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

Reported by Joint ACI-ASCE Committee 352

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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.

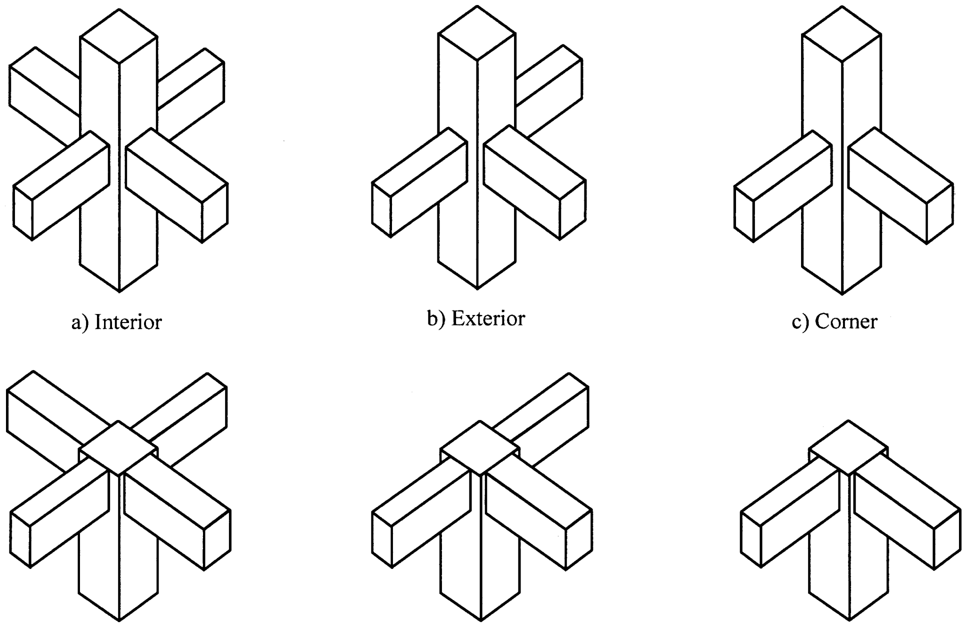

Fig. 1.1—Typical beam-to-column connections (slabs not shown for clarity). Wide-beam cases not shown.

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).

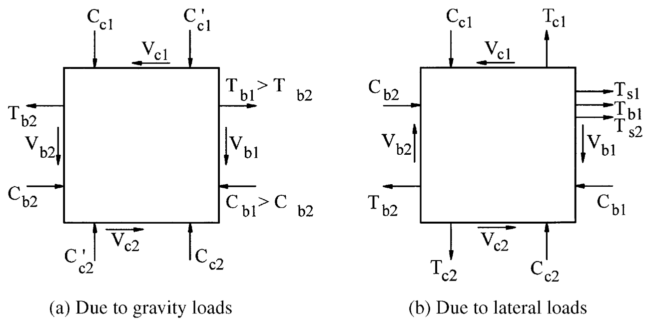

Fig. 3.1—Joint forces at critical sections. T = *tension force;* C = *compression force;* V = *shear force; subscript* b *for beam; subscript* c *for column; and subscript* s *for slab.*

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),