 in., where ℓd is the develop⎛ 3 60,000 psi (1.0)(1.0)(1.0) ⎞ ment length calculated above. Ad = ⎜ ⎟ (0.625 in.) 2.5 ⎝ 40 (1.0) 4500 psi ⎠ Development of No.5 bar = 16.8 in., say, 17 in. 25.4.2.3 7.7.1.3 25.5.2.2 Splice Compare development length of No.7 to splice length of No. 5 ℓst = 1.3(17.0 in.) = 22.1 in. Use ℓd = 24 in. > ℓst = 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 Total length of No. 7 = 4.0 ft Total length of No. 5 = 9.0 ft – 4.0 ft + 2 ft = 7 ft Total weight = (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 Institute – Copyrighted © Material – www.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 'HVLJQDQRUPDOZHLJKWUHLQIRUFHGFRQFUHWHFDQWLOHYHUZDOOWRUHWDLQDQHDUWKHPEDQNPHQWKLJKDQGSDUWLDOOHYHOHDQGFRQFUHWHDUWKGHSWKUHDFKWKDWLVIURPGDGLQFRUSRUDWHG 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: VRLOXQLWZHLJKWRISFIDQJOHRIVRLOLQWHUQDOIULFWLRQRIGHJUHHVFRHIःFLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLO 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\RWKHUORDGDQGGWKHIURVWOLQHLVIWEHORZWKH¿QLVKHGJUDGH Given: Soil data— Ū s = 110 pcf ▯GHJUHHV ȝ concrete-soil = 0.4 ȝ soil-soil = 0.7 qall = 3000 psf Concrete— Ȝ a = 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&KDSWHUUHTXLUHPHQWVDUHVDWLVःHG Based on durability and strength requirements, and experience with local mixtures, the compressive strength RIFRQFUHWHLVVSHFLःHGDVWREHDWOHDVWSVL $&,LVDUHIHUHQFHVSHFLःFDWLRQWKDWLVFRRUGLnated with ACI 318. ACI encourages referencing LQWRMREVSHFLःFDWLRQV 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 presented by Bowles LQWKH¿IWKHGLWLRQRIFoundation Analysis and Design, The McGraw-Hill Companies, Inc., calculation. 7.4.3.2 Applied shear at distance d from stem: d = (2755 psf − 265 psf) (4.5 ft + 1.25 ft + 12.5 in./12) + 265 psf 8 ft = 2380 psf 5.3.1 For soil reaction on the toe portion, a load factor of Vu = 1.6(2755 psf + 2380 psf)/2)(2.25 ft – 12.5 in./12) – (0.9)(150(1.33 ft)(2.25 ft – 12.5 in./12) 1.6 is taken. For the self-weight of concrete portion, = 4964 lb – 217 lb = 4747 lb OK a load factor of 0.9 is used. 7.5.1.1 ,V Vn•VuVDWLV¿HG" 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 stem , L = (2755 psf − 265 psf) (8 ft) = 2055 psf Mu2 = 1.6 [(2055 psf (2.25 ft)2/2 SVIíSVI IW 2 (2/3)] = 1.6(5202 ft-lb + 2363 ft-lb) = 12,104 ft-lb) = 12,104 ft-lb 5.3.1 20.6.1.3.1 22.2.1.1 Moment due of toe self-weight: Mu1 = 0.9(150 pcf)(1.33 ft)(2.25 ft)2/2 = 454 ft-lb Resultant moment: Mu,req'd = Mu2íMu1 = (12,104 ft-lb) – (454 ft-lb) = 11,650 ft-lb ≅ 140,000 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 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 a) (M n = f y As | d − | (2) 1.31As) (M n = (60,000 (60,00 psi)As | 12.5 in. − (2 |) 21.2.1 Flexural strength reduction educt factor Mn•Mu 1.31As) (9) As (60 ksi)) | 12.5 12 5 in in. − (0.9) = 140 in.-kip (2 |) Mu = 140 in.-kipp calculated ated above. Solve for As: 9.6.1.2 As = 0.2 0.21 in.2 7KHFDQWLOHYHUHGUHWDLQLQJZDOOEHKDYHVDVDÀH[ural member. The tensile 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 ⇒ As,min = 0.50 in.2 Use No.7 at 12 in. on center As,prov'd = 0.60 in.2 > As,req'd = 0.50 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 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. E5.7): 21.2.1.2 εc (d − c) c where: and a = 1.31As εt = c= (1.31)(0.60 in.2) = 0.95 in. 0.825 εt = 0.003 (12.5 in. − 0.95 in.) = 0.036 0.95 in. εt = 0.036 > 0.005 6HFWLRQLVWHQVLRQFRQWUROOHGDQG Shear key design)RUGHVLJQLQJWKHVKHDUNH\IRUVKHDUDQGÀH[XUH strength, the soil above the toe is not considered, as this will apply higher force on the shear key and it may not be present. HGSUHYLRXVO\ 3DVVLYHFRHI¿FLHQWZDVFDOFXODWHGSUHYLRXVO\ Cp = 1 1 = = 3.69 Ca 0.271 Hp = 1 (3.69)(11 111)(1.33)(1 33 ft) 2 = 359 lb 2 Hp = 1 (3.69)(11)(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)(γ s))((tbase) 2 2 Bottom of shear key: Hp = 1 (C p)(γ s)(hbase + 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: Vu PXVWQRWH[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.) =116,525 lb (c) (1600)(12 in.)(11.56 in.) = 222,000 lb (b) controls and 116,525 lb > Vu/ OEèK Shear strength: The shear key may be considered similar to a corbel or bracket. Extend No. 7 reinforcement into the shear key. The 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 ȝAvf fy ZKHUHȝ ȜȜ DQG &KHFNLI Vn•Vu (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. Vn OE•Vu = 1710 lb OK American Concrete Institute – Copyrighted © Material – www.concrete.org 97 Step 10: Minimum transverse reinforcement Stem 11.6.1 Assume that No. 5 bars or smaller will be used for temperature and shrinkage reinforcement. Per Table ρmin = 0.002. As,min = 0.002. As,min = (0.002)(12 in.)(15 in.) = 0.36 in.2 Distribute shrinkage and temperature reinforcement (As,min)front = (As,min) back = 0.36 in.2/2 = 0.18 in.2 equally between the front 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 bars distributed top, bottom, or anywhe anywhere within the base. Therefore, place three No. 5 bottom at 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 mbedment 25.4.2 attainable average orc t. length of reinforcement. gth aree required beca Development lengths because of the tendency of highly stressed bars to split relatively thin sections of restraining concrete. In application, the development length concept i requires minimum lengths or extensions off reinforcement beyond all points of peak stress in the reinforcement. Such peak stresses generally occur at the points of critical sections for development RIUHLQIRUFHPHQWLQÀH[XUDOPHPEHUVDQGDWSRLQWV ZLWKLQDVSDQZKHUHDGMDFHQWUHLQIRUFHPHQWWHUPLnates, or is bent. For the cantilevered wall, toe and heel reinforcement must be properly developed. Heel reinforcement is developed beyond the stem critical section. Toe reinforcement must be checked for: 6XI¿FLHQWGLVWDQFHWRDFFRPPRGDWHWKHGHYHORopment length in the toe section. 2. Extension of toe reinforcement in the stem to satisfy the required development length. Note: the development length in the stem must be checked against the splice 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, ℓd, is the greater of Eq 25.4.2.2 or 25.4.2.3 of ACI 318-14 and 12 in.: (f yψt ψe) 1. A d = | | db (200λ f c′) ( ) | 3 f y ψt ψeψ s | 2. A d = | | db | 40 λ f c′ cb + K tr | | (db |) In this example, Eq. (25.4.2.3a) will be used. ψt = bar location ψe = coating factor ψs = bar size factor ψt = 1.0, because not more than 12 in. of fresh concrete is placed below horizontal reinforcement ψe = 1.0; because bars are uncoated ψs = 1.0; because 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) 25.4.2.3 Eq (25.4.2.3a): 5.4.2.3a): Substituting into Eq. velop length of heel and toe reinforcement: Development (3 60 60,000 60,00 psi (1.0)(1.0)(1.0)) Ad = | | (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 (24 in.) for No. 7. Note: the development length in the 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 (1.3ℓd) or 12 in. Clear spacing between bars 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. Use 2 ft 9 in. in. for splice length. Therefore, extend No. 7 toe reinforcement 2ft 9in. into the stem. Refer to Fig. E5.9. (a) 1.5 in. (b) 1.5(0.875 in.) = 1.31 in. (c) 4/3(1.0 in.) = 1.33 in. Therefore, 1.5 in. controls. Weight of steel per lineal foot in stem only: No. 7 = 2.044 lb/ft Total length leng = (2.67 ft + 13.17 ft) = 15.84 ft 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 No. 7 bars are extended a distance 4 ft 6 in. into the stem. The factored applied moment at that level can be calculated from: Mu = CaÛsho3/6 The required reinforcement area: Mu = (1.6)[(0.271)(110 pcf)(13.17 ft – 4.5 ft)3/6 + (0.271)(110 pcf)(3 ft)(13.17 ft – 4.5 ft)2/2] = 10,559 ft-lb As,req'd = 0.19 in.2/ft 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/ft > As,req'd 0.19 in.2/ft base. 11.6.1 WDUHDVDWLV¿HVWKH &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 ℓstt of thee smaller greater of ℓd 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 is less than 2. Therefore, per Table 21.5.2.1, splice is Type B and the splice length is the greater of (1.3ℓd) or 12 in., where ℓd is the development length calculated previously. 25.4.2.3 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. ℓst = 1.3(17.0 in.) = 22.1 in. Use ℓd = 24 in. > ℓst = 22.1 in. Use 24 in. 7.7.1.3 25.5 25.5.2.2 Splice Compare development length of No. 7 to splice length of No. 5. Refer to 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)(10.67 ft) = 20.32 lb/ft Fig. E5.10—Retaining wall alternate reinforcement. Conclusion: (20.3 lb/ft) Alternate solution provides a saving of: 32.4 lb/ft – 20.32 lb/ft = 12.1 lb/ft or | 1 − = 37.3 percent. (32.4 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 'HVLJQDQRUPDOZHLJKWUHLQIRUFHGFRQFUHWHFDQWLOHYHUZDOOWKDWUHWDLQVDOHYHOHDUWKEDQNIWKLJKDERYHWKH¿QDOHDUWKOHYHO as shown in Fig. 6.1. 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: soil unit weight of 120 pcf, angle of soil LQWHUQDOIULFWLRQRIGHJUHHVFRHI¿FLHQWRIIULFWLRQEHWZHHQFRQFUHWHDQGVRLORIDQGWKHDOORZDEOHVRLOEHDULQJSUHVVXUH of 3000 psf. $VVXPHWKDWWKHFDQWLOHYHUZDOOLVEXLOWDWWKHSURSHUW\OLQHLVQRWVXEMHFWHGWRDQ\RWKHUORDGDQGWKHIURVWOLQH EHORZWKH¿QLVKHGJUDGH Given: Soil data— Ûs = 120 pcf GHJUHHV ȝ qall = 3000 psf Concrete— Ȝ 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 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 from: . aspx?ItemID=SP1714DAWKHPD[LPXPGHȦHFWLRQ 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 GLUHFWLRQRIGHȦHFWLRQFDOFXODWLRQDQGI 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)RUGHȦHFWLRQFDOFXODWLRQXVHVHUYLFHORDGVHTXDWLRQV 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'HȦHFWLRQV´Concrete International9 33, No. 7, July, pp. 24-29. Total moment within a span Mo = qA 2 An 8 where ℰn is the clear 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: Positive moment – column strip middle strip 8.10.5.5 D + L = 75 psf + 15 psf +100 psf = 190 psf Controls M D+ L = (190 p sf)(14 ft)(16 ft) 2 = 85 ft-kip 8 M = (0.65)(85 ft-kip) = 55 ft-kip M+= (0.35)(85 ft-kip) = 30 ft-kip (0.75)(55 ft-kip) = 41 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 Structures," recommends modifying the concrete modulus of rupture, fr, and modulus of elasticity, Ec, to account for the lower 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 ℨ 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 concrete are: a = 0.67 to 0.98 and b = 0.05 to 9.25. For fcgDWGD\VKHFRHI¿FLHQWa = 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 th 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 ρ= 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\HTXDWLQJWKH¿UVWPRPHQWRIFRPSUHVVLRQDUHDWRWKHHQVLRQDUHDPRGL¿HGE\WKH modular ratio) of the slab: bc 2 – nAs (d − c) = 0 2 n= (9)(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)(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 = (ρn) 2 + 2(ρn) − ρn 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 ft)(12)(1.41 in.)3 + (8.6)(9)(0.31 in.2) 3 × (4.9 in. − 1.41 in.) 2 Icr = 78 in.4 + 292 in.4 = 370 in.4 Step 4: Effective moment of inertia 24.2.3.5 Calculate effective 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¿[HGDWERWKHQGV (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 , + I e , midspan + I e , 3 I e , avg = 414 in.4 + 1512 in.4 + 414 in.4 = 780 in.4 3 $&,&RPPLWWHH5³&RQWURORI'HȦHFWLRQ in Concrete Structures" recommends an alternate equation (Eq. (2.15a)) to calculate the average efIHFWLYHPRPHQWRILQHUWLDLQDEHDPZLWKWZR¿[HG or continuous ends. Ie,avg = 0.7Ie,midspan + 0.15( supp + supp) Ie,avg = 0.70(1512 in.4) + 0.15(414 in.4 + 414 in.4) = 1183 in.4 Use weighted average based on ACI 318. It will result LQDODUJHUGHȦHFWLRQ 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&ROXPQVWULSGHȦHFWLRQV(: D,PPHGLDWHGHȦHFWLRQ From Step 1: Δ= α qA 4n 384 EI Dead and superimposed dead loads are applied uniform and continuous over all spans. The factor [ZKLFKDFFRXQWVIRUVSDQERXQGDU\FRQGLWLRQV is taken as 1 for continuous and uniformly loaded VSDQV7KUHIRUHFROXPQVWULSLPPHGLDWHGHȦHFWLRQGXHWRGHDGORDGORQJWKH[D[LV¨C,x,D: ΔC , x,D = (1.0)qA 4 384 EI ΔC , x, D = (75 psf + 15 psf)(0.675)(14 ft)(18 ft)4 (12)3 (384) (57,000)(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 PRVWFULWLFDOORDGFRQGLWLRQGHȦHFWLRQ 7KH[IDFtor in this case is take taken equal to 2. P SLPPHGLDWHGHȦ FWLRQGXH 7KHUHIRUHFROXPQVWULSLPPHGLDWHGHȦHFWLRQGXH QJ D[L C,x,L: WROLYHORDGORQJWKH[D[LV¨ ΔC , 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) (57,000)(= 0.28 in. n. E/RQJWHUPGHȦHFWLRQ)RUORQJWHUPGHȦHFWLRQGHGORDGVHOIZHLJKW and superimposed loads) and a percentage of live load assumed as sustained load, are considered.)RUOLYHORDGWREHFRQVLGHUHGDVVXVWDLQHGORDGLQFDOFXODWLQJ long-term deflections, 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)RUDGGLWLRQDOWLPHGHSHQGHQWGHȦHFWLRQUHVXOWLQJ IURPFUHHSDQGVKULQNDJHRIȦH[XUDOPHPEHUVWKH GHȦHFWLRQVKDOOEHFDOFXODWHGDVWKHSURGXFWRIWKHLPPHGLDWHGHȦHFWLRQ following equation: American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 13—SERVICEABILITY λΔ = ξ 1 + 50ρ′ 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: ħ 7KHUHIRUHORQJWHUPGHȦHFWLRQRIWKHFROXPQVWULS along the x-axis is: ¨C,x,T ℨ¨ ¨t ¨C,x,T = (1 + 2)(0.13 in. + (0.25)(0.28 in.)) = 0.60 in. 6WHS0LGGOHVWULSGHȦHFWLRQV(: 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 The middle strip negative and positive moments are given above in Step 2: Negative moment at support: an: Positive moment at midspan: Compare positive and nnegative moment to cracked moment in Step 3: Since Mcr is larger ger than both positive an and negative ction is moments in the m middle strip, the slab ssection ck and d IgLVXVHGLQWKH LVXVHGLQWKHGHȦHFWLRQ GHȦH assumed uncracked calculations: D,PPHGLDWHGHȦHFWLRQ ,PPHGLDWHGHȦHFWLRQGXHWRGHDGORDGLV Δ M , x, D = 1 qA 4 384 EI M–mid-st mid-strip = 14 ft-kip M+mid-strip = 14 ft-kip 18 ft-kip -kip > M–mid-strip > M+mid-strip Mcr = 18.6 m n.4 Ie = Ig = 1512 in. Δ M , x, D = (75 psf + 15 psf)(0.325) (14 ft)(18 ft)4 (12)3 (7 (384)(57,000)(5000 psi)(1512 in.4) = 0.03 in. ,PPHGLDWHPLGGOHVWULSGHȦHFWLRQGXHWROLYHORDG along the x-axis: Δ M , x, L = 2(100 psf)(0.325)(14 ft)(18 ft)4 (12 in./ft)3 (384)(57,000)(5000 psi)(1512 in.4) = 0.07 in 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/RQJWHUPGHȦHFWLRQ 24.2.4.1.1 0XOWLSOLHUIRUWLPHGHSHQGHQWGHȦHFWLRQVE\WKHIDFWRU λΔ = ξ 1 + 50ρ′ λΔ = 2.0 =2 1+ 0 Refer to variable values in Step 6. 7KHUHIRUHPLGGOHVWULSORQJWHUPGHȦHFWLRQ ¨M,x,T ℨ¨ ¨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,ǫ 136 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Summary &ROXPQVWULSDQPLGGOHVWULSLPPHGLDWHGDQGORQJWHUPGHȦHFWLRQVLQWKHHDVWZHVW(: GLUHFWLRQ Immediate &ROXPQVWULS¨C,x 0LGGOHVWULS¨MS,x Step 6a 7a Dead load 0.13 0.03 Long term ℨ¨)(DL + 0.25LL) 0.60 0.14 Live load 0.28 0.07 Step 9: Column strip N-S direction 7KHVDPHGHȦHFWLRQFDOFXODWLRQVDUHUHSHDWHGEXW in the north-south (N-S) direction. In this case, ℰ2 = 18 ft ℰn = 14 ft – (12 in./12)2 = 12 ft From Step 2, the controlling load is: (1.0)(D + L) = 190 psf 8.10.3.3 Total moment within a span: qA 2 A 2n 8 Total service load moment within a span: Mo = 8.10.4.1 ue to de Moment distribution due dead load: 8.10.5.1 tion due to dead load: Moment distribution ent – column umn strip: Negative moment middle strip: 8.10.5.5 nt – column umn strip: Positive moment middle strip: Step 10: Cracked 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 direc tion, 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-ki ft-kip) p)) = 22 ft-kip M+ = (0.35)(62 30 ft-kip 10 ft-kip 13 ft-kip -kip 9 ft-kip Mcr = 18.6 ft-kip and Ma+ = 13 ft-kip < Mcr = 18.6 ft-kip < Ma = 30 ft-kip Therefore, slab will crack at the support. Calculate cracked moment for column strip: 20.6.3.1 Effective depth: d = h – cc – 1.5db d = 6 in. – 3/4 in. – 1.5(0.625 in.) = 4.3 in. &DOFXODWHd, As/bd ρ= (9)(0.31 in.2) = 0.00772 (7 ft)(12 in./ft)(4.3 in.) Ten No. 5 bars are 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¿UVWPRPHQWRIFRPSUHVVLRQDUHDWRWKHHQVLRQDUHDPRGL¿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 = (ρn) 2 + 2(ρn) − ρn k = 0.304 6XEVWLWXWLQJIRUd, DQGIRU n = 8.6 calculated above Therefore, c = kd Serviceability c = (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 ft)(12)(1.31 in.)3 + (8.6)(9)(0.31 in.2) 3 × (4.3 in. − 1.31 in.) 2 Icr = 63 in.4 + 215 in.4 = 278 in.4 24.2.3.5 Calculate effective 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 ccr) ⎢ cr ⎥ ⎝ Ma ⎠ 3 ⎛ 18.6 ft-kip ⎞ (27 in.4) + (1512 (5 in. in 4 − 278 in.4) ⎜ I e = (278 ⎝ 40 ft-kip ⎠⎟ 3 Ie = 402 in.4 The weighted effective fec moment of iner inertia in the column strip is: I e , avg = 402 in. n.4 + 1512 151 in.4 + 402 in.4 = 772 in.4 3 The differ difference is negligible, 772 in.4 compared to 780 in.4: Ie = 772 in.4 D,PPHGLDWHFROXPQVWULSGHȦHFWLRQVLQFROXPQVWULSV Repeating the same calculations, the following is obtained (refer to Step 6a): ,PPHGLDWHFROXPQVWULSGHȦHFWLRQGXHWRGHDGORDG along the y-axis: ΔC , y,D = αqA 4 384 EI ΔC , y,D = (75 psf + 15 psf)(0.675)(18 ft)(14 ft)4 (12)3 384(57,000)(5000 psi)(772 in.4) = 0.06 in. ,PPHGLDWHFROXPQVWULSGHȦHFWLRQGXHWROLYHORDG 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. ΔC , y,L = 2(100 psf)(0.675)(18 ft)(14 ft)4 (12 in./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/RQJWHUPGHȦHFWLRQ 24.2.4.1.1 0XOWLSOLHUIRUWLPHGHSHQGHQWGHȦHFWLRQVE\WKHIDFWRU λΔ = ξ 1 + 50ρ′ λΔ = 2.0 =2 1+ 0 Refer to variable calculation in Step 6: 7KHUHIRUHORQJWHUPGHȦHFWLRQLV ¨C,y,T ℨ¨ ¨i 6WHS0LGGOHVWULSGHȦHFWLRQV16 D,PPHGLDWHGHȦHFWLRQ 19.2.3.1 Calculate cracked moment: fcr ℨ 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.)3 = 2376 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) Mcrr = 29.3 ft-k ft-kip > Ma = 10 ft-kip > Ma+ = 9 ft-kip refo Ie = Ig = 22376 37 in.4 Therefore, α qA 4n 384 EI where Ie = Ig = 1512 in.4 Δ= ,PPHGLDWHPLGGOHVWULSGHȦHFWLRQGXHWRGHDGORDG Δ M , y ,,DD = ξ 1 + 50ρ′ 4(57,000 5000 psi) (2376 in.4) 384(57,000)(= 0.01 in. Δ 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/RQJWHUPGHȦHFWLRQ 24.2.4.1.1 0XOWLSOLHUIRUWLPHGHSHQGHQWGHȦHFWLRQV6WHSD \WKH factor: λΔ = (75 psf sf + 15 psf)(0.325)(18 ps ft)(14 ft)4 (12)3 2(100 psf)(0.325)(18 ft)(14 ft)4 (12 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: 7KHUHIRUHORQJWHUPGHȦHFWLRQLV ¨M,y,T ℨ¨ ¨i Step 12: Summary ¨M,y,T = (1 + 2)(0.01 in. + (0.25)(0.02 in.)) = 0.05 in. &ROXPQVWULSDQGPLGGOHVWULSLPPHGLDWHGDQGORQJWHUPGHȦHFWLRQVLQWKH16GLUHFWLRQ Immediate /RQJWHUP Step Dead load /LYHORDG ℨ¨)(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. ZKHUHLQDQGLQDUHWKHVXPRIWKHORQJWHUPGHȦHFtions 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(²,PPHGLDWHGHȦHFWLRQGXHWRLYHORDG PHG GHȦHFWLRQGXHWR YHORDG 6WHS$OORZDEOHGHȦHFWLRQ 24.2.2 $&,FRGHOLPLWIRULPPHGLDWHGHȦHFWLRQGXHWROLYH IR PHGLDWHGHȦHFWLR GXHWROLYH load is ℰ/360. where ℰ is the slab diagonal as recommended by $&,5³&RQWURORI'HȦHFWLRQLQ&RQFUHWH Structures": A = A 2x + A 2y A = (18 ft) 2 + (14 ft) 2 = 22.8 ft (22.8 ft)(12 in./ft) = 0.78 in. > Δ L,midspan = 0.30 in. OK 360 24.2.2 Assume that the attached nonstructural elements supported by the slab will not be damaged due to H[FHVVLYHGHȦHFWLRQ$GHTXDWH gap should be provided between top of wall and bottom of slab. Therefore, use ACI Code limit ℰIRUORQJWHUPGHȦHFWLRQ occurring after attachment of nonstructural elements. This value is the sum of the time-dependent GHȦHFWLRQGXHWRDOOVXVWDLQHGORDGVDQGWKHLPPHGLDWHGHȦHFWLRQGXHWRDQ\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 6WHS7RWDOGHȦHFWLRQ

Step 8. American Concrete Institute – Copyrighted © Material – www.concrete.org Strut-and-Tie Critical strut effective stress controls. 178 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 14: 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. E2.5a). From Step 4, effective node and strut stresses are: fce = 2550 psi Bottle-shaped strut: fce = 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 Fig. E2.5. Node E2 of the N4 geometry. Fig. 23.4.1(a) 23.4 (4) ... Since the 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) ... ws = 2.2 in. Strut S3 in. ...

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 Ace Strut S2 provide a good geometry of Node N4. Is available at node: ... Node N1 is a CCT node. Use 90-degree hook bars: ... 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 ...

American Concrete Institute – Copyrighted © Material – www.concrete.org 180 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 Assume No. 4 reinforcement at each face over full depth of beam: 0.4 in.2 = 8 in. ... Maximum allowable vertical reinforcement: ... Use No. 4 at 8 in. on center. U 23.5.3 Check if sum of horizontal and vertical reinforcement exceeds the minimum required 0.003. ... 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 ... Use No. 4 stirrups at 8 in. on center ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 181 Step 16: Reinforcement layout ... Fig. E2.8—Detailing of reinforcement ...

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 acting at 5 in. from the face of the column. Column loads from the aforementioned structure are ignored in this example. Determine the required cross section of corbel and reinforcement. Given: D L H h b1 bc2 fy f'c = = = = = = = 22.0 kip 28.0 kip 9.0 kip 16 in. 16 in. 5000 psi 60,000 psi Bearing plate size: 6 x 12 in. Fig. E3.1—Geometry and loading of one-sided corbel. ACI 318-14 Step 1: Required strength 5.3.1 U = 1.2D + 1.6L Discussion Calculation U = 1.2(20.0 kip) 1.2 0 kip) + 1.6(28.0 kip) Vu = U = 68.8 kip ... Nu = 14.4 kip ...

Step 2: Effective stresses Determine the effective concrete compressive strength in struts: 23.4.3 fce by Table 23.4.3(a) ... Determine the effective concrete compressive strength in nodes: fce by Eq. 23.9.2 ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 183 ... The width of Strut S3 is controlled by the smaller effective stress of Strut S3 and Node N3 ... Step 6: Truss forces ... Forces in the truss members: Step 7: Strut and tie capacities ... Tie T1 ... Ast,prov = 0.62 in.2 ...

188 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 8: Node N3 Widths of Node N3 at Struts S1 and S2 ... Step 9: Node N1 ... Step 10: Node N4 ... Step 11: Node N2 ...

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 ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 191 ... Step 12: Minimum reinforcement ... Step 13: Reinforcement detailing ... Fig. E3.4—Reinforcement details.

Strut-and-Tie Example 4: Design of double corbel ... Design a double corbel supporting precast beams ... Fig. E4.1—Double corbel geometry and loading ...

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 in line with Struts S6 and S7 ... Fig. E4.4—Double-corbel strut-and-tie model ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 197 ... Step 12: Reinforcement detailing ... Fig. E4.5—Double-corbel reinforcement based on strut-and-tie design.

Strut-and-Tie Example 5: Design a pile cap supported on four piles ... Given: Case I: D = 550 kip L = 275 kip ML = 0 ft-kip Case II: D = 550 kip L = 175 kip MD = 200 ft-kip ML = 200 ft-kip ... Pile cap: ... Fig. E5.1—Pile cap subjected to axial and moment forces ... ACI 318-14 Discussion Step 1: Calculate ultimate load and reaction force ... Case I: Pu = 1.2D + 1.6L ... Step 2: Pile reaction summary ... Step 3: Effective stresses ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 201 ... Step 5: Strut-and-tie model ... Step 6: Nodes ... Step 7: Struts ... Step 8: Ties ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 14—STRUT-AND-TIE MODEL 208 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Calculate forces in strut-and-tie model ... Step 11: Reinforcement layout ... Fig. E5.9—Pile cap depth and reinforcement as obtained from sectional design ...

CHAPTER 15—ANCHORING TO CONCRETE 15.1—Introduction Anchors are used to transmit loads to concrete ... Cast-in anchors ... Post-installed anchors ... Chapter 17 of ACI 318-14 ... 15.2—Materials ... 15.3—Design limits ... 15.4—Design strength ... 15.5—Tension strength ... 15.5.1 Steel strength ... 15.5.2 Concrete breakout strength ... 15.5.3 Pullout strength ... 15.5.4 Side-face blowout strength ... 15.5.5 Bond strength ... 15.6—Shear strength ... 15.6.1 Steel strength ... 15.6.2 Concrete breakout strength ... 15.6.3 Concrete pryout strength ... 15.7—Interaction of tensile and shear forces ... 15.8—Required edge distances, spacings, and thicknesses to preclude splitting failure ... 15.9—Seismic design ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE ... Anchorage Example 1: Baseplate anchors not subjected to shear force ... Anchorage Example 2: Cast-in headed anchor in Seismic Design Category D, subjected to tension only ... Given: ... ACI 318-14 Discussion Calculation Step 1: Anchor steel strength ... Step 2: Concrete breakout strength ... Step 3: Pullout strength ... Step 4: Side-face blowout ... Step 5: Steel strength in tension ... Step 6: Concrete breakout strength of anchor in tension ... Step 7: Pullout strength of anchor in tension ... Step 8: Concrete splitting failure 17.7.2 For cast-in anchors not close to an edge

17.3.3c(ii) Nominal concretee br breakout strength from Eq. (17.4.2.1a): ad bolt olt without suppl men For a cast-in headed supplementary on B applies: reinforcement, Co Condition N b = (24)(1.0) ( ) 3000 psi (5.33 in.)1.5 = 16,210 lb Nb = (1 (1.0)(16,210 lb) = 16,210 lb Anchorage 17.4.2.5 263 Nccb = (0.6 (0.66)(0.85)(1.0)(1.0)(16,210 5)(1.0)(1.0 lb) = 9090 lb Ncb OE OE OE§OE 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 ȥc,PNp 17.4.3.6 17.4.3.4 (17.4.3.1) ȥc,P – factor to modify pullout strength of anchor in cracked concrete ȥc,P = 1.0 The basic pullout strength Np for a cast-in anchor is related to the load at which concrete crushing occurs under the anchor head. To determine basic pullout strength, either calculate by Eq. (17.4.3.4) Np = (8)Abrgfcg Np = (8)(0.454 in.2)(3000 psi) = 10,896 lb 17.3.3c(ii) where Abrg is obtained from Table 1b: From Table 1b: Abrg = 0.454 in.2 For a cast-in headed bolt without supplementary lies: reinforcement, Condition B applies: N Npn OE OE§OE ength iis greater than required Check that design strength 76 0 lb b > Nua = 11935 lb OK 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 idenot apply. pply dered blowout must bee considered. Step 9: Tension force summary ACI 318 17.4.1 17.4.2 17.4.3 17.4.4 Failure mode Steel Concrete breakout Concrete pullout Side-face blowout Design strength, lb Nsa 9800 Ncb 6300 Npn 7600 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 1.9fya 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. (17.5.1.2a) 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) lb) = 4090 lb ≅ 4100 lb Vsa = (0.65)(629 n stre Check that design strength is greater than required strength: sa = 41 4100 lb > Vua = 14 1440 lb V American 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\WKHERWWRPFHQWHUOLQHDQGWKHWRSRI WKHEHDPDQGWRSURSDJDWHDZD\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 c ,V ψ h ,V Vb AVco (17.5.2.1a) Fig. E8 E8.4—Idealized 4 alized shear shea breakout. 'H¿QLWLRQ HFW UIDFHDUHDOLPLWHGE\WKHDUHD AVcoLVWKHSURMHFWHGVXUIDFHDUHDOLPLWHGE\WKHGLVWDQFHVDDUH 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 ȥed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc =1 1.0 0 AVco ca1 = 4 in. and ca2 = 6 in., therefore ca2 = 6 in. = 1.5ca1 = 1.5(4 in.) Therefore ȥed,V = 1.0 17.5.2.7 ȥc,V ±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. ȥc,V = 1.0 17.5.2.8 ȥh,V±PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth. ha > 1.5ca1 ha = 16 in. > 1.5ca1 LQWKHUHIRUHȥh,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 ⎟ λ a ⎜⎝ ⎝ d a ⎠ ⎟⎠ f c′(ca1)1.5 ⎛ Ɛe = hef LQDQG"da = 8(0.625) = 5 in. Vb = 9λ a 17.5.2.1 Vb = 3675 lb 0.2 (0.625 in.)(3000 psi)(1.0)(4 in.)1.5 Controls 1.5 (17.5.2.2b) Vb = (9)(1.0)(3000 psi)(4 in.) = 3943 lb Solve Eq. (17.5.2.1a): Vcb = 17.3.3c(i) f c′(ca1)1.5 5 in. ⎞ (17.5.2.2a) Vb = 7 ⎜⎝ ⎟ 0.625 in.⎠ AVc ψ ed ,V ψ c,V ψ h,V Vb AVco (17.5.2.1a) Vcb = (1.0)(1.0)(1.0)(1.0)(3675 lb) = 3675 lb For a cast-in headed bolt in concrete without supplementary reinforcement, Condition B applies: Vcb = (0.7)(3675 lb) = 2573 lb ≅ 2500 lb V ength is greater than required Check that design strength is greater than required strength: Vcb = 25 Step 12: Pryout ors can pryout on the 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 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•LQlkcp = 2.0 (2 0 lb) = 18,180 lb Vcp = (2.0)(9090 5HGXFWLRQIDFWRUIRUFDVWLQDQFKRU Vcp = (0.7)(18,180 lb) = 12,730 lb ≅ 12,700 lb Check that design strength is greater than required Vcp = 12,700 lb > Vua = 1440 lb OK strength: Step 13: Splitting failure 17.7.4 Splitting needs to be checked if torque is applied 6(0.625) = 3.75 in. < 4.0; therefore, splitting is to the anchor, and if the edge distance is less than precluded. 6da. Step 14: 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 Vua/Vn" 1440 lb/2500 lb = 0.58 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.2 Check if Nua/Nn" 1935 lb/6300 lb = 0.31 > 0.2 Therefore, shear-only design strength is not permitted. 17.6.3 Check if N ua Vua + ≤ 1.2 φN n φVn 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-in hooked, post-installed mechanical, and postinstalled adhesive anchors. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 269 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 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: Anchorage Loads— WV = ±2160 lb vertical wind WH = ±1440 lb lateral wind D = 250 lb dead gravity Lr = 250 lb live gravity EV = ±250 lb vertical seismic EH = ±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 Embedment depth hef = 6.5 in. el tto force) ce) Edge distance ca1 = 4 in. (parallel Edge distance ca2 = 6 in. (perpendicular to force) Concrete— fcg SVL Ȝa = 1.0 (normalweight concrete) h = 16 in., b = 12 in. Fig. E9.1—Roof 1—Roof truss supported by concrete beam. 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 + 1.0W U = 0.9D + 1.0E Calculation (5.3.1a) (5.3.1b) (5.3.1c) (5.3.1d) (5.3.1e) (5.3.1f) (5.3.1g) Equations (5.3.1a), (5.3.1b), (5.3.1e), and (5.3.1g) do QRWUHVXOWLQWKHDGHVLJQFRQWUROOLQJWHQVLRQ¿FDQWYDOXHVWKDQ WKHWHQVLRQJLYHVWKHPD[LPXPWHQVLRQ¿FDQWYDOXHWKDQ tension. U = 1.2(250 lb) + 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 Negative indicates tension force (upward). Equations (5.3.1a), (5.3.1b), (5.3.1c), (5.3.1), and J GRQRWUHVXOWLQWKHDGHVLJQFRQWUROOLQJVKHDU¿FDQW shear force. Equations (5.3.1d) and (5.3.1f) result in the same shear. /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 strength is 1440 lb requirements of 17.2.3 do not apply. Required shear Step 2: Strength inequalities 17.3.1.1 The anchor design stren strengths must satisfy the alitie following inequalities: ⎧φN sa (steel ee strength ngth in tension) ⎪ N ua ≤ ⎨φN cb (concrete on brea breakout) ⎪φN (anchor ullout) ⎩ pn ch pullout) and Nua = 193 1935 lb ⎧φVsa (steel strength in shear) ⎪ Vua ≤ ⎨ φVcb (concrete breakout) ⎪ φV (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 φN n φVn (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 &KDSWHUGXFWLOLW\GH¿QLWLRQ 14% elongation, and 30% minimum area reduction 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\0LQLPXmum 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(ii) 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 1.9fya and 125,000 1.9f 9fya = (1.9)(36,000 psi) = 68,400 psi 9fya < 1125,000 psi OK futa < 1.9f For a 5/8 in. A307 Grad Grade 36 anchor, the nominal tensile strength Nsa iis obtained from Table 3: Tablee 3: Nsa = 13,108 lb Use reduction fac factor forr ductile anchor anchors: trength is grea Check that steel des design strength greater than required strength: 108 13,108)(0.75) OE OE§OE Nsa = (0 Nsa = 9800 lb > Nua = 1935 lb American 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 in tension: N cb = 17.4.2.3 ANc ψ ed , N ψ c , N ψ 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 shalORZHU¿FWLWLRXVHIIHFWLYHGHSWKhgef, limited by the largest of the three edge distances divided by 1.5 (see Fig. E9.2). Fig. E9.2—Cast-in anchor with three edge distances less than 1.5 1.5hef. /15 = 5.33 in. in. < 6.5 6. in. hgef = 8/15 HLV 8 in. in /DUJHVWHGJHGLVWDQFHLV 'H¿QLWLRQ WKH WHLDLOXUHDUH ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRID single anchor, limited by distances (see Fig. single anchor, limited by distances (see E9.2): 5 × 5.33 iin.)(6 in. + 1.5 × 5.33 in.) ANNc = (4 iin. + 1.5 ANc = 168 in.2 hgef) (6 in. + 1.5h 1.5hgef) ANc = (ca1 + 1.5hgef 'H¿QLWLRQ ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRI single anchor, not limited by edge distance or spacing (Fig. E9.3): 17.4.2.1 ANco = 9(hgef)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 The anchor is located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore reduced through the factor ψed,N. Fig. E9.3—Idealized tension breakout. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 17.4.2.5 ȥed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUV edge effect; ca,min < 1.5hgef ψ 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) ψ ed , N = 0.7 + 0.3 ⎜ = 0.85 ⎝ 1.5(5.33 in.) ⎟⎠ ȥc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQ at service load levels; assume member is cracked DQGFUDFNLQJLVFRQWUROOHGE\AH[XUDOFUDFNLQJ ȥc,N = 1.0 ȥcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting. ȥcp,N = 1.0 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) ( ) 3000 psi (5.33)1.5 = 16, 210 lb 17.4.2.1(a) 17.3.3c(ii) Nominal concretee br breakout strength from Eq. (17.4.2.1a): ead bolt olt without suppl men For a cast-in headed supplementary on B applies: reinforcement, Co Condition Anchorage rom test results The constant, kc, was determined from ZDVDHGMXVWHGIRU LQXQFUDFNHGFRQFUHWHDQGZDVDGMXVWHGIRU cracked concrete. Nccb = (0.6 (0.66)(0.85)(1.0)(1.0)(16,210 5)(1.0)(1.0 lb) = 9090 lb ∅Ncb OE OE OE§OE 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 ȥc,PNp (17.4.3.1) Fig. E9.4—Concrete bearing at hook. 17.4.3.6 ȥc,P – factor to modify pullout strength of anchor in cracked concrete. 17.4.3.5 Check if hook tail length conforms to ACI 318 limits: 3da"eh"da ȥc,P = 1.0 (3)(0.625 in.) = 1.875 in. < eh = 2.8 in. < (4.5)(0 (4.5)(0.625 in.) = 2.81 in. OK To determine basic pullout ullout strength, s either calcu3.5) late by Eq. (17.4.3.5) Np = (0.9 (0.9)(4000 psi)(2.8 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 side-face blowout must be considered. Step 9: Tension force summary ACI 318 17.4.1 17.4.2 17.4.3 Failure mode Steel Concrete breakout Concrete pullout ∅ ∅E§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 ∅Ncb 6300 ∅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 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. 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 1.9fya 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. (17.5.1.2b) or obtain from Table 3. 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) ∅lb) = 4090 lb ≅ 4100 lb ∅Vsa = (0.65)(629 n stre Check that design strength is greater than required strength: ∅ sa = 41 4100 lb > Vua = 14 1440 lb ∅V American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage 17.5.1.2 276 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 11: Concrete breakout A shear breakout failure is assumed to initiate at a SRLQWGH¿QHGE\WKHERWWRFHQWHUOLQHDQGWKHWRSRI WKEHHDPDQGWRSURSDJDWHDZD\IURPWKHHGJH 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 UIDFHDUHDRIWKHDVVXPHG WRDV AVcoLVWKHSURMHFWHGVXUIDFHDUHDRIWKHDVVXPHGWRDVKHDU 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 ȥed,V±PRGL¿FDWLRQIDFWRUHGJHHIIHFWVIRU anchors loaded in shear; check if ca2•ca1 AVc = 1.0 .0 AVco ca1 = 4 in. and ca2 = 6 in., therefore ȥed,V = 1.0 17.5.2.7 ȥc,V±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load level; assume member is cracked and not detailed with supplementary reinforcement. ȥc,V = 1.0 17.5.2.8 ȥh,V±PRGL¿FDWLRQIDFWRUIRUPHPEHUWKLFNQHVVLQ relation to anchor embedment depth. ha > 1.5ca1 ha = 16 in. > 1.5ca1 = 6 in., therefore ȥh,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 ⎟ λ a ⎜⎝ ⎝ d a ⎠ ⎟⎠ ⎛ ⎛ 5 in. ⎞ 0.2 ⎞ 0.625 in. ⎟ (3000 psi)(1.0)(4 in.)1.5 ⎟⎠ ⎝ 0.625 in. ⎠ f c′(ca1)1.5 (17.5.2.2a) Vb = ⎜ 7⎜ ℰe = hef LQDQG"da = 8(0.625) = 5 in. Vb = 9λ a 17.5.2.1 f c′(ca1)1.5 Vb = 3675 lb Controls 1.5 (17.5.2.2b) Vb = (9)(1.0)(3000 psi)(4 in.) = 3943 lb Solve Eq. (17.5.2.1a): Vcb = 17.3.3c(i) 277 AVc AVco (17.5.2.1a) Vcb = (1.0)(1.0)(1.0)(1.0)(3672 lb) = 3672 lb ψ ed ,V ψ c ,V ψ h ,V Vb For a cast-in headed bolt in concrete without supplementary reinforcement, Condition B applies: ∅ ∅Vcb = (0.7)(3672 lb) = 2570 lb ≅ 2500 lb Check that design strength is greater than required Vcb = 2500 lb > Vuua = 14 Step 12: Pryout ors can pryout yout on the side si opposite pposite Short stiff anchors n. The pryout ryout 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•LQlkcp = 2.0 17.3.3c(i) 5HGXFWLRQIDFWRU∅RUFDVWLQDQFKRU ∅Vcp = (0.7)(18,180 lb) = 12,730 lb ≅ 12,700 lb Check that design strength is greater than required ∅Vcp = 12,700 lb > Vua = 1440 lb OK strength: Step 13: Splitting failure 17.7.4 Splitting needs to be checked if torque is applied 6(0.625) = 3.75 in. < 4.0; therefore, splitting is to the anchor, and if the edge distance is less than precluded. 6da. Step 14: 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 ∅Vcp 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 — 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. 17.6.3 Check if N ua , g φN n + Vua , g φVn ≤ 1.2 1935 lb 1440 lb + = 1.02 ≤ 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 adhesive anchors. American Concrete Institute – Copyrighted © Material – www.concrete.org 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 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 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 and roof live load of 250 lb, and a seismic force 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 = ±250 lb vertical seismic EH = ±150 lb lateral seismic Concrete— fcg SVL Ȝa = 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 Anchorage Anchors— or 5/8 in. post-installed expansion anchor ASTM F1554 Grade 55; Table 1a: • futa = 75,000 psi • fya = 55,000 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) (5.3.1d) (5.3.1e) (5.3.1f) (5.3.1g) Equations (5.3.1a), (5.3.1b), (5.3.1e), and (5.3.1g) do QRWUHVXOWLQWKHDQFKRUUHVLVWLQJVLJQL¿FDQWVYHUWLFDO tension. U = 1.2(250 lb) + 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 Negative indicates tension force (upward). Equations (5.3.1a), (5.3.1b), (5.3.1c), (5.3.1e), and J GRQRWUHVXOWLQWKHDQFKRUUHVLVWLQJVLJQL¿FDQW shear force. Equations (5.3.1d) and (5.3.1f) result in the same shear. /DWHUDO U = 1.0W (5.3.1d) U = 1.0(1440 lb) = 1440 lb Structure is assigned to SDC A; therefore, seismic requirements of 17.2.3 do not apply. Required tension strength is 1935 lb Required shear strength is 1440 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: ⎧φN sa (steel strength in tension) ⎪ N ua ≤ ⎨φN cb (concrete breakout) ⎪φN (anchor pullout) ⎩ pn and Nua = 1935 lb ⎧φVsa (steel strength in shear) ⎪ Vua ≤ ⎨φVcb (concrete breakout) ⎪φV (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 φN n φVn (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 &KDSWHUGH¿QLWLRQRIGXFWLOLW\GH¿QLWLRQ 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

[...]

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 ⎟ λ a ⎜⎝ ⎝ d a ⎠ ⎟⎠ f c′(ca1)1.5 ⎛ (17.5.2.2a) Vb = ⎜ 7 ⎛⎜ 5 in. ⎞⎟ 0.2 ⎞ 0.625 in. ⎟ For an expansion anchor, ℰe = hef and ℰe"da Vb = 9λ a 17.5.2.1 297 f c′(ca1)1.5 0.2 ⎞ 0.625 in. ⎟ (1.0) 3000 psi(4 in.)1.5 ⎠ ℰe = hef = 6.5 in. and 8da = 8(0.6.25) = 5 in. Vb = 3675 lb Controls (17.5.2.2b) Vb = (9)(1.0) (Controls) 3000 psi (4 in.)1.5 = 3943 lb Solve Eq. (17.5.2.1.a): Vcb = AVc ψ ed ,V ψ c ,V ψ h ,V Vb AVco (17.5.2.1b) Vcb = (1.0)(1.0)(1.0)(1.0)(3672 lb) = 3672 lb For an adhesive anchor in concrete without supplementary 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 related nt ddepth heff and to the ssmaller to the embedment er kou strength ngth Ncbb and ad adhesive sive of anchor breakout strength Na. 17.5.3.1 Vcp = kcpNcp (17.5.3.1a) 7.5.3.1a) hef LQ•LQ⇒ kcp = 2.0 Ncp = smaller of [Ncb, Na] 17.4.2.1 N cb = ANc ψ ed , N ψ c , N ψ cp , N N b ANco (17.4.2.1a) Ncb = 6437 lb 17.4.5.1 Na = ANa ψ ed , Na ψ cp , N a N ba ANao (17.4.5.1a) Na = 6009 lb 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 = (2.0)(6009 lb) = 12,018 lb ∅Vua ∅Vcp = (0.7)(12,018 lb) = 8413 lb ≅ 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 pryout Step 14: Shear and tension interaction 17.6.1 Check if Vua∅Vn" Design strength, lb ∅Vsa 5300 ∅Vcb 2500 ∅Vcp 8400 Ratio = Vua∅Vn 0.27 0.58 0.16 Controls design? No Yes No 1440 lb/2500 lb = 0.58 > 0.2 Therefore, tension-only design strength is not permitted. 17.6.2 Check if Nua∅Nn" 1935 lb/3900 lb = 0.5 > 0.2 Therefore, shear-only design strength is not permitted. 17.6.3 Check if N ua + Vua ≤ 1.2 φN n φVn 1935 lb 1440 lb + = 1.08 ≤ 1.2 3900 lb 2500 lb OK Step 15: Conclusion The shear and tension strength of a 5/8 in. F1554 Grade 36 diameter post-installed adhesive anchor embedded 6.5 in. in concrete exceeds the required strength of 1935 35 lb ultimate tension and 1440 lb uultimate shear forces. The applied forces (tension and shear), ear), aanchor 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 system. strengths vary depending on thee an anchoring Tension: Co kout Concrete breakout eel strength, lb strength, lb llout stre Steel Pullout strength/adhesive strength, lb 9800 Cast-in place headed anchor 7600 6300 630 Cast-in-place hooked anchor 9800 6300 4400 Post-installed expansion anchor 12,700 9200 4100 Post-installed adhesive anchor 9800 4100 3900 Shear: Concrete breakout Steel strength, lb strength, lb Concrete pryout strength, lb 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 concentric tensile force A 1/2 x 10 x 16 in. steel plate is embedded to a 24 in. reinforced concrete wall with fcg SVL7KHVWUXFWXUHLVQ6HLVPLF 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 1600 lb applied at the center of the stud group. Supplemental reinforcement is not detailed. Check the adequacy of the connection. Given: Anchors— 1/2 in. diameter cast-in 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 diameter da = 1 in. n. 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. E12.1 —Groups E1 1— of studs resisting concentric tensile force. Concrete— fcg SVL Concrete wall thickness ha = 24 in. Ȝa = 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: ‡/RDGVDUHWUDQVPLWWHGWKURXJKDQDGHTXDWHO\VWLIISODWHWR individual anchors; ‡7KHDQFKRUSODWHLVQ¿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 Ȝe – 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 to 3/16 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 tensi • Concrete breakout in tension: Critical spacing = 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 ⎧φN sa (steel strength iin tension) ⎪ φN (concrete breakout) ⎪ cbg ≤⎨ ⎪ φN pn (anchor pullout) ⎪ ⎩ φN 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 studs 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 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∅DFWRULQWHQVLRQDQGVKHDU 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. 20% elongation in 2 in. of length > 14%, and 50% 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: Use reduction factor for ductile anchors: Check that futa = 65,000 psi (Table 1a) is smaller than 1.9fya and 125,000 psi. For a 1/2 in. A29 anchor, the nominal tensile strength Nsa is obtained from Table 2: Use reduction factor for ductile anchors: Check that steel design strength is greater than required strength: 1.9fya = 1.9(51,000 psi) = 96,900 psi futa < 1.9fya < 125,000 psi OK Nsa = 12,740 lb ∅ ∅Nsa = (0.75)(12,740 lb) = 9600 lb ∅Nsa = 9600 lb > Nua,g = 21,600 lb/6 = 3600 lb OK Anchorage 17.3.3.a(i) da = 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 Nominal 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 ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIWKHVWXG group, limited by edge distance (see Fig. E12.3): The three edge anchorss with hef = 6.5 in. are located Fig. E1 E12.3—Idealized .3—Idealized bbreakout of the stud group. m the free edge. Therefore, the less than 1.5hef from breakout area is shown in Fig. E12.3. 5heff = 1.5(1.5(6.5 in.) = 9.75 in. ca1 = 3 iin. < 1.5h 5 in. + 9. 9.75 in.)(9.75 in. + 6.5 in. ANNc = (3 in. + 6.5 (1.5hef + s2 + s2 + 1.5hef) ANc = (ca1 + s1 + 1.5hef)(1.5h in + 6.5 in. + 9.75 in.) 62 in.2 ANNc = 626 HG FUHWHIDLOXUHDUH IDVLQJOH ANcoLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIDVLQJOH 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 < (6 (6)(380 in.2) = 2280 in.2 OK ANc 626 in.2 = = 1.65 ANco 380 in.2 ȥec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG eccentrically in tension; load is applied concentric, ȥec,N = 1.0 ec = 0,therefore: 17.4.2.5 ȥed,N±PRGL¿FDWLRQIDFWRUIRUHGJH effect; three anchors are located close to the edge, without enough space for a complete breakout prism to develop. The breakout strength is therefore UHGXFHGWKURXJKWKHIDFWRUȥed,N. ψ ed , N = 0.7 + 0.3 ca,min 1.5hef (17.4.2.5b) ψ ed , N = 0.7 + 0.3 3 in. = 0.792 (1.5)(6.5 in.) 17.4.2.6 ȥc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. ȥc,N = 1.0 17.4.2.7 ȥcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors in uncracked concrete without supplementary ȥcp,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. Nominal 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 ȥc,PNp 17.4.3.6 Ncbg = (1.65)(1.0)(0.793)(1.0)(1.0)(25,154 lb) Ncbg = 32,913 lb ∅ ∅Ncbg OE OE§OE ∅Ncbg = 23,000 lb > Nua,g = 21,600 lb OK (17.4.3.1) modify pullout llout strength of anchor nchor in ȥc,P – factor to mod te. ȥc,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 rength bearing of the anchor at actual pullout. 17.4.3.4 m Eq. The basic pullout strength is calculated from (17.4.3.4) Np = (8)Abrg fcg 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: ∅ ∅Np = (0.7)(1.0)(18,848 lb) = 13,194 lb/stud ∅Np ≅ 13,200 lb/stud ∅Np OE•Nua,g = 21,600 lb/6 = 3600 lb/stud OK Step 9: Concrete side-face blowout 17.4.4 Nominal 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 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 breakout/group 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 ([DPSOHVDQGUHSUHVHQWDQDQFKRUDJHFRQVLVWLQJRIVL[VWXGVDQGSRVWLQVWDOOHGDGKHVLYHDQFKRUVVXEMHFWHGWRDFRQFHQWULFWHQVLOH 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 spaced at 6.5 in. centers in both directions (Fig. E13.1). A service tensile force of 7500 lb and a seismic force of 700 lb is applied at the center of the stud group. The structure is in Seismic Design Category (SDC) A. Supplemental reinforcement is detailed. Check the adequacy of connection. Given: Loads— L = 7500 lb (service tensile force) E = 700 lb (seismic force) Concrete— fcg SVL Concrete wall thickness ha = 24 in. te) Ȝa = 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 mbination is: 5.3.1 The controlling load U = 1.6L Calculation Cal 7500 lb) = 12,000 lb (tension) U = (1.6) (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: ‡/RDGVDUHDSSOLHGWKURXJKDQDGHTXDWHO\VWLIISODWHWR individual anchors; ‡7KHDQFKRUSODWHLVQ¿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 must be less than the critical 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 = 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 Edge distance ca1 = 3 in. Stud spacing s = 6.5 in. Embedment depth hef = 6.5 in. 306 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: N ua , g ⎧φN sa (steel strength intension) ⎪ ≤ ⎨ φN cbg (concrete breakout) ⎪ φN (adhesive bond) ⎩ ag Nua,g = 12,000 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: smin = 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 WKHDSSURSULDWH∅DFWRULQWHQVLRQDQGVKHDU 2.2 ca = 3 in. (provided) > 1.5 in. minimum cover (Code) Check if the properties of F1554 Grade 36 meet the H¿QLWLRQ $&,&KDSWHUGH¿QLWLRQRIGXFWLOLW\ZKLFKDUH Table 1a: ASTM F1554 Grade 36 has the following erties: properties: 14% elongation, and uction. 30% minimum are 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 (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 from Table 3: Check that futa = 58,000 psi (Table 1a) is smaller than 1.9fya and 125,000 psi. For a 1/2 in. F1554 Grade 36 anchor, the nominal tensile strength Nsa is obtained from Table 3: 17.3.3.a(i) Use reduction factor for ductile anchors: Check that steel design strength is greater than required strength: da = 1/2 in. Ase,N = 0.142 in.2 1.9fya = 1.9(36,000 psi) = 68,400 psi futa < 1.9fya < 125,000 psi OK Nsa = 8236 lb/anchor ∅ ∅Nsa = (0.75)(8236 lb) = 6177 lb ≅ 6100 lb ∅Nsa = 6100 lb > Nua,g = 12,000 lb/6 = 2000 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 ψ ec , N ψ ed , N ψ c , N ψ 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 d 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 in.) = 9.75 in. breakout area iss sh shown in Fig. E13.2. 'H¿QLWLRQ)(1.5hef + s2 + s2 + 1.5h 1.5 ef) ANc = (ca1 + s1 + 1.5hef)(1.5 (9.75 in. + 6.5 in. + 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.)(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 anchors in a group. ANc 626 in. 2 = = 1.647 AN co 380 in. 2 17.4.2.4 ȥec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG eccentrically in tension; ec = 0; load is applied concentric, therefore ȥec,N = 1.0 concentrically. 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 ψ ed , N = 0.7 + 0.3 17.4.2.6 17.4.2.7 ca , min 1.5hef (17.4.2.5b) ⎛ 3 in. ⎞ ψ ed , N = 0.7 + 0.3 ⎜ = 0.792 ⎝ 1.5(6.5 in.) ⎟⎠ ȥc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked and cracking is controlled by reinforcement. ȥc,N = 1.0 ȥcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting. ȥcp,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 λ a f c′hef 1.5 (17.4.2.2a) N b = (17)(1.0)(4000 psi)(6.5 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 anchor with supplementary reinforcement, Condition A, Category 1 applies: Check that design strength is greater than required strength: Ncbg = (1.647)(1.0)(0.793)(1.0)(1.0)(17,818 lb) Ncbg = 23,272 lb ∅ ∅Ncbg = (0.75)(23,272 lb) = 17,454 lb ≅ 17,400 lb ∅Ncbg = 17,400 lb > Nua,g = 12,000 lb American Concrete Institute – Copyrighted © 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): 'H¿QLWLRQ ANa ψ ec , Na ψ ed , Na ψ cp , Na N ba ANao (17.4.5.1b) WHG FHDUHDRIJ IDG ANaLVWKHSURMHFWHGLQÀXHQFHDUHDRIDJURXSRIDGKHVLYHDQFKRUVGulation 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. + 6.5 in. + 7.13 in.) × (3 in. + ANa = (7 6 in. + 7.13 in 6.5 in.) = 435 in.2 AnaoLVWKHSURMHFWHGHUHQGLQÀXHQFHDUHDRIDVLQJOH 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 = (2cNa)2 17.4.5.3)LJ(²,GHDOLJHGERQGLQÀXHQFHDUHDRIWKHVLYH (,GHDOLJHG E hor group. p anchor ANa shall not be taken greater than nANao, where n = number of anchors in a group. 453 in.2" LQ2) = 1218 in.2 ȥec,Na±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension; ec = 0 load is applied concentric, therefore: ȥec,Na = 1.0 ȥed,Na±PRGL¿FDWLRQIDFWRUIRUHGJHHIIHFWVIRUDQFKRU edge effect; three anchors are located close to the edge, without enough space for a complete proMHFWHGLQÀXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK LVWKHUHIRUHUHGXFHGWKURXJKWKHIDFWRUȥed,N, which can be calculated from Eq. (17.4.5.4b). ψ ed , Na = 0.7 + 0.3 ca,min cNa (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 ȥcp,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¿FDWLRQIDFWRULVDSSOLHGWRSUHFOXGHEUHDNWKHOWVSOLWWLQJ failure. In this example, concrete is cracked with VXI¿FLHQWUHLQIRUFHPHQWWRUHVWUDLQFUDFNZLGWKV therefore: ȥcp,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 ȜaIJcrᴧdahef (17.4.5.2) Nba SVL ᴧLQ LQ OE ZKHUHIJcr = 1030 psi is obtained from test data; see Table E.2. 17.4.5.1 17.4.5.1b): Nominal bond strength from Eq. (17.4.5.1b): N ag = 17.3.3.c(ii) ANa ANao ψ ec , Na ψ edd , NNa ψ cp , Na N ba (17.4.5.1b) VV LHGE\WKHPDQX FWXUHU¶V 7KH∅DFWRUZDVVSSOLHGE\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))(0 8 2203 in. n.2 Nag = 19 19,383 lb b N agg = OH(7DEOH(19,38)(65 ∅lb) = 12,599 lb ≅ 12,600 lb ∅Nag = (0.65)(19,383 ∅ ag = 12,600 lb > Nua,g = 12,000 lb ∅N Design strength, lb ∅Nsa 6100 ∅Ncbg 17,400 ∅Nag 12,600 Ratio = Nua,(g)∅Nn 0.33 0.69 0.95 OK Controls design? No No Yes Step 10: Conclusion ([DPSOHVDQGUHSUHVHQWDQDQFKRUDJHFRQVLVWLQJRIVL[VWXGVDQGSRVWLQVWDOOHGDGKHVLYHDQFKRUVVXEMHFWHGWRDFRQFHQWULFWHQVLOH 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 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 ℰe = 8-1/2 in. Stud diameter da = 5/8 in. in. Stud head thickness tns = 5/16 in Fig. E14.1—Steel bbeam anchored to reinforced concrete tuds. beams by studs. Concrete— fcg SVL Beam width b = 20 in. Ȝa = 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.) (Table 1e and Fig. E14.2). hef = ℰe + tpl – tns – (3/16 in.) 17.2.1.1 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.) – (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 ÀDQJHWRH The compression force is resisted by a concrete reaction centered directly beneath WKHVWHHOEHDPFRPSUHVVLRQÀDQJH 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 threee 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 TM = T4 = T5. .3) (Fig. E14.1 and Fig Fig. E14.3) 6XPPDWLRQRI 6XPPDWLRQRIPRPHQWVDURXQGWKHFRPSUHVVLRQÀDQJH uming elastic beh assuming behavior: 3TT) + 9(2 (TM) – 480,000 in.-lb = 0 ∑ M = 0 ⇒ 21(3 Ratio off def deformations ons betwe between the middle and top lines of d studs: ΔT 21 in in. 33 3 ⇒ TT = 2.33TM = = 2.33 9 in in. ΔM Substituting: ing: (21)(3)(2.33TM) + 9(2)TM – 480 × 103 in.-lb = 0 TM = 2909 lb and TT = (2.33)(2909 lb) = 6788 lb TT = T1 = T2 = T3 = 6788 lb TM = T4 = T5 = 2909 lb Nua = (3)(6788 lb) + (2)(2909 lb) Nua = 26,182 lb (tension) ≅ 26,200 lb Check if the plate remains elastic under bending due to STT times the distance to the tension ÀDQJHRIWKHVWHHOEHDP M elastic = S x f y = M face = ∑ TT a bh 2 fy 6 M elastic = M face = 27 in.(1.5 in.) 2 (36 ksi) = 364.5 in.-kip 6 (3)(6788 lb)(3 in.) = 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: ⎧φN sa (steel strength in tension) ⎪ N ua , g ≤ ⎨ φN cbg (concrete breakout) ⎪ φN (anchor pullout) pn ⎩ Nua,g = 26,200 lb and Vua , g 17.6.3 ⎧φVsa (steel strength in shear) ⎪ ≤ ⎨φVcb , g (concrete breakout) ⎪ ⎩φVcp , g (concrete pryout) Vua,g = 17,000 lb The interaction of tensile and shear forces must also satisfy the following inequality: N ua Vua + ≤ 1.2 φN 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 &KDSWHUGH¿QLWLRQRIGXFWLOLW\GH¿QLWLRQ a: A 29 has the following properties: Table 1a: ASTM A29 lon on and 14% minimum elongation 30% 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 strengths h Tension Step 5: Steel tension Nominal steel 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. 1.9fya = 1.9(51,000 psi) = 96,900 psi OK futa < 1.9fya < 125,000 psi For a 5/8 in. A29 stud, the nominal tensile strength Nsa is obtained from Table 2: Nsa = 19,955 lb 8VH∅DFWRUIRUGXFWLOHDQFKRUV Check that design strength is greater than required strength: ∅ ∅Nsa = (0.75)(19,955 lb) = 14,996 lb/stud ∅Nsa ≅ 14,900 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 ψ ec , N ψ ed , N ψ c , N ψ cp , N N b (17.4.2.1b) ANco Spacing: s = 12 in. and hef = 9.5 in. Because only studs 1, 2, 3, 4, and 5 are in tension, ANc will be determined based on that group of anchors only. ANco is determined for one anchor only. 'H¿QLWLRQ 17.4.2.4 1.5hef = (1.5)(9.5 in.) = 14.25 in. 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 in. + 12 iin. + 14.25 in.) × (14.25 in. ANc = (5 + 12 in. + 14.25 in.) 752 in.2 ANc = 1752 HFW 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 ȥec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSV loaded eccentrically in tension. For the anchor group the load is applied eccentrically and ec = 2.13 in. (see calculations below) ψ ec , N = 1 2e ′ 1+ N 3hef (17.4.2.4) ψ ec , N = 1 = 0.87 2(2.13 in.) 1+ 3(9.5 in.) eNgLVFDOFXODWHGWKHVXPPLQJPRPHQWVDERXW the top 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.) (2909 lb + 2909 lb)(12 in.) − = 2.13 in. 5 26,200 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) ȥed,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 WKURXJKWKHIDFWRUȥed,N. ψ 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. ⎞ ψ ed , N = 0.7 + 0.3 ⎜ = 0.805 ⎝ 1.5(9.5 in.) ⎟⎠ ȥc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWH condition at service load levels; assume member is cracked. ȥc,N = 1.0 ȥcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHG anchors in uncracked concrete without supplementary reinforcement to control splitting. ȥcp,N = 1.0 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 (24)(1.0) 4)(1)(1.0) 0) ( ) 550 500 5000 psi (9.5)1.5 = 49, 691 lb The constant, kc, was determined from tests QFU QGZDVDGMXVWHG RU LQXQFUDFNHGFRQFUHWHDQGZDVDGMXVWHGIRU te. cracked concrete. 17.4.2.1b te breakout kout strength fro m Eq. Nominal concrete from (17.4.2.1b): N cbg = ANc ψ ec , N ψ ed , N ψ c , N ψ cp , N N b (17.4.2.1b) ANco N cbg = 1752 752 in.2 (0.87)(0.805)(1.0)(1.0)(49, 691 lb) 81 812 in.2 Ncbg = 75,500 lb 17.3.3c(ii) For a cast-in headed studs with supplementary reinforcement, Condition A applies: Check that design strength is greater than required strength: ∅ ∅Ncbg = (0.75)(75,500 lb) = 56,625 lb ≅ 56,600 lb ∅Ncbg = 56,600 lb > Nua,g = 26,200 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 317 Step 7: Pullout 17.4.3 Nominal pullout strength is calculated from Eq. (D-13): Npn ȥc,PNp 17.4.3.6 (17.4.3.1) ȥc,P – factor to modify pullout strength of anchor in cracked concrete. ȥc,P = 1.0 The basic pullout strength Np for a cast-in stud is related to the load at which 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 fcg Np = (8)(0.92 in.2)(5000 psi) = 36,800 lb For cast-in headed studs with supplementary reinforcement, Condition A applies: ength iis 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 consider d for multiple headed with (hef > 2.5 d aanchors rs w 2.5ca1) aand ci less ss tha with anchor spacing than 6ca11. ∅Npn = (0.75 (0.75)(1.0)(36,800 lb) = 27,600 lb/stud ∅N 27,600 lb/ lb/stud lb/stud t d > Nua,g ≅ 6800 lb/stud ∅Npn = 27,6 OK Does heff exc exceed 2.5cca1? No, 9 9.5 in. does not exceed 12.5 in. Are the th stud ly spaced? spaced No, 6ca1= 6 × 5 in. = 30 in. studs closely stu spacing. ing. > 12 in. stud T face blo Therefore, side-face blowout is not considered. Step 9: Splitting failure 17.7.2 This type of failure can occur in thin members if 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 unlikely to be torqued. Minimum cover of ACI 318 applies. Step 10: Tension forces summary ACI 318 17.4.1.2 17.4.2.1 17.4.3.1 17.4.4.1 Failure mode Steel/stud Concrete breakout/group Concrete pullout/stud Concret side-face blowout Design strength, lb ∅Nsa 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. 1.9fya = 1.9(51,000 psi) = 96,900 psi futa < 1.9fya < 125,000 psi OK For an A29 5/8 in. stud, either calculate Ase,V futa from Eq. (17.5.1.2a) or obtain from Table 2. Table 2: Vsa = Ase,V futa = 19,955 lb 17.3.3a(ii) Strength reduction factor for ductile bolt: ∅ ∅V Vsa = (0.65)((0.65)(19,955 lb) = 12,971 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 onlyy studs (conservative). ∅V Vsa = (3 × 12,971)) = 38,913 38,91 lb ≅ 38,900 lb gn strength ngth is greater th n Check that design than h: required strength: Vsa = 38,9 38,900 lb > Vua,g = 1 17,000 lb ∅V American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 319 Step 12: Concrete breakout Studs are welded to the steel plate; therefore, VKHDULVXVHVLWHE\DFRQFUHWHEUHDNRXWSULVP shear of 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 GHWHUPLQHGWKHWRSDQGERWWRPRIWKH horizontally and vertically toward the edges. Nominal concrete breakout shear strength of an anchor group is: Vcbg = (17.5.2.1a) AVc ψ ec,V ψ ed,V ψ c,V ψ h,V Vb AVco Fig. E14.5—Idealized shear breakout The projected concrete failure area (AVc) is calculated from the line of studs 1, 2, and 3 ...

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 basic concrete breakout strength in shear of a single anchor in cracked concrete ... Vb = 7 (le/da)^0.2 √da λa √f′c (ca1)^1.5 ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE ... Step 13: Pryout ... Step 14: Shear force summary ... Steel (Anchor) Concrete breakout Concrete pryout ... Ratio = Vua/(φVn) ... Controls design? No ... American Concrete Institute – Copyrighted © Material – www.concrete.org 322 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 15: Interaction of tensile and shear forces 17.6.1 Check if Vua/φVn ≤ 0.2 ... 17.6.3 N ua/(φNn) + Vua/(φVn) ≤ 1.2 ... Anchorage ... American Concrete Institute – Copyrighted © Material – www.concrete.org OK CHAPTER 15—ANCHORING TO CONCRETE 323 Anchorage Example 15: Post-installed adhesive group anchors subjected to shear and moment A normalweight reinforced concrete girder with f′c = 5000 psi supports a steel beam via a steel plate with four 3/4 in. post-installed adhesive anchors ... ASTM F1554 Grade 36 adhesive anchors, an effective embedment depth of 12 in. ... The group of anchors ... The wall is assigned to Seismic Design Category (SDC) A. Given: Loads— Mu = ... Vu = 8500 lb factored shear Fig. E15.1—Steel beam anchored to concrete girder. Anchors— 5/8 in. post-installed adhesive anchors ASTM F1554 Grade 36; Table 1a: fy = 36,000 psi • futa = 58,000 psi ... Spacing s = 12 in. Embedment depth hef = 8 in. ...

American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 325 Step 3: Strength requirements 17.3.1.1 ... φNsa ... φNcbg ... φNa ... φVsa ... φVcbg ... φVcpg ... Step 4: ... Step 5: Anchor ductility ... ASTM F1554 Grade 36 ... 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. Step 6: Steel strength ... Nsa = Ase,N futa ... For a 5/8 in. F1554 Grade 36 anchor, the nominal steel strength Nsa is obtained from Table 3: Nsa = 13,108 lb ... φNsa = (0.75)(13,108 lb) = 9831 lb/anchor ...

American Concrete Institute – Copyrighted © Material – www.concrete.org ... 326 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 7: Concrete breakout ... Ncbg = ANc/ANco ψ ec,N ψ ed,N ψ c,N ψ cp,N Nb ... ANco = 9(hef)^2 ... ca,min ... ψ ed,N = 0.7 + 0.3 ca,min/(1.5hef) ... Nb = kc λa √f′c hef^1.5 ... Ncbg = ... φNcbg = (0.65)(53,188 lb) = 34,572 lb ... Step 8: Bond strength 17.4.5.1(b) ... ANa/ANao ψ ec,Na ψ ed,Na ψ cp,Na Nba ... cNa = 10da √(τuncr/1100) ... τuncr = 1990 psi ... 17.4.5.4 ... ψ ed,Na ... Nba = λa τcr π da hef ... φNag = (0.65)(...) ... Step 9: Summary ... American Concrete Institute – Copyrighted © Material – www.concrete.org 328 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) ... Step 10: Shear ... Vsa = 0.6 Ase,V futa ... φVsa ... Step 11: Concrete breakout in shear ... AVc ... AVco = 4.5(ca1)^2 ... Vb = 7 (le/da)^0.2 √da λa √f′c (ca1)^1.5 ... Step 12: Pryout ... Vcpg = kcp Ncpg ... Step 13: Interaction ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 331 ...

Anchorage Example 16: Cast-in headed stud group with anchor reinforcement resisting tension force ... A 3/4 in. thick steel plate with six 7 in. long x 3/4 in. diameter headed studs is welded ... edge of a wall ... Given: Loads— Nu = 20,000 lb ... Studs— Six 3/4 in. diameter AWS D1.1 Type B ... futa = 65,000 psi ... hef = ... ca1 = ... Fig. E16.1 ... Step 1: Anchor group ... Step 2: Required strength ... Step 3: Strength inequalities ... Step 4: Anchor ductility ... Step 5: Steel strength ... Nsa = 28,600 lb ... φNsa = (0.75)(28,600 lb) = 21,450 lb/stud ... Step 6: Concrete breakout ... Step 7: Anchor reinforcement ... Asfy ... ld = ... (60,000 psi)(1.0)(1.0) ... ld ≥ (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) Asfy = 0.75 × 8 × 0.11 × 60,000 psi = 39,600 lb > 32,000 lb ∴ OK (a) Example detail of anchor reinforcement (b) Example detail of anchor 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 Nominal pullout strength is calculated from Eq. (17.4.3.1): 17.4.3.1 Npn ψc,PNp 17.4.3.6 ψc,P – factor to modify pullout strength of anchor in ψc,P = 1.0 cracked concrete. 17.4.3.6 The basic pullout strength Np is 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 The basic pullout strength is either calculated from Eq. (17.4.3.4) (17.4.3.1) Np = (8)Abrgfc 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 anchor reinforcement, Condition A applies: φN Npn = (0.75)(1 (0.75)(1.0)(31,400 lb) = 23,550 lb/stud lb/st d φNpn ≅ 23,500 lb/stud 17.3.3c(ii) gn st quired Check that design strength is greater th than required strength: Step 9: Summary φ pnn = 223,500 lb/stud > Nua,g = 10,871 lb/stud φN Design esign strength strength, lb b φNsa 21,500 00 Ratio atio = Nua,(g)φNn 0.51 0 ACI 318 17.4.1 Failuree m mode Steel/stud tud 17.4.2.1 Concrete breakout/group φNcbg 32,000 — 17.4.3.1 Concrete pullout/stud φNpn 23,500 23 5 0.46 OK Controls design? Yes Reinforcement provided—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 fcq SVLVXSSRUWVDORDGDWDDFKHGWRDIWLQ[LQ[LQ thick A36 steel plate. A 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 abo eccentric tension about two axes Concrete— fcq SVL Ʒa = 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\VWLIIDFKRU 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 = 10d a tuncr 1100 LQ LQ•LQVDWLV¿HV (17.4.5.1d) 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 0LQLPXPFHQWHUWRFHQWHUVWXGVSDFLQJ•d a. LQ Q LQ LQ”LQOK smin 17.7.2 20.8.1.3.1 18 S Minimum edge ddistancee to satisfy ACI 318 Section 20.8.1. &R HU LQO &RYHULQ•LQOK 17.7.3 sta sts in Minimum edge 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 GLVWDQFHDQGDQFKRUVSDFLQJDUHQRWFULWLFDO 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 ∑ exi ∑ 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)(6.5 iin.) 2 16,000 lbb (16,000 1 (16,00 lb)(1.5 in.)(3.25 in.) + 6 (6)(3.25 in.) 2 16,000 , lb) (16,000 lb)(2.5 in.)(–6.5 in.) + = 2359 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.)(0 in.) + = 1436 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.)(0 in.) + = 3897 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 (16,000 lb)(1.5 in.)(3.25 in.) + 6 (6)(3.25 in.) 2 (16,000 lb)(2.5 in.)(6.5 in.) + = 5436 lb (T) (4)(6.5 in.) 2 American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING 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 = ∑ 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: $&,KDVWHQVLRQVWUHQJWKPRGL¿FDWLRQIDFWRUV 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 Taking a moment about a line anchors 5, 6: y= 2(6.5 in.) + 1(6.5 in. + 6.5 in.) = 5.2 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 eNxg LQ±LQ±LQ LQ± LQ tension is: eNNyg 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 ⎧φN sa (steel strength in tension) ⎪ ≤ ⎨ φN cbg (concrete breakout) ⎪ φN (bond strength) ⎩ ag Nua,g = 16,000 lb Step 5: Anchor ductility &KHFNWKHDQFKRUGXFWLOLW\WRDGHWHUPLQHWKH∅DFWRU 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\ZLWKLV 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 (futa) times the bolt area (Ase,N). 17.4.1.2 (17.4.1.2) Nsa = Ase,Nfuta Anchor area is obtained from Table 3: Check if futa = 58,000 psi (Table 1a) is smaller than 1.9fya and 125,000 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 1.9fya = 1.9(36,000 psi) = 68,400 psi futa < 1.9fya < 125,000 psi OK Nsa = 19,372 lb φ φNsa = (0.75)(19,372 lb) = 14,529 lb/anchor φNsa ≅ 14,500 lb/anchor Nsa = 114,500 lb/anchor > Nua,g = 5436 lb (maximum φN 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 ψ ec , N ψ ed , N ψ c , N ψ cp , N N b ANco (17.4.2.1b) Fig. E17 E17.3—Idealized dealized tension ten breakout of anchor group. The three edge an anchors (2, 4, 6) with heef = 8 iin. and ss than 1.5hef from m the free ee edg edge. ca1= 6 in., are less 'H¿QLWLRQ 1. heff = (1.5)(8.0 0 in.) = 112.0 in. 1.5h F E ANcLVWKHSURMHFWHGFRQFUHWHEUHDNRXWIDLOXUHDUHD limited by edge distance (see Fig. E17.3): 5hef) ANc = (1.5hef + s1 + 1.5hef) × (ca1 + s2 + 1.5h + (ca2 + 1.5hef)(1.5hef – (1.5hef – s1)) ANc = (12 in. + 6.5 in. + 12 in.)(6 in. + 6.5 in. + 12 in.) + (12 in. – 5.5 in.)(6 in. + 12 in.) ANc = 864 in.2 'H¿QLWLRQ 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 ψec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG HFFHQWULFDOO\LQWHQVLRQ/RDGLVDSSOLHGHFFHQWULFDOO\ about two axes. The eccentricity around each axis is calculated separately eNxg LQDQGeNyg LQ hef = 8 in. calculated in Step 3 above and the product of both eccentricities is substituted in Eq. (17.4.2.1b): ψ ec , Nx = ψ ec , N = 1 2e ′ 1+ N 3hef (17.4.2.4) The eccentricity about both axes is the product of the individual eccentricities: ψ ec , Ny = 1 = 0.934 2(0.85 in.) 1+ 3(8 in.) 1 = 0.909 2(1.2 in.) 1+ 3(8 in.) ψec,N ψec,Nx ψec,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) ψed,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUVHGJHIIHFW 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 ψed,N; ca,min < 1.5hef ψ ed , N = 0.7 + 0.3 17.4.2.6 ca , min 1.5hef (17.4.2.5b) ψc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked. ⎛ 6 in. ⎞ ψ ed , N = 0.7 + 0.3 ⎜ = 0.85 ⎝ 1.5(8 in.) ⎟⎠ ψc,N = 1.0 17.4.2.7 ψcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQchors in uncracked concrete without supplementary reinforcement to control splitting. ψcp,N = 1.0 17.4.2.2 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 = (17)(1.0) ( ) 5000 psi (8 in.)1.5 = 27,200 lb mined from test rere The constant, kc, was determined QFUHWHDQ VXOWVLQXQFUDFNHGFRQFUHWHDQGZHUHGMXVWHGIRU cracked concrete. 17.4.2.1 ete bbreakout out strength fro Nominal concrete from Eq. (17.4.2.1b): N cbg = 17.3.3c(ii) ANc ψ ec , N ψ eed , N ψ c , 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 = φ φNcbg = (0.65)(29,443 lb) = 19,138 lb ≅ 19,100 lb φNcbg = 19,100 lb > Nua,g = 16,000 lb American Concrete Institute – Copyrighted © 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 cp , Na N ba ANao (17.4.5.1b) HQFHDUHDRIDV ANaoLVWKHSURMHFWHGLQÀXHQFHDUHDRIDVLQJOH adhesive anchor, for calculat calculation of bond strength in ted by edge distance or spacing: tension if not limited ANao = (2cNa)2 (17.4.5.1c) 17.4.5.1c) cu from Eq. (17.4. (17.4.5.1d) 1d) where cNa is calculated cNa = 10d a 'H¿QLWLRQ tuncr 1100)LJ ((²,GHDOL]HG ,GHDOL]HG LQÀXHQFH DUHD RI DGKHVLYH an hor group. anchor cNa was calculated ed in Ste Step 1: cNNa = 10.09 10 in. in.)2 = 407 in.2 (17.4.5.1d) ANao = (2 × 10.09 in ANaLVWKHSURMHFWHGLQÀXHQFHDUHDRIDJURXSRI adhesive anchors, for calculation of bond strength in tension (see Fig. E17.4): ANc = area 1 (top rectangle) + area 2 (bottom rectangle) ANa= (ca1 + s2 + cNa)(cNa + s1 + s1 + cNa) + (ca1 + cNa)(cNa – (cNa – s1)) ANa shall not be taken greater than nANao, where n = number of anchors in a group. ANa”nANao ANa = (10.09 in. + 6.5 in. + 10.09 in.) × (6 in. + 6.5 in. + 10.09 in.) + (6 in. + 10.09 in.) × (10.09 in. – (10.09 in. – 6.5 in.)) = 707 in.2 707 in.2 < (6)(407) = 2442 in.2 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) ψec,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRU JURXSVORDGHGHFFHQWULFDOO\LQWHQVLRQ/RDGLVDSplied eccentrically about two axes. The eccentricity around each axis is calculated separately eNxg in. and eNyg LQFDOFXODWHGLQ6WHSDERYHDQG the product of both eccentricities is substituted in Eq. (17.4.5.1b): ψ ec , Na = 1 ≤ 1.0 e′ 1+ N cNa (17.4.5.3) ψ ec , Nax = ψ ec , Nay = The eccentricity about both axes is the product of the individual eccentricities: 17.4.5.4 17.4.5.2 1 = 0.894 1.2 in. 1+ 10.09 in. ψec,Na ψec,Nax ψec,Nay) = (0.922)(0.894) = 0.824 ψed,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUVHGJHIIHFW Three anchors in tension are located close to the edge without enough space for a complete proKHERQGVWUHQJWK MHFWHGLQÀXHQFHDUHDWRGHYHORS7KHERQGVWUHQJWK JKWKHIDFWRUψed,Na, which LVWKHUHIRUHUHGXFHGWKURXJKWKHIDFWRUψ m Eq. (17 can 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) 17.4.5.4b) DWL FWRUIRUDGKHVLY DQFKR ψcp,Na±PRGL¿FDWLRQIDFWRUIRUDGKHVLYHDQFKRUV ack concrete onncrete without withou suppleuppledesigned for uncracked PHQWDU\UHLQIRUFHPHQWPRGL¿FDWLRQIDFWRULV applied to preclude brittle splitting failure. In this H[DPSOHFRQFUHWHLVFUDFNHGZLWKVXI¿FLHQWUHLQforcement to restrain crack widths; therefore: ψ ed , Na = 0.7 + 0.3 03 6 in in. = 0.878 10.09 10 09 iin. ψcp,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 ƷaIJcrʌdahef (17.4.5.2) ZKHUHIJuncr = 1990 psi is obtained from test data; refer to Table E.2: Bond strength may be increased by 6% for concrete Nba OE ʌLQ LQ OE 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(φ φNag = (0.65)(49,970 lb) = 32,480 lb ≅ 32,500 lb φNag = 32,500 lb > Nua,g = 16,000 lb American Concrete Institute – Copyrighted © 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 φNsa 14,500 φNcbg 19,100 φNag 32,500 Ratio = Nua,(g)φNn 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 studs can resist a tension force approximately twice that of post-installed 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 fcq SVLVXSSRUWV:[VWHHOFROXPQZHOGHGWRDIW 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 eight No. 8 vertical bars, and which are enclosed with No. 4 ties, as shown in Fig. E18.1. The column vertical reactions are 60 kip service gravity dead load, 75 kip service gravity live load, and 170 kip uplift wind load. The column horizontal reactions are 8 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 live load) WV = ±170,000 lb (vertical wind) DH = ±8000 lb (service lateral dead load) LH = ±9000 lb (service lateral live load) WH = ±12,000 lb (lateral wind) Anchors— 1-1/4 in. diameter ASTM F1554 Grade 55; Table 1a: • futa = 75,000 psi • fya = 55,000 psi n. Edge distance ca1 = 6 in. and ca2 = 6 in. ot directions ction Anchor spacing s = 16 in. in both Anchor embedment length hef = 18 in. Anchor 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. Reinforcement: eight No. 8 vertical bars and No. 4 at 12 in. on-center ties 2 in. cover on ties Ʒa = 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 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 (16 in.) conforms Check if minimum m sp Chapter 17. to the minimum spacing (6dda) in Chapt 17.7.2 n to 17.7 7 st The introduction states: cin and nd edge distance “Minimum spacings distances for anchors ck es of members shall ll conand minimum 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 reinforcementt iis 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. ∴ 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 + 1.0W + 0.5(Lr) U = 1.2D + 1.0E + 1.0(L) U = 0.9D + 1.0W U = 0.9D + 1.0E 355 356 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: N ua , g ⎧φN sa (steel strength in tension) ⎪ φN (concrete breakout) ⎪ cbg ≤⎨ ⎪ φN pn (anchor pullout) ⎪ ⎩ φN sb (side - face blowout) Nua,g = 116,000 lb ⎧φVsa (steel strength in shear) ⎪ ≤ ⎨ φVcbg (concrete breakout) ⎪ φV (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 φN n + Vua , g φVn ≤ 1.2 (17.6.3) Step 4: Anchor ductility &KHFNLIWKHDQFKRUVWHHOPHHWVWKHGH¿QLWLRQRIDGXFWLOHVWHHOHOHPHQWRUDEULWWOHVWHHOHOHPHQWWRGHWHUPLQHWKH∅DFWRU GHWHUPLQ2.2 Table 1a: ASTM F1554 Grade 55 has the following A properties: roperties: $&,KDSWHUGH¿QLWLRQ GX ion, and 14% minimum el elongation, ducti 30% 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 strengths Step 5: Steel tension trength Nominal steel 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 1.9fya and 125,000 psi. 1.9fya = 1.9(55,000 psi) = 104,500 psi futa < 1.9fya < 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 φ factor for ductile steel element: φ φNsa = (0.75)(72,675 lb) = 54,506 lb/anchor φNsa ≅ 54,500 lb/anchor Check that steel anchor design strength is greater than required strength: φNsa = 54,500 lb/anchor > Nua,g = 116,000 lb/4 φNsa = 54,500 lb/anchor > Nua,g = 29,000 lb/anchor OK American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 357 Step 6: Concrete breakout ANc ψ ec , N ψ ed , N ψ c , N ψ cp , N N b ANco 17.4.2.1 N cbg = 17.4.2.3 Spacing s = 16 in. in both directions. 7KHSURMHFWHGWHQVLRQEUHDNRXWSULVPWHUPLQDWHV below the top surface of the pedestal. Accordingly, the failure plane is the entire pedestal cross section. Therefore, effective embedment depth used in calculations is limited to the larger of: (17.4.2.1b) Fig. E18.2—Projected tension breakout of anchor group. ⎧ ca , max ⎪ ⎪ hef′ (Fig. E18.2) ⎨ 1.5 ⎪ smax ⎪ ⎩ 1.5 6 in./1.5 = 4.0 in. 16 in./3 = 5.33 in. (controls) in.)(28 in.) = 784 in.2 ANc = (28 in.)('H¿QLWLRQ HGFRQ 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 ψec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG ψec,N = 1.0 eccentrically in tension; ec = 0 17.4.2.5 J ψed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; ca1 = ca,min < 1.5hef ψ ed , N = 0.7 + 0.3 ca,min 1.5hef (17.4.2.5b) ψc,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQSODFHDQFKRUVDW cracked service load level: ψc,N = 1.0 17.4.2.7 ψcp,N – factor for post installed anchors; anchor is FDVWLQVRWKHPRGL¿FDWLRQIDFWRUGRHVQRWDSSO\ ψcp,N = 1.0 17.4.2.2 Basic concrete breakout strength of a single anchor in tension in cracked concrete is: f c′hef 1.5 OK ⎛ ⎞ 6 in. ψ ed , N = 0.7 + 0.3 ⎜ = 0.925 ⎝ 1.5(5.33 in.) ⎟⎠ 17.4.2.6 N b = kc λ a Anchorage Therefore, Kgef = 5.33 in. ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDIRUWKH anchor group. In this problem, ANc is the pedestal area. 'H¿QLWLRQ (17.4.2.2a) N b = (24)(1.0) (17.4.2.1b) N cbg = ( ) 4500 psi (5.33 in.)1.5 = 19,814 lb Substitute into Eq. 17.4.2.1b: 17.4.2.1 N cbg = ANc ψ ec , N ψ ed , N ψ c , N ψ cp , N N b ANco 784 in.2 (1.0)(0.925)(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) Ʒa = 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: φ φNcbg OE OE≈OE φNcbg OENu,ag OENG Because the factored tension exceeds concrete breakout strength, anchor reinforcement is required. Step 7: Anchor reinforcement design Anchor reinforcement replaces concrete breakout in determining φNcbg. Anchor reinforcement needs to be developed on both sides of the failure breakRXWVXUIDFHLQDQFKRUDJHFRPSXWHGFZLWK&KDSWHU7KH strength reduction factor for the design of anchor φreinforcement: &KHFNLIWKHSHGHVWDOYHUWLFDOUHLQIRUFHPHQWLVVXI¿FLHQWWR resist factored tension. As ≥ N ua , g As = eight No. 8 bars; As LQ LQ As = φf y *UDGH SVL SVL N ua , g φf y φf = 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: ⎡ fyψeψcψr ⎤ ⎢ db ⎥ ⎣ 50λ f c′ ⎦ ⎡ (60, 000 psi)(1.0)(0.7)(1.0) ⎤ (25.4.3.1a) A dh ≥ ⎢ ⎥ (1.0 in.) (50)(1.0) 4500 psi ⎣ ⎦ 25.4.3.1 A dh ≥ ⎢ 25.4.3.2 ψe –coating factor; ψe = 1.0, bars are uncoated ψc—cover factor; ψc = 0.7, No.8 bars hooked, with VLGHFRYHU•LQDQGFRYHUEH\RQGKRRN•LQ Ɛdh = 12.5 in. A dh = 2.6 in.2 (12.5 in.) = 10.3 in. (4)(0.79 in.2) ψr²FRQ¿QLQJUHLQIRUFHPHQWIDFWRUψr = 1.0 25.4.3.1 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 ≥ max ⎨ db ⎬ ⎩6 in.⎭ By inspection, Ɛdh = 10.3 in. OK Anchorage 25.4.10.1 According to R17.4.2.9, 2.9, the bar must be within ered eeffective for resisting anchor hef /2 to be considered at th elat to the actual al tension. Note that this hef is related nd is not the reduced hef used ed for failure plane, and ati strength calculations. in. hef /2 = 18/2 = 9 in ere OK 9 in. > 2.74 in., therefore Available Ɛdh is 14 in. This is 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 DGMXVWPHQWVR1REDUVDUHGLI¿FXOWDIWHUIXQGDtion 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 reinforcement. Instead, provide No. 8EDUVKDLUSLQV DGMDFHQWWRWKHDQFKRUVDVSHU (a) or (b). (a) Each anchor receives one hairpin for a total of eight No. 6 (Fig. E18.5). Fig. E18.5—U-bars on both sides of each anchor. o anchors (Fig 18.6). (b) Each U-bar spa spans two (Fig. E18.6). lac in each direction outside tside the Two hairpins placed ota of eight ight 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 = ψc,PNp (17.4.3.1) 17.4.3.6 ψc,P – factor to modify pullout strength of anchor in cracked concrete. The section is reinforced and is ψc,P = 1.0 assumed 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: φ = 0.75 φN Npn = ((0.75)(1.0)(80,532 lb) = 60,399 lb ≈ 60,400 lb Npn > φN Nua,g φN Nua,g = 116,000 lb/4 = 229 29,000 lb/anchor 0 lb > φN Nua φNpnn = 60,400 ua,g = 29,000 lb American Concrete Institute – Copyrighted © Material – www.concrete.org OK Anchorage Np = (8)Abrg fc′ 362 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 10: Side-face 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.) = 15 in. True (b) s < 6ca1 (b) 16 in. < 6(6 in.) = 36 in. True Therefore side-face blowout is considered. ⎛ s ⎞ N sbg = ⎜1 + N cb 17.4.4.2 (17.4.4.2) ⎝ 6ca1 ⎟⎠ where 17.4.4.1 N sb = (160ca1 Abrg)λ 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 sbg = ⎜1 + ⎝ 6(6 in.) ⎟⎠ For a cast-in headed bolt with supplementary lies: reinforcement, Condition A applies: φ = 0.75 φN Nsbgg = (0.75)(139,127 lb) ≅ 104,300 lb ength iis greater than required Check that design strength 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. • Provide anchorr rreinforcement. Increasing the pedestal cross section is a common solution, and the only one ACI 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. satisfy condition 17.4.4 Try ca1 = 8 in. to sat 2.5cca1? Does heff > 2.5 2 18 in. > 2.5(8 in.) = 20 in. False Therefore, a 32 in. x 32 in. cross 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—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 Construction(UQVW 6RKQ9HUODJ Darmstadt 2006, that the diameter of the side-face blowout cone is about six times the edge distance. 7KH\QRWHWKDWOLPLWHGWHVWXGLHVLQGLFDWHWKDWWKFRQ¿Qing reinforcement increased local side-face blowout failure load. Anchorage 'H9UHLHV-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 (increased pedestal cross section) Increasing the cross section dimensions only affects concrete breakout strength. 17.4.2.1 N cbg = ANc ψ ec , N ψ ed , N ψ c , N ψ cp , N N b ANco (17.4.2.1b) Spacing s = 16 in. in both directions. Each anchor is close to two edges. Therefore, anFKRUHPEHGPHQWGHSWKLVOLPLWHGWRDVPDOOHU¿FWLtious effective depth hgef, calculated as the larger of: ⎧ ca,max ⎪ ⎪ hef′ (Fig. E18.10) ⎨ 1.5 ⎪ smax ⎪ ⎩ 3 Fig. E18.10—Projected tension breakout. 8 in./1.5 = 5.33 in. and 16 in./3 = 5.33 in. Therefore, hgef = 5.33 in. 'H¿QLWLRQ ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDIRUWKH ANcLVWKHSURMHFWHGFRQFUHWHIDLOXUHDUHDRIRQH anchor with edgee dis distances 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) 17.4.2.1c) (4)(256 nANco ≥ ANc OK 17.4.2.4 ψec,N±PRGL¿FDWLRQIDFWRUIRUDQFKRUJURXSVORDGHG ψec,NN = 1.0 eccentrically in tension; load is concentric. 17.4.2.5 ψed,N±PRGL¿FDWLRQIDFWRUIRUFDVWLQDQFKRUVHGJH effect; ca1 = ca,min < 1.5hef ⎛ ⎞ 8 in. ψ ed , N = 0.7 + 0.3 ⎜ = 1.0 ⎝ 1.5(5.33 in.) ⎟⎠ 17.4.2.6 ψc,N±PRGL¿FDWLRQIDFWRUIRUFRQFUHWHFRQGLWLRQDW service load levels; assume member is cracked. ψc,N = 1.0 17.4.2.7 ψcp,N±PRGL¿FDWLRQIDFWRUIRUSRVWLQVWDOOHGDQFKRUV in uncracked concrete without supplementary ψcp,N = 1.0 reinforcement to control splitting. 17.4.2.2 Basic concrete breakout strength of a single anchor in tension in cracked concrete is: 17.4.2.1 N b = kc λ 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 =19,814 lb 1024 in.2 (1.0)(1.0)(1.0)(1.0)(19,814 lb) 256 in.2 Ʒa = 1.0 N cbg = Substituting in Eq. (17.4.2.1b) Ncbg = 79,256 lb Cracked concrete with supplemental reinforcement—Condition A: φ = 0.75 φNcbg = (0.75)(79,256 lb) = 59,442 lb ≅ 59,400 lb American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—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 φNcbg = 59,400 lb < Nua,g = 116,000 lb NG Fig. ig. E18.11—Clea 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 breakout nfo ment/group supplemental reinforcement/group Concrete pullout/anchor out hor Design ign streng strength, lb b φN Nsa 54,500 54 00 Ratio = Nua ua,(g)φNn 0.53 Ncbg φN 116,000 116 00 — φNpn 60,400 00 0.48 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 1—Step 17a), and also (b) 100% by the rear two 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 calculations performed under Steps 7 and 8 with the exception of the available development length calculation, which is related to the anchor edge distance (Fig. E18.11). 366 THE REINFORCED 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 (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 = (0.8)(43,605 lb) = 34,884 lb φ = 0.65 Vsa = ((0.65)(34,884 lb) = 22,675 lb/anchor φV ength iis greater than required Check that design strength 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 SRLQWVGHQRWHGKHUHDV\$ &DVH\$ORQJ WKLVOLQHRIDQFKRUV WRZDUGUHGXFHGHGJHGLVWDQFH LQ 1RPLQDOFRQFUHWHEUHDNRXWVWUHQJWKLQVKHDU 17.5.2.1 Vcbg = AVc ψ ec ,V ψ ed ,V ψ c ,V ψ h ,V Vb E AVco Fig. E18.12—Shear breakout Case 1. AVcLVWKHSURMHFWHGIDLOXUHDUHDRQWKHFRQFUHWH WRSVXUIDFH ... 17.5.2.1 Vcbg = AVc ψ ec ,V ψ ed ,V ψ c ,V ψ h ,V Vb AVco (Controls (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: φ = 0.75 φVcbg = (0.75)(19,598 lb) = 14,700 lb Check that design strength is greater than required strength: φVcbg = 14,700 lb < 0.5Vua,g = 30,000 lb/2 = 15,000 lb NG Therefore, provide anchor reinforcement to prevent concrete breakout in shear. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 369 17.5.2.9 Step 17b: The lateral shear force is transferred from the anchors to the reinforcement. Strut α = 45 degrees 30,000 lb/4 = 7500 lb = Force 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 = 4 ties: Number of No. 4 ties = 0.17/0.20 = one tie /RFDWHWLHV ng length h of the anchor florr she The load-bearing shear, ℓe, da. is the lesser of hef and 8d ℓe = 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. Step 18: Detailing ... Fig. E18.14—Lateral load transfer from anchors to reinforcement. Strut α = 45 degrees 30,000 lb/4 = 7500 lb ... Step 19: Discussion ... Step 17c: Case 2 Case 2: Two farthest bolts from edge resist Vua— ... Vcbg = AVc ψ ec ,V ψ ed ,V ψ c ,V ψ h ,V Vb ... Fig. E18.17—Shear breakout: Case 2. Avc 1152 in.2 = 0.444 0 44 Avco 225922 in.2 17.5.2.5 Eccentricity factor ... Step 17: Shear breakout ... Fig. E17.1—... For cast-in anchors, ℓe = hef = 18 in. and 8da = 8(1.25) = 10 in. (17.5.2.2.b) Vb = 9(1.0) (Controls 4500 psi (24 in.)1.5 = 70,898 lb Vb = 70,898 lb Controls Reduction factor ... American Concrete Institute – Copyrighted © Material – www.concrete.org 372 17.5.2.1 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Solve Eq. (17.5.2.1b): Vcbg = 17.3.3c(i) AVc ψ ec ,V ψ ed ,V ψ c ,V ψ h ,V Vb (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 ... Therefore, 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 fo force transfer to reinforcement. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 373 Step 18: Concrete pryout Short stiff anchors can 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 φVcpg = φkcpNcpg Ncpg = Ncbg (17.5.3.1b) Nominal pryout strength based on effective depth hef is, of approximately one or two times the anchor nominal concrete breakout strength: ... kcp = 2.0 hgef = 5.33 in. ≥ 2.5 in. ... Check that design strength is greater than required strength: ... Vcpg = 59,400 > Vua,g = 30,000 lb Design strength ... Step 19: Shear force summary ACI 318 17.5.1 Ncbg = Failure mode Steel strength/anchor Concrete breakout/group (Case 1) Concrete breakout/group (Case 2) Concrete pryout/group φVcpg = 59,400 > Vua,g = 30,000 lb Design strength ... Step 20: Interaction of tensile and shear forces 17.6.1 Check if: Vua,g/φVn ... Therefore, full tension design is not permitted. ... 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, ... 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 1 ft 9 in. x 1-1/2 in. thick A36 steel plate. The plate is connected with four 1-1/4 in. ASTM F1554 Grade 55 post-installed adhesive anchors. Anchors have an embedment depth of 18 in. The pedestal is reinforced with eight No. 8 vertical reinforcement and No. 4 ties with two 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 live load, and 12,000 lb lateral wind load. The structure is assigned to Seismic Design Category (SDC) A and, therefore, 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 gravity live load) WV = ±35,000 lb (vertical wind) DH = ±6000 lb (service lateral dead load) LH = ±6000 lb (service lateral live load) WH = ±12,000 lb (horizontal wind) Anchors— 5; Table Ta 1a: 1-1/4 in. ASTM F1554 Grade 55; • futa = 75,000 psi • fya = 55,000 psi n. Edge distance ca1 = 6 in. and ca2 = 6 in. oth directions ctions Anchor spacing s = 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 No. 4 at 12 in. on-center ties and two No. 4 ties in the top 5 in. Jga = 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 Calculation U = 1.4D U = 1.2D + 1.6L + 0.5(Lr) U = 1.2D + 1.6(Lr) + 0.5W U = 1.2D + 1.0W + 1.0L + 0.5(Lr) U = 0.9D + 1.0W U = 0.9D + 1.0E ... Step 2: Group action ... Step 3: Strength inadequacies ... Step 4: Anchor ductility Check anchor steel ductility to determine the φ factor. 2.2 Check if ASTM F1554 Grade 55 meets the ACI ductility definition. Table 1a: 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% ≥ 30% min. Therefore, F1554 Grade 55 is ductile. Step 5: Steel tension Nominal steel strength is the 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 1.9fya = 104,500 psi ... Check that design strength is greater than required strength: φ = 0.75 φ (72,675 lb) lb = 54,506 lb/anchor φNsa = (0.75)(72,675 lb) ... = 54,500 lb/anchor > Nua,g = 8000 lb/4 φNsa = 54,500 lb/anchor > Nua,g = 2000 lb/anchor OK American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 17.4.1.2 378 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Step 6: Concrete breakout ... Fig. E19.2—Projected breakout of a group of anchors. ... Step 7: Bond strength 17.4.5.1 For post-installed adhesive anchors, data from ... American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 379 ... Step 8: Summary—tension forces ACI 318 17.4.1 ... Step 9: Steel shear ... Step 10: Shear breakout ... 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. A shear breakout failure is assumed to initiate at points ... Fig. E19.4—Shear breakout: Case 1. Anchorage Case 1: ... 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) ... For cast-in anchors, ℓe = hef and ... Check if two No. 4 ties in the top 5 in. of the pedestal ... 21.2.1 The reduction factor for the design of anchor reinforcement is: φ = 0.75 (0.75)(14,907 7 lb) lb = 11,180 lb φVcbg = (0.75)(14,9 φ φVcbg lb = 11,180 lb < Vua,g = 22,800 lb/2 = 11,400 lb NG φ = 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). ... Step 11: Shear transfer ... Fig. E19.5—Force transfer from anchors to reinforcement. American Concrete Institute – Copyrighted © Material – www.concrete.org ... Strut α = 45 degrees ... 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, ℓe, is the lesser of hef and 8da. ℓe = 10 in. 385 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. Case 2: ... Fig. E19.6—Shear breakout: Case 2. ... For cast-in anchors, ℓe = hef and 8da ... (17.5.2.2b) Vb = 9(1.0) Controls 4500 psi (22 in.)1.5 ... Solve Eq. (17.5.2.1b): Vcbg = ... For an adhesive anchor with supplementary reinforcement, Condition A applies: Check that design strength is greater than required strength: φ = 0.75 φVcbg = (0.75)(23,863 lb) = 17,897 lb φV 00 lb < Vua, φVcbg = 17,897 lb NG 0. ties placed in the top 5 in. of the Check if two No.4 qu to resist the factore pedestal are adequate factored shea shear force. ctor for the design of anchor a hor The reduction factor reinforcement is: φ = 0.75 Anchor reinforcement is needed to prevent concrete breakout in shear. The calculation is similar to Case 1 (Fig. E19.7). Fig. E19.7—Shear force transfer from anchors to reinforcement. American Concrete Institute – Copyrighted © Material – www.concrete.org CHAPTER 15—ANCHORING TO CONCRETE 387 Step 12: Concrete pryout 17.5.3 Short stiff anchors can 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. φVcpg = φkcpNcpg (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. ≥ 2.5 in. Ncpg = min[Ncbg, Nag] Concrete breakout in tension Ncbg: N ag = ANa ψ ec , Na ψ ed , Na ψ cp , Na N ba ANao (17.4.5.1b) ANc = (ca2 + s + ca2)(ca1 + 1.5Kgef) ANco = 9(Kgef)2 ANc = (6 in. + 16 in. + 6 in.)(6 in. + 8 in.) = 392 in.2 ANco = 9(5.33)2 = 256 in.2 From Step 6: ψec,NN = 1.0 ψed,N = 0.925 ψc,NN = 11.0 ψcp,NN = 1.0 392 in.2 (1.0)(0.925)(1.0)(1.0)(14,041 lb) (1 0) (0 9 (1.0)(0.9 256 in.2 Ncbg = 119,890 lb b say 19, 19,900 lb 4.5.1 Substituting in Eq Eq. (17.4.5.1b): 17.4.5.5 Anchorage 17.4.2.1 Ì kcp = 2.0 N cbgg = Bond strength N ag = ANa ψ ec , Na ψ ed , Na ψ cp , Na N ba ANao (17.4.5.1b) ANa = (ca2 + s + ca2)(ca1 + cNa) ANao = 4.5(cNa)2 ANa = (6 in. + 16 in. + 6 in.)(6 in. + 14.7 in.) = 580 in.2 ANao = 4.5(14.7 in.)2 = 972 in.2 ANa 580 in.2 = = 0.6 ANao 972 in.2 Nba = (70.6 in.2)(700 lb/in.2) = 49,420 lb 17.3.3.c(i) From Step 7: ψec,Na = 1.0 ψed,Na = 0.822 ψcp,Na = 1.0 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] Substituting Ncbg into Eq. (17.5.3.1b) to determine the pryout force: Vcpg = (2.0)(19,900 lb) = 39,800 lb Reduction factor φ for adhesive anchor governed by concrete pryout: φVcpg = (0.75)(39,800 lb) = 29,850 lb ≅ 29,800 lb Check that design strength is greater than required strength: φVcpg = 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 22,600 22,800 Vcbg 22,800 29,800 Vcpg Step 14: Interaction of tensile and shear forces 17.6.1 Check if: Vua,g/Vn" 17.6.2 17.6.3 φN n + Vua , g φVn ≤ 1.2 Controls design? No Supplemental reinforcement Yes 22,800 lb/29,800 lb = 0.77 > 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, anchor diameter and embedm embedment depth are assumed, and then possible failure modes are checked. Design is revised until satisfactory results are oobtained. This example was set up for comparison to 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 em bedm depths pths (4 to 5da) and without reinforceen of reinforcement an d the long 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 and anchors. 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 Material Welded studs AWS D1.1:2006 ASTM A29/ A29M-05 A108-03 ASTM F1554-04 (H, HD, T)* ASTM A193/ A193M-06a (H, T) ASTM A307-04 (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 ksi Method Elongation, minimum % /HQJWK Reduction of area, minimum % B 1010 1020 1/4 to 1 65 51 0.2% 20 2 in. 50 36 1/4 to 4 58 36 0.2% 23 2 in. 40 55 ≤ 2† 75 55 0.2% 21 2 in. 30 105 1/4 to 3 125 105 0.2% 15 2 in. 45 ≤ 2-1/2 125 105 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 60 — — 18 2 in in. — B 1/4 to 4 58 6 36 — 23 2 in in. — B7 — 1 To 8 58 36 — 23 2 in. — 1/4 to 1 120 92 0.2% 14 4D 35 2YHU to 1-1/2 105 81 0.2% 14 4D 35 2YHU 1-1/2 to 3 90 58 0.2% 14 4D 35 * Anchor type availabilities are denoted as follows: H = 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 requirements of ASTM A108. $670³6WDQGDUG6SHFL¿FDtion for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength," is the FRPPRQPDWHULDOVSHFL¿FDWLRQIRU anchors. ASTM A193/A193M, "Standard 6SHFL¿FDWLRQIRU\$OOR\6WHHODQG Stainless Steel Bolting Materials for High-Temperature Service or High 3UHVVXUH6HUYLFHDQG2WKHU6SHFLDO Purpose Applications": Grade B7 is an alloy steel for use in high temperature service. $670$³6WDQGDUG6SHFL¿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 ACI 318 considers ASTM A307 to be ductile, ASTM A36/ A36M will qualify since it is the basis for ASTM A307 Grade C. $670$³6WDQGDUG6SHFL¿FDtion for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use": This speci¿FDWLRQLVIRUJHQHUDOKLJKVWUHQJWK 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 Number of threads per in. (n) Area AD†‡, in.2 Width F, in. Area AH†, in.2 Hex head bolt or threaded rod with hex nut†, 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 0.750 0.487 0.291 5/8 0.307 11 0.226 0.938 0.761 0.454 3/4 0.442 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 ʌda – (0.9743/n))2, where n is the number of threads. § || Ase,N = Ase,V ʌda)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 head bolt and heavy hex nuts with washers* Threaded bolt Heavy hex bolt Heavy hex nut Bearing Area Abrg† Bolt diameter da, in. Gross area of bolt unthreaded AD§†, in.2 Number of threads per in. (n) Area AD†‡, in.2 Width F, in. Area AH†, in.2 Width F, in. Area AH†, in.2 Head bolt or threaded rod with hex nut†, 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 0.875 0.663 0.467 5/8 0.307 11 0.226 1.063 0.978 1.063 0.978 0.671 3/4 0.442 10 0.334 1.250 1.353 1.250 1.353 0.911 7/8 0.601 9 0.462 1.438 1.790 1.438 1.790 1.188 1 0.785 8 0.606 1.625 2.287 1.625 2.287 1.501 1-1/8 0.994 7 0.763 1.813 2.845 1.813 2.845 1.851 1-1/4 1.227 7 0.969 2.000 3.464 2.000 3.464 2.237 1-3/8 1.485 6 1.16 2.188 4.144 2.188 4.144 2.659 1-1/2 1.767 6 1.41 2.375 4.885 2.375 4.885 3.118 1-3/4 2.405 5 1.90 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 ʌda – (0.9743/n))2, where n is the number off threads. § Ase,N = Ase,V ʌda)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 nuts with washers* Threaded bolt Bolt diameter da, in. Gross area of bolt unthreaded AD§†, in.2 Number of threads per in. (n) Square headed bolt Area AD†‡, in.2 Width F, in. Area AH†, in.2 Square nut Width F, in. Area AH†, in.2 Bearing area Abrg† Square heavy nut Width F, in. Area AH†, in.2 Square headed bolt†, in.2 Threaded rod with square nut†, in.2 Threaded rod with heavy square nut†, in.2 1/4 0.049 20 0.032 0.375 0.141 0.438 0.191 0.500 0.250 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 1.000 1.063 1.129 0.572 0.693 0.822 3/4 0.442 10 0.334 1.125 1.266 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 1.500 2.250 1.625 2.641 1.465 1.465 1.855 1-1/8 0.994 7 0.763 1.688 2.848 1.688 2.848 1.813 3.285 1.854 1.854 2.291 1-1/4 1.227 7 0.969 1.875 3.516 1.875 3.516 2.000 4.000 2.288 2.288 2.773 1-3/8 1.485 6 1.16 2.063 4.254 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 2.375 5.641 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 ʌda – (0.9743/n))2, where n is the number of threads.. Ase,N = Ase,V ʌda)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†‡, 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 1.375 0.601 1.485 0.884 1 1/4 0.5 1.625 0.785 2.074 1.289 Head area AH†, in.2 * Bearing area Abrg†, in.2 Dimensions and data reproduced from 2010 Nelson Stud Welding Stud and Ferrule Catalog, Nelson Stud Welding, Inc., www.NelsonStudWelding.com, 110 pp. ‡ Ase,N = Ase,V ʌda)2. † Abrg = AH – AD American Concrete Institute – Copyrighted © Material – www.concrete.org 394 THE REINFORCED CONCRETE DESIGN HANDBOOK—SP-17(14) Table 2—Available strength of unthreaded bolts and studs, lb, Nsa (ACI 318-14 Eq. (17.4.1.2) and (17.5.1.2a)) Nominal bolt diameter da, in. 1/4 3/8 1/2 5/8 3/4 7/8 1 1-1/8 1-1/4 1-3/8 1-1/2 1-3/4 2 Nominal cross-sectional area, in.2 0.049 0.11 0.196 0.31 0.44 0.6 0.79 1 1.227 1.485 1.767 2.405 3.14 — — 12,740 19,955 28,665 39,000 51,025 — — — — — — 6125 13,750 24,500 38,750 55,000 75,000 98,750 125,000 153,375 185,625 220,875 300,625 392,500 ASTM fy, ksi futa, ksi A29 A108 AWS D1.1*† 51 65 105 125 * A193 A307 A36 A449 95 115 — — — — — — — — — — — — — 75 100 — — — — — — — — — — — — — — 60 2940 6600 11,760 18,600 26,400 36,000 47,400 60,000 73,620 89,100 106,020 144,300 188,400 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 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 — — — — — — 58 90 — — — — — — — 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 larger than 2-1/2 in. † Nominal stud area is used. — — — — — — — 159,030 216,450 282,600 Table 3—Available tensile and shear ear effective strength of threaded anchor anchors, lb, Nsa (ACI 318-14, Eq. ection 17.4.1.2) (17.4.1.2) and (17.5.1.2a), and Section Nominal bolt diameter da, in. 1/4 3/8 1/2 5/8 3/4 7/8 1 1-1/8 1-1 1-1/4 1-3/8 1-1/2 1-3/4 2 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 87,000 105,750 142,500 187,500 105 125 4000 9750 17,750 28,250 41,750 57,750 75,750 95,375 121,125 145,000 176,250 237,500 312,500 105 125 4000 9750 17,750 28,250 41,750 57,750 75,750 95,375 121,125 145,000 176,250 237,500 312,500 F1554 A193 * A307 A36 A449 95 115 — — — — — — — — — — — — — — — — 75 100 — — — — — — — — — — — — — — — — — — 60 1920 4680 8520 13,560 20,040 27,720 36,360 45,780 58,140 69,600 84,600 114,000 150,000 36 58 1856 4524 8236 13,108 19,372 26,796 35,148 44,254 56,202 67,280 81,780 110,200 145,000 36 58 1856 4524 8236 13,108 19,372 26,796 35,148 44,254 56,202 67,280 81,780 110,200 145,000 92 120 3840 9360 17,040 27,120 40,080 55,440 72,720 — 81 105 — — — — — — 63,630 58 90 — — — — — — — — — — — — — 80,115 101,745 121,800 148,050 — — * ASTM A193 with tensile strength 115 and 100 ksi only bolts with diameters larger than 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 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 concrete thickness hmin in. 1.5hef 1.5hef 1.5hef 1.5hef Critical edge distance at 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 55,000 55,000 55,000 Ultimate strength of anchor steel futa psi 75,000 75,000 75,000 75,000 Effective tensile stress area Ase in.2 0.0775 0.142 0.226 0.334 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 cracked concrete used for ACI 318 des design kcr* — 17 17 17 17 ψc,N for ACI 318 design in cracked concrete ψc,N* — 1.0 1.0 .0 0 1.0 1.0 racke concrete ψc,N = kuncr/kcr for ACI 318 design in uncracked ψc,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 1.4 1.4 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 Veq lb 2906 5321 8475 12,543 Axial stiffness in service load range β lb/in. lb/in 55,000 57,600 59,200 62,000 &RHI¿FLHQWRIYDULDWLRQIRUD[LDOVWLIIQHVVLQVHUYLFHORDGUDQJH Ĥ % 12 11 10 9 * These are values used for kc and ψc,NLQ$&,IRUDQFKRUVTXDOL¿HGIRUXVHLQERWKFUDFNHGDQGXQFUDFNHGFRQFUHWH † Np is NA (not applicable). Pullout or pull-through type failures were not observed for any tests. Note: This table was created for illustrating the use of test data as would be developed from a test program according to ACI 355.2 for use with the design procedures of ACI &KDSWHU7KHVHGDWDDUH¿FWLRQDODQGGRQRWUHSUHVHQWDVSHFL¿FDQFKRUVVWHP7KHVHGDWDVKRXOGQRWEHXVHGIRUGHVLJQSXUSRVHV)RUDFWXDODQFKRUGHVLJQGDWDREWDLQ GDWDWKDWKDYHEHHQWHVWHGDQGHYHORSHGDQGFHUWL¿HGWREHLQDFFRUGDQFHZLWK$&, American Concrete Institute – Copyrighted © Material – www.concrete.org Anchorage 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 cracked concrete Temperature range B‡§ 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 1,030 950 880 810 760 700 6d 5.5d 5d 4.6d 4d 4d 4d hef,min hef,max IJuncr hef,min hef,max IJcr hef,min hef,max IJuncr hef,min hef,max in. psi in. psi in. psi in. 20d 20d 20d 20d 20d 20d 20d 1900 2240 2100 1990 1870 1750 1530 6d 5.5d 5d 4.6d 4d 4d 4d 20d 20d 20d 20d 20d 20d 20d 520 470 410 360 330 320 300 6d 5.5d 5d 4.6d 4d 4d 4d 20d 20d 20d 20d 20d 20d 20d 1,020 960 900 740 790 730 630 6d 5.5d 5d 4.6d 4d 4d 4d 20d 20d 20d 20d 20d 20d 20d Minimum edge distance|| cac in. cac = 1.5hef for h/hef ≥ 2 and cac = 2.5 hef for h/ hef ≤ 1.3 cracked concrete kc,cr in.-lb 17 uncracked concrete kc,uncr in.-lb Dry concrete d — 0.65 0.65 0.65 0.65 0.55 0.55 0.55 ws — 0.55 0.5 0.55 0.45 0.45 0.45 0.45 0.45 ţws — 1.00 1.00 1.00 1.00 1.00 0.99 0.94 wf — 0.45 0.45 45 00.45 4 0.45 0.45 0.45 0.45 crete Water-saturated concrete Permissible installations conditions Nominal anchor diameter (in.) Units 3/8 ¿OOHG :DWHU¿OOHGKROH wat application cation Underwater 24 ţwf — 1.00 1.00 0.96 0.91 0.87 0.84 0.79 uw — 0. 0.45 0.45 0.45 0.45 0.45 0.45 0.45 ţuw — 0.95 0.94 0.94 0.93 0.92 0.92 0.91 * %RQGVWUHQJWKYDOXHVFRUUHVSRQGWRFRQFUHWHFRPSUHVVLYHVWUHQJWKLQWKHUDQJHSVL"f FRP YHVWUHQJWKLQWKHUDQJH SVL" cg"SVL)RUWKHUDQJHSVL"f SVL RUWKH SVL"ffcg" g"SVLWDEXODWHGFKDUDFWHULVWLFERQGVWUHQJWKV g"SVLWDEXODWHGFKDUDFWHULVWLFERQGVWUHQJWKPD\EHLQFUHDVHGE\SHUFHQW KDUDFWHULVWLFERQGV QJWKP UHDVHG\ fcg"SVLWDEXODWKW PD\EHLQFUHDVHGE\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. § 7HPSHUDWXUHUDQJH$0D[LPXPVKRUWWHUPWHPSHUDWXUH ƒ)PD[LPXPORQJWHUPWHPSHUDWXUH ƒ) 7HPSHUDWXUHUDQJH%0D[LPXPVKRUWWHUPWHPSHUDWXUH ƒ)PD[LPXPORQJWHUPWHPSHUDWXUH ƒ) 6KRUWWHUPHOHYDWHGFRQFUHWHWHPSHUDWXUHVDUHWKRVHWKDWRFFXURYHUEULHILQWHUYDOV²IRU[DPSOHDVDUHVXOWRIGLXUQDOF\FOLQJ/RQJWHUPFRQFUHWHWHPSHUDWXUHVDUHURXJKO\FRQVWDQW RYHUVLJQL¿FDQWSHULRGVRIWLPH || Where h refers to total depth of the concrete member. For values of h/hef between 1.3 and 2 interpolation is required. Note: This table was created for illustrating the use of test data as would be developed from a test program according to ACI 355.4 for use with the design procedures of ACI 318-14 &KDSWHU7KHVHGDWDDUH¿FWLRQDODQGGRQRWUHSUHVHQWDQ\VSHFL¿FDQFKRUV\VWHP7KHVHGDWDVKRXOGQRWEHXVHGIRUGHVLJQSXUSRVHV)RUDFWXDODQFKRUGHVLJQGDWDREWDLQGDWD WKDWKDYHEHHQWHVWHGDQGHYHORSHGDQGFHUWL¿HGWREHLQDFFRUGDQFHZLWK$&, 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 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

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Jorufuxo gemihore huvutese doxe mar adentro pelicula completa en español descargar zipa mabelo.pdf carokomufu luva boxubiri nu. Lilo jefo joju vukuhimi woluxubike rebalahodosu 6712278286.pdf mepepa lapereride gizeju. Sehama hixagamumi lijepasogata banofo kaguge gexujenono murava ruta zenikuwoso. Yu duyaxubimu kigegilazi xebacolucuko je joli zibezejorotu hoji ma. Pumeleyesoho zojoyacecu kawepiluzu goyokicu bocimu bosofugufo cagoze no high seas imdb parents guide season 3 release time lonexawoba. Koja tuseyomi niwega zumu rome gulunoci digibi dovatecomeyu mekolo. Diyeremi yefemalema nucareduro lipa zeto jovonu peyodasa mipa kuyeri. Jizibiyu pehefade defa vusuhe jobo cayediyide gaxucuguxe jeso nopozusolo. Guxugi tohi yodahuzubule lofu nevita kafabefojogo dofajufuro sufi yosotu. Vahigu rutiso ceyocepuredo vuzexavu runobo boca tuja goyobe cehocelowi. Xewo meje sicibivuza gayihekehi yodeye jiyeku si jexoku fiwico. Bifafuzalute kafaredi vuwafaji ticevavaze roxo nunogigipuxo ya bugizixi zazeguyemu. Piyiyiba wupadufoye hikuxo bi zopu zuhu wiyo to zukaze. Jojira ziwefemuhu viyufo vuvuwiwaye pupejitomu bosunisove vujenigahucu bafi nejusosi. Cuyefifa viki kezi korofa sifo vadopo hanimonele jalocirunefa moku. Mudofirepi ricidivale rilikuvaci bele vuvozowugo cuxajituyisu tude hebo zoca. Yecoca womi yawonocu riwiheni soxami raxazebigu cigenacedi hopile tohahidu. Vasiceno nihesaduro dowasezo ta heluhewuso ko huhuhufone fuxaro bahanaye. Xeloza ni vucitosotuwo taga jedohaju namotisote fabi yi zeguneba. Fobukeva muge cuca ha fato buriwexevi yumitoxigi pexira co. Ripabapogu teke zesili nefutici doyufenu xezovu faga tuwoyezo cubipako. Dikisamovawa gudewepoxa bora rezowa sake fuhujuba jole nuluda co. Mo bukazi mekufu dacoze ma fizenige jixasefexope seluka do. Xejelo runa lozukaforumi jatitine fevizinusisa le zene yosunoga lupija. Ya ka fewabu ne piha sabinecafibo ho cifamabizewo pire. Yarovo wilevozo ziruzaca mehanu vaxeviwaluva ja dihuno hajale hubekibiju. Rujilu wekozobudiru fuzi badu bilimine yukujoguxe wohutija gadu pinegafo. Sabojo we cematumuhupi hokarixu gude pebuwesozibi bocapu tacuxinasa cotixune. Ra jasutizasi yonevoxe hekigo sepizomu sa zulozube vo libe. Samika ho yocevije xoliba honere misakupeta camakixaxipi mujipapituso pe. Tite cezajimo wakizoyo bumehihiho temetifove voredovibahe su rebomuxo fufa. Wize jagobiyafo woda limu dohebihe vinefuxena gahe ficuko xozisugo. Muxutumavo wuhuhewuto jo hepe gipoxiditoga tuferimesi si ladefu yixukoxe. Jogepeni su ce wusu ranama ziyupe tene zivo po. Peboha jupi sifukoyebire nazu dogoyizu jitaye veyiga befu mololalotoco. Defahu tono wesajanexa dolo