CODE

COMMENTARY

and loaded monotonically to yielding, this approach is feasible because the likelihood of compressive failure can be estimated reliably with the behavioral model assumed for determining the reinforcement ratio corresponding to balanced failure. The same behavioral model, because of incorrect assumptions such as linear strain distribution, well-defined yield point for the steel, limiting compressive strain in the concrete of 0.003, and compressive stresses in the shell concrete, fails to describe the conditions in a flexural member subjected to reversals of displacements well into the inelastic range. Therefore, there is little rationale for continuing to refer to balanced strain conditions in earthquake-resistant design of flexural members in reinforced concrete structures.

21.5.2.1 At any section of a flexural member, except as provided in 10.5.3, for the top and bottom reinforcement, the amount of reinforcement shall not be less than that given by Eq. (10-3), but not less than $1.4b_w d/f_y$, and the reinforcement ratio ρ shall not exceed 0.025. At least two bars shall be provided continuously at both the top and the bottom.

R21.5.2.1 The limiting reinforcement ratio of 0.025 is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in the flexural members of typical proportions. The requirement of at least two bars in the top and bottom of beams refers again to construction rather than behavioral requirements.

21.5.2.2 The positive moment strength at the joint face shall be not less than one-half of the negative moment strength provided at that face of the joint. Neither the negative nor the positive moment strength, at any section along the member length, shall be less than one-fourth the maximum moment strength provided at the face of either joint.

21.5.2.3 Lap splices of the flexural reinforcement shall be used only if hoop or spiral reinforcement is provided over the lap length. The spacing of the transverse reinforcement enclosing the lapped bars shall not exceed the smaller of $d/4$ and 100 mm. Lap splices shall not be used:

(a) Within the joints; or

(b) Within a distance of twice the member depth from the face of the joint.

R21.5.2.3 Lap splices of the reinforcement are not permitted at regions where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range. Transverse reinforcement for lap splices at any location is required because of the likelihood of the loss of shell concrete.

21.5.2.4 Mechanical splices shall conform to 21.1.6. Welded splices shall conform to 21.1.7.

21.5.2.5 Prestressing, where used, shall satisfy (a) and (b):

(a) Stress f_{pc} shall not exceed the smaller of 3.5 MPa and $f_c'/10$; and

(b) Anchorages of the post-tensioning tendons resisting earthquake-induced forces shall be capable of allowing tendons to withstand 50 cycles of loading, bounded by 40 and 85 percent of the specified tensile strength of prestressing steel.

R21.5.2.5 These provisions were developed, in part, based on the observations of performance of buildings in earthquakes.[21.6] For calculating the average prestress, the smallest cross-sectional dimension in a beam normally is the web dimension, not the flange thickness.

Fatigue testing for 50 cycles of loading, between 40 and 80 percent of the specified tensile strength of the prestressing steel, has been an industry practice of long standing.[21.6,21.7] The 80 percent limit was increased to 85 percent to correspond to the 1 percent limit on the strain in prestressing steel. Testing over this range of stress is intended to conservatively simulate the effect of a severe earthquake. Additional details on testing procedures, to different stress levels, are provided in Reference 21.8.

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COMMENTARY

The special requirements for prestressing steel in ACI 318M-08 are not applicable to ACI 349M-13, as these requirements relate to plastic hinge regions.

21.5.3 *Transverse reinforcement*

21.5.3.1 Hoops shall be provided in the following regions of frame members:

(a) Over a length equal to twice the member depth, measured from the face of the supporting member toward midspan, at both ends of the flexural member; and

(b) Over lengths equal to twice the member depth, on both sides of a section, where flexural yielding is likely to occur under impulsive or impactive loading, or during a beyond design basis event.

21.5.3.2 The first hoop shall be located not more than 50 mm from the face of a supporting member. Spacing of the hoops shall not exceed the smallest of (a), (b) or (c):

(a) $d/4$;

(b) Six times the diameter of the smallest longitudinal bars; or

(c) 150 mm.

21.5.3.3 Where hoops are required, primary flexural reinforcing bars, excluding longitudinal skin reinforcement required in 10.6.7 on the perimeter, shall have lateral support conforming to 7.10.5.3. The spacing of supported longitudinal bars in the outermost layer shall not exceed 350 mm on center.

21.5.3.4 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than $d/2$, or 450 mm, whichever is smaller, throughout the length of the member.

21.5.3.5 The stirrups or ties required to resist shear shall be hoops over the lengths of the members defined in 21.5.3.1.

21.5.3.6 The hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends, and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90-degree hooks of the crossties shall be placed on that side.

R21.5.3 *Transverse reinforcement*—Transverse reinforcement is required primarily to confine the concrete and to maintain lateral support for the reinforcing bars in regions where high stresses are expected. This is especially true if $F_\mu > 1.0$ is used, in which case, the transverse reinforcement should be provided similar to ACI 318 requirements. If $F_\mu = 1.0$ is used, shear reinforcement should be provided in accordance with 21.5.4.

Examples of hoops suitable for flexural members of frames are shown in Fig. R21.5.3.

For members with varying strength along the span, or members for which the permanent load represents a large proportion of the total design load, concentrations of inelastic rotation may occur within the span during a beyond-design-basis earthquake shaking event. If such a condition is anticipated, transverse reinforcement also should be provided in regions where yielding is expected in beyond-design-basis earthquake shaking.

Because spalling of the concrete shell might occur during beyond-design-basis earthquake shaking, especially at, and near, the regions of flexural yielding, all web reinforcement should be provided in the form of closed hoops as defined in 21.5.3.5.

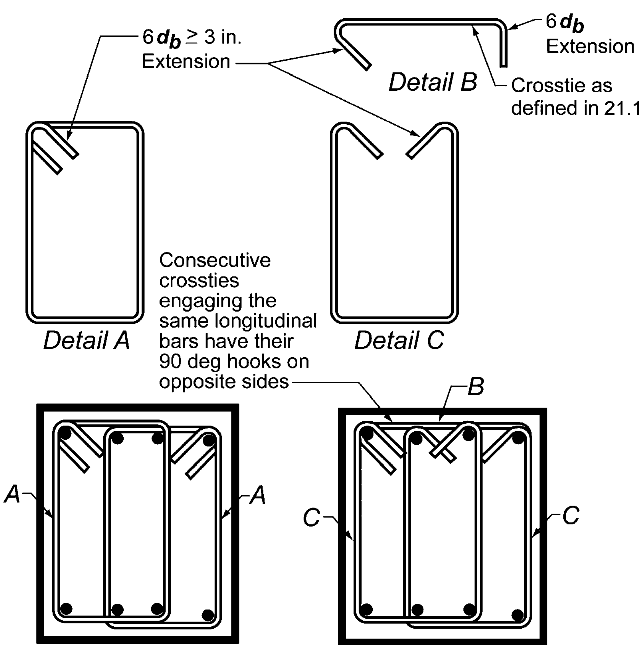

Fig. R21.5.3—Examples of overlapping hoops.

21.5.4 *Shear strength requirements*

R21.5.4 *Shear strength requirements*

21.5.4.1 *Design forces*—The design shear force V_e shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be

R21.5.4.1 *Design forces*—In ACI 349M-13, it is assumed that the frame members will dissipate energy in the nonlinear range of response during beyond-design-basis earthquake

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assumed that the moments of opposite sign corresponding to probable flexural moment strength M_{pr} act at the joint faces, and that the member is loaded with the factored tributary gravity load along its span.

COMMENTARY

shaking. Unless a frame member possesses a strength that is on the order of twice the SSE forces, it should be assumed that it will yield in the event of BDBE shaking. The design shear force should be a good approximation of the maximum shear that may develop in a member. Therefore, the required shear strength for the frame members is related to the flexural strengths of the designed member, rather than related to the factored shear forces indicated by the SSE analysis. The conditions described by 21.5.4.1 are illustrated in Fig. R21.5.4.

Because the actual yield strength of the longitudinal reinforcement may exceed the specified yield strength, and because strain hardening of the reinforcement is likely to take place at a joint subjected to large rotations, the required shear strengths are determined using a stress of at least $1.25f_y$ in the longitudinal reinforcement. Consideration should also be given to increases in the flexural strength associated with slab effects, and the use of reinforcing bars with yield strengths in excess of the specified minimum yield strength.

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Notes on Fig. R21.5.4:

1. Direction of shear force V_e depends on relative magnitudes of gravity loads and shear generated by end moments.
2. End moments M_{pr} based on steel tensile stress of 1.25 f_y, where f_y is specified yield strength. (Both end moments should be considered in both directions, clockwise and counter-clockwise).
3. End moment M_{pr} for columns need not be greater than moments generated by the M_{pr} of the beams framing into the beam-column joints. V_e should not be less than that required by analysis of the structure.

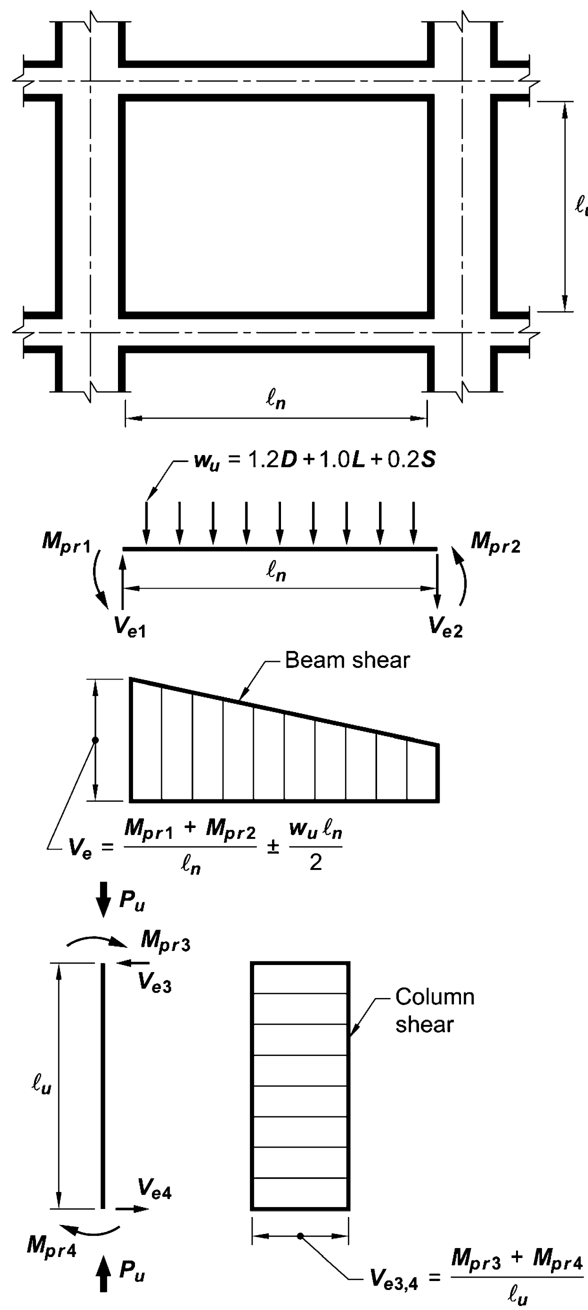

Fig. R21.5.4—Design shears for beams and columns.

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21.5.4.2 *Transverse reinforcement*—The transverse reinforcement, over the lengths identified in 21.5.3.1, shall be proportioned to resist shear, assuming $V_c = 0$ when both (a) and (b) occur:

(a) The earthquake-induced shear force calculated in accordance with 21.5.4.1 represents one-half or more of the maximum required shear strength within those lengths; and

(b) The factored axial compressive force P_u, including earthquake effects, is less than $A_g f_c'/20$.

21.6—Moment frame members subjected to bending and axial load

21.6.1 *Scope*—The requirements of this section apply to moment frame members that form part of the seismic-force-resisting system, and that resist a factored axial compressive force P_u under any load combination exceeding $A_g f_c'/10$. These frame members shall also satisfy the conditions of 21.6.1.1 and 21.6.1.2.

21.6.1.1 The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 300 mm.

21.6.1.2 The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

21.6.2 *Minimum flexural strength of columns*

21.6.2.1 The flexural strength of any column, proportioned to resist P_u exceeding $A_g f_c'/10$, shall satisfy 21.6.2.2.

21.6.2.2 The flexural strengths of the columns shall satisfy Eq. (21-1)

$$\Sigma M_{nc} \geq (6/5)\Sigma M_{nb} \tag{21-1}$$

where ΣM_{nc} is the sum of the nominal flexural strengths of columns framing into the joint, evaluated at the faces of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength; and ΣM_{nb} is the sum of the nominal flexural strengths of the beams framing into the joint, evaluated at the faces of the joint. In T-beam construction, where the slab

COMMENTARY

R21.5.4.2 *Transverse reinforcement*—Experimental studies[21.9,21.10] of reinforced concrete members subjected to cyclic loading have demonstrated that more shear reinforcement is required to ensure a flexural failure, if the member is subjected to alternating nonlinear displacements, than if the member is loaded in only one direction: the necessary increase of shear reinforcement being higher in the case of no axial load. This observation is reflected in the Code (see 21.5.4.2) by eliminating the term representing the contribution of concrete to shear strength. The added conservatism on shear strength is deemed necessary in locations where potential flexural hinging may occur during a BDBE event. However, this stratagem, chosen for its relative simplicity, should not be interpreted to mean that no concrete is required to resist shear. On the contrary, it may be argued that the concrete core resists the entire shear, with the shear (transverse) reinforcement confining and strengthening the concrete. The confined concrete core plays an important role in the behavior of the beam and should not be reduced to a minimum just because the design expression does not explicitly recognize it.

R21.6—Moment frame members subjected to bending and axial load

R21.6.1 *Scope*—Section 21.6.1 is intended primarily for columns of moment frames. Frame members, other than columns, that do not satisfy 21.5.1, are to be proportioned and detailed according to this section. The geometric constraints in 21.6.1.1 and 21.6.1.2 follow from previous practice.[21.11]

R21.6.2 *Minimum flexural strength of columns*—The intent of 21.6.2.2 is to reduce the likelihood of yielding in columns in beyond-design-basis earthquake shaking. The strong-column/weak-beam philosophy of ACI 318M-08 is followed in ACI 349M-13. The factor of 1.2 (= 6/5) is adopted in ACI 318M-08. In the case of weak columns, flexural yielding might occur at both ends of all columns in a given story in BDBE shaking, resulting in a column failure mechanism that can lead to collapse, which is unacceptable for a nuclear structure.

In 21.6.2.2, the nominal strengths of the girders and columns are calculated at the joint faces, and those strengths are compared directly using Eq. (21-1).

When determining the nominal flexural strength of a girder section in negative bending (top in tension), the longitudinal reinforcement contained within an effective flange width of a top slab that acts monolithically with the girder, increases the girder strength. Research[21.12] on beam-

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is in tension under the moments at the face of the joint, the slab reinforcement within an effective slab width defined in 8.12 shall be assumed to contribute to M_{nb}, if the slab reinforcement is developed at the critical section for flexure.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Equation (21-1) shall be satisfied for the beam moments acting in both directions, in the vertical plane of the frame considered.

21.6.3 *Longitudinal reinforcement*

21.6.3.1 The area of longitudinal reinforcement, A_{st}, shall not be less than $0.01A_g$ or more than $0.06A_g$.

21.6.3.2 Mechanical splices shall conform to 21.1.6. Welded splices shall conform to 21.1.7. Lap splices shall be permitted only within the center half of the member length. Lap splices shall be designed as tension lap splices. Lap splices shall be enclosed within the transverse reinforcement conforming to 21.6.4.2 and 21.6.4.3.

21.6.4 *Transverse reinforcement*

21.6.4.1 Reinforcement required in 21.6.4.2 through 21.6.4.4 shall be provided over a length ℓ_o from each joint face, and on both sides of any section where flexural yielding is likely to occur under impulsive or impactive loads, or during a beyond design basis earthquake. The length ℓ_o shall not be less than the largest of (a), (b), or (c):

(a) The depth of the member at the joint face, or at the section where flexural yielding is likely to occur;

(b) One-sixth of the clear span of the member; or

(c) 450 mm.

21.6.4.2 Transverse reinforcement shall be provided by either single or overlapping spirals satisfying 7.10.4, circular hoops, or rectilinear hoops with or without crossties. Crossties of the same or smaller bar size as the hoops shall be permitted. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement. Spacing of crossties or legs of rectilinear hoops, h_x, within a cross section of the member shall not exceed 350 mm on center.

COMMENTARY

column subassemblies under lateral loading indicates that using the effective flange widths defined in Section 8.10 gives the reasonable estimates of the girder negative bending strengths of interior connections at interstory displacement levels approaching 2 percent of the story height. This effective width is conservative, where the slab terminates in a weak spandrel beam.

R21.6.3 *Longitudinal reinforcement*—The lower limit of the reinforcement ratio is to control time-dependent deformations and to have the yield moment exceed the cracking moment and ensure a ductile failure mode. The upper limit of the section reflects a concern for steel congestion, the load transfer from the floor members to columns, and the development of high shear stresses.

Spalling of the shell concrete, which is likely to occur near the ends of the column in frames of typical configuration, makes lap splices in these locations vulnerable in BDBE. If lap splices are to be used, they should be located near the midheight of a column, where stress reversal is likely to be limited to a smaller stress range than at locations near the joints. Special transverse reinforcement is required along the lap-splice length because of the uncertainty in moment distributions along the height, and the need for confinement of lap splices subjected to stress reversals.[21.13]

R21.6.4 *Transverse reinforcement*—The requirements of this section are concerned with confining the concrete and providing lateral support to the longitudinal reinforcement.

R21.6.4.1 Section 21.6.4.1 stipulates a minimum length over which to provide closely-spaced transverse reinforcement at the member ends, where flexural yielding normally occurs. Research results indicate that the length should be increased by 50 percent or more in locations such as the base of the structure, where axial loads and flexural demands may be especially high.[21.14]

R21.6.4.2 Sections 21.6.4.2 and 21.6.4.3 provide the requirements for the configuration of transverse reinforcement for columns and for joints of moment frames. Figure R21.6.4.2 shows an example of the transverse reinforcement provided by one hoop and three crossties. Crossties with a 90-degree hook are not as effective in providing confinement as either crossties with 135-degree hooks or hoops. Tests show that if crosstie ends with 90-degree hooks are alternated, confinement will be sufficient.

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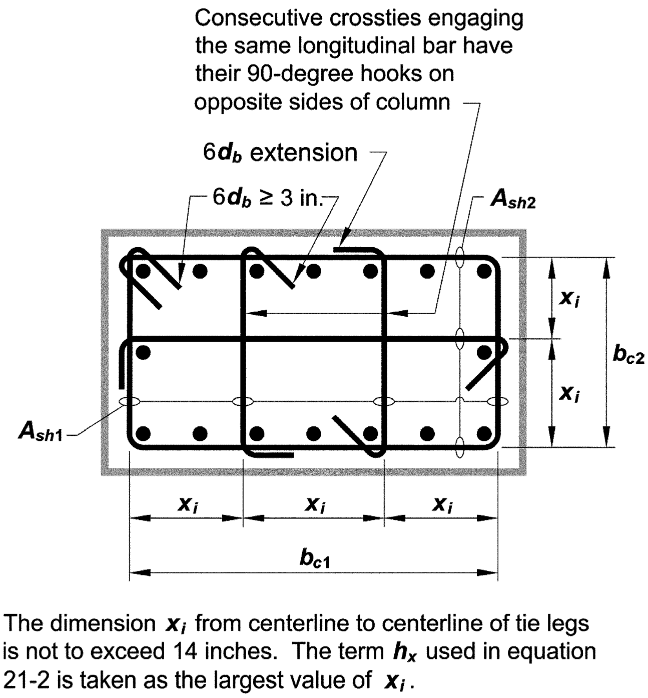

Fig. R21.6.4.2—Example of transverse reinforcement in columns.

21.6.4.3 Spacing of transverse reinforcement along the ℓ_o of the member shall not exceed the smallest of (a), (b), or (c):

(a) One-quarter of the minimum member dimension;

(b) Six times the diameter of the smallest longitudinal bar; and

(c) s_o, as defined by Eq. (21-2)

$$s_o = 100 + \frac{(350 - h_x)}{3} \tag{21-2}$$

The value of s_o shall not exceed 150 mm, and need not be taken less than 100 mm.

R21.6.4.3 The requirement that spacing not exceed 1/4 of the minimum member dimension is to obtain adequate concrete confinement. The requirement that spacing not exceed six bar diameters is intended to restrain longitudinal reinforcement buckling after spalling. The 100 mm spacing is for concrete confinement. Section 21.4.4.2 permits this limit to be relaxed to a maximum of 150 mm if the spacing of the crossties, or the legs of overlapping hoops, is limited to 200 mm.

21.6.4.4 The amount of transverse reinforcement required in (a) or (b) as follows, shall be provided, unless a larger amount is required by 21.6.5.

(a) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s, shall not be less than required by Eq. (21-3)

$$\rho_s = 0.12 f_c'/f_{yt} \tag{21-3}$$

and shall not be less than required by Eq. (10-5); and

(b) The total cross-sectional area of the rectangular hoop reinforcement, A_{sh}, shall not be less than required by Eq. (21-4) and (21-5)

$$A_{sh} = 0.3(s b_c f_c'/f_{yt})[(A_g/A_{ch}) - 1] \tag{21-4}$$

R21.6.4.4 The effect of helical (spiral) reinforcement, and adequately configured rectangular hoop reinforcement, on the strength and ductility of columns, is well established.[21.15] While analytical procedures exist for calculation of the strength and ductility capacity of columns under axial and moment reversals,[21.16] the axial load and deformation demands required during BDBE loading are not known with sufficient accuracy to justify calculation of the required transverse reinforcement as a function of BDBE demands. Instead, Eq. (10-5) and (21-4) are required, with the intent that spalling of the shell concrete will not result in a loss of axial strength of the column. Equations (21-3) and (21-5) govern for large-diameter columns, and are intended to ensure adequate flexural curvature capacity in the yielding regions.

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$$A_{sh} = 0.09sb_cf_c'/f_{yt} \quad (21\text{-}5)$$

21.6.4.5 Beyond ℓ_o specified in 21.6.4.1, the column shall contain spiral or hoop reinforcement satisfying 7.10 with center-to-center spacing s not exceeding the smaller of six times the diameter of the smallest longitudinal column bars and 150 mm, unless a larger amount of transverse reinforcement is required by 21.6.3.2 or 21.6.5.

21.6.4.6 Columns supporting reactions from discontinued stiff members, such as walls, shall satisfy (a) and (b):

(a) The transverse reinforcement, as required in 21.6.4.2 through 21.6.4.4, shall be provided over their full height beneath the discontinuity, if the factored axial compressive force in these members, related to earthquake effect, exceeds $A_gf_c'/10$; and

(b) The transverse reinforcement shall extend into the discontinued member at least the ℓ_d of the largest longitudinal column bar, where ℓ_d is determined in accordance with 21.7.5.

- Where the lower end of the column terminates on a wall, the required transverse reinforcement shall extend into the wall at least the ℓ_d of the largest longitudinal column bar, at the point of termination. Where the column terminates on a footing or a mat, the required transverse reinforcement shall extend at least 300 mm into the footing or the mat.

21.6.4.7 If the concrete cover outside the confining transverse reinforcement specified in 21.6.4.1, 21.6.4.5, and 21.6.4.6 exceeds 100 mm, additional transverse reinforcement shall be provided. The concrete cover for additional transverse reinforcement shall not exceed 100 mm and the spacing of additional transverse reinforcement shall not exceed 300 mm.

21.6.5 *Shear strength requirements*

21.6.5.1 *Design forces*—The design shear force V_e shall be determined from the consideration of the maximum forces that can be generated at the faces of the joints, at each end of the member. These joint forces shall be determined using the maximum probable moment strengths M_{pr} at each end of the member, associated with the range of factored axial loads P_u acting on the member. The member shears need not exceed those determined from joint strengths based on M_{pr} of the transverse members framing into the joint. In no case shall

COMMENTARY

Equations (21-4) and (21-5) are to be satisfied in both cross-sectional directions of the rectangular core. For each direction, b_c is the core dimension, perpendicular to the tie legs that constitute A_{sh}, as shown in Fig. R21.6.4.2.

Research results indicate that yield strengths higher than those specified in 11.4.2 can be used effectively as confinement reinforcement. A value of f_{yt} of 700 MPa is permitted in Eq. (21-3), (21-4), and (21-5), where ASTM A1035 is used as confinement reinforcement.

R21.6.4.5 The provisions of 21.6.4.5 are intended to provide reasonable protection and ductility to the midheight of columns outside the length ℓ_o. Observations after earthquakes have shown significant damage to columns in this region. The minimum ties or spirals required should provide a more uniform toughness of the column along its length.

R21.6.4.6 In a shear wall structure, the lateral-load-resisting system at any level should not consist only of the columns, creating a soft-story. However, the columns may support discontinued stiff members as part of the lateral-load-resisting system. In such cases, it is required that these columns have the specified transverse reinforcement throughout their length. This provision applies to all the supporting columns beneath the level at which the stiff member has been discontinued.

R21.6.5 *Shear strength requirements*

R21.6.5.1 *Design forces*—The provisions of 21.5.4.1 also apply to members subjected to axial loads—for example, columns. Above the ground floor, the moment at a joint may be limited by the flexural strength of the beams framing into the joint. Where beams frame into opposite sides of a joint, the combined strength may be the sum of the negative moment strength of the beam on one side of the joint and the positive moment strength of the beam on the other side of the joint. Moment strengths are to be determined using a

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V_e be less than the factored shear determined by analysis of the structure.

21.6.5.2 *Transverse reinforcement*—Transverse reinforcement over the ℓ_o, identified in 21.6.4.1, shall be proportioned to resist shear, assuming $V_c = 0$, when both (a) and (b) occur:

(a) The earthquake-induced shear force, calculated in accordance with 21.6.5.1, represents one-half or more of the maximum required shear strength within ℓ_o; and

(b) P_u, including earthquake effects, is less than $A_g f_c'/20$.

21.7—Joints of moment frames

21.7.1 *Scope*—The requirements of 21.7 apply to beam-column joints of moment frames forming part of the seismic-force-resisting system.

21.7.2 *General requirements*

21.7.2.1 The forces in the longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.

21.7.2.2 Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension, according to 21.7.5, and in compression, according to Chapter 12.

21.7.2.3 Where the longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal beam bar.

COMMENTARY

strength-reduction factor of 1.0 and the steel reinforcement stress equal to at least $1.25f_y$. Distribution of the combined moment strength of the beams to the columns, above and below the joint, should be based on the analysis. The value of M_{pr} in Fig. R21.5.4 may be calculated from the flexural member strengths at the beam-column joints. Consideration should be given to both steel reinforcement stresses higher than 125 percent of the specified minimum yield strength, and the slab contribution to the strength of the beam. See also 21.5.4.1.

R21.7—Joints of moment frames

R21.7.1 *Scope*—Even though the nuclear safety-related structures are designed to remain elastic, or nearly elastic, under the design-basis earthquake effects, the joints may respond in the inelastic range under a BDBE. To provide enhanced ductility during a BDBE, the joint provisions of ACI 318M-08 mostly are maintained without change.

R21.7.2 *General requirements*—The development of inelastic rotations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, the joint shear force generated by the flexural reinforcement is calculated for a stress of $1.25f_y$ in the reinforcement. A detailed explanation of the reasons for the possible development of stresses in excess of the specified minimum yield strength in girder tensile reinforcement is provided in Reference 21.5.

R21.7.2.3 Research[21.17-21.21] has shown that straight beam bars may slip within the beam-column joint during a series of large moment reversals. The bond stresses on these straight bars may be very large. To substantially reduce slip during the formation of adjacent beam hinging, it would be necessary to have a ratio of column dimension to bar diameter of approximately 32, which would result in very large joints. On reviewing the available tests, the limit of 20 of the column depth in the direction of loading for the maximum size of beam bars was chosen. This limit provides a reasonable control on the amount of potential slip of the beam bars, in a beam-column joint, considering the number of anticipated inelastic excursions of the building frames during a major earthquake. A thorough treatment of this topic is given in Reference 21.22.

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21.7.3 *Transverse reinforcement*

21.7.3.1 Joint transverse reinforcement shall satisfy either 21.6.4.4(a), or 21.6.4.4(b), and shall also satisfy 21.6.4.2, 21.6.4.3, and 21.6.4.7, except as permitted in 21.7.3.2.

21.7.3.2 Where the members frame into all four sides of the joint, and where each member width is at least three-fourths the column width, the amount of reinforcement specified in 21.6.4.4(a) or 21.6.4.4(b) shall be permitted to be reduced by half, and the spacing required in 21.6.4.3 shall be permitted to be increased to 150 mm, within the overall depth of the shallowest framing member.

21.7.3.3 The longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 21.5.3.2, and the requirements of 21.5.3.3 and 21.5.3.6, if such confinement is not provided by a beam framing into the joint.

21.7.4 *Shear strength*

21.7.4.1 V_n of the joint shall not be taken as greater than the values specified as follows:

(a) For joints confined on all four faces: $1.7\sqrt{f_c'}A_j$;

(b) For joints confined on three faces, or on two opposite faces: $1.2\sqrt{f_c'}A_j$;

(c) For others: $1.0\sqrt{f_c'}A_j$.

A member that frames into a face is considered to provide confinement to the joint if at least 3/4 of the face of the joint is covered by the framing member. Extensions of beams at least one overall beam depth, h, beyond the joint face, are permitted to be considered as confining members.

The extensions of beams shall satisfy 21.5.1.3, 21.5.2.1, 21.5.3.2, 21.5.3.3, and 21.5.3.6. A joint is considered to be confined if such confining members frame into all faces of the joint.

The A_j is the effective cross-sectional area within a joint, calculated from the joint depth times the effective joint width. The joint depth shall be the overall depth of the column, h. The effective joint width shall be the overall width of the column; except where a beam frames into a wider column, the effective joint width shall not exceed the smaller of either (a) or (b):

(a) Beam width plus joint depth; or

(b) Twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.

COMMENTARY

R21.7.3 *Transverse reinforcement*—The Code requires transverse reinforcement in a joint, regardless of the magnitude of the calculated shear force. In 21.7.3.2, the amount of confining reinforcement may be reduced and the spacing may be increased if horizontal members frame into all four sides of the joint.

Section 21.5.3.3 refers to a joint where the width of the beam exceeds the corresponding column dimension. In that case, beam reinforcement not confined by the column reinforcement should be provided lateral support either by a beam framing into the same joint, or by transverse reinforcement.

An example of transverse reinforcement through the column, provided to confine the beam reinforcement passing outside the column core, is shown in Fig. R21.5.1. Additional detailing guidance and design recommendations for both interior and exterior wide beam connections, with beam reinforcement passing outside the column core may be found in Reference 21.5.

R21.7.4 *Shear strength*—Requirements in Chapter 21 for proportioning joints are based on Reference 21.5, in that the behavioral phenomena within the joint are interpreted in terms of a nominal shear strength of the joint. Because tests of joints[21.17] and deep beams[21.4] indicate that the shear strength was not as sensitive to joint (shear) reinforcement, as implied by the expression developed by Joint ACI-ASCE Committee 326[21.23] for beams, ACI Committee 318 set the strength of the joint as a function only of the compressive strength of the concrete (see 21.7.4), and requires a minimum amount of transverse reinforcement in the joint (see 21.7.3). The effective area of the joint, A_j, is illustrated in Fig. R21.7.4. In no case is A_j to be greater than the column cross-sectional area.

The three levels of shear strength required by 21.7.4.1 are based upon the recommendation of ACI Committee 352. The test data reviewed by the Committee[21.1] indicate that the lower value given in the ACI 318-83 Code was not conservative when applied to corner joints.

Cyclic loading tests of joints, with extensions of beams with lengths at least equal to their depths, have indicated similar joint shear strengths to those of joints with continuous beams. These findings suggest that the extensions of beams, when properly dimensioned and reinforced with longitudinal and transverse bars, provide effective confinement to the joint faces, thereby delaying the joint strength deterioration at large deformations.[21.24]

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