

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. Saïid Saïidi

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

Chapter 1—Introduction, scope, and definitions, p. 2

- 1.1—Introduction
- 1.2—Scope
- 1.3—Definitions

ACI Committee Reports, Guides, Manuals, and Commentaries are intended for guidance in planning, designing, executing, and inspecting construction. This document is intended for the use of individuals who are competent to evaluate the significance and limitations of its content and recommendations and who will accept responsibility for the application of the material it contains. The American Concrete Institute disclaims any and all responsibility for the stated principles. The Institute shall not be liable for any loss or damage arising therefrom.

Reference to this document shall not be made in contract documents. If items found in this document are desired by the Architect/Engineer to be a part of the contract documents, they shall be restated in mandatory language for incorporation by the Architect/Engineer.

Chapter 2—Classification of beam-column connections, p. 3

- 2.1—Loading conditions
- 2.2—Connection geometry

Chapter 3—Design considerations, p. 3

- 3.1—Design forces and resistance
- 3.2—Critical sections
- 3.3—Member flexural strength
- 3.4—Serviceability

Chapter 4—Nominal strength and detailing requirements, p. 6

- 4.1—Column longitudinal reinforcement
- 4.2—Joint transverse reinforcement
- 4.3—Joint shear for Type 1 and Type 2 connections
- 4.4—Flexure
- 4.5—Development of reinforcement
- 4.6—Beam transverse reinforcement

Chapter 5—Notation, p. 17

Chapter 6—References, p. 17

- 6.1—Referenced standards and reports
- 6.2—Cited references

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

Copyright © 2002, American Concrete Institute.

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by electronic or mechanical device, printed, written, or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system of device, unless permission in writing is obtained from the copyright proprietor.

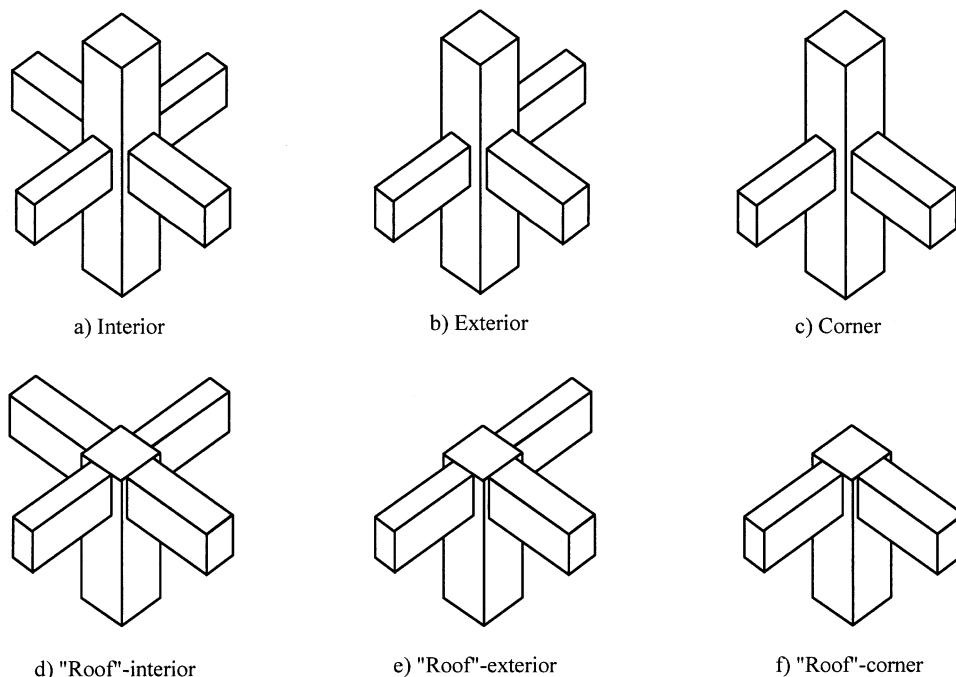


Fig. 1.1—Typical beam-to-column connections (slabs not shown for clarity). Wide-beam cases not shown.

Appendix A—Areas needing research, p. 20

- A.1—Effect of eccentric beams on joints
- A.2—Lightweight aggregate concrete in joints
- A.3—Limit on joint shear
- A.4—Behavior of indeterminate systems
- A.5—Distribution of plastic hinges
- A.6—Innovative joint designs
- A.7—Special joint configurations and loadings
- A.8—Joints in existing structures

Appendix B—Design examples, p. 21

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).

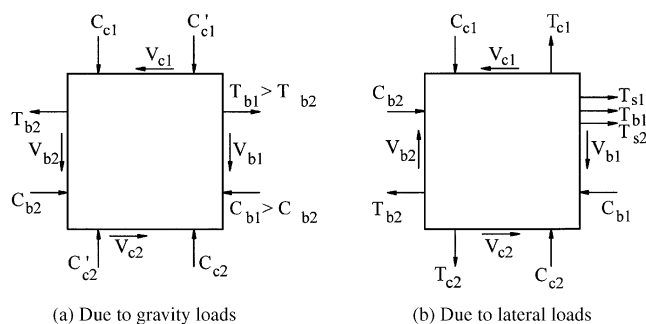


Fig. 3.1—Joint forces at critical sections. T = tension force; C = compression force; V = shear force; subscript b for beam; subscript c for column; and subscript s for slab.

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 $2b_f$, where b_f 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; Raffaele 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 subassemblies 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 $b_e/4$ and 6 in. (150 mm).

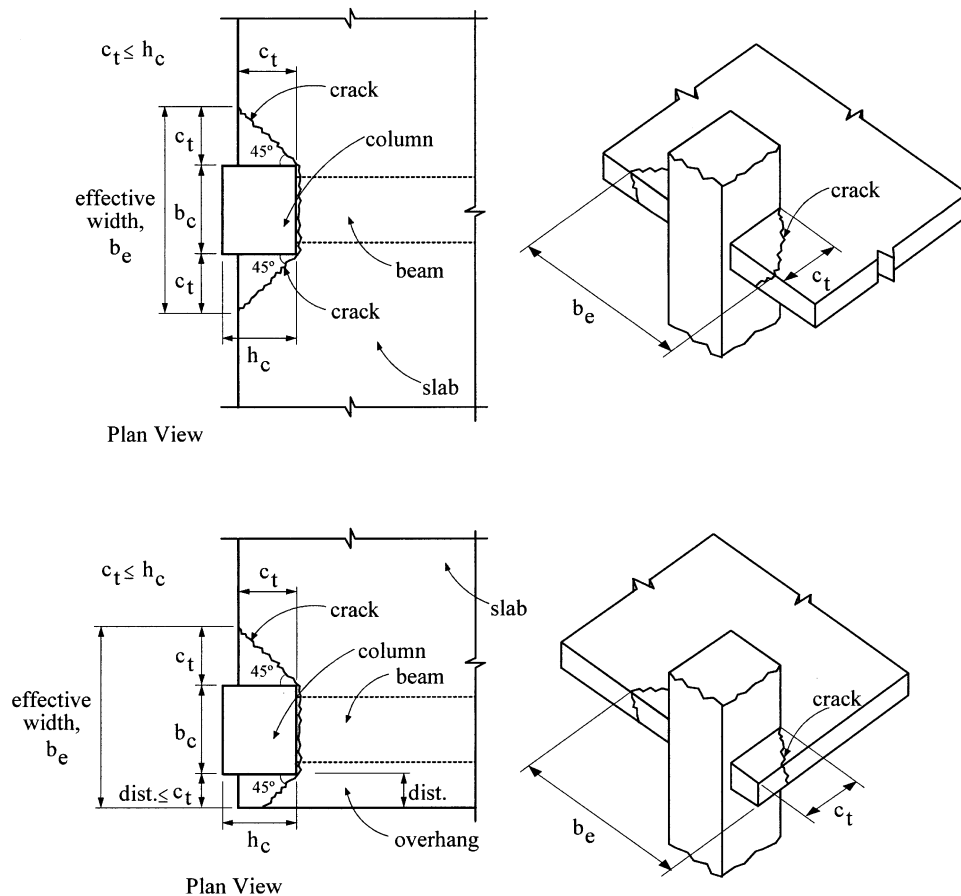


Fig. 3.2—Effective width at exterior connections with no transverse beam.

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 and column tension and compression forces under factored loading.

The value of $\alpha = 1.25$ 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 $\alpha = 1.25$ 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 $\alpha = 1.0$ 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 nonseismic 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.

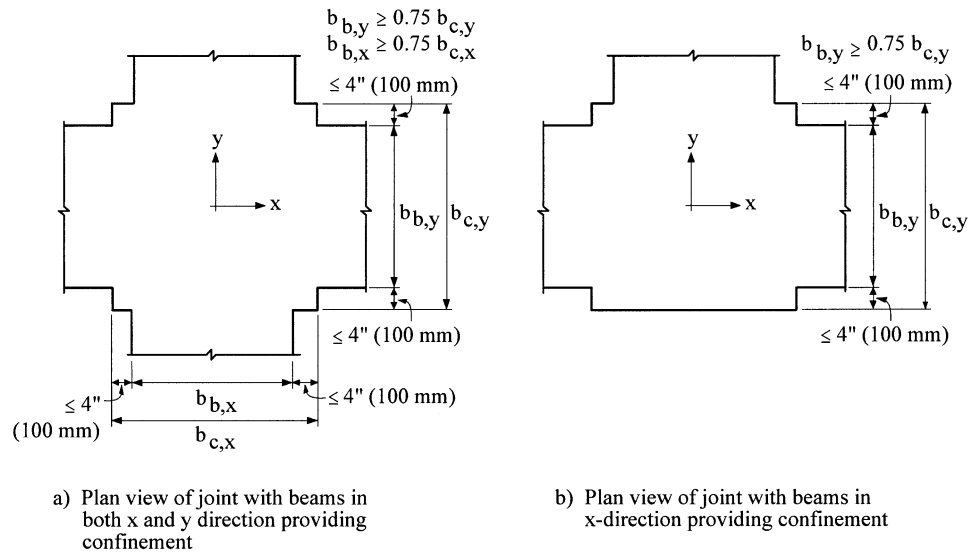


Fig. 4.1—Definition of adequate lateral confining members for evaluating joint transverse reinforcement.

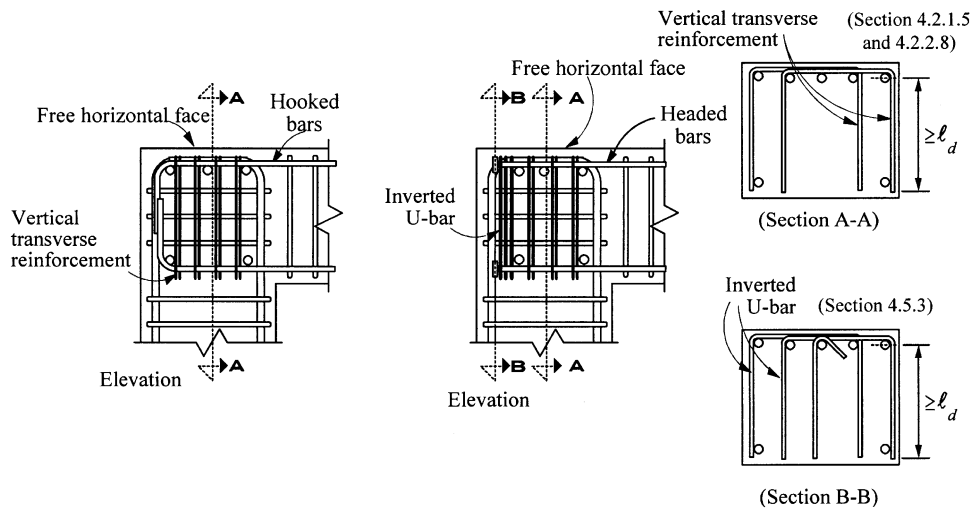


Fig. 4.2—Vertical transverse reinforcement in connections with discontinuous columns.

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

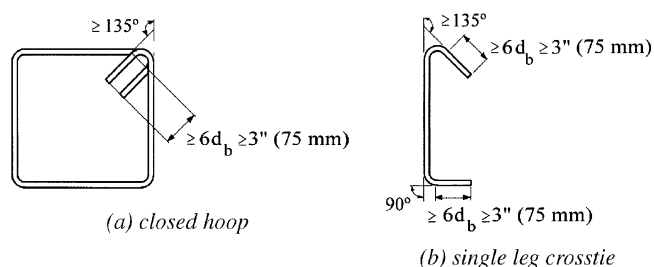


Fig. 4.3—Required dimensions of transverse reinforcement.

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

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \quad (4.2)$$

but should not be less than

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

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

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

but should not be less than

$$A_{sh} = 0.09 \frac{s_h b_c f'_c}{f_{yh}} \quad (4.5)$$

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 longitudinal 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

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

Classification	Connection type	
	1	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

should be taken as the centerline of the beam longitudinal reinforcement nearest to the unconfined face.

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

$$\phi V_n \geq V_u \quad (4.6)$$

where $\phi = 0.85$ and V_n , the nominal shear strength of the joint, is

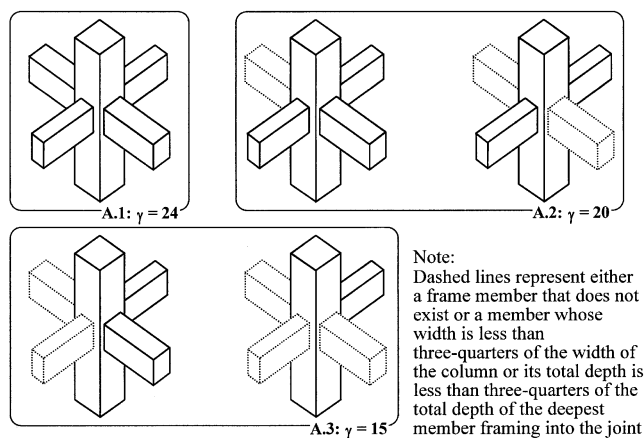
$$V_n = \gamma \sqrt{f'_c} b_j h_c \text{ (psi)}$$

$$V_n = 0.083 \gamma \sqrt{f'_c} b_j h_c \text{ (MPa)} \quad (4.7)$$

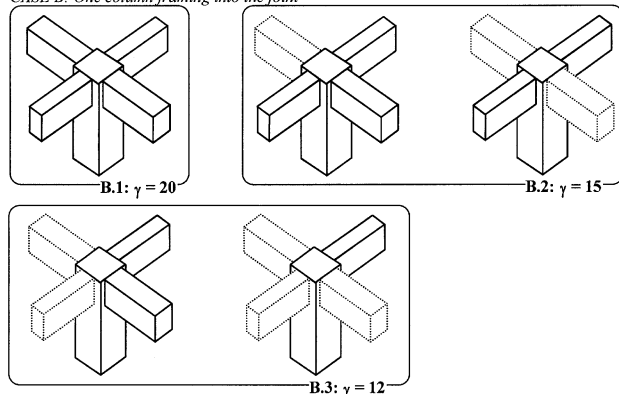
where b_j is the effective joint width as defined in Eq. (4.8), and h_c is the depth of the column in the direction of joint

TYPE 1 CONNECTIONS

CASE A: Two columns framing into the joint



CASE B: One column framing into the joint

Fig. 4.4— γ -values for Type 1 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

$$b_b + \sum \frac{mh_c}{2} \quad (4.8)$$

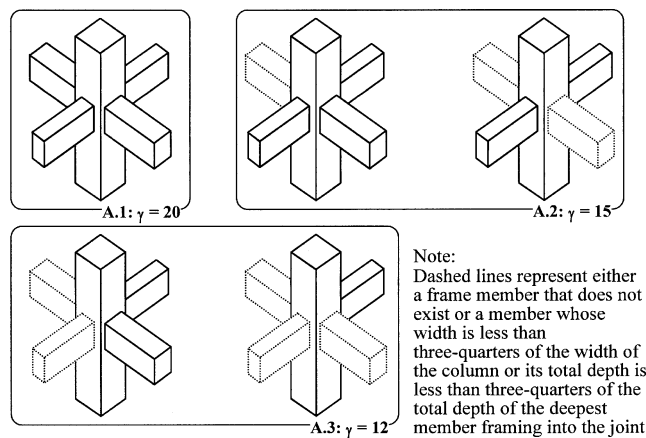
and

$$b_c$$

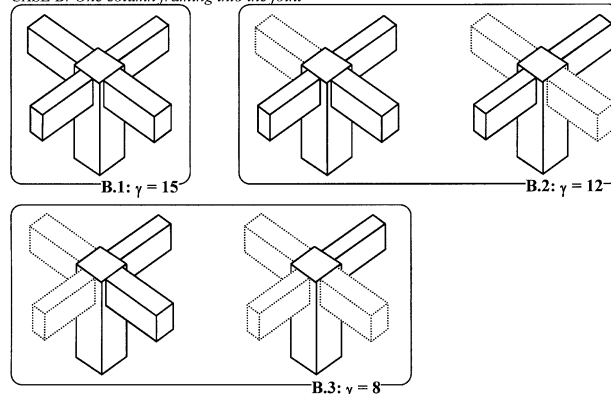
The term b_b is the width of the longitudinal beam. For joints where the eccentricity between the beam centerline and the column centroid exceeds $b_c/8$, $m = 0.3$ should be used; for all other cases, $m = 0.5$. The summation term should be applied on each side of the joint where the edge of the column extends beyond the edge of the beam. The value of $mh_c/2$ should not be taken larger than the extension of the

TYPE 2 CONNECTIONS

CASE A: Two columns framing into the joint



CASE B: One column framing into the joint

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

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 measured strengths (Alcocer 1993, Alcocer and Jirsa 1993).

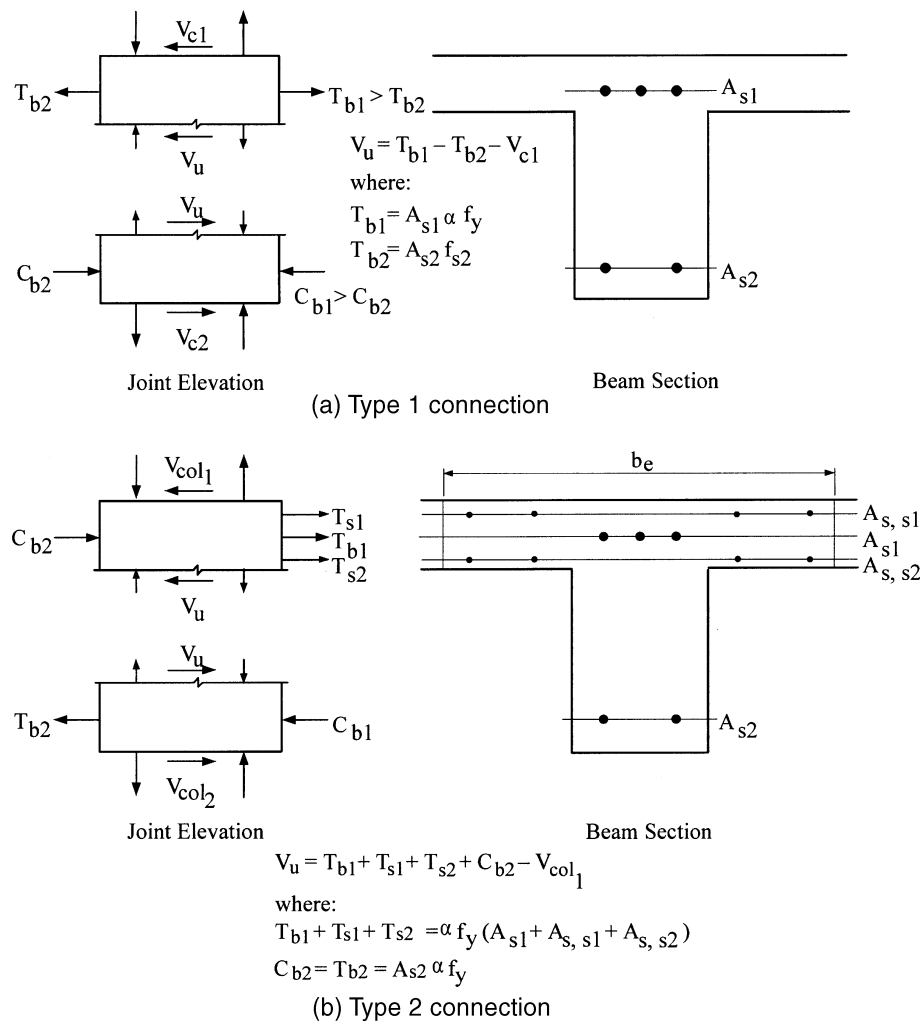
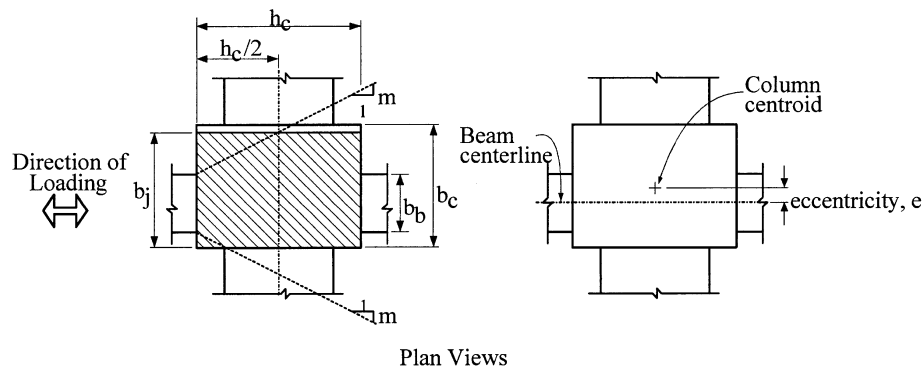


Fig. 4.6—Evaluation of horizontal joint shear.

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; Raffaele 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.