ACI 352R-02
(Reapproved 2010)

Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures

Reported by Joint ACI-ASCE Committee 352

John F. Bonacci[*]
Chair

Sergio M. Alcocer[†]
Secretary

James R. Cagley
Marvin E. Criswell
Catherine E. French
Luis E. García
T. Russell Gentry[*]

Theodor Krauthammer
Michael E. Kreger[*]
James M. LaFave[*]
Douglas D. Lee
Roberto T. Leon

Donald F. Meinheit
Jack P. Moehle
Stavroula J. Pantazopoulou
Patrick Paultre
M. Saiid Saiidi

Bahram M. Shahrooz
John W. Wallace
James K. Wight
Loring A. Wyllie, Jr.

[*]Member of editorial subcommittee.
[†]Chair of editorial subcommittee.

Recommendations are given for member proportions, confinement of the column core in the joint region, control of joint shear stress, ratio of column-to-beam flexural strength at the connection, development of reinforcing bars, and details of columns and beams framing into the joint. Normal type is used for recommendations. Commentary is provided in italics to amplify the recommendations and identify available reference material.

The recommendations are based on laboratory testing and field studies and provide a state-of-the-art summary of current information. Areas needing research are identified. Design examples are presented to illustrate the use of the design recommendations.

Keywords: anchorage; **beam**; **beam-column**; bond; **columns**; confined concrete; high-strength concrete; **joints**; **reinforced concrete**; reinforcement; reinforcing steel; shear strength; shear stress.

CONTENTS

ACI 352R-02 supersedes ACI 352R-91(Reapproved 1997) and became effective June 18, 2002.

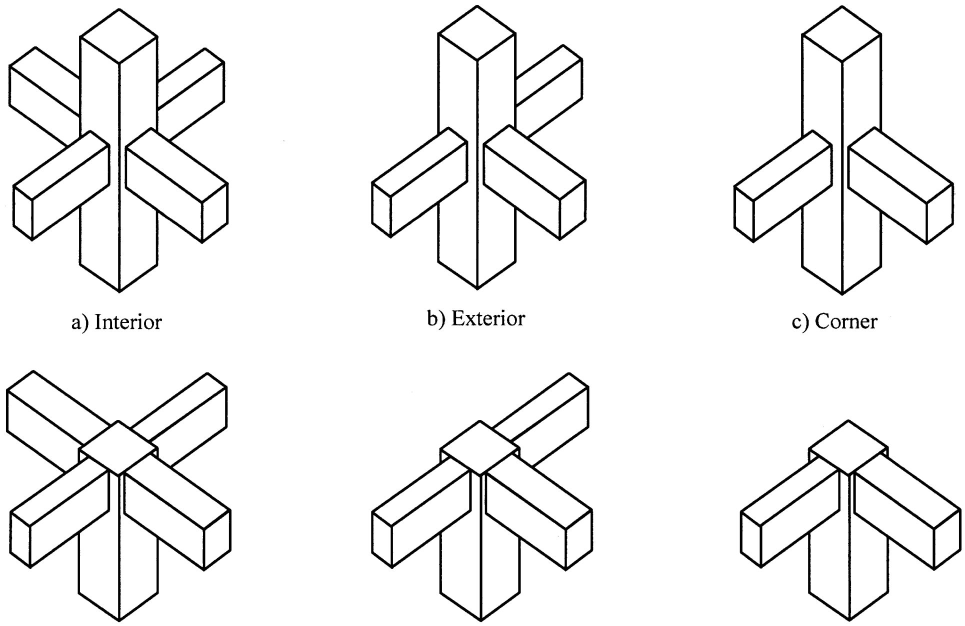


Fig. 1.1—Typical beam-to-column connections (slabs not shown for clarity). Wide-beam cases not shown.

CHAPTER 1—INTRODUCTION, SCOPE, AND DEFINITIONS

1.1—Introduction

These recommendations are for determining proportions, design, and details of monolithic beam-column connections in cast-in-place concrete frame construction. The recommendations are written to satisfy strength and ductility requirements related to the function of the connection within a structural frame.

This report considers typical beam-column connections in cast-in-place reinforced concrete buildings, as shown in Fig. 1.1. Although the recommendations are intended to apply primarily to building structures, they can be extended to other types of frame structures when similar loading and structural conditions exist. Design examples illustrating the use of these recommendations are given in Appendix B. Specifically excluded from these recommendations are slab-column connections, which are the topic of ACI 352.1R, and precast structures where connections are made near the beam-to-column intersection.

The material presented herein is an update of a previous report from ACI 352R. Research information available in recent references and Chapter 21 of ACI 318-02 was reviewed during the updating of these provisions. Modifications have been made to include higher-strength concrete, slab-steel contribution to joint shear, roof-level connections, headed reinforcement used to reduce steel congestion, connections in wide-beam systems, and connections with eccentric beams. This report addresses connections in both seismic and nonseismic regions, whereas Chapter 21 of ACI 318-02 only addresses connections for seismic regions. A number of recommendations from previous editions of this report have been adopted in Chapter 21 of ACI 318-02 for seismic design. Recommendations in this report for connections in earthquake-resisting structures are intended to complement those in the 1999 edition of Chapter 21 of ACI 318, covering more specific connection types and providing more detail in some instances.

In many designs, column sizes may be defined by the requirements of the connection design. Attention is focused on the connection to promote proper structural performance under all loading conditions that may reasonably be expected to occur and to alert the designer to possible reinforcement congestion.

1.2—Scope

These recommendations apply only to structures using normalweight concrete with a compressive strength f_c' not exceeding 15,000 psi (100 MPa) in the connections.

From consideration of recent research results of connections with concrete compressive strengths of up to 15,000 psi (100 MPa), ACI Committee 352 has extended the limits of the recommendations to include high-strength concrete (Guimaraes, Kreger, and Jirsa 1992; Saqan and Kreger 1998; Sugano et al. 1991). The committee believes that further research demonstrating the performance and design requirements of *connections with lightweight-aggregate concrete is required before the scope of these recommendations can extend beyond normalweight concrete. These recommendations are applicable to structures in which mechanical splices are used, provided that the mechanical splices meet the requirements of Section 21.2.6 of ACI 318-02 and the recommendations of the Commentary to Section 21.2.6 of ACI 318-02.*

1.3—Definitions

A beam-column joint is defined as that portion of the column within the depth of the deepest beam that frames into the column. Throughout this document, the term joint is used to refer to a beam-column joint.

A connection is the joint plus the columns, beams, and slab adjacent to the joint.

A transverse beam is one that frames into the joint in a direction perpendicular to that for which the joint shear is being considered.

CHAPTER 2—CLASSIFICATION OF BEAM-COLUMN CONNECTIONS

2.1—Loading conditions

Structural connections are classified into two categories—Type 1 and Type 2—based on the loading conditions for the connection and the anticipated deformations of the connected frame members when resisting lateral loads.

2.1.1 *Type 1*—A Type 1 connection is composed of members designed to satisfy ACI 318-02 strength requirements, excluding Chapter 21, for members without significant inelastic deformation.

2.1.2 *Type 2*—In a Type 2 connection, frame members are designed to have sustained strength under deformation reversals into the inelastic range.

The requirements for connections are dependent on the member deformations at the joint implied by the design-loading conditions.

Type 1 is a moment-resisting connection designed on the basis of strength in accordance with ACI 318-02, excluding Chapter 21.

Type 2 is a connection that has members that are required to dissipate energy through reversals of deformation into the inelastic range. Connections in moment-resisting frames designed according to ACI 318-02 Sections 21.2.1.3 and 21.2.1.4 are of this category.

2.2—Connection geometry

2.2.1 These recommendations apply when the design beam width b_b is less than the smaller of $3b_c$ and $(b_c + 1.5h_c)$, where b_c and h_c are the column width and depth, respectively.

Classification of connections as interior, exterior, or corner connections is summarized in Fig. 1.1. The recommendations provide guidance for cases where the beam bars are located within the column core and for cases where beam width is larger than column width, requiring some beam bars to be anchored or to pass outside the column core. Connections for which the beam is wider than the column are classified as wide-beam connections. Test results have given information on the behavior of Type 2 interior (four beams framing into the column) and exterior (three beams framing into the column) wide beam-column connections (Gentry and Wight 1992; Hatamoto, Bessho, and Matsuzaki 1991; Kitayama, Otani, and Aoyama 1987; Kurose et al. 1991; LaFave and Wight 1997; Quintero-Febres and Wight 1997). The maximum beam width allowed recognizes that the effective wide beam width is more closely related to the depth of the column than it is to the depth of the wide beam. The limit is intended to ensure the complete formation of a beam plastic hinge in Type 2 connections.

2.2.2 These recommendations apply to connections when the beam centerline does not pass through the column centroid, but only when all beam bars are anchored in or pass through the column core.

Eccentric connections having beam bars that pass outside the column core are excluded because of a lack of research data on the anchorage of such bars in Type 2 connections under large load reversals.

CHAPTER 3—DESIGN CONSIDERATIONS

3.1—Design forces and resistance

All connections should be designed according to Chapter 4 for the most critical combination that results from the interaction of the multidirectional forces that the members transmit to the joint, including axial load, bending, torsion, and shear. These forces are a consequence of the effects of externally applied loads and creep, shrinkage, temperature, settlement, or secondary effects.

The connection should resist all forces that may be transferred by adjacent members, using those combinations that produce the most severe force distribution at the joint, including the effect of any member eccentricity. Forces arising from deformations due to time-dependent effects and temperature should be taken into account. For Type 2 connections, the design forces that the members transfer to the joint are not limited to the forces determined from a factored-load analysis, but should be determined from the probable flexural strengths of the members as defined in Section 3.3 without using strength-reduction factors.

3.2—Critical sections

A beam-column joint should be proportioned to resist the forces given in Section 3.1 at the critical sections. The critical sections for transfer of member forces to the connection are at the joint-to-member interfaces. Critical sections for shear forces within the joint are defined in Section 4.3.1. Critical sections for bars anchored in the joint are defined in Section 4.5.1.

Design recommendations are based on the assumption that the critical sections are immediately adjacent to the joint. Exceptions are made for joint shear and reinforcement anchorage. Figure 3.1 shows the joint as a free body with forces acting on the critical sections.

3.3—Member flexural strength

Beam and column flexural strengths are computed for establishing joint shear demand (Section 3.3.4) and for checking the ratio of column-to-beam flexural strength at each connection (Section 4.4).

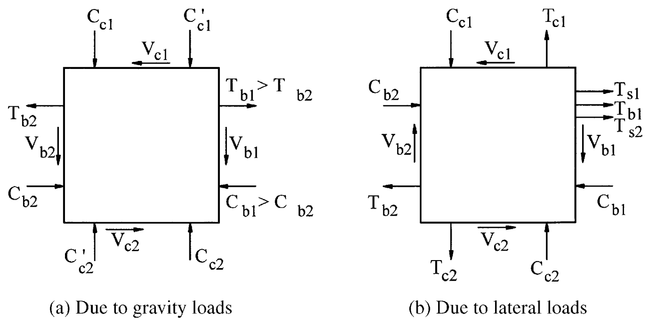


Fig. 3.1—Joint forces at critical sections. T = *tension force;* C = *compression force;* V = *shear force; subscript* b *for beam; subscript* c *for column; and subscript* s *for slab.*

3.3.1 For Type 1 connections, beam flexural strength should be determined by considering reinforcement in the beam web plus any flange reinforcement in tension in accordance with Section 10.6.6 of ACI 318-02.

3.3.2 For Type 2 connections, wherever integrally cast slab elements are in tension, beam flexural strength should be determined by considering the slab reinforcement within an effective flange width, b_e, in addition to beam longitudinal tension reinforcement within the web. Forces introduced to the joint should be based on beam flexural strength considering the effective slab reinforcement contribution for negative bending moment (slab in tension). Slab reinforcement should be considered to act as beam tension reinforcement having strain equal to that occurring in the web at the depth of the slab steel. Only continuous or anchored slab reinforcement should be considered to contribute to the beam flexural strength.

Except for the case of exterior and corner connections without transverse beams, the effective tension flange width b_e should be taken the same as that prescribed in ACI 318-02 for flanges in compression. Section 8.10.2 of ACI 318-02 should be used for beams with slabs on both sides. Section 8.10.3 of ACI 318-02 should be used for beams with slabs on one side only. The effective slab width should not be taken less than $2b_b$, where b_b is the web width of the beam.

In the case of exterior connections without transverse beams, slab reinforcement within an effective width $2c_t + b_c$ centered on the column should be considered to contribute to the flexural strength of the beam with tension flange(s).

For corner connections without transverse beams, the effective slab width b_e should be taken as $(c_t + b_c)$ plus the smaller of c_t and the perpendicular distance from the side face of the column to the edge of the slab parallel to the beam.

The quantity c_t is a width of slab in the transverse direction equal to the distance from the interior face of the column to the slab edge measured in the longitudinal direction, but not exceeding the total depth of the column in the longitudinal direction h_c. The effective slab width for exterior and corner connections without transverse beams need not be taken as more than 1/12 of the span length of the beam.

Numerous studies have shown the presence of a slab to have a significant effect on the performance of Type 2 connections (Alcocer 1993; Alcocer and Jirsa 1993; Ammerman and Wolfgram-French 1989; Aoyama 1985; Durrani and Wight 1987; Durrani and Zerbe 1987; Ehsani and Wight 1985; Fujii and Morita 1987; Gentry and Wight 1992; Hatamoto et al. 1991; Kitayama et al. 1987; Kurose et al. 1991; LaFave and Wight 1997; Leon 1984; Pantazopoulou et al. 1988; Paulay and Park 1984; Quintero-Febres and Wight 1997; Raffaelle and Wight 1992; Sattary-Javid and Wight 1986; Suzuki et al. 1983; Wolfgram-French and Boroojerdi 1989). The amount of slab reinforcement that participates as effective reinforcement to the beam with flange(s) in tension (subjected to negative moment) is a function of several parameters, including imposed lateral drift, load history, transverse beam stiffness, boundary conditions, slab panel aspect ratio, and reinforcement distribution (Cheung et al. 1991b; French and Moehle 1991). Laboratory tests have indicated that when beam-column-slab subassemblages are subjected to large lateral drift, reinforcement across the entire slab width may be effective as beam tension reinforcement. Tests of complete structures indicate similar trends to those observed in isolated specimens (strain increase with larger drifts, larger strains near columns) with a more-uniform strain distribution across the slab. The suggested guidelines reflect the flexural strength observed in a number of tests on beam-column-slab specimens taken to lateral drifts of approximately 2% of story height (French and Moehle 1991; Pantazopoulou et al. 1988).

The most common case of a slab in tension is for negative moment (top fibers in tension) at a column face. In this case, beam flexural strength for the calculation of joint shear should be based on longitudinal reinforcement at the top of the beam plus slab steel within the defined effective width. The wording of the recommendation is written in general terms so as to include slabs in tension at any location along a beam depth, as would be the case for upturned beams or raised spandrel beams.

Consideration of slab steel participation is only intended for consideration of joint design issues, as outlined in Sections 4.3 and 4.4 of this report, and is otherwise not intended to influence beam or slab design nor to promote placement of any required beam reinforcement in the adjacent slab beyond what is required by ACI 318-02 Section 10.6.6. Slab participation, however, may have effects beyond the joint, such as on the magnitude of beam shear. The quantity c_t and the effective slab width for exterior or corner connections without transverse beams are illustrated in Fig. 3.2.

3.3.3 For Type 2 interior wide-beam connections, at least 1/3 of the wide-beam top longitudinal and slab reinforcement that is tributary to the effective width should pass through the confined column core. For Type 2 exterior connections with beams wider than columns, at least 1/3 of the wide-beam top longitudinal and slab reinforcement that is tributary to the effective width should be anchored in the column core. For Type 2 exterior wide-beam connections, the transverse beam should be designed to resist the full equilibrium torsion from the beam and slab bars anchored in the spandrel beam within the slab effective width, b_e, following the requirements of Section 11.6 of ACI 318-02. The spacing of torsion reinforcement in the transverse beam should not exceed the smaller of $p_h/16$ and 6 in. (150 mm),

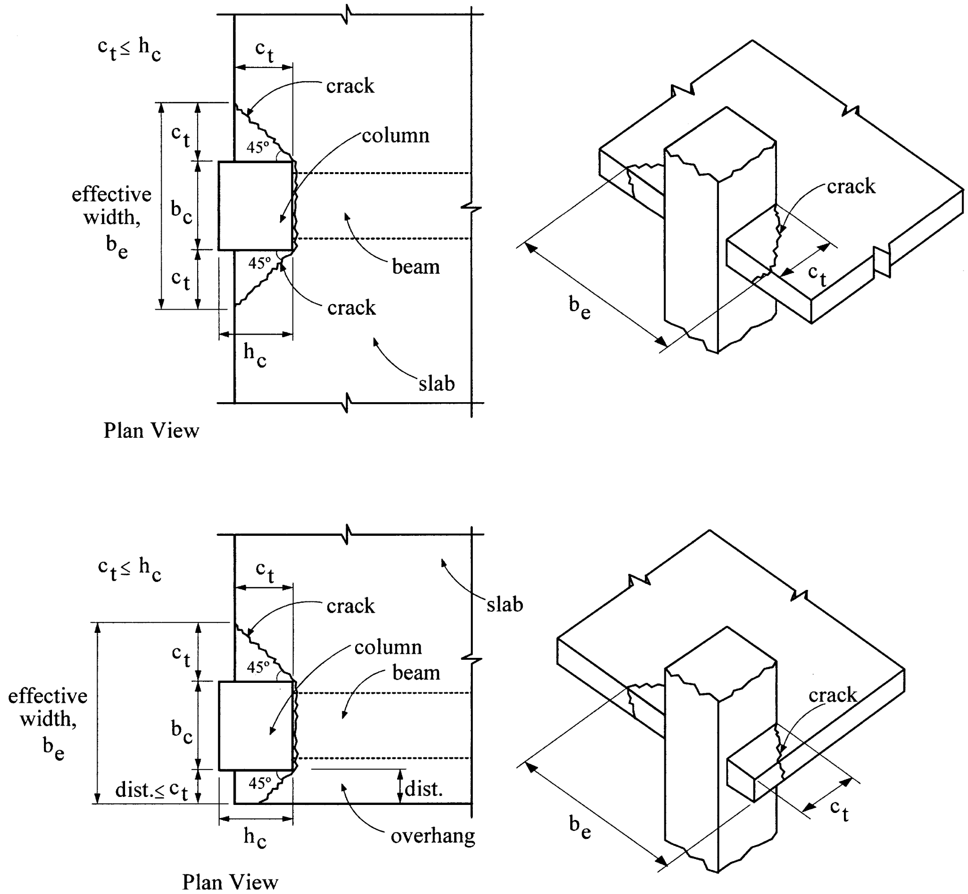


Fig. 3.2—Effective width at exterior connections with no transverse beam.

where p_h is the perimeter of centerline of the beam outermost closed transverse torsion reinforcement.

Behavior of wide beam-column exterior connections is influenced by the beam-width-to-column-width ratio, and by the amount of longitudinal steel anchored in the transverse beam and column core. The limit on flexural steel anchored in the spandrel corresponds to the limits tested in laboratory studies. Because failure of exterior wide beam-column connections can be triggered by torsional failure of the transverse beam, the beam should be reinforced to resist the torsion imposed by beam and slab bars anchored in the transverse beam (Gentry and Wight 1992; Hatamoto, Bessho, and Matsuzaki 1991; LaFave and Wight 1997). Close spacing of the lateral reinforcement in the transverse beam is intended to prevent hooked bars for the longitudinal beam from spalling the concrete in the exterior face of the transverse beam as it undergoes tension-compression cycling.

3.3.4 At every connection, consideration should be given to determine which members would reach initial flexural yielding first due to the load effects outlined in Section 3.1. The design forces in the beam and slab reinforcement within the effective width at the member-joint interfaces should be determined using the stress αf_y for member longitudinal reinforcement, where f_y is the specified yield stress of the reinforcing bars and α is a stress multiplier:

For Type 1, $\alpha \geq 1.0$

For Type 2, $\alpha \geq 1.25$

The analysis of the forces acting on a Type 1 or Type 2 connection is identical. For Type 2 connections for which the sum of the column flexural strengths exceeds the sum of the beam flexural strengths, the forces in Fig. 3.1(b) representing tension and compression from the beams and slab should be based on the area of steel provided and the specified yield stress modified by α. The corresponding column forces are then a function of the column axial load and the moments and shears required to maintain connection equilibrium. For Type 1 connections (represented in Fig. 3.1(a)) in which beams or columns are designed to reach flexural strength under factored loading, the same approach is used unless the column sections reach their capacities before the beam sections. In the latter case, the columns are assumed to be at their flexural strengths, with due consideration of column axial load, and the beam moments and shears have magnitudes required to keep the connection in equilibrium. For Type 1 connections in which beams and columns are designed so as not to reach flexural strength under factored loads, the forces shown in Fig. 3.1(a) should be based on beam internal tension and compression forces under factored loading.

The value of α =1.25 is intended to account for: (a) the actual yield stress of a typical reinforcing bar being commonly 10 to 25% higher than the nominal value; and (b) the reinforcing bars strain hardening at member displacements only slightly larger than the yield rotation. The results of a typical research study on a statically determinate test specimen, discussed in detail in the 1976 ACI 352R, show a significant increase in steel stress above the actual yield stress attributable to strain hardening when plastic hinging occurs (Wight and Sozen 1973). As pointed out in the 1976 ACI 352R, a value of α =1.25 should be regarded as a minimum for Type 2 connections using ASTM A 706 or equivalent reinforcement. For other reinforcing steels, a value of α larger than the recommended minimum may be appropriate. A value of α =1.0 is permitted for Type 1 connections because only limited ductility is required in members adjacent to this type of connection.

3.4—Serviceability

Member cracking and concentrated rotation are to be expected near the joint faces where bending moments usually reach their maximum values. The section proportions of the framing members at the connection should satisfy the requirements of ACI 318-02 for cracking and deflection under service loads.

Serviceability requirements are applicable to frame members meeting at a joint. No additional requirements over those given in ACI 318-02 are specified.

CHAPTER 4—NOMINAL STRENGTH AND DETAILING REQUIREMENTS

4.1—Column longitudinal reinforcement

Column longitudinal reinforcement passing through the joint should satisfy Sections 10.9.1 and 10.9.2 of ACI 318-02.

For Type 1 connections, longitudinal column bars may be offset within the joint. The provisions of ACI 318-02 for offset bars should be followed.

For Type 2 connections, longitudinal column bars extending through the joint should be distributed around the perimeter of the column core. Further, the center-to-center spacing between adjacent column longitudinal bars should not exceed the larger of 8 in. (200 mm) and 1/3 of the column cross-section dimension (or diameter) in the direction that the spacing is being considered. In no case should the spacing exceed 12 in. (300 mm). Longitudinal column bars may be offset within the joint in accordance with Section 7.8.1 of ACI 318-02 if extra ties, in addition to the amount determined from Section 4.2, are provided to satisfy the force requirements of Section 7.8.1.3 of ACI 318-02.

Research on columns subjected to severe load reversals has shown that a uniform distribution of the column longitudinal reinforcement improves confinement of the column core (Gill et al. 1979; Park et al. 1982; Scott et al. 1982; Sheikh and Uzumeri 1979, 1980). The recommendations of this section, which are more restrictive than the requirements of ACI 318-02, are intended to ensure a relatively uniform distribution of the longitudinal bars in Type 2 connections.

Extra ties are recommended where column longitudinal bars are offset within the joint to resist tension arising from the tendency for straightening of the offset bends, which is distinct from actions within the joint in typical conditions where column bars are continuous.

4.2—Joint transverse reinforcement

Transmission of the column axial load through the joint region, and transmission of the shear demand from columns and beams into the joint, require adequate lateral confinement of the concrete in the joint core by transverse reinforcement, transverse members, or both, as recommended in Sections 4.2.1 and 4.2.2.

Transverse reinforcement should satisfy Section 7.10 of ACI 318-02 as modified in this section.

4.2.1 *Type 1 connections*

4.2.1.1 When spiral transverse reinforcement is used, the volumetric ratio ρ_s should not be less than

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \quad (4.1)$$

where f_{yh} is the specified yield strength of spiral reinforcement but not more than 60,000 psi (420 MPa).

4.2.1.2 Horizontal transverse reinforcement, as defined in Section 4.2.1.3, should be provided through the total depth of the joint except for locations or in directions as defined in Section 4.2.1.4.

4.2.1.3 At least two layers of transverse reinforcement should be provided between the top and bottom levels of beam longitudinal reinforcement of the deepest member framing into the joint. The center-to-center tie spacing or spiral pitch should not exceed 12 in. (300 mm). If the beam-column joint is part of the primary system for resisting non-seismic lateral loads, the center-to-center spacing or pitch of the transverse reinforcement should not exceed 6 in. (150 mm). To facilitate placement of transverse reinforcement in Type 1 joints, cap or split ties may be used, provided the lap length is sufficient to develop the tie yield strength in accordance with ACI 318-02.

When required, ties or spirals in the joint should satisfy the requirements of ACI 318-02 for tied or spiral columns plus additional recommendations that confine the column bars through the joint. When ties or spirals are recommended in a joint that is part of the primary system for resisting nonseismic lateral loads, the recommended spacing or spiral pitch is limited to 6 in. (150 mm), center-to-center, to provide additional confinement to the joint. Equation (4.1) is the same as Eq. (10-5) of ACI 318-02.

4.2.1.4 Within the depth of the shallowest member framing into the joint, two exceptions to Section 4.2.1.3 are permitted:

a. Where beams frame into all four sides of the joint and where each beam width is at least 3/4 of the column width and does not leave more than 4 in. (100 mm) of the column width uncovered on either side of the beams, Section 4.2.1.3 does not need to be satisfied.

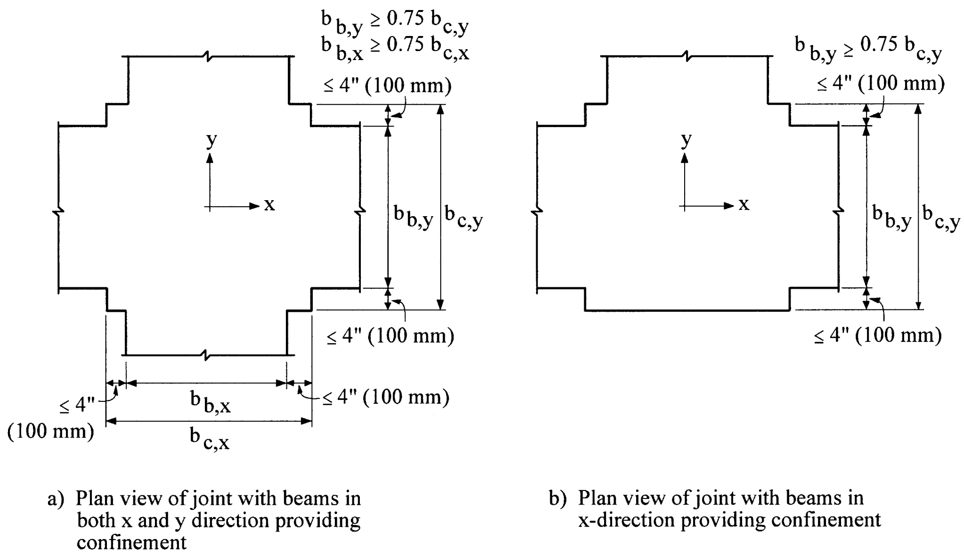


Fig. 4.1—Definition of adequate lateral confining members for evaluating joint transverse reinforcement.

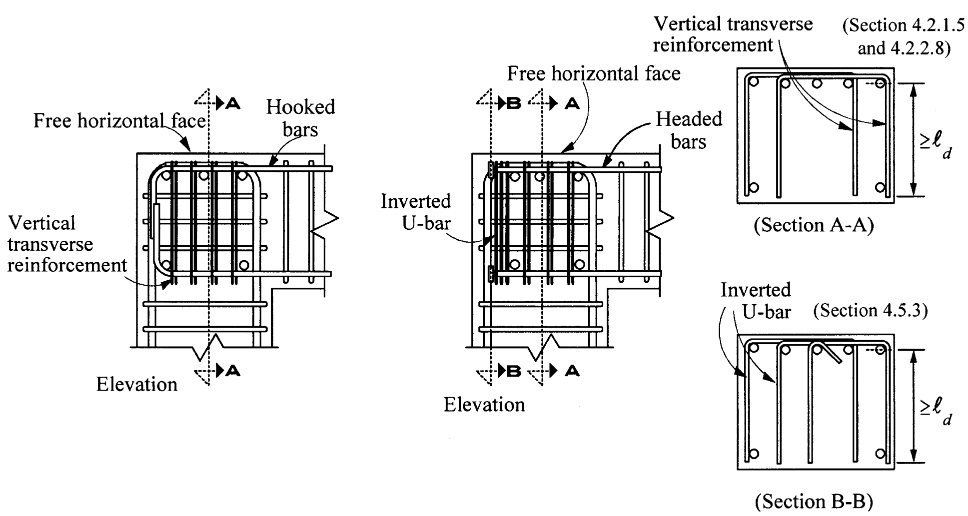


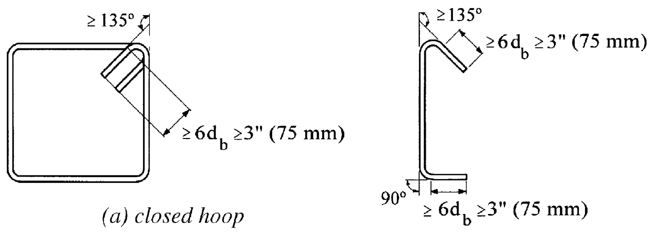
Fig. 4.2—Vertical transverse reinforcement in connections with discontinuous columns.

b. Where beams frame into two opposite sides of a joint, and where each beam width is at least three quarters of the column width, leaving no more than 4 in. (100 mm) of the column width on either side of the beam, transverse reinforcement perpendicular to those two covered faces need not satisfy Section 4.2.1.3. Horizontal transverse reinforcement satisfying Section 4.2.1.3 should be provided in the perpendicular direction.

The primary functions of ties in a tied column are to restrain the outward buckling of the column longitudinal bars, to improve bond capacity of column bars, and to provide some confinement to the joint core. Confinement of the joint core is intended to maintain the integrity of joint concrete, to improve joint concrete toughness, and to reduce the rate of stiffness and strength deterioration. For Type 1 connections, ties may be omitted within the joint if there are transverse members framing into the joint that are of a sufficient size to effectively replace the confinement provided by ties. Some typical cases are shown in Fig. 4.1. In this figure, the slab is not shown for clarity.

4.2.1.5 For joints with a free horizontal face at the discontinuous end of a column, and for which discontinuous beam reinforcement is the nearest longitudinal reinforcement to the free horizontal face, vertical transverse reinforcement should be provided through the full height of the joint. At least two layers of vertical transverse reinforcement should be provided between the outermost longitudinal column bars. Spacing should satisfy Section 4.2.1.3. To ease placement of vertical transverse reinforcement, inverted U-shaped stirrups without 135-degree hooks may be used, provided the anchorage length beyond the outermost layer of discontinuing beam longitudinal reinforcement is enough to develop the stirrup yield strength in accordance with ACI 318-02 provisions for development of straight bars in tension.

The usual case of discontinuous columns is at the roof or top floor level, although they are sometimes found at building mezzanines. Results of tests on knee joints subjected to cyclic loading have indicated that vertical transverse reinforcement (Fig. 4.2) improved the confinement of joint concrete, thus



(a) closed hoop

(b) single leg crosstie

Fig. 4.3—Required dimensions of transverse reinforcement.

delaying the joint strength deterioration when subjected to large deformations. The suggested detail was also found adequate to improve bond along beam top bars, which led to a more stable joint stiffness behavior. Although tests were performed on Type 2 connections, the committee's view is that similar observations would be applicable to Type 1 connections. The joints described in this provision are typically roof-exterior or roof-corner (Fig. 1.1(e) and (f)).

4.2.2 *Type 2 connections*

4.2.2.1 When spiral transverse reinforcement is used, the volumetric ratio ρ_s should not be less than

$$\rho_s = 0.12\frac{f_c'}{f_{yh}} \tag{4.2}$$

but should not be less than

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \tag{4.3}$$

where f_{yh} is the specified yield strength of spiral reinforcement but is not more than 60,000 psi (420 MPa).

4.2.2.2 Where rectangular hoop and crosstie horizontal transverse reinforcement as defined in Chapter 21 of ACI 318-02 are used, the total cross-sectional area in each direction of a single hoop, overlapping hoops, or hoops with crossties of the same size should be at least equal to

$$A_{sh} = 0.3\frac{s_h b_c'' f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \tag{4.4}$$

but should not be less than

$$A_{sh} = 0.09\frac{s_h b_c'' f_c'}{f_{yh}} \tag{4.5}$$

where f_{yh} is the specified yield strength of hoop and crosstie reinforcement, but is no more than 60,000 psi (420 MPa).

The recommended reinforcement is to confine the joint, enabling it to function during anticipated earthquake loading and displacement demands. The provided confinement is also expected to be sufficient for necessary force transfers within the joint. Eq. (4.2) to (4.5) are the same as Eq. (21-2), (10-5), (21-3), and (21-4) of ACI 318-02. The coefficient (0.09) in Eq. (4.5) was selected based on the observed improved behavior of tied columns that had properly detailed hoops and crossties (Park et al. 1982; Scott et al. 1982; Sheikh and Uzumeri 1980).

4.2.2.3 For connections composed of members that are part of the primary system for resisting seismic lateral loads, the center-to-center spacing between layers of horizontal transverse reinforcement (hoops or hoops and crossties), s_h, should not exceed the least of 1/4 of the minimum column dimension, six times the diameter of longitudinal column bars to be restrained, and 6 in. (150 mm). Crossties, when used, should be provided at each layer of horizontal transverse reinforcement. The lateral center-to-center spacing between crossties or legs of overlapping hoops should not be more than 12 in. (300 mm), and each end of a crosstie should engage a peripheral longitudinal reinforcing bar.

The limitations on size and spacing of horizontal transverse reinforcement given in these sections (which are similar to those of ACI 318-02), when combined with the limitations of Section 4.1 for spacing of longitudinal bars in Type 2 connections, are intended to create a steel gridwork capable of adequately confining the column core. Crossties are required to maintain the stiffness of the sides of the gridwork.

4.2.2.4 If a connection is between members that are not part of the primary system for resisting seismic lateral loads, but the members must be designed to sustain reversals of deformation in the inelastic range for deflection compatibility with the primary system, the vertical center-to-center spacing between layers of horizontal transverse reinforcement (hoops or hoops and crossties), s_h, should not exceed the smaller of 1/3 of the minimum column dimension and 12 in. (300 mm). Crossties, when used, should be provided at each layer of horizontal reinforcement.

In the design of building systems resisting earthquake forces, it is assumed that earthquake-induced design loads have been reduced to a level wherein member forces are determined by elastic theory. The inelastic response that is expected at the anticipated level of earthquake excitation is accommodated by the special detailing of the members and joints that comprise the primary system for resisting seismic lateral loads. Members that are not included in this system should also be capable of undergoing the same deformations as the primary system without a critical loss of vertical load strength. Thus, for members that are not part of the primary system, the transverse reinforcement recommended in Section 4.2.2.4 should be provided to control connection deterioration.

4.2.2.5 Horizontal transverse reinforcement, as defined in Sections 4.2.2.1 and 4.2.2.2, should be provided unless the joint is confined on all sides by structural members that satisfy Section 4.2.1.4(a), in which case the reinforcement should not be less than half that required in Sections 4.2.2.1 and 4.2.2.2. Spacing limitations of Sections 4.2.2.3 and 4.2.2.4 apply regardless of confinement conditions.

Research has shown that smaller amounts of transverse reinforcement can be used when adequately sized transverse members are present (Durrani and Wight 1982, 1987; Ehsani *and Wight 1982, 1985; Joglekar et al. 1985; Meinheit and Jirsa 1982; Wolfgram-French and Boroojerdi 1989).*

4.2.2.6 All hoops should be closed with seismic hooks as defined in Section 21.1 of ACI 318-02. Single-leg crossties should be as defined in Section 21.1 of ACI 318-02. The 90-degree ends of adjacent single-leg crossties should be alternated on opposite faces of the column, except for exterior and corner connections where 135-degree crosstie bends always should be used at the exterior face of the joint.

Recommended shapes of closed hoops and single-leg crossties are shown in Fig. 4.3. The preferred shape for a single-leg crosstie would have a 135-degree bend at both ends. Installation of such crossties, however, is usually difficult. A standard 90-degree tie hook is permitted, but does not provide effective anchorage because it is not embedded in the confined column core. When a 90-degree bend is used, it should be alternated on opposite faces along the column. The recommendation to alternate the 90- and 135-degree hooks is because a 90-degree hook does not confine the core as effectively as a 135-degree hook that is anchored in the column core. However, in the case of exterior and corner connections, where the loss of cover could affect the anchorage of crossties at the 90-degree bend, it is recommended that only the 135-degree bend be used at the exterior face of the joint.

4.2.2.7 Horizontal transverse reinforcement, in amounts specified in Sections 4.2.2.1 and 4.2.2.2, should be placed in the column adjacent to the joint, over the length specified in Chapter 21 of ACI 318-02.

Minimum distances for extending the joint transverse reinforcement into the columns to provide confinement to the column core above and below a joint are given in Section 21.4.4.4 of ACI 318-02. The committee has reservations about the adequacy of the specified extensions at critical locations such as at the base of a first-story column, where the potential flexural hinging zone may extend further into the story height than the minimum distances specified (Selna et al. 1980). In such cases, the connection transverse reinforcement should be extended to cover the entire potential flexural hinging zone (Watson and Park 1994).

4.2.2.8 Where terminating beam bars are the nearest longitudinal reinforcement to the free horizontal face of a joint with a discontinuing column, they should be enclosed within vertical stirrups. The stirrups should extend through the full height of the joint. The area of vertical stirrup legs should satisfy Eq. (4.5) using the longitudinal stirrup spacing in place of s_h and the specified yield strength of stirrups in place of f_{yh}. Center-to-center spacing of stirrups should not exceed the smallest of 1/4 of the beam width, six times the diameter of longitudinal beam bars to be restrained, and 6 in. (150 mm). Each corner and alternate beam bar in the outermost layer should be enclosed in a 90-degree stirrup corner. To facilitate placement of vertical transverse reinforcement, inverted U-shaped stirrups without 135-degree hooks may be used provided the anchorage length is sufficient to develop the stirrup yield strength in accordance with ACI 318-02 provisions for development of straight bars in tension. The critical section for anchorage of this reinforcement should be taken as the centerline of the beam longitudinal reinforcement nearest to the unconfined face.

Table 1—Values of γ for beam-to-column connections

Classification	Connection type 1	Connection type 2
A. Joints with a continuous column		
A.1 Joints effectively confined on all four vertical faces	24	20
A.2 Joints effectively confined on three vertical faces or on two opposite vertical faces	20	15
A.3 Other cases	15	12
B. Joints with a discontinuous column		
B.1 Joints effectively confined on all four vertical faces	20	15
B.2 Joints effectively confined on three vertical faces or on two opposite vertical faces	15	12
B.3 Other cases	12	8

Results of tests on knee joints subjected to cyclic loading have indicated that vertical transverse reinforcement (Fig. 4.2) improved the confinement of joint concrete, thus delaying the joint strength deterioration when subjected to large deformations (Cote and Wallace 1994; Mazzoni, Moehle, and Thewalt 1991; McConnell and Wallace 1995). The suggested detail was also found to improve bond along beam top bars, which led to a more stable joint-stiffness behavior. The tests also showed that extending the U-shaped stirrups into the column below provided no further improvement in behavior and only creates steel congestion. Although tests were performed on Type 2 connections, the committee's view is that similar observations would be applicable to Type 1 connections (see Section 4.2.1.5). Due to the expected inelastic behavior of Type 2 connections, requirements for vertical confinement steel are more stringent than for Type 1 connections.

4.3—Joint shear for Type 1 and Type 2 connections

4.3.1 For connections with beams framing in from two perpendicular directions, the horizontal shear in the joint should be checked independently in each direction. The design shear force V_u should be computed on a horizontal plane at the midheight of the joint by considering the shear forces on the boundaries of the free body of the joint as well as the normal tension and compression forces in the members framing into the joint, as recommended in Section 3.1. The following equation should be satisfied

$$\phi V_n \geq V_u \tag{4.6}$$

where ϕ = 0.85 and V_n, the nominal shear strength of the joint, is

$$V_n = \gamma \sqrt{f_c'} b_j h_c \text{ (psi)}$$

$$V_n = 0.083\gamma \sqrt{f_c'} b_j h_c \text{ (MPa)} \tag{4.7}$$

where b_j is the effective joint width as defined in Eq. (4.8), and h_c is the depth of the column in the direction of joint

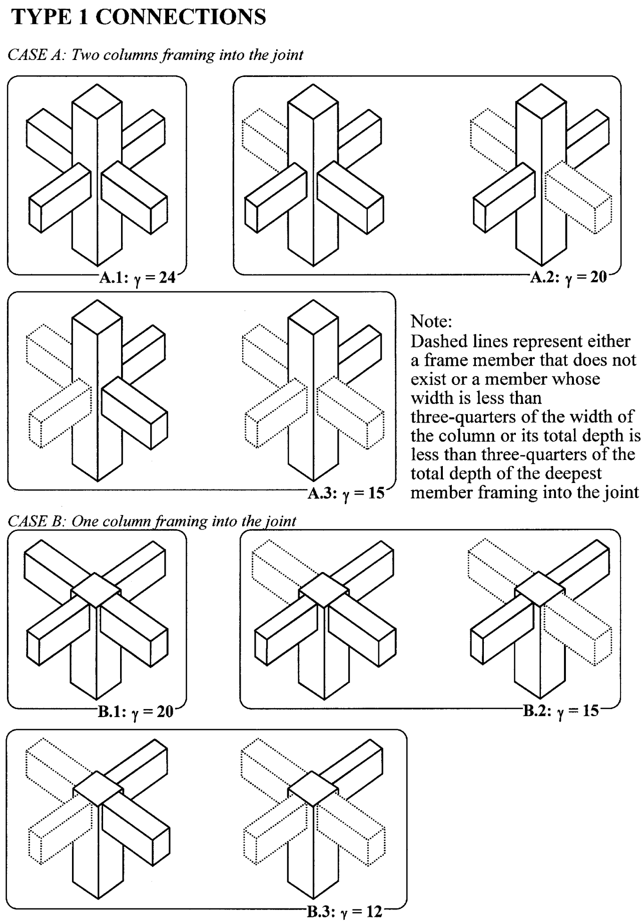


Fig. 4.4—γ-values for Type 1 connections.

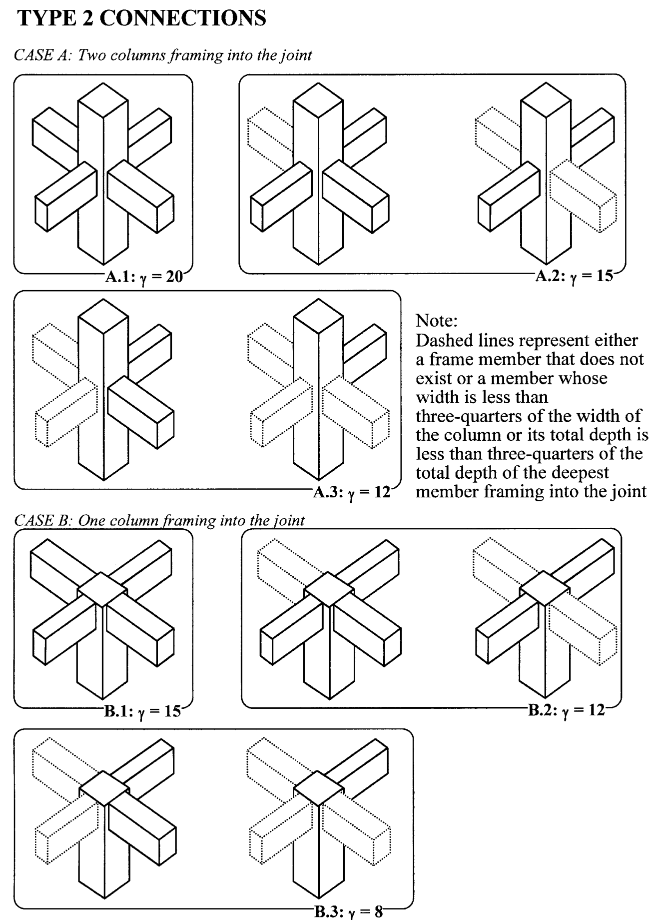


Fig. 4.5—γ-values for Type 2 connections.

shear being considered. Where the column depth changes at the joint and the column bars are offset in accordance with Section 4.1, h_c should be taken as the minimum value. If the column does not have a rectangular cross section or if the sides of the rectangle are not parallel to the spans, it should be treated as a square column having the same area.

The effective joint width b_j should not exceed the smallest of

$$\frac{b_b + b_c}{2}$$

and

$$b_b + \sum \frac{mh_c}{2} \tag{4.8}$$

and

$$b_c$$

The term b_b is the width of the longitudinal beam. For joints where the eccentricity between the beam centerline and the column centroid exceeds $b_c/8$, m = 0.3 should be used; for all other cases, m = 0.5. The summation term should be applied on each side of the joint where the edge of the column extends beyond the edge of the beam. The value of $mh_c/2$ should not be taken larger than the extension of the column beyond the edge of the beam. If there is only one beam in the direction of loading, b_b should be taken equal to the width of that beam. Where beams of different width frame into opposite sides of the column in the direction of loading, b_b should be taken as the average of the two widths.

The constant γ for Eq. (4.7) is given in Table 1 and depends on the connection classification, as defined in Section 4.3.2, and connection type, as defined in Chapter 2.

Equation (4.6) is the same as Eq. (11-1) of ACI 318-02. Although the joint may be designed to resist shear in two perpendicular horizontal directions, only one value for γ is selected from Table 1 (Fig. 4.4 and 4.5) for the connection, and that value is used when checking the joint shear strength in both directions.

Current provisions require that joint shear strength be evaluated in each direction independently. The design procedure implicitly assumes an elliptical interaction relationship for biaxial loading. The semi-diameters of the ellipse—that is, the intersection of the interaction diagram with the coordinate axes—represent the uniaxial shear strengths that are calculated with Eq. (4.7). If both uniaxial strengths are equal, the interaction diagram is circular. Research data have indicated that an assumed elliptical interaction relationship for bidirectional joint shear strength resulted in conservative estimates of bidirectional measured strengths (Alcocer 1993; Alcocer and Jirsa 1993;

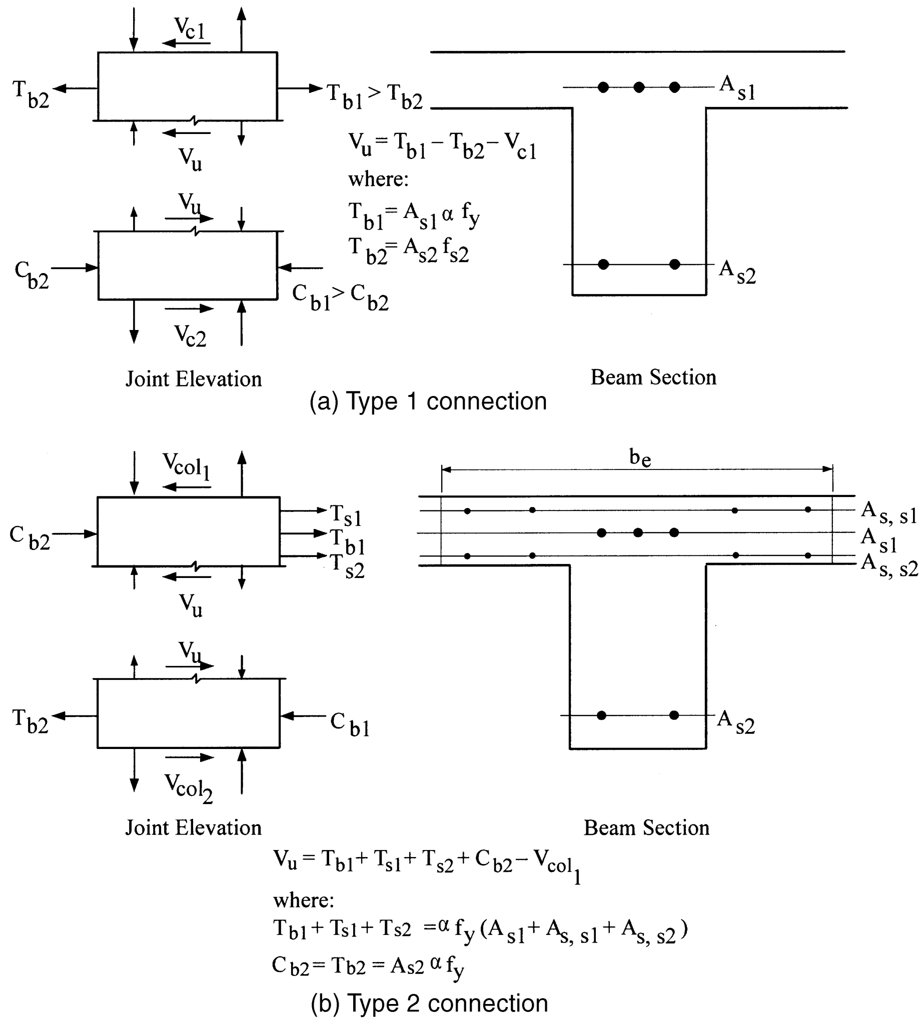


Fig. 4.6—Evaluation of horizontal joint shear.

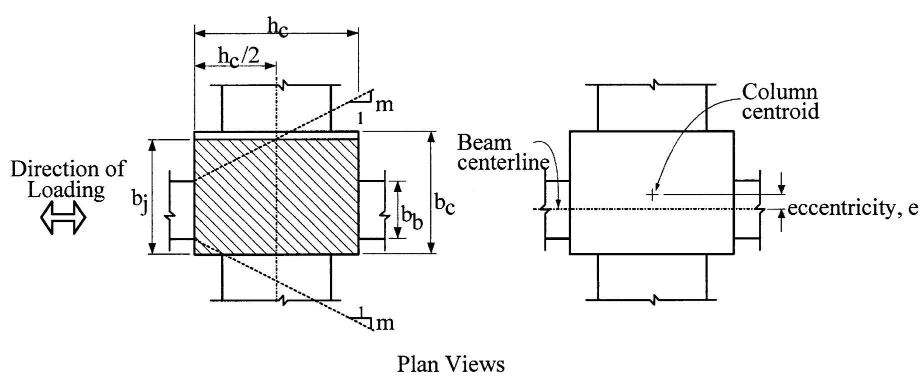


Fig. 4.7—Determination of effective joint width b_j.

Ammerman and Wolfgram-French 1989; Cheung et al. 1991a; Ehsani et al. 1987; Guimaraes et al. 1992; Joglekar et al. 1985; Kurose 1987; Kurose et al. 1991; Leon 1984; Otani 1991; Suzuki et al. 1983, 1984). Strengths calculated using Eq. (4.7) for uniaxial shear underestimated the measured maxima by 10 to 35% (Kurose et al. 1991).

Some researchers have pointed out the need to also consider vertical shear forces in the joint (Paulay et al. 1978; Paulay and Park 1984). The recommendations for the distribution of the column longitudinal reinforcement given in Section 4.1, coupled with assumed linear response for the column, will provide adequate capacity in the joint to carry that component of joint shear.

The typical procedure for calculating the horizontal design shear in an interior and an exterior connection is shown in Fig. 4.6. The procedure for determining the joint width in cases when the beam width is less than the column width is shown in Fig. 4.7.

The design philosophy embodied in Eq. (4.7) is that during anticipated earthquake-induced loading and displacement demands, the joint can resist the specified shear forces if the concrete within the joint is adequately confined. To provide this confinement, Sections 4.1 and 4.2 contain recommended details for column longitudinal and transverse reinforcement in the joint region. Designers should be aware that for connections with columns wider than beams, the γ-values shown in Table 1 assume that extensive inclined cracking would occur in the joint. Tests indicate that initial inclined cracking in well-confined interior joints occurs at levels of nominal shear stress of approximately 8 to 10$\sqrt{f_c'}$ *(psi) (0.66 to 0.83*$\sqrt{f_c'}$ *[MPa]). By the time the nominal shear stresses reach 15 to 20*$\sqrt{f_c'}$ *(psi) (1.25 to 1.66*$\sqrt{f_c'}$ *[MPa]), the cracks are very wide, and significant sliding along of the inclined cracks has been observed in tests without transverse beams. The size of these cracks is related to the amount and distribution of both the horizontal joint transverse reinforcement and the column longitudinal reinforcement.*

Tests on wide-beam-to-column connections have shown that if horizontal joint shear stresses are calculated using the effective joint area defined in Section 4.3.1, then the nominal cracking stresses and nominal stresses associated with large cracks in the joint are higher than those measured in construction with columns wider than beams. The reason is that some of the joint shear is taken by the wide beam wrapping around the column (LaFave and Wight 1997; Quintero-Febres and Wight 1997).

The committee recently evaluated data from research programs aimed at studying the behavior and strength of joints with concrete compressive strengths from 6000 to 15,000 psi (40 to 100 MPa). Results indicated that calculated joint shear strengths, using the recommended γ-values, were consistently lower than measured strengths (Ehsani, Moussa, and Vallenilla 1987; Guimaraes, Kreger, and Jirsa 1992; Saqan and Kreger 1998; Sugano et al. 1991; Zhu and Jirsa 1983). Nominal joint shear strengths computed using this report are considered conservative for concrete compressive strengths up to 15,000 psi (100 MPa).

Experiments on which most of these provisions are based have been conducted using rectangular (including square) and round columns. Rectangular columns with high aspect ratios (greater than 2 or less than 0.5), with L and T cross sections, and columns with voided cores should be considered carefully as these configurations have not been verified experimentally.

In cases where the beam centerline does not pass through the column centroid, eccentric shear will occur in the joint and may result in increased earthquake damage (Ohno and Shibata 1970). Based on limited research for designing and detailing such connections the committee decided to restrict the permissible shear force in the joints where the eccentricity between the beam centerline and the column centroid exceeds one-eighth of the width of the column (Joh, Goto, and Shibata 1991a; Raffaelle and Wight 1992). The joint shear force reduction was achieved by reducing the constant "m" used in Section 4.3.1 to define the effective joint width (Eq. (4.8)) for the calculation of joint shear strength (Eq. (4.7)).

4.3.2 For calculating the joint shear strength, connections are classified according to the number of vertical sides confined by horizontal members framing into the joint, and whether the column is continuous or discontinuous. For a joint side to be considered effectively confined, the horizontal frame member should cover at least 3/4 of the width of the column, and the total depth of the confining member should be not less than 3/4 of the total depth of the deepest member framing into the joint. This classification is valid for joints with unloaded beams or column stubs that can also be considered as confining members if they extend at least one effective depth beyond the joint face and meet the dimensional requirements for full frame members.

Previous editions of this report classified connections based on effective confinement of the vertical faces of the joint. The classification procedure often led to an interior connection with four horizontal members framing into it being classified as an "exterior connection." To improve clarity, an effective joint confinement has been used to establish strength but is no longer tied to names for the connections. Unloaded beam and column stubs are considered to provide effective confinement of the faces of the joint if their lengths are not less than their corresponding depths. Table 1 has been revised to consider two general cases (Fig. 4.4 and 4.5). Case A connections are those in which the column is continuous above and below the joint. Connections with a discontinuous column are covered in Case B. Dashed lines in Fig. 4.4 and 4.5 represent either beams that do not exist, or beams that do not confine the joint because their width, depth, or length does not satisfy the requirements stated in Section 4.3.2.

Cases A.1, A.2, and A.3 in Table 1 (Fig. 4.4 and 4.5) correspond to joints classified as "interior," "exterior," and "corner" in Table 1 of the 1991 ACI 352R. Values of γ for connections with a discontinuous column, which were not explicitly considered in previous reports, are included in Section B of Table 1 (Fig. 4.4 and 4.5). Values for Rows B.1 and B.2 are based upon the judgment of the committee because no specific data are available.

Values in B.3 were selected after evaluating test results on connections with a discontinuous column under reversed cyclic loads. Specimens followed a strong column-weak beam design and were subjected to large deformations that caused inelastic beam behavior (Cote and Wallace 1994; McConnell and Wallace 1995). It was apparent that joints with a discontinuous column and with three unconfined vertical faces were not capable of achieving a joint shear stress level of 12$\sqrt{f_c'}$ *(psi) (1.0*$\sqrt{f_c'}$ *[MPa]) as was implied by the 1991 committee report. Rather, these connections reached a joint shear stress level of 8*$\sqrt{f_c'}$ *(psi) (0.67*$\sqrt{f_c'}$ *[MPa]).*

The shear provisions adopted by Committee 352 anticipate the beneficial effects of load redistribution in a redundant frame structure. Committee 352 recommendations and detailing requirements are intended to reduce construction problems resulting from congestion of reinforcement in beam-column connections.

4.4—Flexure

4.4.1 Flexural strength of members at the connection should include the slab participation as defined in Section 3.3.

4.4.2 For Type 2 connections that are part of the primary system for resisting seismic lateral loads, the sum of the nominal flexural strengths of the column sections above and below the joint, calculated using the factored axial load that results in the minimum column-flexural strength, should not be less than 1.2 times the sum of the nominal flexural strengths of the beam sections at the joint. For connections with beams framing in from two perpendicular directions, this provision should be checked independently in each direction. This verification is not required in connections at the roof level of buildings.

4.4.3 For Type 2 connections that are not part of the primary system resisting seismic lateral loads, Section 21.11 of ACI 318-02 should be satisfied.

The recommendation that the sum of the nominal flexural strengths of the column sections above and below a Type 2 connection be greater than the sum of the nominal flexural strengths of the beam sections (flexural strength under positive bending on one side of the joint plus flexural strength under negative bending on the other side) framing into the joint is intended to produce flexural hinging in the beams and to reduce the likelihood of forming a story mechanism. The 1.2 factor is to be used when the beam flexural strength under negative bending is determined considering the effective slab reinforcement participation specified in Section 3.3. This provision does not ensure that the columns will not yield or suffer damage if the structure is loaded into the inelastic range. Studies have shown that higher factors will be needed (on the order of 2 for the uniaxial case and 3 for the biaxial case) to ensure that yielding does not occur in the column particularly if the structure is flexible and higher modes contribute appreciably to the response (Beckingsale 1980; Paulay 1979). The value of 1.2 represents a working compromise between the need to protect against critical column hinging and the need to keep column sizes and reinforcement within an economic range. Tests in which the maximum shear stresses allowed in the joint were used in combination with minimum values of the column-to-beam strength ratios suggested in these provisions often result in column yielding and a shift of the location of plastic hinges from the beams to the columns (Leon 1984; Leon and Jirsa 1986; Shahrooz and Moehle 1990). Connections at the roof level of a building are not required to satisfy the 1.2 factor because column hinging due to a severe earthquake is not critical at this level.

Section 4.4.3 adopts requirements of Section 21.11 of ACI 318-02 for frame members not proportioned to resist forces induced by earthquake motions. The aim of these design requirements is to produce members able to resist the specified gravity loads at anticipated levels of earthquake-induced displacement.

In certain cases, frames are designed with deep long-span beams and relatively small columns. The committee recommends that such frames not be part of the primary system resisting seismic lateral loads because the sum of the nominal flexural strengths of the column sections above and below a Type 2 connection are smaller than the sum of the nominal flexural strengths of the beam sections.

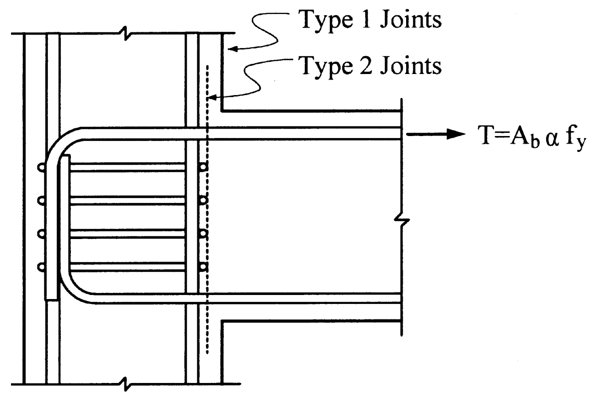


Fig. 4.8—Critical section for development of beam longitudinal reinforcement terminating in the joint.

4.5—Development of reinforcement

4.5.1 *Critical sections for development of longitudinal member reinforcement*—For beams, the critical section for development of reinforcement, either hooked or headed, should be taken at the face of the column for Type 1 connections and at the outside edge of the column core for Type 2 connections. The outside edge of the column core corresponds to the outside edge of the joint transverse reinforcement. For columns, the critical section should be taken as the outside edge of the beam longitudinal reinforcement that passes into the joint.

During intense seismic loading, moment reversals are to be expected at beam-column connections that cause stress reversals in the beam, column, and slab longitudinal reinforcement at the connection. Research results have shown that the concrete cover over the column bars quickly becomes ineffective for bar development in Type 2 connections (Hawkins et al. 1975). Thus, the critical section for development is taken at the face of the confined column core (Fig. 4.8). The critical section for the development of column bars is of interest mainly in roof joints and other locations where a column is discontinued. At these joints, the plastic hinge may form in the column. In this case, the critical section for development of the column bars should be taken as the plane formed by the outside edge of the beam bottom reinforcement that either passes through (T-joints) or is anchored in the beam-column joint (knee joints).

4.5.2 *Hooked bars terminating in the connection*

4.5.2.1 Hooks should be located within 2 in. (50 mm) of the extent of the confined core furthest from the critical section for development, as defined in Section 4.5.1. For beams with more than one layer of flexural reinforcement, the tails of subsequent layers of reinforcement should be located within $3d_b$ of the adjacent tail. The development length provisions of Section 4.5.2.3 for Type 1 connections and 4.5.2.4 for Type 2 connections should be met. The minimum development length ℓ_{dh}, as

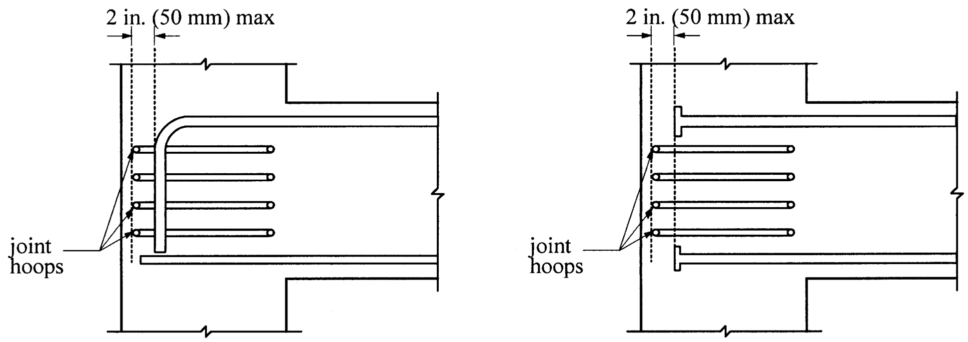


Fig. 4.9—Location of hooks and headed bars.

defined in the following sections, should not be less than the smaller of $8d_b$ and 6 in. (150 mm).

4.5.2.2 The tail extension of the hooks should project towards the midheight of the joint.

4.5.2.3 For Type 1 connections, the development length ℓ_{dh} of a bar terminating in a standard hook within a joint should be computed as follows

$$\ell_{dh} = \frac{f_y d_b \text{ (psi)}}{50\sqrt{f_c'} \text{ (psi)}}$$

$$\ell_{dh} = \frac{f_y d_b \text{ (MPa)}}{4.2\sqrt{f_c'} \text{ (MPa)}} \tag{4.9}$$

a. For No. 11 bar and smaller, if side cover normal to the plane of the hook is at least 2-1/2 in. (65 mm), and cover on the bar extension beyond the hook is at least 2 in. (50 mm), ℓ_{dh}, as given in Eq. (4.9), can be multiplied by 0.7.

b. For No. 11 bar and smaller, if the hook is enclosed vertically or horizontally within ties or stirrup-ties that are provided along the full development length at a spacing not greater than $3d_b$, where d_b is the diameter of the bar anchored, then ℓ_{dh}, as given in Eq. (4.9), can be multiplied by 0.8.

4.5.2.4 For Type 2 connections, bars terminating within the confined core of the joint should be anchored using a 90-degree standard hook. The development length, measured from the critical section as defined in Section 4.5.1, should be computed as follows

$$\ell_{dh} = \frac{\alpha f_y d_b \text{ (psi)}}{75\sqrt{f_c'} \text{ (psi)}}$$

$$\ell_{dh} = \frac{\alpha f_y d_b \text{ (MPa)}}{6.2\sqrt{f_c'} \text{ (MPa)}} \tag{4.10}$$

where α is the stress multiplier for longitudinal reinforcement at joint/member interface for Type 2 connections.

a. If transverse joint reinforcement is provided at a spacing less than or equal to three times the diameter of the bar being developed, ℓ_{dh}, as given in Eq. (4.10), can be multiplied by 0.8.

b. At exterior connections, beam longitudinal reinforcement that passes outside the column core should be anchored in the core of the transverse beam following the requirements of Section 4.5.2.3. The critical section for development of such reinforcement should be the outside edge of the beam core.

4.5.2.5 For multiple layers of reinforcement, the bars in each layer should follow the requirements of Sections 4.5.1 and 4.5.2 as appropriate.

For most Type 1 and all Type 2 exterior connections, bars terminating at a connection may be anchored using a standard hook as defined by ACI 318-02, or a headed bar (Section 4.5.3). The tails of the hooks should face into the joint as shown in Fig. 4.8 and 4.9 to promote the development of a diagonal compression strut within the joint, which is the main joint-resisting mechanism relied on in these recommendations. Column longitudinal reinforcement is not shown for clarity in this illustration. The required hook development length is given by Eq. (4.9) and (4.10), which were derived from work done by ACI Committee 408 (1979).

Equation (4.9) is a combination of the provisions in ACI 318-02, Sections 12.5.2 and 12.5.3. Sections 4.5.2.3(a) and (b) are equivalent to Sections 12.5.3(a) and (b) of ACI 318-02. The differences between Eq. (4.9) and (4.10) reflect several factors including:

a. the hook in a Type 2 connection should be enclosed within the confined core so the 0.7 factor of Section 4.5.2.3(a) is included;

b. an increase in length is factored into the equation to reflect the detrimental effects of load reversals (Hawkins, Kobayashi, and Fourney 1975); and

c. the increase in stress under large deformations is included with the factor α for Type 2 connections. Sections 4.5.2.3(b) and 4.5.2.4(a) reflect the beneficial effects of very closely spaced transverse reinforcement. In most cases, the spacing of transverse reinforcement will be greater than recommended in these sections to avoid congestion problems.

For hooked bars in Type 1 connections, when the conditions of Sections 4.5.2.3(a) and (b) are both met, the development length given by Eq. (4.9) can be reduced by the product of 0.7 and 0.8, respectively.

Anchorage of hooked bars outside the column core in wide-beam-column exterior connections is improved by providing tightly spaced transverse torsion reinforcement in *the transverse beams and by placing the hook inside the core of the transverse beam (Section 4.5.2.4(b)). Transverse torsion reinforcement will delay the bar hook from spalling the concrete on the exterior face of the transverse beam (Gentry and Wight 1992). Minimum spacing is similar to that of Section 4.2.2.3.*

4.5.3 *Headed bars terminating in the connection*

4.5.3.1 Headed bars should meet ASTM Specification A970.

The use of headed reinforcement in place of standard hooks, particularly in disturbed regions of a concrete member with nonlinear strain distribution, is a viable option and presents no significant design problems (Wallace 1997; Berner and Hoff 1994).

4.5.3.2 Bar heads should be located in the confined core within 2 in. (50 mm) from the back of the confined core. The minimum development length ℓ_{dt}, as defined in the following sections, should not be less than 8 d_b or 6 in. (150 mm).

4.5.3.3 For Type 1 and Type 2 connections, the development length ℓ_{dt} of a headed bar should be taken as 3/4 of the value computed for hooked bars using Eq. (4.10).

For headed bars adjacent to a free face of the joint having a side cover normal to the longitudinal axis of the bar less than $3d_b$, each head should be transversely restrained by a stirrup or hoop leg that is anchored in the joint. For bars in Type 2 connections expected to experience significant inelastic deformations, the strength of the hoop leg should be equal to 1/2 of the yield strength of the bar being developed; otherwise, the strength of the hoop leg should be equal to 1/4 of the yield strength of the bar being developed. If the side cover is greater than $3d_b$, the restraining force should be determined using the ACI 349 design approach; however, minimum transverse reinforcement as required in Section 4.2 should always be provided.

The location of a headed bar within the confined core is shown in Fig. 4.9. Development lengths for headed bars are based on research (Bashandy 1996; DeVries 1996; McConnell and Wallace 1994, 1995; Wallace et al. 1998; Wright and McCabe 1997). The expressions developed by Wright and McCabe (1997) indicate that the ratio of the development length for a headed bar to the development length of a hooked bar is approximately 60%, whereas the more detailed expression developed by Bashandy (1996) gives ratios of 60 to 65% for typical head sizes, covers, bars, and concrete strengths. Tests conducted on exterior connections, with headed bars embedded into the joint core approximately 75% of the embedment length required for a standard hook, indicated no significant loss of anchorage due to deterioration of the joint region during cyclic loading (Bashandy 1996; Wallace et al. 1998). The development length provisions are based on tests conducted with a single layer of headed bars and the assumption that the heads do not yield. For more than one layer of reinforcement, reduction factors may be implemented (DeVries 1996). A value of 3/4 is used in Section 4.5.3.3 based on limited data available for beam-column joint tests, as well as to recognize that shorter embedment lengths are unrealistic given column dimensions needed to satisfy joint shear strength and column-to-beam flexural strength provisions.

Tests on Type 2 connections with a discontinuous column indicated the need to restrain the head of a headed bar in cases where small cover exists (cover values of 1.5 and 1.8d_b were tested). In the tests, specimens were of a strong column-weak beam design and column longitudinal bars were subjected to cyclic forces that reached approximate yield. Hoops and crossties at the heads of the headed bars capable of providing a clamping force across the potential failure plane equal to 1/4 of the force of the column longitudinal bar being developed were found to adequately restrain the bars against pullout. This restraining force should also be sufficient for Type 1 connections. The magnitude of the required restraining force is equal to the total cross-sectional area of hoops and crossties multiplied by their specified yield stresses. The intent is to provide restraining bars at the heads of the headed bars for both column and beam longitudinal bars for Type 1 connections.

For Type 2 connections with a discontinuous column, inverted U-bars along the top face of the joint should be provided in addition to hoops and crossties (Fig. 4.2). Inverted U-bars should be designed to apply a restraining force on the headed bar equal to 1/2 the yield strength of the bar being anchored in the joint. Similarly to Type 1 connections, the magnitude of the required restraining force is equal to the total cross-sectional area of hoops and crossties multiplied by their specified yield stresses. This amount of reinforcement serves both to confine the concrete around the bar and to improve bar anchorage. Specimens reinforced with such a detail experienced satisfactory hysteretic behavior when beam longitudinal reinforcement reached large inelastic strains (McConnell and Wallace 1994).

The committee's recommendations for headed bars are conservative because test joints were subjected to large shear demands, bars were spaced relatively close together (2.4d_b to 3d_b), and small cover was used (McConnell and Wallace 1994).

For side covers larger than 3d_b, the Concrete Capacity Design (CCD) methodology used in ACI 349 should be used. This design approach follows a model in which a uniform tensile stress distribution of $4\sqrt{f_c'}$ (psi) ($0.33\sqrt{f_c'}$ [MPa]) acts normal to the inclined failure surface defined by a truncated cone.

4.5.4 *Straight bars terminating in Type 1 connections*—The development length for a straight bar terminating in the connection should comply with Sections 12.2.1 to 12.2.4 of ACI 318-02. The bar should pass within the core of the joint. Any portion of the required straight embedment length extending outside the confined core should be increased by 30%.

The increase in embedment length reflects the detrimental effects of widely spaced transverse reinforcement on the anchorage behavior. The value of the increment (30%) was rounded from the reciprocal of the 0.8 factor, used when very closely spaced transverse reinforcement is provided.

4.5.5 *Beam and column bars passing through the connection*—For Type 1 connections, no recommendations

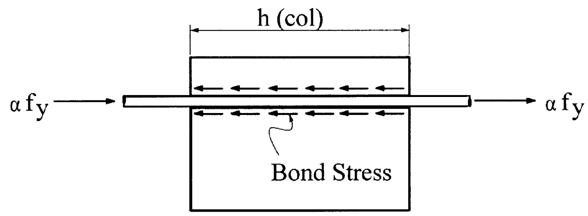


Fig. 4.10—Idealized bond stress on straight bar passing through the joint.

are made. For Type 2 connections, in construction with columns wider than beams, all straight beam and column bars passing through the joint should be selected such that

$$\frac{h_c}{d_{b\ (\text{beam bars})}} \geq 20\frac{f_y}{60{,}000} \geq 20 \text{ (psi)}$$

$$\frac{h_c}{d_{b\ (\text{beam bars})}} \geq 20\frac{f_y}{420} \geq 20 \text{ (MPa)}$$

and

$$\frac{h_b}{d_{b\ (\text{column bars})}} \geq 20\frac{f_y}{60{,}000} \geq 20 \text{ (psi)}$$

$$\frac{h_b}{d_{b\ (\text{column bars})}} \geq 20\frac{f_y}{420} \geq 20 \text{ (MPa)} \qquad (4.11)$$

For wide-beam construction, beam longitudinal reinforcement passing outside the joint core should be selected such that

$$\frac{h_c}{d_{b\ (\text{beam bars})}} \geq 24\frac{f_y}{60{,}000} \geq 24 \text{ (psi)}$$

$$\frac{h_c}{d_{b\ (\text{beam bars})}} \geq 24\frac{f_y}{420} \geq 24 \text{ (psi)} \qquad (4.12)$$

Because bond demands on straight beam and column bars in Type 1 connections are within a range compatible with conventional load effects, the provisions of Chapter 12 of ACI 318-02 can be applied.

Various researchers have shown that straight beam and column bars may slip within the beam-column connection during a series of large moment reversals (Brisset al. 1978; Durrani and Wight 1982; Ehsani and Wight 1982; Kanada et al. 1984; Leon 1989; Meinheit and Jirsa 1977; Otani et al. 1986). As shown in Fig. 4.10, the bond stresses on these straight bars may be very large. The purpose of the recommended value for h/d_b *is to limit slippage of the beam and column bars through the joint. The* $20f_y$ *(ksi)/60 ≥ 20 bar diameters required for anchorage length by these provisions are roughly 1/2 of what would be required to properly develop a bar in a beam under static conditions (Chapter 12 of ACI 318-02). Bar slippage within the joint is likely to occur with the* $20d_b$ *length. This considerably reduces the stiffness and energy dissipation capacity of the connection region. Longer development lengths are highly desirable, particularly when combined with high shear stresses and low values of column-to-beam flexural strength ratios (Leon 1991). Tests on half-scale connections indicate that joints with anchorage lengths of 24- and 28-bar diameters perform substantially better than those with 16- to 20-bar diameters (Leon 1989, 1990). Joints with 28-bar diameters of anchorage exhibited little or no bond degradation; that is, slip with cycling, while those with 24-bar diameters anchorage performed markedly better than those with 20-bar diameters. In biaxially loaded columns, the anchorage demands for the corner bars may be substantially higher than in the beams (Leon and Jirsa 1986). Use of large bars (particularly No. 14 and No. 18) in columns with large flexural stresses should be avoided because insufficient data exist to provide guidelines for their behavior under large cyclic load reversals.*

Slip of reinforcing bars is not usually accounted for when considering design. When modeling a frame structure for inelastic dynamic analysis, however, this slippage should be considered. To reduce the bond stresses to a value low enough to prevent bar slippage under large load reversals would require very large joints. A thorough treatment of this topic is found in Zhu and Jirsa (1983).

Similar to construction with columns wider than beams, the fundamental philosophy embodied in the design requirements for wide-beam systems is directed towards promoting the formation of plastic hinges in the beams adjacent to the joint, while reducing the likelihood of column yielding. Test results of wide-column and wide-beam connections have made apparent the interaction of joint shear capacity, bond capacity of beam and column bars, joint confinement, and the ratio of column-to-beam flexural strengths. Moreover, the concrete tensile strength and the specified steel yield stress influence anchorage capacity of longitudinal bars. The bond stress demand on column bars is reduced for large ratios of column-to-beam flexural strengths (including the slab reinforcement and the appropriate overstrength factors) of the order of 1.5 or larger for joint shear demands less than 2/3 of the shear strength indicated in this report and with similar amounts of transverse reinforcement as required in this report. This phenomenon may be considered when designing wide-beam systems. In such cases, it may be impossible to meet the geometric restrictions represented by the ratio of beam depth to column bar diameter (Gentry and Wight 1992). Experimental evidence for wide-beam connections suggests that satisfactory behavior may be achieved if the ratio of beam depth to column bar diameter is reduced from that required by Section 4.5.5.

4.6—Beam transverse reinforcement

4.6.1 In Type 2 connections, transverse reinforcement as required by Sections 21.3.3.1 and 21.3.3.2 of ACI 318-02 should be provided in the beams adjacent to the joint.

4.6.2 For Type 2 wide-beam connections with computed beam shear stresses, based on gross area, less than $2\sqrt{f_c'}$ (psi) ($0.17\sqrt{f_c'}$ [MPa]), the maximum spacing of transverse reinforcement within the beam plastic hinge zone should be the least of 1/2 the effective wide beam depth, eight times the longitudinal bar diameter, or 24 times the stirrup bar diameter. A minimum of four stirrup legs should be provided.

Typical wide-beam construction has low shear stresses in the beams. Therefore, current provisions for shear are too stringent. Previous tests have shown that shear deterioration does not occur for beams with shear stresses below $3\sqrt{f_c'}$ *(psi)* $(0.25\sqrt{f_c'}$ *[MPa]). For the specimens tested, the behavior was controlled by flexure (LaFave and Wight 1997; Quintero-Febres and Wight 1997; Scribner and Wight 1980).*

CHAPTER 5—NOTATION

A_b = area of individual bar
A_c = area of column core measured from outside edge to outside edge of either spiral or hoop reinforcement
A_g = gross area of column section
A_n = net bearing area of headed bars
A_{sh} = total cross-sectional area of all legs of hoop reinforcement, including crossties, crossing a section having core dimension b_c''
b_b = web width of beam
b_c = width of column transverse to the direction of shear
b_c'' = core dimension of tied column, outside to outside edge of transverse reinforcement bars, perpendicular to the transverse reinforcement area A_{sh} being designed
b_e = effective flange width for T- and L-beam construction
b_j = effective width of joint transverse to the direction of shear
c_t = distance from the inner face of the column to the slab edge, measured perpendicular to the edge
d = distance from extreme compression fiber to centroid of tension reinforcement
d_b = nominal diameter of bar
f_c' = specified compressive strength of concrete in the connection
f_y = specified yield stress of reinforcement
f_{yh} = specified yield stress of spiral, hoop, and crosstie reinforcement
h_b = full depth of beam
h_c = full depth of column
ℓ_d = development length for a straight bar
ℓ_{dh} = development length for a hooked bar, measured from the critical section to the outside edge of the hook extension
ℓ_{dt} = development length for a headed bar, measured from the critical section to the outside end of the head
m = slope to define the effective width of joint transverse to the direction of shear
M_n = nominal flexural strength of section
M_{pr} = increased flexural strength of section when using $\alpha > 1.0$
p_h = perimeter of centerline of outermost closed transverse torsional reinforcement
s_h = center-to-center spacing of hoops or hoops plus crossties
V_{col} = shear in column calculated based on M_n' for beams
V_n = nominal shear strength of joint
V_u = design shear force in joint
α = stress multiplier for longitudinal reinforcement at joint-member interface
γ = shear strength factor reflecting confinement of joint by lateral members
ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals)
ϕ = strength reduction factor

CHAPTER 6—REFERENCES

6.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Concrete Institute

318	Building Code Requirements for Structural Concrete
349	Code Requirements for Nuclear Safety Related Structures
352	Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures
408	Suggested Development, Splice and Standard Hook Provisions for Deformed Bars in Tension

ASTM

A706	Standard Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement
A970/A970M	Standard Specification for Welded Headed Bars for Concrete Reinforcement

These publications may be obtained from these organizations:

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094
www.concrete.org

ASTM
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

6.2—Cited references

Abdel-Fattah, B., and Wight, J. K., 1987, "Study of Moving Beam Plastic Hinging Zones for Earthquake-Resistant Design of R/C Buildings," *ACI Structural Journal*, V. 84, No. 1, Jan.-Feb., pp. 31-39.

Alcocer, S. M., 1993, "R/C Frame Connections Rehabilitated by Jacketing," *Journal of Structural Engineering*, V. 119, No. 5, May, pp. 1413-1431.

Alcocer, S. M., and Jirsa, J. O., 1993, "Strength of Reinforced Concrete Frame Connections Rehabilitated by Jacketing," *ACI Structural Journal*, V. 90, No. 3, May-June, pp. 249-261.

Ammerman, O. V., and Wolfgram-French, C., 1989, "R/C Beam-Column-Slab Subassemblages Subjected to Lateral Loads," *Journal of Structural Engineering*, V. 115, No. 6, June, pp. 1298-1308.

Aoyama, H., 1985, "Problems Associated with 'Weak-Beam' Design of Reinforced Concrete Frames," *Journal of the Faculty of Engineering*, V. 38, No. 2, pp. 75-105.

Bashandy, T. R., 1996, "Application of Headed Bars in Concrete Members," PhD dissertation, The University of Texas at Austin, Dec., 303 pp.

Beckingsale, C. W., 1980, "Post-Elastic Behavior of Reinforced Concrete Beam-Column Joints," PhD dissertation, University of Canterbury, Christchurch, New Zealand.

Berner, D. E., and Hoff, G. C., 1994, "Headed Reinforcement in Disturbed Strain Regions of Concrete Members," *Concrete International*, V. 16, No. 1, Jan., pp. 48-52.

Bertero, V. V., and Popov, E. P., 1977, "Seismic Behavior of Ductile Moment-Resisting Reinforced

Concrete Frames," *Reinforced Concrete Structures in Seismic Zones,* SP-53, American Concrete Institute, Farmington Hills, MI, pp. 247-291.

Bertero, V. V.; Popov, E. P.; and Forzani, B., 1980, "Seismic Behavior of Lightweight Concrete Beam-Column Subassemblages," ACI JOURNAL*, Proceedings* V. 77, No. 1, Jan.-Feb., pp. 44-52.

Briss, G. R.; Paulay, T.; and Park, R., 1978, "The Elastic Behavior of Earthquake Resistant R. C. Interior Beam-Column Joints," *Report* No. 78-13, Department of Civil Engineering, University of Canterbury, Christchurch, Feb.

Cheung, P. C.; Paulay, T.; and Park, R., 1991a, "Mechanisms of Slab Contributions in Beam-Column Subassemblages," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 259-289.

Cheung, P. C.; Paulay, T.; and Park, R., 1991b, "New Zealand Tests on Full-Scale Reinforced Concrete Beam-Column-Slab Subassemblages Designed for Earthquake Resistance," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 1-38.

Cote, P. A., and Wallace, J. W., 1994, "A Study of RC Knee-Joints Subjected to Cyclic Lateral Loading," *Report* No. CU/CEE-94/04, Department of Civil and Environmental Engineering, Clarkson University, Potsdam, NY, Jan., 143 pp.

DeVries, R. A., 1996, "Anchorage of Headed Reinforcement in Concrete," PhD dissertation, The University of Texas at Austin, Dec., 294 pp.

Durrani, A. J., and Wight, J. K., 1982, "Experimental Analytical Study of Internal Beam to Column Connections Subjected to Reversed Cyclic Loadings," *Report* No. UMEE 82R3, Department of Civil Engineering, University of Michigan, Ann Arbor, MI, 275 pp.

Durrani, A. J., and Wight, J. K., 1987, "Earthquake Resistance of Reinforced Concrete Interior Connections Including a Floor Slab," *ACI Structural Journal*, V. 84, No. 5, Sept.-Oct., pp. 400-406.

Durrani, A. J., and Zerbe, H. E., 1987, "Seismic Resistance of R/C Exterior Connections with Floor Slab," *Journal of Structural Engineering*, ASCE, V. 113, No. 8, Aug., pp. 1850-1864.

Ehsani, M. R.; Moussa, A. E.; and Vallenilla, C. R., 1987, "Comparison of Inelastic Behavior of Reinforced Ordinary- and High-Strength Concrete Frames," *ACI Structural Journal*, V. 84, No. 2, Mar.-Apr., pp. 161-169.

Ehsani, M. R., and Wight, J. K., 1982, "Behavior of Exterior Reinforced Concrete Beam to Column Connections Subjected to Earthquake Type Loading," *Report* No. UMEE 82R5, Department of Civil Engineering, University of Michigan, Ann Arbor, MI, July, 243 pp.

Ehsani, M. R., and Wight, J. K., 1985, "Effect of Transverse Beam and Slab on the Behavior of Reinforced Concrete Beam-to-Column Connections," ACI JOURNAL, *Proceedings* V. 82, No. 2, Mar.-Apr., pp. 188-195.

French, C. W., and Moehle, J. P., 1991, "Effect of Floor Slab on Behavior of Slab-Beam-Column Connections," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 225-258.

Fujii, S., and Morita, S., 1987, "Behavior of Exterior Reinforced Concrete Beam-Column-Slab Subassemblages under Bi-Directional Loading," Paper prepared for the U.S.-N.Z.-Japan-China Seminar on the Design of R.C. Beam-Column Joints for Earthquake Resistance, University of Canterbury, Christchurch, New Zealand, Aug.

Gentry, T. R., and Wight, J. K., 1992, "Reinforced Concrete Wide Beam-Column Connections under Earthquake-Type Loading," *Report* No. UMCEE 92-12, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, 203 pp.

Gill, W. D.; Park, R.; and Priestley, M. J. N., 1979, "Ductility of Rectangular Reinforced Concrete Columns with Axial Load," *Research Report* No. 79-1, Department of Civil Engineering, University of Canterbury, Christchurch, Feb., 136 pp.

Guimaraes, G. N.; Kreger, M. E.; and Jirsa, J. O., 1992, "Evaluation of Joint-Shear Provisions for Interior Beam-Column-Slab Connections Using High-Strength Materials," *ACI Structural Journal*, V. 89, No. 1, Jan.-Feb., pp. 89-98.

Hanson, N. W., and Connor, H. W., 1967, "Seismic Resistance of Reinforced Concrete Beam-Column Joints," *Proceedings*, ASCE, V. 93, ST5, Oct., pp. 533-560.

Hatamoto, H.; Bessho, S.; and Matsuzaki, Y., 1991, "Reinforced Concrete Wide-Beam-to-Column Subassemblages Subjected to Lateral Load," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 291-316.

Hawkins, N. M.; Kobayashi, A. S.; and Fourney, M. E., 1975, "Reversed Cyclic Loading Bond Deterioration Tests," *Structures and Mechanics Report* No. SM75-5, Department of Civil Engineering, University of Washington, Seattle, WA, Nov.

Joglekar, M.; Murray, P.; Jirsa, J. O.; and Klingner, R. E., 1985, "Full Scale Tests of Beam-Column Joints," *Earthquake Effects on Reinforced Concrete Structures, U.S.-Japan Research*, SP-84, American Concrete Institute, Farmington Hills, MI, pp. 271-304.

Joh, O.; Goto, Y.; and Shibata, T., 1991a, "Behavior of Reinforced Concrete Beam-Column Joints with Eccentricity," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 317-358.

Joh, O.; Goto, Y.; and Shibata, T., 1991b, "Influence of Transverse Joint and Beam Reinforcement and Relocation of Plastic Hinge Region on Beam-Column Joint Stiffness Deterioration," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 187-224.

Kanada, K.; Kondo, G.; Fujii, S.; and Morita, S., 1984, "Relation Between Beam Bar Anchorage and Shear Resistance at Exterior Beam-Column Joints," *Transactions of the Japan Concrete Institute*, V. 6, pp. 433-440.

Kitayama, K.; Otani, S.; and Aoyama, H., 1987, "Behavior of Reinforced Concrete Beam-Column Connections with Slabs," *Paper Prepared for the U.S.-N.Z.-Japan-China Seminar on the Design of R.C. Beam-Column Joints for*

Earthquake Resistance, University of Canterbury, Christchurch, New Zealand, Aug.

Kurose, Y., 1987, "Recent Studies on Reinforced Concrete Beam-Column Joints in Japan," *PMFSEL Report* No. 87-8, Phil M. Ferguson Structural Engineering Laboratory, University of Texas, Austin, TX, 164 pp.

Kurose, Y. et al., 1991, "Evaluation of Slab-Beam-Column Connections Subjected to Bidirectional Loading," *Design of Beam-Column Joints for Seismic Resistance,* SP-123, American Concrete Institute, Farmington Hills, MI, pp. 39-68.

LaFave, J. M., and Wight, J. K., 1997, "Behavior of Reinforced Concrete Exterior Wide Beam-Column-Slab Connections Subjected to Lateral Earthquake Loading," *Report* No. UMCEE 97-01, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, Jan., 217 pp.

Leon, R. T., 1984, "The Effect of Floor Member Size on the Behavior of Reinforced Concrete Beam-Column Joints," *Proceedings*, 8th World Conference on Earthquake Engineering, San Francisco, CA, July, pp. 445-452.

Leon, R. T., 1989, "Interior Joints with Variable Anchorage Length," *Journal of Structural Engineering*, ASCE, V. 115, No. 9, Sept., pp. 2261-2275.

Leon, R. T., 1990, "Shear Strength and Hysteretic Behavior of Beam-Column Joints, *ACI Structural Journal*, V. 87, No. 1, Jan.-Feb., pp. 3-11.

Leon, R. T., 1991, "Towards New Bond and Anchorage Provisions for Interior Joints," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 425-442.

Leon, R. T., and Deierlein, G. G., 1996, "Consideration for the Use of Quasi-Static Testing," *Earthquake Spectra*, V. 12, No. 1, Feb., pp. 87-110.

Leon, R. T., and Jirsa, J. O., 1986, "Bi-directional Loading of RC Beam-Column Joints," *Earthquake Spectra*, V. 2, No. 3, pp. 537-564.

Mazzoni, S.; Moehle, J. P.; and Thewalt, C. R., 1991, "Cyclic Response of RC Beam-Column Knee Joints: Test and Retrofit," *Report No. UCB/EERC-91/14*, Earthquake Engineering Research Center, University of California, Berkeley, CA, Oct., 24 pp.

McConnell, S. W., and Wallace, J. W., 1994, "Use of T-Headed Bars in Reinforced Concrete Knee-Joints Subjected to Cyclic Loads," *Report* No. CU/CEE-94/10, Department of Civil and Environmental Engineering, Clarkson University, Potsdam, NY, June, 44 pp.

McConnell, S. W., and Wallace, J. W., 1995, "Behavior of Reinforced Concrete Beam-Column Knee-Joints Subjected to Reversed Cyclic Loading," *Report* No. CU/CEE-95/07, Department of Civil and Environmental Engineering, Clarkson University, Potsdam, NY, June, 197 pp.

Meinheit, D. F., and Jirsa, J. O., 1977, "The Shear Strength of Reinforced Concrete Beam-Column Joints," *Report* No. 77-1, Department of Civil Engineering, Structures Research Laboratory, University of Texas at Austin, Jan.

Meinheit, D. F., and Jirsa, J. O., 1982, "Shear Strength of R/C Beam-Column Connections," *Proceedings*, ASCE, V. 107, ST11, Nov., pp. 2227-2244.

Ohno, K., and Shibata, T., 1970, "On the Damage to the Hakodate College by the Tokachioki Earthquake, 1968," *Proceedings*, U.S.-Japan Seminar of Earthquake Engineering with Emphasis on the Safety of School Buildings, Sendai, Sept., pp. 129-144.

Otani, S., 1991, "The Architectural Institute of Japan (AIJ) Proposal of Ultimate Strength Design Requirements for RC Buildings with Emphasis on Beam-Column Joints," *Design of Beam-Column Joints for Seismic Resistance,* SP-123, American Concrete Institute, Farmington Hills, MI, pp. 125-144.

Otani, S.; Kitayama, K.; and Aoyama, H., 1986, "Beam Bar Bond Requirements for Interior Beam-Column Connections," *Proceedings of the International Symposium on Fundamental Theory of Reinforced and Prestressed Concrete*, Nanjing Institute of Technology, China, Sept.

Pantazopoulou, S. J.; Moehle, J. P.; and Shahrooz, B. M., 1988, "Simple Analytical Model for T-Beam in Flexure," *Journal of Structural Engineering*, V. 114, No. 7, July, pp. 1507-1523.

Park, R.; Priestley, M. J. N.; and Gill, W. D., 1982, "Ductility of Square-Confined Concrete Columns," *Proceedings*, ASCE, V. 108, ST4, Apr., pp. 929-950.

Paulay, T., 1979, "Developments in the Design of Ductile Reinforced Concrete Frames," *Bulletin of the New Zealand National Society for Earthquake Engineering*, V. 12, No. 1, Mar., pp. 35-43.

Paulay, T., and Park, R., 1984, "Joints in Reinforced Concrete Frames Designed for Earthquake Resistance," *Research Report 84-9*, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, June.

Paulay, T.; Park, R.; and Priestley, M. J. N., 1978, "Reinforced Concrete Beam-Column Joints under Seismic Actions," ACI JOURNAL, *Proceedings* V. 75, No. 11, Nov., pp. 585-593.

Qi, X., 1986, "The Behavior of a R.C. Slab-Column Subassemblage under Lateral Load Reversals," *CE 299 Report*, Structural Engineering and Structural Mechanics, Department of Civil Engineering, University of California, Berkeley, CA.

Quintero-Febres, C. G., and Wight, J. K., 1997, "Investigation of the Seismic Behavior of RC Interior Wide Beam-Column Connections," *Report* No. UMCEE 97-15, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, Sept., 292 pp.

Rabbat, B. G.; Daniel, J. I.; Weinmann, T. L.; and Hanson, N. W., 1982, "Seismic Behavior of Lightweight Concrete Columns," PCA Construction Technology Laboratory/ National Science Foundation, Washington, DC, Sept. (Available as PB83-204 891 from NTIS.)

Raffaelle, G. S., and Wight, J. K., 1992, "R/C Eccentric Beam-Column Connections Subjected to Earthquake-Type Loading," *Report* No. UMCEE 92-18, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI, 234 pp.

Saqan, E. I., and Kreger, M. E., 1998, "Evaluation of U.S. Shear Strength Provisions for Design of Beam-Column Connections Constructed with High-Strength Concrete," *High Strength Concrete in Seismic Regions*, SP-176, C. W.

French and M. E. Kreger, eds., American Concrete Institute, Farmington Hills, MI, pp. 311-328.

Sattary-Javid, V., and Wight, J. K., 1986, "Earthquake Load on R/C Beams: Building Versus Single Beam," *Journal of Structural Engineering*, V. 112, No. 7, July, pp. 1443-1508.

Scott, B. D.; Park, R.; and Priestley, M. J. N., 1982, "Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates," ACI JOURNAL, *Proceedings* V. 79, No. 1, Jan.-Feb., pp. 13-27.

Scribner, C. F., and Wight, J. K., 1980, "Strength Decay in Reinforced Concrete Beams under Load Reversals," *Journal of the Structural Division*, ASCE, V. 106, No. ST4, Apr.

Seckin, M., and Uzumeri, S. M., 1978, "Examination of Design Criteria for Beam-Column Joints," European Conference on Earthquake Engineering, Dubrovnik, Sept.

Selna, L.; Martin, I.; Park, R.; and Wyllie, L., 1980, "Strong and Tough Concrete Columns for Seismic Forces," *Proceedings*, ASCE, V. 106, No. ST8, Aug., pp. 1717-1734.

Shahrooz, B. M., and Moehle, J. P., 1990, "Seismic Response and Design of Setback Buildings," *Journal of Structural Engineering*, V. 116, No. 5, May, pp. 1423-1439.

Sheikh, S. A., and Uzumeri, S. M., 1979, "Properties of Concrete Confined by Rectangular Ties," *AICAP-CEB Symposium on Structural Concrete under Seismic Actions*, Bulletin d'Information No. 132, Comite Euro-International Du Beton, Paris, Apr., pp. 53-60.

Sheikh, S. A., and Uzumeri, S. M., 1980, "Strength and Ductility of Tied Concrete Columns," *Proceedings*, ASCE, V. 106, ST5, May, pp. 1079-1102.

Sugano, S. et al., 1991, "Behavior of Beam-Column Joints Using High-Strength Materials," *Design of Beam-Column Joints for Seismic Resistance*, SP-123, American Concrete Institute, Farmington Hills, MI, pp. 359-378.

Suzuki, N.; Otani, S.; and Aoyama, H., 1983, "The Effective Width of Slabs in Reinforced Concrete Structures," *Transaction of the Japan Concrete Institute*, V. 5, pp. 309-316.

Suzuki, N.; Otani, S.; and Aoyama, H., 1984, "Three-Dimensional Beam-Column Subassemblages under Bidirectional Earthquake Loadings," *Proceedings*, 8th World Conference on Earthquake Engineering, V. 6, San Francisco, CA, July, pp. 453-460.

Wallace, J. W., 1997, "Headed Reinforcement: A Viable Option," *Concrete International*, V. 19, No. 12, Dec., pp. 47-53.

Wallace, J. W.; McConnell, S. W.; Gupta, P.; and Cote, P. A., 1998, "Use of Headed Reinforcement in Beam-Column Joints Subjected to Earthquake Loads," *ACI Structural Journal,* V. 95, No. 5, Sept.-Oct., pp. 590-606.

Watson, S., and Park, R., 1994, "Simulated Seismic Load Tests on Reinforced Concrete Columns," *Journal of Structural Engineering*, V. 120, No. 6, June, pp. 1825-1849.

Wight, J. K.; and Sozen, M. A., 1973, "Shear Strength Decay in Reinforced Concrete Columns Subjected to Large Deflection Reversals," *Report* No. SRS 403, Department of Civil Engineering, University of Illinois, Urbana-Champaign, Aug., 290 pp.

Wolfgram-French, C., and Boroojerdi, A., 1989, "Contribution of R/C Floor Slab in Resisting Lateral Loads," *Journal of Structural Engineering*, V. 115, No. 1, Jan., pp. 1-18.

Wright, J. L., and McCabe, S. L., 1997, "The Development Length and Anchorage Behavior of Headed Reinforcing Bars," *SM Report* No. 44, Structural Engineering and Engineering Materials, University of Kansas Center for Research, Lawrence, Kans., Sept., 147 pp.

Zerbe, H. E., and Durrani, A. J., 1989, "Seismic Response of Connections in Two-Bay R/C Frame Subassemblies," *Journal of Structural Engineering*, V. 115, No. 11, Nov., pp. 2829-2844.

Zhang, L., and Jirsa, J. O., 1982, "A Study of Shear Behavior of Reinforced Concrete Beam-Column Joints," *PMFSEL Report* No. 82-1, University of Texas at Austin, Feb.

Zhu, S., and Jirsa, J. O., 1983, "A Study of Bond Deterioration in Reinforced Concrete Beam-Column Joints," *PMFSEL Report* No. 83-1, Department of Civil Engineering, University of Texas at Austin, July.

APPENDIX A—AREAS NEEDING RESEARCH

The following list identifies areas needing further research. As a guide for interested researchers, listed below are some of the most recent references related to the individual topics. The ordering of the items listed is arbitrary.

A.1—Effect of eccentric beams on joints

Most connections tested to date had concentric beams; the axes of the column and beams were coincident. Connections in which beam axes are eccentric to the column axis are common, however, particularly in exterior building frames where beams connect to columns so that the outside faces of beams and columns are flush. Additional research is needed on the effects of eccentricity on the behavior in and adjacent to the joint particularly in torsion (Joh et al. 1991a; Raffaelle and Wight 1992).

A.2—Lightweight aggregate concrete in joints

Additional studies are needed to evaluate all aspects of joint behavior when various classes of lightweight aggregate concrete are used (Bertero et al. 1980; Rabbat et al. 1982).

A.3—Limit on joint shear

Some of the current limits on joint shear may be overly conservative for certain combinations of connection configuration, member size, and material strength. More experimental studies are needed to determine if these joint shear limits can be relaxed.

A.4—Behavior of indeterminate systems

Experimental results for beam-column joints have primarily been obtained from tests of statically determinate joint assemblies. The effect of force redistribution and joint deformation on the behavior of statically indeterminate structural systems needs to be determined (Leon and Deierlein 1996; Qi 1986; Seckin and Uzumeri 1978; Zerbe and Durrani 1989).

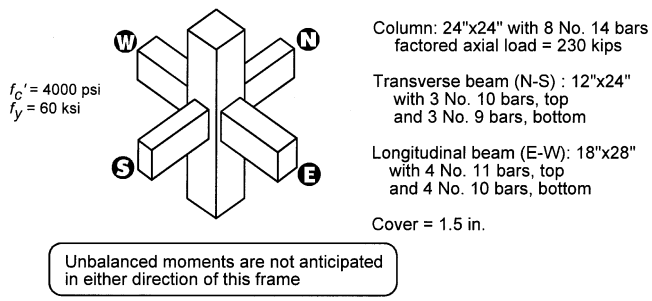


Fig. E1.1

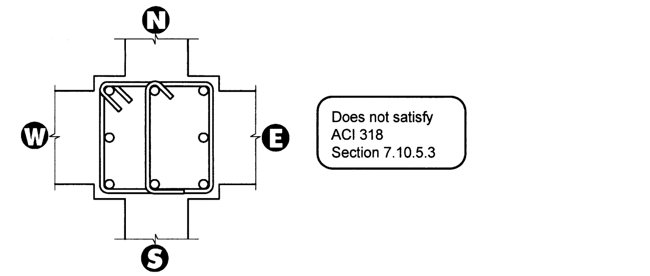


Fig. E1.2

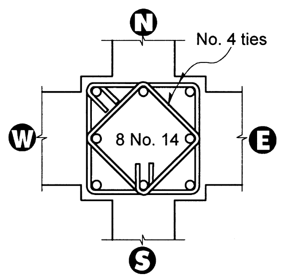


Fig. E1.3

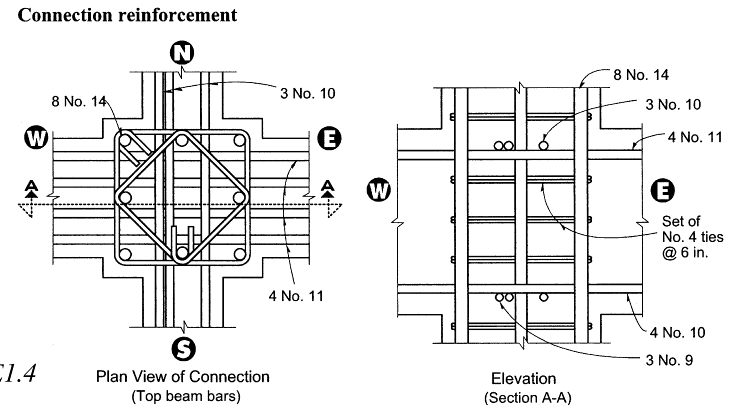


Fig. E1.4

A.5—Distribution of plastic hinges

Not all joints in a structure located in a high-seismicity area will experience significant inelastic deformations. Guidelines are needed to identify "Type 2" joints within a structure without having to do a comprehensive static or dynamic inelastic frame analysis.

A.6—Innovative joint designs

Studies have been performed on relocating beam plastic hinges away from the joint region (Abdel-Fattah and Wight 1987; Bertero and Popov 1977; Joh, Goto, and Shibata 1991b). Other innovative joint designs also need to be proposed and investigated, such as using fiber reinforcement in the joint region or post-tensioning the joint. Innovative joint designs that are able to reduce reinforcement congestion are particularly desirable, and along those lines, additional research on use of T-headed bars in joints is needed.

A.7—Special joint configurations and loadings

Certain categories of joints have not been thoroughly studied, examples are roof joints that have continuous beams (as opposed to knee-joints) and joints that are likely to be subjected to biaxial loading.

A.8—Joints in existing structures

Joints in structures built before the development of current design guidelines do not conform to the current requirements. These joints need to be studied in detail to establish their adequacy and to develop evaluation guidelines for building rehabilitation. Methods for improving performance of older joints need to be studied. Scarce information is available on connection repair and strengthening (Alcocer 1993; Alcocer and Jirsa 1993).

APPENDIX B—DESIGN EXAMPLES

Seven design examples are presented. The preliminary member sizes and reinforcement are given for each example, and the supporting calculations demonstrate the application of the committee's connection design recommendations. In all examples, it is assumed that the joints are part of the primary structural system for resisting lateral loads; that is, wind loads for Type 1 connections and earthquake loads for Type 2 connections. For Type 1 connections, examples are similar to those in the previous committee report.

DESIGN EXAMPLE 1—INTERIOR TYPE 1 CONNECTION (FIG. E1.1)

Transverse reinforcement (Section 4.2.1)

Only two opposite sides of the joint are effectively confined, that is, beams in the E-W direction cover three-quarters of the column width. Therefore, horizontal transverse reinforcement is required in the direction parallel to the axis of the N-S beam. According to Section 4.2.1.4(b), the amount of horizontal transverse reinforcement could be reduced in the direction perpendicular to the effectively confined joint faces. To satisfy Section 4.2.1.4(b), a No. 4 perimeter tie and a No. 4 cross tie parallel to the effectively confined sides are suggested (Fig. E1.2).

However, to comply fully with Section 7.10.5.3 of ACI 318-02, every corner and alternate longitudinal bar should be restrained, and no bar should be farther than 6 in. clear on each side along the tie from a laterally supported bar; thus, the proposed transverse reinforcement should be modified. A permissible arrangement of No. 4 ties is shown in Fig. E1.3.

Spacing between sets of ties should be less than or equal to 6 in. (Section 4.2.1.3)

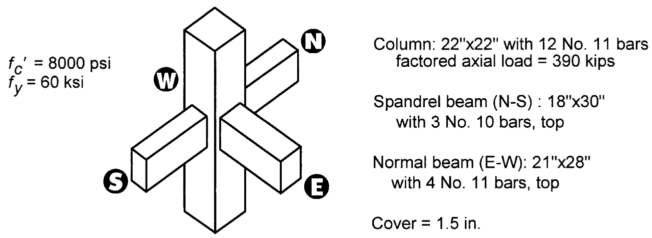


Fig. E2.1

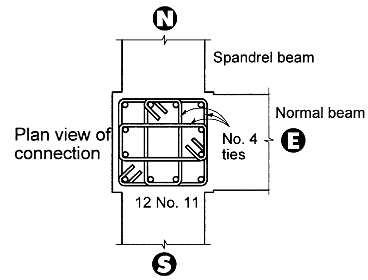


Fig. E2.2

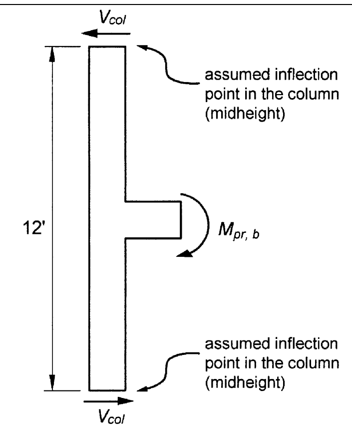


Fig. E2.3

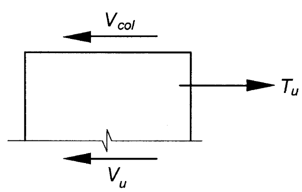


Fig. E2.4

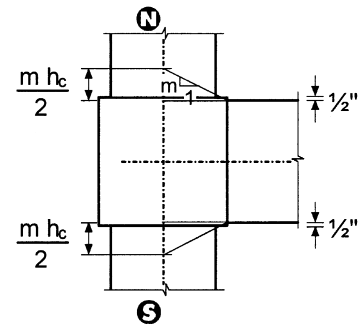
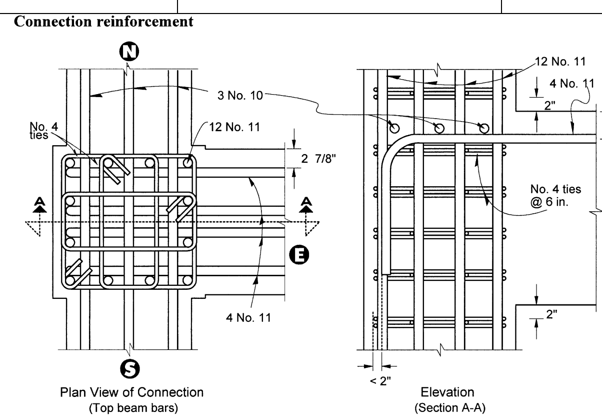


Fig. E2.5

Fig. E2.6



Shear

Shear is not a problem because large unbalanced moments are not anticipated in either direction.

Anchorage

Top beam bars should be continuous through the joint. The bottom bars should also be continuous through the joint because the joint is part of the primary system for resisting lateral loads (Fig E1.4).

DESIGN EXAMPLE 2—EXTERIOR TYPE 1 CONNECTION (FIG. E2.1)

Column longitudinal reinforcement (Section 4.1)

The indicated arrangement of twelve No. 11 bars is acceptable (Fig. E2.2).

Transverse reinforcement (Section 4.2.1)

A permissible arrangement of No. 4 ties is shown in Fig. E2.2 (ACI 318-02 Section 7.10.5.3). Spacing between sets of ties should be less than or equal to 6 in. (Section 4.2.1.3).

Joint shear force (Section 4.3.1)

Shear is not a problem in the north-south (spandrel) direction because large unbalanced moments are not anticipated in this direction. Positive bending at the face of the column in the E-W direction is not critical because gravity effects dominate. For shear in the normal direction (E-W), the maximum possible joint shear is a function of the flexural strength of the beam normal to the connection (Fig. E2.3).

$$d = 28 - 3.9 \text{ in.} = 24.1 \text{ in.}$$

$$M_{pr,\,b} = A_s \alpha f_y \left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s \alpha f_y}{0.85 f_c' b} = \frac{4(1.56 \text{ in.}^2)(1.0)(60 \text{ ksi})}{0.85(8 \text{ ksi})(21 \text{ in.})}$$

$$a = 2.62 \text{ in.}$$

$$M_{pr,\,b} = 4(1.56 \text{ in.}^2)(1.0)(60 \text{ ksi})\left(24.1 \text{ in.} - \frac{2.62 \text{ in.}}{2}\right)$$

$$M_{pr,b} = 8533 \text{ k-in.} = 711 \text{ k-ft}$$

$$V_{col} = M_{pr,b}/12 \text{ ft} = 59.3 \text{ kips}$$

Joint shear (Fig. E2.4)

$$T_u = A_s \alpha f_y = 374 \text{ kips}$$

$$V_u = T_u - V_{col} = 315 \text{ kips}$$

Joint shear strength (Section 4.3)

This is a Type 1 connection with a continuous column that meets the confinement restrictions of case A.2 in Table 1. Therefore, use $\gamma = 20$.

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} \\ b_b + \sum \dfrac{m \cdot h_c}{2} \\ b_c \end{cases}$$

According to Section 4.3.1 (Fig. E2.5):
$(m \cdot h_c)/2 \le$ extension of the column beyond the edge of the beam;

$$\frac{0.5(22 \text{ in.})}{2} = 5.5 \text{ in.};$$

extension of the column beyond the edge of the beam = 0.5 in., then $(m \cdot h_c)/2 = 0.5$ in.

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} = \dfrac{22 \text{ in.} + 21 \text{ in.}}{2} = 21.5 \text{ in. (governs)} \\ b_b + \sum \dfrac{m \cdot h_c}{2} = 21 \text{ in.} + 2(0.5 \text{ in.}) = 22 \text{ in.} \\ b_c = 22 \text{ in.} \end{cases}$$

$$V_n = 20\sqrt{f_c'}\, b_j h_c$$

$$V_n = 20\sqrt{8000 \text{ psi}}(21.5 \text{ in.})(22 \text{ in.})$$

$$V_n = 846{,}000 \text{ lb} = 846 \text{ kips}$$

$$\phi V_n = 0.85(846 \text{ kips}) = 719 \text{ kips} > 315 \text{ kips (OK)}$$

Hooked bar anchorage (Fig. E2.6) (Section 4.5.2)

$$\ell_{dh} = \frac{f_y d_b}{50\sqrt{f_c'}} = \frac{(60{,}000 \text{ psi})(1.41 \text{ in})}{50\sqrt{8000 \text{ psi}}} = 18.9 \text{ in.}$$

Table B.1—Minimum column depth for Type 2 connections*

Bar size, No. (1)	d_b, in. (mm) (2)	ℓ_{dh}, in. (mm) (3)	h (min) for column: For column hoops at a spacing > $3d_b$, in. (mm) (4)	h (min) for column: For column hoops at a spacing ≤ $3d_b$, in. (mm) (5)
6	0.750 (19.0)	11.9 (300)	15.4 (390)	13.0 (330)
7	0.875 (22.2)	13.8 (350)	17.3 (440)	14.6 (370)
8	1.000 (25.4)	15.8 (401)	19.3 (491)	16.1 (411)
9	1.128 (28.6)	17.8 (451)	21.3 (541)	17.8 (451)
10	1.270 (32.3)	20.1 (502)	23.6 (592)	19.6 (491)
11	1.410 (35.8)	22.3 (551)	25.8 (641)	21.3 (530)

*Based on anchorage of terminating beam longitudinal reinforcement.

The reduction factor of Section 4.5.2.3(a) applies, so

$$\text{mod } \ell_{dh} = (18.9 \text{ in.})(0.7) = 13.2 \text{ in.}$$

Available space = 22 in. – 1.5 in. (back cover) – 0.5 in. (tie diameter) = 20 in. (OK)

Hook is located within 2 in. from the back of the confined core (Section 4.5.2.1).

MEMBER DEPTH CONSIDERATIONS FOR TYPE 2 CONNECTIONS

Before starting the examples for Type 2 connections, it is important to point out that in the course of design, column sizes adequate for member strength requirements may have to be increased to satisfy anchorage and shear requirements within the joint. Wider beam sections may be necessary to cover column faces and allow the use of higher joint shear stress values.

Table B.1 is based on anchorage requirements for hooked bars terminating in a joint (Section 4.5.2). Table B.2 is based on requirements for the ratio of joint dimensions (actually beam and column dimensions) to the diameter of beam and columns bars (Section 4.5.5). These tables should be useful for selecting main reinforcing bar diameters and joint dimensions. Values for ℓ_{dh} were calculated from Eq. (4.10) using $\alpha = 1.25$, $f_y = 60$ ksi (414 MPa), and $f_c' = 4000$ psi (28 MPa). In Table B.1, an extra 3-1/2 in. (90 mm) has been added to ℓ_{dh} to determine the minimum column dimension to anchor a given bar. The quantity 3-1/2 in. (90 mm) comes from twice the clear cover (typical 1-1/2 in. [38 mm] front and back) plus one tie-bar diameter. The 0.8 multiplier for close spacing of transverse reinforcement of 4.5.2.4a is included in Column 5 of Table B.1.

DESIGN EXAMPLE 3—INTERIOR TYPE 2 CONNECTION (FIG. E3.1)

Column longitudinal reinforcement (Section 4.1)

Change the number of longitudinal bars to give a more uniform distribution of longitudinal steel. The arrangement of column longitudinal bars of 12 No. 9 bars shown is acceptable (Fig. E3.2). Column reinforcement is well

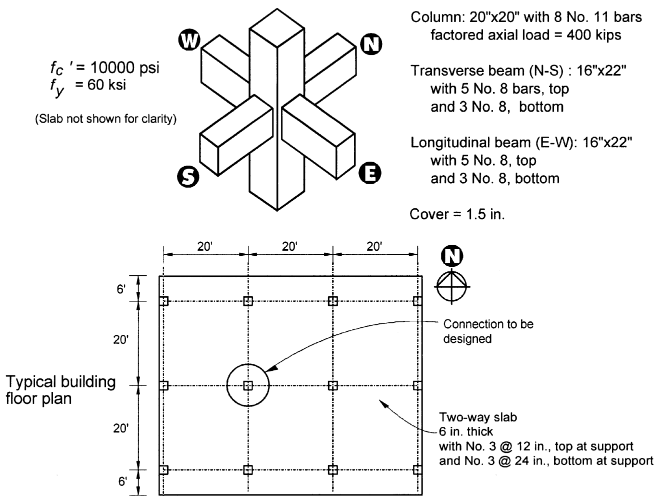


Fig. E3.1

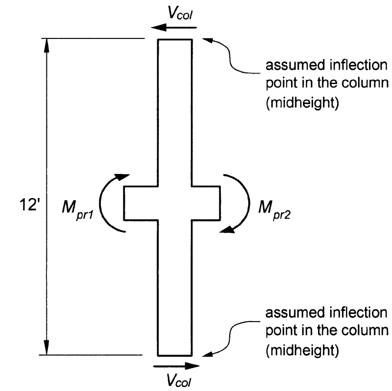


Fig. E3.4

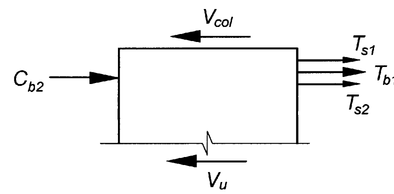


Fig. E3.5

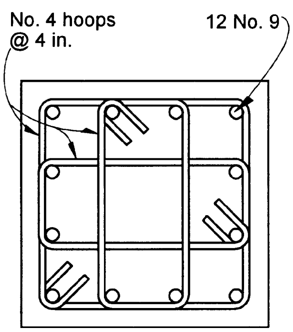


Fig. E3.2

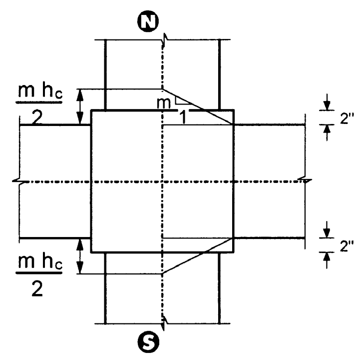


Fig. E3.6

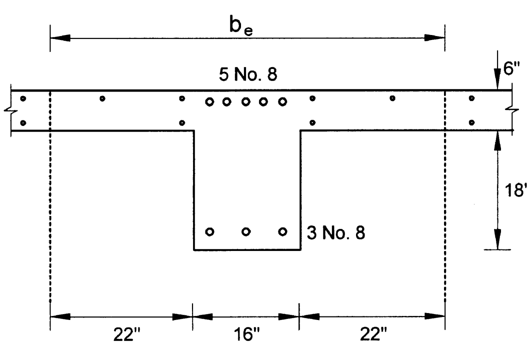


Fig. E3.3

Connection reinforcement

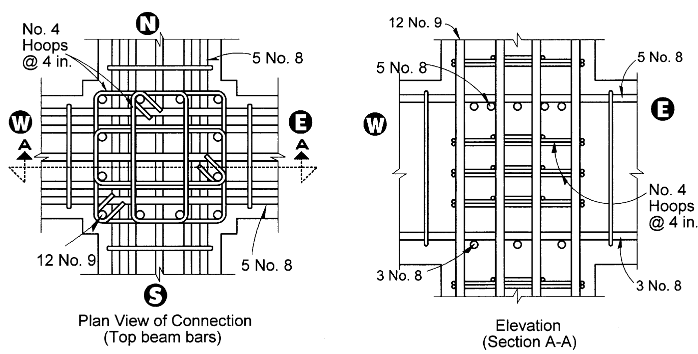


Fig. E3.7

distributed around the perimeter and the maximum spacing between supported bars satisfies Section 4.1.

From Table B.2, the minimum beam depth is 22.6 in. for a No. 9 column longitudinal bar; beams are 22 in. deep. To comply with this requirement, 24 in. deep beams will be considered.

Transverse reinforcement (Section 4.2.2)

Provide A_{sh} = 4 legs (0.20 in.^{2}/leg) = 0.80 in.^{2} (in each direction).

Because beam dimensions satisfy Section 4.2.2.5, the value for A_{sh} obtained from Eq. (4.4) and (4.5) is reduced by 50% in the joint.

From Section 4.2.2.3

$$s_h \le \begin{cases} b_c/4 \;=\; 5 \text{ in. (governs)} \\ 6d_b \;=\; 6(1.128 \text{ in.}) \;=\; 6.8 \text{ in.} \\ 6 \text{ in.} \end{cases}$$

From Eq. (4.4)

$$A_{sh} = 0.3\frac{s_h b_c'' f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) =$$

$$0.3\frac{(5\text{ in.})(17\text{ in.})(10\text{ ksi})}{60\text{ ksi}}\left(\frac{20^2}{17^2} - 1\right) = 1.63\text{ in.}^2$$

From Eq. (4.5)

$$A_{sh} = 0.09\frac{s_h b_c'' f_c'}{f_{yh}} =$$

$$0.09\frac{(5\text{ in.})(17\text{ in.})(10\text{ ksi})}{60\text{ ksi}} = 1.28\text{ in.}^2$$

Required A_{sh} = 0.5 (1.63 in.^{2}) = 0.82 in.^{2} > 0.80 in.^{2} (inadequate).

A 4 in. spacing will be used for the No. 4 hoop reinforcement

$$A_{sh} = 0.5(1.63)(4/5) = 0.65\text{ in.}^2 < 0.80\text{ in.}^2\text{ (OK)}$$

For the hoops, it was decided to stay with No. 4 bars at reduced spacing rather than increasing to No. 5 bars, because tests have demonstrated that for the same amount of reinforcement, the use of a smaller-diameter bar enhances member strength and ductility.

Joint shear (Section 4.3)

For calculating the beam flexural strength (Section 3.3.2), it is necessary to assess the slab participation under negative bending (top fibers in tension). Beam flexural strength under positive and negative bending is determined according to ACI 318-02 requirements.

For negative bending moment:

According to Section 8.10.2 of ACI 318-02, the slab effective as a T-beam flange should not exceed:

a) one-quarter of the span length of the beam = 20 ft/4 = 5 ft = 60 in. (governs);

b) web width + eight times the slab thickness on each side = 16 in. + 8 (6 in.) × 2 = 112 in.;

c) web width + one-half the clear distance to the next web on each side = 16 + 0.5 (20 ft × 12 in./ft – 16 in.) + 0.5 (20 ft × 12 in./ft – 16 in.) = 240 in. (Fig. E3.3)

$$b_e = 60\text{ in.} > 2b_b = 32\text{ in.}$$

Within the effective flange width (60 in.), a total of six No. 3 slab bars (top and bottom) should be considered for bending analysis. Both top and bottom slab bars are assumed to be continuous through the connection.

For positive bending moment:

Similarly to negative bending moment, b_e = 60 in.

For the bending analysis that follows, ignore the effect of compression reinforcement and assume, in most locations, $d = h - 2.7$ in. In locations where there is interference between bars from the normal and spandrel beams, assume $d = h - 3.7$ in. for the spandrel beam.

Longitudinal beam (E-W) (Fig. E3.4)

$$M_{pr,b} = A_s \alpha f_y\left(d - \frac{a}{2}\right)$$

$$a = \frac{A_s \alpha f_y}{0.85 f_c' b}$$

For positive bending moment

$$a = \frac{3(0.79\text{ in.}^2)(1.25)(60\text{ ksi})}{0.85(10\text{ ksi})(60\text{ in.})} = 0.35\text{ in.}$$

$$M_{pr1} = (2.37\text{ in.}^2)(1.25)(60\text{ ksi})\left(24 - 3.7 - \frac{0.35}{2}\right)$$

$$= 3577\text{ k-in.} = 298\text{ k-ft}$$

For negative bending moment

$$a = \frac{(3.95 + 0.66\text{ in.}^2)(1.25)(60\text{ ksi})}{0.85(10\text{ ksi})(16\text{ in.})} = 2.54\text{ in.}$$

$$M_{pr2} = (4.61\text{ in.}^2)(1.25)(60\text{ ksi})\left(24 - 2.7 - \frac{2.54}{2}\right)$$

$$= 6925\text{ k-in.} = 577\text{ k-ft}$$

(For thick or heavily reinforced slabs the actual effective depth should be calculated.)

Therefore, from Fig. E3.5, the column shear is:

$$V_{col} = \frac{M_{pr1} + M_{pr2}}{12\text{ ft}} = \frac{298 + 577\text{ k ft}}{12\text{ ft}} = 73\text{ kips}$$

$$V_u = T_{b1} + T_{s1} + T_{s2} + C_{b2} - V_{col}$$
$$= \alpha f_y (A_{s1} + A_{s,s1} + A_{s,s2} + A_{s2}) - V_{col}$$

Table B.2—Minimum column or beam depth for Type 2 connections*

Bar size, No. (1)	d_b, in. (mm) (2)	h (min) for column based on size of beam longitudinal reinforcement or h (min) for beam based on size of column longitudinal reinforcement, in. (mm) (3)
6	0.750 (19.0)	15.0 (380)
7	0.875 (22.2)	17.5 (444)
8	1.000 (25.4)	20.0 (508)
9	1.128 (28.6)	22.6 (572)
10	1.270 (32.3)	25.4 (636)
11	1.410 (35.8)	28.2 (698)
14	1.693 (44.5)	33.9 (890)

*Based on size of longitudinal reinforcement.

$= 1.25\ (60\text{ ksi})\ (3.95 + 0.44 + 0.22 + 2.37\text{ in.}^2) - 73 =$ 451 kips

For joint shear strength

$$V_n = \gamma\sqrt{f_c'}\,b_j h_c$$

Because beams are wide enough, the joint can be classified as case A.1 in Table 1 and Fig. 4.5 "joints effectively confined on all four vertical faces." So $\gamma = 20$ (Fig. E3.6)

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} \\ b_b + \sum \dfrac{m \cdot h_c}{2} \\ b_c \end{cases}$$

According to Section 4.3.1:
$(m \cdot h_c)/2 \le$ extension of the column beyond the edge of the beam

$$\frac{0.5(20\text{ in.})}{2} = 5\text{ in.};$$

extension of the column beyond the edge of the beam = 2 in., then $(m \cdot h_c)/2 = 2$ in.

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} = \dfrac{20\text{ in.} + 16\text{ in.}}{2} = 18\text{ in. (governs)} \\ b_b + \sum \dfrac{m \cdot h_c}{2} = 16\text{ in.} + 2(2\text{ in.}) = 20\text{ in.} \\ \cancel{b_c = 20\text{ in.}} \end{cases}$$

$$\phi V_n = 0.85(20)\sqrt{10{,}000\text{ psi}}(18\text{ in.})(20\text{ in.})\frac{1\text{ kip}}{1000\text{ lb}}$$

$$= 612\text{ kips} > 451\text{ kips (OK)}$$

Flexural strength ratio (Section 4.4.5)

When determining the column flexural strength, the factored axial load that results in the lowest column flexural strength was assumed in this example to be 400 kips (this will normally depend on actual load combinations). Also, α was set equal to 1.0 for this calculation. Using these assumptions, $M_{n,c} = 700$ k-ft.

The beam flexural strengths have been found earlier using $\alpha = 1.25$. Those beam strengths will be divided by 1.25 to obtain an approximate value for the beam flexural strength if $\alpha = 1.0$. If the strength ratio is close to the allowable value, a more accurate determination of the beam flexural strength for $\alpha = 1.0$ could be made.

$$M_{n1} \cong 298\text{ k-ft}/1.25 = 238\text{ k-ft}$$

$$M_{n2} \cong 577\text{ k-ft}/1.25 = 462\text{ k-ft}$$

Flexural strength ratio =

$$\frac{\Sigma M_{n,c}}{\Sigma M_{n,b}} = \frac{2(700)}{238 + 462} = 2.0 > 1.2\text{ (OK)}$$

Beam and column bars passing through the joints (Section 4.5.5) (Fig. E3.7)

The column dimension is governed by the largest beam bar (Eq. (4.11))

$$h_c > 20(60{,}000/60{,}000)(1.00\text{ in.}) = 20.0\text{ in.} = 20\text{ in. (OK)}$$

Beam depths are controlled by the column bars:

$$h_b > 20(60{,}000/60{,}000)(1.128\text{ in.}) = 22.6\text{ in.} < 24\text{ in. (OK)}$$

DESIGN EXAMPLE 4—CORNER TYPE 2 CONNECTION (FIG. E4.1)

Column longitudinal reinforcement (Section 4.1)

The arrangement of 14 No. 9 bars shown in Fig. E4.2 is acceptable. Column reinforcement is well distributed around the perimeter. Maximum spacing between supported bars satisfies Section 4.1.

From Table B.2, the minimum depth is 22.6 in. for a No. 9 column longitudinal bar; beams are 28 in. deep.

Transverse reinforcement (Section 4.2.2)

N-S direction:

Provided A_{sh} = 4 legs (0.31 in.^{2}/leg) = 1.24 in.^{2}; assume 4 in. spacing.

From Eq. (4.4)

$$A_{sh} = 0.3\frac{(4\text{ in.})(25\text{ in.})(8\text{ ksi})}{60\text{ ksi}}\left(\frac{24(28)}{21(25)} - 1\right) = 1.12\text{ in.}^2$$

From Eq. (4.5)

$$A_{sh} = 0.09\frac{(4\text{ in.})(25\text{ in.})(8\text{ ksi})}{60\text{ ksi}} = 1.2\text{ in.}^2\text{(governs)}$$

Section 4.2.2.5 states that full A_{sh} is to be provided for Type 2 joints, unless beams provide effective confinement to all four column faces.

Required A_{sh} = 1.2 in.^{2} < Provided A_{sh} = 1.24 in.^{2} (OK)

For the E-W direction

Provided A_{sh} = 1.02 in.^{2}

From Eq. (4.4)

$$A_{sh} = 0.3\frac{(4\text{ in.})(21\text{ in.})(8\text{ ksi})}{60\text{ ksi}}\left(\frac{24(28)}{21(25)} - 1\right) = 0.94\text{ in.}^2$$

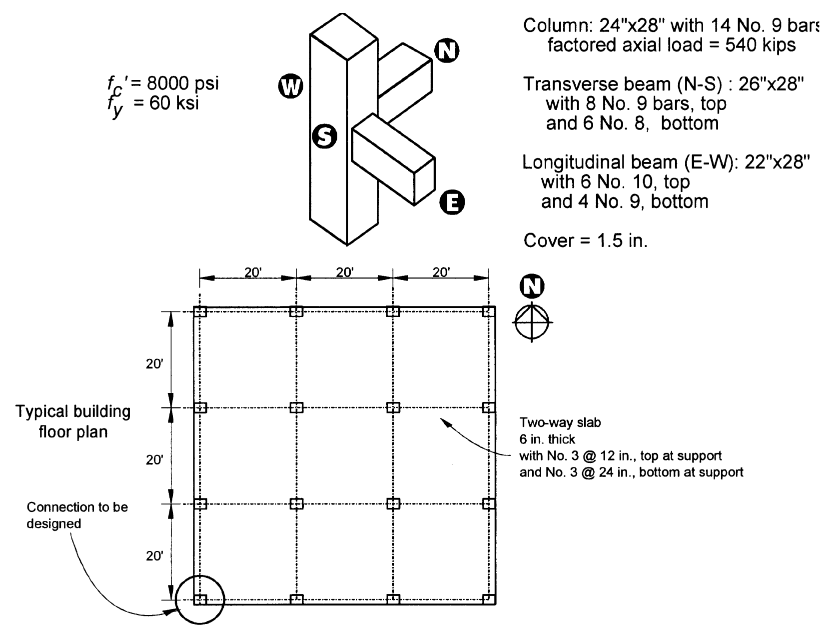


Fig. E4.1

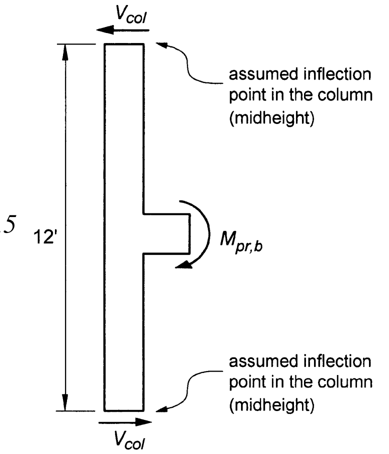
Fig. E4.5


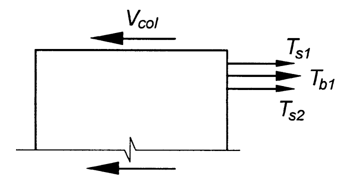
Fig. E4.6


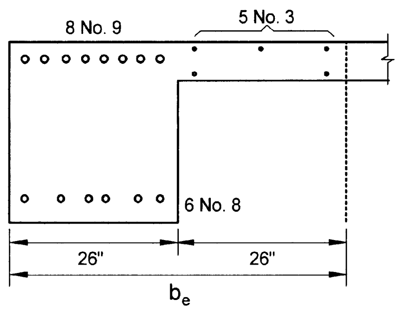
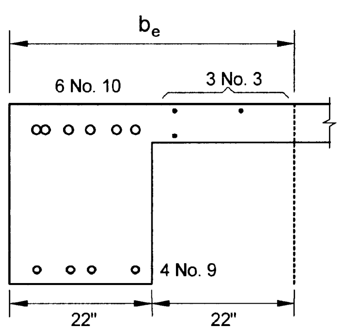
Fig. E4.3


Fig. E4.4


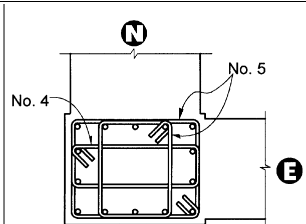
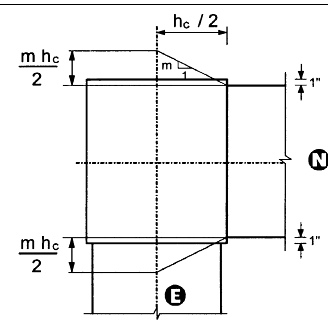


Fig. E4.2

Fig. E4.7


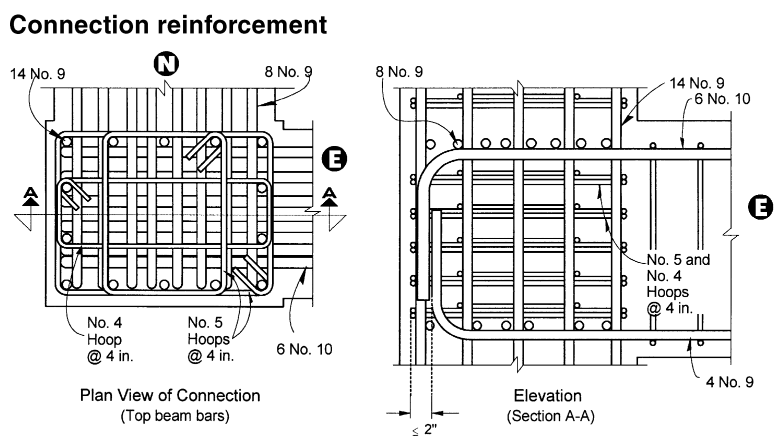


Fig. E4.8

From Eq. (4.5)

$$A_{sh} = 0.09\frac{(4\text{ in.})(21\text{ in.})(8\text{ ksi})}{60\text{ ksi}} = 1.0\text{ in.}^2(\text{governs})$$

Use sets of No. 5 perimeter hoop with No. 5 and No. 4 interior hoops spaced at 4 in. (Provided $A_{sh} = 1.24$ in.^{2} in the N-S direction and 1.02 in.^{2} in the E-W direction).

Joint shear (Section 4.3)

For calculating the beam flexural strength (Section 3.3.2), it is necessary to assess the slab participation under negative bending moment. Only negative flexure is considered because strength is larger than that for positive moment.

According to Section 8.10.3 of ACI 318-02, effective flange width should not exceed:

N-S Beam (Fig. E4.3):

a) one-twelfth the span length of the beam + web width = 20 ft × 12 in./ft /12 + 26 in. = 46 in.

b) web width + six times the slab thickness = 26 in. + 6(6 in.) = 62 in.

c) web width + one-half the clear distance to the next web = 26 + 0.5 (20 ft × 12 in./ft – 26) = 133 in.

According to Section 3.3.2, however, $b_e \geq 2b_b$. In this case, $2b_b$ = 52 in. controls for negative flexure.

Within $b_{e,N\text{-}S}$, five No. 3 slab bars must be considered for bending analysis.

E-W beam similarly (Fig. E4.4):

a) 42 in.

b) 58 in.

c) 131 in.

42 in. < $2b_b$ = 44 in., thus $b_{e,E\text{-}W}$ = 44 in. controls for negative flexure.

Within $b_{e,E\text{-}W}$, three No. 3 slab bars must be considered for bending analysis.

Both top and bottom slab bars are assumed to be anchored with standard hooks at the transverse beams.

For the bending analysis ignore the effect of compression reinforcement and assume $d = h - 2.7$ in. For the N-S beam, and $d = h - 3.7$ in. for the E-W beam (Fig. E4.5).

N-S beam

$$M_{pr,N-S} = [8(1.0\text{ in.}^2) + 5(0.11\text{ in.}^2)](1.25)(60\text{ ksi})$$

$$\left(28 - 2.7 - \frac{3.63}{2}\right) = 15{,}060\text{ kip-in.} = 1255\text{ k-ft}$$

E-W beam

$$M_{pr,E-W} = [6(1.27\text{ in.}^2) + 3(0.11\text{ in.}^2)](1.25)(60\text{ ksi})$$

$$\left(28 - 3.7 - \frac{3.99}{2}\right) = 13{,}300\text{ kip-in.} = 1108\text{ k-ft}$$

Column shear (Fig. E4.6)

N-S direction

$$V_{col,N-S} = \frac{M_{pr,N-S}}{12\text{ ft}} = \frac{1255}{12} = 105\text{ kips}$$

E-W direction

$$V_{col,E-W} = \frac{M_{pr,E-W}}{12\text{ ft}} = \frac{1108}{12} = 92\text{ kips}$$

$$V_u = T_{b1} + T_{s1} + T_{s2} - V_{col}$$

N-S direction

$$V_u = (8.55\text{ in.}^2)(1.25)(60\text{ ksi}) - 105\text{ kips} = 536\text{ kips}$$

E-W direction

$$V_u = (7.95\text{ in.}^2)(1.25)(60\text{ ksi}) - 92\text{ kips} = 504\text{ kips}$$

Joint shear strength

$$V_n = \gamma\sqrt{f_c'}b_j h_c$$

From Table 1 and Fig. 4.5, γ = 12:

N-S direction

$$b_j \leq \begin{cases} \dfrac{b_c + b_b}{2} \\ b_b + \sum \dfrac{m \cdot h_c}{2} \\ b_c \end{cases}$$

According to Section 4.3.1 (Fig. E4.7):

$(m \cdot h_c)/2 \leq$ extension of the column beyond the edge of the beam = 1 in.,

$$\frac{0.5(24\text{ in.})}{2} = 6\text{ in.}; \text{ then } (m \cdot h_c)/2 = 1\text{ in.}$$

$$b_j \leq \begin{cases} \dfrac{b_c + b_b}{2} = \dfrac{28\text{ in.} + 26\text{ in.}}{2} = 27\text{ in.(governs)} \\ b_b + \sum \dfrac{m \cdot h_c}{2} = 26\text{ in.} + 2(1\text{ in.}) = 28\text{ in.} \\ b_c = 28\text{ in.} \end{cases}$$

$$\phi V_n = 0.85(12)\sqrt{8000\text{ psi}}(27\text{ in.})(24\text{ in.})$$

$$= 591\text{ kips} > 536\text{ kips (OK)}$$

E-W direction

$$\frac{0.5(28\text{ in.})}{2} = 7\text{ in.}; \text{ then}$$

$$\frac{m \cdot h_c}{2} = 1\text{ in.}$$

$$b_j \leq \begin{cases} \dfrac{24\text{ in.} + 22\text{ in.}}{2} = 23\text{ in. (governs)} \\ 22\text{ in.} + 2(1\text{ in.}) = 24\text{ in.} \\ 24\text{ in.} \end{cases}$$

$$\phi V_n = 0.85(12)\sqrt{8000\text{ psi}}(23\text{ in.})(28\text{ in.})$$

$$= 588\text{ kips} > 504\text{ kips (OK)}$$

Flexural strength ratio (Section 4.4.2)

When determining the column flexural strength, the factored axial load that results in the lowest column flexural strength was assumed in this example to be 540 kips. Also, α was set equal to 1.0 for this calculation. Using these assumptions:

N-S direction, $M_{nc,N\text{-}S} = 1086$ k-ft

E-W direction, $M_{nc,E\text{-}W} = 1262$ k-ft

Beam flexural strengths for $\alpha = 1.0$ are approximated as was done in Example 3.

$M_{n,N\text{-}S} \cong 1255/1.25 = 1004$ k-ft

$M_{n,E\text{-}W} \cong 1108/1.25 = 887$ k-ft

Thus,

N-S direction

$$\text{Flexural strength ratio} = \frac{2(1086)}{1004} = 2.2 > 1.2 \text{ (OK)}$$

E-W direction

$$\text{Flexural strength ratio} = \frac{2(1262)}{887} = 2.8 > 1.2 \text{ (OK)}$$

Hooked bars terminating in the connection (Section 4.5.2)

N-S direction: the No. 9 bars need to be checked (Fig. E4.8):

$$\ell_{dh} = \frac{\alpha f_y d_b}{75\sqrt{f_c'}} = \frac{1.25(60{,}000 \text{ psi})(1.128 \text{ in.})}{75\sqrt{8000 \text{ psi}}} = 13 \text{ in.}$$

$\ell_{dh} = 13$ in. is less than the provided depth of the column core minus one hoop diameter:

$$13 \text{ in.} < 24 - 2(1.5) - 0.625 = 20.375 \text{ in. (OK)}$$

E-W direction: analogously for the No. 10 bars:

$\ell_{dh} = 14$ in. which is less than $28 - 2(1.5) - 0.625 = 24.375$ in. (OK)

Hooks should be located within 2 in. from the back of the confined core.

Column bars passing through joint (Section 4.5.5)

The total beam depths are governed by the column bar (Eq. (4.11)).

$$20\,(60{,}000/60{,}000)(1.128 \text{ in.}) = 22.6 \text{ in.} < h_b = 28 \text{ in. (OK)}$$

DESIGN EXAMPLE 5—EXTERIOR TYPE 2 CONNECTION WITH A DISCONTINUOUS COLUMN AND WITHOUT TRANSVERSE BEAM (FIG. E5.1)

Welded headed bars are used for column and beam longitudinal reinforcement.

Anticipated changes

Change beam dimensions to 22 x 32 in. The beam width is increased to help satisfy confinement and shear requirements.

Column longitudinal reinforcement (Section 4.1)

An acceptable arrangement of column longitudinal reinforcement is shown in Fig. E5.2. Longitudinal reinforcement is uniformly distributed around the perimeter to enhance concrete confinement.

Horizontal transverse reinforcement (Section 4.2.2)

The joint concrete should be adequately confined with hoops calculated with Eq. (4.4) and (4.5).

Provided:

$$A_{sh} = 5 \text{ legs } (0.20 \text{ in.}^2/\text{leg}) = 1.0 \text{ in.}^2 \text{ (each direction)}$$

From Eq. (4.4)

$$A_{sh} = 0.3\frac{(4 \text{ in.})(27 \text{ in.})(6 \text{ ksi})}{60 \text{ ksi}}\left(\frac{30^2}{27^2} - 1\right) = 0.76 \text{ in.}^2$$

From Eq. (4.5)

$$A_{sh} = 0.09\frac{(4 \text{ in.})(27 \text{ in.})(6 \text{ ksi})}{60 \text{ ksi}} = 0.97 \text{ in.}^2$$

Required $A_{sh} = 0.97$ in.^{2} < Provided $A_{sh} = 1.0$ in.^{2} (OK)

Thus, use No. 4 hoops and crosstie spaced at 4 in.

Joint shear (Section 4.3)

For the bending analysis, ignore the effect of compression reinforcement and assume $d = h - 2.7$ in. Negative bending strength will be evaluated because it is larger than positive bending strength, and thus controls the demand on the joint.

For negative bending moment, slab reinforcement within a width $2c_t + b_c$ shall be considered to contribute to the flexural strength of the beam (Section 3.3.2b):

$$2c_t + b_c = 60 \text{ in.} + 30 \text{ in.} = 90 \text{ in.}$$

but, $b_e \leq$ one-twelfth the span length of the beam + $b_b = 42$ in. (governs) (Fig. E5.3).

Thus, $b_e = 42$ in.

Both top and bottom slab bars are assumed to be anchored with standard hooks at the exterior edge of the slab (Fig. E5.4).

$$A_s = 6(0.79) + 2(0.2) + 2(0.11) = 5.36 \text{ in.}^2$$

$$V_u = T_{b1} + T_{s1} + T_{s2} = \alpha f_y(A_{s,b} + A_{s,s1} + A_{s,s2}) = 1.25\,(60 \text{ ksi})(5.36) = 402 \text{ kips}$$

According to Table 1 and Fig. 4.5, $\gamma = 8$. Thus, the joint shear strength is

$$\phi V_n = \phi\gamma\sqrt{f_c'}b_j h_c$$

According to Section 4.3.1:

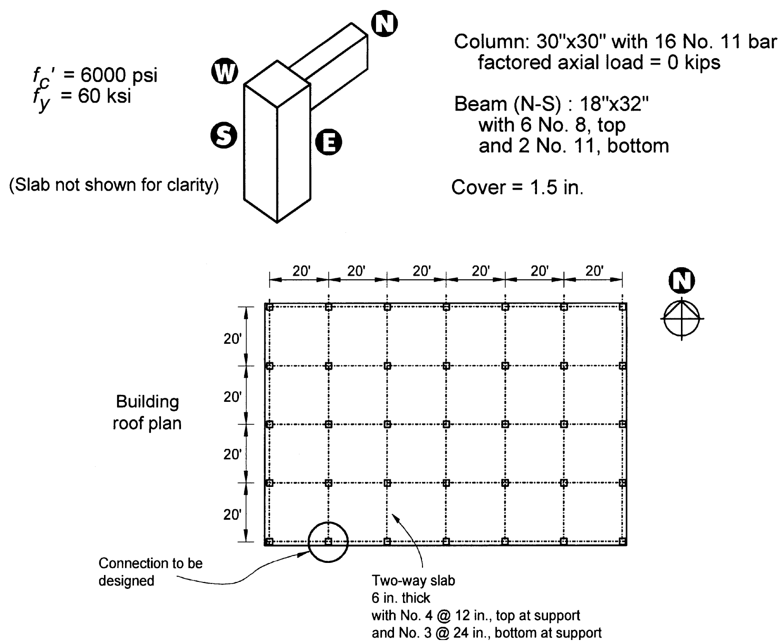


Fig. E5.1

Fig. E5.3

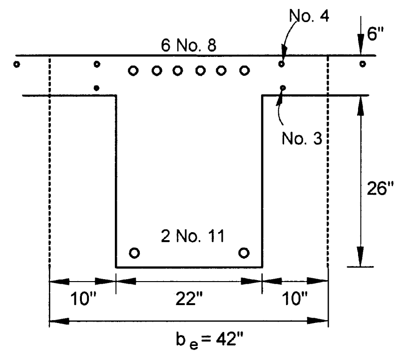


Fig. E5.4

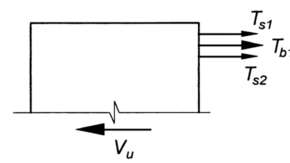


Fig. E5.2

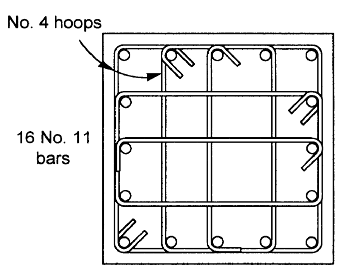


Fig. E5.5

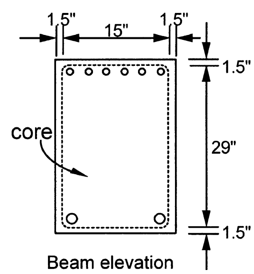
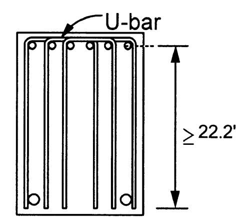


Fig. E5.6



$(m \cdot h_c)/2 \leq$ extension of the column beyond the edge of the beam = 4 in.,

$$\frac{0.5(30\text{ in.})}{2} = 7.5\text{ in.}\text{ ; then }(m \cdot h_c)/2 = 4\text{ in.}$$

$$b_j \leq \begin{cases} \dfrac{b_c + b_b}{2} = \dfrac{30\text{ in.} + 22\text{ in.}}{2} = 26\text{ in.(governs)} \\ b_b + \sum \dfrac{m \cdot h_c}{2} = 22\text{ in.} + 2(4\text{ in.}) = 30\text{ in.} \\ b_c = 30\text{ in.} \end{cases}$$

Therefore,

$$\phi V_n = 0.85(8)\sqrt{6000\text{ psi}}(26\text{ in.})(30\text{ in.})\frac{1\text{ kip}}{1000\text{ lb}}$$

$$= 411\text{ kips} > 402\text{ kips (OK)}$$

Flexural strength ratio (Section 4.4.2)

Per Section 4.4.2, the flexural strength ratio need not be checked.

Headed bars terminating in the joint (Section 4.5.3)

Checking anchorage length for largest bar diameter (No. 11).

$$\ell_{dt} = \begin{cases} 8d_b = 11.3\text{ in.} \\ 6\text{ in.} \\ \dfrac{3}{4}\dfrac{\alpha f_y d_b}{75\sqrt{f_c'}} = 13.7\text{ (governs)} \end{cases}$$

Provided anchorage length for beam bars is

h_c – front cover – back cover – horizontal hoop diameter = 30 in. – 1.5 in. – 1.5 in. – 0.5 in. = 26.5 in. >> 13.7 in. (OK)

Vertical transverse reinforcement (Sections 4.2.2.8 and 4.5.3.3)

For connections with a discontinuous column above the floor, vertical transverse reinforcement should be provided in the joint region (Fig. E5.5).

According to Sections 4.2.2.2 and 4.2.2.3, inverted U-bars are needed to confine the unrestrained joint face (Fig. E5.6).

Provided

$$A_{sh,U\text{-}bar} = 2 \text{ legs } (0.44 \text{ in.}^2/\text{leg}) = 0.88 \text{ in.}^2$$

From Eq. (4.4)

$$A_{sh} = 0.3\frac{(4.5 \text{ in.})(15 \text{ in.})(6 \text{ ksi})}{60 \text{ ksi}}\left(\frac{18(32)}{15(29)} - 1\right) = 0.66 \text{ in.}^2$$

From Eq. (4.5)

$$A_{sh} = 0.09\frac{(4.5 \text{ in.})(15 \text{ in.})(6 \text{ ksi})}{60 \text{ ksi}} = 0.61 \text{ in.}^2$$

Required $A_{sh} = 0.66$ in.^{2} < Provided $A_{sh,Ubar} = 0.88$ in.^{2} (OK)

Thus, use No. 6 U-bar spaced at 4.5 in.

According to Section 4.5.3.3, a restraining force equal to 1/2 of the yield strength of the bar being developed should be provided. The amount of U-bars is determined assuming that they have reached their specified yield stress.

For six No. 8 beam bars: $A_{s,U\text{-}bars} \geq 0.5(6)(0.79) = 2.37$ in.^{2}

With three No. 6 bars from U-shaped ties: $A_{s,U\text{-}bars} = 2(3)(0.44) = 2.64$ in.^{2} > 2.37 (OK).

Anchorage length shall be sufficient to develop the tie yield strength. Thus, according to ACI 318-02 Section 12.2

$$\ell_d \geq 12 \text{ in., or}$$

$$\frac{\ell_d}{d_b} = \frac{f_y \alpha \beta \lambda}{25\sqrt{f_c'}}$$

For this case:

$\alpha = 1.0$ because it is a vertical bar;

$\beta = 1.0$ because it is uncoated reinforcement;

$\lambda = 1.0$ because it is normalweight concrete; so that

$$\ell_d = \frac{60{,}000(0.75 \text{ in.})}{25\sqrt{6000}} = 23.2 \text{ in.}$$

Because the depth of the beam is 32 in. > 23.2 in., inverted U-bars are adequate if extended along the joint height.

According to Section 4.5.3.3, headed bars for beam and column longitudinal reinforcement must be restrained with layers of transverse reinforcement perpendicular to the unconfined face and anchored within the joint. Horizontal transverse reinforcement, calculated according to Section 4.2, also serves this purpose.

DESIGN EXAMPLE 6—INTERIOR TYPE 2 WIDE-BEAM CONNECTION (FIG. E6.1)

Because $b_b = 50$ in. < $3b_c = 66$ in., and $b_b = 50$ in. < $(b_c + 1.5h_c) = 55$ in. (Section 2.2), these recommendations are applicable.

Column longitudinal reinforcement (Section 4.1)

An increase in the number of longitudinal bars is required to give a more uniform distribution of longitudinal steel. An acceptable arrangement of column longitudinal bars is shown in Fig. E6.2.

From Section 4.5.5, the minimum beam depth is 15 in. for a No. 6 column longitudinal bar.

Transverse reinforcement (Section 4.2.2)

Provided $A_{sh} = 4$ legs (0.11 in.^{2}/leg) = 0.44 in.^{2} (in each direction).

Because beam dimensions satisfy Section 4.2.2.5, the value for A_{sh} obtained from Eq. (4.4) and (4.5) may be reduced by 50% in the joint.

From Eq. (4.4)

$$A_{sh} = 0.3\frac{(5 \text{ in.})(19 \text{ in.})(4 \text{ ksi})}{60 \text{ ksi}}\left(\frac{22^2}{19^2} - 1\right) = 0.65 \text{ in.}^2$$

From Eq. (4.5)

$$A_{sh} = 0.09\frac{(5 \text{ in.})(19 \text{ in.})(4 \text{ ksi})}{60 \text{ ksi}} = 0.57 \text{ in.}^2$$

Required $A_{sh} = 0.5(0.65$ in.$^2) = 0.33$ in.^{2} < 0.44 in.^{2} (OK)

Joint shear (Section 4.3)

The E-W direction is critical because the column is square and symmetrically reinforced, and because E-W beams have larger longitudinal reinforcement amounts than N-S beams. Similar to Example 3, beam flexural strengths are calculated considering the slab participation (Fig. E6.3).

For negative bending moment, the effective slab width is:

a) one-quarter of the span length of the beam = 18 ft/4 = 4.5 ft = 54 in. (governs);

b) web width + eight times the slab thickness on each side = 50 in. + 8 (4 in.) × 2 = 114 in.; and

c) web width + one-half the clear distance to the next web on each side = 50 in. + 0.5(18 ft × 12 in./ft – 50 in.) × 2 = 216 in.

According to Section 3.3.2, however, $b_e \geq 2b_b$. In this case, $2b_b = 100$ in. controls for negative flexure.

Within the effective flange width of 100 in., six No. 4 bars shall be considered for the bending analysis. It is assumed that slab bars are continuous through the connection.

For positive bending moment, the effective beam flange width (according to ACI 318 Section 8.10.2) is: $b_e = 54$ in. using the same assumptions for the flexural analysis as were made in previous examples (Fig E6.4 and E6.5).

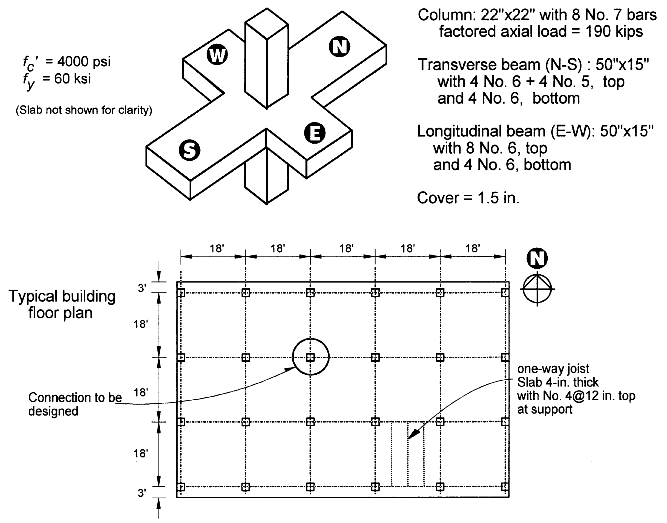


Fig. E6.1

Fig. E6.2

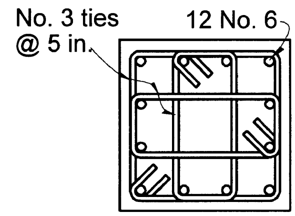
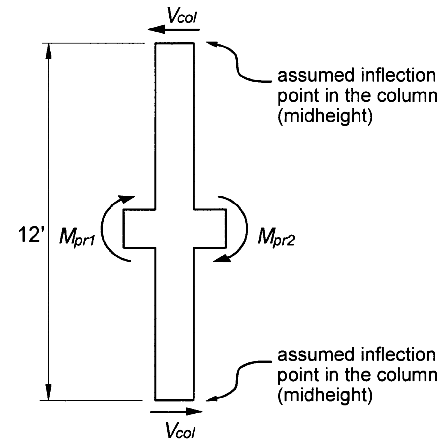


Fig. E6.4



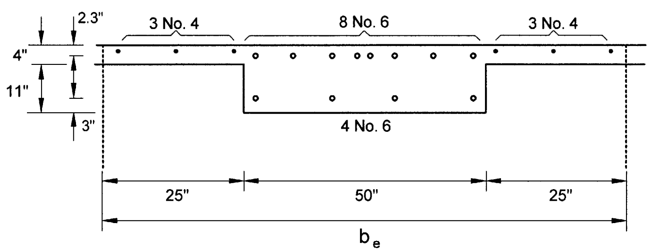


Fig. E6.3

Fig. E6.5

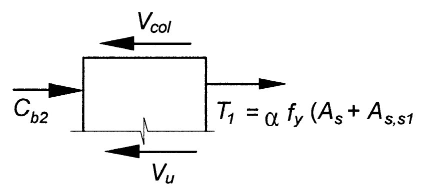
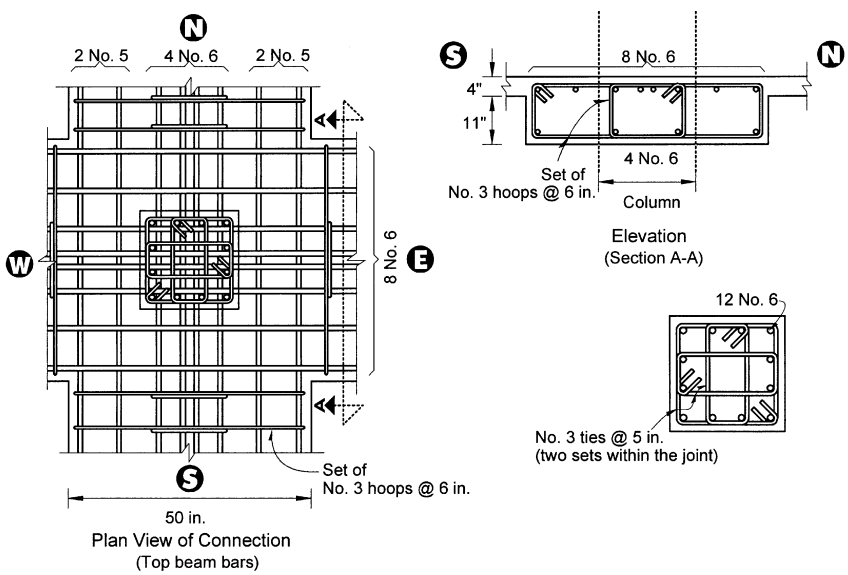


Fig. E6.6



$$M_{pr1} = (1.76\ \text{in.}^2)(1.25)(60\ \text{ksi})\left(12 - \frac{0.72}{2}\right)$$

$$= 1536\ \text{kip-in.} = 128\ \text{k-ft}$$

$$M_{pr2} = (4.72\ \text{in.}^2)(1.25)(60\ \text{ksi})\left(12.7 - \frac{2.08}{2}\right)$$

$$= 4127\ \text{kip-in.} = 344\ \text{k-ft}$$

$$V_{col} = \frac{M_{pr1} + M_{pr2}}{12\ \text{ft}} = \frac{128 + 344}{12} = 39.3\ \text{kips}$$

$$T_1 = (1.25)(60)[8(0.44\ \text{in.}^2) + 6(0.20\ \text{in.}^2)] = 354\ \text{kips}$$

$$C_{b2} = (1.25)(60)(4)(0.44\ \text{in.}^2) = 132\ \text{kips}$$

$$V_u = T_1 + C_{b2} - V_{col} = 354 + 132 - 39.3 = 447\ \text{kips}$$

Joint sides are effectively confined, thus $\gamma = 20$ (Table 1 and Fig. 4.5).

$$b_j = b_c = 22 \text{ in.}$$

$$\phi V_n = 0.85(20)\sqrt{4000 \text{ psi}}(22 \text{ in.})(22 \text{ in.})\frac{1 \text{ kip}}{1000\ell bs}$$

$$= 520 \text{ kips} > 447 \text{ kips (OK)}$$

Flexural strength ratio (Section 4.4.2)

The factored axial load that results in the lowest column flexural strength was assumed in this example to be 190 kips. Also, α was set equal to 1.0 for this calculation. Using these assumptions, $M_{n,c} = 362$ k-ft.

Only the longitudinal (E-W) beams need to be considered because they are stronger than the transverse (N-S) beams.

$$M_{n1} \cong 128 \text{ k-ft}/1.25 = 102 \text{ k-ft}$$

$$M_{n2} \cong 344 \text{ k-ft}/1.25 = 275 \text{ k-ft}$$

Flexural strength ratio =

$$\frac{\Sigma M_{n,c}}{\Sigma M_{n,b}} = \frac{2(362)}{102 + 275} = 1.9 > 1.2 \text{ (OK)}$$

Shear reinforcement in wide beam plastic hinge region (Section 4.6.2)

An estimate of the maximum shear force at the column face is

$$V_b = \frac{M_{pr2}}{0.5(18 - 22/12)\text{ft}} = \frac{344 \text{ k-ft}}{8.08 \text{ ft}} = 42.6 \text{ kips}$$

and

$$V_{max} = 2\sqrt{f_c'}b_b d =$$

$$2\sqrt{4000 \text{ psi}}(50 \text{ in.})(12.7 \text{ in.})\frac{1 \text{ kip}}{1000 \text{ lb}} = 80.3 \text{ kips}$$

Because $V_{max} > V_b$, the maximum spacing of shear reinforcement shall be the lesser of:

a) $d/2 = 12.7$ in./2 = 6.3 in.;

b) $8d_{b,beam} = 8(0.750$ in.) = 6 in. (governs); and

c) $24d_{b,stirrup} = 24(0.375$ in.) = 9 in.

Within the plastic hinge region ($2h_b = 2 \times 15$ in. = 30 in.), No. 3 stirrups with four legs spaced at 6 in. shall be used.

Beam and column bars through the joint (Section 4.5.5) (Fig. E6.6)

The column dimension is governed by the large beam bar (Eq. (4.11))

$$h_c > 24\ (60{,}000/60{,}000)(0.750 \text{ in.}) = 18 \text{ in.} < 22 \text{ in. (OK)}$$

Beam depths are controlled by the column bars

$$h_b = 15 \text{ in.} = 20(60{,}000/60{,}000)(0.75 \text{ in.}) \text{ (OK)}$$

Connection reinforcement

About 40% of the negative flexural reinforcement in the wide beam and slab are anchored in the column core, thus satisfying Section 3.3.3.

DESIGN EXAMPLE 7—EXTERIOR TYPE 2 WIDE-BEAM CONNECTION (FIG. E7.1)

Because $b_b = 50$ in. $< 3b_c = 60$ in., and $b_b < (b_c + 1.5h_c) =$ 50 in., these recommendations are applicable (Section 2.2).

Column longitudinal reinforcement (Section 4.1)

The increase in the number of longitudinal bars is required to give a more uniform distribution of longitudinal steel. An acceptable arrangement of the column longitudinal reinforcement is shown in Fig. E7.2.

From Section 4.5.5, the minimum beam depth is 15 in. for a No. 6 column longitudinal bar; wide-beam is 15 in. deep.

Transverse reinforcement (Section 4.2.2)

Provided $A_{sh} = 2\ (0.20 + 0.11) = 0.62$ in.^{2} (in each direction)

From Eq. (4.4)

$$A_{sh} = 0.3\frac{(4 \text{ in.})(17 \text{ in.})(4\text{ksi})}{60 \text{ ksi}}\left(\frac{20^2}{17^2} - 1\right) = 0.52 \text{ in.}^2$$

(governs)

From Eq. (4.5)

$$A_{sh} = 0.09\frac{(4 \text{ in.})(17 \text{ in.})(4 \text{ ksi})}{60 \text{ ksi}} = 0.41 \text{ in.}^2$$

Required $A_{sh} = 0.52$ in.^{2} < Provided $A_{sh} = 0.62$ in.^{2} (OK)

Hoop spacing equal to 4 in. satisfies Section 4.2.2.3.

Design of spandrel beam for torsion (Section 3.3.3)

According to Section 3.3.3, a spandrel beam should be designed for full equilibrium torsion from the beam and slab bars anchored in the spandrel beam within the effective flange width b_e (Fig. E7.3). Wide-beam and slab bars on each side of the column are assumed to yield.

The effective flange width of the wide-beam (E-W direction) should be obtained. The effective slab width shall not exceed (Section 8.10.2 of ACI 318):

a) one-quarter of the span length of the beam = 18 ft/4 = 4.5 ft = 54 in. (governs);

b) web width + eight times the slab thickness on each side = 50 in. + 8(6 in.) × 2 = 146 in.; and

c) web width + one-half the clear distance to the next web on each side = 50 in. + 0.5(18 ft × 12 in./ft – 50 in.) × 2 = 216 in.

But $b_e = 54$ in. $< 2b_b = 100$ in.

Therefore, $b_e = 100$ in.

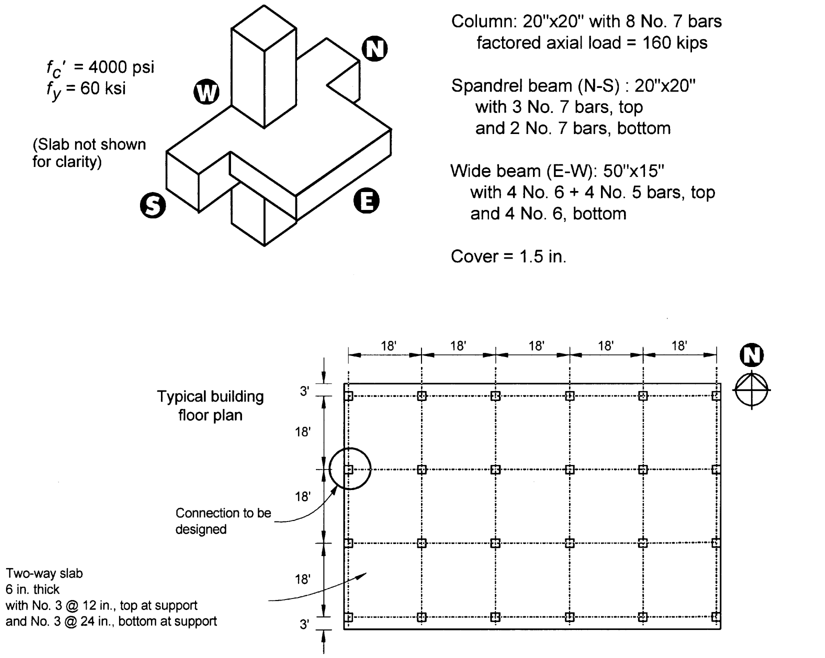


Fig. E7.1

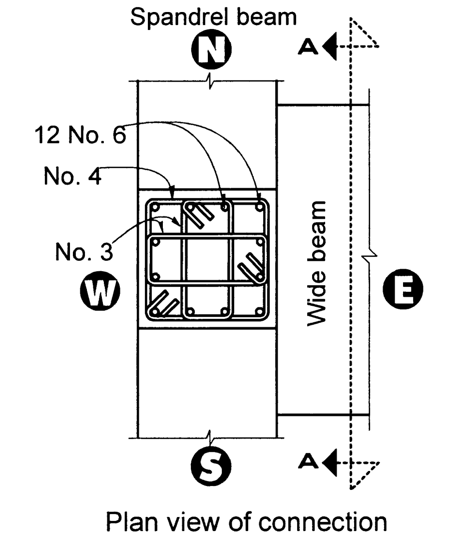


Fig. E7.2

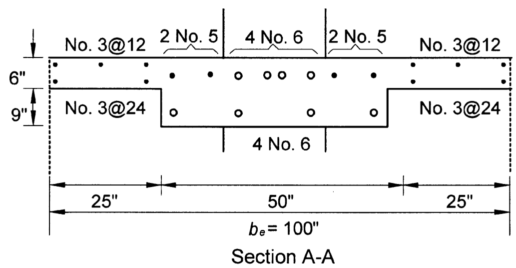


Fig. E7.3

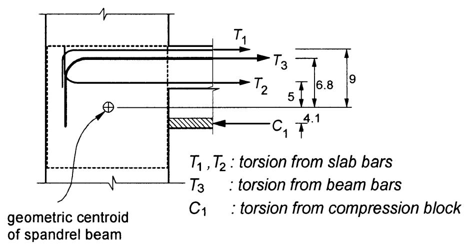


Fig. E7.4

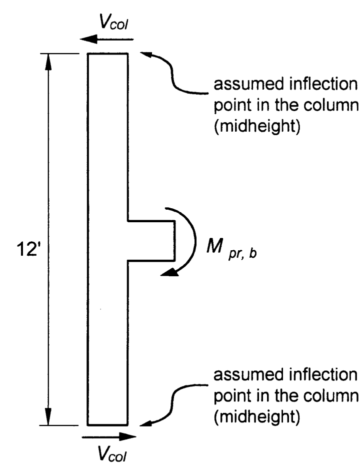


Fig. E7.5

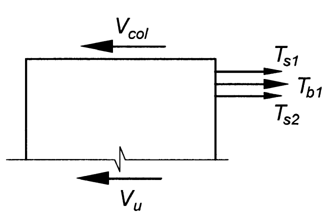


Fig. E7.6

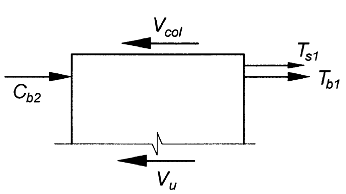


Fig. E7.7

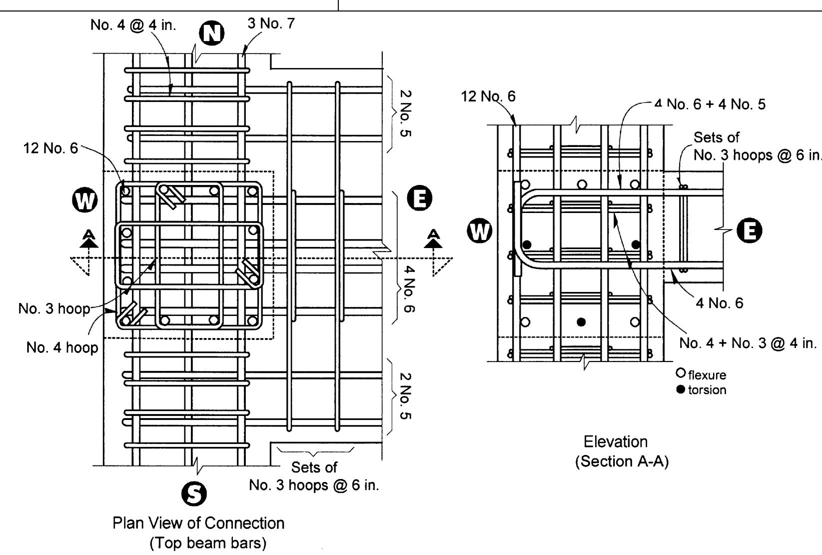


Fig. E7.8

Thus, from Fig. E7.4, torsion is calculated as the sum of torsional moments produced by slab and wide-beam bars with respect to the centroid of the spandrel beam.

From the flexural strength of wide-beam

$$a = \frac{A_s \alpha f_y}{0.85 f_c' b} = \frac{4.1\ \text{in.}^2 (1.25)(60\ \text{ksi})}{0.85(4\ \text{ksi})(50\ \text{in.})} = 1.81\ \text{in.}$$

T_i = (No. bars) $A_s f_y y_i$:
T_1 = 3(0.11 in.^{2})(60 ksi)(9 in.)= 178 kips-in.
T_2 = 2(0.11 in.^{2})(60 ksi)(5 in.) = 66 kips-in.
T_3 = 2(0.31 in.^{2})(60 ksi)(6.8 in.) = 253 kips-in.
C_1 = 0.85(4 ksi)(15 × 1.81 in.^{2})(4.1 in.) = 378 kips-in.

$T = T_1 + T_2 + T_3 + C_1 = 875$ kips-in.
$T_u = \alpha T = 1.25(875) = 1094$ kips-in.

From ACI 318-02 Section 11.6.3, the spacing of No. 4 hoops is 4 in., and an additional three No. 7 longitudinal beam bars are necessary to resist torsion. Additional bars are distributed on the perimeter of spandrel beam.

From Section 3.3.3 of these recommendations, the spacing of the transverse reinforcement shall not exceed:

a) $p_h/16 = 66$ in./16 = 4.125 in.; and

b) 6 in.

So, use No. 4 closed hoops spaced at 4 in.

Joint shear (Section 4.3)

Bending analysis is performed using the same assumptions of Example 3.

Both top and bottom slab bars are assumed to be anchored with standard hooks at the transverse beams.

Wide-beam (E-W direction) (Fig. E7.5 and E7.6)

Within the effective flange width of 100 in., four No. 6, four No. 5, and ten No. 3 bars should be considered for bending analysis.

$$M_{pr,E\text{-}W} = 4.1\ \text{in.}^2(1.25)(60\ \text{ksi})\left(15 - 3.2 - \frac{1.81}{2}\right)$$
$$= 3350\ \text{kip-in.} = 279\ \text{k-ft}$$

$$V_{col} = \frac{M_{pr,E-W}}{12\ \text{ft}} = \frac{279}{12} = 23.3\ \text{kips}$$

$$V_u = T_{b1} + T_{s1} + T_{s2} - V_{col} = (4.1\ \text{in.}^2)(1.25)(60\ \text{ksi}) - 23.3$$
$$= 284.2\ \text{kips}$$

From Table 1 and Fig. 4.5, $\gamma = 15$, $b_j = b_c = 20$ in.

$$\phi V_n = 0.85(15)\sqrt{4000\ \text{psi}}\,(20\ \text{in.})^2 = 323\ \text{kips} > 284\ \text{kips (OK)}$$

Spandrel beam (N-S direction) (Fig. E7.7)

The effective slab width should not exceed (Section 8.10.3 of ACI 318):

a) web width + one-twelfth the span length of the beam = 20 in. + 18 ft(12 in./ft)/12 = 38 in.;

b) web width + six times the slab thickness = 20 in. + 36 in. = 56 in.; and

c) web width + one-half the clear distance to the next web = 20 in. + 0.5(196) = 118 in., and should be $b_e \geq 2b_b = 2$ (20 in.) = 40 in. (governs).

Within the effective flange width of 40 in., three No. 3 slab bars should be considered for the bending analysis.

$$M_{pr1,N\text{-}S} = 1.8\ \text{in.}^2(1.25)(60\ \text{ksi})\left(17.7 - \frac{1.04}{2}\right) =$$
$$2319\ \text{kip-in.} = 193\ \text{k-ft}$$

$$M_{pr2,N\text{-}S} = 2.13\ \text{in.}^2(1.25)(60\ \text{ksi})\left(17.7 - \frac{2.35}{2}\right) =$$
$$2640\ \text{kip-in.} = 220\ \text{k-ft}$$

$$V_{col} = \frac{(M_{pr,N-S})^{(-)} + (M_{pr,N-S})^{(+)}}{12\ \text{ft}} = \frac{220 + 193}{12}$$
$$= 34.4\ \text{kips}$$

$$V_u = (2.13 + 1.8\ \text{in.}^2)(1.25)(60\ \text{ksi}) - 34 = 260\ \text{kips} < \phi V_n$$
$$= 323\ \text{kips (same as E-W check) (OK).}$$

Flexural strength ratio (Section 4.4.2)

The factored axial load that results in the lowest column flexural strength was assumed in this example to be 160 kips. Also, α was set equal to 1.0 for this calculation. Using these assumptions, $M_{n,c} = 299$ k-ft.

$$M_{n,E\text{-}W} \cong 279/1.25 = 223\ \text{k-ft}$$

$$(M_{n,N\text{-}S})^{(-)} \cong 220/1.25 = 176\ \text{k-ft}$$

$$(M_{n,N\text{-}S})^{(+)} \cong 193/1.25 = 154\ \text{k-ft}$$

Wide-beam direction (E-W)

$$\text{Flexural strength ratio} = \frac{2(299)}{223} = 2.7 > 1.2\ \text{(OK)}$$

Spandrel beam direction (N-S)

$$\text{Flexural strength ratio}\ \frac{2(299)}{176 + 154} = 1.8 > 1.2\ \text{(OK)}$$

Shear reinforcement in wide-beam plastic hinge region (Section 4.6.2)

An estimate of the maximum shear force at the column face is

$$V = \frac{M_{pr,E-W}}{0.5(18 - 20/12)\ \text{ft}} = \frac{279}{8.17} = 34.2\ \text{kips}$$

and V_{max} =

$$2\sqrt{f_c'}\,b_d d = 2\sqrt{4000\text{ psi}}(50\text{ in.})(11.8\text{ in.})\left(\frac{1\text{ kip}}{1000\text{ lb}}\right)$$

$$= 74.6\text{ kips}$$

Because $V_{max} > V_b$, the maximum spacing of shear reinforcement should be the lesser of:

a) $d/2$ = 11.8 in./2 = 5.9 ≅ 6 in. (governs);

b) $8d_{b,beam}$ = 8(0.750 in.) = 6 in.; and

c) $24d_{b,stirrup}$ = 24(0.375 in.) = 9 in.

Within the plastic hinge region (30 in.), use No. 3 stirrups with four legs spaced at 6 in.

Beam and column bars through the joint (Section 4.5.5) (Fig. E7.8)

The column dimension (in the N-S direction) is governed by the spandrel longitudinal bar (Eq. (4.11))

$$h_c = 20\text{ in.} > 20(60{,}000/60{,}000)(0.875\text{ in.}) = 17.5\text{ in. (OK)}$$

Beam depths are controlled by the column bars (E-W direction)

$$h_b = 15\text{ in.} \geq 20(60{,}000/60{,}000)(0.75\text{ in.}) = 15\text{ in. (OK)}$$

For N-S direction

$$h_b = 20\text{ in.} > 20(60{,}000/60{,}000)(0.75\text{ in.}) = 15\text{ in. (OK)}$$

Hooked bars anchored in the joint and in the spandrel beam should satisfy Section 4.5.3. Checking for largest beam bar, No. 6.

$$\ell_{dh} = \frac{\alpha f_y d_b}{75\sqrt{f_c'}} = \frac{(1.25)(60{,}000\text{ psi})(0.75\text{ in.})}{75\sqrt{4000\text{ psi}}} = 11.9\text{ in.}$$

ℓ_{dh} = 12 in. is larger than 6 in. and $8d_b$ = 6 in.

The space available in the column from the critical section is 20 in. – back cover (1.5 in.) – column hoop (0.5 in.) – front cover (1.5) = 16.5 in. > 12 in. (OK)

The hooks are located within 2 in. from the back of the confined core (Section 4.5.2.1).

DESIGN EXAMPLE 3 IN SI UNITS (INTERIOR TYPE 2 CONNECTION) (FIG. E8.1)

Column longitudinal reinforcement (Section 4.1)

Change the number of longitudinal bars to give a more uniform distribution of longitudinal steel (Fig. E8.2). The arrangement of column longitudinal bars of 12 No. 9 bars shown below is acceptable. Column reinforcement is well distributed around the perimeter and the maximum spacing between supported bars satisfies Section 4.1.

From Table B.2, the minimum beam depth is 572 mm for a No. 9 column longitudinal bar; beams are 550 mm deep. To comply with this requirement, 600 mm deep beams will be considered.

Transverse reinforcement (Section 4.2.2)

Provided A_{sh} = 4 legs (127 mm²/leg) = 508 mm² (in each direction).

Because beam dimensions satisfy Section 4.2.2.5, the value for A_{sh} obtained from Eq. (4.4) and (4.5) is reduced by 50% in the joint.

From Section 4.2.2.3

$$s_h \leq \begin{cases} b_c/4 = 125\text{ mm (governs)} \\ 6d_b = 6(28.6\text{ mm}) = 172\text{ mm} \\ 150\text{ mm} \end{cases}$$

From Eq. (4.4)

$$A_{sh} = 0.3\frac{s_h b_c'' f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) =$$

$$0.3\frac{(125\text{ mm})(424\text{ mm})(70\text{ MPa})}{414\text{ MPa}}\left(\frac{500^2}{424^2} - 1\right) = 1050\text{ mm}^2$$

From Eq. (4.5)

$$A_{sh} = 0.09\frac{s_h b_c'' f_c'}{f_{yh}} = 0.09\frac{(125\text{ mm})(424\text{ mm})(70\text{ MPa})}{414\text{ MPa}}$$

$$= 807\text{ mm}^2$$

Required A_{sh} = 0.5(1050 mm²) = 525 in.² > 508 mm² (inadequate).

A 100-mm spacing will be used for the No. 4 hoop reinforcement

$$A_{sh} = 0.5(1050)(100/125) = 420\text{ mm}^2 < 508\text{ in.}^2\text{ (OK)}$$

For hoops, No. 4 bars are used instead of No. 5 bars, because tests have demonstrated that for the same amount of reinforcement, use of a smaller-diameter bar enhances member strength and ductility.

Joint shear (Section 4.3)

For calculating the beam flexural strength (Section 3.3.2), it is necessary to assess the slab participation under negative bending (top fibers in tension). Beam flexural strength under positive and negative bending is determined according to ACI 318 requirements.

For negative bending moment

According to Section 8.10.2 of ACI 318-02, the slab effective as a T-beam flange should not exceed (see Fig. E8.3):

a) one-quarter of the span length of the beam = 6.1/4 = 1.52 m (governs);

b) web width + eight times the slab thickness on each side = 0.4 + 8 (0.15) × 2 = 2.8 m; and

c) web width + one-half the clear distance to the next web on each side = 0.4 + 0.5 (6.1 – 0.4) + 0.5 (6.1 – 0.4) = 6.1 m.

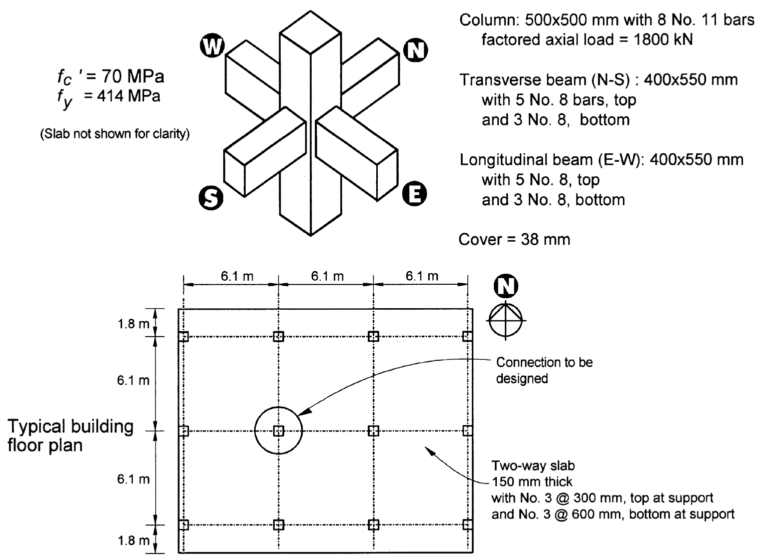


Fig. E8.1

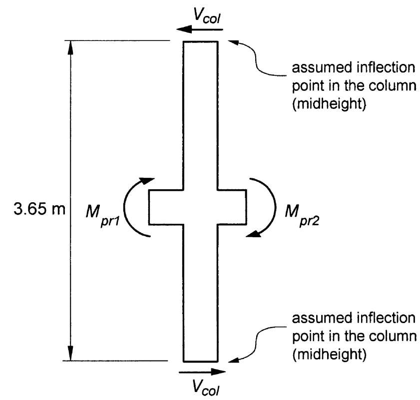


Fig. E8.4

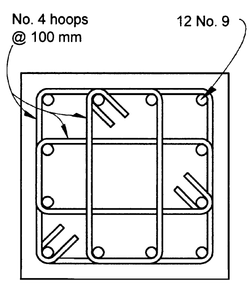


Fig. E8.2

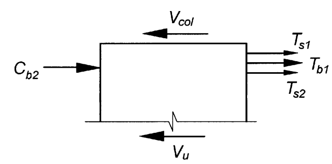


Fig. E8.5

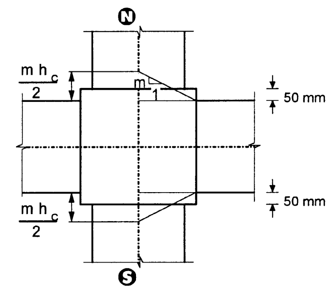


Fig. E8.6

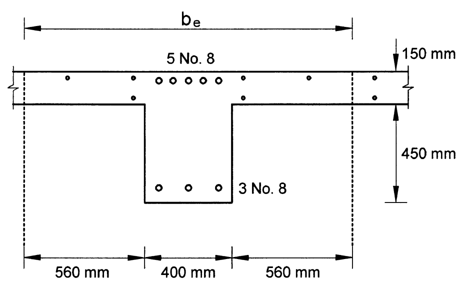


Fig. E8.3

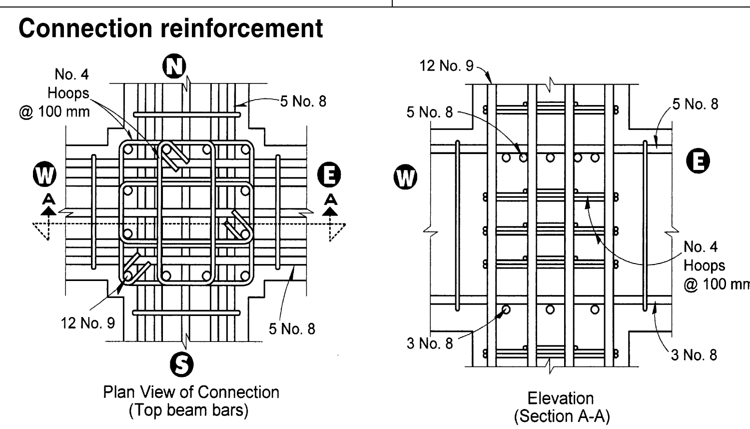


Fig. E8.7

$$b_e = 1.52 \text{ m} > 2b_b = 0.8 \text{ m}$$

Within the effective flange width (1.52 m), six No. 3 slab bars (top and bottom) should be considered for bending analysis. It is assumed that both top and bottom slab bars are continuous through the connection.

For positive bending moment

Similarly to negative bending moment, $b_e = 1.52$ m.

For the bending analysis that follows, ignore the effect of compression reinforcement and assume, in most locations, $d = h - 70$ mm. In locations where there is interference between bars from the normal and spandrel beams, assume $d = h - 95$ mm for the spandrel beam.

Longitudinal beam (E-W) (Fig. E8.4)

$$M_{pr,b} = A_s \alpha f_y \left(d - \frac{a}{2} \right)$$

$$a = \frac{A_s \alpha f_y}{0.85 f_c' b}$$

For positive bending moment

$$a = \frac{3(507 \text{ mm}^2)(1.25)(414 \text{ MPa})}{0.85(70 \text{ MPa})(1520 \text{ mm})} = 8.7 \text{ mm}$$

$$M_{pr1} = (1521 \text{ mm}^2)(1.25)(414 \text{ MPa})\left(600 - 95 - \frac{8.7}{2}\right)$$

$$= 394 \times 10^6 \text{ N-mm} = 394 \text{ kN-m}$$

For negative bending moment

$$a = \frac{(2535 + 213 \text{ mm}^2)(1.25)(414 \text{ MPa})}{0.85(70 \text{ MPa})(400 \text{ mm})} = 60 \text{ mm}$$

$$M_{pr2} = (2748\text{ mm}^2)(1.25)(414\text{ MPa})\left(600 - 70 - \frac{60}{2}\right)$$

$$= 711 \times 10^6\text{ N-mm} = 711\text{ kN-m}$$

Therefore, the column shear disregarding the beam shear for simplicity is (see Fig. E8.5)

$$V_{col} = \frac{M_{pr1} + M_{pr2}}{3.65\text{ m}} = \frac{394 + 711\text{ kN-m}}{3.65\text{ m}} = 302\text{ kN}$$

$$V_u = T_{b1} + T_{s1} + T_{s2} + C_{b2} - V_{col}$$
$$= \alpha f_y(A_{s1} + A_{s,s1} + A_{s,s2} + A_{s2}) - V_{col}$$

$$= 1.25\,(414\text{ MPa})(2535 + 284 + 142 + 1521\text{ mm}^2)/(1000\text{ N/kN}) - 302 = 2017\text{ kN}$$

For thick or heavily reinforced slabs the actual effective depth should be calculated.

Joint shear strength

$$V_n = 0.083\gamma\sqrt{f_c'}\,b_j h_c$$

Because beams are wide enough, the joint can be classified as case A.1 in Table 1 and Fig. 4.5 "joints effectively confined on all four vertical faces." So $\gamma = 20$

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} \\ b_b + \sum\dfrac{m \cdot h_c}{2} \\ b_c \end{cases}$$

According to Section 4.3.1 (Fig. E8.6):
$(m \cdot h_c)/2 \le$ extension of the column beyond the edge of the beam

$$\frac{0.5(500\text{ mm})}{2} = 125\text{ mm};$$

extension of the column beyond the edge of the beam = 50 mm, then $(m \cdot h_c)/2 = 50$ mm

$$b_j \le \begin{cases} \dfrac{b_c + b_b}{2} = \dfrac{500\text{ mm} + 400\text{ mm}}{2} = 450\text{ mm (governs)} \\ b_b + \sum\dfrac{m \cdot h_c}{2} = 400\text{ mm} + 2(50\text{ mm}) = 500\text{ mm} \\ b_c = 500\text{ mm} \end{cases}$$

$$\phi V_n = 0.85(0.083)(20)\sqrt{70\text{ MPa}}(450\text{ mm})(500\text{ mm})$$

$$\frac{1\text{kN}}{1000\text{ N}} = 2656\text{ kN} > 2017\text{ kN (OK)}$$

Flexural strength ratio (Section 4.4.5)

When determining the column flexural strength, the factored axial load that results in the lowest column flexural strength was assumed in this example to be 1800 kN (this will normally depend on actual load combinations). Also, α was set equal to 1.0 for this calculation. Using these assumptions, $M_{n,c} = 934$ kN-m.

The beam flexural strengths have been found earlier using $\alpha = 1.25$. Those beam strengths will be divided by 1.25 to obtain an approximate value for the beam flexural strength if $\alpha = 1.0$. If the strength ratio is close to the allowable value, a more accurate determination of the beam flexural strength for $\alpha = 1.0$ could be made.

$$M_{n1} \cong 394\text{ kN-m}/1.25 = 315\text{ kN-m}$$

$$M_{n2} \cong 711\text{ kN-m}/1.25 = 569\text{ kN-m}$$

$$\text{Flexural strength ratio} = \frac{\Sigma M_{n,c}}{\Sigma M_{n,b}} = \frac{2(934)}{315 + 569}\ 2.1 > 1.2\text{ (OK)}$$

Beam and column bars passing through the joints (Section 4.5.5) (Fig. E8.7)

The column dimension is governed by the largest beam bar (Eq. (4.11))

$$h_c > 20(414/414)(25.4\text{ mm}) = 508\text{ mm} \approx 500\text{ mm (OK)}$$

Beam depths are controlled by the column bars

$$h_b > 20(414/414)(28.6\text{ mm}) = 572\text{ mm} < 600\text{ mm (OK)}$$