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Seismic Design of Reinforced Concrete Special Moment Frames:

A Guide for Practicing Engineers

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National Institute of Standards and Technology
U.S. Department of Commerce

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Cover photo – Reinforced concrete special moment frame under construction.

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Contents

1. Introduction

Reinforced concrete special moment frames are used as part of seismic force-resisting systems in buildings that are designed to resist earthquakes. Beams, columns, and beam-column joints in moment frames are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sways through multiple displacement cycles during strong earthquake ground shaking. Special proportioning and detailing requirements result in a frame capable of resisting strong earthquake shaking without significant loss of stiffness or strength. These moment-resisting frames are called "Special Moment Frames" because of these additional requirements, which improve the seismic resistance in comparison with less stringently detailed Intermediate and Ordinary Moment Frames.

The design requirements for special moment frames are presented in the American Concrete Institute (ACI) Committee 318 *Building Code Requirements for Structural Concrete* (ACI 318). The special requirements relate to inspection, materials, framing members (beams, columns, and beam-column joints), and construction procedures. In addition, requirements pertain to diaphragms, foundations, and framing members not designated as part of the seismic force-resisting system. The numerous interrelated requirements are covered in several sections of ACI 318, not necessarily arranged in a logical sequence, making their application challenging for all but the most experienced designers.

This guide was written for the practicing structural engineer to assist in the application of ACI 318 requirements for special moment frames. The material is presented in a sequence that practicing engineers have found useful. The guide is intended especially for the practicing structural engineer, though it will also be useful for building officials, educators, and students.

This guide follows the requirements of the 2008 edition of ACI 318, along with the pertinent seismic load requirements specified in the American Society of Civil Engineers (ASCE) publication ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* (ASCE 2006). The International Building Code, or IBC, (ICC 2006), which is the code generally adopted throughout the United States, refers to ASCE 7 for the determination of seismic loads. The ACI Building Code classifies design requirements according to the Seismic Design Categories designated by the IBC and ASCE 7 and contains the latest information on design of special moment frames at the time of this writing. Because the 2008 edition of ACI 318 may not yet be adopted in many jurisdictions, not all of its provisions will necessarily apply.

Most special moment frames use cast-in-place, normal-weight concrete having rectilinear cross sections without prestressing. Interested readers are referred to ACI 318 for specific requirements on the use of lightweight concrete, prestressed beams, spiral-reinforced columns, and precast concrete, which are not covered in this guide.

The main body of text in this guide emphasizes code requirements and accepted approaches to their implementation. It includes background information and sketches to help understand the requirements. Additional guidance is presented in sidebars appearing alongside the main text. Sections 2 through 6 present analysis, behavior, proportioning, and detailing requirements for special moment frames and other portions of the building that interact with them. Section 7

Sidebars in the guide

Sidebars are used in this guide to illustrate key points, to highlight construction issues, and to provide additional guidance on good practices and open issues in special moment frame design.

presents construction examples to illustrate detailing requirements for constructability. Cited references, notation and abbreviations, and credits are in Sections 8, 9, and 10 respectively.

ACI 318: 2005 versus 2008

ACI 318-05 (ACI 2005) is currently the referenced document for concrete seismic construction in most jurisdictions in the U.S. In the interest of incorporating the most recent developments, however, this guide is based on ACI 318-08 (ACI 2008). Most of the technical requirements of the two documents for special moment frames are essentially the same. One notable difference is the effective stiffness requirements for calculating lateral deflections in Chapter 8. In addition, Chapter 21 was revised to refer to Seismic Design Categories directly, and was reorganized so the requirements for special systems, including special moment frames, are later in the chapter than in earlier editions of the code. As a result, section numbers 21.6 through 21.13 in ACI 318-08, the reference document used in this guide, correspond generally to sections 21.4 through 21.11 in ACI 318-05.

Code Requirements versus Guide Recommendations

Building codes present minimum requirements for design, construction, and administration of buildings, and are legal requirements where adopted by the authority having jurisdiction over the building. Thus, where adopted, the *Building Code Requirements for Structural Concrete* (ACI 318-08) must, as a minimum, be followed. In addition to the Building Code, the American Concrete Institute also produces guides and recommended practices. An example is *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures* (ACI 352R-02) (ACI 2002). In general, guides of this type present recommended good practice, which as a minimum also meets the requirements of the Building Code.

This guide is written mainly to clarify requirements of the Building Code, but it also introduces other guides such as ACI 352R-02 and it presents other recommendations for good design and construction practices. This guide is written to clearly differentiate between Building Code requirements and other recommendations.

2. The Use of Special Moment Frames

2.1 Historic Development

Reinforced concrete special moment frame concepts were introduced in the U.S. starting around 1960 (Blume, Newmark, and Corning 1961). Their use at that time was essentially at the discretion of the designer, as it was not until 1973 that the Uniform Building Code (ICBO 1973) first required use of the special frame details in regions of highest seismicity. The earliest detailing requirements are remarkably similar to those in place today.

In most early applications, special moment frames were used in all framing lines of a building. A trend that developed in the 1990s was to use special moment frames in fewer framing lines of the building, with the remainder comprising gravity-only framing that was not designated as part of the seismic force-resisting system. Some of these gravity-only frames did not perform well in the 1994 Northridge Earthquake, leading to more stringent requirements for proportioning and detailing these frames. The provisions for members not designated as part of the seismic force-resisting system are contained in ACI 318 - 21.13 and apply wherever special moment frames are used in Seismic Design Category D, E, or F. Because the detailing requirements for the gravity-only elements in those cases are similar to the requirements for the special moment frames, some economy may be achieved if the gravity-only frames can be made to qualify as part of the seismic force-resisting system.

Special moment frames also have found use in dual systems that combine special moment frames with shear walls or braced frames. In current usage, the moment frame is required to be capable of resisting at least 25 % of the design seismic forces, while the total seismic resistance is provided by the combination of the moment frame and the shear walls or braced frames in proportion with their relative stiffnesses. ASCE 7 - 12.2.1 limits the height of certain seismic force-resisting systems such as special reinforced concrete shear walls and special steel concentrically braced frames. These height limits may be extended when special moment frames are added to create a dual system.

2.2 When To Use Special Moment Frames

Moment frames are generally selected as the seismic force-resisting system when architectural space planning flexibility is desired. When concrete moment frames are selected for buildings assigned to Seismic Design Categories D, E, or F, they are required to be detailed as special reinforced concrete moment frames. Proportioning and detailing requirements for a special moment frame will enable the frame to safely undergo extensive inelastic deformations that are anticipated in these seismic design categories. Special moment frames may be used in Seismic Design Categories A, B, and C, though this may not lead to the most economical design. If special moment frames are selected as the seismic force-resisting system, ALL requirements for the frames must be satisfied to help ensure ductile behavior.

2.3 Frame Proportioning

Typical economical beam spans for special moment frames are in the range of 20 to 30 feet. In general, this range will result in beam depths that will support typical gravity loads and the requisite seismic forces without overloading the adjacent beam-column joints and columns. The clear span of a beam must be at least four times its effective depth per ACI 318 - 21.5.1.2. Beams are allowed to be wider than the supporting columns as noted in ACI 318 - 21.5.1.4, but beam width normally does not exceed the width of the column, for constructability. The provisions for special moment frames exclude use of slab-column framing as part of the seismic force-resisting system.

Special moment frames with story heights up to 20 feet are not uncommon. For buildings with relatively tall stories, it is important to make sure that soft (low stiffness) and/or weak stories are not created.

The ratio of the cross-sectional dimensions for columns shall not be less than 0.4 per ACI 318 - 21.6.1.2. This limits the cross section to a more compact section rather than a long rectangle. ACI 318 - 21.6.1.1 sets the minimum column dimension to 12 inches, which is often not practical to construct. A minimum dimension of 16 inches is suggested, except for unusual cases or for low-rise buildings.

2.4 Strength and Drift Limits

Both strength and stiffness need to be considered in the design of special moment frames. According to ASCE 7, special moment frames are allowed to be designed for a force reduction factor of $R = 8$. That is, they are allowed to be designed for a base shear equal to one-eighth of the value obtained from an elastic response analysis. Moment frames are generally flexible lateral systems; therefore, strength requirements may be controlled by the minimum base shear equations of the code. Base shear calculations for long-period structures, especially in Seismic Design Categories D, E, and F, are frequently controlled by the approximate upper limit period as defined in ASCE 7 - 12.8.2. Wind loads, as described in ASCE 7, must also be checked and may govern the strength requirements of special moment frames. Regardless of whether gravity, wind, or seismic forces are the largest, proportioning and detailing provisions for special moment frames apply wherever special moment frames are used.

The stiffness of the frame must be sufficient to control the drift of the building at each story within the limits specified by the building code. Drift limits in ASCE 7 are a function of both occupancy category (IBC 1604.5) and the redundancy factor, ρ, (ASCE 7 - 12.3.4) as shown in **Table 2-1**.

Redundancy Factor	Occupancy Category		
	I and II	III	IV
$\rho = 1.0$	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$
$\rho = 1.3$	$0.015h_{sx}$	$0.012h_{sx}$	$0.008h_{sx}$

Table 2-1 - Allowable story drift per ASCE 7. h_{sx} = story height.

The drift of the structure is to be calculated using the factored seismic load, amplified by C_d (ASCE 7 - 12.8.6), when comparing it with the values listed in **Table 2-1**. Furthermore, effective stiffness of framing members must be reduced to account for effects of concrete cracking (see Section 4.2 of this guide). The allowable wind drift limit is not specified by ASCE 7; therefore, engineering judgment is required to determine the appropriate limit. Consideration should be given to the attachment of the cladding and other elements and to the comfort of the occupants.

P-delta effects, discussed in ASCE 7 - 12.8.7, can be significant in a special moment frame and must be checked.

3. Principles for Design of Special Moment Frames

The design base shear equations of current building codes (e.g., IBC and ASCE 7) incorporate a seismic force-reduction factor *R* that reflects the degree of inelastic response expected for design-level ground motions, as well as the ductility capacity of the framing system. As mentioned in Section 2.4, the *R* factor for special moment frames is 8. Therefore, a special moment frame should be expected to sustain multiple cycles of inelastic response if it experiences design-level ground motion.

The proportioning and detailing requirements for special moment frames are intended to ensure that inelastic response is ductile. Three main goals are: (1) to achieve a strong-column/weak-beam design that spreads inelastic response over several stories; (2) to avoid shear failure; and (3) to provide details that enable ductile flexural response in yielding regions.

3.1 Design a Strong-column / Weak-beam Frame

When a building sways during an earthquake, the distribution of damage over height depends on the distribution of lateral drift. If the building has weak columns, drift tends to concentrate in one or a few stories (**Figure 3-1a**), and may exceed the drift capacity of the columns. On the other hand, if columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed (**Figure 3-1c**), and localized damage will be reduced. Additionally, it is important to recognize that the columns in a given story support the weight of the entire building above those columns, whereas the beams only support the gravity loads of the floor of which they form a part; therefore, failure of a column is of greater consequence than failure of a beam. Recognizing this behavior, building codes specify that columns be stronger than the beams that frame into them. This strong-column/weak-beam principle is fundamental to achieving safe behavior of frames during strong earthquake ground shaking.

ACI 318 adopts the strong-column/weak-beam principle by requiring that the sum of column strengths exceed the sum of beam strengths at each beam-column connection of a special moment frame. Studies (e.g. Kuntz and Browning 2003) have shown that the full structural mechanism of **Figure 3-1c** can only be achieved if the column-to-beam strength ratio is relatively large (on the order of four). As this is impractical in most cases, a lower strength ratio of 1.2 is adopted by ACI 318. Thus, some column yielding associated with an intermediate mechanism (**Figure 3-1b**) is to be expected, and columns must be detailed accordingly. Section 5.4 of this guide summarizes the column hoop and lap splice requirements of ACI 318.

3.2 Avoid Shear Failure

Ductile response requires that members yield in flexure, and that shear failure be avoided. Shear failure, especially in columns, is relatively brittle and can lead to rapid loss of lateral strength and axial load-carrying capacity (**Figure 3-2**). Column shear failure is the most frequently cited cause of concrete building failure and collapse in earthquakes.

Shear failure is avoided through use of a capacity-design approach. The general approach is to identify flexural yielding regions, design those regions for code-required moment strengths, and then calculate design shears based on equilibrium assuming the flexural yielding regions develop probable moment strengths. The probable moment strength is calculated using procedures that produce a high estimate of the moment strength of the as-designed cross section. Sections 5.3 and 5.4 discuss this approach more thoroughly for beam and column designs.

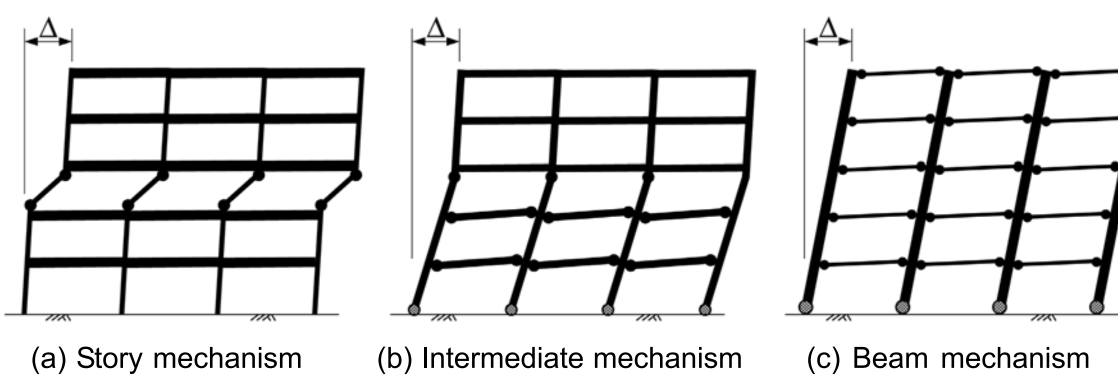


Figure 3-1 - Design of special moment frames aims to avoid the story mechanism (a) and instead achieve either an intermediate mechanism (b) or a beam mechanism (c).

Figure 3-2 - Shear failure can lead to a story mechanism and axial collapse.

3.3 Detail for Ductile Behavior

Ductile behavior of reinforced concrete members is based on the following principles.

Confinement for heavily loaded sections
Plain concrete has relatively small usable compressive strain capacity (around 0.003), and this might limit the deformability of beams and columns of special moment frames. Strain capacity can be increased ten-fold by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain dilation of the core concrete as it is loaded in compression, and this confining action leads to increased strength and strain capacity.

Hoops typically are provided at the ends of columns, as well as through beam-column joints, and at the ends of beams. **Figure 3-3** shows a column hoop configuration using rectilinear hoops. Circular hoops and spirals, which can be very efficient for column confinement, are not covered in this guide.

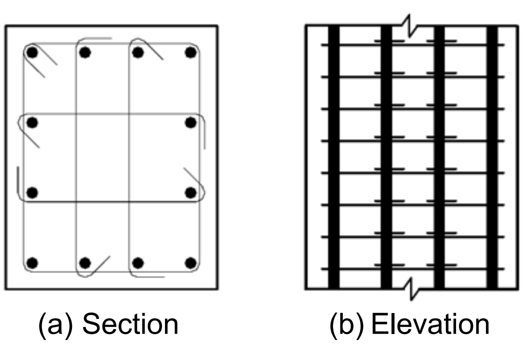


Figure 3-3 - Hoops confine heavily stressed cross sections of columns and beams, with (a) hoops surrounding the core and supplementary bars restraining longitudinal bars, all of which are (b) closely spaced along the member length.

To be effective, the hoops must enclose the entire cross section except the cover concrete, which should be as small as allowable, and must be closed by 135° hooks embedded in the core concrete; this prevents the hoops from opening if the concrete cover spalls off. Crossties should engage longitudinal reinforcement around the perimeter to improve confinement effectiveness. The hoops should be closely spaced along the longitudinal axis of the member, both to confine the concrete and restrain buckling of longitudinal reinforcement. Crossties, which typically have 90° and 135° hooks to facilitate construction, must have their 90° and 135° hooks alternated along the length of the member to improve confinement effectiveness.

Ample shear reinforcement
Shear strength degrades in members subjected to multiple inelastic deformation reversals, especially if axial loads are low. In such members ACI 318 requires that the contribution of concrete to shear resistance be ignored, that is, $V_c = 0$. Therefore, shear reinforcement is required to resist the entire shear force.

Avoidance of anchorage or splice failure
Severe seismic loading can result in loss of concrete cover, which will reduce development and lap-splice strength of longitudinal reinforcement. Lap splices, if used, must be located away from sections of maximum moment (that is, away from ends of beams and columns) and must have closed hoops to confine the splice in the event of cover spalling. Bars passing through a beam-column joint can create severe bond stress demands on the joint; for this reason, ACI 318 restricts beam bar sizes. Bars anchored in exterior joints must develop yield strength (f_y) using hooks located at the far side of the joint. Finally, mechanical splices located where yielding is likely must be Type II splices (these are splices capable of developing at least the specified tensile strength of the bar).

4. Analysis Guidance

4.1 Analysis Procedure

ASCE 7 allows the seismic forces within a special moment frame to be determined by three types of analysis: equivalent lateral force (ELF) analysis, modal response spectrum (MRS) analysis, and seismic response history (SRH) analysis. The ELF analysis is the simplest and can be used effectively for basic low-rise structures. This analysis procedure is not permitted for long-period structures (fundamental period T greater than 3.5 seconds) or structures with certain horizontal or vertical irregularities.

The base shear calculated according to ELF analysis is based on an approximate fundamental period, T_a, unless the period of the structure is determined by analysis. Generally, analysis will show that the building period is longer than the approximate period, and, therefore, the calculated base shear per ASCE 7 Equations 12.8-3 and 12.8-4 can be lowered. The upper limit on the period ($C_u T_a$) will likely limit the resulting base shear, unless the minimum base shear equations control.

An MRS analysis is often preferred to account for the overall dynamic behavior of the structure and to take advantage of calculated, rather than approximated, building periods. Another advantage of the MRS analysis is that the combined response for the modal base shear can be less than the base shear calculated using the ELF procedure. In such cases, however, the modal base shear must be scaled to a minimum of 85 % of the ELF base shear.

If an MRS or SRH analysis is required, 2-D and 3-D computer models are typically used. A 3-D model is effective in identifying the effects of any inherent torsion in the lateral system, as well as combined effects at corner conditions.

ASCE 7 - 12.5 specifies the requirements for the directions in which seismic loads are to be applied to the structure. The design forces for the beams and columns are independently based on the seismic loads in each orthogonal direction. It is common to apply the seismic loads using the orthogonal combination procedure of ASCE 7 - 12.5.3a in which 100 % of the seismic force in one direction is combined with 30 % of the seismic force in the perpendicular direction. Multiple load combinations are required to bound the orthogonal effects in both directions. The design of each beam and column is then based on an axial and biaxial flexural interaction for each load combination. The orthogonal force combination procedure is not required for all moment frame conditions, however. The ASCE 7 requirements should be reviewed and the frame should be designed accordingly.

ACI 318 - 21.1.2.1 requires that the interaction of all structural and nonstructural members that affect the linear and nonlinear response of the structure to earthquake motions be considered in the analysis. This can be especially important for special moment frames, which may be flexible in comparison with other parts of the building, including parts intended to be nonstructural in nature. Important examples include interactions with masonry infills (partial height or full height), architectural concrete walls, stair wells, cast-in-place stairways, and inclined parking ramps.

While permitting use of rigid members assumed not to be part of the seismic force-resisting system, ACI 318 - 21.1.2.2 requires that effects of these members be considered and accommodated by the design. Furthermore, effects of localized failures of one or more of these elements must be considered. For example, the failure of a rigid architectural element in one story could lead to formation of a story mechanism, as illustrated in **Figure 3-1(a)**. Generally, it is best to provide an ample seismic separation joint between the special moment frame and rigid elements assumed not to be part of the seismic force-resisting system. If adequate separation is not provided, the interaction effects specified in ASCE 7 - 12.7.4 must be addressed.

4.2 Stiffness Recommendations

When analyzing a special moment frame, it is important to appropriately model the cracked stiffness of the beams, columns, and joints, as this stiffness determines the resulting building periods, base shear, story drifts, and internal force distributions. **Table 4-1** shows the range of values for the effective, cracked stiffness for each element based on the requirements of ACI 318 - 8.8.2. For beams cast monolithically with slabs, it is acceptable to include the effective flange width of ACI 318 - 8.12.

Element	I_e/I_g
Beam	0.35-0.50
Column	0.50-0.70

Table 4-1 - Cracked stiffness modifiers.

More detailed analysis may be used to calculate the reduced stiffness based on the applied loading conditions. For example, ASCE 41 recommends that the following (**Table 4-2**) I_e/I_g ratios be used with linear interpolation for intermediate axial loads.

Compression Due to Design Gravity Loads	I_e/I_g
$\geq 0.5 A_g f_c'$	0.7
$\leq 0.1 A_g f_c'$	0.3

Table 4-2 - ASCE 41 Supplement No. 1 effective stiffness modifiers for columns.

Note that for beams this produces I_e/I_g = 0.30. When considering serviceability under wind loading, it is common to assume gross section properties for the beams, columns, and joints.

ACI 318 does not contain guidance on modeling the stiffness of the beam-column joint. In a special moment frame the beam-column joint is stiffer than the adjoining beams and columns, but it is not perfectly rigid. As described in ASCE 41 (including Supplement No. 1) the joint stiffness can be adequately modeled by extending the beam flexibility to the column centerline and defining the column as rigid within the joint.

4.3 Foundation Modeling

Base restraint can have a significant effect on the behavior of a moment frame. ASCE 7 - 12.7.1 (Foundation Modeling) states "for purposes of determining seismic loads, it is permitted to consider the structure to be fixed at the base. Alternatively, where foundation flexibility is considered, it shall be in accordance with Section 12.13.3 or Chapter 19." Therefore, the engineer has to decide the most appropriate analytical assumptions for the frame, considering its construction details. **Figure 4-1** illustrates four types of base restraint conditions that may be considered.

Modeling pinned restraints at the base of the columns, **Figure 4-1 (a)**, is typical for frames that do not extend through floors below grade. This assumption results in the most flexible column base restraint. The high flexibility will lengthen the period of the building, resulting in a lower calculated base shear but larger calculated drifts. Pinned restraints at the column bases will also simplify the design of the footing. Where pinned restraints have been modeled, dowels connecting the column base to the foundation need to be capable of transferring the shear and axial forces to the foundation.

One drawback to the pinned base condition is that the drift of the frame, especially the interstory drift in the lowest story, is more difficult to control within code-allowable limits. This problem is exacerbated because the first story is usually taller than typical stories. In addition, a pinned base may lead to development of soft or weak stories, which are prohibited in certain cases as noted in ASCE 7 - 12.3.3.1 and 12.3.3.2.

If the drift of the structure exceeds acceptable limits, then rotational restraint can be increased at the foundation by a variety of methods, as illustrated in **Figure 4-1 (b), (c), and (d)**. Regardless of which modeling technique is used, the base of the column and the supporting footing or grade beam must be designed and detailed to resist all the forces determined by the analysis, as discussed in Section 6.3.2. The foundation elements must also be capable of delivering the forces to the supporting soil.

ASCE 7 - 12.2.5.5 outlines requirements where special moment frames extend through below-grade floors, as shown in **Figure 4-2**. The restraint and stiffness of the below-grade diaphragms and basement walls needs to be considered. In this condition, the columns would be modeled as continuous elements down to the footing. The type of rotational restraint at the column base will not have a significant effect on the behavior of the moment frame. Large forces are transferred through the grade level diaphragm to the basement walls, which are generally very stiff relative to the special moment frame.

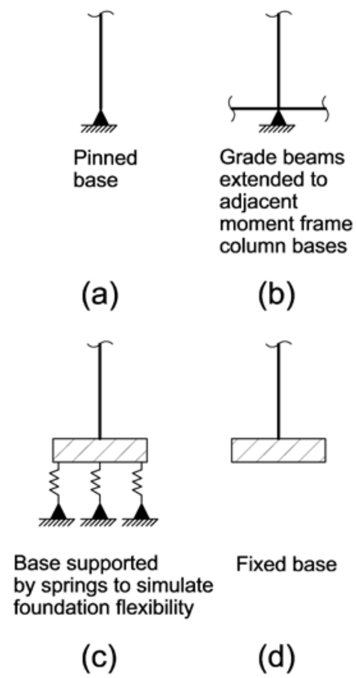


Figure 4-1 - Column base restraint conditions.

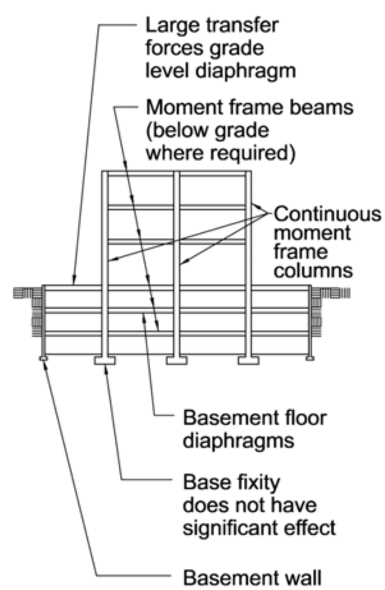


Figure 4-2 - Moment frame extending through floors below grade.

5. Design Guidance

5.1 Beam Flexure and Longitudinal Reinforcement

A capacity design approach is used to guide the design of a special moment frame. The process begins by identifying where inelastic action is intended to occur. For a special moment frame, this is intended to be predominantly in the form of flexural yielding of the beams. The building is analyzed under the design loads to determine the required flexural strengths at beam plastic hinges, which are almost always located at the ends of the beams. Beam sections are designed so that the reliable flexural strength is at least equal to the factored design moment, that is, $\phi M_n > M_u$.

Flexural Strength of Beams Cast Monolithically with Slabs

When a slab is cast monolithically with a beam, the slab acts as a flange, increasing the flexural stiffness and strength of the beam. ACI 318 is not explicit on how to account for this T-beam behavior in seismic designs, creating ambiguity, and leading to different practices in different design offices. One practice is to size the beam for the code required moment strength considering only the longitudinal reinforcement within the beam web. Another practice is to size the beam for this moment including developed longitudinal reinforcement within both the web and the effective flange width defined in ACI 318 - 8.12. Regardless of the approach used to initially size the beam, it is important to recognize that the developed flange reinforcement acts as flexural tension reinforcement when the beam moment puts the slab in tension. ACI 318 - 21.6.2.2 requires this slab reinforcement to be considered as beam longitudinal tension reinforcement for the purpose of calculating the relative strengths of columns and beams.

Probable Moment Strength, M_{pr}

The overstrength factor 1.25 is thought to be a low estimate of the actual overstrength that might occur for a beam. Reinforcement commonly used in the U.S. has an average yield stress about 15 percent higher than the nominal value (f_y), and it is not unusual for the actual tensile strength to be 1.5 times the actual yield stress. Thus, if a reinforcing bar is subjected to large strains during an earthquake, stresses well above 1.25 f_y are likely. The main reason for estimating beam flexural overstrength conservatively is to be certain there is sufficient strength elsewhere in the structure to resist the forces that develop as the beams yield in flexure. The beam overstrength is likely to be offset by overstrength throughout the rest of the building as well. The factor 1.25 in ACI 318 was established recognizing all these effects.

Once the beam is proportioned, the plastic moment strengths of the beam can be determined based on the expected material properties and the selected cross section. ACI 318 uses the probable moment strength M_{pr} for this purpose. Probable moment strength is calculated from conventional flexural theory considering the as-designed cross section, using $\phi = 1.0$, and assuming reinforcement yield strength equal to at least 1.25 f_y. The probable moment strength is used to establish requirements for beam shear strength, beam-column joint strength, and column strength as part of the capacity-design process. Because the design of other frame elements depends on the amount of beam flexural reinforcement, the designer should take care to optimize each beam and minimize excess capacity.

Besides providing the required strength, the flexural reinforcement must also satisfy the requirements illustrated in **Figure 5-1.** Although ACI 318 - 21.5.2.1 allows a reinforcement ratio up to 0.025, 0.01 is more practical for constructability and for keeping joint shear forces within reasonable limits.

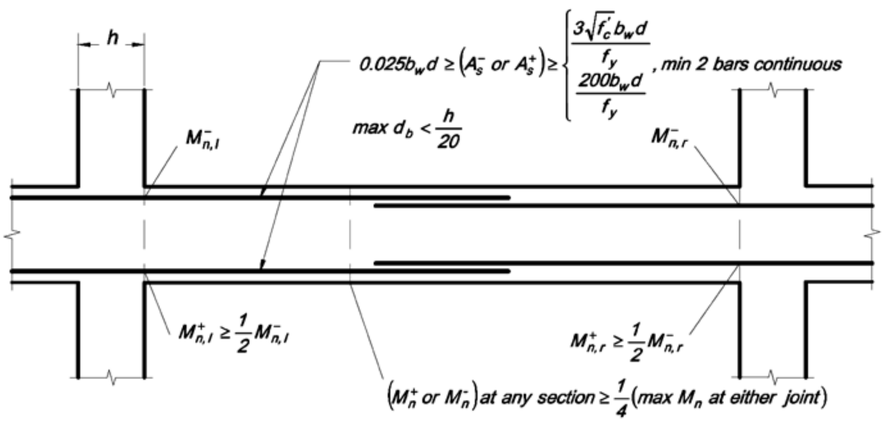


Figure 5-1 - Beam flexural reinforcement requirements.

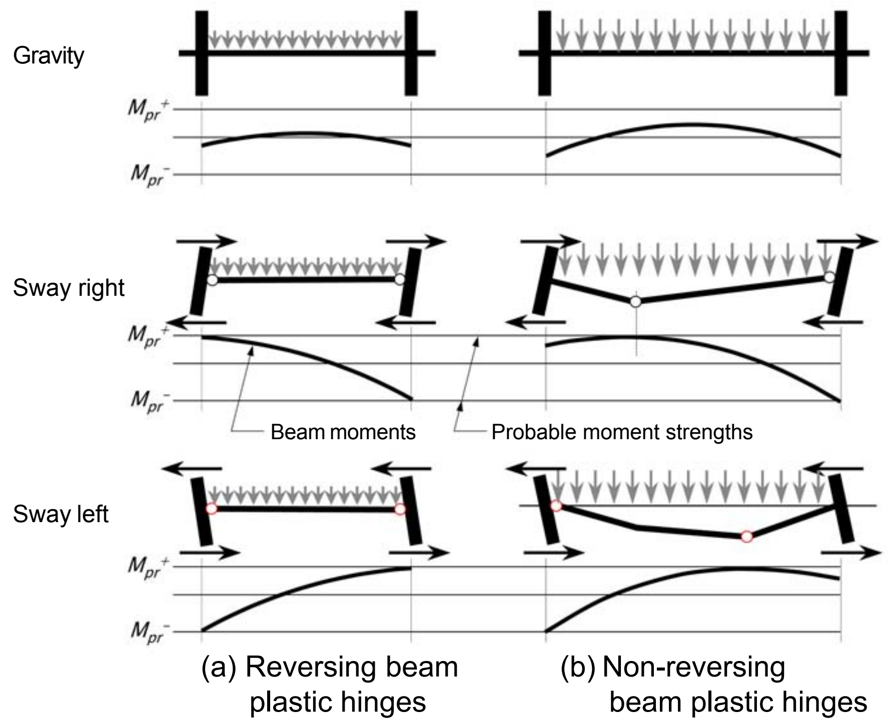


Figure 5-2 – (a) Reversing beam plastic hinges (preferred) tend to occur when spans are relatively short and gravity loads relatively low; (b) non-reversing plastic hinges (undesirable) tend to occur for longer spans or heavier gravity loads.

An objective in the design of special moment frames is to restrict yielding to specially detailed lengths of the beams. If the beam is relatively short and/or the gravity loads relatively low compared with seismic design forces, beam yielding is likely to occur at the ends of the beams adjacent to the beam-column joints, as suggested in **Figure 5-2(a)**. Where this occurs, the beam plastic hinges undergo reversing cycles of yielding as the building sways back and forth. This is the intended and desirable behavior.

In contrast, if the span or gravity loads are relatively large compared with earthquake forces, then a less desirable behavior can result. This is illustrated in **Figure 5-2(b)**. As the beam is deformed by the earthquake, the moments demands reach the plastic moment strengths in negative moment at the column face and in positive moment away from the column face. The deformed shape is shown. Upon reversal, the same situation occurs, but at the opposite ends of the beam. In this case, beam plastic hinges do not reverse but instead continue to build up rotation. This behavior results in progressively increasing rotations of the plastic hinges. For a long-duration earthquake, the rotations can be very large and the vertical movement of the floor can exceed serviceable values.

This undesirable behavior can be avoided if the beam probable moment strengths are selected to satisfy the following:

$$M_{pr}^{+} + M_{pr}^{-} \geq w_u \ell_n^2 / 2$$

This expression is valid for the common case where nearly equal moment strengths are provided at both ends and the moment strength does not change dramatically along the span. For other cases, the mechanism needs to be evaluated from first principles.

5.2 Joint Shear and Anchorage

Once the flexural reinforcement in the beams has been determined, the next design step is to check the joint shear in the beam-column joints. Joint shear is a critical check and will often govern the size of the moment frame columns.

To illustrate the procedure, consider a column bounded by two beams (**Figure 5-3**). As part of the frame design, it is assumed that the beams framing into the column will yield and develop their probable moment strengths at the column faces. This action determines the demands on the column and the beam-column joint.

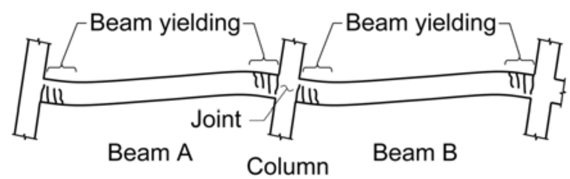
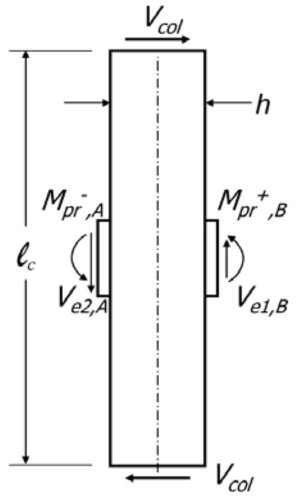


Figure 5-3 – The frame yielding mechanism determines the forces acting on the column and beam-column joint.



$$V_{col} = \left[(M_{pr}^{+},_B + M_{pr}^{-},_A) + (V_{e2,A} + V_{e1,B})\frac{h}{2}\right]/\ell_c$$

Figure 5-4 – Free body diagram of column used to calculate column shear V_{col}.

A free body diagram is made by cutting through the beam plastic hinges on both sides of the column and cutting through the column one-half story height above and below the joint as shown in **Figure 5-4**. In this figure, subscripts A and B refer to beams A and B on opposite sides of the joint, and $V_{e2,A}$ and $V_{e1,B}$ are shears in the beams at the joint face corresponding to development of M_{pr} at both ends of the beam (see Section 5.3.1 for discussion on how to calculate these shears). For a typical story, the column midheight provides a sufficiently good approximation to the point of contraflexure; for a pin-ended column it would be more appropriate to cut the free body diagram through the pinned end.

Having found the column shear, V_{col}, the design horizontal joint shear V_j is obtained by equilibrium of horizontal forces acting on a free body diagram of the joint as shown in **Figure 5-5**. Beam longitudinal reinforcement is assumed to reach a force at least equal to $1.25A_sf_y$. Assuming the beam to have zero axial load, the flexural compression force in the beam on one side of the joint is taken equal to the flexural tension force on the same side of the joint.

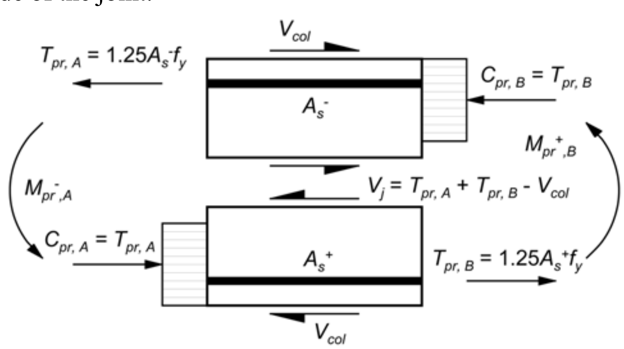


Figure 5-5 - Joint shear free body diagram.

It is well established that for monolithic construction, the slab longitudinal reinforcement within an effective width also contributes to the beam flexural strength. Although not required by ACI 318, ACI 352-02 recommends including the slab reinforcement within this effective width in the quantity A_s used to calculate the joint shear force. Except for exterior and corner connections without transverse beams, the effective width in tension is to be taken equal to the width prescribed by ACI 318 - 8.12 for the effective flange width in compression. For corner and exterior connections without transverse beams, the effective width is defined as the beam width plus a distance on each side of the beam equal to the length of the column cross section measured parallel to the beam generating the shear.

The design strength ϕV_n is required to be at least equal to the required strength V_j shown in **Figure 5-5**. The design strength is defined as

$$\phi V_n = \phi\gamma\sqrt{f_c'}A_j$$

in which ϕ equals 0.85; A_j is the joint area defined in **Figure 5-6**; and γ is a strength coefficient defined in **Figure 5-7**.

Though **Figure 5-6** shows the beam narrower than the column, ACI 318 - 21.5.1 contains provisions allowing the beam to be wider than the column. The effective joint width, however, is not to be taken any larger than the overall width of the column as stated in ACI 318 - 21.7.4.

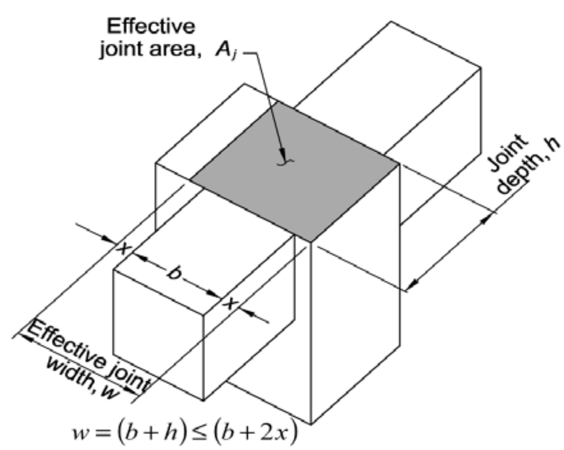


Figure 5-6 – Definition of beam-column joint dimensions.

The strength coefficients shown in **Figure 5-7** are from ACI 352-02. ACI 318 does not define different strengths for roof and typical floor levels but instead specifies using the typical values (upper row in **Figure 5-7**) for all levels. As shown, strength is a function of how many beams frame into the column and confine the joint faces. If a beam covers less than three quarters of the column face at the joint, it must be ignored in determining which coefficient γ applies.

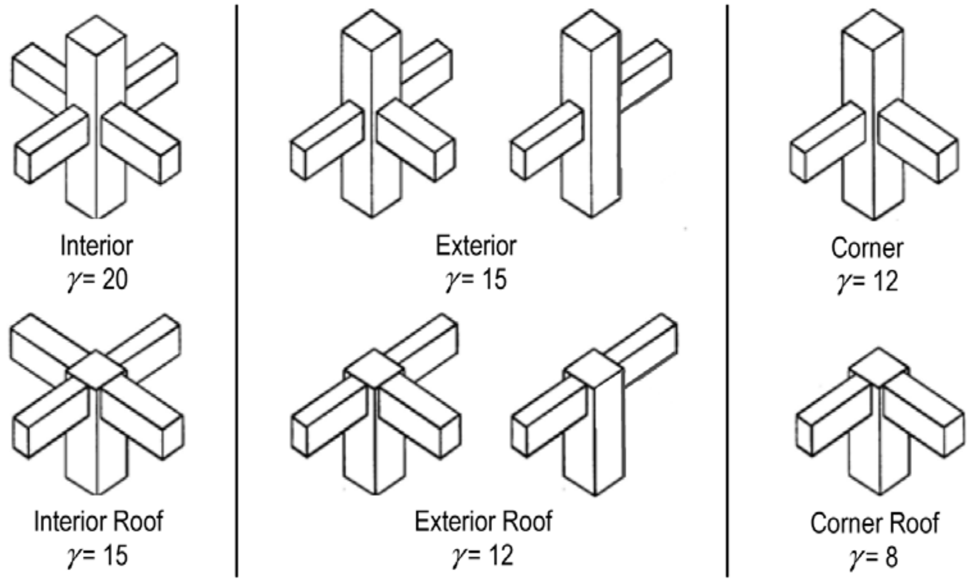


Figure 5-7 – Joint configurations and strength coefficients.

Detailing beam-column joints is an art requiring careful attention to several code requirements as well as construction requirements. **Figures 5-8** and **5-9** show example details for interior and exterior beam-column joints, respectively. Note that beam bars, possibly entering the joint from two different framing directions, must pass by each other and the column longitudinal bars. Joint hoop reinforcement is also required. Large-scale drawings or even physical mockups of beam-column joints should be prepared prior to completing the design so that adjustments can be made to improve constructability. This subject is discussed in more detail in Section 7.

It is important for beam and column longitudinal reinforcement to be anchored adequately so that the joint can resist the beam and column moments. Different requirements apply to interior and exterior joints. In interior joints, beam reinforcement typically extends through the joint and is anchored in the adjacent beam span. ACI 318 requires that the column dimension parallel to the beam longitudinal reinforcement be at least 20 longitudinal bar diameters for normal weight concrete (**Figure 5-8**). This requirement helps improve performance of the joint by resisting slip of the beam bars through the joint. Some slip, however, will occur even with this column dimension requirement

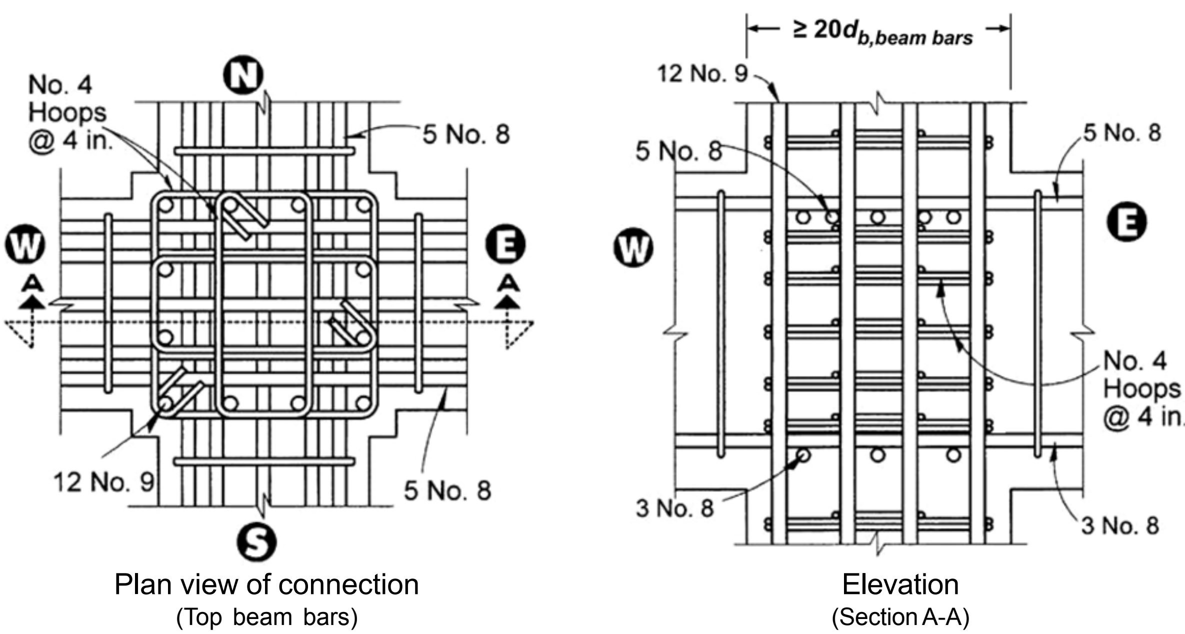


Figure 5-8 – Example interior joint detailing.

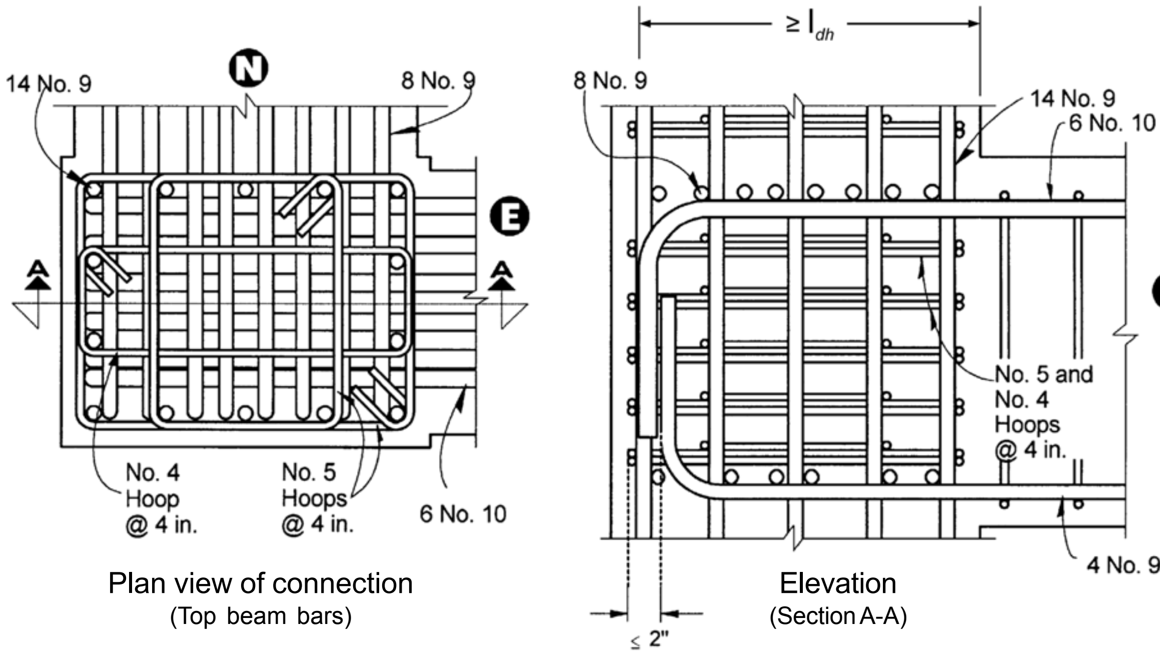


Figure 5-9 – Example exterior joint detailing.

ACI 352 recommends that the beam depth be at least 20 times the diameter of the column longitudinal reinforcement for the same reason. ACI 318 does not include this requirement.

For exterior joints, beam longitudinal reinforcement usually terminates in the joint with a standard hook (**Figure 5-9**). The tail of the hook must project toward the mid-depth of the joint so that a joint diagonal compression strut can be developed. The length for a standard 90° hook in normal-weight concrete must be the largest of 8 bar diameters, 6 inches, and the length required by the following expression:

$$\ell_{dh} = f_y d_b \Big/ 65\sqrt{f_c^{'}}$$

The latter expression almost always governs. This expression assumes that the hook is embedded in a confined beam-column joint. The expression applies only to bar sizes No. 3 through No. 11.

In addition to satisfying the length requirements of the previous paragraph, hooked beam bars are required to extend to the far side of the beam-column joint (ACI 318 - 21.7.2.2). This is to ensure the full depth of the joint is used to resist the joint shear generated by anchorage of the hooked bars. It is common practice to hold the hooks back an inch or so from the perimeter hoops of the joint to improve concrete placement.

Joint transverse reinforcement is provided to confine the joint core and improve anchorage of the beam and column longitudinal reinforcement. The amount of transverse hoop reinforcement in the joint is to be the same as the amount provided in the adjacent column end regions (see Section 5.4). Where beams frame into all four sides of the joint, and where each beam width is at least three-fourths the column width, then transverse reinforcement within the depth of the shallowest framing member may be relaxed to one-half the amount required in the column end regions, provided the maximum spacing does not exceed 6 inches.

5.3 Beam Shear and Transverse Reinforcement

5.3.1 Beam Design Shear

The beam design shear is determined using the capacity design approach as outlined in Section 3.2. **Figure 5-10** illustrates this approach applied to a beam. A free body diagram of the beam is isolated from the frame, and is loaded by factored gravity loads (using the appropriate load combinations defined by ASCE 7) as well as the moments and shears acting at the ends of the beam. Assuming the beam is yielding in flexure, the beam end moments are set equal to the probable moment strengths M_{pr} described in Section 5.1. The design shears are then calculated as the shears required to maintain moment equilibrium of the free body (that is, summing moments about one end to obtain the shear at the opposite end).

This approach is intended to result in a conservatively high estimate of the design shears. For a typical beam in a special moment frame, the resulting beam shears do not trend to zero near mid-span, as they typically would in a gravity-only beam. Instead, most beams in a special moment frame will have non-reversing shear demand along their length. If the shear does reverse along the span, it is likely that non-reversing beam plastic hinges will occur (see Section 5.1).

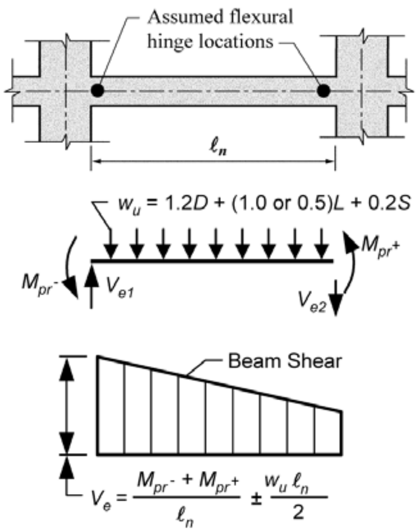


Figure 5-10 – Beam shears are calculated based on provided probable moment strengths combined with factored gravity loads.

Typical practice for gravity-load design of beams is to take the design shear at a distance d away from the column face. For special moment frames, the shear gradient typically is low such that the design shear at d is only marginally less than at the column face. Thus, for simplicity the design shear value usually is evaluated at the column face. Design for beam shear is outlined in Section 5.3.2.

5.3.2 Beam Transverse Reinforcement

Beams in special moment frames are required to have either hoops or stirrups along their entire length. Hoops fully enclose the beam cross section and are provided to confine the concrete, restrain longitudinal bar buckling, improve bond between reinforcing bars and concrete, and resist shear. Stirrups, which generally are not closed, are used where only shear resistance is required.

Beams of special moment frames can be divided into three different zones when considering where hoops and stirrups can be placed: the zone at each end of the beam where flexural yielding is expected to occur; the zone along lap-spliced bars, if any; and the remaining length of the beam.

The zone at each end, of length $2h$, needs to be well confined because this is where the beam is expected to undergo flexural yielding and this is the location with the highest shear. Therefore, closely spaced, closed hoops are required in this zone, as shown in **Figure 5-11**. Note that if flexural yielding is expected anywhere along the beam span other than the end of the beam, hoops must also extend $2h$ on both sides of that yielding location. This latter condition is one associated with non-reversing beam plastic hinges (see Section 5.1), and is not recommended. Subsequent discussion assumes that this type of behavior is avoided by design.

Hoop reinforcement may be constructed of one or more closed hoops. Alternatively, it may be constructed of typical beam stirrups with seismic hooks at each end closed off with crossties having 135° and 90° hooks at opposite ends. Using beam stirrups with crossties rather than closed hoops is often preferred for constructability so that the top longitudinal beam reinforcement can be placed in the field, followed by installation of the crossties. See **Figure 5-12** for additional detail requirements for the hoop reinforcement.

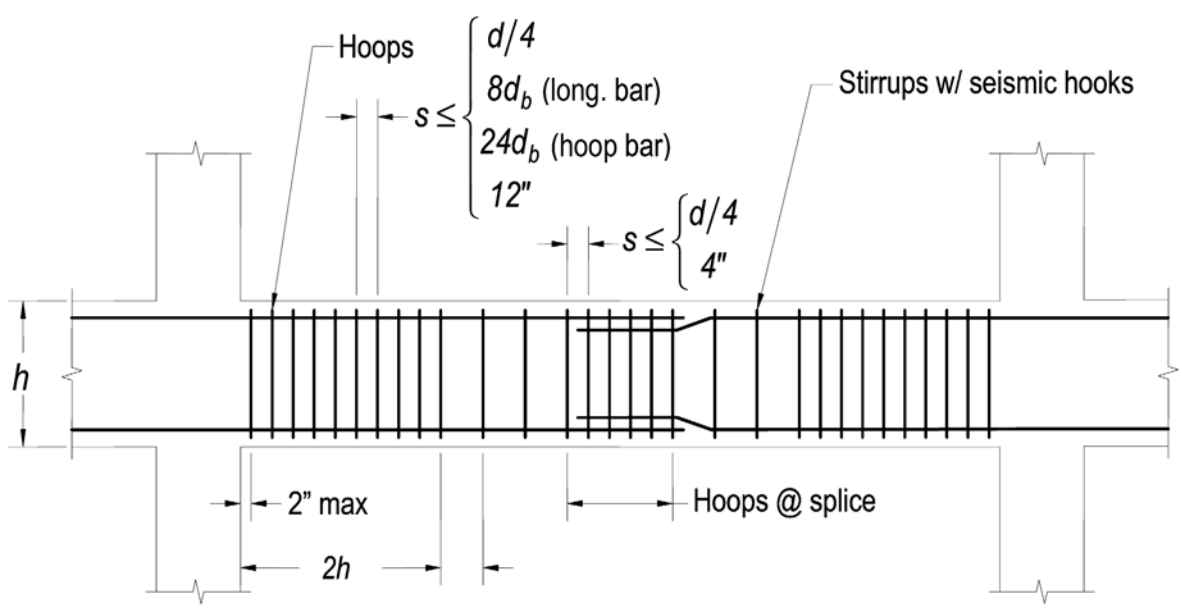


Figure 5-11 - Hoop and stirrup location and spacing requirements.

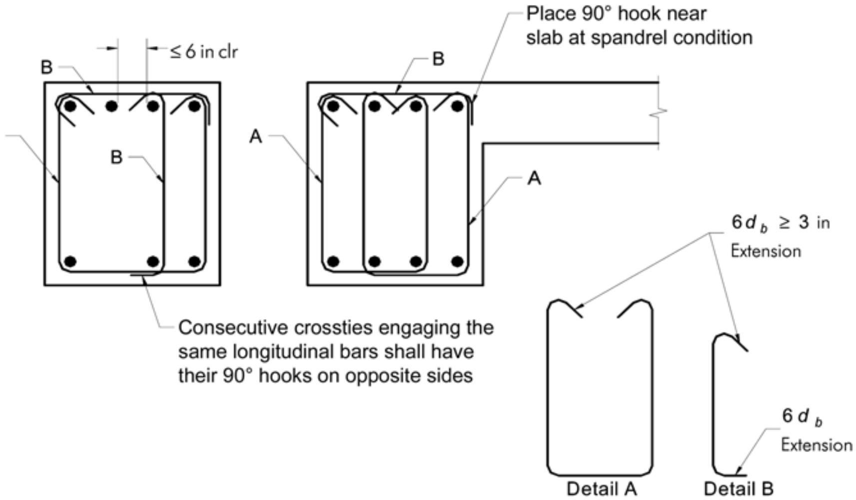


Figure 5-12 - Hoop reinforcement detail.

> **Placement of Hoops and Stirrups**
>
> Where hoops are being provided at each end of a beam and along a reinforcement splice, there may not be much length of the beam left where stirrups are acceptable. Because of this aspect, and to prevent placement errors, it is practical to extend the hoop detail and spacing over the entire length of the beam. A quick quantity comparison should be conducted to see the difference in the amount of detailed reinforcement. Both the weight of reinforcement and the number of pieces to be placed in the field affect the cost and should be considered when specifying the hoops and stirrups. If a design with hoops and stirrups with different configurations and spacing is specified, more rigorous observations need to be conducted by the engineer to ensure that ironworkers and special inspectors have a clear understanding of the placement requirements. These observations are most crucial early in the construction process when the first level of beams is constructed. Generally after the first level, the reinforcement pattern is properly replicated.

Wherever hoops are required, longitudinal bars on the perimeter must have lateral support conforming to ACI 318 - 7.10.5.3. This is to ensure that longitudinal bars are restrained against buckling should they be required to act in compression under moment reversals within potential flexural yielding locations.

When sizing the hoops in the end zones of a special moment frame beam, the shear strength of the concrete itself must be neglected (i.e., $V_c = 0$) except where specifically allowed per ACI 318 - 21.5.4.2. Thus, along the beam end zones, the shear design requirement typically is $\phi V_s \geq V_e$, where $\phi = 0.75$. Note that V_e is determined using capacity design as discussed in Section 5.3.1. Outside the end zones, design for shear is done using the conventional design equation $\phi (V_c + V_s) \geq V_e$.

If beam longitudinal bars are lap-spliced, hoops are required along the length of the lap, and longitudinal bars around the perimeter of the cross section are required to have lateral support conforming to ACI 318 - 7.10.5.3. Beam longitudinal bar lap splices shall not be used (a) within the joints; (b) within a distance of twice the member depth from the face of the joint; and (c) where analysis indicates flexural yielding is likely due to inelastic lateral displacements of the frame. Generally, if lap splices are used, they are placed near the mid-span of the beam. See **Figure 5-11** for hoop spacing requirements.

> Hoops are required along the beam end zones (where flexural yielding is expected) and along lap splices, with spacing limits as noted in **Figure 5-11**. Elsewhere, transverse reinforcement is required at a spacing not to exceed $d/2$ and is permitted to be in the form of beam stirrups with seismic hooks.

5.4 Column Design and Reinforcement

There are several strength checks associated with columns of a special moment frame. As a first approximation, the columns can be designed for the maximum factored gravity loads while limiting the area of reinforcement to between 1 % and 3 % of the gross cross-sectional area. ACI 318 allows the longitudinal reinforcement to reach 6 % of the gross section area, but this amount of reinforcement results in very congested splice locations. The use of mechanical couplers should be considered where the reinforcement ratio is in excess of 3 %.

Column Axial Load

Laboratory tests demonstrate that column performance is negatively affected by high axial loads. As axial loads increase, demands on the compressed concrete increase. At and above the balanced point, flexural yielding occurs by "yielding" the compression zone, which can compromise axial load-carrying capability. Although ACI 318 permits the maximum design axial load for a tied column as high as $0.80\phi P_o = 0.52P_o$, good design practice would aim for lower axial loads. It is recommended to limit the design axial load to the balanced point of the column interaction diagram.

Seismic forces acting on a moment frame generally do not make large contributions to the axial load at interior columns. Special attention should be given to the axial load in the exterior and corner columns because the seismic forces may be large in comparison with the gravity loads.

According to ACI 318 - 21.6.1, if the factored column axial load under any load combination exceeds $A_g f'_c/10$ in compression, the column must satisfy the strong-column/weak-beam requirement for all load combinations. As discussed in Section 3.1, this requirement is intended to promote an intermediate or beam yielding mechanism under earthquake load as illustrated in **Figure 3-1 (b) and (c)**. This requirement generally controls the flexural strength of the column.

To meet the strong-column/weak-beam requirement of ACI 318 - 21.6.2.2, the sum of the nominal flexural strengths, M_n, of the columns framing into a joint must be at least 1.2 times the sum of the nominal flexural strengths of the beams framing into the joint, as illustrated in **Figure 5-13**. It is required to include the developed slab reinforcement within the effective flange width (ACI 318 - 8.12) as beam flexural tension reinforcement when

Design versus Expected Column Axial Loads

Design axial loads are calculated using ASCE 7 load combinations, usually based on analysis of a linear-elastic model of the structure. During a strong earthquake, structural elements may respond nonlinearly, with internal forces different from those calculated using a linear model. For example, if the building shown in **Figure 3-1** developed a beam hinging mechanism over its entire height, every beam would develop the probable moment strength M_{pr}. This moment is higher than the design moment determined from the linear analysis and will generally lead to higher internal forces in other elements such as columns.

For the exterior column at the right side of **Figure 3-1(c)**, the axial load could be as high as the sum of the shears V_{e2} from the yielding beams over the height of the building (see **Figure 5-10**) plus the loads from the column self-weight and other elements supported by the column. Of course, there is no way to know if the full-height beam yielding mechanism will be realized, so there is no way to know with certainty how high the axial loads will be. Since high axial loads reduce column performance this is another reason why good design practice aims to keep design axial loads low.

computing beam strength. This check must be verified independently for sway in both directions (for example, East and West) and in each of the two principal framing directions (for example, EW and NS). When this flexural strength check is done, consideration needs to be given to the maximum and minimum axial loads in the column, because the column flexural strength is dependent on the axial load as shown in **Figure 5-14**. The load combinations shown in **Figure 5-14** are from ASCE 7 - 2.3.2 and 12.14.3.1.3. Refer to ASCE 7 for the live load factor requirements.

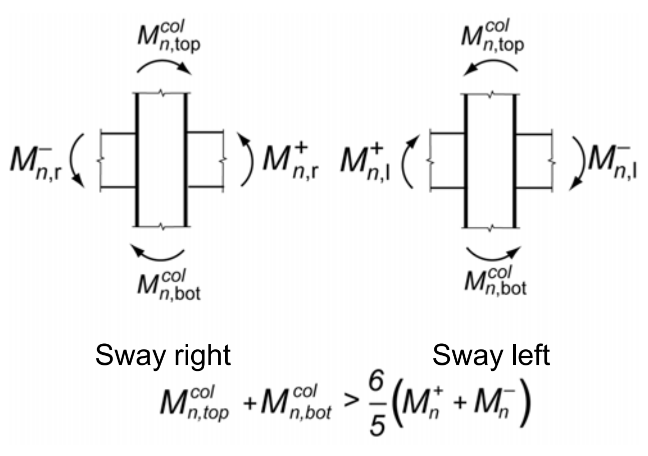


Figure 5-13 – Strong column/weak beam design moments.

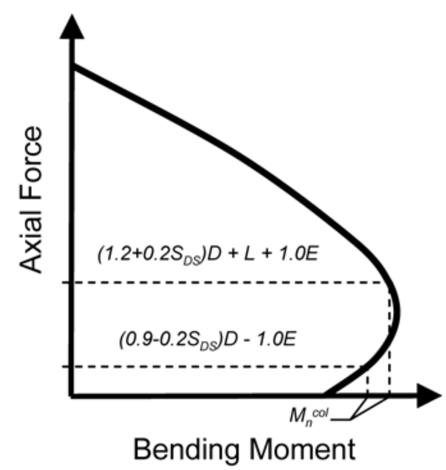


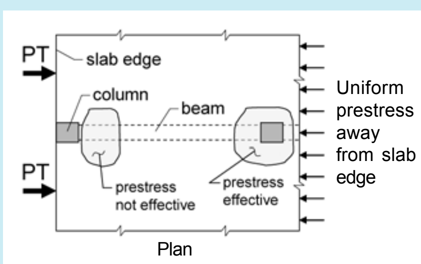
Figure 5-14 – Nominal column moments must be checked at maximum and minimum axial forces.

In some cases it may not be practical to satisfy the strong-column/weak-beam provisions for a small number of columns. The strength and stiffness of such columns cannot be considered as part of the special moment frame. These columns must also satisfy the requirements of ACI 318 - 21.13, that is, columns not designated as part of the seismic force-resisting system.

Strong-column / Weak-beam Check

The requirement that the column be stronger than the beam is important to avoid formation of story mechanisms such as the one illustrated in **Figure 3-1(a)**. ACI 318 requires that the contribution of the slab to flexural strength be considered in this case, especially including the contribution of the developed slab reinforcement within the effective flange width defined in Section 8.12.

A common construction form in modern buildings uses unbonded post-tensioned slabs cast monolithically with conventionally reinforced beams. Placing the unbonded strands outside the effective flange width does not mean those strands do not contribute to beam flexural strength. This is because, away from the slab edge, the post tensioning produces a fairly uniform compressive stress field across the plate including the beam cross section (see sketch).



A reasonable approach is to calculate the average prestress acting on the combined slab-beam system, then apply this prestress to the T-beam cross section to determine the effective axial compression on the T-beam. This axial load, acting at the level of the slab, is used along with the beam longitudinal reinforcement to calculate the T-beam flexural strength. This recommendation applies only for interior connections that are far enough away from the slab edge so as to be fully stressed by the post-tensioning. It need not apply at an exterior connection close to the slab edge because the post-tensioning will not effectively compress the beam at that location.

The transverse column reinforcement will vary over the column length, as illustrated in **Figure 5-15**. Longitudinal bars should be well distributed around the perimeter. Longitudinal bar lap splices, if any, must be located along the middle of the clear height and should not extend into the length ℓ_0 at the column ends. Such lap splices require closely spaced, closed hoops along the lap length. Closely spaced hoops are also required along the length ℓ_0 measured from both ends, to confine the concrete and restrain longitudinal bar buckling in case column flexural yielding occurs. Along the entire length, shear strength must be sufficient to resist the design shear forces, requiring hoops at maximum spacing of $d/2$, where d is commonly taken equal to 0.8 times the column cross-sectional dimension h.

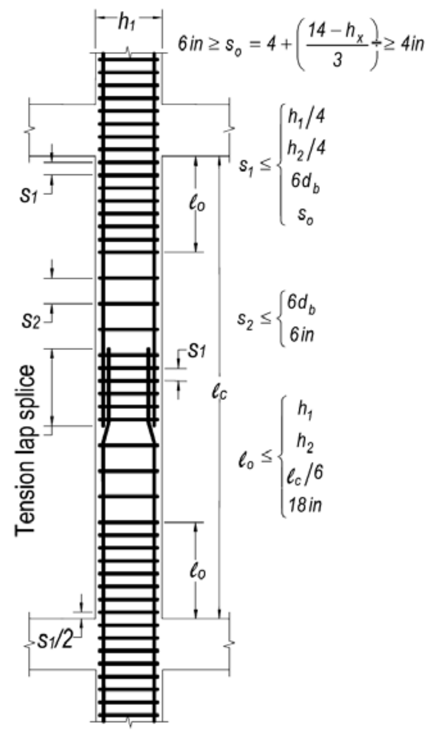


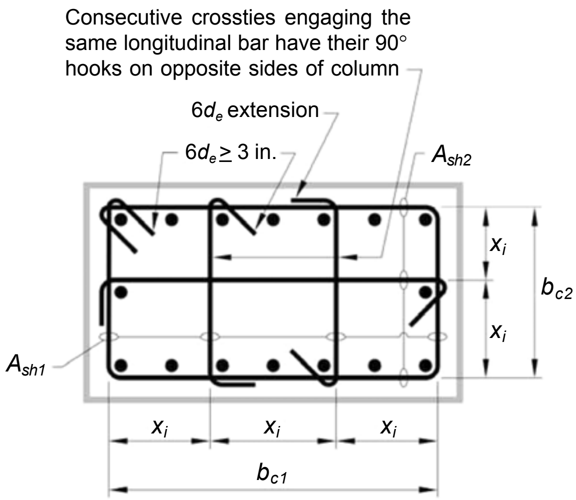
Figure 5-15 – Column transverse reinforcement spacing requirements.

The column transverse reinforcement should initially be selected based on the confinement requirements of ACI 318 - 21.6.4. For rectangular cross sections, the total cross-sectional area of rectangular hoop reinforcement is not to be less than that required by either of the following two equations, whichever gives the larger amount.

$$A_{sh} = 0.3(sb_c f'_c / f_{yt})[(A_g / A_{ch}) - 1]$$

$$A_{sh} = 0.09 sb_c f'_c / f_{yt}$$

Both of these equations must be checked in both principal directions of the column cross section. Thus, as illustrated in **Figure 5-16**, to determine total hoop leg area A_{sh1}, the dimension b_{c1} is substituted for b_c in each of these two equations, while to determine A_{sh2}, dimension b_{c2} is used.

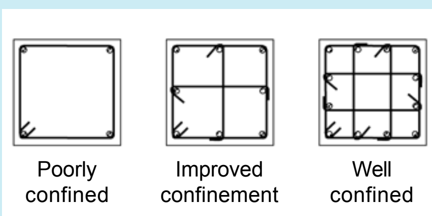


The dimension x_i from centerline of centerline to tie legs is not to exceed 14 inches. The term h_x is taken as the largest value of x_i.

Figure 5-16 – Column transverse reinforcement detail.

Hoop Configuration

Column hoops should be configured with at least three hoop or crosstie legs restraining longitudinal bars along each face. A single perimeter hoop without crossties, legally permitted by ACI 318 for small column cross sections, is discouraged because confinement effectiveness is low.



Once the transverse reinforcement is selected, the shear strength of the section needs to be checked. ACI 318 is ambiguous about the shear design requirements, reflecting the uncertainty that remains about what is an adequate minimum design requirement. Three distinct procedures for calculating design shear are given. The column design shear is defined as the larger of the shear from procedure **a** and the shear from either procedure **b** or procedure **c**. These are summarized below.

a. According to ACI 318 - 21.6.5.1, V_e shall not be taken less than the shear obtained by analysis of the building frame considering the governing design load combinations. See **Figure 5-17(a)**. For reference in subsequent paragraphs, this shear will be denoted V_{code}.

b. V_e can be determined using the capacity design approach as illustrated in **Figure 5-17(b)**. As with beams, M_{pr} is calculated using strength reduction factor $\phi = 1.0$ and steel yield stress equal to at least 1.25 f_y. Furthermore, M_{pr} is to be taken equal to the maximum value associated with the anticipated range of axial forces. As shown in **Figure 5-18**, the axial force under design load combinations ranges from P_{u1} to P_{u2}. The moment strength is required to be taken equal to the maximum moment strength over that range of axial loads.

This approach is considered to be conservative because, barring some unforeseeable accidental loading, no higher shear can be developed in the column. This approach is recommended where feasible. For some columns, however, the shear obtained by this approach is much higher than can reasonably be accommodated by transverse reinforcement, and much higher than anticipated shears, so alternative **c** is offered in ACI 318 - 21.6.5.1.

c. By this alternative column design shear can be taken equal to the shear determined from joint strengths based on M_{pr} of the beams framing into the joint. See **Figure 5-17(c)**. The concept behind this approach is that the column shears need

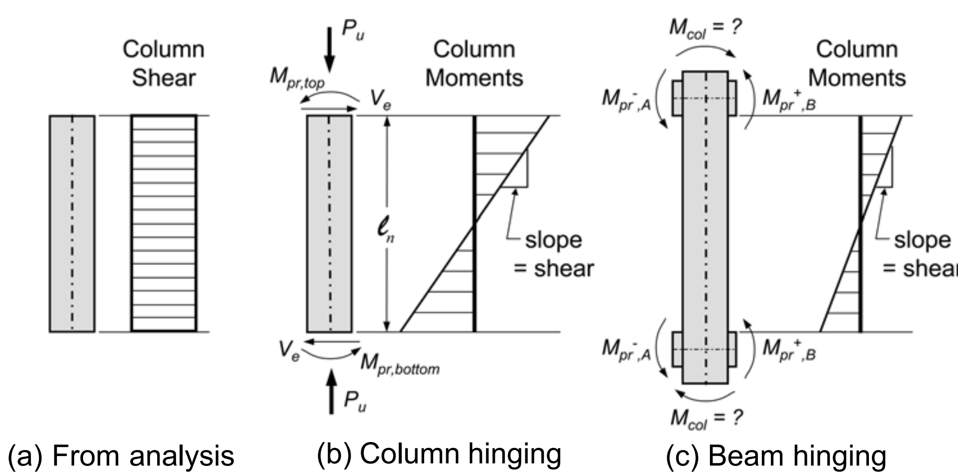


Figure 5-17 – Column shear calculation options.

not be taken as any greater than the shear that develops when the beams develop their probable moment strengths in the intended beam-yielding mechanism. The problem with this approach is that the distribution of column resisting moments above and below the joint is indeterminate. A common assumption is to distribute the moments to the columns in proportion with the column flexural rigidity. *Analytical studies have shown this approach can be unconservative by a wide margin, so it is not recommended here*.

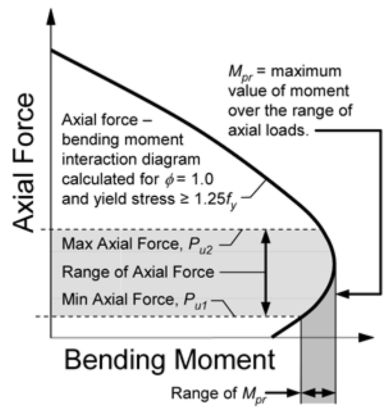


Figure 5-18 – To find M_{pr} for a column, first determine the range of axial loads under design load combinations. M_{pr} is the largest moment for that range of axial loads.

Column Tie Spacing

Similar to the discussion on beam hoops and stirrups, when a lap splice of the vertical column reinforcement is present, there is often not much space left to take advantage of the more relaxed column tie spacing outside the ℓ_0 regions shown in **Figure 5-15**. For this reason, it is common practice to specify a uniform tie spacing to prevent misplaced ties during construction. Where bars are not spliced at every floor, perhaps every other floor, more economy can be realized by specifying a larger spacing between the ℓ_0 regions. The benefit can be seen by simply counting the number of ties that can be saved as the spacing is relaxed.

This guide recommends an alternative way to apply procedure **c**. First, determine the column shear V_{code} as defined for procedure **a**. V_{code} might be a reasonable estimate of the true shear forces if the frame was proportioned with strengths exactly corresponding to the design requirements. Actual beam flexural strengths likely exceed the minimum requirements because of section oversizing, materials overstrength, and other design conservatism. If beams develop average moment strengths M_{pr}, compared with average design moment strengths M_u, it is reasonable to anticipate shear forces reaching values equal to $M_{pr}/M_u \times V_{code}$. This is the shear force recommended for the column design by procedure **c**.

This shear design approach thus simplifies to the following: V_e, is either (1) the shear obtained by procedure **b**, or (2) the shear obtained by the modified procedure **c** as described in the preceding paragraph.

The design shear strength for the column is $\phi\,(V_c + V_s) \geq V_e$, with $\phi = 0.75$. V_c must be set to zero over the length of ℓ_0, shown in **Figure 5-15**, for any load combination for which the column has low axial load ($< A_g f'_c/20$) and high seismic shear demand ($V_e = V_u/2$). Note that both of these conditions must occur to require $V_c = 0$. In Seismic Design Categories D, E, and F, V_e will be the dominant force.

According to ACI 318 - 21.1.2.3, if columns of a special moment frame extend below the base of the structure as shown in **Figure 4-2**, and those columns are required to transmit forces resulting from earthquake effects to the foundation, then those columns must satisfy the detailing and proportioning requirements for columns of special moment frames. In most conditions, the columns of a special moment frame will be carrying seismic forces over their entire height, and providing full-height ductile detailing is required.

Where a column frames into a strong foundation element or wall, such that column yielding is likely under design earthquake loading, a conservative approach to detailing the confinement reinforcement is warranted. ACI 318 refers to this condition in the commentary to Section 21.6.4.1. It is recommended that the length of the confinement zone be increased to $1.5\ell_0$. Tests have shown that 90° bends on crossties tend to be less effective than 135° bends for yielding columns with high axial loads. The 90° bend on crossties at this location should be avoided if the axial load on the column is above the balanced point. Alternatively, double crossties can be used so there is a 135° bend at each end, though this may create construction difficulties.

At the roof location the axial demand in the columns is generally low. If the axial demand is less than or equal to $A_g f'_c/10$, the strong column/weak provisions are not required. Therefore, it is more likely to develop a hinge at the top of a column just below the roof. For this case, a column must satisfy the requirements of Section 21.5 for flexural members of special moment frames. Accordingly, the length of ℓ_0 shown in **Figure 5-13** needs to be extended to twice the maximum column dimension, h. At the top of the column the longitudinal bars must also be hooked toward the center of the column to allow for diagonal compression struts to be developed within the joint.

6. Additional Requirements

6.1 Special Inspection

Reinforced concrete special moment frames are complex structural elements whose performance depends on proper implementation of design requirements during construction. Therefore, wherever a special moment frame is used, regardless of the Seismic Design Category, the IBC requires continuous inspection of the placement of the reinforcement and concrete by a qualified inspector. The inspector shall be under the supervision of the licensed design professional responsible for the structural design or under the supervision of a licensed design professional with demonstrated capability for supervising inspection of construction of special moment frames. Continuous special inspection generally is interpreted to mean that the special inspector is on the site at all times observing the work requiring special inspection.

Generally, the special inspector is required to observe work assigned for conformance to the approved design drawings and specifications. Contract documents specify that the special inspector will furnish inspection reports to the building official, the licensed design professional, or other designated persons. Discrepancies are to be brought to the immediate attention of the contractor for correction, then, if uncorrected, to the proper design authority and the building official. A final signed report is to be submitted stating whether the work requiring special inspection was, to the best of the inspector's knowledge, in conformance with the approved plans and specifications and the applicable workmanship provisions of the IBC.

6.2 Material Properties

Wherever a special moment frame is used, regardless of the Seismic Design Category, ACI 318 stipulates that materials shall conform to special requirements. These requirements are intended to result in a special moment frame capable of sustaining multiple inelastic deformation cycles without critical degradation.

6.2.1 Concrete

According to ACI 318 - 21.1.4.2, the specified compressive strength of concrete, f'_c, shall be not less than 3,000 psi. Additional requirements apply where lightweight concrete is used; the reader is referred to ACI 318 for these requirements. Where high-strength concrete is used, the value of $\sqrt{f'_c}$ is restricted to an upper-bound value of 100 psi for any shear strengths or anchorage/development strengths derived from Chapters 11 and 12 of ACI 318. The limit does not apply to beam-column joint shear strength or to development of bars at beam-column joints, as covered by ACI 318 - 21.7. Beam-column joint shear strengths calculated without the 100 psi limit were conservative for laboratory tests having concrete compressive strengths up to 15,000 psi (ACI 352). Based on local experiences, some jurisdictions impose additional restrictions on the use of high-strength concrete.

6.2.2 Reinforcement

Inelastic flexural response is anticipated for special moment frames subjected to design-level earthquake shaking. ACI 318 aims to control the flexural strength and deformability of yielding regions by controlling the properties of the longitudinal reinforcement. It is important that the reinforcement yield strength meet at least the specified yield strength requirement, and also that the actual yield strength not be too much higher than the specified yield strength. If it is too much higher, the plastic moment strength of yielding members will be higher than anticipated in design, resulting in higher forces being transmitted to adjacent members as the intended yield mechanism forms. Therefore, ACI 318 requires reinforcement to meet the specified yield strength and that the actual yield strength not exceed the specified yield strength by a large margin.

Additionally, it is important that flexural reinforcement strain harden after yielding so that inelastic action will be forced to spread along the length of a member. Therefore, ACI 318 also requires that strain hardening meet specified requirements.

According to ACI 318, deformed reinforcement resisting earthquake-induced flexural and axial forces in frame members must conform with the American Society for Testing and Materials (ASTM) publication ASTM A706. According to this specification, the actual yield strength must not exceed the specified yield strength by more than 18,000 psi, and the ratio of the actual tensile strength to the actual yield strength must be at least 1.25. A706 also has excellent strain ductility capacity and chemical composition that makes it more suitable for welding. Alternatively, ASTM A615 Grades 40 and 60 reinforcement are permitted by ACI 318 if (a) the actual yield strength based on mill tests does not exceed f_y by more than 18,000 psi; and (b) the ratio of the actual tensile strength to the actual yield strength is not less than 1.25. The optional use of A615 reinforcement sometimes is adopted because A615 reinforcement may be more widely available in the marketplace and may have lower unit cost.

Market forces and construction efficiencies sometimes promote the use of higher yield strength longitudinal reinforcement (for example, Grade 75). This reinforcement may perform suitably if the elongation and stress requirements match those of A706 reinforcement. Higher strength reinforcement results in higher unit bond stresses and requires longer development and splice lengths.

Even higher-strength reinforcement, up to 100-ksi nominal yield strength, is permitted to be used for transverse reinforcement. This reinforcement can reduce congestion problems especially for large members using higher strength concrete. Where used, the value of f_{yt} used to compute the amount of confinement reinforcement shall not exceed 100,000 psi, and the value of f_{yt} used in design of shear reinforcement shall conform to ACI 318 - 11.4.2 (that is, the maximum value is 60,000 psi except 80,000 psi is permitted for welded deformed wire reinforcement).

6.2.3 Mechanical Splices

Longitudinal reinforcement in special moment frames is expected to undergo multiple yielding cycles in prescribed locations during design-level earthquake shaking. If mechanical splices are used in these locations, they should be capable of developing nearly the tensile strength of the spliced bars. Outside yielding regions, mechanical splices, if used, can have reduced performance requirements.

According to ACI 318, mechanical splices shall be classified as either Type 1 or Type 2 mechanical splices, as follows: (a) Type 1 mechanical splices shall conform to ACI 318 - 12.14.3.2, that is, they shall be capable of 1.25 f_y in tension or compression, as required; (b) Type 2 mechanical splices shall develop the specified tensile strength of the spliced bar.

Where mechanical splices are used in beams or columns of special moment frames, only Type 2 mechanical splices are permitted within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacements. Either Type 1 or Type 2 mechanical splices are permitted in other locations.

6.2.4 Welding

Special moment frames are anticipated to yield when subjected to design-level earthquake ground motions, so special care is required where welding is done. Welded splices in reinforcement resisting earthquake-induced forces must develop at least 1.25 f_y of the bar and shall not be used within a distance equal to twice the member depth from the column or beam face or from sections where yielding of the reinforcement is likely to occur.

Welding of stirrups, ties, inserts, or other similar elements to longitudinal reinforcement that is required by design is not permitted because cross-welding can lead to local embrittlement of the welded materials. (The small added bars shown in **Figure 7-2** are an example of reinforcement that is not required by design; generally such bars should be of small diameter so as to not materially affect flexural response.) Welded products should only be used where test data demonstrate adequate performance under loading conditions similar to conditions anticipated for the particular application.

6.3 Additional System Design Requirements

Where special moment frames are used, certain other requirements of the code must be followed. In some cases these additional requirements apply only in Seismic Design Categories D, E, and F.

6.3.1 Structural Diaphragms

ACI 318 - 21.11 presents requirements for diaphragms that are applicable wherever a special moment frame is used in Seismic Design Category D, E, or F. For elevated diaphragms in frames without vertical irregularities, the diaphragm forces are predominantly associated with transferring inertial forces from the diaphragm to the special moment frames. ASCE 7 contains requirements for determining these diaphragm forces. For elevated diaphragms in dual systems, or in buildings with vertical irregularities, the diaphragm also resists forces associated with interaction among the different elements of the lateral-force-resisting system. For buildings with a podium level (that is, widened footprint at the base or in the bottom-most stories), such as shown in **Figure 4-2**, the diaphragm serves to transmit the seismic forces from the special moment frames to the basement walls or other stiff elements of the podium. Collectors of diaphragms must be designed for forces amplified by the factor Ω_o intended to account for structural overstrength of the building.

6.3.2 Foundations

ACI 318 - 21.12.1 presents requirements for foundations that are applicable wherever a special moment frame is used in Seismic Design Category D, E, or F. This includes specific requirements for the foundation elements (footings, foundation mats, pile caps, grade beams, etc.) as well as requirements for longitudinal and transverse reinforcement of columns framing into these foundation elements.

Where grade beams connect adjacent column bases, the longitudinal and transverse reinforcement must meet the requirements of ACI 21.5 as described earlier is Sections 5.1 and 5.3.

6.3.3 Members Not Designated as Part of the Seismic Force-resisting System

Section 2 of this guide described the progression of building design practices from the early days, when special moment frames were used in most framing lines, to more recent practices, in which special moment frames are used in a few framing lines with the remainder of the structural framing not designated as part of the seismic force-resisting system. Sometimes referred to as “gravity-only systems,” those parts of the building not designated as part of the seismic force-resisting system need to be capable of safely supporting gravity loads as they are subjected to the drifts and forces generated as the building sways under the design earthquake ground motions. Failure to provide this capability has resulted in building collapses in past earthquakes.

Where special moment frames are used as part of the seismic force-resisting system in Seismic Design Category D, E, or F, it is required to satisfy requirements of ACI 318 - 21.13, titled Members Not Designated as Part of the Seismic-Force-Resisting System. These requirements apply to columns, beams, beam-column connections, and slab-column connections of "gravity-only systems." In some cases, the requirements approach those for the special moment frame that serves as part of the primary seismic force-resisting system. In some cases, it may prove more economical, and may improve performance, to spread the seismic force resistance throughout the building rather than concentrating it in a few specially designated elements.

7. Detailing and Constructability Issues

A special moment frame relies on carefully detailed and properly placed reinforcement to ensure that it can maintain its strength through multiple cycles beyond the yield deformation. Architectural requirements often push to get the beams and columns as small as possible, resulting in beams, columns, and joints that become very congested. Early in the design process, it is important to ensure that the required reinforcement not only fits within the geometric confines of the elements, but also can be properly placed in the field.

The text that follows is based on construction experiences, both good and bad, and draws from Wyllie and LaPlante (2003).

7.1 Longitudinal Bar Compatibility

When laying out the beam and column reinforcement, it is helpful to establish planes of reinforcement for the longitudinal steel. The column longitudinal bars are located around the perimeter of the column cross section, establishing vertical planes of reinforcement for the column. The beam longitudinal reinforcement within the width of the column must pass between these planes. Horizontal planes are created with the top and bottom beam longitudinal reinforcement. With orthogonal beams framing into the same joint, there are four horizontal planes, two at the top and two at the bottom. As all these planes need to extend through the beam-column joint, they cannot overlap. **Figure 7-1** shows a well coordinated joint with three beam bars passing though a column face that has four vertical bars.

Figure 7-1 – Beam-column joint with beam corner bars swept to inside of column corner bars.

The beam-column joint is the critical design region. By keeping the column and beam dimensions large, beam and column longitudinal reinforcement ratios can be kept low and beam-column joint volumes kept large so that joint shear stresses are within limits. Large joints with low reinforcement ratios also help with placement of reinforcing bars and concrete.

Beams and columns always need bars close to their faces and at corners to hold the stirrups or ties. When the beam and column are the same width, these bars are in the same plane in the beam and the column, and they conflict at the joint. This requires bending and offsetting one set of bars, which will increase fabrication costs. Offsetting the bars can also create placement difficulties and results in bar eccentricities that may affect ultimate performance. If the beam is at least 4 inches wider or narrower than the column (2 inches on each side), the bars can be detailed so that they are in different planes and thus do not need to be offset.

One option, pictured in **Figure 7-1**, is to gently sweep the corner beam bars to the inside of the column corner bars. This will work if the hoops are detailed as stirrups with a cap tie. Near the column, the corner beam bars will be closer together and the vertical legs of the stirrups are usually flexible enough that they can be pulled over to allow the corner bar to be placed within the 135° hooks. Corner bars will not fit tightly within the bends of the cap tie, but the hook extensions of the 135° hooks are normally long enough so they are still anchored into the core of the beam. One might consider using 135° hooks at both ends of the cap tie to improve the anchorage into the core of the beam.

Figure 7-2 – Beam-column joint with small diameter corner bars.

Another option, pictured in **Figure 7-2**, is to provide a smaller, discontinuous bar to support the stirrups at the edge of the beam. This requires additional reinforcement that is not contributing to the strength of the moment frame and requires more pieces to be placed. The added reinforcement should be of small diameter so it does not create a large discontinuity in flexural strength of the beam in the potential plastic hinge region.

Making the beam wider or narrower than the column may create undesirable conditions along the exterior edge of a floor and may increase forming costs for both exterior and interior framing locations. Consideration needs to be given to the architectural condition along this exterior location. Even though different beam and column widths work well for the structure, this may create a complicated enclosure detail that is more costly.

Figure 7-3 – Beam-column joint having multiple layers of beam reinforcement hooked at back side of joint. Note the upturned beam (the slab is cast at the bottom face of the moment frame beam).

To support the beam hoops and stirrups, some of the top bars must be made continuous with lap splices or mechanical couplers near mid-span. To meet the negative moment requirements, shorter bars passing though the column can be added to the continuous top bars.

Multiple layers of longitudinal bars should usually be avoided where possible, as this condition makes placement very difficult, especially when two or more layers of top bars must be hooked down into the joint at an exterior column. If more than one layer of bars is required, it may be because the beam is too small; if this is the case, enlarging the beam is recommended, if possible. This situation also occurs where lateral resistance is concentrated in a few moment frames, requiring large, heavily reinforced beams (**Figure 7-3**)

7.2 Beam and Column Confinement

Confinement of beams and columns is crucial to the ductile performance of a special moment frame. Usually confinement is provided by sets of hoops or hoops with crossties. Several examples are shown in the figures of this section.

As shown in **Figure 5-16**, hoops are required to have 135° hooks; crossties are permitted to have a 135° hook at one end and a 90° hook at the other end, provided the crossties are alternated end for end along the longitudinal axis of the member (as shown in several photographs in this section). The 135° hooks are essential for seismic construction; alternating 135° and 90° hooks is a compromise that improves constructability. The concrete cover on beams and columns may spall off during response to the ground shaking, exposing the stirrup and tie hooks. A 90° hook can easily be bent outward from internal pressure. If this happens, the stirrup or tie will lose its effectiveness. In contrast, a 135° hook will remain anchored in the core of the member when the concrete cover spalls. There is no real cost premium for 135° hooks and their performance in extreme loadings is superior to 90° hooks.

Another option besides crossties with hooks is to use headed reinforcement (that is, deformed reinforcing bars with heads attached at one or both ends to improve bar anchorage). It is important to ensure that the heads are properly engaged. Special inspection of their final placement is very important. Yet another option is to use continuously bent hoops, that is, hoops constructed from a single piece of reinforcement (**Figure 7-4**). Whereas these hoops can result in reinforcement cages with excellent tolerances, the pre-bent shape limits field adjustments that may be required when interferences arise.

As shown in **Figure 5-16**, ACI 318 permits the horizontal spacing between legs of hoops and crossties to be as large as 14 inches in columns. Confinement can be improved by reducing this spacing. It is recommended that longitudinal bars be spaced around the perimeter no more than 6 or 8 inches apart. According

Figure 7-4 – Column cage with hoops constructed from single reinforcing bar.

to ACI 318 - 21.6.4.3, vertical spacing of hoop sets can be increased from 4 inches to 6 inches as horizontal spacing of crosstie legs decreases from 14 inches to 8 inches. The extra vertical spacing can reduce the total number of hoop sets and facilitate working between hoop sets. Because a typical hoop set comprises a three-layer stack of bars (crossties in one direction, then the hoop, then the crossties in the other direction), the actual clear spacing between hoop sets can be quite small. The ties and stirrups should be kept to #4 or #5 bars. Number 6 and larger bars have very large diameter bends and are difficult to place.

Although spirally reinforced columns are not treated in detail in this guide, it can be noted that they are more ductile than columns with ties and are therefore better for extreme seismic loads. The spirals need to be stopped below the beam-column joint because it is very difficult, if not impossible, to integrate the spirals with the longitudinal beam reinforcement. Because transverse reinforcement is required to extend through the joint per ACI 318 - 21.7.3, the spirals can be replaced within the joint by circular hoop reinforcement.

7.3 Bar Splices

Lap splices of longitudinal reinforcement must be positioned outside intended yielding regions, as noted in Sections 5.3 and 5.4. Considering that column and beam ends, as well as lap splice lengths, all require closely spaced hoops, it commonly becomes simpler to specify closely spaced hoops along the entire beam or column length. This is especially common for columns.

Large diameter bars require long lap splices. In columns, these must be detailed so they do not extend outside the middle half of the column length and do not extend into the length ℓ_0 at the end of the column. If longitudinal bars are offset to accommodate the lap splice, the offset also should be outside the length ℓ_0 (**Figure 7-5**).

Lap splices of the longitudinal reinforcement create a very congested area of the column as the number of vertical bars is doubled and the hoops must be tightly spaced. Splicing the vertical bars at every other floor as shown in **Figure 7-6** will eliminate some of the congestion. Mechanical splices also may help reduce congestion.

7.4 Concrete Placement

Regardless of the effort to make sure the reinforcing bars fit together, reinforcement congestion is higher in the beams, columns, and joints than in other structural elements such as slabs. To help achieve proper consolidation of the concrete in these congested areas, maximum aggregate size should be limited accordingly. Specifying 1/2-inch maximum aggregate size is common for special moment frames. Sometimes small aggregate size will result in lower concrete strength, but other components of the concrete mixture can be adjusted to offset the lost strength. Another key to well-consolidated concrete in congested areas is having a concrete mixture with a high slump. A slump in the range of 7 to 9 inches may be necessary to get the concrete to flow in the congested areas.

It may be difficult to achieve good consolidation with internal vibration in highly congested areas because the reinforcement blocks insertion of the equipment. On occasion, contactors will position internal vibration equipment prior to placing the reinforcement. Alternatively, external vibration may be considered if there is adequate access to all sides of the formwork.

Difficulties with vibration do not come into play if self-consolidating concrete is used. These concrete mixtures are extremely fluid and easily flow around congested reinforcement. There is a cost premium associated with the self-consolidating concrete itself. This premium diminishes with increasing strength. The formwork required to hold this type of concrete must also be much tighter than with a standard concrete mixture. Using self-consolidating concrete successfully is highly dependent on the experience and preference of the contractor. For this reason, it is recommended not to specify self-consolidating concrete in the structural documents unless it has been previously discussed with the contractor.

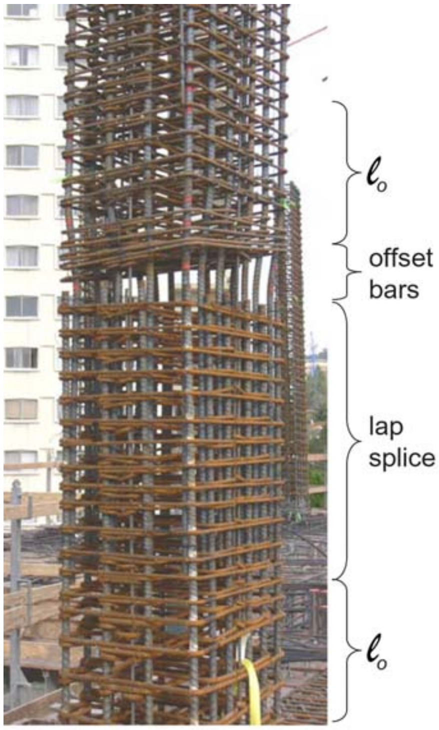


Figure 7-5 – Column cage lap splices are not permitted to extend outside the middle half of the column length and should not extend into the length ℓ_o at the column end.

Figure 7-6 – Longitudinal column reinforcement spliced every other floor to reduce congestion.

8. References

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9. Notation and Abbreviations

The terms in this list are used in the Guide.

A_{ch} cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, in.^{2}

A_g gross area of concrete section, in.^{2}

A_j effective cross-sectional area within a joint in a plane parallel to plane of reinforcement generating shear in the joint, in.^{2}

A_s area of nonprestressed longitudinal tension reinforcement, in.^{2}

A_s' area of compression reinforcement, in.^{2}

A_{sh} total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension b_c , in.^{2}

A_{st} total area of nonprestressed longitudinal reinforcement, in.^{2}

b width of compression face of member, in.

b_c cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh}, in.

b_w web width, in.

C_d deflection amplification factor defined in ASCE 7

C_{pr} flexural compression force, associated with M_{pr} in beam, acting on vertical face of the beam-column joint, lb

C_u coefficient for upper limit on calculated period as defined in ASCE 7

d distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in.

D dead loads, or related internal moments and forces

d_b nominal diameter of bar, wire, or prestressing strand, in.

E load effects of earthquake, or related internal moments and forces

f'_c specified compressive strength of concrete, psi

f_y specified yield strength of reinforcement, psi

f_{yt} specified yield strength f_y of transverse reinforcement, psi

h overall thickness or height of member, in.

h_x largest value of x_i measured around a column cross section (**Figure 5-16**), in.

h_{sx} story height below story level x (note: x refers to a story level, which is different from the definition of x in **Figure 5-6**)

h_x maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the column, in.

I_e effective moment of inertia for computation of deflection, in.^{4}

I_g moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in.^{4}

ℓ_c length of compression member in a frame, measured center-to-center of the joints in the frame, in.

ℓ_{dh} development length in tension of deformed bar with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), in.

ℓ_n length of clear span measured face-to-face of supports, in.

ℓ_0 length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, in.

L live loads, or related internal moments and forces

M_n nominal flexural strength at section, in.-lb

M_{pr} probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming yield strength in the longitudinal bars of at least 1.25f_y and a strength reduction factor, ϕ , of 1.0, in.-lb (M_{pr}^+ and M_{pr}^- refer respectively to positive and negative moment strengths that develop at opposite ends of a member when developing the intended hinging mechanism as in **Figure 5-3**)

M_u factored moment at section, in.-lb

P expected axial load, commonly taken as $D + 0.1L$

P_o nominal axial strength at zero eccentricity, lb, = $0.85f'_c(A_g - A_{st}) + f_y A_{st}$

P_u factored axial force; to be taken as positive for compression and negative for tension, lb

R response modification coefficient defined in ASCE 7

s center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, prestressing tendons, wires, or anchors, in.

S snow load, or related internal moments and forces

s_0 center-to-center spacing of transverse reinforcement within the length λ_o , in.

S_{DS} design, 5-percent damped, spectral response acceleration parameter at short periods defined in ASCE 7

T fundamental period of the building defined in ASCE 7, sec

T_a approximate fundamental period of building defined in ASCE 7, sec

T_{pr} flexural tension force, associated with M_{pr} in beam, acting on vertical face of the beam-column joint, lb

V_c nominal shear strength provided by concrete, lb

V_{code} column shear force calculated using code design load combinations, lb

V_{col} column shear force for use in calculating beam-column joint shear, lb

V_e design shear force corresponding to the development of the probable moment strength of the member, lb

V_j beam-column joint shear for assumed frame yield mechanism, lb

V_n nominal shear strength, lb

V_s nominal shear strength provided by shear reinforcement, lb

V_u factored shear force at section, lb

w effective width of beam-column joint for joint shear strength calculations, in.

w_u factored load per unit length of beam or one-way slab

x where supporting column is wider than the framing beam web, the shorter extension of the column beyond the beam web in the direction of the beam width (**Figure 5-6**), in.

x_i dimension from centerline to centerline of adjacent tie legs measured along member face perpendicular to member longitudinal axis, in.

γ coefficient defining joint nominal shear strength as function of joint geometry

σ redundancy factor based on the extent of structural redundancy present in a building defined in ASCE 7

Ω_o amplification factor to account for overstrength of the seismic-force-resisting system defined in ASCE 7

ϕ strength reduction factor

Abbreviations

ACI American Concrete Institute
IBC International Building Code
ASCE American Society of Civil Engineers
ELF Equivalent Lateral Force
MRS Modal Response Spectrum
SRH Seismic Response History
ASTM American Society for Testing and Materials

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