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NEHRP Seismic Design Technical Brief No. 1

Seismic Design of Reinforced Concrete Special Moment Frames

A Guide for Practicing Engineers

SECOND EDITION

Jack P. Moehle
John D. Hooper

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NEHRP Seismic Design Technical Briefs

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Prepared for
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With references to ASCE 7-16 and ACI 318-14

U.S. Department of Commerce
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Disclaimers

Cover photo—Reinforced concrete special moment frames under construction.

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1. Introduction

Building structures generally comprise a three-dimensional framework of structural elements configured to support gravity and lateral loads. Although the complete three-dimensional system acts integrally to resist loads, the seismic force-resisting system is commonly conceived as being composed of vertical elements, diaphragms, and the foundation (**Figure 1-1**). For reinforced concrete buildings assigned to the highest Seismic Design Categories (D, E, and F) in the United States, as defined in ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 2016), the applicable building codes permit the vertical elements to be either special moment frames or special structural walls. This Guide is written to describe the use, analysis, design, and construction of special reinforced concrete moment frames. NIST GCR 11-917-11REV-1, *Seismic design of cast-in-place concrete special structural walls and coupling beams: A guide for practicing engineers, NEHRP Seismic Design Technical Brief No. 6* (Moehle et al. 2011) and NIST GCR 16-917-42, *Seismic design of cast-in-place diaphragms, chords, and collectors: A guide for practicing engineers, NEHRP Seismic Design Technical Brief No. 3, Second Edition,* (NIST 2016) are companion guides.

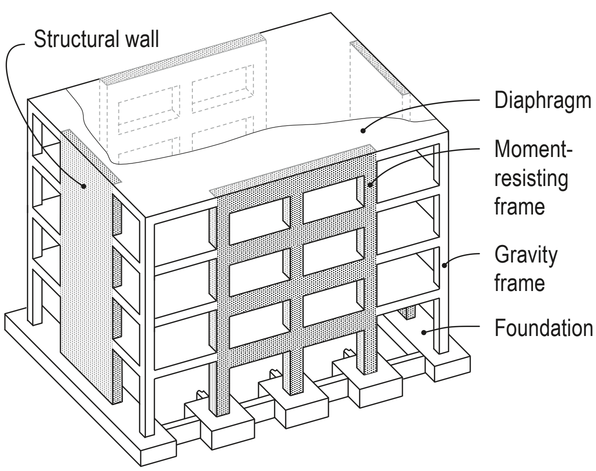


Figure 1-1. Basic building structural system.

Reinforced concrete special moment frames are made up of beams, columns, and beam-column joints. The frames are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sways through multiple displacement cycles during 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 ACI 318-14, *Building Code Requirements for Structural Concrete* (ACI 2014). The special requirements relate to inspection, materials, framing members (beams, columns, and beam-column joints), and to 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, making their identification and 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 will also be useful for building officials, educators, and students.

Most special moment frames use cast-in-place, normal-weight concrete without prestressing, and the member cross sections are rectilinear. ACI 318 contains requirements on the use of lightweight concrete, prestressed beams, columns with spiral reinforcement, 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 illustrations to help explain the requirements. Additional guidance is presented in sidebars. Section 2 through Section 6 present analysis, behavior, proportioning, and detailing requirements for special moment frames and other portions of the building that interact with them. Section 7 presents construction examples to illustrate detailing requirements for constructability. Cited references, notation and abbreviations, and credits are in Section 8, Section 9, and Section 10, respectively.

Sidebars in this 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 concrete special moment frame design.

Editions of Standards and the Building Code

This Guide follows the requirements of ACI 318-14, *Building Code Requirements for Structural Concrete* (ACI 2014), along with the pertinent requirements of the 2015 *International Building Code* (ICC 2015), and ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 2016). In this document, the terms ACI 318, IBC, and ASCE 7, used without a publication date, refer to these documents. These editions may not have been adopted yet by many jurisdictions. Design engineers are responsible for verifying the current applicable building code provisions adopted by the authority having jurisdiction over their project. Discussion with and approval by the building official should occur to verify that a later version of a code or standard not yet adopted locally may be used.

At the time of this writing, most jurisdictions in the United States have adopted the provisions of ACI 318-11, *Building Code Requirements for Structural Concrete* (ACI 2011). Most, though not all, of the technical requirements for special moment frames are the same in ACI 318-11 and ACI 318-14. Notable technical differences are (1) requirements for columns with high axial forces, (2) provisions on use of headed reinforcement in beam-column joints, and (3) restrictions on aspect ratio of beam-column joints. In terms of its format and organization, ACI 318-14 has been completely revised as compared with ACI 318-11, with most seismic provisions now appearing in Chapter 18.

Also as of this writing, most jurisdictions in the United States adopt ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010). Most of the seismic requirements of ASCE 7-10 and ASCE 7-16 are the same. Notable technical differences that may affect the design of special moment frames include (1) modal response spectrum analysis must account for 100 percent of the mass of the structure; (2) modal response spectrum base shear is scaled to 100 percent of the equivalent lateral force base shear, not 85 percent; and (3) for structures with a Type 4 out-of-plane offset irregularity as defined in ASCE 7-16 Table 12.3-1, transfer forces are increased by the overstrength factor, Ω_o. There are also many changes in Chapter 11 and Chapter 22 related to seismic maps and coefficients.

The First Edition of this Guide (NIST GCR 8-917-1) was published in August 2008. The codes and standards referenced in that edition were current as of then but have been updated by the documents referenced in this Second Edition. The First Edition, which may still be relevant in some engineering applications with regard to buildings constructed under the earlier editions of the codes and standards, references ACI 318-08, *Building Code Requirements for Structural Concrete* (ACI 2008), ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2006), and the 2006 *International Building Code* (ICC 2006).

Code Requirements versus the Recommendations of Guidance Documents

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, ACI 318 must, as a minimum, be followed. In addition to the ACI 318, the American Concrete Institute also produces guides and recommended practices. An example is ACI 352-02, *Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures* (ACI 2002). In general, guides of this type present recommended good practice, which as a minimum also meets the requirements of the current edition of ACI 318 at the time of publication of this Guide.

This Guide is written mainly to clarify requirements of the ACI 318, but it also refers to other guides such as ACI 352, and it presents other recommendations for good design and construction practices. This Guide is written to clearly differentiate between ACI 318 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 United States starting around 1960 (Blume et al. 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 have many similarities with 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 (**Figure 1-1**). 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 in ACI 318 §18.14 and apply wherever special moment frames are used with gravity frames in Seismic Design Category D, E, or F as defined in ASCE 7. The detailing requirements for the gravity-only elements may approach those for the special moment frame, such that in many cases it may be more economical to include those elements as part of the seismic force-resisting system if they can be made to satisfy all of the applicable requirements.

Special moment frames have also found use in dual systems that combine special moment frames with structural (shear) walls. In current usage, the moment frame is required to be capable of resisting at least 25 percent of the design seismic forces, while the total seismic resistance is provided by the combination of the moment frame and the shear walls in proportion with their relative stiffnesses. The use of special moment frames to create a dual system permits the use of a larger response modification coefficient, R, and thereby may reduce the overall seismic strength requirements. However, the added formwork and detailing required to construct special moment frames may increase construction cost compared with cost for a system using only shear walls.

2.2 When To Use Special Moment Frames

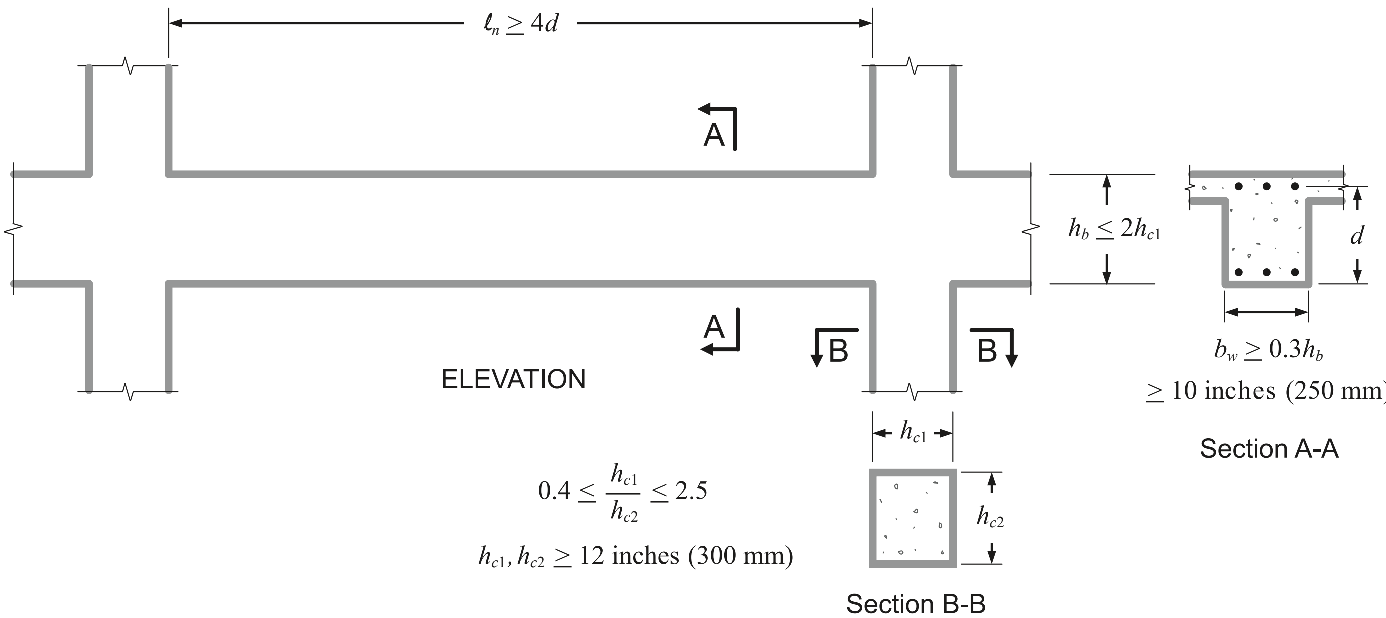
Moment frames are generally selected as the seismic force- resisting system when architectural space planning flexibility is desired. For many seismic force-resisting systems, such as special reinforced concrete shear walls, ASCE 7 §12.2.1 limits the building height. These height limits do not apply to special moment frames used alone or used in combination with shear walls to create a dual system.

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. Special moment frames may also be used in Seismic Design Categories A, B, and C, although 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 regardless of Seismic Design Category.

2.3 Frame Proportioning

Special frames must be proportioned such that they are capable of providing required lateral force resistance within specified story drift limits. The term “story drift” refers to the lateral displacement of one floor relative to the floor below it. Section 2.4, Section 4, and Section 5 of this Guide provide guidance on analysis and design to satisfy these requirements.

Typical economical beam spans for special moment frames are in the range of 20 to 30 feet (6 to 9 m). 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. Dimensional limits are covered in ACI 318 §18.6.2, §18.7.2, and §18.8.2.4. **Figure 2-1** summarizes the dimensional limits. The clear span of a beam must be at least four times its effective depth. Beam depth must not exceed twice the column depth in the framing direction, which limits the beam-column joint aspect ratio to improve force transfer. Beams are allowed to be wider than the supporting columns within ACI 318 limits, but beam width normally does not exceed the width of the column, for reasons of constructability. The



Note: For beams wider than columns, the beam width beyond the column on each side shall not exceed the smaller of h_{c2} and $0.75h_{c1}$. The longitudinal steel in column cross section and traverse steel in both beam and column cross sections are not shown for clarity.

Figure 2-1. Dimensional limits of beams and columns of special moment frames according to ACI 318.

ratio of the cross-sectional dimensions for columns shall not be less than 0.4, and beam width b_w shall be at least $0.3h_b$, which limits the cross sections to more compact sections rather than elongated rectangles. The minimum column dimension is 12 inches (300 mm), which is often too small for practical construction.

Special moment frames with first-story heights up to 20 feet (6 m) are common in practice. For buildings with relatively tall stories, it is important to make sure that soft (low stiffness) and/or weak stories are not created (see ASCE 7 §12.3.2.2).

Slab-column moment frames generally cannot be used as special moment frames because they do not satisfy the dimensional and reinforcement requirements for special moment frames.

2.4 Strength and Story 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 using a response modification coefficient of $R = 8$. Thus, they are allowed to be designed for a base shear equal to one-eighth of the value obtained from a linear elastic response analysis. Moment frames are generally flexible lateral force-resisting systems. If the building is relatively tall, its fundamental vibration period may fall within the long-period portion of the design response spectrum, resulting in a calculated base shear that may be lower than the required minimum base shear. In such cases, the required design strength is controlled by the minimum base shear equations of ASCE 7. Base shear calculations for long-period structures, especially in Seismic Design Categories D, E, and F, are frequently controlled by the upper limit on calculated period as defined in ASCE 7 §12.8.2. Wind loads specified in ASCE 7 must also be considered 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 story drift of the building within the limits specified by the building code. Story drift limits in ASCE 7 are a function of both risk category (IBC §1604.5) and the redundancy factor, ρ (ASCE 7 §12.3.4), as shown in **Table 2-1**.

Redundancy Factor	Risk 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, where h_{sx} is the story height.

The story drifts of the structure are to be calculated using the strength-level seismic load and amplified by C_d (ASCE 7 §12.8.6) when comparing them 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 story drift limit is not specified by ASCE 7; therefore, engineering judgment is required to determine the appropriate limit. Attachment of the cladding and other elements, and the comfort of the occupants, should be considered.

P-delta effects, addressed in ASCE 7 §12.8.7, can appreciably increase design moments and, therefore, must be considered in the frame design.

3. Principles for Design of Special Moment Frames

As noted in Section 2.4, ASCE 7 uses a design base shear that is considerably less than the base shear required for linear response at the anticipated earthquake intensity. Consequently, it is likely that design-level earthquake ground motions will drive a building structure well beyond the linear range of response. Consistent with this expectation, ACI 318 specifies proportioning and detailing requirements for special moment frames that are intended to produce a structure capable of multiple cycles of inelastic response without critical loss of strength. Three main goals are (1) to achieve a strong-column/weak-beam design that spreads inelastic response over multiple stories, (2) to provide details that enable ductile flexural response in the intended yielding regions, and (3) to avoid nonductile failures. Additionally, connections with nonstructural elements, such as stairs and infills, must be detailed such that they do not interfere with the intended frame behavior.

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 story drift. If the building has weak columns or weak beam-column joints, story drift tends to concentrate in one or a few stories (**Figure 3-1a**) and may exceed the story drift capacity of the columns. On the other hand, if columns provide a stiff and strong spine over the building height, story drift will be more uniformly distributed (**Figure 3-1c**), and localized damage will be reduced. Additionally, columns in a given story support the weight of the entire building above those columns, whereas the beams support only the gravity loads of the floor of which they form a part. Consequently, failure of a column is of greater consequence than failure of a beam. Therefore, building codes specify that columns shall 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 moment strengths exceed the sum of beam moment strengths at each beam-column connection of a special moment frame. Studies (e.g., Kuntz and Browning 2003, Moehle 2014) have shown that the full structural mechanism of **Figure 3-1c** can be achieved only when the column-to-beam strength ratio is relatively large (about four or more). Because this ratio 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 5.5 of this Guide summarizes the column detailing requirements of ACI 318.

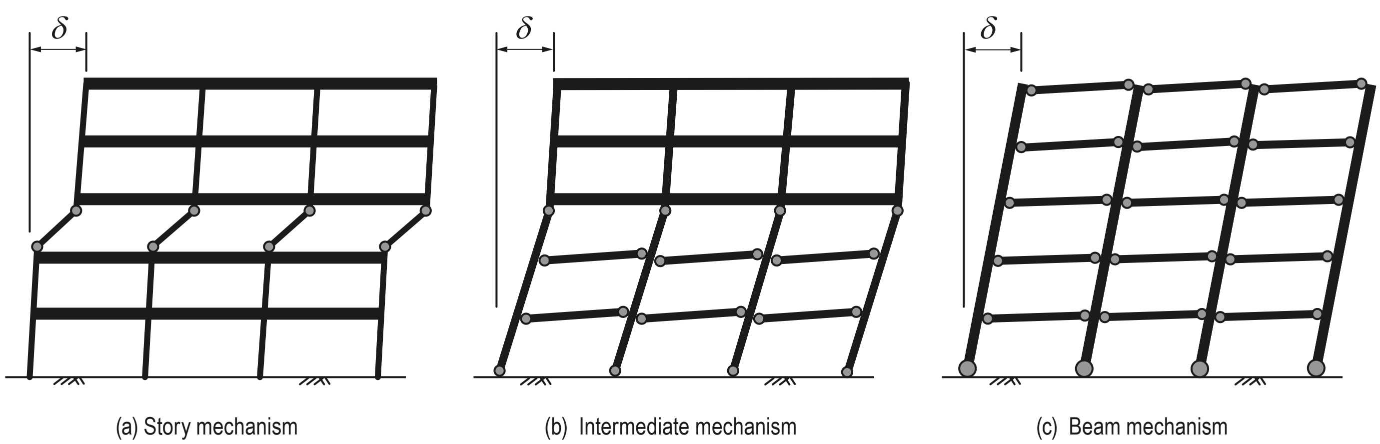


Figure 3-1. Design of special moment frames aims to avoid the story mechanism (a) and instead achieve either an intermediate mechanism (b) or a beam mechanism (c).

3.2 Detail Beams and Columns for Ductile Flexural Behavior

The ideal yield mechanism involves yielding of the beams throughout the height of the structure plus the columns at the base. Realistically, however, some column yielding along the height of the structure also has to be anticipated unless the columns are much stronger than the beams (see Section 3.1). Therefore, the end regions of the beams and columns at every beam-column joint should be detailed so these regions can undergo inelastic flexural response without critical strength decay. In the plane of a moment frame, column longitudinal reinforcement at all floor joints and beam longitudinal reinforcement at all interior joints should be continuous through the joints without splices unless such splices are proven to be capable of sustaining multiple post-yielding cycles. Transverse reinforcement should confine the core concrete and provide restraint against buckling of the longitudinal reinforcement. This transverse reinforcement should extend from the joint face along a length that will envelope the likely yielding region at the ends of beams and columns. **Figure 3-2** illustrates the typical required details near a beam-column connection.

3.3 Avoid Nonductile Failures

Ductile response requires that members yield in flexure and that shear, axial, and other nonductile failure modes be avoided. Nonductile failures usually can be avoided through a capacity design approach. The general approach is to identify flexural yielding regions, design those regions for code-required moment strengths, and then calculate other design forces 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. Section 5.2 through Section 5.4 describe the capacity design approach in greater detail

3.3.1 Column and Beam Shear

Shear failure, especially in columns, is relatively brittle and can lead to rapid loss of lateral strength and axial load-carrying capacity (**Figure 3-3**). Column shear failure is the most frequently cited cause of concrete building failure and collapse in earthquakes. ACI 318 uses a capacity design approach that requires the design shear strength be at least equal to the shear that occurs when yielding sections reach probable moment strengths.

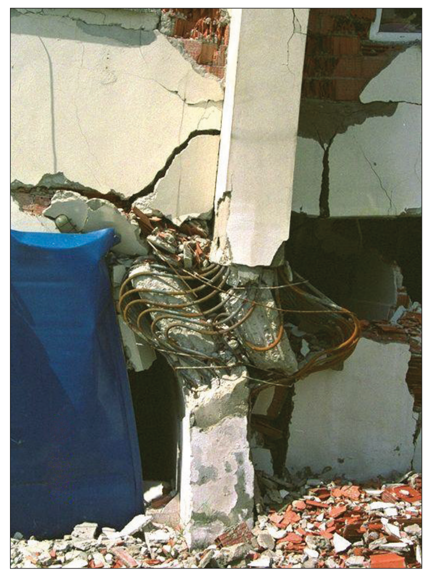


Figure 3-3. Shear failure can lead to a story mechanism and vertical collapse.

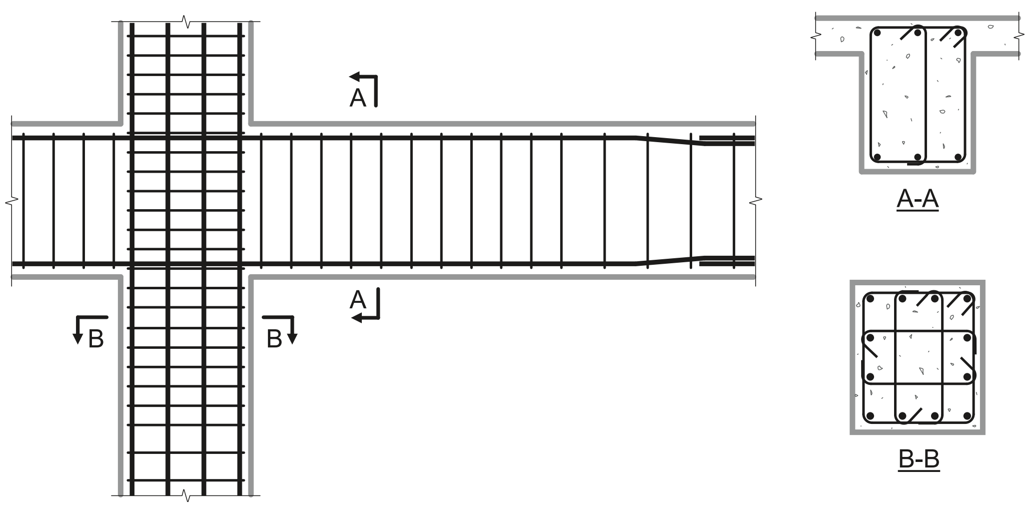


Figure 3-2. Frame elevation and sections showing typical reinforcement details required for ductile flexural response.

3.3.2 Column Axial Load

Deformation capacity can be severely limited for columns supporting high axial loads. Adequate performance of such columns can be achieved, but only through provision of relatively costly reinforcement details. To the extent practicable, high axial forces should be avoided in columns of special moment frames.

3.3.3 Beam-column Joints

Beam-column joints in special moment frames are required to transfer moments, shears, and axial forces among the interconnected beams and columns. To do so, the joints need to be stronger than the members framing into them. Joint transverse reinforcement helps the joint maintain strength under expected deformation reversals during earthquake shaking.

3.3.4 Reinforcement Anchorages and Splices

Severe seismic loading can result in loss of concrete cover, which will reduce the strength of developed or lap-spliced longitudinal reinforcement. Therefore, 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 within the joint; for this reason, ACI 318 restricts beam bar sizes. Bars anchored in exterior joints must have hooks or headed bars extended to the exterior side of the joint, and the tail of the hooked bars must project toward mid-depth of the joint. Finally, mechanical splices located where yielding is likely must be Type 2 splices, as noted in ACI 318 §18.2.7, capable of sustaining multiple cycles to stress levels well above the yield stress.

4. Analysis Guidance

4.1 Analysis Procedure

ASCE 7 permits the seismic forces within a special moment frame to be determined by four types of analysis procedures: Equivalent Lateral Force (ELF) analysis, Modal Response Spectrum (MRS) analysis, Linear Response History (LRH) analysis, and Nonlinear Response History (NRH) analysis. ASCE 7 Table 12.6-1 specifies the permitted analytical procedures based on the building's structural characteristics.

The ELF analysis is the simplest and can be used effectively for many structures. 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 Equation 12.8-3 and Equation 12.8-4 can typically be lowered. The upper limit on the period (C_uT_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 a calculated period rather than the approximate period defined in ASCE 7. Where the MRS analysis base shear is less than the ELF base shear, it must be scaled to 100 percent of the ELF value.

If an MRS or LRH analysis is required, three-dimensional computer models are typically used, although two-dimensional models occasionally are used. A three-dimensional model is effective in identifying the effects of any inherent torsion in the lateral system, as well as combined effects at corner conditions. A three-dimensional model must be used when certain horizontal structural irregularities exist (ASCE 7 §12.7.3).

ASCE 7 §12.5 specifies the requirements for the directions in which seismic loads are to be applied to the structural model. Member design forces can be determined from analysis of the structure with seismic forces applied independently in each of the two orthogonal directions, except for structures assigned to Seismic Design Categories C through F and having nonparallel systems or plan irregularity Type 5. For such structures, design must consider the interaction of orthogonal loading in one of two ways. If the ELF and MRS analysis are used, 100 percent of the effects in one primary direction are to be combined with 30 percent of the effects in the orthogonal direction. Alternatively, if design is based on response history analysis, orthogonal pairs of ground motion histories are to be applied simultaneously. In Seismic Design Categories D through F, columns that form part of two or more intersecting seismic force-resisting frames and have seismic axial loads in excess of 20 percent of the axial design strength shall also be subject to the above orthogonal loading. Although ASCE 7 only requires use of the orthogonal combination as noted above, it may be preferable to consider the orthogonal combination for all designs. This Guide recommends that approach.

ACI 318 §18.2.2.1 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 partial height or full height masonry infill walls, architectural concrete walls, stair wells, cast-in-place stairways, and inclined parking ramps.

Although it permits use of rigid members assumed not to be part of the seismic force-resisting system, ACI 318 §18.2.2.2 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 a seismic separation joint between the special moment frame and rigid elements assumed not to be part of the seismic force-resisting system. When this is not practicable, the interaction effects specified in ASCE 7 §12.7.4 must be addressed.

4.2 Stiffness Recommendations

When a special moment frame is analyzed, the cracked stiffness of the beams, columns, and joints must be appropriately modeled, because this stiffness determines the resulting building periods, base shear, story drifts, and internal force distributions. **Table 4-1** shows a typical range of values for the effective, cracked moment of inertia compared to moment of inertia of gross concrete section for beams and columns. These values fall within the limits specified in ACI 318 §6.6.3. For beams cast monolithically with slabs, including the effective flange width of ACI 318 §6.3.2 is acceptable.

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/SEI 41 (2014) recommends that the ratio of effective to gross-section moments of inertia, I_e/I_g, of **Table 4-2** be used with linear interpolation for intermediate axial loads. For beams, this interpolation results in $I_e/I_g = 0.30$.

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

Table 4-2. ASCE 41 effective stiffness modifiers for columns (ASCE 2014).

When serviceability under wind loading is considered, assuming higher effective stiffness and, in some cases, gross-section properties, for the beams, columns, and joints is common.

ACI 318 permits modeling of beam-column joints as rigid zones. Using this model in practice is common. An alternative is to model the joint as partially rigid, using the assumptions shown in **Figure 4-1** (Birely et al. 2012).

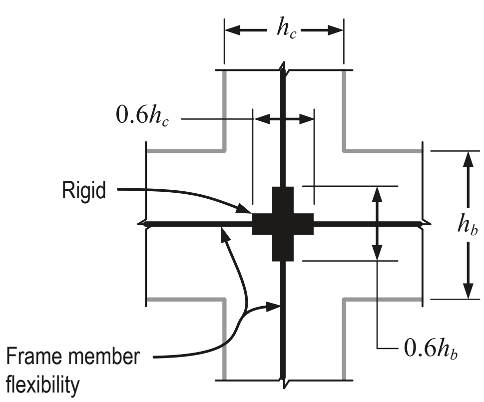


Figure 4-1. Partially rigid joint model.

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 4-2** illustrates four types of base restraint conditions that may be considered.

Modeling pinned restraints at the base of the columns, **Figure 4-2 (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 lengthens the period of the building, resulting in a smaller calculated base shear but larger calculated story drifts. Pinned restraints at the column bases 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. Even if not designed as such, a dowelled connection has some moment strength, which should be considered when determining design shears.

One drawback to the pinned-base condition is that the story drift of the frame, especially the story drift in the lowest story, is more difficult to keep 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 story 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 4-2 (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 of this Guide. The foundation elements must also be capable of delivering the forces to the supporting soil.

For structures assigned to Seismic Design Category D, E, or F, ASCE 7 §12.2.5.5 requires that special moment frames be continuous to the base where the frame is

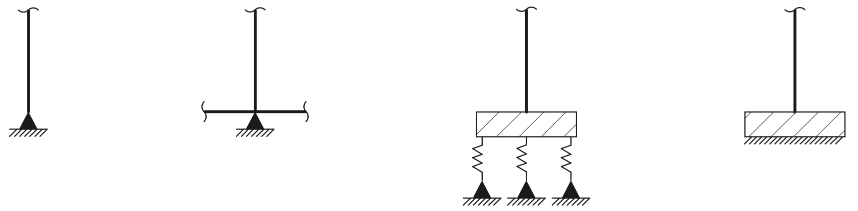


Figure 4-2. Column base restraint conditions.

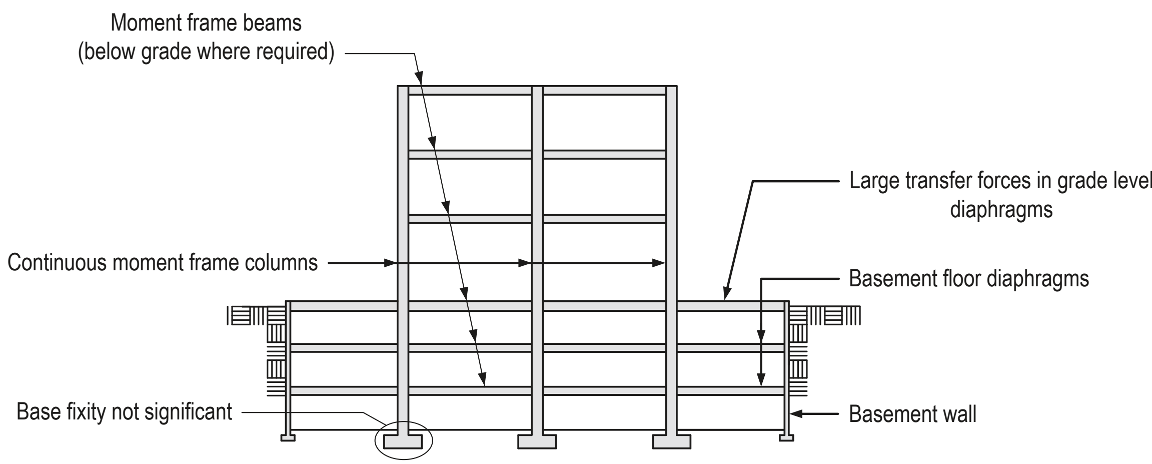


Figure 4-3. Moment frame extending through floors below grade.

required by ASCE 7 Table 12.2-1. A special frame that is used but not required by Table 12.2-1 is permitted to be discontinued above the base and supported by a more rigid system with a lower *R* factor, provided the requirements of ASCE 7 §12.2.3.1 and §12.3.3.4 are met.

The restraint and stiffness of the below-grade diaphragms and basement walls shown in **Figure 4-3** need 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 because the basement walls prevent significant drift below grade. Large forces are transferred through the grade level diaphragm to the basement walls, which are generally very stiff relative to the special moment frames.

5. Design Guidance

Design begins by identifying the layout of the special moment frames within the building. Member preliminary sizes are selected based on experience and the constraints outlined in Section 2.3. The building is then analyzed to determine design forces at critical sections of frame members. Design of a special moment frame uses a combination of strength design and capacity design. As described in Section 3.1, the general intent is to design a strong-column/weak-beam frame such that inelastic response is predominantly through flexural yielding at the beam ends. Thus, member sizing for required strength begins with moment strength design of the beams at the intended yielding locations and then progresses to the other design requirements. This section presents the design procedure in a sequence that most engineers find to be efficient.

> **Strength Design Method**
>
> ACI 318 uses the strength design method to provide the intended level of safety. The basic requirement for strength design can be expressed as *design strength* ≥ *required strength*. The design strength is written in the general form ϕS_n, in which ϕ is a strength reduction factor and S_n is the nominal strength. The required strength is expressed in terms of factored loads or related internal moments and forces. The strength design method is the same as the Load and Resistance Factor Method (LRFD) used for the design of some other materials

5.1 Load and Strength Reduction Factors

Chapter 2 of ASCE 7 defines the load combinations applicable to the design of beams and columns of special moment frames. The load combinations require consideration of horizontal seismic effects, vertical seismic effects, dead load, live load, and other applicable loads such as soil pressures, snow, and fluid. ASCE 7 §12.4.2 defines the horizontal seismic effect as $E_h = \rho Q_E$ and the vertical seismic effect as $E_v = 0.2 S_{DS} D$. In general, E_h and E_v must be applied in all combinations in both positive and negative directions. The basic seismic load combinations are

$$1.2D + E_v \pm E_h + 1.0L + 0.2S$$

$$0.9D - E_v \pm E_h$$

The load factor on L is permitted to equal 0.5 for all occupancies in which unreduced design live load is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly. The loads are applied to the structure either independently along orthogonal principal directions or using orthogonal combination rules as discussed in Section 4.1.

> **Governing Load Combinations**
>
> The load combinations specified in IBC §1605 are based on the load combinations from ASCE 7 Chapter 2. However, the IBC factors on snow load are different from those in ASCE 7. Where there is a discrepancy between the two documents, the IBC governs for building design in the United States.

For combined flexure and axial force in beams and columns, the strength reduction factor ϕ varies with the net tensile strain, ε_t, defined as the net tensile strain in the extreme tension steel when the section reaches nominal strength (that is for $\varepsilon_{cu} = 0.003$). If $\varepsilon_t \geq 0.005$, $\phi = 0.9$. If $\varepsilon_t \leq \varepsilon_y$ (taken as 0.002 for Grade 60 reinforcement), $\phi = 0.65$ for tied columns or 0.75 for columns with spiral reinforcement. The value of ϕ is interpolated for intermediate values of ε_t. For beams of special moment frames, ϕ is usually 0.9. The exception is where a beam acts as a collector or chord of a diaphragm, in which case design axial compressive force may result in a lower value.

For column or beam shear, $\phi = 0.75$. For beam-column joint shear, $\phi = 0.85$.

5.2 Beam Moment Strength and Longitudinal Reinforcement

The building is analyzed under the design loads to determine the required flexural strengths at beam plastic hinges, which should be located at the ends of the beams. **Figure 5-1** illustrates the intended plastic hinge locations along with a typical moment envelope obtained by analyzing the frame under required load combinations. At each plastic hinge location, and for both positive and negative moment, the beam section is designed such that the design moment strength is at least equal to the required moment strength calculated using applicable factored load combinations, that is, $\phi M_n \geq M_u$.

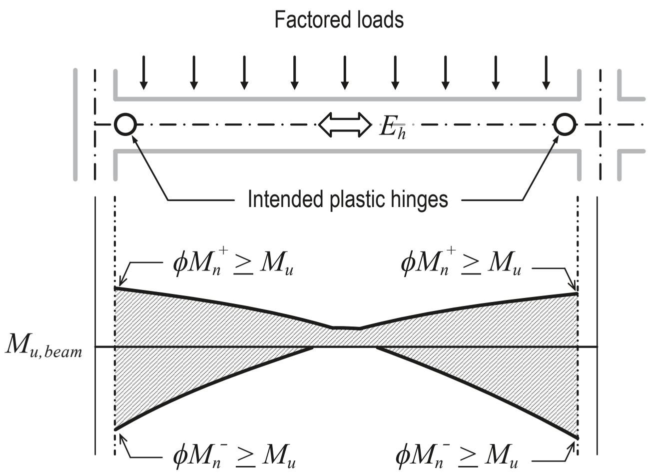


Figure 5-1. Design moment strengths must be at least as large as required moment strengths at intended plastic hinge locations.

Moment Strength of Beams Cast Monolithically with Slabs

Where a slab is cast monolithically with a beam, the slab acts as a flange, increasing the flexural stiffness and strength of the beam. The slab can act both as a compression flange and as a tension flange, depending on the direction of bending moment. 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 §6.3.2. Regardless of the approach used to initially size the beam, the developed flange reinforcement acts as flexural tension reinforcement when the beam moment puts the slab in tension. ACI 318 §18.7.3.2 requires this slab reinforcement to be considered as beam longitudinal tension reinforcement for calculating the relative strengths of columns and beams.

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 ϕ = 1.0, and assuming reinforcement yield strength equal to $1.25f_y$. 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 provided beam moment strength, the designer should take care to minimize excess capacity.

Probable Moment Strength, M_{pr}

ACI 318 defines probable moment strength, M_{pr}, as moment strength of a member, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile stress in longitudinal bars of "at least 1.25 f_y", where f_y in the nominal yield strength of the reinforcement and a strength reduction factor ϕ of 1.0. It is uncommon to use an assumed yield strength greater than 1.25 f_y. Therefore, this Guide assumes the value is always 1.25 f_y. It is common to calculate M_{pr} using the usual assumptions of strain compatibility and steel stress-strain relation that is linear to a yield strength of 1.25 f_y, with stress at a constant value of 1.25 f_y for strains beyond the yield point. For beams, $1.25M_n$ can be a good approximation to M_{pr}. M_{pr} is a measure of the flexural overstrength that may develop in a beam or column and is used to estimate force demands in other parts of the structure that develop when flexural yielding occurs.

Reinforcement commonly used in the United States has an average yield strength 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 strength. Thus, if a reinforcing bar is subjected to large strains during an earthquake, stresses well above $1.25f_y$ are likely. Such effects, however, are likely to be offset by inherent overstrength throughout the rest of the building as well. The factor 1.25 in ACI 318 was established recognizing all of these effects.

Besides providing the required strength, the longitudinal reinforcement must also satisfy the requirements illustrated in **Figure 5-2**. Although ACI 318 §18.6.3 allows a reinforcement ratio up to 0.025, 0.01 is more practical for constructability and for keeping joint shear forces within reasonable limits.

The designer also needs to specify requirements for reinforcement splicing and bar cutoffs. Where lap splices are used, these should be located at least $2h_b$ away from critical sections where flexural yielding is likely to occur (**Figure 5-2**). Mechanical splices, if used, shall be Type 2; although ACI 318 §18.6.3.4 permits these at any location, it is better to locate them at least $2h_b$ away from critical sections where flexural yielding is likely to occur.

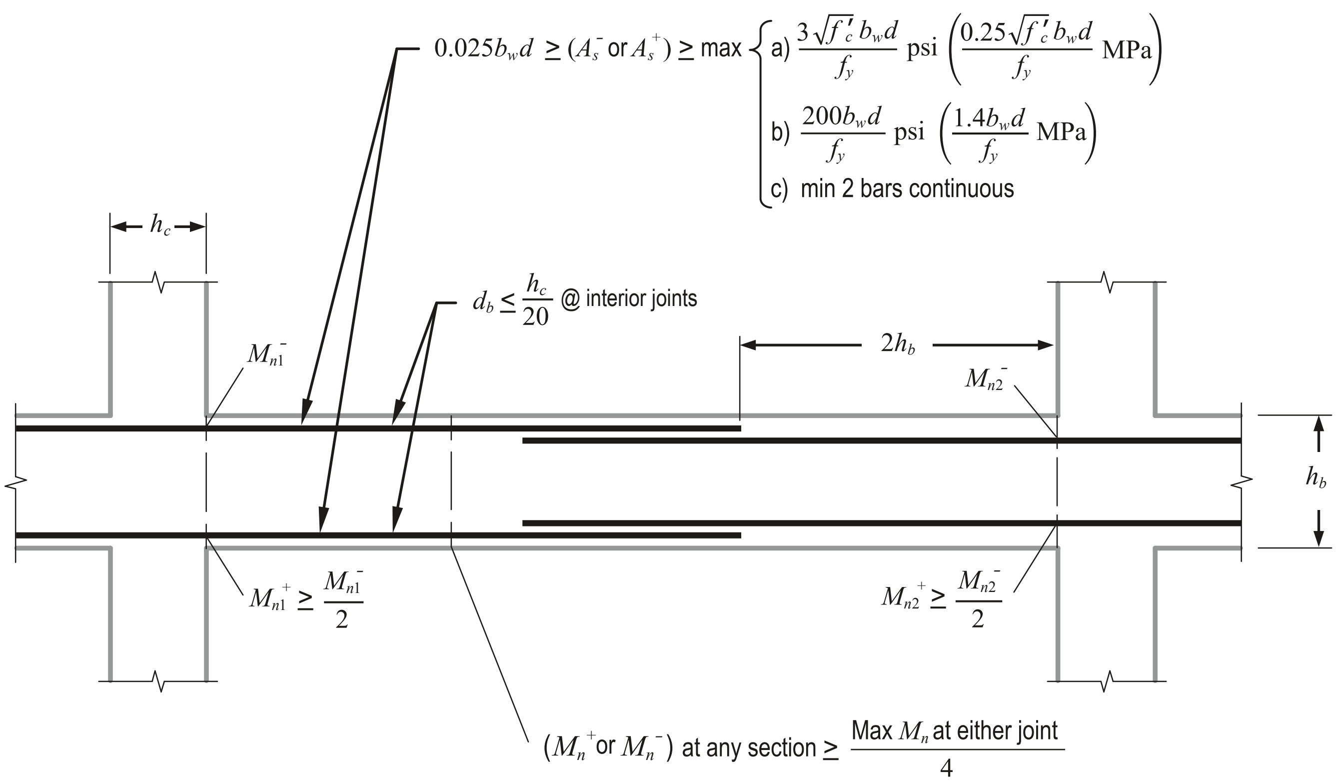


Figure 5-2. Beam longitudinal reinforcement requirements.

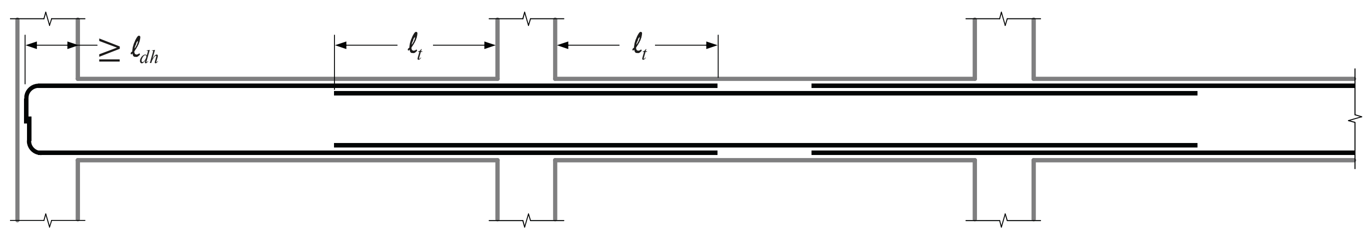


Figure 5-3. Beam longitudinal reinforcement arrangement to avoid splicing.

Figure 5-3 presents an optional layout for beam longitudinal reinforcement that avoids lap splices by staggering bar cutoffs along adjacent spans. Bar extension ℓ_t is determined using the usual bar termination requirements for beams (ACI 318 §9.7.3).

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, producing small gravity load moments compared with seismic design moments, then beam yielding is likely to occur at the ends of the beams adjacent to the beam-column joints, as suggested in **Figure 5-4(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, producing large gravity load moments compared with seismic design moments, then a less desirable behavior can result, as illustrated in **Figure 5-4(b)**. As the beam is deformed by the earthquake, the moment 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:

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

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.

5.3 Joint Shear and Anchorage

Once the longitudinal 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 often governs the size of the moment frame columns.

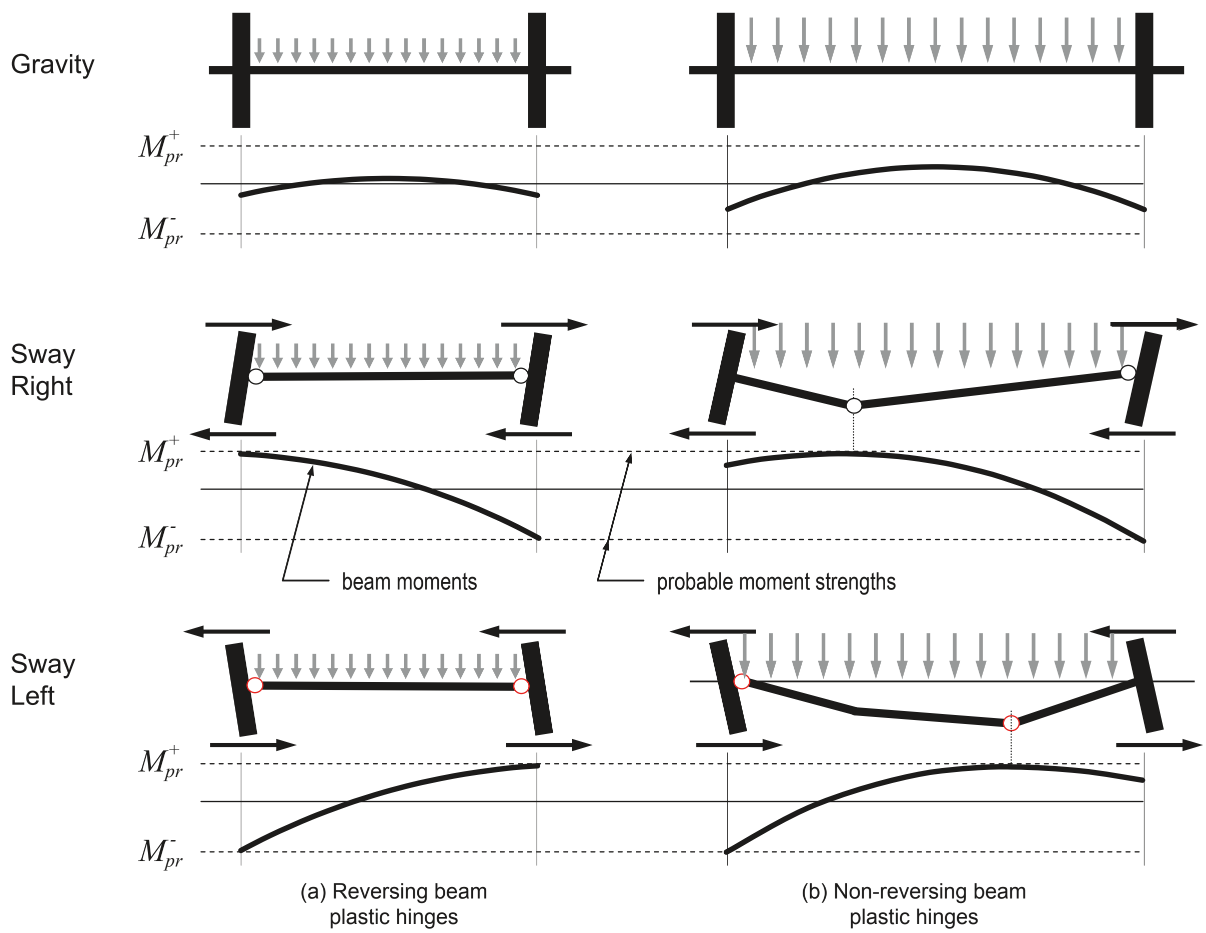


Figure 5-4. (a) Reversing beam plastic hinges (preferred) tend to occur when spans are relatively short and gravity loads relatively low; (b) non-reversing plastic hinges (undesirable) tend to occur for longer spans or heavier gravity loads.

To illustrate the procedure, consider a column bounded by two beams (**Figure 5-5**). 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.

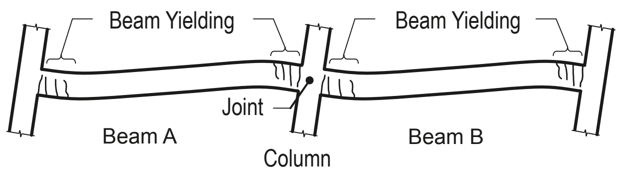


Figure 5-5. The frame yielding mechanism determines the forces acting on the column and beam-column joint.

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 5-6**. 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 5.4.1 for discussion on how to calculate these shears). For a typical story, the column mid-height 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. From the free body diagram of **Figure 5-6**, the column shear is calculated as

$$V_{col} = \left[\left(M^-_{pr,A} + M^+_{pr,B}\right) + \left(V_{e2,A} + V_{e1,B}\right)\frac{h_c}{2}\right] / \ell_c$$

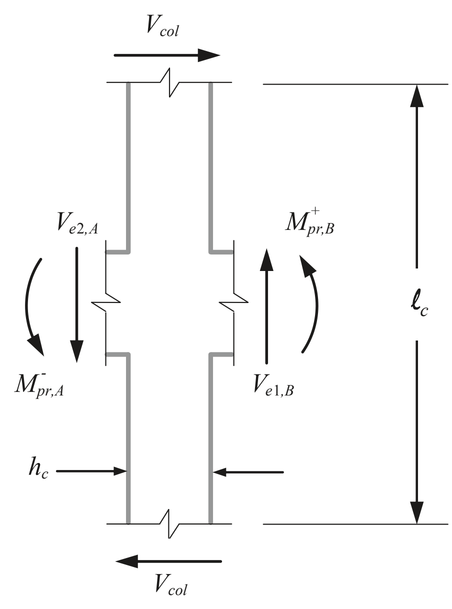


Figure 5-6. Free body diagram of column used to calculate column shear V_{col}.

Having found the column shear, V_{col}, the 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 5-7**. Beam longitudinal reinforcement is assumed to reach a force equal to $1.25A_s f_y$ or $1.25A'_s f_y$. Assuming the beam to have zero axial force, 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. Thus, the required joint shear strength is

$$V_u = V_j = T_{pr} + T'_{pr} - V_{col}$$

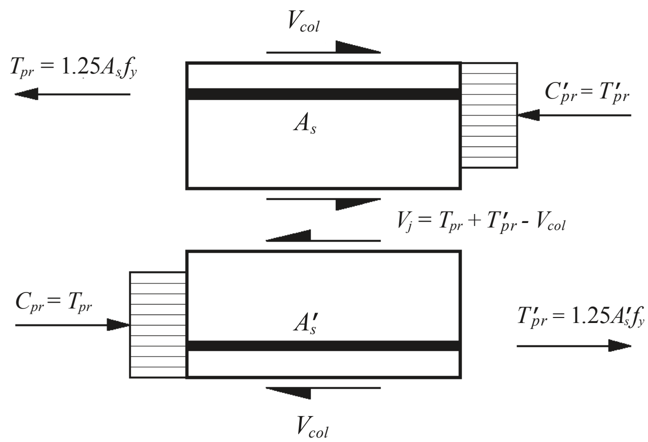


Figure 5-7. Joint shear free body diagram.

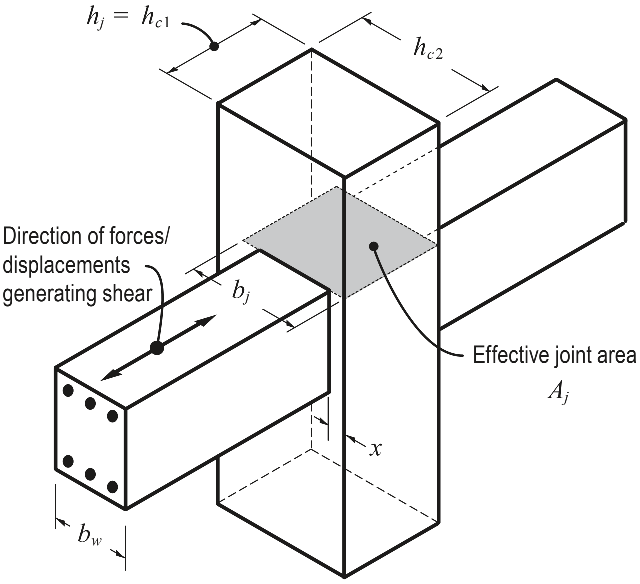
It is well established that for monolithic construction, the slab longitudinal reinforcement within an effective width also contributes to the beam flexural strength. Although not required by ACI 318, ACI 352 recommends including the slab reinforcement within this effective width in the quantity A_s 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 §6.3.2 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 is required to be at least equal to the required strength, that is, $\phi V_n \geq V_j$. Where V_j is shown in **Figure 5-7**. The strength reduction factor is $\phi = 0.85$. The nominal strength V_n is defined as

$$V_n = \gamma\sqrt{f'_c}A_j$$

in which A_j is the joint area defined in **Figure 5-8**, and γ is a strength coefficient defined in **Figure 5-9**.

Though **Figure 5-8** shows the beam narrower than the column, ACI 318 §18.6.2 contains provisions allowing the beam to be wider than the column, as described in the note in **Figure 2-1**. The effective joint width, however, is limited to the overall width of the column h_{c2}.



$$b_j = min[(h_{c2}), (b_w + h_{c1}), (b_w + 2x)]$$

$$A_j = b_j h_j$$

Notes

1. For columns wider than beam, x = the smaller of the dimension from face of beam to edge of column.
2. Effective area of joint for forces in each direction of framing is to be considered separately.

Figure 5-8. Definition of beam-column joint dimensions.

The nominal shear strength coefficients shown in **Figure 5-9** are from ACI 352-02. ACI 318 does not define different strengths for roof and typical floor levels but instead directs the designer to use the typical values (left half of **Figure 5-9**) 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.

Detailing beam-column joints is a task requiring careful attention to several code requirements as well as construction requirements. **Figure 5-10** and **Figure 5-11** show example details for interior and exterior beam-column joints, respectively. Beam bars, possibly entering the joint from two different framing directions, must pass by each other and by the column longitudinal bars. Joint hoop reinforcement is also required. Large-scale drawings, three-dimensional models, 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.

Beam and column longitudinal reinforcement must 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

Case A: Two columns framing into the joint

Case B: One column framing into the joint

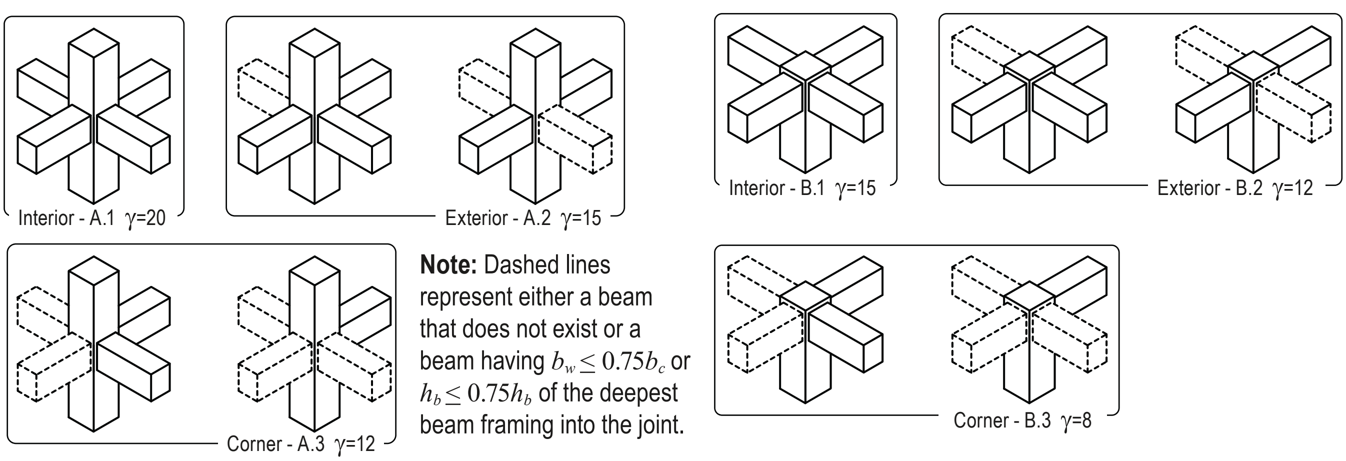


Figure 5-9. Joint configurations and nominal shear strength coefficients. Values of γ are for in-lb units. For SI units, divide γ by 12.

normal-weight concrete (**Figure 5-10**). This requirement helps improve performance of the joint by reducing slip of the beam bars through the joint relative to slip that would occur with a smaller column dimension. Some slip, however, will occur even with this column dimension requirement. 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 5-11**). The tail of the hook must project toward the mid-depth of the joint so that a joint diagonal

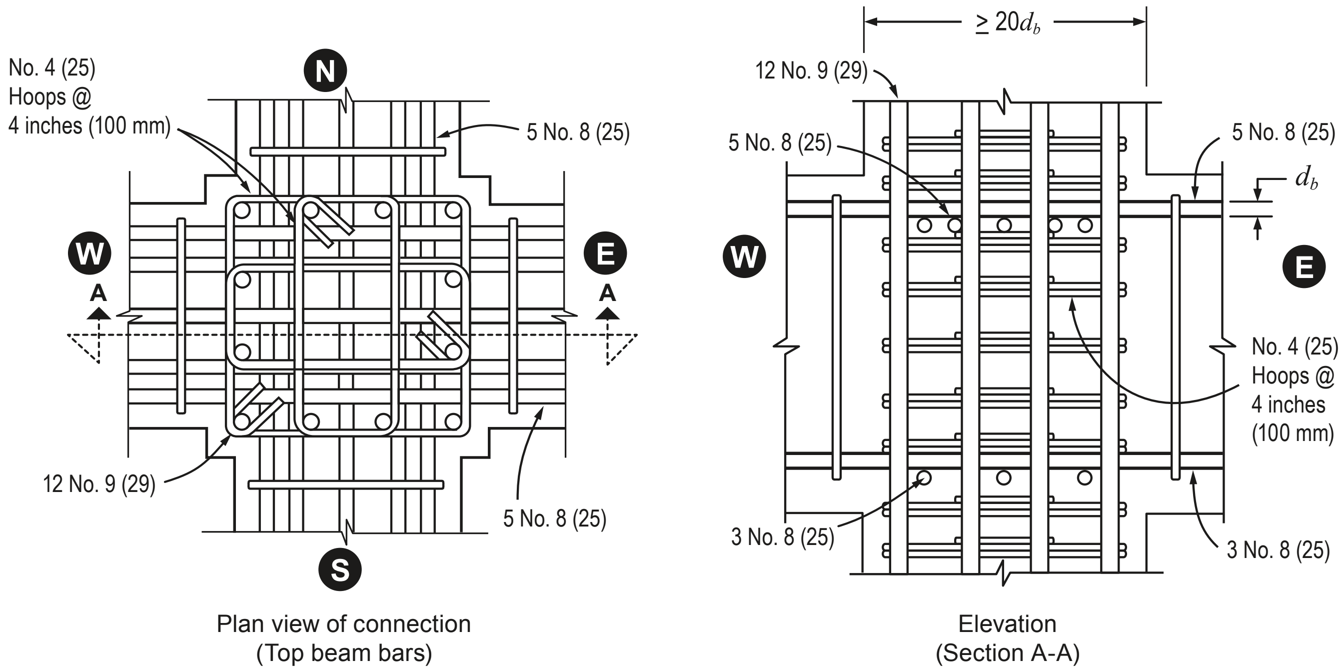


Figure 5-10. Example interior joint detailing.

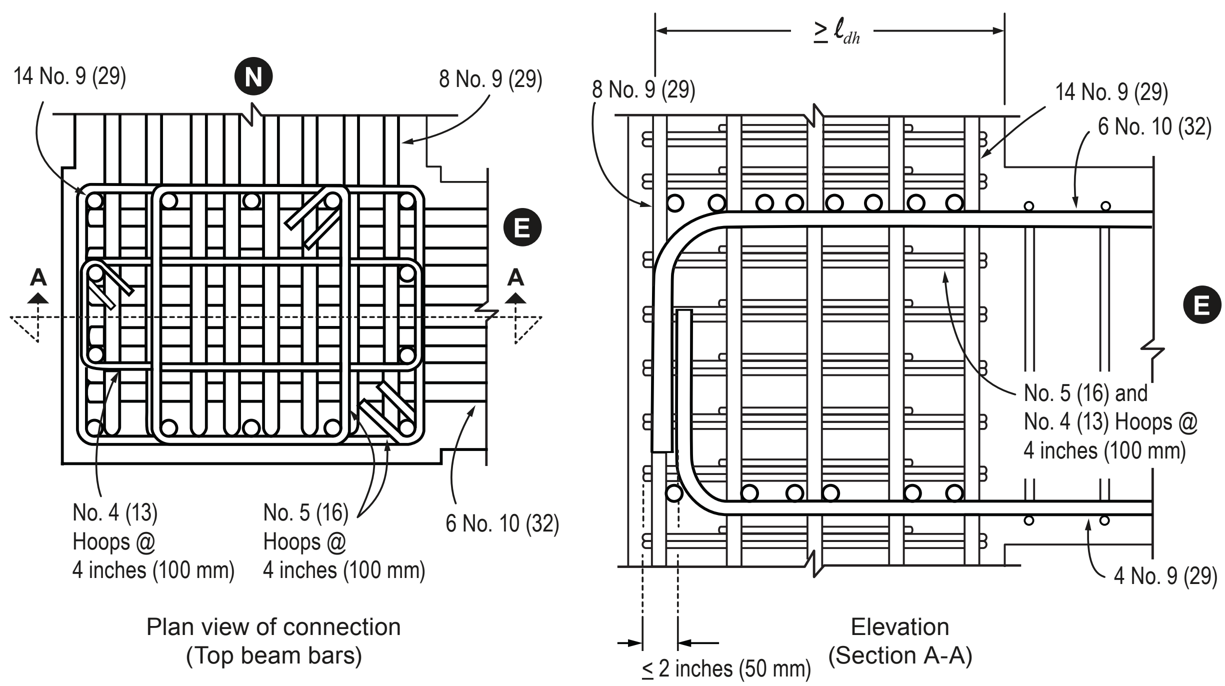


Figure 5-11. Example exterior joint detailing.

compression strut can be developed from the interior of the curved portion of the bar and extend diagonally across the joint to the compression blocks for the column and beam. The development length in tension of a deformed bar with a standard 90° hook in normal-weight concrete must be the longest of $8d_b$, 6 inches (150 mm), and the length required by the following equation:

$$\ell_{dh} = f_y\, d_b \,/\, (65\lambda\sqrt{f'_c}) \text{ (psi)}$$

$$\ell_{dh} = f_y\, d_b \,/\, (5.4\lambda\sqrt{f'_c}) \text{ (MPa)}$$

The length required by the equation almost always governs. This equation assumes that the hook is embedded in a confined beam-column joint. The equation applies only to bar sizes No. 3 through No. 11 (SI designations 10 through 36).

In addition to satisfying the development length requirements of the previous paragraph, hooked beam bars are required to extend to the far side of the beam-column joint (ACI 318 §18.8.2.2). This requirement 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 approximately one inch (25 mm) from the perimeter hoops of the joint to improve concrete placement.

Beam and column longitudinal reinforcement is sometimes terminated using headed deformed bars rather than hooks. ACI 318 covers headed bars conforming to the American Society for Testing Materials ASTM A970 (ASTM 2013), including Annex A1 requirements for Class HA head dimensions. For beams, the development length for headed bars in normal-weight concrete must be the longest of $8d_b$, 6 inches (150 mm), and the length required by the following equation:

$$\ell_{dt} = \frac{0.016 f_y}{\sqrt{f'_c}}\, d_b \text{ (psi)}$$

$$\ell_{dt} = \frac{0.19 f_y}{\sqrt{f'_c}}\, d_b \text{ (MPa)}$$

The equation applies only to Grade 60 (420) reinforcement, bar sizes No. 3 through No. 11 (10 through 36), and concrete compressive strength not exceeding 6,000 psi (41 MPa). Clear spacing of bars in a layer shall be at least $3d_b$.

Where beam top longitudinal reinforcement is provided by headed deformed bars that terminate in the joint and the column does not extend at least h_c above the top of the joint, there is the potential for the beam top bars to split out of the top of the joint. In this case, ACI 318 requires additional vertical joint reinforcement to enclose the longitudinal bars and clamp them in place. ACI 352 provides example details to address this condition.

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 5.5). Where beams frame into all four sides of the joint and where each beam width is at least three-fourths the column width, the 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 (150 mm).

5.4 Beam Shear and Transverse Reinforcement

5.4.1 Beam Design Shear

The beam design shear is determined using the capacity design approach outlined in Section 3.3. **Figure 5-12** illustrates this approach applied to a beam. A free body diagram of the beam is isolated from the frame and is loaded by transverse loads w_u as well as the moments and shears acting at the ends of the beam. According to ASCE 7, the controlling transverse load combination is $1.2D + E_v +$ (1.0 or 0.5)$L + 0.2S$. Assuming the beam is yielding in flexure, the beam end moments are set equal to the probable moment strengths M_{pr} described in Section 5.2. 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). **Figure 5-12** shows the conditions for determining shear for loading in one direction. The process is repeated for the opposite direction to obtain the shear envelope for design.

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 midspan, as they typically would in a gravity-only beam. Instead, most beams in a special moment frame will have non-reversing shear demands along their length. If the shear does reverse along the span, non-reversing beam plastic hinges will likely occur (see Section 5.2).

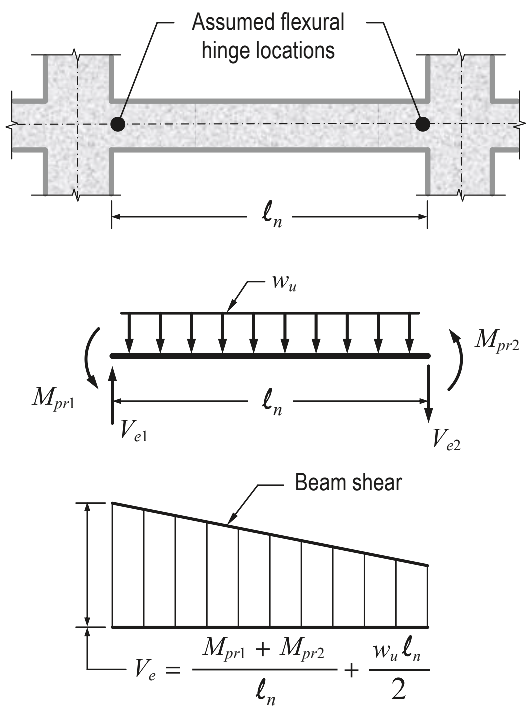


Figure 5-12. Beam shears are calculated based on provided probable moment strengths combined with factored gravity loads.

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 5.4.2.

5.4.2 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, confine lap splices where present, 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 or stirrups can be placed: the zones where flexural yielding is expected to occur, preferably at beam ends; the zone along lap-spliced bars, if any; and the remaining lengths of the beam.

The zones where plastic hinging is intended to occur, of length $2h_b$ on either side of the plastic hinge, needs to be well confined because this is where the beam is expected to undergo flexural yielding and, if yielding is at the beam ends, this is the location with the highest shear. Therefore, closely spaced, closed hoops are required in these zones, as shown in **Figure 5-13**. If flexural yielding is expected anywhere along the beam span other than the ends of the beam, hoops must extend $2h_b$ on both sides of that yielding location. This latter condition is one associated with non-reversing beam plastic hinges (see Section 5.2), and avoiding this condition is recommended. Subsequent discussion assumes that this type of behavior is avoided by design.

Placement of Hoops and Stirrups

Hoops are required along the beam end zones (where flexural yielding is expected) and along lap splices, with spacing limits as noted in **Figure 5-13.** Elsewhere, transverse reinforcement is required at a spacing not to exceed $d/2$ or $d/4$, depending on the level of shear, and is permitted to be in the form of beam stirrups with seismic hooks.

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 may be practical to extend the hoop detail and spacing over the entire length of the beam. A quick quantity comparison should be conducted to determine 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, ironworkers and special inspectors need to have a clear understanding of the placement requirements. Some engineers take it upon themselves to communicate these unique conditions to the special inspector. This is typically most crucial early in the construction process when the first few levels of beams are constructed. Generally after the first few levels, the reinforcement pattern is properly replicated.

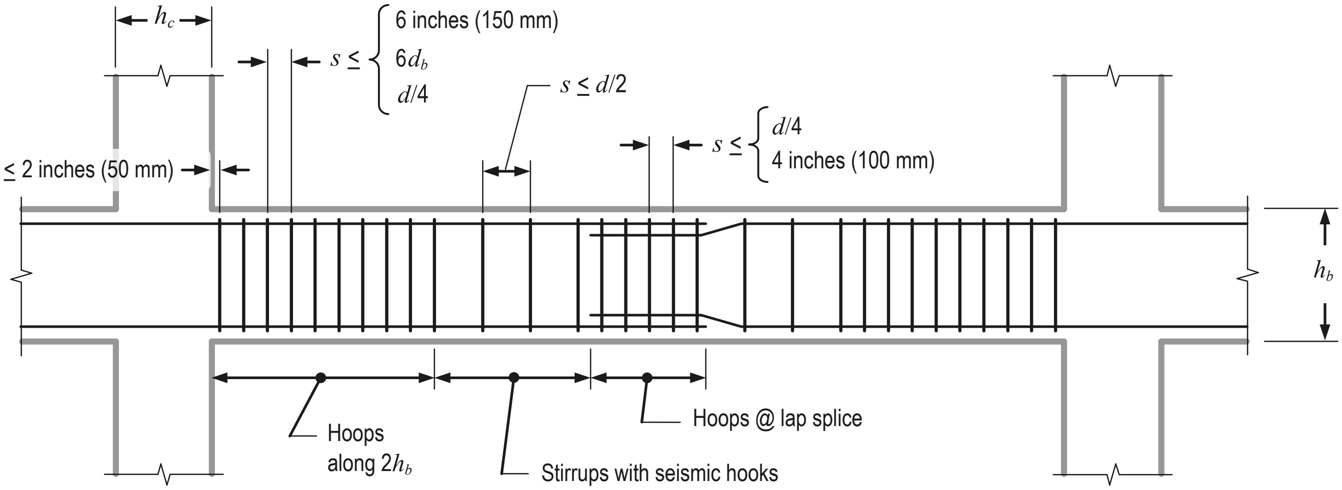


Figure 5-13. Hoop and stirrup location and spacing requirements.

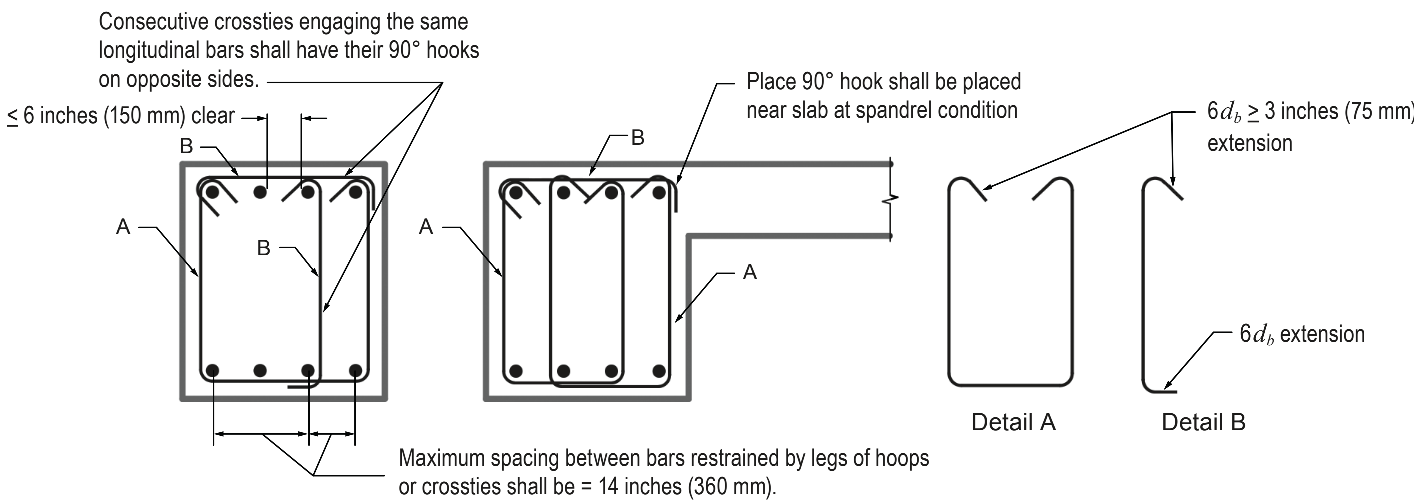


Figure 5-14. Hoop reinforcement detail.

Hoop reinforcement may be constructed of one or more closed hoops. Alternatively, hoops 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 5-14** for additional detailing requirements for the hoop reinforcement.

Wherever hoops are required, they must be configured such that (a) every corner and alternate longitudinal bar on the perimeter has lateral support provided by the corner of a tie or crosstie with an included angle of not more than 135° and (b) no unsupported bar is farther than 6 inches (150 mm) clear on each side along the tie from a laterally supported bar. This requirement 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 regions.

When sizing the hoops within the potential flexural yielding regions, typically within lengths $2h_b$ at the beam ends, the shear strength of the concrete itself must be neglected (i.e., $V_c = 0$) except where specifically allowed per ACI 318 §18.6.5.2. Thus, within the potential yielding regions, the shear design requirement typically is $\phi V_s \ge V_e$, where $\phi = 0.75$. V_e is determined using capacity design, as discussed in Section 5.4.1. Outside the potential flexural yielding regions, design for shear is done using the conventional design equation $\phi(V_c + V_s) \ge V_e$.

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 as described previously for the end zones. Beam longitudinal bar lap splices shall not be used (a) within the joints, (b) within a distance of $2h_b$ 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 midspan of the beam. See **Figure 5-13** for hoop spacing requirements.

5.5 Column Design and Reinforcement

5.5.1 Preliminary Sizing

The column cross-sectional dimensions can be controlled by one of several requirements:

- Shear and anchorage requirements for beam-column joints determine the required joint dimensions and consequently may dictate column dimensions. See Section 5.3.
- Column strength must be sufficient to meet the strong-column/weak-beam requirements.
- Cross sections should provide combined moment/axial strengths using longitudinal reinforcement ratios that are within constructible limits. Preferably, reinforcement ratios are in the range of 0.01 to 0.03. ACI 318 permits a ratio as large as 0.06, but this amount of reinforcement results in very congested splice locations. Mechanical splices should be considered when the reinforcement ratio exceeds 0.03.
- Cross sections should be sufficient for expected axial forces. The maximum permitted axial force for a tied column is $P_u = 0.52P_o$, but columns near this limit must be detailed with transverse reinforcement that will complicate construction. Columns having $P_u \leq 0.3A_g f'_c$ can perform well with relaxed detailing that facilitates construction.
- Column shear strength must be sufficient to resist demands associated with flexural yielding.
- The dimensions must conform to the prescriptive dimensional requirements of **Figure 2-1**.

5.5.2 Moment/Axial Strength and Longitudinal Reinforcement

According to ACI 318, a column that is considered part of the seismic force-resisting system must satisfy the strong-column/weak-beam requirement for all load combinations. As discussed in Section 3.1, this requirement is intended to promote formation of mechanisms under earthquake load as illustrated in **Figure 3-1 (b)** and **(c)**. This requirement usually controls the flexural strength of the column.

To meet the strong-column/weak-beam requirement of ACI 318, the sum of the nominal moment strengths, M_{nc}, of the columns framing into each beam-column joint must be at least 1.2 times the sum of the nominal moment strengths, M_{nb}, of the beams framing into the joint, as illustrated in **Figure 5-15**. It is required to include the developed slab reinforcement within the effective flange width (ACI 318 §6.3.2) as beam flexural tension reinforcement when calculating beam strength. This check must be verified independently for sway in each direction (for example, east and west) and in each of the two principal framing directions (for example, EW and NS). The variation in column axial force for sway in the two directions must be considered because the column moment strength is dependent on the axial force, as shown in **Figure 5-16**.

$$M_{nc}^{top} + M_{nc}^{bot} \geq \frac{6}{5}\left(M_{nb}^{+} + M_{nb}^{-}\right)$$

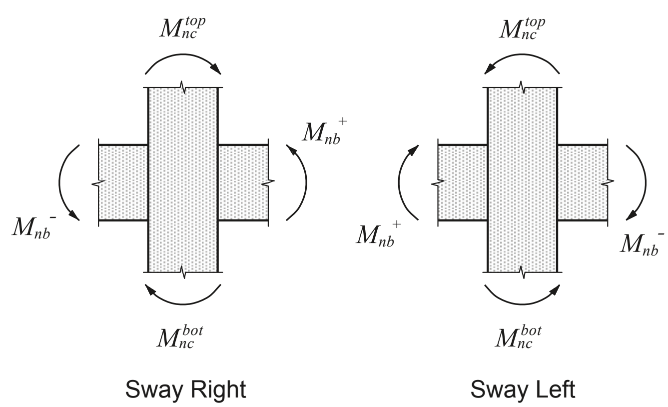


Figure 5-15. Strong column/weak beam design moments.

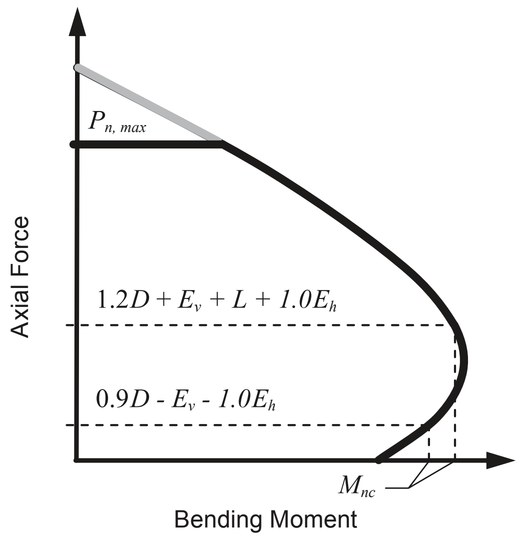


Figure 5-16. Nominal column moments must be checked at maximum and minimum axial forces.

In some cases it may not be practical to satisfy the strong-column/weak-beam provisions for all of the columns. The strength and stiffness of such columns cannot be considered as part of the special moment frame. These columns must satisfy the requirements of ACI 318 §18.14, that is, columns not designated as part of the seismic force-resisting system.

It may be reasonable to make an exception to the ACI 318 strong-column/weak-beam requirement at the roof level of a building or at other equivalent levels where a column does not extend above the beam-column joint. At such locations, a single interior column may be required to resist moments from two beams in a given framing direction. Columns at such locations commonly support relatively low axial forces, and flexural hinging of the columns at this level will not adversely affect the overall frame mechanism. Where a column is weaker than the beams framing into the column at such locations, the column should be detailed to enable it to develop a flexural hinge without critical strength degradation.

In addition to satisfying the strong-column/weak-beam requirement, each column section must satisfy the basic strength design requirement, that is, $(\phi M_n, \phi P_n) \geq (M_u, P_u)$ for each load combination. This is checked by plotting all combinations of (M_u, P_u) and ensuring that they fall within the design strength envelope given by $(\phi M_n, \phi P_n)$. For columns subjected to bi-directional moments, the requirement is $(\phi M_{nx}, \phi M_{ny}, \phi P_n) \geq (M_{ux}, M_{uy}, P_u)$.

Strong-column/Weak-beam Check

The requirement that the columns be stronger than the beams is intended 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 6.3.2.

A common construction approach 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 (PT in **Figure 5-17**) produces a fairly uniform compressive stress field across the plate including the beam cross section.

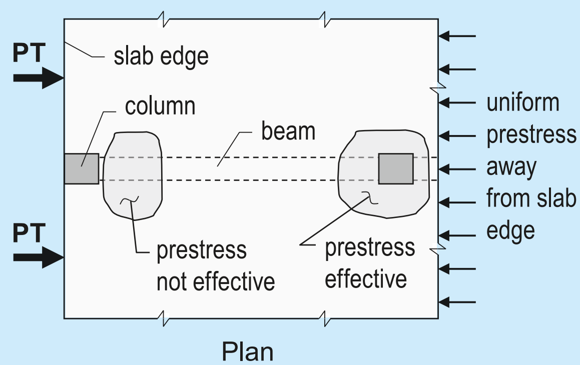


Figure 5-17. Beam with integral slab.

A reasonable approach is to calculate the average prestress acting on the combined slab-beam system and 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 that they are 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.

Expected Column Axial Forces

As specified in ASCE 7, column design axial forces are to be calculated using design load combinations, including earthquake load effects as appropriate. The usual approach is to calculate axial forces using a linear-elastic model of the building subjected to the ASCE 7 prescribed seismic design forces. Interior columns of special moment frames usually do not experience large variations in axial forces because of seismic effects. In contrast, the exterior and corner columns typically experience large variations in axial force because of seismic effects, which should be considered in design.

When a building responds to strong earthquake shaking, the beams may yield and develop probable moments and corresponding beam shears, as shown in **Figure 5-12**. These shears are transferred to the columns at each floor, accumulating forces over the height of the building and producing column axial forces that almost certainly exceed the forces obtained from linear analysis of the frames.

An upper bound on the column axial force variations can be obtained by assuming the frame develops a full beam yielding mechanism, as shown in **Figure 3-1(c)**. For an exterior column, the axial force could be as high as the sum of the shears V_{e2} from the yielding beams over the height of the building plus the loads from the column self-weight and other elements supported by the column (Moehle 2014).

Research shows that tall building frames are unlikely to develop the full beam yielding mechanism, so axial forces are unlikely to reach the upper bound described in the previous paragraph. Nonetheless, if the full beam yielding mechanism produces axial forces approaching P_o, the column capacity may be insufficient for actual earthquake demands and should be re-evaluated. Where project fees permit, nonlinear response history analysis under representative earthquake ground motions can provide a good measure of the expected column behavior. Where this approach is not feasible, this Guide recommends a conservative column design approach that keeps axial stresses caused by design axial forces low.

Column longitudinal bars should be well distributed around the perimeter of the column. This distribution improves confinement of the core concrete and improves transfer of forces through the beam-column joints. The exact requirements vary depending on column axial load and concrete compressive strength (see later discussion). Longitudinal bar lap splices, if any, must be located along the middle of the clear height and should not extend into the length ℓ_o at the column ends. Mechanical splices, if used, should be Type 2.

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 crushing of the concrete in 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 aims for lower axial loads. Limiting the design axial load to that corresponding to the balance point of the column interaction diagram is recommended. ACI 318 requires enhanced detailing of transverse reinforcement for $P_u > 0.3A_g f'_c$.

5.5.3 Shear and Confinement Reinforcement

Transverse reinforcement is required in columns to (a) confine the core concrete, (b) provide lateral support to longitudinal reinforcement, (c) confine longitudinal reinforcement lap splices, and (d) provide shear strength. To perform these various functions, the required reinforcement varies over the column length, as illustrated in **Figure 5-18**. The provided transverse reinforcement can simultaneously serve as confinement reinforcement, longitudinal bar support, lap splice confinement, and shear reinforcement. It is not required to sum the reinforcement required for each purpose, but, instead, independently satisfy all the requirements.

The column transverse reinforcement should initially be selected based on the confinement requirements of ACI 318 §18.7.5. These confinement requirements apply at the ends of the column where flexural yielding may occur, along a length ℓ_o (**Figure 5-18**). For rectangular cross sections, the total cross-sectional area of rectangular hoop reinforcement in each principal direction of the column cross section is not to be less than that required by the equations in **Table 5-1**. Of the applicable equations (a), (b), and (c), the required confinement is determined by the expression that gives the larger/largest amount.

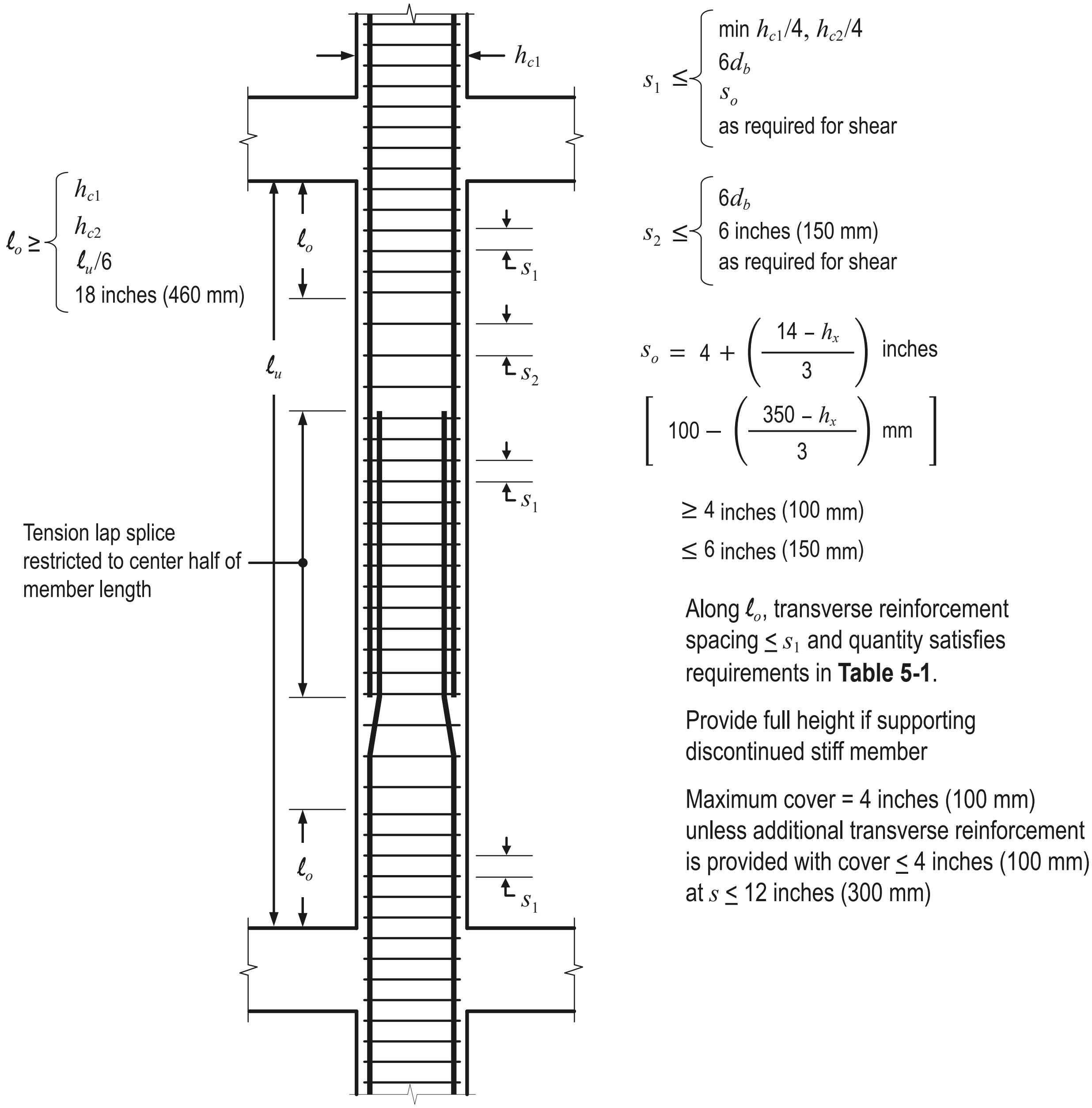


Figure 5-18. Column transverse reinforcement spacing requirements.

Conditions	Applicable Equations	
$P_u \le 0.3A_g f'_c$ and $f'_c \le$ 10,000 psi (70 MPa)	(a) and (b)	$\frac{A_{sh}}{sb_c} \ge 0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$ (a) $\frac{A_{sh}}{sb_c} \ge 0.09\frac{f'_c}{f_{yt}}$ (b) $\frac{A_{sh}}{sb_c} \ge 0.2\, k_f k_n \frac{P_u}{f_{yt} A_{ch}}$ (c)
$P_u > 0.3A_g f'_c$ or $f'_c >$ 10,000 psi (70 MPa)	(a), (b), and (c)	

Table 5-1. Required transverse reinforcement along lengths ℓ_o (ACI 318 §18.7.5.4).

In Equation (c) of **Table 5-1**, variables k_f and k_n are defined by

$$k_f = \frac{f'_c}{25{,}000} + 0.6 \ge 1.0 \text{ (psi)}$$

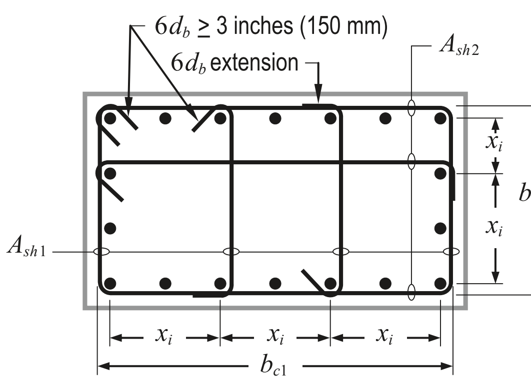
$$k_f = \frac{f'_c}{175} + 0.6 \ge 1.0 \text{ (MPa)}$$

$$k_n = \frac{n_l}{n_l - 2}$$

where n_l is the number of longitudinal bars or bar bundles around the perimeter of a column core that are laterally supported by the corner of hoops or by seismic hooks.

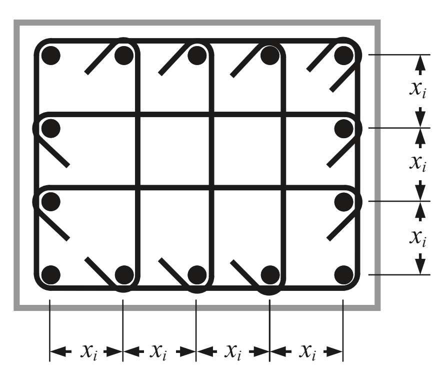
In addition to the requirements of **Table 5-1**, other confinement reinforcement detailing requirements also apply depending on the axial force and specified concrete compressive strength. If $P_u \le 0.3A_g f'_c$ and $f'_c \le$ 10,000 psi (70 MPa), it is acceptable to support alternate longitudinal bars using crossties with 135° and 90° hooks, provided no unsupported bar is farther than 6 inches (150 mm) clear from a laterally supported bar, and supported bars are not more than 14 inches (360 mm) apart (**Figure 5-19a**). If $P_u > 0.3A_g f'_c$ or $f'_c >$10,000 psi (70 MPa), then every longitudinal bar must be supported by a crosstie having included angle not less than 135° on both ends with maximum spacing between supported bars not exceeding 8 inches (200 mm) (**Figure 5-19b**).

- Every corner and alternate longitudinal bar shall have lateral support, and no bar shall be farther than 6 inches (150 mm) clear from a laterally supported bar.
- Consecutive crossties around the perimeter and along the length have their 90° hooks on opposite sides of column.
- The dimension x_i from centerline to centerline of supported bars shall not exceed 14 inches (360 mm).



(a) $P_u \le 0.3A_g f'_c$ and $f'_c \le$ 10,000 psi (70 MPa)

- Every longitudinal bar around the perimeter of the column core shall have lateral support, provided by the corner of a hoop or by a seismic hook.
- The dimension x_i from centerline to centerline of supported bars shall not exceed 8 inches (200 mm).



(b) $P_u > 0.3A_g f'_c$ or $f'_c >$ 10,000 psi (70 MPa)

Figure 5-19. Column transverse reinforcement detail (ACI 318 §18.7.5.2).

To illustrate the requirements, consider a column having $P_u \leq 0.3A_g f'_c$ and $f'_c \leq 10{,}000$ psi (70 MPa), such that the details of **Figure 5-19a** apply. In this case, Equation (a) and Equation (b) of **Table 5-1** apply. 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. If $P_u > 0.3A_g f'_c$ or $f'_c > 10{,}000$ psi (70 MPa), then the details of **Figure 5-19b** apply. In this case, Equation (a), Equation (b), and Equation (c) of **Table 5-1** apply, with A_{sh} in each direction determined in an analogous way. In this case, $k_l = 14/(14 - 1) = 1.17$.

Within the length ℓ_o, the maximum longitudinal spacing of hoop sets cannot exceed the smallest of (a), (b), and (c) below:

(a) The smaller of $h_{c1}/4$ and $h_{c2}/4$

(b) $6d_b$ of the smallest longitudinal bar

(c) s_o as calculated by

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \text{(inches)}$$

$$s_o = 100 + \left(\frac{350 - h_x}{3}\right) \text{(mm)}$$

The value of s_o shall not be taken greater than 6 inches (150 mm) and need not be taken less than 4 inches (100 mm).

Beyond the length ℓ_o, the column must have hoops with spacing not exceeding the smaller of $6d_b$ and 6 inches (150 mm). The exception is where the column longitudinal reinforcement is lap spliced, in which case hoops along the lap splice are required to satisfy the same requirements as specified for the length ℓ_o.

Column Hoop 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 hoop spacing outside the ℓ_o regions shown in **Figure 5-18**. For this reason, it is common practice to specify a uniform hoop spacing to prevent misplaced hoops 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_o regions. The benefit can be seen by counting the number of hoops that can be saved as the spacing is relaxed.

Hoop Configuration

Column hoops (see **Figure 5-20**) should be configured with at least three hoop or crosstie legs restraining longitudinal bars along each face. A single perimeter hoop without crossties, although permitted by ACI 318 for small column cross sections, is discouraged because confinement effectiveness is low.

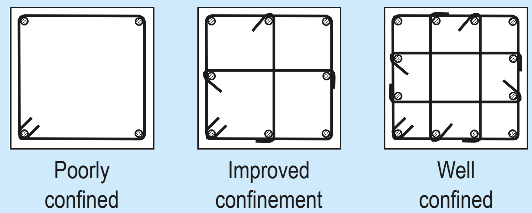


Figure 5-20. Column hoops.

ACI 318 does not permit the perimeter of a hoop to be made up of interlocking headed deformed bars because there are concerns about the heads becoming disengaged during construction or during an earthquake. ACI 318 is silent about the use of headed deformed bars as crossties. If used, the heads must fully engage the supported longitudinal reinforcement.

Once the transverse reinforcement has been selected in accordance with the preceding paragraphs, the shear strength of the column needs to be checked. ACI 318 §18.7.6.1 presents three distinct procedures a, b, and c for determining the design shear force V_e. 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 procedures are summarized below.

a. V_e shall not be less than the shear obtained by analysis of the building frame considering the governing design load combinations. See **Figure 5-21(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 5-21(b)**. As with beams, M_{pr} is calculated using strength reduction factor $\phi = 1.0$ and steel yield stress equal to $1.25f_y$. Furthermore, M_{pr} is to be taken equal to the maximum value associated with the anticipated range of axial forces. As shown in **Figure 5-22**, 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 forces.

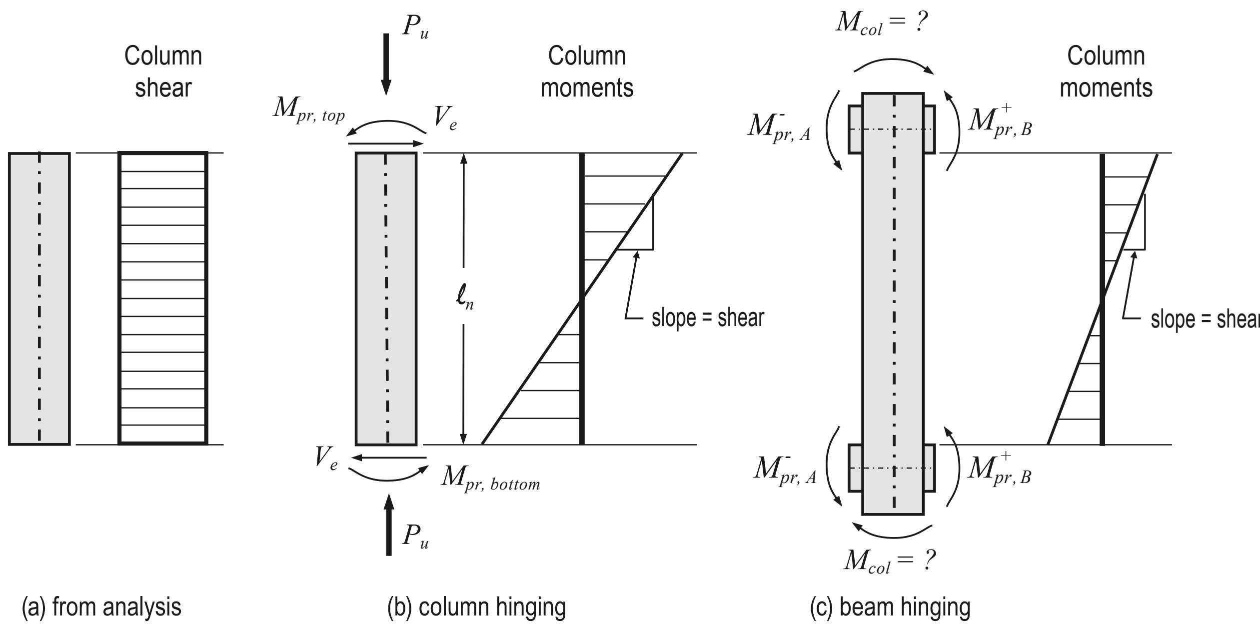


Figure 5-21. Column shear calculation options.

This approach is considered to be conservative because, barring some unforeseeable accidental loading, no higher shear can be developed in the column and 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 an alternative is offered.

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 5-21(c)**. The concept behind this approach is that the column shears need 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. A problem with this approach is that the distribution of column resisting moments above and below the joint is indeterminate. A common solution is to distribute the moments to the columns in proportion with the column flexural rigidity or to divide the moments equally to the columns above and below the joint. Analytical studies (Visnjic et al. 2014) have shown that both of these solutions can be unconservative by a wide margin, so neither is recommended here.

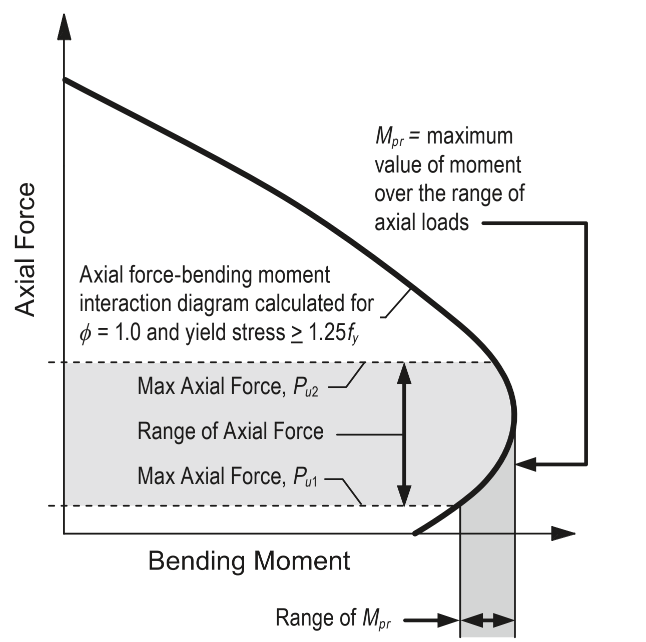


Figure 5-22. To find M_{pr} for a column, first determine the range of axial loads under design load combinations. M_{pr} is the largest moment for that range of axial loads.

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 is 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 required moment strengths M_u, where M_u is the moment at the beam section due to seismic loads alone, it is reasonable to anticipate shear forces reaching values equal to M_{pr}/M_u x 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_o, shown in **Figure 5-17**, for any load combination for which the column has low axial load ($< A_g f'_c/20$) and high seismic shear demand ($V_e \geq V_u/2$). 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.

5.5.4 Other Column Considerations

According to ACI 318 §18.2.2.3, if columns of a special moment frame extend below the base of the structure as shown in **Figure 4-3** 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 such that 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 §18.7.5.1. Increasing the length of the confinement zone to $1.5\ell_o$ is recommended.

At the roof level or other similar location, either the column should extend a short distance above the roof level, or the longitudinal bars should 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 systems whose performance depends on proper implementation of design requirements and detailing during construction. Therefore, wherever a special moment frame is used, regardless of the Seismic Design Category, ACI 318 §26.13 requires continuous special inspection of the placement of the reinforcement and concrete by a qualified special inspector. The special 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 that requires special inspection.

The special inspector is required to inspect work for conformance to the approved design drawings and specifications. Per IBC §1704, the engineer of record should designate the specific inspections and tests to be performed in a Statement of Special Inspections, submitted as part of the permit application. Contract documents should specify that the special inspector will furnish inspection reports to the building official, the engineer of record, owner, and contractor. Discrepancies should 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, completed in conformance with the approved plans and specifications and the applicable workmanship provisions of the IBC and its referenced standards.

6.2 Material Properties

Wherever a special moment frame is used, regardless of the Seismic Design Category, the concrete shell conform to special requirements of ACI 318 §18.2.5.1 and that the reinforcement shall conform to special requirements of ACI 318 §18.2.6.1. 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 §18.2.5.1 and Table 19.2.1.1, the specified compressive strength of concrete, f'_c, shall be not less than 3,000 psi (21 MPa). 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 (8.3 MPa) for any calculated shear strength or anchorage/development length derived from Chapters 22 and 25 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 §18.8. Beam-column joint shear strengths calculated without the 100 psi (8.3 MPa) limit were found conservative in laboratory tests having concrete compressive strengths up to 15,000 psi (100 MPa) (ACI 2002). 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. The reinforcement yield strength must meet at least the specified yield strength requirement, and the actual yield strength must not be too much higher than the specified yield strength. If it is much larger, the moment strength of yielding members will be greater than anticipated in design, resulting in higher forces being transmitted to adjacent members as the yield mechanism forms.

Additionally, flexural reinforcement must 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 ASTM International specification ASTM A706 (ASTM 2015). According to this specification, the actual yield strength must not exceed the specified yield strength by more than 18,000 psi (120 MPa), 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 (ASTM 2016) Grades 40 (275 in the SI designation) and 60 (420) reinforcement are permitted by ACI 318 if (a) actual yield strength based on mill tests does not exceed f_y by more than 18,000 psi (120 MPa); (b) ratio of the actual tensile strength to the actual yield strength is not less than 1.25; and (c) minimum elongation in 8 inches (200 mm) is at least 14 percent for bar sizes No. 3 through No. 6 (10 through 19), at least 12 percent for bar sizes No. 7 through No. 11 (22 through 36), and at least 10 percent for bar sizes No. 14 and No. 18 (43 and 57). 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 (520) and 80 (550)]. This reinforcement may perform suitably if the elongation and stress requirements match those of A706 reinforcement. However, higher strength reinforcement results in higher unit bond stresses, requires longer development and splice lengths, and may require closer spacing of transverse reinforcement to provide adequate bar support.

A706 Grade 80 Reinforcement

The use of A706 Grade 80 (550) reinforcement is a subject of current research (NIST 2014) and consideration by ACI Committee 318, but at the time of this writing, it is not generally permitted for longitudinal reinforcement of special moment frames. Where results of tests and analytical studies demonstrate its suitability, its use may be permitted under the alternative construction materials provisions of building codes.

Reinforcement with even higher strength, up to 100-ksi (690-MPa) 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 (690 MPa), and the value of f_{yt} used in design of shear reinforcement shall conform to ACI 318 §18.2.6.1 and §20.2.2 (that is, the maximum value is 60,000 psi (420 MPa) except 80,000 psi (550 MPa) 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 Type 2 splices, capable of developing nearly the tensile strength of the spliced bars. Outside yielding regions, mechanical splices, if used, are permitted to 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 §18.2.7; that is, they shall be capable of developing $1.25 f_y$ in tension or compression, as required; (b) Type 2 mechanical splices shall develop the specified tensile strength of the spliced bar but not necessarily the actual tensile strength.

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 reinforcement is welded where reinforcement is welded and proper attention paid to reinforcing bar metallurgy and to the welding process including filler metal to be employed. 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. 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 §18.12 presents requirements for diaphragms that are applicable wherever a special moment frame is used in Seismic Design Category D, E, or F. Additional guidance is provided in NIST (2016). 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 4-3**, the diaphragm serves to transmit the seismic forces from the special moment frames to the basement walls or other stiff elements of the podium. According to ASCE 7, transfer forces at such in-plane discontinuities in the vertical elements of the seismic force-resisting system must be amplified by the overstrength factor, Ω_o, and the resulting Ω_o-amplified forces apply to design of all components of the diaphragm. At other diaphragms, only the collectors and their connections need to be designed for forces amplified by the overstrength factor Ω_o.

6.3.2 Foundations

ACI 318 §18.13 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 318 §18.6.3, as described earlier in Section 5.2 and Section 5.4.

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

Section 2 of this Guide describes 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 frames," 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 story 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 §18.14, 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 frames.

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 the design team to make the beams and columns as small as possible, resulting in beams, columns, and joints that become very congested with reinforcement. 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).

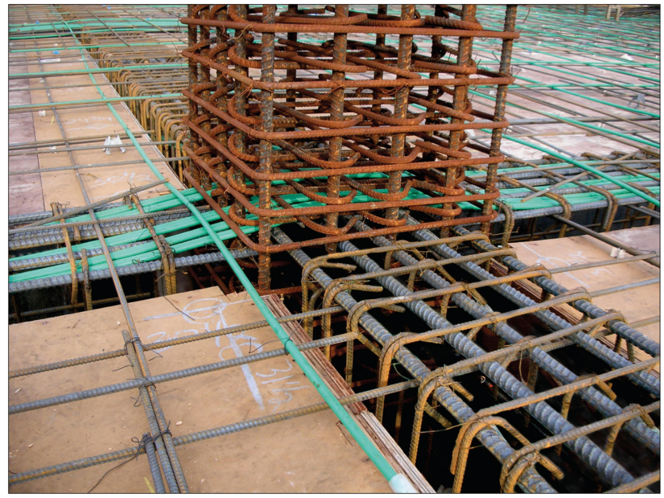


Figure 7-1. A well-detailed beam-column joint.

7.1 Longitudinal Bar Compatibility

In establishing the dimensions of beams and columns, the designer should recognize that larger member cross sections may enable use of smaller longitudinal reinforcement ratios, which helps with placement of reinforcing bars and concrete. The larger sections with lower reinforcement ratios also reduce design shear stresses in beams, columns, and joints, thereby improving performance during earthquake shaking.

When laying out the beam and column reinforcement, it is helpful to establish planes of reinforcement for the longitudinal bars. 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. Because all these planes need to extend through the beam-column joint, they cannot overlap. A good practice can be to have an even number of column bars on a face and an odd number of beam bars entering the face, or vice versa, such that the beam bars can easily fit between the column bars. Where longitudinal bars are well spaced, the column or beam bars can be shifted slightly at the intersection between the members, such that this odd-even guidance need not be followed. **Figure 7-1** shows a well-coordinated joint with five beam bars passing through a column face that has five vertical bars.

Beams and columns always need longitudinal bars close to their faces and at corners to hold the hoops and ties. Where the beam and column are the same width, these longitudinal bars are in the same plane in the beam and the column, and they conflict at the joint. Some solutions to this detailing conflict that have been implemented in practice include the following:

1. Bend and offset the outermost beam longitudinal bars near the joint (**Figure 7-2**). This solution moves the longitudinal bars out of the corners of the hoops, which may create interferences with seismic hooks, and creates bar eccentricities, both of which might reduce performance.

2. Move the main beam longitudinal bars inboard such that they can pass between the column bars without bending, and place smaller, discontinuous longitudinal bars in the corners of the beam hoops. Additional hoop or crosstie legs may be required to support the main beam longitudinal reinforcement. The minor discontinuity in longitudinal reinforcement created by this detailing practice might reduce performance.

3. Move the beam longitudinal bars inboard such that they can pass between the column bars without bending, and decrease the size of the beam hoops to tightly enclose the beam bars. This arrangement of bars results in increased cover on the beam hoops, which might reduce beam performance if the larger concrete cover spalls during earthquake loading.

4. Make the beam wider or narrower than the column. This solution may increase forming costs but reduces reinforcement fabrication costs relative to the other solutions and improves performance during earthquake loading. **Figure 7-1** illustrates a beam that is narrower than the supporting column. Where a beam is wider than the supporting column, ACI 318 requires additional transverse reinforcement to enclose the beam longitudinal bars through the joint.

Figure 7-2. A beam with longitudinal bars swept inward near the beam-column joint. The longitudinal bars are not tightly held within corners of hoops, which might reduce performance.

Making the beam wider or narrower than the column may create undesirable conditions along the exterior edge of a floor. The architectural condition along this exterior location must be considered. Even though different beam and column widths work well for the structure, this solution may create a complicated façade detail that increases cost. This Guide has a preference for the fourth detailing option above.

To support the beam hoops and stirrups, some of the top bars must be made continuous with lap splices or mechanical couplers near midspan. 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 because this condition makes placement very difficult, especially when two or more layers of bars must be hooked into the joint at an exterior column (**Figure 7-3**). If more than one layer of bars is required, it may be that 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. Beam-column joint having multiple layers of beam reinforcement hooked at back side of joint. The beam is upturned (the slab is cast at the bottom face of the moment frame beam).

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 5-19**, 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 exception is at exterior beams with a slab on one side only, in which case the crossties are required to be arranged such that the 90° hook is on the interior side of the beam. 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 loses its effectiveness. In contrast, a 135° hook remains 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). The heads must be properly engaged. Special inspection of their final placement is very important. ACI 318 does not permit the perimeter hoops to be made up of interlocking headed bars. 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.

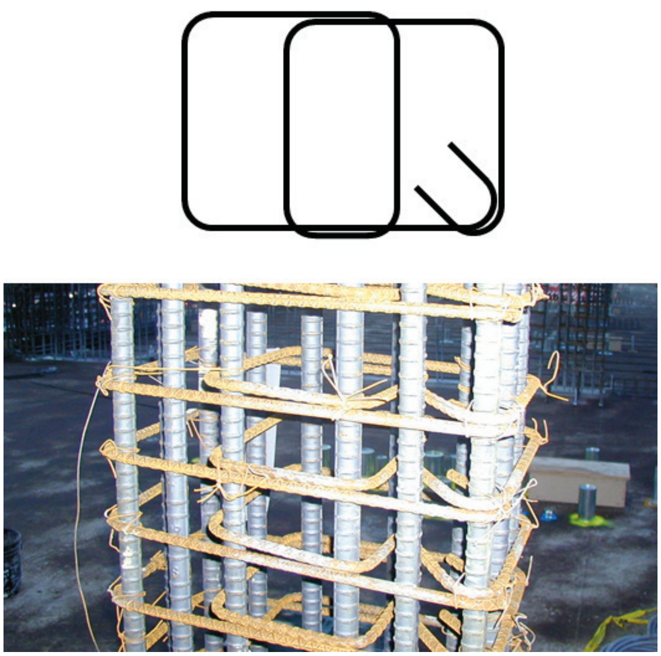


Figure 7-4. Column cage with hoops constructed from single reinforcing bar.

As described in Section 5.5, ACI 318 permits the horizontal spacing between legs of hoops and crossties to be as large as 14 inches (360 mm) in columns with low axial loads. Confinement can be improved by reducing this spacing. Longitudinal bars spaced around the perimeter no more than 6 or 8 inches (150 to 200 mm) apart is recommended. According to ACI 318 §18.7.5.3, vertical spacing of hoop sets can be increased from 4 inches to 6 inches (100 mm to 150 mm) as horizontal spacing of crosstie legs decreases from 14 inches to 8 inches (360 mm to 200 mm). 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, the hoop, and 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 No. 4 (13) or No. 5 (16) bars. Number 6 (19) and larger bars have large diameter bends and are difficult to place.

Although spirally reinforced columns are not treated in detail in this Guide, 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 §18.8.3.1, the spirals can be replaced within the joint by circular or rectangular hoop reinforcement.

7.3 Bar Splices

Lap splices of longitudinal reinforcement must be positioned outside intended yielding regions, as noted in Section 5.2 and Section 5.5. 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, especially 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_o 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_o (**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 in special moment frames. Mechanical splices also may help reduce congestion.

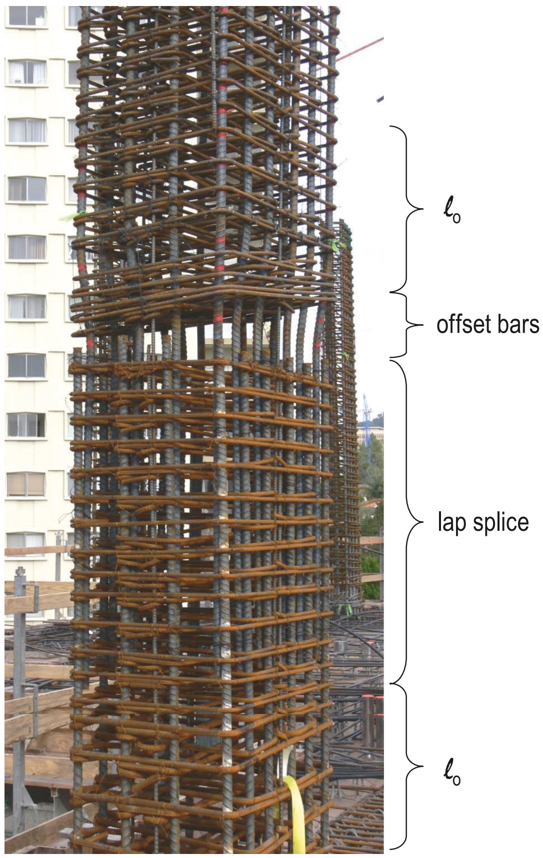


Figure 7-5. Column cage lap splices are not permitted to extend outside the middle half of the column length and should not extend into the length ℓ_o at the column end.

7.4 Anchoring to Concrete

ACI 318 §17.2.3 addresses anchoring to concrete in structures assigned to Seismic Design Categories C through F. The provisions of that section do not apply to design of anchors in potential plastic hinge regions because the anticipated higher degrees of cracking and spalling in such regions is beyond the conditions for which the ACI provisions are applicable. Plastic hinge regions are considered to extend a distance equal to twice the member depth from any beam face, column face, or any other section that may yield under design earthquake actions. The commentary of ACI 318 §17.2.3 provides additional discussion.

7.5 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 ½-inch (12 mm) 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 (180 to 230 mm) may be necessary to get the concrete to flow in the congested areas.

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

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 ties required to hold this type of concrete must also be spaced closer together than with a standard concrete mixture. The successful use of self-consolidating concrete is highly dependent on the experience and techniques of the contractor. For this reason, it is not recommended to specify self-consolidating concrete in the structural documents unless it has been previously discussed with the contractor.

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9. Notation and Abbreviations

Notations

A_{ch} cross-sectional area of a structural member measured to the outside edges of transverse reinforcement

A_g gross area of concrete section

A_j effective cross-sectional area within a joint in a plane parallel to plane of beam reinforcement generating shear in the joint

A_s area of nonprestressed longitudinal tension reinforcement

A'_s area of compression reinforcement

A_{sh} total cross-sectional area of transverse reinforcement, including crossties, within spacing s and perpendicular to dimension b_c

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

b_j effective width of beam-column joint

b_w web width

C_d deflection amplification factor as given in ASCE 7

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

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

D effect of service dead load

d_b nominal diameter of bar

E_h effect of horizontal seismic forces defined in ASCE 7

E_v effect of vertical seismic forces defined in ASCE 7

f'_c specified compressive strength of concrete, psi

f_y specified yield strength of nonprestressed reinforcement

f_{yt} specified yield strength of f_y transverse reinforcement

h_b value of h for beam

h_c value of h for column

h_{c1} dimension of rectangular or equivalent rectangular column measured in the direction of the span for which moments are being determined

h_{c2} dimension of rectangular or equivalent rectangular column measured in the direction perpendicular to h_{c1}

h_j effective depth of beam-column joint

h_{sx} story height below story x (note: x refers to a story, which is different from the definition of x in **Figure 5-8**)

h_x maximum value of x_i measured around the perimeter of the column

I_e effective moment of inertia for calculation of deflection

I_g moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

k_f concrete strength factor

k_n confinement effectiveness factor

ℓ_{dh} development length in tension of deformed bar with a standard hook, measured from outside end of hook, point of tangency, toward the critical section

ℓ_{dt} development length in tension of headed deformed bar, measured from the bearing face of the head toward the critical section

ℓ_n length of clear span measured face-to-face of supports

ℓ_o length, measured from joint face along axis of member, over which special transverse reinforcement must be provided

ℓ_t longitudinal bar extension beyond face of joint

ℓ_u unsupported length of column

L effect of service live load

M_n nominal flexural strength at section

M_{nb} nominal flexural strength of beam framing into joint, including slab where in tension

M_{nc} nominal flexural strength of column framing into joint, calculated for factored axial force, consistent with the direction of lateral forces considered, resulting in lowest flexural strength

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

M_u factored moment at section

n_l number of longitudinal bars around the perimeter of a column core with rectilinear hoops that are laterally supported by the corner of hoops or by seismic hooks. A bundle of bars is counted as a single bar

P_n nominal axial compressive strength

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

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

Q_E effects of horizontal seismic forces defined in ASCE 7

R response modification coefficient defined in ASCE 7

S_n nominal moment, shear, or axial strength

s_o center-to-center spacing of transverse reinforcement within the length ℓ_o

T fundamental period of the building defined in ASCE 7

T_a approximate fundamental period of building defined in ASCE 7

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

V_c nominal shear strength provided by concrete

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

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

V_e design shear force for load combinations including earthquake effects, assuming moments of opposite sign corresponding to probable flexural strength, M_{pr}, act at the joint faces

V_j beam-column joint shear for assumed frame yield mechanism

V_n nominal shear strength

V_s nominal shear strength provided by shear reinforcement

V_u factored shear force at section

w_u factored load per unit length of beam

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 5-8**)

x_i center-to-center distance between longitudinal bars supported by hoops or crossties

δ drift

ε_{cu} maximum concrete compressive strain

ε_t net tensile strain in extreme layer of longitudinal tension reinforcement at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature

ε_y value of net tensile strain in the extreme layer of longitudinal tension reinforcement used to define a compression-controlled section

γ coefficient defining joint nominal shear strength

λ modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal-weight concrete of the same compressive strength

ρ redundancy factor defined in ASCE 7

ϕ strength reduction factor

Ω_o overstrength factor

Abbreviations

ACI American Concrete Institute

ATC Applied Technology Council

ASCE American Society of Civil Engineers

ASTM ASTM International (formerly American Society for Testing and Materials)

CUREE Consortium of Universities for Research in Earthquake Engineering

EEG Earthquake Engineering Group

ELF Equivalent Lateral Force

IBC International Building Code

LRFD Load and Resistance Factor Method

LRH Linear Response History

MRS Modal Response Spectrum

NEHRP National Earthquake Hazards Reduction Program

NIST National Institute of Standards and Technology

10. Credits

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