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

Reported by ACI-ASCE Committee 352

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Recommendations are given for determining proportions and details of monolithic, reinforced concrete slab-column connections.

Included are recommendations regarding appropriate uses of slab-column connections in structures resisting gravity and lateral forces, procedures for determination of connection design forces, procedures for determination of connection strength, and reinforcement details to insure adequate strength, ductility, and structural integrity. The recommendations are based on a review of currently available information. A commentary is provided to amplify the recommendations and identify available reference material. Design examples illustrate application of the recommendations. (Design recommendations are set in standard type. Commentary is set in italics.)

Keywords: anchorage (structural); beams (supports); collapse; columns (supports); concrete slabs; connections; earthquake-resistant structures; joints (junctions); lateral pressure: loads (forces); reinforced concrete; reinforcing steels; shear strength; stresses; structural design; structures.

CONTENTS

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents they should be phrased in mandatory language and incorporated into the Project Documents.

CHAPTER 1-SCOPE

These recommendations are for the determination of connection proportions and details that are intended to provide for adequate performance of the connection of cast-in-place reinforced concrete slab-column connections. The recommendations are written to satisfy serviceability, strength, and ductility requirements related to the intended functions of the connection.

*Members of the slab-column subcommittee.

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Design of the connection between a slab and its supporting member requires consideration of both the joint (the volume common to the slab and the supporting element) and the portion of the slab or slab and beams immediately adjacent to the joint. No reported cases of joint distress have been identified by the Committee. However, several connection failures associated with inadequate performance of the slab adjacent to the joint have been reported.[1-7] Many of these have occurred during construction when young concrete received loads from more than one floor as a consequence of shoring and reshoring.[8-10] The disastrous consequences of some failures, including total collapse of the structure, emphasize the importance of the design of the connection. It is the objective of these recommendations to alert the designer to those aspects of behavior that should be considered in design of the connection and to suggest design procedures that will lead to adequate connection performance.

Previous reports[5,11] and codes (ACI 318) have summarized available information and presented some design recommendations. The present recommendations are based on data presented in those earlier reports and more recent data.

The recommendations are intended to serve as a guide to practice.

These recommendations apply only to slab-column connections in monolithic concrete structures, with or without drop panels or column capitals, without slab shear reinforcement, without prestressed reinforcement, and using normal weight or lightweight concrete having design compression strength assumed not to exceed 6000 psi. Construction that combines slab-column and beam-column framing in orthogonal directions at individual connections is included, but these recommendations are limited to problems related to the transfer of loads in the direction perpendicular to the beam axis. The provisions are limited to connections for which severe inelastic load reversals are not anticipated. The recommendations do not apply to multistory slab-column construction in regions of high seismic risk in which the slab connection is a part of the primary lateral load resisting system. Slab-column framing is inappropriate for such applications.

These recommendations are limited to slab-column connections of cast-in-place reinforced concrete floor construction, including ribbed floor slab construction[12] and slab-column connections with transverse beams. Recommendations are made elsewhere (ACI 352R) for connections in which framing is predominantly by action between beams and columns.

The recommendations do not consider connections with slab shear reinforcement, slab-wall connections, precast or prestressed connections, or slabs on grade. The Committee is continuing study of these aspects of connection design. Relevant information on these subjects can be found in the literature. (See References 5, 11, and 13 through 18 for slab shear reinforcement, References 19 and 20 for slab-wall connections, and ACI 423.3R, and References 21 through 26 for prestressed connections.) Although structures having concrete compressive strength exceeding 6000 psi are within the realm of this document, the recommendations limit the assumed maximum value of compressive strength to 6000 psi.

Slab-column framing is generally inadequate as the primary lateral load resisting system of multistory buildings located in regions of high seismic risk (such as Zones 3 and 4 as defined in ANSI A.58.1 and UBC) because of problems associated with excessive lateral drift and inadequate shear and moment transfer capacity at the connection. In regions of high seismic risk, if designed according to provisions of these recommendations, slab-column framing may be acceptable in low-rise construction and multistory construction in which lateral loads are carried by a stiffer lateral load resisting system. In regions of low and moderate seismic risk (such as Zones I and 2 as defined in ANSI A.58.1 and UBC), slab-column frames may be adequate as the primary lateral load resisting system, provided the connection design recommendations in this document are followed.

CHAPTER 2-DEFINITIONS AND CLASSIFICATIONS

2.1 -Definitions

Joint-The part of the column within the depth of the slab including drop panel and having plan dimensions equal to those of the column at the intersection between the column and the bottom surface of the slab or drop panel.

Connection-The joint plus the region of the slab and beams adjacent to the joint.

Column-A cast-in-place vertical supporting element, including column capital if provided, with or without construction joints, designed to resist forces from the slab at the connection, and having a ratio of long to short cross-sectional dimensions not exceeding four.

Column capital-A flared portion of the column below the slab, cast at the same time as the slab, and having effective plan dimensions assumed equal to the smaller of the actual dimensions and the part of the capital lying within the largest right circular cone or pyramid with a 90-deg vertex that can be included within the outlines of the supporting column.

Drop panel-A thickened portion of the slab around the column having thickness not less than one-quarter of the surrounding slab thickness and extending from the column centerline in each principal direction a distance not less than one-sixth of the center-to-center span between columns.

Shear capital-A thickened portion of the slab around the column not satisfying plan dimension requirements for drop panels.

Slab critical section-A cross section of the slab near the column, having depth d perpendicular to the slab and extending around the column (including capital). A critical section should be considered around the column so that its perimeter b_o is a minimum, but it need not approach closer than the lines located $d/2$ from the column face and parallel to the column boundaries. Alternate critical sections should be investigated at other sections that might result in reduced shear strength. For the purpose of defining the slab critical section, a support of circular cross section may be replaced by a square support having an equal cross-sectional area.

Direction of moment-Defined to be parallel to the flexural reinforcement placed to resist that moment. In connection design and analysis, moments may be idealized as acting about two orthogonal axes, in which case orthogonal directions are defined for the moments.

Transfer moment-The portion of the slab total moment transferred to the supporting element at a connection. The transfer moment is identical in meaning to the unbalanced moment as defined in ACI 318.

Performance of a connection can be affected by behavior of the joint (including slip of reinforcement embedded in the joint) and by the region of the slab or slab and beams surrounding the joint. In general, the region of slab that directly affects behavior of the connection extends from the joint face not more than approximately twice the development length of the largest slab bars or four slab thicknesses, whichever is greater." The joint definition is illustrated in Fig. 2. 1.

The slab critical section, used for slab strength determination, is the same as that specified in ACI 318, although the definition has been modified to clarify that slab critical sections for rectangular supports may be assumed to have a rectangular shape. The slab critical sections for several support geometries are shown in Fig. 2.2. Punching shear strengths for circular columns have been observed'" to exceed the punching shear strengths for square columns having the same cross-sectional area. Thus, it is conservative and may be analytically simpler to represent circular columns by square columns having the same cross-sectional area [Fig. 2.2(c)]. Two critical sections are defined for connections with drop panels or shear capitals because failure may occur either through the thickened portion of the slab near the column or through the slab outside the drop panel or shear capital [Fig. 2.2(d)].

Fig. 2.3 illustrates the limitation on the aspect ratio of the column cross-sectional dimensions. As the aspect ratio becomes elongated, behavior deviates from that which is assumed in this report.[20] In such instances, the connection between the supporting member and the slab should be designed as a slab-wall connection. No recommendations for such connections are made in this report. Information is available in the literature.[19,20]

The direction of moment is parallel to slab reinforcement placed to resist that moment. For example, in a one-way slab (Fig. 2.4), the direction of moment is parallel to the span of the slab. Using vector notation, the moment vector [Fig. 2.5(c)] is perpendicular to the moment direction.

2.2-Classifications

Connections are classified according to geometry in Section 2.2.1 and according to anticipated performance in Section 2.2.2.

2.2.1 A slab-column connection is an exterior connection if the distance from any discontinuous edge to the nearest support face is less than four slab thicknesses. An edge connection is an exterior connection for which a discontinuous edge is located adjacent to one support face only. A corner connection is an exterior connection for which discontinuous edges are located adjacent to two support faces. A vertical slab opening located closer than four slab thicknesses to the support face should be classified as a discontinuous edge if radial lines projecting from the centroid of the support area to the boundaries of the opening enclose a length of the slab critical section that exceeds the adja-

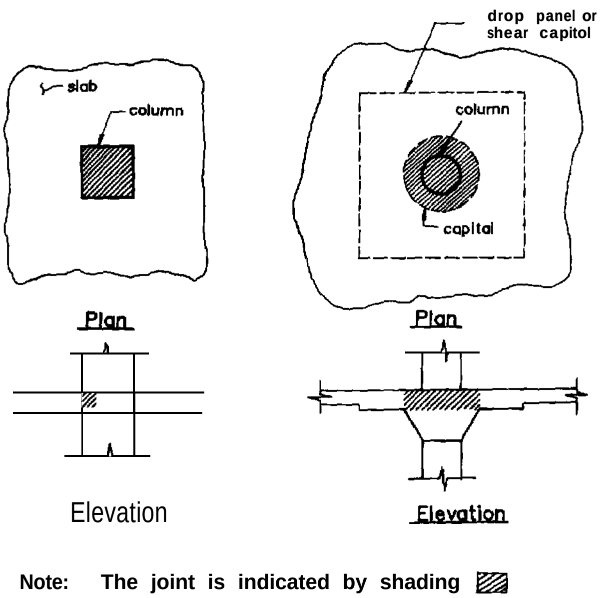
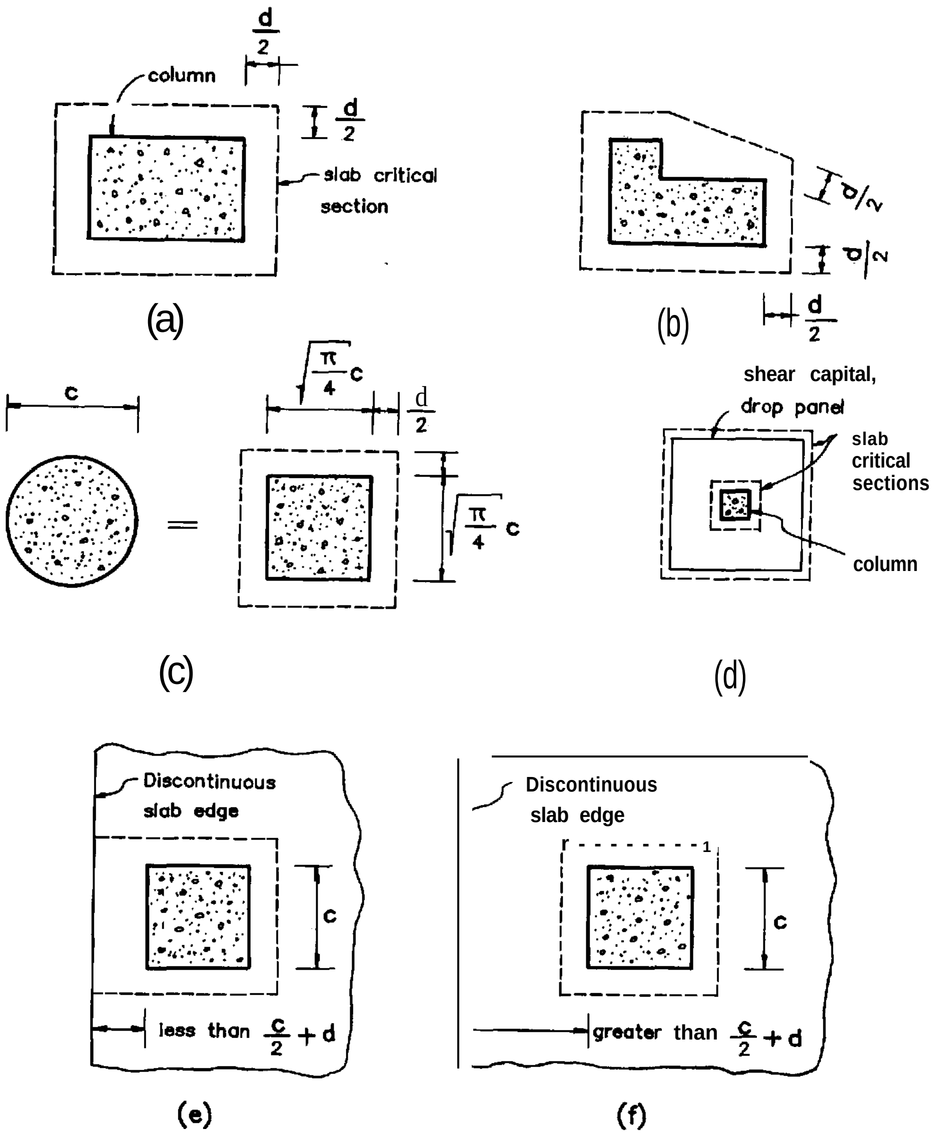


Fig. 2.1-Joint in typical slab-column connections



Note: For exterior connections, the slab critical section should extend to the slab edge as shown in (e) if such extension will reduce the critical section perimeter. Otherwise, the slab critical section is as shown in (f)

Fig. 2.2-Examples of slab critical sections

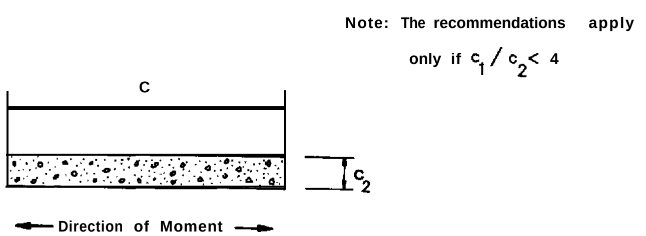


Fig. 2.3-Limitation on column aspect ratio

cent support dimension. A connection not defined as an exterior connection is considered to be an interior connection.

Openings or slab edges located close to the support interrupt the shear flow in the slab, induce moment transfer to supports, reduce anchorage lengths, and reduce the effective joint confinement. The distance of four times the slab thickness is based on considerations related to strength of the slab near the support.[11] *Several examples of exterior connections are in Fig. 2.5.*

Where openings are located closer than four slab thicknesses, the connection may behave as an exterior connection, depending on the size and proximity of the opening. To gage approximately the effect of the opening, radial lines are drawn from the centroid of the support area to the boundaries of the opening [Fig. 2.5(e)]. If the length of the slab critical section enclosed within the radial lines exceeds the adjacent support dimension, the connection is classified as an exterior connection. In the preceding, if there are no shear capitals, a support should be interpreted as being the column plus column capital if present. If there are shear capitals, the effect of the opening should first be checked considering the column to act as the support, and secondly, considering the shear capital to act as the

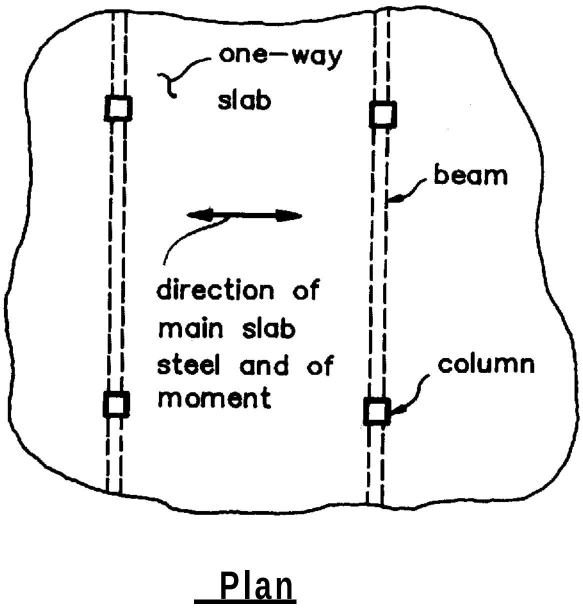


Fig. 2.4-Moment direction for one-way slab

support. For the purpose of classifying a connection as interior or exterior, the effect of openings on the critical section around a drop panel need not be considered.

Where distances to openings and free edges exceed the aforementioned requirements, the connection may be defined as being interior. In such cases, the diameter of the longitudinal bars should be iimited so that adequate development is available between the column and the opening or edge. Recommendations given elsewhere" suggest that bars should be selected so that the development length is less than half the distance from the column face to the edge or opening.

2.2.2 A connection is classified as either Type 1 or Type 2 depending on the loading conditions of the connection as follows:

(a) Type 1: A connection between elements that are designed to satisfy ACI 318 strength and serviceability requirements and that are not expected to undergo deformations into the inelastic range during the service life.

(b) Type 2: A connection between elements that are designed to satisfy ACI 318 strength and serviceability requirements and that are required to possess sustained strength under moderate deformations into the inelastic range, including but not limited to connections subjected to load reversals.

The design recommendations for connections are dependent on the deformations implied for the design loading conditions. A Type I connection is any connection in a structure designed to resist gravity and normal wind loads without deformations into the inelastic range for expected loads. Some local yielding of slab reinforcement may be acceptable for Type I connections. Slabs designed by conventional yield-line methods may be included in this category, except if required to resist loads as described for Type 2 connections. A Type 2 connection is a connection between members that may be required to absorb or dissipate moderate amounts of energy by deformations into the inelastic range. Typical examples of Type 2 connections are those in structures designed to resist earthquakes or very high winds. In structures subjected to very high winds or seismic loads, a slab-column connection that is rigidly connected to the primary lateral load resisting system should be classified as a Type 2 connection even though it may not be considered during design as a part of that primary lateral load resisting system. As noted in Chapter 1, these recommendations do not apply to multistory frames in regions of high seismic risk in which slab-column framing is considered as part of the primary lateral load resisting system.

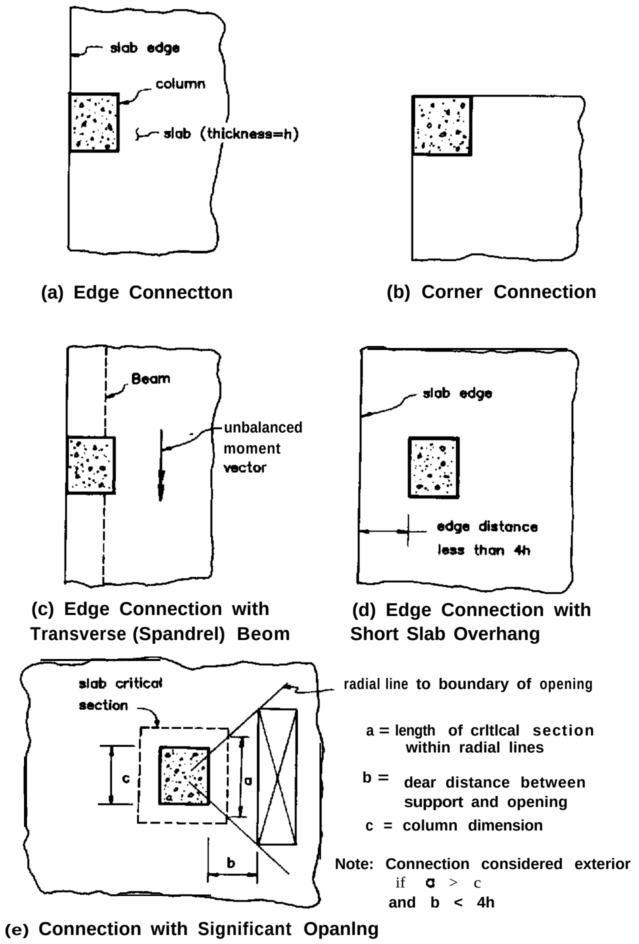


Fig. 2.5-Examples of exterior connections

CHAPTER 3-DESIGN CONSIDERATIONS

3.1-Connection performance

The connection should be proportioned for serviceability, strength, and ductility to resist the actions and forces specified in this chapter.

3.2-Types of actions on the connection

3.2.1 The design should account for simultaneous effects of axial forces, shears, bending moments, and torsion applied to the connection as a consequence of external loads, creep, shrinkage, temperature, and foundation movements. Loads occurring during construction and during the service life should be considered.

The connection should be designed for the forces due to applied external loads and due to time-dependent and temperature effects where they are significant. Effects of construction loads and early concrete strengths are of particular importance for slabs without beams, as demonstrated by several catastrophic failures during construction.[1-4] Effects of heavy construction equipment and of shoring and reshoring[8,27,28] should be considered. Effects of simultaneous bidirectional moment transfer should be considered in design of the connection, except wind or seismic lateral loads generally are not considered to act simultaneously along both axes of the structure in design.

3.2.2 Moment transfer about any principal axis should be included in evaluating connection resistance if the ratio between the factored transfer moment and factored slab shear at the slab critical section exceeds 0.2d, where d is the slab effective depth. The moment should be taken at the geometric centroid of the slab critical section defined in Section 2.1. Where biaxial moments are transferred to the support, the $0.2d$ limitation can be applied independently about both principal axes of the connection.

Moment transfer at a connection can reduce the shear strength of a slab-column connection. However, the strength reduction for eccentricity less than 0.2d is within the experimental scatter for nominally identical connections transferring shear only."

3.3-Determination of connection forces

3.3.1 Forces on the connection may be determined by any method satisfying requirements of equilibrium and geometric compatibility for the structure. Time-dependent effects should be evaluated.

3.3.2 For normal gravity loads, the recommendations of Section 3.3.1 may be satisfied using the Direct Design Method or the Equivalent Frame Method of ACI 318. For uniformly loaded slabs, slab shears at the connection may be determined for loads within a tributary area bounded by panel centerlines; slab shears at first interior supports should not be taken less than 1.2 times the tributary area values unless a compatibility analysis shows lower values are appropriate.

The design should account for the worst combinations of actions at the connection. Analysis for connection forces should consider at least (a) loads producing the maximum slab shear on the slab critical section, and (b) loads producing the maximum moment transfer at the slab critical section.

Factored slab shear at the connection can be determined by several procedures, including yield line and strip design methods[13,29] and the equivalent frame method. However, in typical designs, simpler procedures such as the use of tributary areas are acceptable. The designer is cautioned that the shear at first interior supports is likely to be higher (by as much as 20 percent) than the tributary area shears[30,31] because of continuity effects.

3.3.3 For lateral loads, effects of cracking, compatibility, and vertical loads acting through lateral displacements (P-delta effects) should be considered.

Cracking in the connection has been shown[32-34] to reduce connection lateral-load stiffness to a value well below the stiffness calculated by the elastic theory.[32,35] The reduction in stiffnes can result in lateral drift exceeding that anticipated by a conventional elastic analysis. Effects of gravity loads acting through lateral displacements (P-delta effects) are consequently amplified and may play an important role in behavior and stability of slab-column frames. Methods of estimating reduced lateral-load stiffness are discussed in References 32, 33, and ACI 318R.

CHAPTER 4-METHODS OF ANALYSIS FOR DETERMINATION OF CONNECTION STRENGTH

4.1 -General principles and recommendations

Connection strength may be determined by any method that satisfies the requirements of equilibrium and geometric compatibility and that considers the limiting strengths of the slab, the column, and the joint. In lieu of a general analysis, strength of the slab included in the connection may be determined according to the procedures given in Sections 4.2, 4.3, and 4.4, and strength of the joint may be determined according to Section 4.5.

Methods of computing strength of the slab in shear and moment transfer have received considerable attention in literature in recent years. Available methods include applications of yield line theory, elastic plate theory, beam analogies, truss models, and others.[5,36-41] The explicit procedures given in Sections 4.2, 4.3, and 4.4 provide acceptable estimates of connection strength with a reasonable computational effort. It is noted that moment transfer strength of a connection may be limited by the sum of the strengths of columns above and below the joint; hence, connection strength should not be assumed to exceed this limiting value.

4.2-Connections without beams

The connection should be proportioned to satisfy Sections 4.2.1 and 4.2.2.

4.2.1 Shear

4.2.1.1 *Connections transferring shear*-Shear strength V_o in the absence of moment transfer is given by

$$V_o = \phi V_n, \text{ where } V_n = C_v V_c \qquad (4\text{-}1)$$

in which $\phi = 0.85$, V_n = the nominal shear strength, V_c = basic shear strength carried by concrete, and C_v is the product of all appropriate modification factors given in Table 4.1 and is taken equal to 1.0 if none of the modification factors of Table 4.1 are applicable

$$V_c = (2 + 4/\beta_c)\sqrt{f'_c}\, A_{cs} \leqslant 4\, A_{cs}\sqrt{f'_c} \qquad (4\text{-}2)$$

in which β_c = ratio of long to short cross-sectional dimensions of the supporting column, A_{cs} = cross-sectional area of the slab critical section = $b_o d$, and f'_c = concrete compressive strength in units of psi and not to exceed 6000 psi.

Eq. (4-l) defines shear strength in the absence of moment transfer. The presence of moment may result in decreased shear strength. Therefore, the designer is cautioned when computing the required connection moment strength to consider effects of pattern loads, lateral loads, construction loads, and possible accidental loads.

Eq. (4-l) is based on a similar equation for two-way shear strength as presented in the ACI 318. However, modification factors not included in ACI 318 are included in these recommendations. The basic shear strength should be multiplied by each of the applicable modification factors in Table 4.1 to arrive at the nominal shear strength V_n. The modification factors reflect how each variable individually affects shear strength. There is little experimental information to show that the effects are cumulative. The Committee recommendation is intended to be conservative.

The maximum value of $4\sqrt{f'_c}\,A_{cs}$ for the basic shear strength given in Eq. (4-2) exceeds the nominal strength of $2\sqrt{f'_c}\,bd_{beam}$ used for beams largely because of the geometric confinement afforded to the slab shear failure surface. As the supporting column cross section becomes elongated, the confinement due to lateral compression along the long face is diminished. The term β_c in Eq. (4-2) reflects the reduction in strength due to reduction in lateral confinement. A similar phenomenon arises if the critical section perimeter b_o greatly exceeds the depth d of the slab,[42] as occurs for the critical section around drop panels and shear capitals. The values of the modification factors as a function of b_o/d are based subjectively on trends observed in References 42 and 43. Research on interior connections with shearhead reinforcement[43] shows that the nominal strength decreases as the distance between the critical section and the column face increases. An evaluation of the data by the Committee indicates that the reduction may also have been attributable to the increase in the ratio of the critical section dimension to slab depth.

Lightweight aggregate concretes have been observed[44] to exhibit lower shear strengths reiative to normal weight concretes having the same compressive strength.

Connections subjected to widespread flexural yielding have been observed[42] to exhibit shear strengths lower than those observed for connections failing in shear prior to flexural yielding. Nominal shear strength for this case is reduced by a factor of 0.75. This provision should be applied for all Type 2 connections and for some Type 1 connections. Included in the latter category are slabs designed by yield-line methods. The possibility of yield should be considered in flat-slab and flat-plate floor systems for which column layouts are irregular.

The basic shear strength given by Eq. (4-2) is written as a function of the square root of the concrete compressive strength. Some research[5,45] suggests that the relation should be in terms of the cube root of concrete strength rather than the square root. Thus, it is possible that shear strength given by Eq. (4-2) is unconservative for concrete strengths exceeding 6000 psi, the upper bound of strengths reported in tests of slab-column connections.

During construction, young and relatively weak concrete may need to carry heavy loads. Low concrete strength has a greater effect on shear strength than flexural strength. Thus, there is a tendency toward connection shear failures. In checking resistance to construction loads that occur before the full design concrete strength develops, it is important to use the concrete strength corresponding to the age at which the load occurs rather than the design strength.

Table 4.1 — Modification factors for basic shear strength

Condition	Modification factor
All-lightweight concrete	0.75
Sand-lightweight concrete	0.85
Flexural yielding anticipated in slab, including all Type 2 connections	0.75
$20 < b_o/d \leqslant 40$	0.75
$b_o/d > 40$	0.5

4.2.1.2 *Connections transferring shear and moment*-Any connection may be designed in accordance with the recommendations of Section 4.2.1.2(a). Connections satisfying the limitations of Sections 4.2.1.2(b) or 4.2.1.2(c) may be designed by the procedures listed in those sections in lieu of the procedure in Section 4.2.1.2(i). All Type 2 connections should satisfy the recommendation of Section 4.2.1.2(d) in addition to the other recommendations of this section. All connections should meet the recommendations of Section 4.2.2.

(a) The fraction of the transfer moment given by

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\beta_{cr}}} \qquad (4\text{-}3)$$

should be considered resisted by shear stresses acting on the slab critical section. In Eq. (4-3), β_{cr} is the ratio of the lengths of the sides of the slab critical section measured parallel and transverse to the direction of moment transfer, respectively. The shear stresses due to moment transfer should be assumed to vary linearly about the centroid of the slab critical section. The algebraic sum of shear stresses due to direct shear and moment transfer should not exceed the value of V_o/A_{cs}.

(b) Corner connections, and edge connections transferring moments only perpendicular to the slab edge, may be assumed to have adequate shear strength if the factored direct shear transferred to the column does not exceed 0.75 V_o, with V_o defined by Eq. (4-l).

(c) Connections supported on columns having a ratio of long to short cross-sectional dimensions less than or equal to two may be assumed to have adequate shear strength to transfer the factored connection shear and moment if

$$V_o \geqslant V_u + \alpha(M_{ub1} + M_{ub2})/b_o \quad (4\text{-}4)$$

in which b_o = perimeter of the slab critical section, V_u = factored direct shear on the slab critical section, and M_{ub1} and M_{ub2} are the factored moments transferred simultaneously to the support in the two principal directions at the geometric centroid of the slab critical section. For exterior connections, moments perpendicular to the slab edge may be taken equal to zero in Eq. (4-4) if V_u does not exceed 0.75 V_o, with V_o defined by Eq. (4-1). The value of α should be taken equal to 5 for interior connections and 3.5 for edge connections.

(d) For all Type 2 connections, the maximum shear acting on the connection in conjunction with inelastic moment transfer should not exceed $0.4V_c$.

Shear strength may be reduced when moments are transferred simultaneously to the connection. In Section 4.2.1.2, several alternate procedures for considering the effects of moment transfer are recommended. The most general of the recommended procedures, which can be applied to connections of any geometry and loading, is described in Section 4.2.1.2(a). However, connections can be designed with less computational effort if they satisfy the loading and geometric requirements of Section 4.2.1.2(b) or 4.2.1.2(c).

The design method described in Section 4.2.1.2(a) is identical to the eccentric shear stress model embodied in ACI 318. It is assumed that shear stresses due to direct shear on the connection are uniformly distributed on the slab critical section. In addition, a portion of the unbalanced moment given by Eq. (4-3) is resisted by a linear variation of shear stresses on the slab critical section. The algebraic sum of shear stresses due to direct shear and moment transfer should not exceed the value of V_o/A_{cs}. The portion of moment not carried by eccentric shear stresses is to be carried by slab flexural reinforcement according to Section 4.2.2. The method is described in detail in several references (e.g., ACI 318R, and Reference 13).

For corner connections, and for edge connections transferring moment only perpendicular to the slab edge, a simple computational design procedure is given in Section 4.2.1.2(b). The procedure is based on research[46] on slab-column edge connections for which the outside face of the column is flush with the slab edge. For such connections, moment transfer strength perpendicular to the slab edge is governed by slab flexural reinforcement within an effective transfer width, and apparently is not influenced significantly by shear on the connection. Failure apparently occurs when the connection moment reaches the flexural strength of slab reinforcement, or the connection shear reaches the shear strength of the slab critical section. In cases where moments induce yield in slab flexural reinforcement, shear failure can apparently occur for shear less than that given by Eq. (4-1) because of loss of in-plane restraint when the flexural reinforcement yields. For that reason, an upper limit equal to three-quarters of the value given by Eq. (4-1) is recommended. Recommendations for moment transfer reinforcement are given in Section 4.2.2.

For interior or edge connections having a ratio between long and short column dimensions less than or equal to two, effects of moment transfer on shear strength can be accounted for by proportioning the connection to satisfy the recommendations of Section 4.2.1.2(c). Eq. (4-4) of that section essentially emulates, in algebraic form, the eccentric shear stress model described in Section 4.2.1.2(a). The form of Eq. (4-4) was originally presented by ACI-ASCE Committee 426,[11] which recommended the equation for interior connections with a value of α equal to 5.2. The value of α has been modified to 5.0 for interior connections. For edge connections transferring moment only parallel to the slab edge, a value of α equal to 3.5 is appropriate. For edge connections also transferring moment perpendicular to the slab edge, the shear V_u is usually less than $0.75V_o$, in which case moments perpendicular to the slab edge can be ignored in Eq. (4-4). This equation may be unconservative for connections not satisfying the requirement for column cross section aspect ratio.

The recommendation in Section 4.2.1.2(d) should be applied to all connections without beams for which inelastic moment transfer is anticipated. The recommendation is based on a review[47] of data reported in References 33, 34, and 48 through 52, and some previously unpublished tests, which reveal that lateral displacement ductility of interior connections without shear reinforcement is inversely related to the level of shear on the connection. Connections having shear exceeding the recommended value exhibited virtually no lateral displacement ductility under lateral loading. The recommendation of Section 4.2.1.2(d) may be waived if calculations demonstrate that lateral interstory drifts will not induce yield in the slab system. For multistory construction, stiff lateral load resisting structural systems comprising several structural walls may be adequate.

4.2.2 Flexure-Slab flexural reinforcement should be provided to carry the moment transferred to the connection in accordance with Section 5.1.1.

4.3-Connections with transverse beams

If a connection has beams transverse to the span of the slab, shear and moment transfer strength of the connection may be determined as follows:

4.3.1 Shear strength is the smaller of the following:

(a) Design shear strength limited by beam action with a critical section extending across the entire slab width in a plane parallel to the beam and located a distance d from the face of the beam, where d is the slab effective depth. Design shear strength for this condition is calculated according to ACI 318 for beams.

(b) Design shear strength limited by the sum of design strengths in shear of only the transverse beams. Design shear strength of the transverse beams at a distance d_{beam} from the support face should be computed considering interaction between shear and torsion, where d_{beam} is the beam effective depth.

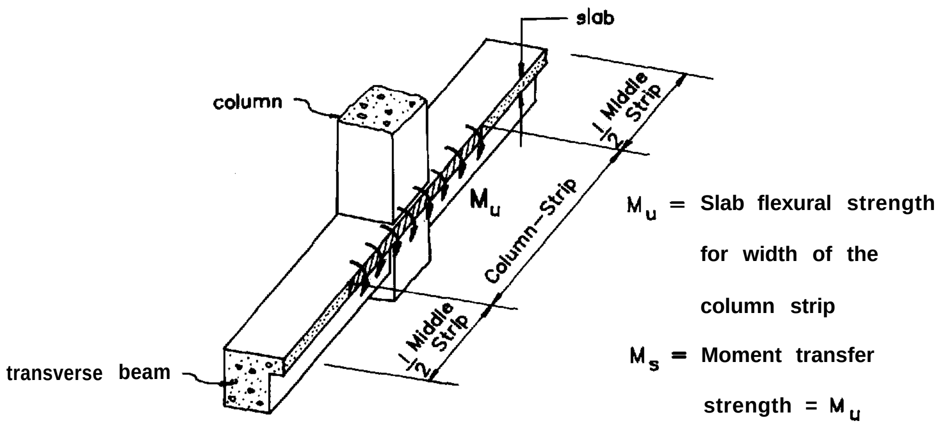
4.3.2 Moment transfer strength is the smaller of the following:

(a) Design flexural strength of the slab at the face of the support over a width equal to that of the column strip.

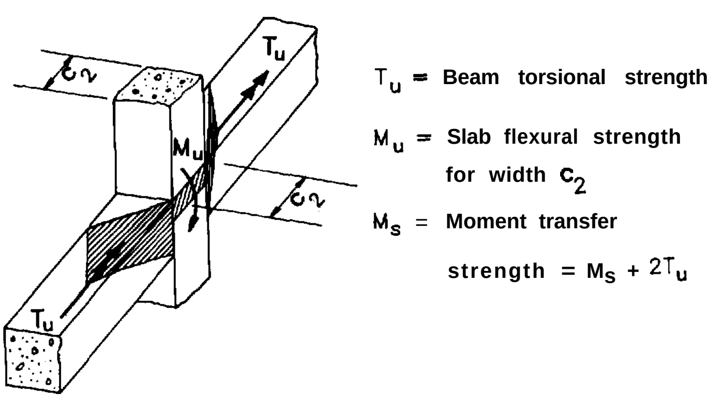
(b) Sum of the design flexural strength of the slab and the design torsional strengths of the transverse beams. Slab design flexural strength is computed over a width equal to that of the support face.

The procedure described is based on concepts of the beam analogy as presented in Reference 38. The procedure assumes the shear strength is limited by either beam action in the slab or by development of shear strengths of the beams at the side faces of the connection. For connections having substantial transverse beams, it is unlikely that the beams and slab will develop design shear strengths simultaneously, so shear strength should be limited to the contribution of the beams only.

Flexural strength is limited by development of a flexural yield line across the slab column-strip width, in which case the transverse beams do not reach their design strengths [Fig. 4.1(a)], or by development of a yield surface around the connection that involves flexural yield of the slab and torsional yield of the transverse beams [Fig. 4.1(b)]. Beam torsional strength is calculated considering interaction between shear and torsion. The beam shear may be determined by the procedure given in Reference 16, or more simply, all shear may be assumed distributed to beams in proportion to their tributary areas if the beams have equal



(a) Strength Limited by Slab Column-Strip Capacity



(b) Strength Limited by Combined Flexural/Torsional Capacities

Fig. 4.1-Unbalanced moment strength of connections with transverse beams

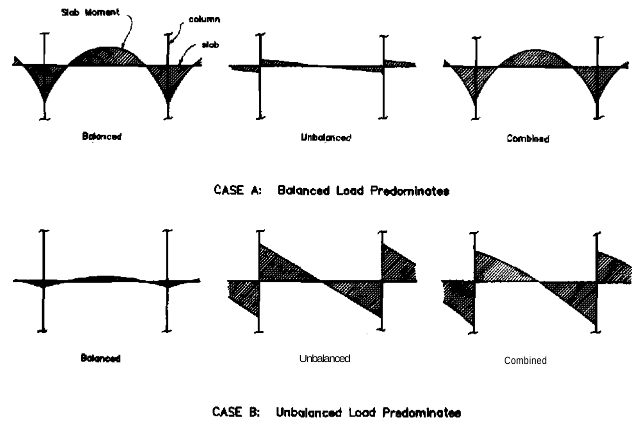


Fig. 5.1-Illustration of cases where balanced and unbalanced connection moments predominate

stiffness. Combined shear and torsion strength may be represented as in ACI 318 or can be based on other methods such as those described in References 53 and 16.

4.4-Effect of openings

When openings perpendicular to the plane of the slab are located closer to a slab critical section than four times the slab thickness, the effect of such openings should be taken into account. This may be done using a general analysis that satisfies requirements of equilibrium and compatibility. In lieu of a general analysis, Section 4.2 or 4.3 should be followed as appropriate, except that portions of the slab critical section enclosed within lines from the centroid of the support area to the extreme edges of the opening should be considered ineffective. The eccentricity of the applied shear caused by the opening should also be taken into account, except where the ineffective length of the slab critical section is less than either d or half the length of the adjacent support face. The support should be considered the column including column capital if the critical section under consideration is adjacent to the column, and should be considered the shear capital or drop panel if the critical section under consideration is adjacent to the shear capital or drop panel.

Slab perforations and embedded service ducts disrupt the flow of flexural and shear stresses in the vicinity of the connection and generally result in decreased strength. The influence is a function of proximity and size of the disruption. Effects of slab perforations and of embedded service ducts are described in Reference 54.

4.5-Strength of the joint

4.5.1 ***Axial compression***--If the design compressive strength of concrete in the column is less than or equal to 1.4 times that of the floor system, strength of the joint in axial compression can be assumed equal to strength of the column below the joint. Otherwise, axial strength should be determined according to Section 10.13 of ACI 318. The column longitudinal reinforcement should be continuous through the joint, with or without splices, and the joint should be confined as specified in Section 5.2.2 of these recommendations.

4.5.2 *Shear*-Calculations for joint shear strength in slab-column connections are not required.

The committee is aware of no cases of joint shear failure in flat slab or flat plate connections. The absence of joint shear failures is likely to be attributable to two phenomena: (1) For slabs of usual proportions, the magnitudes of moment transfer that can be developed, and hence of the joint shear, are not excessive; and (2) confinement afforded by the slab concrete enhances joint shear strength.

CHAPTER 5-REINFORCEMENT REQUIREMENTS

5.1 -Slab reinforcement for moment transfer

5.1.1

(a) Interior connections-Reinforcement required in each direction to resist the moment $\gamma_f M_{ub}$, where γ_f = $1 - \gamma_v$, should be placed within lines $1.5h$ either side of a column (including capital), where M_{ub} = the moment transferred to the column in each principal direction, h = the slab thickness including drop panel, and γ_f = fraction of moment transferred by flexure. The reinforcement should be anchored to develop the tensile forces at the face of the support. Reinforcement placed to resist slab flexural moments or placed as structural integrity reinforcement (as recommended in Section 5.3) may be assumed effective for moment transfer.

The optimum placement of reinforcement for moment transfer has not been clearly established by available experimental data. Current practice (ACI 318) considers reinforcement placed within 1.5 slab thicknesses both sides of the column to be effective in transferring the flexural moment $\gamma_f M_{ub}$ *and observed performance of connections designed by this procedure has generally been acceptable. Whether the reinforcement required for moment transfer is placed totally as top reinforcement, or whether some bottom reinforcement should be used, is less clear and requires judgment on the part of the engineer. As guidance, consider the two extreme cases illustrated in Fig. 5.1.*

In Case A of Fig. 5.1, the connection loading is predominated by a large balanced moment. If a small eccentric loading is introduced, the slab moment increases on one side of the connection and decreases slightly (but still remains negative) on the other side of the connection. In this case, the designer would be prudent to place all the moment transfer reinforcement as top steel.

In the other extreme (Case B of Fig. 5.1), the connection is loaded by a small balanced moment and a large moment transfer due to lateral loads. In this case, the loading results in nearly equal slab moments of opposite sign on opposite sides of the column. Consequently, the total area of reinforcement required by Section 5.1.1(a) for moment transfer should be divided equally between the top and bottom of the slab. Because the loading condition shown in Case B of Fig. 5.1 *normally involves moment reversals, both the top and the bottom reinforcement should be effectively continuous over the column.*

(b) Exterior connections-For resistance to moment transfer parallel to the edge of edge connections, the recommendations of Section 5.1.1(a) for interior connections should be followed.

For resistance to moment transfer perpendicular to the edge, including corner connections, sufficient reinforcement should be placed within a width $2c_t + c_2$, centered on the column, to resist the total moment to be transferred to the column at the centroid of the slab critical section, unless the edge is designed to transfer the torsion due to required slab reinforcement outside this width. The quantity c_t is the distance from the inner face of the column to the slab edge measured perpendicular to the edge, but not to exceed c_1. In cases where the edge is designed for torsion, recommendations of Section 5.1.1(a) for interior connections should be followed.

Experimental results[46,55,56] indicate that slab reinforcement for moment transfer perpendicular to the edge is fully effective in resisting the edge moment only if it is anchored within torsional yield lines projecting from the interior column face to the slab edge (Fig. 5.2). Because of the large twist that occurs in the edge member after torsional yield, reinforcement beyond the projection of the yield line cannot be fully developed until large connection rotations occur. For the typical torsional yield line having a projection of approximately 45 deg, only that reinforcement within the width $2c_t + c_2$ is considered effective, as shown in Fig. 5.2.

If the edge has been designed for torsion, the edge member is likely to possess greater torsional stiffness so that reinforcement beyond the torsional yield line might be effective. In this case, the column strip should be capable of resisting the total moment, and sufficient reinforcement should be placed within the effective width as defined in Section 5.1.1(a). There is some experimental evidence to verify the performance of this type of connection.[16]

5.1.2 At least two of the main top slab bars in each direction and all the structural integrity reinforcement required by Section 5.3 should pass within the column cage. Maximum spacing of slab flexural reinforcement placed in both directions in the connection should not exceed twice the slab thickness.

5.1.3 Continuous bottom slab reinforcement should be provided at the connection in accordance with the following:

(a) Where analysis indicates that positive slab moments develop at the connection, sufficient bottom reinforcement should be provided within the column strip to resist the computed moment.

(b) Where moment transfer alone develops positive slab moments, and the maximum shear stress on the slab critical section due to moment transfer computed in accordance with Section 4.2.1.2(a) exceeds 0.4 V_o/A_{cs}, or when the quantity $5(M_{ub1} + M_{ub2})/b_o V_u$ computed according to Section 4.2.1.2(c) exceeds 0.6, bottom reinforcement should be provided in both directions. The value of $\rho' f_y$ for that reinforcement within lines $2h$ either side of the column in each direction should be not less than 100 psi, where ρ' is the reinforcement ratio of bottom slab reinforcement.

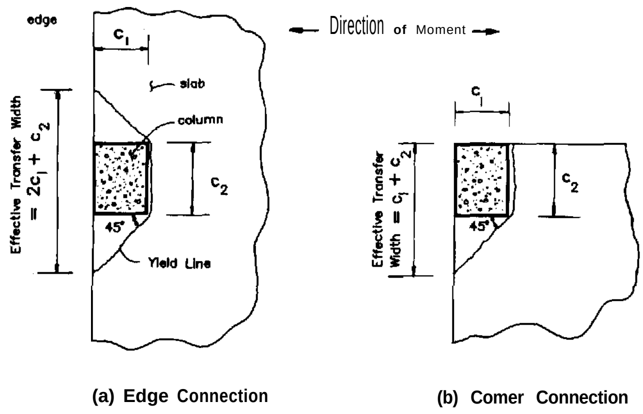


Fig. 5.2-Plan views showing yield lines at edge and corner connections

(c) Structural integrity reinforcement should be provided according to provisions of Section 5.3.

Slab reinforcement is required through the column cage to insure that there is continuity between the slab and column. Minimum reinforcement in the slab surrounding the supporting column is necessary to control cracking. Concentration of reinforcement at the connection delays flexural yield of reinforcement and, thus, enhances shear strength.[49] For exterior slab-column connections in which the slab extends beyond the outer face of the column, the slab overhang should be provided with temperature and shrinkage reinforcement as a minimum.

In designs where lateral loads are of sufficient magnitude that positive slab moments are computed at the column face, reinforcement should be provided in the column strip to resist the computed moments (Case B in Fig. 5.1). This can occur even in buildings with structural wall systems designed to resist the lateral load.

In designs where moment transfer is of lesser magnitude, the total slab moment at the column face may be computed to be negative (Case A in Fig. 5.1). However, it is still possible that positive slab moments will develop near the column,[5] and reinforcement (Section 5.1.3(b)] should be provided to resist this moment.[11] At edge connections where the column is flush with the slab edge and the connection is loaded by an unbalanced moment that produces tension at the top of the slab, the provision of Section 5.1.3(b) does not apply.

The recommendations for continuity and anchorage of bottom reinforcement presented in this and other sections of this document differ from minimum requirements of many codes (e.g., ACI 318). Minimum requirements of these codes are considered to be inadequate for many common design situations.

5.1.4 Where bottom reinforcement is placed to satisfy the recommendations of Section 5.1.3(a) or

5.1.3(b), the sum of the top and bottom reinforcement within the width $c_2 + 3h$ should not exceed three-quarters of the balanced reinforcement computed for the area having total width $c_2 + 3h$ and depth d, unless both the bottom and top flexural reinforcement can be developed within the column.

The upper limit on the sum of continuous top and bottom reinforcement applies for cases where the column dimension is not sufficient to develop the reinforcement, according to Section 5.4.5. In the presence of significant moment transfer at such connections, a bar in tension due to flexural stresses on one face of the column may, because of inadequate anchorage, be in tension also at the opposite face of the column. Thus, both the top and bottom reinforcement may be stressed in tension on a single face. To insure that the extra tensile forces will not result in local crushing of slab concrete, the sum of top and bottom reinforcement ratios should not exceed three-quarters of the balanced ratio.

5.1.5 At discontinuous edges of exterior connections, all top slab reinforcement perpendicular to the edge should be anchored to develop the yield stress at the face of the column, and the edge should be reinforced to satisfy the recommendations of Sections 5.1.5(a) or 5.1.5(b).

(a) A beam should be provided having depth equal to or greater than the slab depth and having longitudinal reinforcement and closed stirrups designed to resist the torsion transmitted from the discontinuous slab edge. The transverse reinforcement should extend a distance not less than four times the slab thickness from both sides of the support and should be spaced at not more than $0.5d_{beam}$, where d_{beam} is the beam effective depth, except it need not be spaced less than 0.75 times the slab effective depth.

(b) An effective beam formed within the slab depth and reinforced by slab reinforcement should be provided. For this effective beam, within a distance not less than two slab thicknesses on both sides of the support, the top reinforcement perpendicular to the edge should be spaced not more than 0.75 times the slab effective depth and should have a 180-deg hook with extension returning along the bottom face of the slab a distance not less than l_d, as defined in Section 5.4.5. In lieu of hooked bars hairpin bars of diameter not less, than that of the top slab bars may be inserted along the edge to overlap the top bars. At least four bars, of diameter not less than the diameter of the main slab bars, should be placed parallel to the discontinuous edge as follows: Two of the bars should be top bars, one along the slab edge and one not less than $0.75\ c_1$ nor more than c_1 from the slab edge. The other two bars should be bottom bars, placed so one bar is directly below each of the two top bars.

At discontinuous edges, the use of spandrel beams is encouraged to insure adequate serviceability and torsional strength. Where spandrel beams are absent, the slab edge should be reinforced to act as a spandrel beam. The recommended slab edge reinforcement is intended to control cracking. It is not intended that the slab edge without spandrel beams be designed for torsion. Additionally, it is noted that the recommended edge reinforcement may be inadequate to act as a diaphragm chord or strut tie. Typical examples of reinforcement at edge connections are shown in Fig. 5.3.

For edge connections without beams, the bars running parallel to the slab edge should be placed (where practicable) within the bars perpendicular to the edge or within the stirrups, if present.

5.2--Recommendations for the joint

5.2.1 *Column longitudinal reinforcement*-Column longitudinal reinforcement passing through the joint should satisfy Sections 10.9.1 and 10.9.2 of ACI 318. Offsets that satisfy requirements of ACI 318 are permitted within the joint.

In addition, the column reinforcement for Type 2 joints should be distributed around the perimeter of the column core. The center-to-center spacing between adjacent longitudinal bars should not exceed the larger of 8 in. or one-third of the column cross-sectional dimension in the direction for which the spacing is being determined.

Researchers have pointed out the need for well-distributed longitudinal reinforcement to confine concrete.[57] *The recommendations for distribution of longitudinal reinforcement for Type 2 connections are intended to insure adequate column ductility by improving column confinement.*

5.2.2 *Transverse reinforcement*

5.2.2.1 *Type 1* connections-Transverse reinforcement is not required for interior connections. For exterior connections, horizontal transverse joint reinforcement should be provided. Within the depth of the slab plus drop panel, the reinforcement should satisfy Section 7.10 of ACI 318, with the following modifications.

(a) At least one layer of transverse reinforcement should be provided between the top and bottom levels of slab longitudinal reinforcement.

(b) If the connection is part of the primary system for resisting nonseismic lateral loads, the center-to-center spacing of the transverse reinforcement should not exceed 8 in.

5.2.2.2 *Type 2 connections*-Column transverse reinforcement above and below the joint should conform to requirements of Appendix A of ACI 318.

For interior connections, transverse reinforcement is not required within the depth of the joint. For exterior connections, as defined in Section 2.2.1, the column transverse reinforcement should be continued through the joint, with at least one layer of transverse reinforcement between the top and bottom slab reinforcement. Maximum spacing of transverse reinforcement within the slab depth should not exceed the smallest of (a) one-half the least column dimension, (b) eight times the smallest longitudinal bar diameter, or (c) 8 in. All hoops should be closed with hooks at their ends of not less than 135 deg. Where required, crossties should be provided at each layer of transverse reinforcement, and

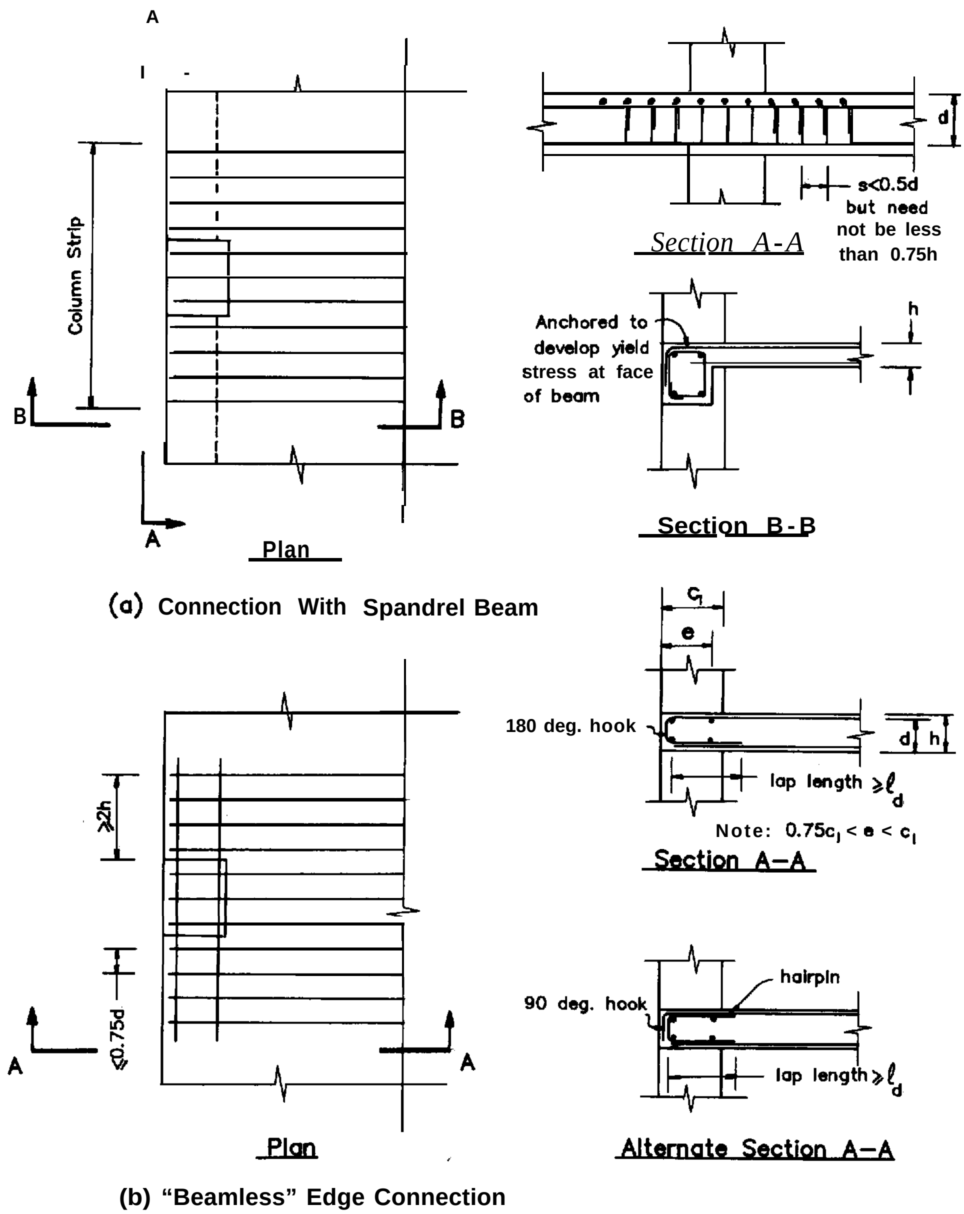


Fig. 5.3-Typical details at discontinuous edges

each end of a crosstie should engage a perimeter longitudinal bar. Single-leg crossties should have a 135 deg or greater bend on one end, and the other end may have a standard 90-deg tie hook as defined in Section 7.1 in ACI 3 18. If 90-deg hooks are used, the hooks should be placed at the interior face of the joint within the slab depth. All 135-deg hooks should have minimum extensions not less than the greater of 6 tie bar diameters and 3 in.

For Type 1 connections, joint confinement by transverse reinforcement is advised for exterior connections where at least one face of the joint is not confined by the slab. Because the joint may be thin in elevation, the requirements of ACI 318 are modified to recommend at least one layer of transverse steel within the joint. An additional requirement is made for the more severe loading case where the slab resists lateral loads.

For Type 2 connections, the recommendations for transverse reinforcement are the same as those given by ACI 318 for columns in frames that are not part of the lateral force resisting system in regions of high seismic risk, and for frames in regions of moderate seismic risk, as appropriate.

For interior connections, adequate confinement is afforded by the slab. Reinforcement above and below the slab should conform to the recommendations.

Within the depth of the joint of exterior connections, column longitudinal bars should be restrained laterally by spirals or by ties as required in Section 7.10.5.3 of ACI 318 and as modified here.

5.3-Structural integrity reinforcement

Reinforcement as specified in 5.3.1 and 5.3.2 should be provided to increase the resistance of the structural system to progressive collapse.

5.3.1 *Connections without beams*-At interior connections, continuous bottom reinforcement passing

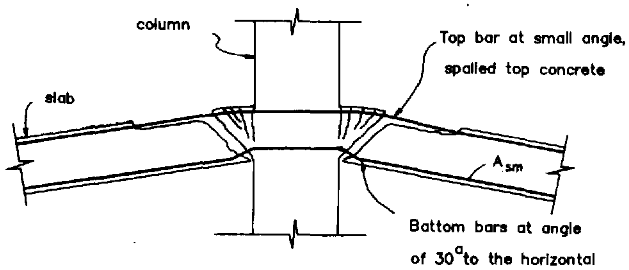


Fig. 5.4-Model of connection during punching failure

within the column cage in each principal direction should have an area at least equal to

$$A_{sm} = \frac{0.5 w_u l_1 l_2}{\phi f_y} \tag{5-1}$$

in which A_{sm} = minimum area of effectively continuous bottom bars or mesh in each principal direction placed over the support, w_u = factored uniformly distributed load, but not less than twice the slab service dead load, l_1 and l_2 = center-to-center span in each principal direction, f_y= yield stress of steel A_{sm}, and ϕ = 0.9. The quantity of reinforcement A_{sm} may be reduced to two thirds of that given by Eq. (5-l) for edge connections, and to one-half of that given by Eq. (5-l) for corner connections. Where the calculated values of A, in a given direction differ for adjacent spans, the larger value should be used at that connection.

Bottom bars having area A, may be considered continuous if (1) they are lap spliced outside a distance $2l_d$ from the column face with a minimum lap splice length equal to l_d; (2) they are lap spliced within the column plan area with a minimum lap splice length of l_d; (3) they are lap spliced immediately outside the column with a minimum lap splice of $2l_d$, provided the lap splice occurs within a region containing top reinforcement; or (4) they are hooked or otherwise anchored at discontinuous edges to develop yield stress at the column face.

Catastrophic progressive collapses have occurred in slab-column structures.1-4 Many of the failures have occurred during construction when young, relatively weak concrete was subjected to heavy construction loads. Procedures for considering the effects of construction loads have been described.[8,27,28]

For Type 1 connections, the minimum bottom reinforcement given by Eq. (5-l) should be continuous over the columns to reduce the likelihood of progressive collapse. Although not presently required by ACI 318, such reinforcement is frequently called out by many design offices.

For Type 2 connections, the design loading conditions may result in general yielding of the top and/or bottom slab reinforcement at the connection. Experimental data[42] *indicate that under such conditions the punching shear strength may be reduced considerably below the nominal value of $4\sqrt{f'_c}A_{cs}$ permitted by ACI 318, thereby reducing the margin of safety against collapse. Thus, minimum continuous bottom reinforcement as specified by Eq. (5-l) is recommended to support the slab in the event of a punching shear failure.*

Eq. (5-l) was developed using the conceptual model of Fig. 5.4. In the model, the slab is supported after punching by bottom reinforcement draped over the support in the two directions. If the bottom reinforcement is considered to assume an angle of 30 deg with respect to the horizontal, reinforcement having an area equal to that given by Eq. (5-l) will be capable of supporting the load w_u within a tributary area equal to $l_1 l_2$. Identical expressions have been obtained by other investigators using different interpretations of the basic mechanism.[58,42] *The adequacy of Eq. (5-l) has been demonstrated by numerous experiments.*[58,42] *The reductions permitted for corner and edge connections result in an area of reinforcement providing the same theoretical resistance as provided for interior connections. For these exterior connections, l_1 and l_2 are intended to be the full span dimensions, not the tributary area dimensions.*

It is noted that only bottom reinforcement is capable of significant post-punching resistance. To perform as intended, the bottom reinforcement must be effectively continuous, and it must be placed directly over the column and within the column cage. As depicted in Fig. 5.4, top reinforcement is less effective than bottom reinforcement because it tends to split the top concrete cover.

The minimum recommended value of w_u equal to twice the slab dead load is based on Reference 8, which indicates that the total load resisted by a connection during construction may be approximately twice the slab dead load. Where detailed calculations and field monitoring of construction loads indicate lower loads, the design may be based on the lower loads.

5.3.2 *Connections with beams*

5.3.2.1 If the beam depth is less than twice the slab depth at the support, the provisions of Section 5.3.1 should be followed in both directions.

5.3.2.2 If the beam depth is at least equal to twice the slab depth, adequate integrity is provided if provisions of ACI 318 are followed for the transverse beams, including minimum embedment of bottom bars in the support.

Progressive collapse has not been a prominent problem in structures having beams between supports. Nonetheless, the value of well anchored bottom bars as provision against collapse should not be overlooked.

5.4-Anchorage of reinforcement

5.4.1 *General recommendations*-Reinforcement should be anchored on each side of the critical section by embedment length or end anchorage. At connections, the critical section for development of reinforcement is at the location of maximum bar stress. At connections in structures having rectangular bays, the critical section may be taken along a line intersecting the joint face and perpendicular to the direction of the mo ment.

5.4.2 *Recommendations for Type I connections*-Reinforcement at connections may be developed by using hooked bars according to Section 5.4.4, by using straight bars passing through the connection according to Section 5.4.5, or by using straight bars terminating at the connection according to Section 5.4.5.

5.4.3 *Recommendations for Type 2 connections*-Reinforcement at connections may be developed by using hooked bars according to Section 5.4.4, except all bars terminating in the joint should be hooked within the transverse reinforcement of the join using a 90-deg hook. Alternately, anchorage may be provided by straight bars passing through the connection according to Section 5.4.6. Straight bars should not be terminated within the region of slab comprising the connection.

5.4.4 *Hooked bars terminating at the connection*-The development length l_{dh} of a bar terminating in a standard hook is

$$l_{dh} = \frac{f_y d_b}{50\sqrt{f'_c}} \tag{5-2}$$

with the following modifications:

(a) The development length should be increased by 30 percent for all-lightweight and sand-lightweight concrete.

(b) If transverse reinforcement in the joint is provided at a spacing less than or equal to three times the diameter of the bar being developed, l_{dh} may be reduced by 20 percent within the joint.

(c) For Type 1 connections, if side cover normal to the plane of the hook is not less than 2½ in., and cover on the bar extension is not less than 2 in., l_{dh} may be reduced by 30 percent.

(d) For Type 1 connections, if reinforcement in excess of that required for strength is provided, l_{dh} may be reduced by the ratio A_s(required)/A_s(provided).

In no case should the length l_{dh} be less than the greater of 6 in. or $8d_b$.

For most Type 1 and all Type 2 exterior connections, bars terminating at a connection will be anchored using a standard hook as defined by ACI 318. The tail extension of the hook should project toward the midheight of the joint. The development length given by Eq. (5-2) is similar to that required by ACI 318 and is evaluated more fully in work done by ACI Committee 408.[59] *The modifications are to be applied concurrently.*

The same length is specified for Type I and Type 2 connections, based on the assumption that the effects of load reversals for Type 2 connections will be offset by more stringent recommendations for joint confinement. These confinement recommendations are equivalent to the benefits from increased concrete cover over the hook; hence, the modification of Section 5.4.4(c) is not applicable to Type 2 connections. In addition, given that yield is generally anticipated in Type 2 connections, the modification of Section 5.4.4(d) is not to be applied for the Type 2 connection.

Where significant strain hardening of reinforcement is anticipated due to inelastic deformations, 1.25 f_y *should be substituted for* f_y *in Eq. (S-2).*

5.4.5 *Straight bars terminating at the connection*-The development length l_d for a straight bar terminating at a Type 1 connection should be computed as

$$l_d = \frac{f_y A_b}{25\sqrt{f'_c}} \geqslant 0.0004 d_b f_y \tag{5-3}$$

provided the bar is contained within the core of the column, with the following modifications:

(a) The length l_d should be increased by 30 percent for bars not terminating within the core of the column. For bars anchored partially within the column core, any portion of the embedment length not within the confined core should be increased by 30 percent.

(b) The length l_d should be increased by 30 percent if the depth of concrete cast in one lift beneath the bar exceeds 12 in.

(c) The length l_d should be multiplied by 1.33 for all-lightweight concrete, or by 1.18 for sand-lightweight concrete.

(d) The length l_d may be reduced for Type 1 connections by multiplying by the factor A_s (required)/ A_s (provided) where reinforcement is provided in excess of that required for strength.

The recommended development length is similar to that required by ACI 318.

Where the bar is not contained within the core of the column, l_d *should be increased as recommended in Reference 59 to account for the greater tendency toward splitting when concrete cover is small.*

For Type 2 connections, straight bars should not terminate in the region of slab comprising the connection.

5.4.6 *Bars passing through the joint*-For Type 2 connections, all straight slab bars passing through the joint should be selected such that

$$h_j/d_b \geqslant 15 \tag{5-4}$$

where h_j is the joint dimension parallel to the bar. No special restrictions are made for column bars or for Type 1 connections.

Straight slab bars are likely to slip within a joint during repeated inelastic lateral load reversals (ACI 352R, Reference 60). In slabs of usual thickness, slip of reinforcement can result in significant reduction of lateral load stiffness.[61] *The purpose of the recommended ratio between bar size and joint dimension is to limit, but not eliminate, slippage of the bars through the connection. The recommended ratio is intended to avoid unusually large diameter slab bars and will not influence proportions in typical designs.*

CHAPTER 6-REFERENCES

6.1-Recommended references

American Concrete Institute

318-83	(1986) Building Code Requirements for Reinforced Concrete
318R-83	Commentary on Building Code Requirements for Reinforced Concrete
423.3R-83	Recommendations for Concrete Members Prestressed with Unbonded Tendons
352 R-85	Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures

American National Standards Institute

ANSI A.58.1-82	Building Code Requirements for Minimum Design Loads in Buildings and Other Structures

International Conference of Building Officials

UBC-1985	Uniform Building Code

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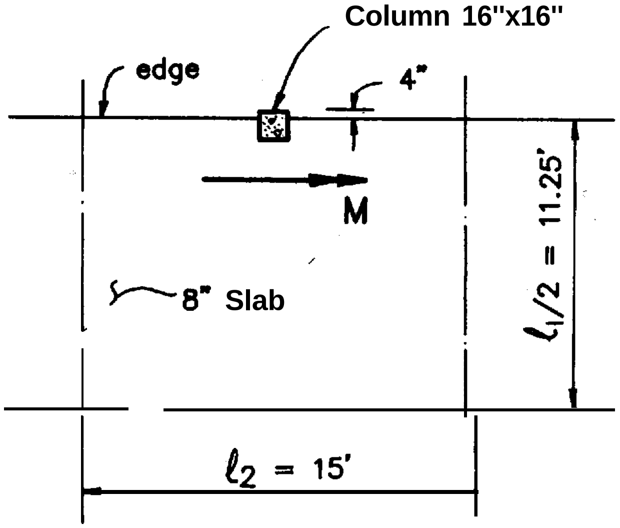
EXAMPLES*

Example 1 — Design of an edge connection subjected to gravity loading



f'_c = 4000 psi
f_y = 60,000 psi

L = 40 psf
D= 115psf

Type 1 connection (2.2.2)

Design forces
U = 1.4D + 1.7L
V_u = 38.6 kips
M_{ub} = 580 kip-in. at centroid of slab critical section

Check shear

Assume #4 bars each way, ¾-in. cover
$d = (7 + 6.5)/2 = 6.75$ in.
$b_o = 16 + 6.75 + 2(12 + 6.75/2) = 53.5$ in. (2.1)
$A_{CS} = b_o d = 53.5 \times 6.75 = 361$ in. (4.2.1.1)
$V_n = V_c = 4 A_{cs} \sqrt{f'_c} = 4 \times 361 \times \sqrt{4000}$
$= 91{,}300$ lb $= 91.3$ kips

$V_o = \phi V_n = 0.85 \times 91.3 = 77.6$ kips
$V_u/V_o = 38.6/77.7 = 0.50 < 0.75$, therefore, OK (4.2.1.2b)

Check moment transfer

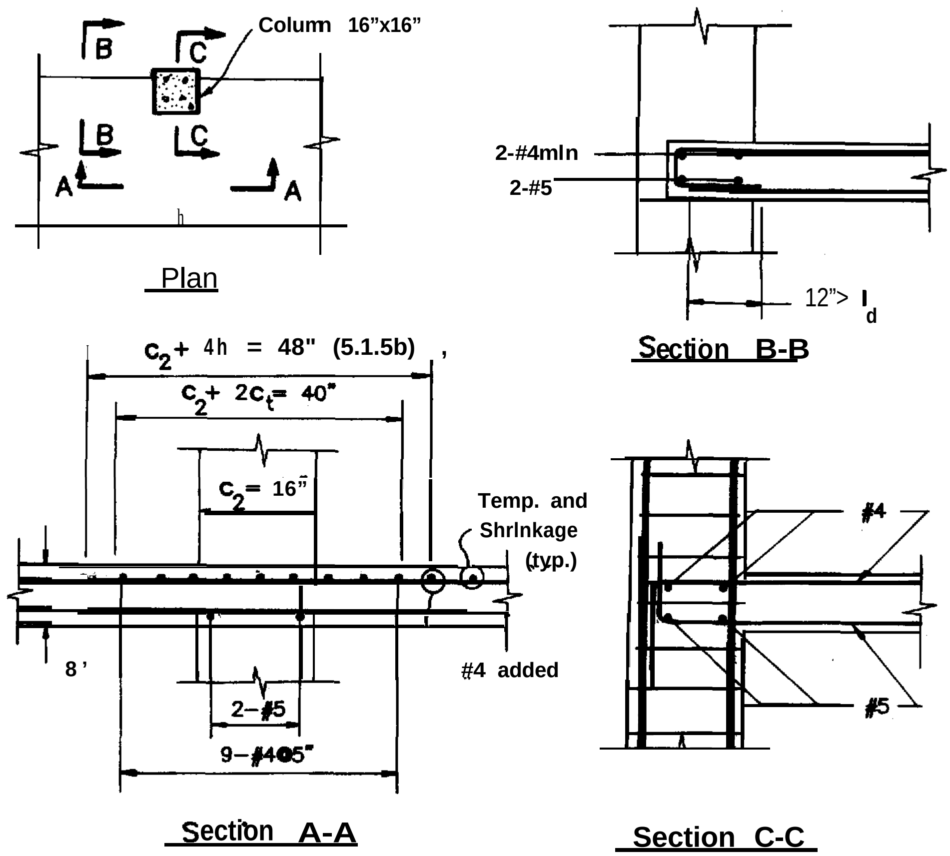
$c_2 + 2c_1 = 16 + 2 \times 12 = 40$ in.
$\phi M_n = \phi \rho b d^2 f_y (1 - 0.59 \rho f_y / f'_c)$ (5.1.lb)
$\geqslant M_u = 580$ kips-in..
which requires $\rho = 0.0058$; $A_s = 1.62$ in.^{2}
Use nine #4 bars

Reinforcement details

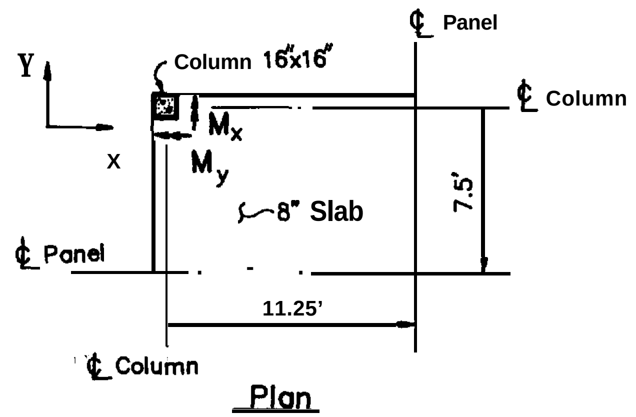
Top reinforcement perpendicular to the slab edge (5.1.5b)
spacing $\leqslant 0.75d = 5.1$ in.
Development length of hooks
$l_{dh} = (f_y d_b)/(50\sqrt{f'_c})$
$= (60{,}000 \times 0.5)/(50\sqrt{4000}) = 9.5$ in. (5.4.4)
Structural integrity reinforcement (5.3.1)
w_u = greater of (1.40 + 1.7L) and (2D) = 0.230 ksf
$A_s = (2/3)(0.5 w_u l_1 l_2)/(\phi f_y)$
$= (2/3)(0.5 \times 0.230 \times 22.5 \times 15)/(0.9 \times 60)$
$= 0.48$ in^2.
Use two #5 bottom bars each way passing through column cage.

*Numbers in parentheses refer to sections of this report.

Final design — Example 1



Example 2 — Design of a corner connection subjected to gravity loading



f'_c = 4000 psi
f_y = 60,000 psi

L = 40psf
D = 115 psf

Type 1 connection (2.2.2)

Design forces

$U = 1.4D + 1.7L$
V_u = 19.3 kip
M_{ubx} = 290 kip-in.
M_{uby} = 190 kip-in.
at centroid of slab critical section

Check shear

Assume #3 bars each way, ¾-in. cover
d = (7.06 + 6.69)/2 = 6.88 in.
b_o = 2(16 + 6.88/2) = 38.9 in. (2.1)
A_{CS} = 38.9 x 6.88 = 268 in. (4.2.1.1)
$V_n = V_c = 4\,A_{CS}\sqrt{f'_c}$ = 4 x 268 x $\sqrt{4000}$ = 67,800 lb = 67.8 kips
$V_o = \phi V_n$ = 0.85 x 67.8 = 57.5 kips
V_u/V_o = 19.3/57.5 = 0.34 < 0.75, therefore, OK (4.2.1.2b)

Check moment transfer

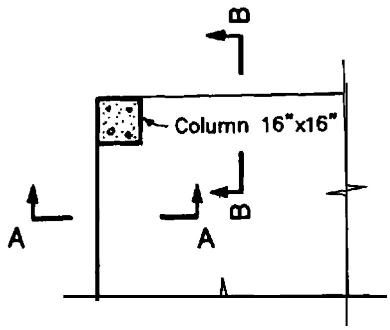
$c_2 + c_t$ = 16 + 16 = 32 in.
$\phi M_{nx} \geqslant M_{ubx}$ = 290 kip-in.
which requires ρ_x = 0.0035; A_s = 0.78 in.^{2} (5.1.1b)
Use eight #3 bars
$\phi M_{ny} \geqslant M_{uby}$ = 190 kip-in.
which requires ρ_y = 0.0025; A_{sy} = 0.54 in.^{2}
Use five #3 bars

Reinforcement details

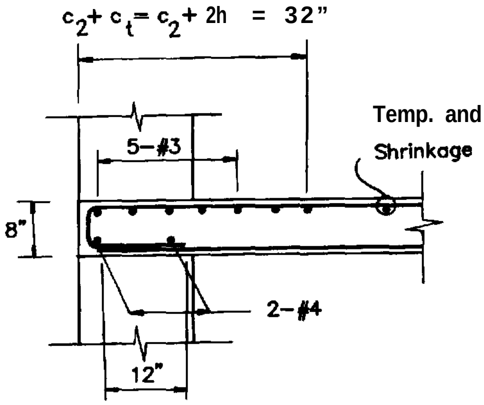
Top reinforcement perpendicular to the slab edge spacing $\leqslant 0.75d$ = 5.1 in. (5.1.5b)

Development length of hooks
$l_{dh} = (f_y d_b)/50\sqrt{f'_c}$ = 7.1 in. (5.4.4)
Structural integrity reinforcement (5.3.1)
w_u = 0.230 ksf as in Example 1
$A_{sm} = (½)(0.5 w_u l_1 l_2)/(\phi f_y)$
= (½)(0.5 x 0.230 x 22.5 x 15)/(0.9 x 60)
= 0.36 in.^{2}
Use two #4 bottom bars each way passing through column cage.

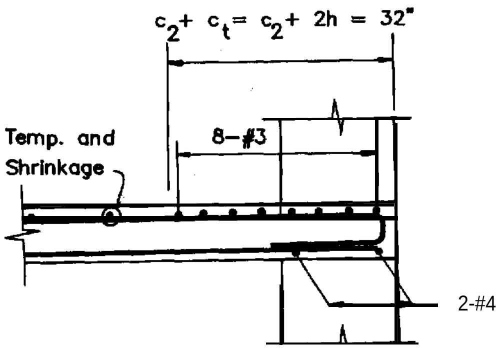
Final design — Example 2



Plan



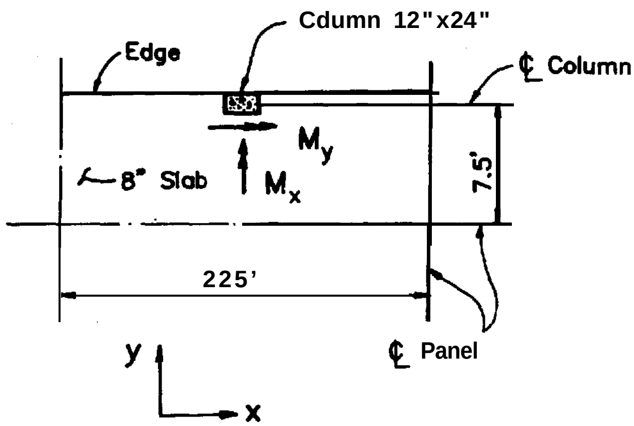
Section A-A



Secti on B-B

Note: #4 bottom bars placed through column for protection against progressive collapse shall have standard hooks (not shown).

Example 3 — Edge connection subjected to biaxial moment due to gravity and wind loading



Load combination	Wind direction	V_u, kips	M_{ux}, kips-in.	M_{uy}, kips-in.	M_{ubx}, kips-in.	M_{uby} kips-in.
(1) $1.4D + 1.7L$	—	40.5	-572	-394	—	-413
(2) $0.75(1.4D + 1.7L + 1.7W)$	x	30.4	- 743	- 296	690	-310
(3) $0.75(1.4D + 1.7L + 1.7W)$	y	34.8	- 429	-610	—	- 623
(4) $0.9D + 1.3W$	x	18.3	+ 59	- 177	703	- 185
(5) $0.9D + 1.3W$	y	22.1	- 257	+144	—	+ 137

Notes: M_{ux} and M_{uy} are flexural moments in the slab column strip.
M_{ubx} and M_{uby} are moments transferred to the connection at the centroid of the slab critical section. For moment in the x-direction, no sign is indicated. For moment in the y-direction, values are positive if the transfer moment tends to place bottom slab steel in tension.
V_u varies depending on whether the wind is considered along the positive or negative direction of the x- or y-axes. Only the larger value for each load combination is tabulated.

f'_c = 4000 psi
f_y = 60,000 psi

L = 40psf
D = 115psf
Type 1 connection (2.2.2)

Design forces
$U = 1.4D + 1.7L$
$\geqslant 0.75(1.4D + 1.7L + 1.7W)$
$\geqslant 0.9 + 1.3W$

Check shear

Basic data
d = 6.75 in.
$b_o = 24 + 6.75 + 2(12 + 6.75/2) = 61.5$ in. (2.1)

$A_{cs} = 61.5 \times 6.75 = 415$ in.^{2}
$V_n = V_c = 4\sqrt{f'_c}\, A_{cs} = 4 \times \sqrt{4000} \times 415 = 105$ kips (4.2.1.1)

$V_o = \phi V_n = 0.85 \times 105 = 89.3$ **kips**

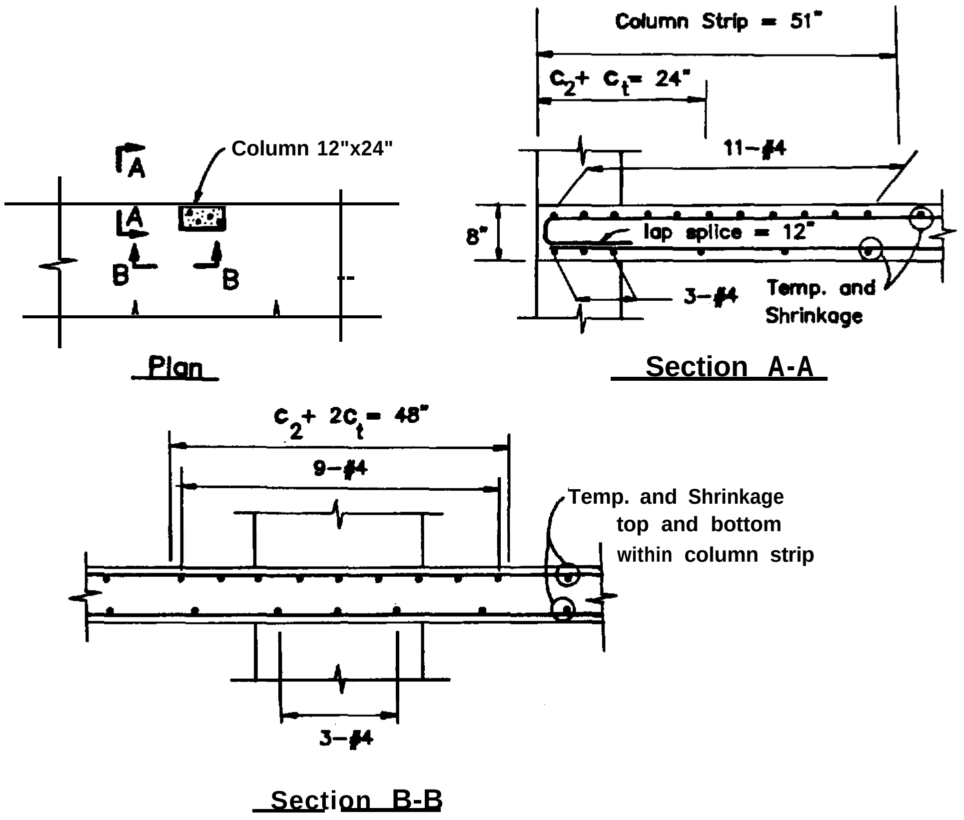
Moment transfer in x-direction:

The maximum transfer moments in the x-direction occur for loading cases (2) and (4). Loading case (4) must be checked because it has the larger moment, and loading case (2) must be checked because it has the larger shear. Both cases involve biaxial moment transfer. Section 4.2.1.2(c) is followed.

For loading case (2)
$V_o \geqslant V_u + a\,(M_{ubx} + M_{uby})/b_o$
$= 30.4 + 3.5(690)/61.5 = 69.7$ kips, OK

For loading case (4)
$V_o \geqslant 18.3 + 3.5(703)/61.5 = 58.3$ kips, OK

Final design-Example 3



Note: M_{uby} is taken equal to zero in the preceding because $V_u/V_o < 0.75$.

Moment transfer in y-direction:

The maximum moment transfer in the y-direction occurs under uniaxial moment transfer. According to Section 4.2.1.2(b), effects of moment transfer on shear are ignored because $V_u/V_o < 0.75$.

Check flexure

Reinforcement in x-direction:

The column strip (51 in.) is designed to carry the total column strip flexural moment M_{ux}, requiring eleven #4 top and #4 at 12 in. bottom (temperature and shrinkage).

For moment transfer

$\gamma_v = 1 - 1/(1 + 0.66\sqrt{\beta_{cr}}) =$ (4.2.1.2a)

$= 1 - 1/(1 + 0.66\sqrt{30.8/15.4}) = 0.49$

$\gamma_f M_{ubx} = (1 - \gamma_v)(703) = 359$ kip-in. (5.1.1b)

Transfer width $= c_2 + 1.5h = 24$ in.

The column strip bars already in place will suffice if distributed uniformly in column strip.

Note: $5M_{ubx}/b_oV_u > 0.6$, therefore place temperature and shrinkage reinforcement at the bottom. (5.1.3b)

Reinforcement in y-direction:

The entire moment M_{uby} is to be resisted in flexure by reinforcement within transfer width equal to (5.1.1b)

$$c_2 + 2c_t = 24 + 2 \times 12 = 48 \text{ in.}$$

For $M_{uby} = -623$ kip-in., provide nine #4 top.

For $M_{uby} = +146$ kip-in., provide #4 @ 12 in, (temperature and shrinkage).

Check spacing $\leqslant 0.75d$, OK (5.1.5b)

Check development, $l_{dh} = 9.5$ in., OK (5.4.4)

Structural integrity reinforcement (5.3.1)

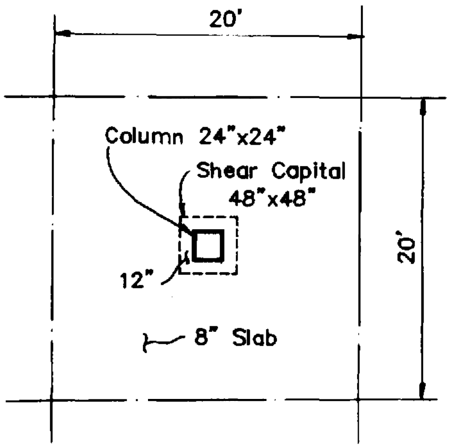
$A_{sm} = (2/3)(0.5 w_u l_1 l_2)/(\phi f_y)$

$= (2/3)(0.5 \times 0.229 \times 22.5 \times 15)/(0.9 \times 60)$

$= 0.48 \text{ in.}^2$

Use three #4 bottom bars each direction through column cage.

Example 4 — Design of an interior connection with shear capital



$f'_c = 4000$ psi

$f_y = 60{,}000$ psi

L = 250 psf

D = 20 psf plus self weight

Type 1 connection (2.2.2)

Design forces

$U = 1.4D + 1.7L$

Slab reinforcement #4 bars each way.

Check shear

(a) Around column

V_u = 233 kips, M_{ub} = 300 k-in,
M_{ub}/V_u = 1.29 in. < 0.2d. therefore, ignore moment transfer (3.2.2)

d = 10.75 in.
b_o = 4 x (10.75 + 24) = 139in.
A_{cs} = 139 x 10.75 = 1490 in.^{2} (4.2.1.1)
$V_n = V_c = 4\sqrt{f'_c}\,A_{cs} = 4 \times 1490 \times \sqrt{4000} = 377$ kips
$V_o = \phi V_n$ = 0.85 x 377 = 320 kips
$V_u < V_o$, therefore, OK (4.2.1.1)

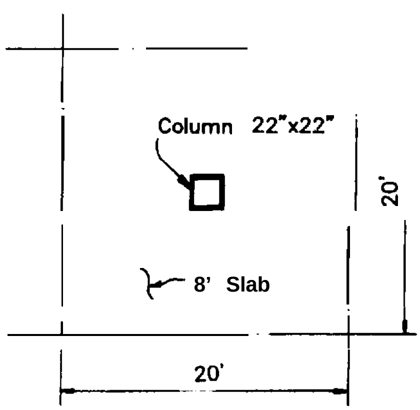
(b) Around shear capital

V_u = 225 kips
d = 6.75 in.
b_o = 4 X (48 + 6.75) = 219 in.
b/d = 219/6.75 = 32.4, therefore, C_v = 0.75 (Table 1)
A_{cs} = 219 x 6.75 = 1480 in.^{2} (4.2.1.1)
$V_n = C_v V_c$ = 0.75 x 4 x 1480 x $\sqrt{4000}$ = 281 kips
$V_o = \phi V_n$ = 0.85 x 281 = 238 kips
$V_u < V_o$, therefore, OK (4.2.1.1)

Reinforcement details

Provide slab flexural steel to resist total slab moments as per AC1 318.
No requirements for moment transfer.
Provide structural integrity reinforcement as per Section 5.3.1 and as illustrated in previous examples.

Example 5 — Design of an interior connection resisting seismic loads



f'_c = 4000 psi
f_y = 60,000 psi

L = 50 psf
D = 115 psf

Type 2 connection (2.2.2)

Design forces
U = 1.4D + 1.7L
$\geqslant$ 0.75(1.4D + 1.7L + 1.87E)
$\geqslant$ 0.9D + 1.43W

Load combination	V_u, kip	M_u, kip-in.	M_{ub}, kip-in.
(1) 1.4D + 1.7L	97	1450	---
(2) 0.75(1.4D + 1.7L + 1.87E)	73	1440	780
(3) 0.9D + 1.43E	41	960	780

Notes: M_u = column strip total moment.
M_{ub} = transfer moment.

Check shear

Assume #4 bars each way, %-in. cover.
d = 6.75 in. (as per Example 1)
b_o = 4 x (22 + 6.75) = 115 in. (2.1)
A_c = 6.75 x 115 = 776 in. (4.2.1.1)

For nonseismic loads
$V_n = V_c$ = 4 $A_c\sqrt{f'_c}$ = 4 X 776 x $\sqrt{4000}$ = 196 kips
$V_o = \phi V_n$ = 0.85 x 196 = I67 kips > 97 kips, OK

For seismic loads
$V_n = C_v V_c$ = 0.75 x 196 = 147 kips
(C_v = 0.75 for seismic loads as per Table 1) (4.2.1.1)
$V_o = \phi V_n$ = 0.85 x 147 = 125 kips

Check moment transfer
$V_u + \alpha(M_{ub1} + M_{ub2})/b_o$ = 73 + 5 x 780/l 15
= 107 kips < V_o, OK (4.2.1.2c)

Check maximum permitted vertical shear
V_{max} = 0.4 V_c = 0.4 x 196 = 78 kips > 73 kips, OK (4.2.1.2d)

Reinforcement requirements

Column strip flexural strength requirements are met by placing 14 #8 bars uniformly across the 10 ft wide column strip.

Moment transfer strength is checked as follows (5.1.la)
γ_f = 1 − γ_v = 0.6
Steel is required within $c_2 + 3h$ = 46 in. to resist flexural moment of value $\gamma_f M_{ub}$ = 0.6 x 780 = 470 kip-in.
Reinforcement placed for total column strip moment (as per previous paragraph) is adequate.

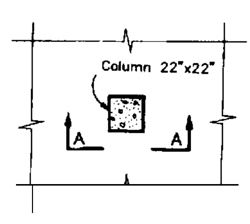
Check if bottom steel is required for moment transfer (5.1.3b)
$5M_{ub1}/b_o V_u$ = 5 x 780/(115 x 41) = 0.83 > 0.6
Therefore minimum reinforcement requirements must be met.
Provide # 4 at 16 in. within width c_2 + 4 h = 54 in., resulting in $\rho' f_y$ = 111 psi > 100 psi, OK.

Structural integrity reinforcement (5.3.1)
A_{sm} = $(0.5 w_u l_1 l_2/(\phi f_y)$
= (0.5 x 0.246 x 20 x 20)/(0.9 x 60) = 0.91 in.'
Use three #5 bottom bars each way passing through column cage.

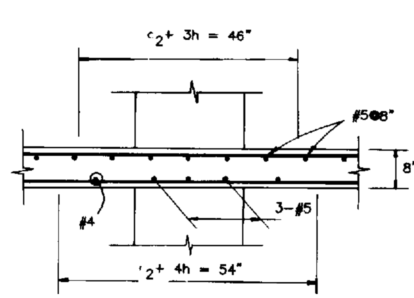
Check maximum reinforcement
Within $c_2 + 3h$ = 46 in., $\rho + \rho' \leqslant 0.75\ \rho_{bal}$, OK (5.1.4)

Check maximum bar spacing (5.1.2)
$S_{max} = 2h$ = 16 in., OK

Final design — Example 5



Plan



Section A-A

NOTATION

A_b = cross-sectional area of reinforcing bar, in.^{2}
A_{cs} = cross-sectional area of the slab critical section, in.^{2}
A_s = total area of steel at a cross section, in.^{2}
A_{sm} = minimum area of effectively continuous bottom slab bars in each principal direction placed over the support for resistance to progressive collapse, in.^{2}
b = beam width, in.
b_o = perimeter of the slab critical section, in.
c_1 = dimension of the column transverse to the direction of moment transferred to the column, in.
c_2 = dimension of the column transverse to the direction of moment transferred to the column, in.
c_t = distance from the inner face of the column to the slab edge measured perpendicular to the edge, but not to exceed c_1
c_v = product of all appropriate modification factors in Table 4.1
d = slab effective depth, taken as the average of the depths from extreme concrete compression fiber to tension steel in two orthogonal directions, in.
d_b = diameter of slab reinforcing bar, in.
d_{beam} = effective depth of transverse beam at connection, in.
f'_c = concrete compression strength, psi
f_y = design yield stress of slab reinforcement, psi
h = slab thickness, in.
h_j = joint dimension in direction parallel to that of a straight slab bar passing through the joint, in.
l_d = development length of straight bar, in.
l_{dh} = development length of hooked bar, in.
l_1, l_2 = center-to-center spans in each principal direction
M_{ub} = moment transferred to the column
M_{ub1}, M_{ub2} = simultaneous moments transferred to the column and acting in the two principal directions about the geometric centroid of the slab critical section
V_c = basic shear strength of concrete without modifications in Table 4.1, lb
V_n = nominal shear strength in the absence of moment transfer, lb
V_o = design shear strength in the absence of moment transfer, lb
V_u = factored direct shear force acting on slab critical section
w_u = factored ultimate load, but not less than twice the slab dead load, to be considered for resistance to progressive collapse
α = coefficient to Eq. (4.4)
β_c = ratio of long to short dimensions of the column cross section
β_{cr} = ratio of lengths of the slab critical section measured parallel and transverse to the direction of moment transfer, respectively
ρ' = steel ratio for bottom slab steel in one direction at the connection
ϕ = strength reduction factor
γ_f = fractin of transfer moment at slab-column connection that is to be carried by slab flexure, same as ACI 3 18 definition of γ_f
γ_v = fraction of transfer moment at slab-column connection that is to be carried by eccentric shear stresses on the slab critical section, same as ACI 318 definition of γ_v

CONVERSION FACTORS

1 in. = 25.4 mm
1 psi = 6895 N/m^2
1 lb = 4.448 N
1 kip-in. = 0.113 kN-m

This report was submitted to letter ballot of the committee and was approved in accordance with ACI balloting procedures.

THE FOLLOWING DISCUSSIONS, WHICH WERE PUBLISHED IN THE JULY-AUGUST 1989 ACI Structural Journal (PP. 496-499), ARE NOT PART OF THE REPORT ACI 352.1 R-89, BUT ARE PROVIDED AS ADDITIONAL INFORMATION TO THE READER.

Recommendations for Design of Slab-Column Connections in Monolithic Reinforced Concrete Structures. Report by ACI-ASCE Committee 352

Discussion by Amin Ghali, B. Vijaya Rangan, and Committee

By AMIN GHALI

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Section 4.2.1.2 of the report recommends three alternate methods for calculating the strength of slab-column connections transferring shearing forces and bending moments. Method (a) is general and applies to any critical section at interior, edge, or corner columns. In this method, a fraction γ_v of the moment is assumed transferred by shear stress, which varies linearly about the centroid of the slab critical section. Method (c) uses Eq. (4-4) which "emulates, in algebraic form, the eccentric shear model" adopted in Method (a). Thus, it can be expected that Method (a) and Eq. (4-4) give the same result. Method (b) ignores the moment transfer in corner and edge connections and considers that they have adequate strength when the shear stress caused by V_u does not exceed 75 percent of V_o/A_{cs}.

The assumption involved in Method (a) leads to the following equation for the shear stress at any point on the critical section

$$v = \frac{V}{A} + \frac{M_x}{I_x} y + \frac{M_y}{I_y} x \qquad (4\text{-}5)$$

where V, M_x, and M_y are the shear force and the moments about centroidal principal axes x and y of the critical section; A, I_x, and I_y are the area and second moments of area about the same axes.

The positive directions of the coordinates x and y and the forces V, M_x, and M_y are indicated in Fig. A. The arrows represent a force and moments exerted by the column on the critical section. Equal and opposite force and moments representing the effect of the critical section on the column* exist but are not shown in Fig. A.

The symbols M_x and M_y represent the fraction of the moments transferred by shear; that is γ_v multiplied by the moment transferred between column and slab.

When using Eq. (4-5), it should be noted that x and y are the critical section centroidal principal axes, which are not necessarily parallel to the slab edges or to the principal axes of the column cross section. This will be the case for the critical section at a corner column or at any column adjacent to nonsymmetrical openings.

The basic mechanics Eq. (4-5) is derived from the assumption of linear variation of v over the critical section and the conditions that v has stress resultants equal

*The double-headed arrows shown on the plans of the slabs in Examples 2 and 3 of the report do not indicate the moment directions unless a mention is made that the arrows represent the action of the column on the critical section or the effect of critical section on the column.

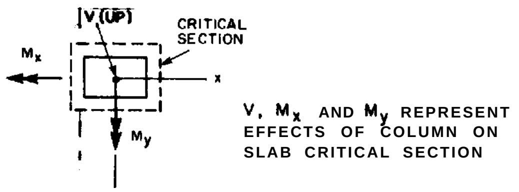

Fig. A — Positive directions of coordinates x and y and of V, $\mathbf{M_x}$, *and* $\mathbf{M_y}$

to V, M_x, and M_y. Eq. (4-5) differs from the equation in Section 11.12.2.4 of the ACI 318-83 Commentary[62] in that the critical section property J_c, which the Commentary describes as "analogous to polar moment of inertia," is here replaced by the second moment of Area I. The reasons for the change are given in Reference 63, where it is shown that the Commentary equation gives erroneous results when x and y are not centroidal principal axes, and when they are, the equation gives slightly smaller stresses than the stresses by Eq. (4-5). For the remainder of the present discussion, Eq. (4-5) will be considered applicable in Method (a) of Section 4.2.1.2. of the Committee 352 report.

Eq. (4.4) of the report implies that the maximum shear stress at the critical section can be determined by

$$v_{u\,max} = \frac{V_u}{A} + \frac{\alpha}{A b_o}(M_{ub1} + M_{ub2}) \qquad (4\text{-}6)$$

where b_o is the perimiter of the critical section, and M_{ub1} and M_{ub2} are the factored moments transferred to the column about the centroidal principal directions at the centroid of the critical section. The values of α recommended are 5 for interior and 3.5 for edge connections. No value is given for corner connections, which probably means that Eq. (4-6) does not apply in this case. In fact, when Eq. (4-6) is used for a corner column, with α = 3.5, it gives a substantially different result from Eq. (4-5).

The Committee report states that Eq. (4-6) does not apply when the long-to-short cross-sectional dimensions of the column are greater than two. There are several other cases not mentioned in the report for which Eq. (4-6) cannot possibly give the correct maximum shear stress because the equation does not include the necessary parameters. Examples of such cases are: columns with nonrectangular cross-sections, nonsymmetrical critical sections due to the presence of openings, and edge connections with slab overhang.

Eq. (4-5) is basic and general and does not need to be simplified by Eq. (4-6), which has so many limitations.

Method (b) is based on tests on edge connections that have indicated that the slab strength in transfer of moment perpendicular to slab edge is not influenced significantly by the shearing force. This phenomenon can mean that the fraction of moment transferred by shear is smaller for exterior columns than for interior columns. Thus, in Method (a), different values of the coefficient γ_v should be used for interior, edge, and

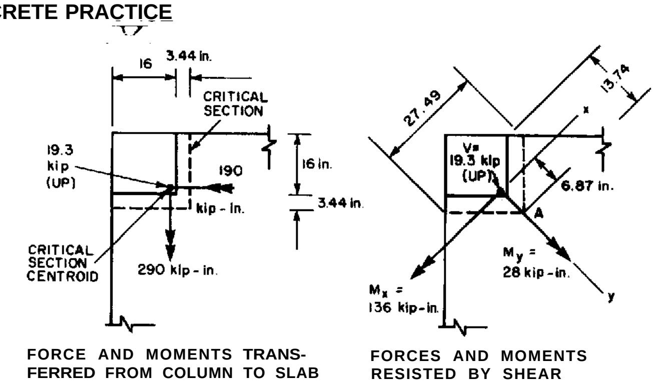

Fig. B — Top views of a corner-column connection example

corner columns. It is also expected that, for edge and corner columns, the coefficient γ_v becomes zero when the critical section is sufficiently far from the column faces. However, research is needed before an adjustment of γ_v can be made.

Method (b) allows substantially higher force and moment transfer compared to Method (a), as will be shown below by a numerical example of a corner column connection. Method (b) can lead to unsafe design because it extends the results of a test series of edge connections to corner connections without sufficient experimental evidence.

EXAMPLE

A corner column of cross section 16 x 16 in.² is connected to an 8-in. slab with d = 6.88 in. The factored force and moments transferred from the column to the slab are indicated in Fig. B.* It is required to determine, using Method (a), a multiplier η_a which, when applied to the transferred force and moment, will make the connection just safe. Repeat the design using Method (b) to determine a corresponding multiplier η_b. Assume that normal weight concrete having f'_c = 4000 psi is used, that flexural yielding in the slab is not anticipated, and that the connection is of Type 1.

Method (a) — The principal axes of the slab critical section are inclined 45 deg to the slab edges. The properties of the critical section are

$$A = 2(6.875)(19.44) = 267 \text{ in.}^2 \quad I_x = \frac{A}{12}(13.74)^2 = 4208 \text{ in.}^4$$

$$I_y = \frac{A}{12}(27.49)^2 = 16{,}800 \text{ in.}^4$$

The transferred moments are multiplied by γ_v = 0.4 and replaced by equivalent components in the principal x- and y-directions. The shear in the critical section are to be determined for V = 19.3 kips; M_x = 136 kips-in., M_y = 28 kips-in.

*The data for this example are the same as for Example 2 of the Committee report. with the exception of the directions of the transferred moments. Here the directions of the transferred moments are chosen such that they produce tensile stress in the top slab fiber in directions perpendicular to the inner faces of the columns. This represents the common case in practice where the moments are caused by gravity forces on the slab.

The maximum shear stress occurs at Point A, whose x and y coordinates are (0 and 6.87). Maximum shear stress by Eq. (4-5)

$$v_{max} = \frac{19300}{267} + \frac{136000}{4208}(6.87) + \frac{28000}{16800}(0)$$
$$= 72 + 222 + 0 = 294 \text{ psi}$$

None of the modification factors of Table 4.1 apply; thus C_v = 1.0. The connection will be just safe when η_a multiplied by v_{max} is equal to $V_o/A_{cs} = \phi(4\sqrt{f'_c})$

$$294\ \eta_a = 0.85\ (4\sqrt{4000});\ \text{thus}\ \eta_a = 0.73$$

Method (b) — According to this method, the connection will be just safe when η_b multiplied by 72 psi, which is the shear stress due to the a vertical force, is equal to 0.75 $\phi(4\sqrt{f'_c})$. This gives

$$72\ \eta_b = 0.75(0.85)\ (4\sqrt{4000});\ \text{thus}\ \eta_b = 2.24$$

From the example just given it can be seen that Method (b) considers the connection to be safe under load more than three times the load allowed by Method (a). Method (b) can hardly be considered an alternate to Method (a).

CONCLUSION

In view of the preceding example, it is suggested that only Method (a) be retained, with the maximum stress calculated by Eq. (4-5). A mention may be made that the value of γ_v can be smaller than the value given by Eq. (4-3) when the connection is of an exterior column.

REFERENCE

62. ACI Committee 318, "Commentary on Building Code Requirements for Reinforced Concrete (ACI 318R-83)." American Concrete Institute, Detroit, 1983, 155 pp., and 1986 Supplement.

63. Ghali, Amin, Discussion of Section 11.12.6.2 of "Proposed Revision to: Building Code Requirements for Reinforced Concrete (ACI 318-83) (Revised 1986)," reported by ACI Committee 318, *ACI Structural Journal*, V. 86, No. 3, May-June 1989. p. 329.

By B. VIJAYA RANGAN

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The members of Committee 352 should be congratulated for their report. This discussion deals mainly with Sections 4.2.1.2(b), and Example 1. The design method described in these sections of the report is based on the work of Professor Moehle.[46] I am concerned that this method would lead to overconservative designs in practice. The following points support my concern:

1. I have reworked Example 1 using ACI 318-83. According to the ACI Building Code method, the moment transferred by direct flexure is 375 kips-in., and therefore 580 – 375 = 205 kips-in. is transferred as torsion. The combined maximum shear stress due direct shear and moment transfer is found to be 211 psi, which is less than $4\phi\sqrt{f'_c} = 4 \times 0.85\sqrt{4000} = 215$ psi. The shear strength of the slab is therefore adequate. To transfer a moment of 375 kips-in. by direct flexure, adequate area steel must be provided in the vicinity of the column over a width of $c_2 + 3h = 16 + (3 \times 8) = 40$ in. This requires ρ = 0.0039; A, = 1.05 in. which should be compared with ρ = 0.0058; A, = 1.62 in. given in Example 1. In other words, the proposed method requires over 50 percent more steel than that needed by the ACI Code method within the same slab width of 40 in. The ACI method has been in use for more than 20 years, and I am not aware of any evidence showing that it is not adequate. With the advent of microcomputers and programmable calculators, very little effort is required to check a slab for adequate shear strength using the ACI method. For these reasons, I fail to see the necessity for the proposed method that would lead to overconservative designs. Also, the supporting data for limiting the spacing of bars to a maximum of 0.75d is no t given in the report.

2. The overconservative nature of the proposed method is further supported by the results obtained from a slab specimen tested recently at the University of New South Wales. The test specimen is similar to the one I have tested earlier,[16] except that there are no closed ties in the slab at the edge.

The test specimen is a half-scale model of an edge connection with the following details: slab thickness = 100 mm (3.94 in.), d = 82 mm (3.23 in.), c, = 250 mm (9.84 in.), c_2 = 200 mm (7.87 in.), f'_c = 48.3 MPa (7004 psi), and slab steel perpendicular to the free edge consisted of 6.3 mm (0.25 in.) diameter bars at spacings of 100 mm (3.94 in.) at the top and 115 mm (4.53 in.) at the bottom. In addition, two 8 mm (0.31 in.) diameter bars and two 6.3 mm (0.25 in.) diameter bars were also placed at the top within the column width. The yield strength of 6.3 mm bar is 460 MPa (66.7 ksi) and that of 8 mm bar is 535 MPa (77.6 ksi). The specimen failed in punching shear and the measured forces at failure are V_u = 108.2 kN (24.4 kips) and M_{ub} = 27.9 kNm (247 kips-in.). For this specimen, b_o = 864 mm (34.0 in.), $V_o = 864 \times 82 \times 0.34\sqrt{48.3}$ = 167.4 kN (37.7 kips) and $V_u/V_o = 108.2/167.4 = 0.65 < 0.75$. According to the proposed method, therefore, the strength of this edge connection is given by the moment transfer strength of the slab flexural steel within the width of $c_2 + 2c_t = 200 + (2 \times 250) = 700$ mm (27.6 in.), which is found to be 14.0 kNm (124 kips-in.). The ratio of test strength/predicted strength = 27.9/14.0 ≈ 2.0.

I have calculated the strength of this test specimen using the ACI Building Code method. The predicted moment transfer strength is 20 kNm (177 kips-in.) and therefore test value/calculated value = 27.9/20.0 = 1.40.

I have also calculated the strength of this connection using the simple formula given in the Australian Standard.[64] The predicted shear strength is 77.3 kN (17.4 kips) and the test/calculated ratio is 108.2/77.3 = 1.40.

REFERENCE

64. "Australian Standard for Concrete Structures," (AS 3600-1988), Standards Association of Australia, North Sydney, Mar. 1988, 108 pp.

COMMITTEE CLOSURE

The committee thanks Professors Ghali and Rangan for their discussions of the recommendations. The Committee will consider seriously the points made in the discussions in its future deliberations. Response to their comments follows.

The committee agrees with Dr. Ghali that the three methods of Section 4.2.1.2 for determining connection shear and moment transfer strength do not produce identical results. Method (a) of that section is the familiar shear and moment transfer method of the ACI Building Code. Because this method has been successful for design for many years, the committee did not attempt to modify this method. The committee cannot comment on the modifications to this method that were proposed by Dr. Ghali. Those modifications and their bases were submitted as discussion to the Committee 318 proposed revisions and are not available to the committee.

Method (b) of Section 4.2.1.2 is not intended to produce designs that are exactly the same as those produced by Method (a); it is an alternative that has been found to match experimental data better than does Method (a). Recent comparisons with experimental data[65] indicate that Method (b) is applicable to corner connections. Dr. Ghali suggests in his example that the shear stress for a corner connection should be calculated about an axis 45 deg relative to the column principal axes. The committee will consider this recommendation further. However, the committee notes that connections tested in the laboratory typically display yield lines across the slab prior to punching of the connection.

The committee thanks Dr. Ghali for pointing out that Method (c) of Section 4.2.1.2 is applicable only to rectangular interior connections and rectangular exterior transferring moment parallel to the edge. For columns having other cross sections, Method (a) of Section 4.2.1.2 should be used.

The committee agrees with Dr. Rangan that the recommendations may result in more reinforcement near exterior columns than is required by the ACI Building Code. The concentration of reinforcement is recommended to improve performance of the connection. As reinforcement is spread over wider distances, the connection becomes more flexible, and, especially under lateral loading, may not be able to develop its strength within reasonable deformation limits. Further, it should be noted that the recommendations, although requiring concentration of reinforcement near the exterior column, do not require use of more reinforcement in total. According to either the ACI Building Code or the Committee 352 recommendations, the total quantity of reinforcement at the edge is determined by the total slab moment at the edge.

The additional experimental data provided by Dr. Rangan are welcome and will be studied further. As noted in the background to the recommendations[66] the eccentric shear stress model of Section 4.2.1.2(a) of the recommendations usually is more conservative than is the model of Section 4.2.1.2(b).

The committee agrees that computers and calculators facilitate design but disagrees that such equipment obviates simplified techniques. When simplified techniques provide insight into the proportioning and detailing process, or simply aid conceptual design, they become tools as valuable as the computer or calculator that are purported to replace them.

REFERENCES

65. Hwang, S-J., "An Experimental Study of Flat-Plate Structures Under Vertical and Lateral Loads," graduate thesis, University of California, Berkeley, Jan. 1989, 271 pp.

66. Moehle, Jack P.; Kreger, Michael E.; and Leon, Roberto, "Background to Recommendations for Design of Reinforced Concrete Slab-Column Connections," *ACI Structural Journal*, V. *85, No. 6,* Nov.-Dec. 1988, pp. 636-644.