# CHAPTER 16—CONNECTIONS BETWEEN MEMBERS

## CODE

### 16.1—Scope

**16.1.1** This chapter shall apply to the design of joints and connections at the intersection of concrete members and for load transfer between concrete surfaces, including (a) through (d):

(a) Connections of precast members
(b) Connections between foundations and either cast-in-place or precast members
(c) Horizontal shear strength of composite concrete flexural members
(d) Brackets and corbels

### 16.2—Connections of precast members

**16.2.1** *General*

**16.2.1.1** Transfer of forces by means of grouted joints, shear keys, bearing, anchors, mechanical connectors, steel reinforcement, reinforced topping, or a combination of these, shall be permitted.

**16.2.1.2** Adequacy of connections shall be verified by analysis or test.

**16.2.1.3** Connection details that rely solely on friction caused by gravity loads shall not be permitted.

**16.2.1.4** Connections, and regions of members adjacent to connections, shall be designed to resist forces and accommodate deformations due to all load effects in the precast structural system.

**16.2.1.5** Design of connections shall consider structural effects of restraint of volume change in accordance with 5.3.6.

**16.2.1.6** Design of connections shall consider the effects of tolerances specified for fabrication and erection of precast members.

## COMMENTARY

### R16.2—Connections of precast members

**R16.2.1** *General*

Connection details should be arranged to minimize the potential for cracking due to restrained creep, shrinkage, and temperature movements. The Precast/Prestressed Concrete Institute (MNL 123) provides information on recommended connection details for precast concrete structures.

**R16.2.1.1** If two or more connection methods are used to satisfy the requirements for force transfer, their individual load-deformation characteristics should be considered to confirm that the mechanisms work together as intended.

**R16.2.1.4** The structural behavior of precast members may differ substantially from that of similar members that are cast-in-place. Design of connections to minimize or transmit forces due to shrinkage, creep, temperature change, elastic deformation, differential settlement, wind, and earthquake require particular consideration in precast construction.

**R16.2.1.5** Connections should be designed to either permit the displacements or resist the forces induced by lack of fit, volume changes caused by shrinkage, creep, thermal, and other environmental effects. Connections intended to resist the forces should do so without loss of strength. Restraint assumptions should be consistent in all interconnected members. There are also cases in which the intended force may be in one direction, but it may affect the strength of the connection in another. For example, shrinkage-induced longitudinal tension in a precast beam may affect the vertical shear strength on the corbel supporting it.

**R16.2.1.6** Refer to R26.9.1(a).

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**16.2.1.7** Design of a connection with multiple components shall consider the differences in stiffness, strength, and ductility of the components.

**16.2.1.8** Integrity ties shall be provided in the vertical, longitudinal, and transverse directions and around the perimeter of a structure in accordance with 16.2.4 or 16.2.5.

### 16.2.2 *Required strength*

**16.2.2.1** Required strength of connections and adjacent regions shall be calculated in accordance with the factored load combinations in Chapter 5.

**16.2.2.2** Required strength of connections and adjacent regions shall be calculated in accordance with the analysis procedures in Chapter 6.

**16.2.2.3** For bearing connections, $N_{uc}$ shall be (a) or (b), but need not exceed $N_{uc,max}$, where $N_{uc,max}$ is the maximum restraint force that can be transmitted through the load path of a bearing connection multiplied by the load factor used for live loads in combinations with other factored load effects.

(a) For connections not on bearing pads, $N_{uc}$ shall be calculated simultaneously with $V_u$ using factored load combinations in accordance with 5.3.6. The restraint force shall be treated as a live load.
(b) For connections on bearing pads, $N_{uc}$ shall be 20% of the sustained unfactored vertical reaction multiplied by a load factor of 1.6.

**16.2.2.4** If the friction coefficient for a bearing material has been determined by results of tests, $N_{uc}$ shall be permitted to be determined by multiplying the sustained unfactored vertical reaction by the friction coefficient and a load factor of 1.6.

## COMMENTARY

**R16.2.1.8** Appendix B of the *PCI Design Handbook* (PCI MNL 120) provides a review of structural integrity and minimum integrity ties for precast concrete bearing wall structures.

### R16.2.2 *Required strength*

**R16.2.2.3** Bearing connections subjected to sustained loads will experience volume change restraint forces due to the effects of creep, shrinkage, and temperature change. Sustained loads are dead loads and any other permanent loads such as soil loads or equipment loads that may be included with live loads. Section 5.3.6 prescribes the general consideration for restraint of volume change and differential settlement in combination with other loading but does not define a specific load factor for precast concrete bearing conditions. Load factors are provided with these provisions. $N_{uc}$ provides a capacity-design limit.

For mechanical connections, steel-to-steel contact, or other high-friction bearings, the horizontal force is usually due to volume change restraint. Such bearing connections will experience volume change restraint forces due to the effects of creep, shrinkage, and temperature change. Because the magnitude of volume change restraint forces acting on bearing connections cannot usually be determined with a high degree of accuracy, it is required to treat the restraint force $N_{uc}$ as a live load in 16.2.2.3(a) when using the factored load combinations of 5.3.6 and multiplied by 1.6 in 16.2.2.3(b).

Common precast concrete bearing connections use elastomeric pads or other structural bearing media that limit transferred forces by pad deformation or slip. The limiting load of such connections can be taken as 20% of the sustained unfactored reaction, as recognized by 16.2.2.3(b).

**R16.2.2.4** Bearings explicitly designed for low friction, such as polytetrafluoroethylene (PTFE)-faced sliding bearings, may reduce volume change restraint forces. If the friction coefficient has been reliably determined for a bearing material considering service conditions such as temperature, aging, and exposure, that information can be used to calculate the maximum restraint force.

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**16.2.3** *Design strength*

**16.2.3.1** For each applicable load combination, design strengths of precast member connections shall satisfy

$$\phi S_n \geq U \tag{16.2.3.1}$$

**16.2.3.2** $\phi$ shall be determined in accordance with 21.2.

**16.2.3.3** At the contact surface between supported and supporting members, or between a supported or supporting member and an intermediate bearing element, nominal bearing strength for concrete surfaces, $B_n$, shall be calculated in accordance with 22.8. $B_n$ shall be the lesser of the nominal concrete bearing strengths for the supported or supporting member surface, and shall not exceed the strength of intermediate bearing elements, if present.

**16.2.3.4** If shear is the primary result of imposed loading and shear transfer occurs across a given plane, it shall be permitted to calculate $V_n$ in accordance with the shear-friction provisions in 22.9.

**16.2.4** *Minimum connection strength and integrity tie requirements*

**16.2.4.1** Except where the provisions of 16.2.5 govern, longitudinal and transverse integrity ties shall connect precast members to a lateral-force-resisting system, and vertical integrity ties shall be provided in accordance with 16.2.4.3 to connect adjacent floor and roof levels.

**16.2.4.2** Where precast members form floor or roof diaphragms, the connections between the diaphragm and those members being laterally supported by the diaphragm shall have a nominal tensile strength of not less than 300 lb per linear ft.

**16.2.4.3** Vertical integrity ties shall be provided at horizontal joints between all vertical precast structural members, except cladding, and shall satisfy (a) or (b):

(a) Connections between precast columns shall have vertical integrity ties, with a nominal tensile strength of at least $200A_g$ lb, where $A_g$ is the gross area of the column. For columns with a larger cross section than required by consideration of loading, a reduced effective area based on the cross section required shall be permitted. The reduced effective area shall be at least one-half the gross area of the column.

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**R16.2.4** *Minimum connection strength and integrity tie requirements*

**R16.2.4.1** It is not intended that these minimum requirements supersede other applicable provisions of the Code for design of precast concrete structures.

The overall integrity of a structure can be substantially enhanced by minor changes in the amount, location, and detailing of member reinforcement and in the detailing of connection hardware. The integrity ties should constitute a complete load path, and load transfers along that load path should be as direct as possible. Eccentricity of the load path, especially within any connection, should be minimized.

**R16.2.4.2** The connection between the diaphragm and the member laterally supported by the diaphragm may be direct or indirect. For example, a column may be connected directly to the diaphragm, or it may be connected to a spandrel beam, which is connected to the diaphragm.

**R16.2.4.3** Base connections and connections at horizontal joints in precast columns and wall panels, including structural walls, are designed to transfer all design forces and moments. The minimum integrity tie requirements of this provision are not additive to these design requirements. Common practice is to place the wall integrity ties symmetrically about the vertical centerline of the wall panel and within the outer quarters of the panel width, wherever possible.

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(b) Connections between precast wall panels shall have at least two vertical integrity ties, with a nominal tensile strength of at least 10,000 lb per tie.

**16.2.5** *Integrity tie requirements for precast concrete bearing wall structures three stories or more in height*

## COMMENTARY

**R16.2.5** *Integrity tie requirements for precast concrete bearing wall structures three stories or more in height*

Section 16.2.4 gives requirements for integrity ties that apply to all precast concrete structures. The specific requirements in this section apply only to precast concrete bearing wall structures with three or more stories, often called large panel structures. If the requirements of this section conflict with the requirements of 16.2.4, the requirements in this section control.

These minimum provisions for structural integrity ties in large panel bearing wall structures are intended to provide an alternate load path in case of loss of a bearing wall support (Portland Cement Association 1980). Tie requirements calculated for specific load effects may exceed these minimum provisions. The minimum integrity tie requirements are illustrated in Fig. R16.2.5, and are based on PCI's recommendations for design of precast concrete bearing wall buildings (PCI Committee on Precast Concrete Bearing Wall Buildings 1976). Integrity tie strength is based on yield strength. Appendix B of the *PCI Design Handbook* (PCI MNL 120) provides a review of structural integrity and minimum integrity ties for precast concrete bearing wall structures.

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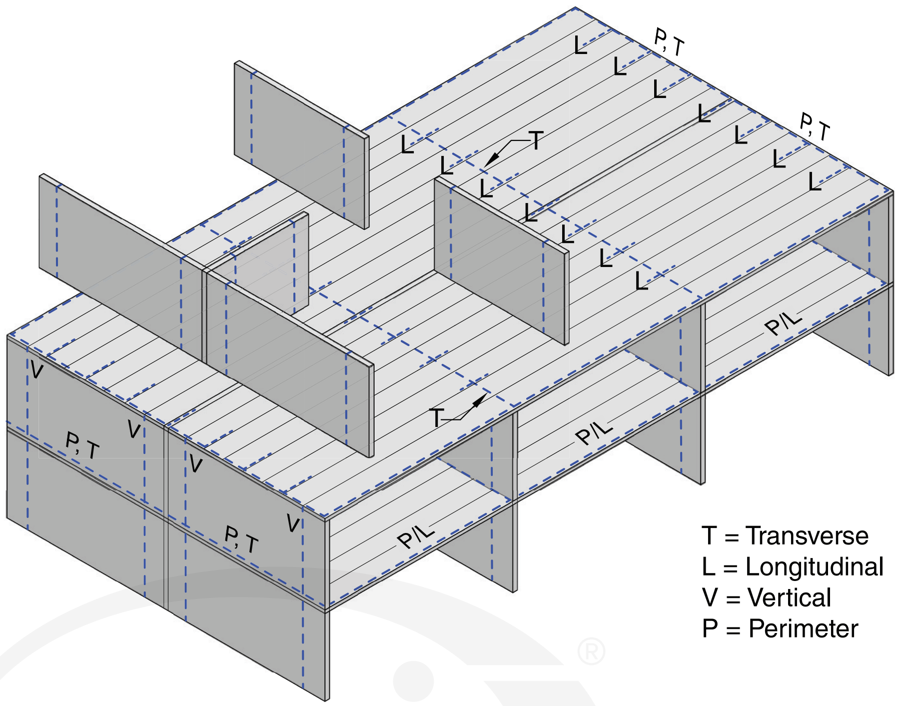


*Fig. R16.2.5—Typical arrangement of integrity ties in large panel structures.*

**16.2.5.1** Integrity ties in floor and roof systems shall satisfy (a) through (f):

(a) Longitudinal and transverse integrity ties shall be provided in floor and roof systems to provide a nominal tensile strength of at least 1500 lb per foot of width or length.

(b) Longitudinal and transverse integrity ties shall be provided over interior wall supports and between the floor or roof system and exterior walls.

(c) Longitudinal and transverse integrity ties shall be positioned in or within 2 ft of the plane of the floor or roof system.

(d) Longitudinal integrity ties shall be oriented parallel to floor or roof slab spans and shall be spaced not greater than 10 ft on center. Provisions shall be made to transfer forces around openings.

(e) Transverse integrity ties shall be oriented perpendicular to floor or roof slab spans and shall be spaced not greater than the bearing wall spacing.

(f) Integrity ties at the perimeter of each floor and roof, within 4 ft of the edge, shall provide a nominal tensile strength of at least 16,000 lb.

**R16.2.5.1**

(a) Longitudinal integrity ties may project from slabs and be lap spliced, welded, mechanically connected, or embedded in grout joints with sufficient length and cover to develop the required force. Bond length for non-tensioned prestressing reinforcement, if used, should be sufficient to develop the yield strength (Salmons and McCrate 1977; PCA 1980).

(c) It is not uncommon to have integrity ties positioned in the walls reasonably close to the plane of the floor or roof system.

(e) Transverse integrity ties may be uniformly spaced and either encased in the panels or in a topping, or they may be concentrated at the transverse bearing walls.

(f) The perimeter integrity tie requirements need not be additive with the longitudinal and transverse integrity tie requirements.

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**16.2.5.2** Vertical integrity ties shall satisfy (a) through (c):

(a) Integrity ties shall be provided in all wall panels and shall be continuous over the height of the building.
(b) Integrity ties shall provide a nominal tensile strength of at least 3000 lb per horizontal foot of wall.
(c) At least two integrity ties shall be provided in each wall panel.

### 16.2.6 *Minimum dimensions at bearing connections*

**16.2.6.1** Dimensions of bearing connections shall satisfy 16.2.6.2 or 16.2.6.3 unless shown by analysis or test that lesser dimensions will not impair performance.

**16.2.6.2** For precast slabs, beams, or stemmed members, minimum design dimensions from the face of support to end of precast member in the direction of the span, considering specified tolerances, shall be in accordance with Table 16.2.6.2.

**Table 16.2.6.2—Minimum design dimensions from face of support to end of precast member**

| Member type | Minimum distance, in. | |
|---|---|---|
| Solid or hollow-core slab | Greater of: | $\ell_n/180$ |
| | | 2 |
| Beam or stemmed member | Greater of: | $\ell_n/180$ |
| | | 3 |

**16.2.6.3** Bearing pads adjacent to unarmored faces shall be set back from the face of the support and the end of the supported member a distance not less than 0.5 in. or the chamfer dimension at a chamfered face.

## 16.3—Connections to foundations

### 16.3.1 *General*

**16.3.1.1** Factored forces and moments at base of columns, walls, or pedestals shall be transferred to supporting foundations by bearing on concrete and by reinforcement, dowels, anchor bolts, or mechanical connectors.

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### R16.2.6 *Minimum dimensions at bearing connections*

This section differentiates between bearing length and length of the end of a precast member over the support (refer to Fig. R16.2.6).

Bearing pads distribute concentrated loads and reactions over the bearing area, and allow limited horizontal and rotational movements for stress relief. To prevent spalling under heavily loaded bearing areas, bearing pads should not extend to the edge of the support unless the edge is armored. Edges can be armored with anchored steel plates or angles. Section 16.5 gives requirements for bearing on brackets or corbels.

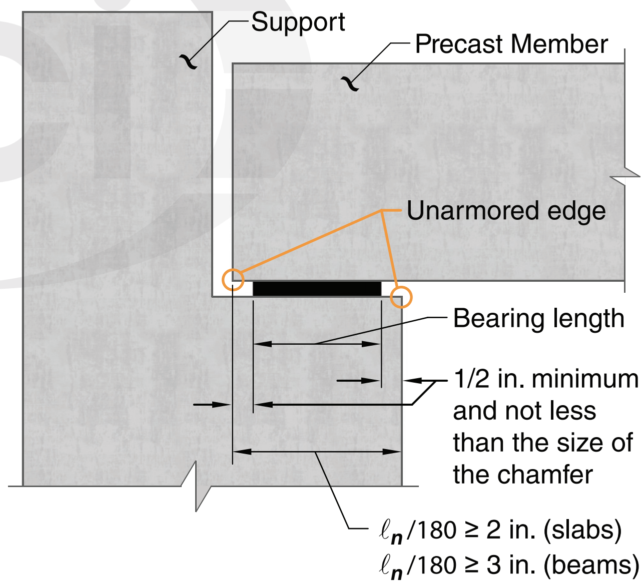


*Fig. R16.2.6—Bearing length on support.*

## R16.3—Connections to foundations

**R16.3.1** The requirements of 16.3.1 through 16.3.3 apply to both cast-in-place and precast construction. Additional requirements for cast-in-place construction are given in 16.3.4 and 16.3.5, while additional requirements for precast construction are given in 16.3.6.

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**16.3.1.2** Reinforcement, dowels, or mechanical connectors between a supported member and foundation shall be designed to transfer (a) and (b):

(a) Compressive forces that exceed the lesser of the concrete bearing strengths of either the supported member or the foundation, calculated in accordance with 22.8
(b) Any calculated tensile force across the interface

**16.3.1.3** At the base of a composite column with a structural steel core, (a) or (b) shall be satisfied:

(a) Base of structural steel section shall be designed to transfer the total factored forces from the entire composite member to the foundation.
(b) Base of structural steel section shall be designed to transfer the factored forces from the steel core only, and the remainder of the total factored forces shall be transferred to the foundation by compression in the concrete and by reinforcement.

### 16.3.2 *Required strength*

**16.3.2.1** Factored forces and moments transferred to foundations shall be calculated in accordance with the factored load combinations in Chapter 5 and analysis procedures in Chapter 6.

### 16.3.3 *Design strength*

**16.3.3.1** Design strengths of connections between columns, walls, or pedestals and foundations shall satisfy Eq. (16.3.3.1) for each applicable load combination. For connections between precast members and foundations, requirements for vertical integrity ties in 16.2.4.3 or 16.2.5.2 shall be satisfied.

$$\phi S_n \geq U \qquad (16.3.3.1)$$

where $S_n$ is the nominal flexural, shear, axial, torsional, or bearing strength of the connection.

**16.3.3.2** $\phi$ shall be determined in accordance with 21.2.

**16.3.3.3** Combined moment and axial strength of connections shall be calculated in accordance with 22.4.

**16.3.3.4** At the contact surface between a supported member and foundation, or between a supported member or foundation and an intermediate bearing element, nominal bearing strength $B_n$ shall be calculated in accordance with 22.8 for concrete surfaces. $B_n$ shall be the lesser of the nominal concrete bearing strengths for the supported member or foundation surface, and shall not exceed the strength of intermediate bearing elements, if present.

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### R16.3.3 *Design strength*

**R16.3.3.4** In the common case of a column bearing on a footing, where the area of the footing is larger than the area of the column, the bearing strength should be checked at the base of the column and the top of the footing. In the absence of dowels or column reinforcement that continue into the foundation, the strength of the lower part of the column should be checked using the strength of the concrete alone.

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**16.3.3.5** At the contact surface between supported member and foundation, $V_n$ shall be calculated in accordance with the shear-friction provisions in 22.9 or by other appropriate means.

**16.3.3.6** At the base of a precast column, pedestal, or wall, anchor bolts and anchors for mechanical connections shall be designed in accordance with Chapter 17. Forces developed during erection shall be considered.

**16.3.3.7** At the base of a precast column, pedestal, or wall, mechanical connectors shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete.

**16.3.4** *Minimum reinforcement for connections between cast-in-place members and foundation*

**16.3.4.1** For connections between a cast-in-place column or pedestal and foundation, $A_s$ crossing the interface shall be at least $0.005A_g$, where $A_g$ is the gross area of the supported member.

**16.3.4.2** For connections between a cast-in-place wall and foundation, area of vertical reinforcement crossing the interface shall satisfy 11.6.1.

**16.3.5** *Details for connections between cast-in-place members and foundation*

**16.3.5.1** At the base of a cast-in-place column, pedestal, or wall, reinforcement required to satisfy 16.3.3 and 16.3.4 shall be provided either by extending longitudinal bars into the supporting foundation or by dowels. The reinforcement extended into the supporting foundation shall be developed in accordance with 25.4.

**16.3.5.2** Where continuity is required, splices and mechanical connectors for the longitudinal reinforcement or dowels shall satisfy 10.7.5 and, if applicable, 18.13.2.2.

**16.3.5.3** If a pinned or rocker connection is used at the base of a cast-in-place column or pedestal, the connection to foundation shall satisfy 16.3.3.

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**R16.3.3.5** Shear-friction may be used to check for transfer of lateral forces to the supporting pedestal or footing. As an alternative to using shear-friction across a shear plane, shear keys may be used, provided that the reinforcement crossing the joint satisfies 16.3.4.1 for cast-in-place construction or 16.3.6.1 for precast construction. In precast construction, resistance to lateral forces may be provided by mechanical or welded connections.

**R16.3.3.6** Chapter 17 covers anchor design, including seismic design requirements. In precast concrete construction, erection considerations may control base connection design and need to be considered.

**R16.3.4** *Minimum reinforcement for connections between cast-in-place members and foundation*

The Code requires a minimum amount of reinforcement between all supported and supporting members to ensure ductile behavior. This reinforcement is required to provide a degree of structural integrity during the construction stage and during the life of the structure.

**R16.3.4.1** The Code does not require that all bars in a column be extended through and be anchored into a footing. However, reinforcement with an area of 0.005 times the column area or an equal area of properly spliced dowels is required to extend into the footing with proper anchorage.

**R16.3.5** *Details for connections between cast-in-place members and foundation*

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**16.3.5.4** At footings, compression lap splices of No. 14 and No. 18 bars that are in compression for all factored load combinations shall be permitted in accordance with 25.5.5.3.

COMMENTARY

**R16.3.5.4** Satisfying 16.3.3.1 might require that each No. 14 or 18 bar be spliced in compression to more than one No. 11 or smaller dowel bar.

**16.3.6** *Details for connections between precast members and foundation*

**16.3.6.1** At the base of a precast column, pedestal, or wall, the connection to the foundation shall satisfy 16.2.4.3 or 16.2.5.2.

**16.3.6.2** If the applicable load combinations of 16.3.3 result in no tension at the base of precast walls, vertical integrity ties required by 16.2.4.3(b) shall be permitted to be developed into an adequately reinforced concrete slab-on-ground.

## 16.4—Horizontal shear transfer in composite concrete flexural members

**16.4.1** *General*

## R16.4—Horizontal shear transfer in composite concrete flexural members

**R16.4.1** *General*

**16.4.1.1** In a composite concrete flexural member, full transfer of horizontal shear forces shall be provided at contact surfaces of interconnected elements.

**R16.4.1.1** Full transfer of horizontal shear forces between segments of composite concrete members can be provided by horizontal shear strength at contact surfaces through interface shear, properly anchored ties, or both.

**16.4.1.2** Where tension exists across any contact surface between interconnected concrete elements, horizontal shear transfer by contact shall be permitted only where transverse reinforcement is provided in accordance with 16.4.6 and 16.4.7.

**16.4.1.3** Surface preparation assumed for design shall be specified in the construction documents.

**R16.4.1.3** Section 26.5.6 requires the licensed design professional to specify the surface preparation in the construction documents.

**16.4.2** *Required strength*

**16.4.2.1** Except as permitted by 16.4.5.3, factored forces transferred along the contact surface in composite concrete flexural members shall be calculated in accordance with the factored load combinations in Chapter 5.

**16.4.2.2** Required strength shall be calculated in accordance with the analysis procedures in Chapter 6.

**16.4.3** *Design strength*

**16.4.3.1** Design strength for horizontal shear transfer shall satisfy Eq. (16.4.3.1) at all locations along the contact surface in a composite concrete flexural member, unless 16.4.5 is satisfied:

$$\phi V_{nh} \geq V_u \qquad (16.4.3.1)$$

where nominal horizontal shear strength $V_{nh}$ is calculated in accordance with 16.4.4.

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**16.4.3.2** $\phi$ shall be determined in accordance with 21.2.

### 16.4.4 *Nominal horizontal shear strength*

**16.4.4.1** Nominal horizontal shear strength $V_{nh}$ shall be calculated in accordance with one of the shear transfer mechanisms in Table 16.4.4.1.

## COMMENTARY

### R16.4.4 *Nominal horizontal shear strength*

**R16.4.4.1** Rows (a) and (c) of Table 16.4.4.1 correspond to the transverse shear force associated with the nominal horizontal shear stress capacity assuming the horizontal shear stress at the contact surface in a cracked flexural member is equal to $V_u/b_vd$ (Park and Paulay 1975). This allows the comparison with the factored transverse shear $V_u$ in Eq. (16.4.3.1). $V_n$ calculated in accordance with rows (b) and (d) is intended to be taken as $V_{nh}$ for use in Eq. (16.4.3.1). The permitted horizontal shear strength in (a) of Table 16.4.4.1 is appropriate for bonded topping slabs. The 80 psi shear stress limit is based on tests by Mones and Brena (2013). To promote bond, a surface that is at least lightly textured is specified, but surface texture alone does not ensure adequate bond. Proven mixture design, placement and curing practices are needed to avoid debonding and curling of topping slabs. If the horizontal shear stress exceeds $\phi$80 psi, the required shear transfer mechanism is shear-friction across a cracked interface.

The permitted horizontal shear strengths and the requirement of 1/4 in. amplitude for intentional roughness are based on tests discussed in Kaar et al. (1960), Saemann and Washa (1964), and Hanson (1960).

**Table 16.4.4.1—Nominal horizontal shear strength**

<table>
<tr><th>Shear transfer mechanism</th><th>Contact surface preparation</th><th>$A_{vf,min}$</th><th colspan="2">$V_{nh}$</th><th></th></tr>
<tr><td>Cementitious bond</td><td>Concrete placed against hardened concrete that is clean, free of laitance, and at least lightly textured[1]</td><td>$A_{vf}$ not required</td><td colspan="2">$80b_vd$</td><td>(a)</td></tr>
<tr><td rowspan="3">Shear-friction</td><td>Concrete placed against hardened concrete that is clean and free of laitance</td><td rowspan="3">In accordance with 16.4.6.1</td><td colspan="2">Determined in accordance with 22.9 assuming a horizontal shear plane over a distance of $d$ where $A_c = b_vd$</td><td>(b)</td></tr>
<tr><td rowspan="2">Concrete placed against hardened concrete that is clean, free of laitance, and intentionally roughened to a trough-to-peak amplitude of approximately 1/4 in.[2]</td><td rowspan="2">Either</td><td>$\lambda\left(260 + 0.6\frac{A_{vf}f_{yt}}{b_vs}\right)b_vd \leq 500b_vd$</td><td>(c)</td></tr>
<tr><td>Determined in accordance with 22.9 assuming a horizontal shear plane over a distance of $d$ where $A_c = b_vd$</td><td>(d)</td></tr>
</table>

[1]Refer to 26.5.6.2(f).

[2]Refer to 26.5.6.2(e) for compliance requirements for intentional roughening.

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**16.4.4.2** Transverse reinforcement in the previously cast concrete that extends into the cast-in-place concrete and is detailed to develop $f_y$ in tension on both sides of the interface shall be permitted to be included as $A_{vf}$.

**16.4.4.3** In Table 16.4.4.1, *d* shall be the distance from extreme compression fiber for the entire composite concrete section to the centroid of prestressed and nonprestressed longitudinal tension reinforcement, if any, but need not be taken less than 0.80*h* for prestressed concrete members.

**16.4.5** *Alternative method for calculating design horizontal shear strength*

**16.4.5.1** As an alternative to 16.4.3.1, it shall be permitted to provide for horizontal shear transfer between points of zero and maximum moment in accordance with 16.4.5.2 through 16.4.5.5.

**16.4.5.2** Horizontal shear shall be transferred by shear-friction and shall satisfy $\phi V_{nh} \geq V_{uh}$.

**16.4.5.3** Factored horizontal shear $V_{uh}$ applied over width $b_v$ and length $\ell_{vh}$ shall be taken as the lesser of (a) and (b):

(a) Force corresponding to nominal compressive strength of the concrete and longitudinal reinforcement on the compression side of the interface. The nominal compressive strength shall be calculated in accordance with Eq. (22.4.2.2) using the effective compression flange width defined in 6.3.2.

(b) Force in prestressed and nonprestressed reinforcement on the tension side of the interface at nominal flexural strength, $A_s f_y + A_{ps} f_{ps}$.

**16.4.5.4** Horizontal shear strength $V_{nh}$ shall be calculated in accordance with (b), (c) or (d) of Table 16.4.4.1 with $b_v \ell_{vh}$ substituted for $b_v d$.

**16.4.5.5** $A_{vf}$ shall be detailed such that $A_{vf}$ per unit length is proportional to the vertical shear force in the member.

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**R16.4.4.3** In composite prestressed concrete members, the depth of the tension reinforcement may vary along the member. The definition of *d* used in Chapter 22 for determining the vertical shear strength is also appropriate for determining the horizontal shear strength.

**R16.4.5** *Alternative method for calculating design horizontal shear strength*

The alternative method differs from the method in 16.4.3 in two ways: 1) the alternative method is based on development of forces corresponding to development of nominal flexural strength, and 2) the strength over the entire distance between points of zero and maximum moment is considered, although distribution of reinforcement in proportion to shear force is required. This method is often used for simply supported precast beams intended to act compositely with topping slabs. An example is provided in the *PCI Design Handbook* (PCI MNL 120). The alternative method is not considered suitable for overlays relying on cementitious bond because redistribution of horizontal shear after interface separation is not possible.

**R16.4.5.5** Detailing of shear-friction reinforcement in proportion to shear force is required because horizontal

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shear stresses along the contact surface in a composite member will reflect the distribution of vertical shear along the member. This requirement is usually satisfied by specifying zones of reinforcement within which groups of bars are consistently spaced. Horizontal shear failure is most likely to initiate where the horizontal shear stress is a maximum and will spread to regions of lower stress. Because the slip at peak horizontal shear resistance is small for a concrete-to-concrete contact surface, longitudinal redistribution of horizontal shear resistance is very limited. Therefore, the spacing of ties along the contact surface should provide horizontal shear resistance distributed approximately the same as the distribution of shear stress along the contact surface.

**16.4.6** *Minimum reinforcement for horizontal shear transfer*

**16.4.6.1** Where shear transfer reinforcement is designed to resist horizontal shear, $A_{vf,min}$ shall be $50(b_{vs}/f_y)$.

**R16.4.6** *Minimum reinforcement for horizontal shear transfer*

**R16.4.6.1** The requirements for minimum area of shear transfer reinforcement are based on test data given in Kaar et al. (1960), Saemann and Washa (1964), Hanson (1960), Grossfield and Birnstiel (1962), and Mast (1968). Beginning with the 2025 Code, minimum reinforcement for horizontal shear is independent of minimum vertical shear reinforcement and is based on the width of the interface through which the reinforcement extends, $b_v$, which may exceed the web width, $b_w$.

**16.4.7** *Reinforcement detailing for horizontal shear transfer*

**16.4.7.1** Shear transfer reinforcement shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire reinforcement.

**16.4.7.2** Where shear transfer reinforcement is designed to resist horizontal shear, longitudinal spacing of shear transfer reinforcement shall not exceed the lesser of 24 in. and four times the least dimension of the supported element.

**16.4.7.3** Shear transfer reinforcement shall be developed in interconnected elements in accordance with 25.7.1.

**R16.4.7** *Reinforcement detailing for horizontal shear transfer*

**R16.4.7.3** Proper anchorage of ties extending across the interface is required to maintain contact along the interface.

## 16.5—Brackets and corbels

**16.5.1** *General*

## R16.5—Brackets and corbels

**R16.5.1** *General*

Brackets and corbels are short cantilevers that tend to act as simple trusses or deep beams, rather than beams, which are designed for shear according to 22.5. The corbel shown in Fig. R16.5.1a and Fig. 16.5.1b may fail by shearing along the interface between the column and the corbel, yielding of the tension tie, crushing or splitting of the compression strut, or localized bearing or shearing failure under the loading plate. These failure modes are illustrated and discussed in Elzanaty et al. (1986).

The method of design addressed in this section has only been validated experimentally for $a_v/d \leq 1.0$. In addition, an upper limit is provided for $N_{uc}$ because this method of design has only been validated experimentally for $N_{uc} \leq V_u$.

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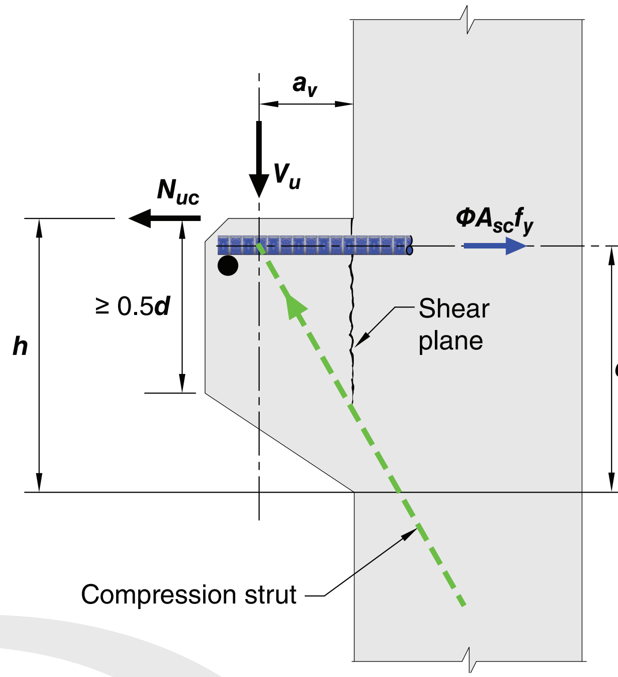


*Fig. R16.5.1a—Structural action of a corbel.*

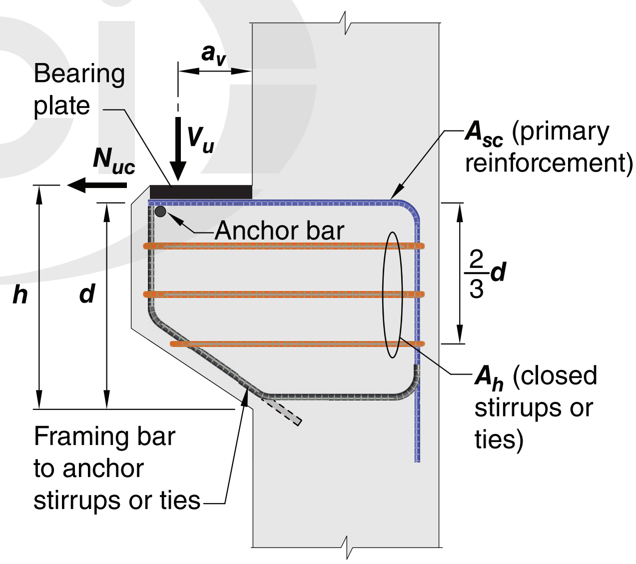


*Fig. R16.5.1b—Notation used in Section 16.5.*

**16.5.1.1** Brackets and corbels with shear span-to-depth ratio $a_v/d \leq 1.0$ and with factored restraint force $N_{uc} \leq V_u$ shall be permitted to be designed in accordance with 16.5.

**R16.5.1.1** Design of brackets and corbels in accordance with Chapter 23 is permitted, regardless of shear span.

**16.5.2** *Dimensional limits*

**R16.5.2** *Dimensional limits*

**16.5.2.1** Effective depth $d$ for a bracket or corbel shall be calculated at the face of the support.

**16.5.2.2** Overall depth of bracket or corbel at the outside edge of the bearing area shall be at least 0.5$d$.

**R16.5.2.2** A minimum depth, as shown in Fig. R16.5.1a and R16.5.1b, is required at the outside edge of the bearing area so that a premature failure will not occur due to a major crack propagating from below the bearing area to the sloping face of the corbel or bracket. Failures of this type have been observed (Kriz and Raths 1965) in corbels having depths at the outside edge of the bearing area less than required in 16.5.2.2.

**16.5.2.3** No part of the bearing area on a bracket or corbel shall project farther from the face of support than (a) or (b):

(a) End of the straight portion of the primary tension reinforcement
(b) Interior face of the transverse anchor bar, if one is provided

**R16.5.2.3** The restriction on the location of the bearing area is necessary to ensure development of the specified yield strength of the primary tension reinforcement near the load.

If the corbel is designed to resist restraint force $N_{uc}$, a bearing plate should be provided and anchored to the primary tension reinforcement (Fig. R16.5.1b).

**16.5.2.4** For normalweight concrete, the bracket or corbel dimensions shall be selected such that $V_u/\phi$ shall not exceed the least of (a) through (c):

(a) $0.2f_c'b_wd$
(b) $(480 + 0.08f_c')b_wd$
(c) $1600b_wd$

**R16.5.2.4** These limits impose dimensional restrictions on brackets and corbels necessary to comply with the maximum shear friction strength allowed on the critical section at the face of support.

**16.5.2.5** For lightweight concrete, the bracket or corbel dimensions shall be selected such that $V_u/\phi$ shall not exceed the lesser of (a) and (b):

(a) $\left(0.2 - 0.07\frac{a_v}{d}\right)f_c'b_wd$

(b) $\left(800 - 280\frac{a_v}{d}\right)b_wd$

**R16.5.2.5** Tests (Mattock et al. 1976a) have shown that the maximum shear friction strength of lightweight concrete brackets and corbels is a function of both $f_c'$ and $a_v/d$.

**16.5.3** *Required strength*

**R16.5.3** *Required strength*

**16.5.3.1** The section at the face of the support shall be designed to resist simultaneously the factored shear $V_u$, the factored restraint force $N_{uc}$, and the factored moment $M_u$.

**R16.5.3.1** Figure R16.5.1b shows the forces applied to the corbel. $M_u$ can be calculated as $[V_ua_v + N_{uc}(h - d)]$.

**16.5.3.2** Factored restraint force, $N_{uc}$, and shear, $V_u$, shall be the maximum values calculated in accordance with the factored load combinations in Chapter 5. It shall be permitted to calculate $N_{uc}$ in accordance with 16.2.2.3 or 16.2.2.4, as appropriate.

**R16.5.3.2** In editions of the Code prior to ACI 318-19, specific provisions for restraint forces at bearing connections were included only for corbels and brackets. In 2019, 16.2.2.3 and 16.2.2.4 were added to include consideration of restraint forces at all bearing connections. Consequently the provisions applicable only to brackets or corbels were removed and a reference made to 16.2.2.3 or 16.2.2.4.

**16.5.3.3** Required strength shall be calculated in accordance with the analysis procedures in Chapter 6, and the requirements in this section.

**16.5.4** *Design strength*

**16.5.4.1** Design strength at all sections shall satisfy $\phi S_n \geq U$, including (a) through (c). Interaction between load effects shall be considered.

(a) $\phi N_n \geq N_{uc}$
(b) $\phi V_n \geq V_u$
(c) $\phi M_n \geq M_u$

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**16.5.4.2** $\phi$ shall be determined in accordance with 21.2.

**16.5.4.3** Nominal tensile strength $N_n$ provided by $A_n$ shall be calculated by

$$N_n = A_n f_y \tag{16.5.4.3}$$

**16.5.4.4** Nominal shear strength $V_n$ provided by $A_{vf}$ shall be calculated in accordance with provisions for shear-friction in 22.9, where $A_{vf}$ is the area of reinforcement that crosses the assumed shear plane.

**16.5.4.5** Nominal flexural strength $M_n$ provided by $A_f$ shall be calculated in accordance with the design assumptions in 22.2.

### 16.5.5 *Reinforcement limits*

**16.5.5.1** Area of primary tension reinforcement, $A_{sc}$, shall be at least the greatest of (a) through (c):

(a) $A_f + A_n$
(b) $(2/3)A_{vf} + A_n$
(c) $0.04(f_c'/f_y)(b_w d)$

**16.5.5.2** Total area of closed stirrups or ties parallel to primary tension reinforcement, $A_h$, shall be at least:

$$A_h = 0.5(A_{sc} - A_n) \tag{16.5.5.2}$$

### 16.5.6 *Reinforcement detailing*

**16.5.6.1** Concrete cover shall be in accordance with 20.5.1.3.

**16.5.6.2** Minimum spacing for deformed reinforcement shall be in accordance with 25.2.

## COMMENTARY

### R16.5.5 *Reinforcement limits*

**R16.5.5.1** Test results (Mattock et al. 1976a) indicate that the total amount of primary tension reinforcement, $A_{sc}$, required to cross the face of the support should be the greatest of:

(a) The sum of the amount of reinforcement needed to resist demands from flexure, $A_f$, plus the amount of reinforcement needed to resist the axial force, $A_n$, as determined by 16.5.4.3.
(b) The sum of two-thirds of the total required shear-friction reinforcement, $A_{vf}$, as determined by 16.5.4.4, plus the amount of reinforcement needed to resist the axial force, $A_n$, determined by 16.5.4.3. The remaining $A_{vf}/3$ should be provided as closed stirrups parallel to $A_{sc}$ as required by 16.5.5.2.
(c) A minimum amount of reinforcement, multiplied by the ratio of concrete strength to steel strength. This amount is required to prevent the possibility of sudden failure should the bracket or corbel crack under the action of flexure and outward tensile force.

**R16.5.5.2** Closed stirrups parallel to the primary tension reinforcement are necessary to prevent a premature diagonal tension failure of the corbel or bracket. Distribution of $A_h$ is required to be in accordance with 16.5.6.6. The total amount of reinforcement required to cross the face of the support, as shown in Fig. R16.5.1b, is the sum of $A_{sc}$ and $A_h$.

### R16.5.6 *Reinforcement detailing*

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**16.5.6.3** At the front face of a bracket or corbel, primary tension reinforcement shall be anchored by (a), (b), or (c):

(a) A weld to a transverse bar of at least equal size that is designed to develop $f_y$ of primary tension reinforcement
(b) Bending the primary tension reinforcement back to form a horizontal loop
(c) Other means of anchorage to develop $f_y$

## COMMENTARY

**R16.5.6.3** For brackets and corbels of variable depth (refer to Fig. R16.5.1a), the stress at ultimate in the reinforcement is almost constant at approximately $f_y$ from the face of support to the load point. This is because the horizontal component of the inclined concrete compression strut is transferred to the primary tension reinforcement at the location of the vertical load. Therefore, reinforcement should be anchored at its outer end (refer to 16.5.6.3) and extended beyond the face of the support to be developed in tension at the face of the support. Satisfactory anchorage at the outer end can be obtained by bending the primary tension reinforcement bars in a horizontal loop as specified in 16.5.6.3b, or by welding a bar of equal diameter or a suitably sized angle across the ends of the primary tension reinforcement bars. The weld detail used successfully in the corbel tests reported in Mattock et al. (1976a) is shown in Fig. R16.5.6.3b. Refer to ACI Committee 408 (1966).

An end hook in the vertical plane, with the minimum diameter bend, is not totally effective because a zone of unreinforced concrete beneath the point of loading will exist for loads applied close to the end of the bracket or corbel. For wide brackets (perpendicular to the plane of the figure) and loads not applied close to the end, U-shaped bars in a horizontal plane provide effective end hooks.

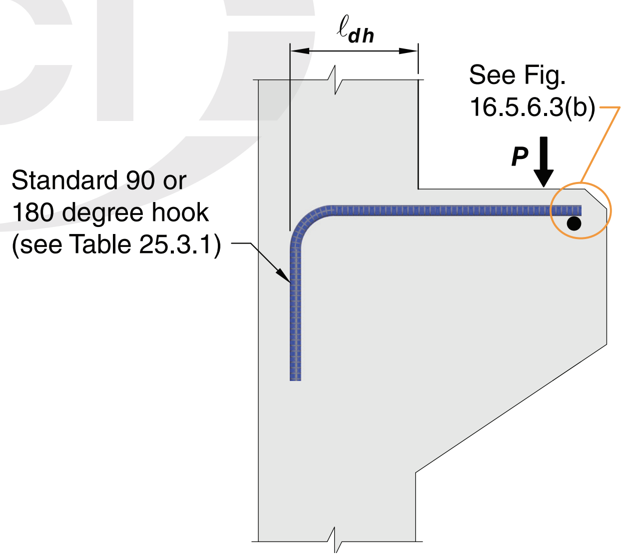


*Fig. R16.5.6.3a—Member largely dependent on support and end anchorages.*

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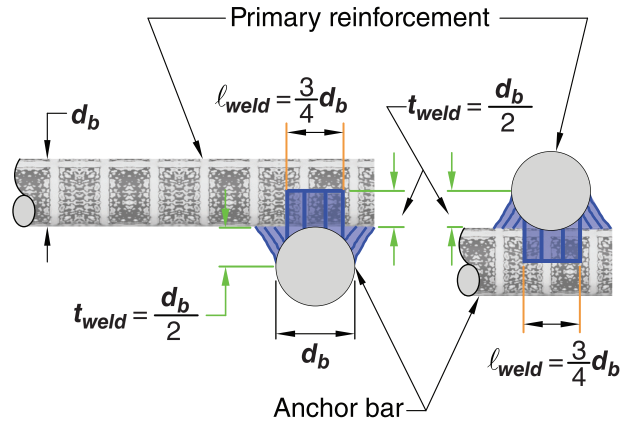


*Fig. R16.5.6.3b—Weld details used in tests of Mattock et al. (1976a).*

**16.5.6.4** Primary tension reinforcement shall be developed at the face of the support.

**16.5.6.5** Development of tension reinforcement shall account for distribution of stress in reinforcement that is not directly proportional to the bending moment.

**R16.5.6.5** Calculated stress in reinforcement at service loads, $f_s$, does not decrease linearly in proportion to a decreasing moment in brackets, corbels, and members of variable depth. Additional consideration is required for proper development of the flexural reinforcement.

**16.5.6.6** Closed stirrups or ties shall be spaced such that $A_h$ is uniformly distributed within (2/3)$d$ measured from the primary tension reinforcement.

**R16.5.6.6** Refer to R16.5.5.2.

16 Connections

# Notes