Chapter 6

Seismic Design

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6.1 Introduction

Seismic design of reinforced concrete buildings is performed by determining earthquake design forces for the anticipated seismic activity in the region, from the building code adopted by the local authority. The structural elements are then proportioned and detailed following the requirements of Chapter 21 of ACI 318-05. Seismic design forces are determined on the basis of earthquake risk levels associated with different regions. Seismic risk levels have been traditionally characterized as low, moderate and high. These risk levels are considered in structural design to produce buildings with compatible seismic performance levels. ACI 318-05 has three design and performance levels, identified as *ordinary, intermediate* and *special*, corresponding to low, moderate and high seismic risk levels, respectively. Ordinary building design is attained for structures located in low seismic regions without the need to follow the special seismic design requirements of Chapter 21. These structures are expected to perform within the elastic range of deformations when subjected to seismic excitations. Buildings in moderate to high seismic risk regions are often designed for earthquake forces that are less than those corresponding to elastic response at anticipated earthquake intensities. Lateral force resisting systems for these buildings may have to dissipate earthquake induced energy through significant inelasticity in their critical regions. These regions require special design and detailing techniques to sustain cycles of inelastic deformation reversals without a significant loss in strength. The latter can be ensured by following the seismic provisions of ACI 318-05 outlined in Chapter 21.

The design and detailing requirements of ACI 31-05 are compatible with the level of energy dissipation assumed in selecting force modification factors and the resulting design force levels. While the level of detailing required may be *intermediate* for a building located in a moderate seismic risk region, it may be at the *special* category (more stringent) for a building in a high seismic risk region. This ensures appropriate level of toughness in the building. It is permissible, however, to design buildings for high toughness in the lower seismic zones to take advantage of lower design forces.

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Structural members not designed as part of the lateral-force resisting system may have to be designed as gravity load carrying members. These members, if present in special moment-resisting frames or special structural wall systems located in high seismic regions, must be protected during strong earthquakes as they continue carrying gravity loads tributary to them. They are often referred to as "gravity elements" and they "go for the ride" during the earthquake motion. Chapter 21 provides design and detailing requirements for such members in Sec. 21.11. Therefore, the structural engineer should first identify if the member under consideration for design is part of a lateral load resisting system, and if so, establish if the element is to be designed as intermediate or special seismic resisting element. **It is important to note that the requirements of Chapter 21 of ACI 318-05 are intended to be *additional* provisions, over and above those stated in other chapters of the Code for *ordinary* building design.**

6.2 Limitations on Materials

Certain limitations are imposed on materials used for seismic resistant construction to ensure deformability of members within the inelastic range of deformations. The following are the limits placed on concrete and reinforcing steel used in earthquake resistant designs:

i) $f'_c \geq 3000$ psi

ii) $f'_c < 5000$ psi for light-weight concrete, unless suitability is demonstrated by tests.

iii) Reinforcement shall comply with ASTM A-706. ASTM A-615 Grades 40 and 60 are permitted if they satisfy items iv and v below.

iv) $(f_y)_{Actual} - (f_y)_{Specified} \leq 18{,}000$ psi

v) $f_u / f_y \geq 1.25$

vi) $f_{yt} \leq 60{,}000$ psi for all transverse reinforcement, including spirals.

In addition to the above limitations, mechanical and welded splices of reinforcement and anchorage to concrete should meet the requirements of 21.26 through 21.2.8.

6.3 Flexural Members of Special Moment Frames

6.3.1 Flexural Design

Members designed to resist primarily flexure ($P_u \leq A_g f'_c/10$) are subject to additional design and detailing considerations for improved seismic performance. These requirements consist of geometric constraints, minimum positive and negative moment capacities along member length, confinement of critical regions of elements for improved deformability, promotion of ductile flexural response and the prevention of premature shear failure. Design aid **Seismic 1** illustrates the geometric constraints, as well as minimum top and bottom reinforcement requirements for minimum moment capacity in each section during lateral load reversals. The same design aid also shows the spacing requirements for concrete confinement at potential plastic hinge locations at member ends (within a distance equal to twice the member depth). The transverse confinement reinforcement consists of hoops, which may be made up of two pieces as illustrated in **Seismic 2**. Where hoops are not required outside the plastic hinge region, stirrups with seismic hoops should be used as also illustrated in **Seismic 2**.

6.3.2 Shear Design

Seismic induced energy in special moment resisting frames is expected to be dissipated through flexural yielding of members. During inelastic response, however, the members should be protected against premature brittle shear failure. This is ensured by providing sufficient shear capacity to resist seismic design shear forces. Seismic design shear V_e in plastic hinge regions is associated with maximum inelastic moments that can develop at the ends of members when the longitudinal tension reinforcement is in the strain hardening range (assumed to develop 1.25 f_y). This moment level is labeled as *probable flexural strength*, M_{pr}. Figure 6-1 illustrates the internal forces of a section that develop at probable moment resistance.

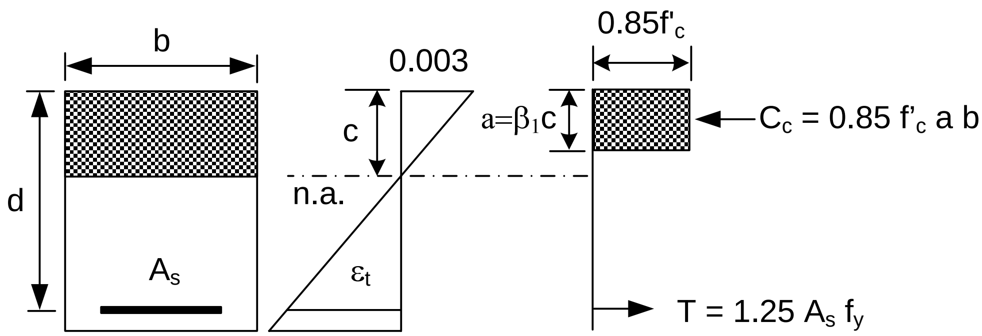


Fig. 6-1 Internal forces in a reinforced concrete section at probable moment resistance

M_{pr} for a rectangular section with tension reinforcement can be obtained from **Seismic 3.** This design aid provides values for coefficient K_{pr}, which is used to solve the following equation:

$$\mathbf{M_{pr} = 1.25\,A_s f_y (d - \frac{a}{2})} \qquad \textbf{(6-1)}$$

Where;

$$\mathbf{a = \frac{1.25\,A_s f_y}{0.85 f_c' \, b}} \qquad \textbf{(6-2)}$$

$$\mathbf{A_s = \rho bd} \qquad \textbf{(6-3)}$$

Substituting Eqs. 6-2 and 6-3 into 6-1 gives;

$$\mathbf{M_{pr} = K_{pr} bd^2} \qquad \textbf{(6-4)}$$

Where;

$$\mathbf{K_{pr} = 1.25\rho f_y (1 - 0.735\rho \frac{f_y}{f_c'})} \qquad \textbf{(6-5)}$$

Once M_{pr} is obtained, the seismic design shear can be computed from the equilibrium of forces shown in **Seismic 4**.

The contribution of concrete to shear, V_c within the plastic hinge region (length equal to twice the member depth at each end) may be negligibly small upon the formation of hinge due to the deterioration of concrete. Therefore, when V_e within the hinging region is equal to one-half or more of the maximum required shear strength, and the factored axial compression including earthquake effects is less than $A_g f'_c / 20$, V_c should be ignored completely in design ($V_c = 0$).

6.4 Special Moment Frame Members Subjected to Bending and Axial Load

6.4.1 Flexural Design

Members designed to resist earthquake forces while subjected to factored axial compressive force of $P_u > A_g f'_c/10$ are designed following the requirements of Sec. 21.4 of ACI 318-05. Columns that fall in this category are designed using the interaction diagrams provided in Chapter 3, with minimum and maximum reinforcement ratios of 1% and 6%, respectively. The 2% reduction in the maximum limit of reinforcement ratio from the 8% limit specified for ordinary building columns is intended to reduce the congestion of reinforcement that may occur in seismic resistant construction. ACI 318-05 also provides limitations on column cross sectional dimensions as illustrated in **Seismic 5.**

6.4.2 Strong-Column Weak-Beam Concept

In multistory reinforced concrete buildings it is desirable to dissipate earthquake induced energy by yielding of the beams rater than the columns. The columns are responsible from overall strength and stability of the structure, with severe consequences of failure. Furthermore, columns are compression members and axial compression reduces the ductility of reinforced concrete columns, thus necessitating more stringent confinement reinforcement. Therefore, it is preferable to control inelasticity in columns, to the extent possible, while dissipating most of the energy through yielding of the beams. This is known as the "strong-column weak-beam concept."

The strong-column weak-beam concept is enforced in the ACI Code through Sect. 21.4.2.2, which states that the flexural strength of columns should be 6/5 of that of the adjoining beams, as indicated below.

$$\sum \mathbf{M}_{nc} \geq \frac{6}{5} \sum \mathbf{M}_{nb} \qquad \textbf{(6-6)}$$

Where;

$\sum \mathbf{M}_{nc}$ is the sum of nominal flexural strengths of the columns framing into the joint, computed at the faces of the joint under factored axial forces such that they give the lowest flexural strength. Nominal flexural strengths of columns can be computed using the column interaction diagrams in Chapter 3.

$\sum \mathbf{M}_{nb}$ is the sum of the nominal flexural strengths of the beams framing into the joint, computed at the faces of the joint. For negative moment capacity calculations, the slab reinforcement in the effective slab width, as defined in Sec. 8.10 of ACI 318-05 and illustrated in **Flexure 6** should also be included, provided that they have sufficient development length beyond the critical section. The nominal flexural strength of a beam can be computed using the appropriate design aids in Chapter 1. (**Flexure 1 to Flexure 8**)

If Eq. (6-6) is not satisfied, the confinement reinforcement required at column ends, as presented in the next section and as required by Sec. 21.4.4 of ACI 318-05 will continue through the full height of the column.

6.4.3 Confinement Reinforcement

The behavior of reinforced concrete compression members is dominated by concrete (as opposed to reinforcement), which tends to be brittle unless confined by properly designed transverse reinforcement. In seismic resistant columns, where inelastic response is expected, sufficient ductility must be ensured through the confinement of core concrete. This can be achieved by using spiral reinforcement or closely spaced hoops, overlapping hoops, and crossties. The increased inelastic deformability is assumed to be met if the column core is confined sufficiently to maintain column concentric load capacity beyond the spalling of cover concrete. This performance criterion results in the following minimum confinement reinforcement as stated in Sec. 21.4.4 of ACI318-05:

i) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s shall not be less than;

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \qquad \textbf{(6-7)}$$

$$\rho_s = 0.12\frac{f_c'}{f_{yt}} \qquad \textbf{(6-8)}$$

ii) The total cross sectional area of rectangular hoop reinforcement, A_{sh} shall not be less than;

$$A_{sh} = 0.3\, s\, b_c \frac{f_c'}{f_{yt}}\left(\frac{A_g}{A_{sh}} - 1\right) \qquad \textbf{(6-9)}$$

$$\rho_s = 0.09\, s\, b_c \frac{f_c'}{f_{yt}} \qquad \textbf{(6-10)}$$

The above reinforcement should be provided with due considerations given to their spacing, both along the column height and column cross-sectional plane, for increased effectiveness of concrete confinement. The spacing requirements of ACI 318-05 for transverse confinement reinforcement are indicated in **Seismic 5**. The volumetric ratio of spiral and circular hoop reinforcement for circular columns and the reinforcement ratio of rectilinear transverse reinforcement in square and rectangular columns can be obtained from **Seismic 6** and **Seismic 7**, respectively.

6.4.4 Shear Design

Seismic design shear in columns is computed from **Seismic 4** as shear force associated with the development of probable moment strength (M_{pr}) at column ends when the associated factored axial force, P_u is acting on the column. These moments are computed with reinforcement strengths in tension equal to 1.25 f_y, reflecting the contribution of longitudinal column reinforcement in the strain hardening range. However, the column capacity is often governed by the crushing of compression concrete without excessive yielding of tension reinforcement. Therefore, the engineer should exercise judgment in selecting the probable moment strength for columns, depending on the level of accompanying axial compression. A conservative approach for estimating column M_{pr} for shear calculations is to use nominal moment capacity at balanced section, since this would be the maximum moment capacity for the column. The seismic shear V_e obtained in this manner need not exceed the seismic shear force associated with the formation of plastic hinges at the ends of the framing beams, i.e., column shear balancing seismic shear at the ends of the beams when probable moment resistance of beams, M_{pr} are developed at beam ends. However, at no case shall V_e computed above be less than the factored column shear force determined by analysis of the structure under seismic loading.

Once the seismic design shear force is computed, the plastic hinge regions at the ends of the column (ℓ_0 defined in **Seismic 5**) will be designed for V_e. In the design, however, the shear resistance provided by concrete, V_c will be neglected ($V_c = 0$) if *both* of the following conditions are met:

i) $V_e \geq$ 50% of the maximum shear strength required within ℓ_0 due to the factored column shear force determined by structural analysis.

ii) P_u (including earthquake effects) < $A_g f'_c / 20$.

6.5 Joints of Special Moment Frames

The formation of plastic hinges at the ends of beams may result in significant shear force reversals in beam-column joints. Joint shear can be determined by computing the internal forces acting on the joint while assuming that the tension beam reinforcement anchored into the joint develops 1.25 f_y. **Seismic 8** and **Seismic 9** illustrate the shear force acting in interior and exterior joints.

6.5.1 Joint Shear Strength

The joint shear produces diagonal tension and compression reversals which may be critical for premature diagonal tension or compression failures, unless properly reinforced. The joint shear may especially be critical in edge and corner joints, which are not confined by the adjoining beams on all four faces. A member that frames into a joint face is considered to provide confinement to the joint if at least ¾ of the face of the joint is covered by the framing member. The shear capacity of beam-column joints in special moment resisting frames can be computed by the following expressions given in Sec. 21.5.3 of ACI 31805.

i) For joints confined on all four faces: $\mathbf{V_n \leq 20\sqrt{f'_c}\ A_j}$ **(6-11)**

ii) For joints confined on three or two opposite faces: $\mathbf{V_n \leq 15\sqrt{f'_c}\ A_j}$ **(6-12)**

iii) For other joints: $\mathbf{V_n \leq 12\sqrt{f'_c}\ A_j}$ **(6-13)**

Where, A_j is the effective joint cross-sectional area defined in Fig. 6-2 as the column depth (column dimension in the direction of joint shear) times the effective width of the column, which is equal to the column width except where the beams frame into a wider column.

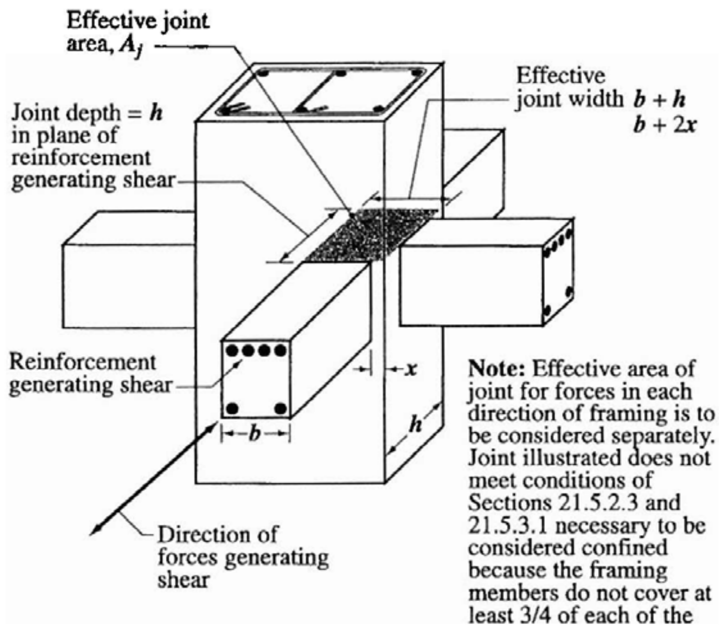


Fig. 6-2 Definition of effective joint area A_j

6.5.2 Joint Reinforcement

The column confinement reinforcement provided at the ends of columns should continue into the beam-column joint if the joint is *not* confined by the framing beams on all four faces, as described in the previous section. For interior joints, with attached beams externally confining the joint on all four faces, the spacing of joint reinforcement can be relaxed to 6 in.

6.6 Members of Intermediate Moment Frames

6.6.1 Flexural Design

Members of intermediate moment frames located in regions of moderate seismicity and are designed to resist primarily flexure ($P_u \leq A_g f'_c/10$), will meet the beam design requirements of **Seismic 10**. This design aid provides guidance for both the longitudinal flexural reinforcement and transverse confinement reinforcement. Members subjected to higher axial loads will be designed as columns following the requirements for columns outlined also in **Seismic 10,** unless the column is designed to have spiral reinforcement.

The transverse reinforcement in beam-column joints of intermediate moment frames will conform to Sec. 11.11.2 of ACI 318-05.

6.6.2 Shear Design

The shear strength ϕV_n of members of intermediate moment frames will be at least equal to the shear force associated with the development of nominal capacities of members at their ends while also subjected to the effects of factored gravity loads. Also, the shear strength should not be lower that the maximum shear obtained from the design load combinations where the earthquake loading is assumed to be twice the magnitude prescribed by the governing code. **Seismic 11** shows the design shear force V_u associated with the development of nominal member strengths at the ends.

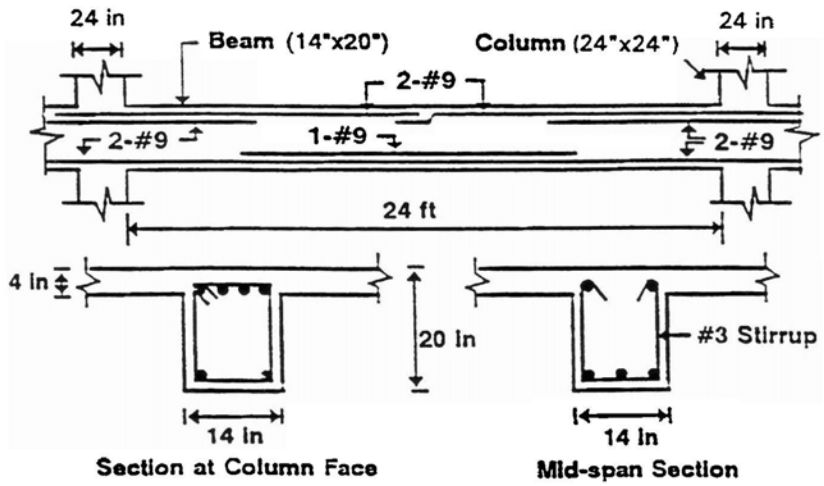
6.7 Members not Designed as Part of the Lateral-Force-Resisting System

Members of structures located in regions of high seismic risk, but not forming part of the lateral force resisting system, must be investigated for sufficient deformability during seismic response. These members, although not designed to resist seismic forces will deform along with the seismic lateral force resisting system. Therefore, they should have adequate strength and deformability to allow the development of design displacement δ_u, as per the requirements of Sec. 21.11 of ACI 318-05.

6.8 Seismic Design Examples

SEISMIC DESIGN EXAMPLE 1 - Adequacy of beam flexural design for a special moment frame.

The beam shown is designed for flexure using factored design loads. Check if the beam meets the seismic design requirements for flexure if it is part of a special moment frame located in a seismically active region.



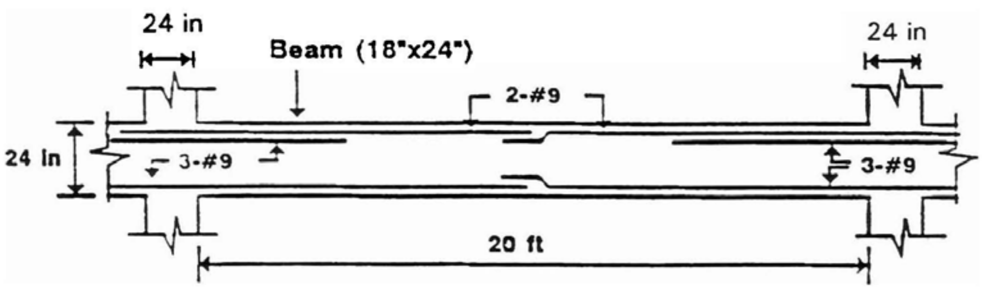
<u>**Given:**</u>
f'_c = 4,000 psi
f_y = 60,000 psi
Clear cover: 1.5 in

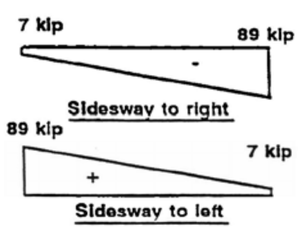
Procedure	Calculation	ACI 318-05 Section	Design Aid
Check geometric constraints for the beam.	d = 20 - 1.5 - 0.375 - 1.128/2 = 17.6 in i) Clear span l_n= 24 ft ≥ 4d = 5.8 ft O.K. ii) b_w/h = 14/20 = 0.7 > 0.3 O.K. iii) b_w > 10 in O.K. and $b_w = c_2$ O.K.	21.3.1.2 21.3.1.3 21.3.1.3	Seismic 1
Check for minimum and maximum ratio of longitudinal reinforcement	i) $(\rho_{min})_{top} = (\rho_{min})_{bott.} = 3\sqrt{f'_c}/f_y$ = 0.32% ii) $(\rho_{min})_{top} = (\rho_{min})_{bott.} = 200/f_y$ = 0.33 % iii) 2 # 9 bars result in ρ = 0.81% O.K. 2 # 9 top and bottom continuous bars. iv) ρ_{max} = 2.5 % O.K.	21.3.2.1	Seismic 1
Check for minimum positive and negative moment capacity at each section.	i) $M_n^+ \geq 0.5M_n^-$ at column face; ρ^- = 4 (1.0) / [(14)(17.6)] = 1.62 % $M_n^- = K_n bd^2/12000$ = (834) $(14)(17.6)^2$ /12000 = 301 ft-kips ρ^+ = 2 (1.0) / [(14)(17.6)] = 0.81 % $M_n^+ = K_n bd^2/12000$ = (452) $(14)(17.6)^2$ /12000 = 163 ft-kips 163 > 0.5 (301) = 151 ft-kips O.K. ii) $M_n^+ \geq 0.25 (M_n^-)_{max}$ at any section; $(M_n^+)_{min}$=163 > 0.25 (301) = 75 ft-kip iii) $M_n^- \geq 0.25 (M_n^-)_{max}$ at any section; $(M_n^-)_{min}$= 163 ft-k > 0.25(301)= 75 ft-k	21.3.2.2	Seismic 1

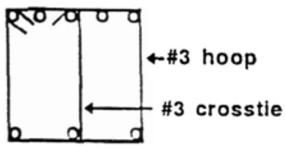
SEISMIC DESIGN EXAMPLE 2 - Design of the critical end regions of a beam in a special moment frame for shear and confinement.

The beam shown below is part of a special moment frame located in a high seismic risk area. Design the potential hinging regions of the beam for transverse reinforcement. Maximum shear strength required by analysis under factored loads is 82 kips.

Given:
f'_c = 4,000 psi
f_y = 60,000 psi
Clear cover: 1.5 in
Live load: 1.20 k/ft
Dead load: 2.45 k/ft
Interior beam
$(V_{max})_{req.}$ = 82 kips



Procedure	Calculation	ACI 318-05 Section	Design Aid
Determine design shear force V_e associated with the formation of plastic hinges at beam ends. First compute probable moment strength (M_{pr}) for positive and negative bending.	Assuming #3 hoops, effective depth d: d = 24 - 1.5 - 0.375 - 1.128/2 = 21.6 in $\rho^- = 5\ (1.0) / [18(21.6)] = 0.0129$ K^-_{pr} = 830 psi; $M^-_{pr} = K^-_{pr}\ bd^2/12000$ $M^-_{pr} = 830(18)(21.6)^2/12000 = 581$ ft-k $\rho^+ = 3\ (1.0) / [18(21.6)] = 0.0077$ K^+_{pr} = 528 psi; $M^+_{pr} = K^+_{pr}\ bd^2/12000$ $M^+_{pr} = 528[(18)(21.6)^2]/12000 = 370$ ft-k	21.3.4.1	Seismic 3
Compute design shear force V_e associated with the formation of M_{pr} at member ends while the member is loaded with factored gravity loads. Shear force diagrams;  	$w_u = 1.2\ D + 1.0\ L + 0.2S$ Int. beam; S=0 $w_u = 1.2(2.45) + (1.0)(1.20)] = 4.14$ k/ft $V_e = \frac{M_{pr1} + M_{pr2}}{\ell_n} \pm \frac{w_u \ell_n}{2}$ $V_e = \frac{370 + 581}{20} \pm \frac{4.14(20)}{2}$ $V_e = 48 \pm 41 = 89$ kips	21.3.4.1	Seismic 4
Check the magnitude of seismic induced shear relative to the maximum design shear required under factored loads and determine the contribution of concrete to shear strength, V_c..	$(V_{max})_{req.}/2$ = 82/2 = 42 kips $V_e > (V_{max})_{req.}/2$ Also, $P_u < A_g f'_c / 20$ (beam) Therefore, $V_c = 0$ (within the hinging region; 2h)	21.3.4.2	

Procedure	Calculation	ACI 318-05 Section	Design Aid
Determine vertical shear reinforcement at the critical section. 	Use #3 perimeter hoops and cross ties as shown in the figure. $\phi V_s = V_e$; $V_s = 89/0.75 = 119$ kips $s = (A_v f_y d)/V_s$ s = (3x0.11)(60)(21.6)/119 = 3.6 in	21.3.4.2 11.5.7	
Provide hoop steel in the potential hinge region at member ends for concrete confinement.	$s < d/4 = 21.6/4 = 5.4$ in $< 8\ (d_b)_{long.} = 8(1.128) = 9$ in $< 24\ (d_b)_{hoop} = 24(0.375) = 9$ in < 12 in spacing required for shear is 3.6 in. Therefore, use s = 3.5 in within 2h = 2(24) = 48 in (4 ft) distance from the column face at each end, with the first hoop located not more than 2 in from the column face.	21.3.3	Seismic 1
Check hoop detailing.	Perimeter hoops and cross ties provide lateral support to at least every other longitudinal reinforcement on the perimeter by the corner of a hoop or the hook of a cross tie. No longitudinal bar is farther than 6 in from a laterally supported bar. Hooks should extend $6d_b$ or 3 in.	21.3.3.3 7.10.5.3 21.3.3.6	Seismic 2

SEISMIC DESIGN EXAMPLE 3 - Design of a column of a special moment frame for longitudinal and confinement reinforcement.

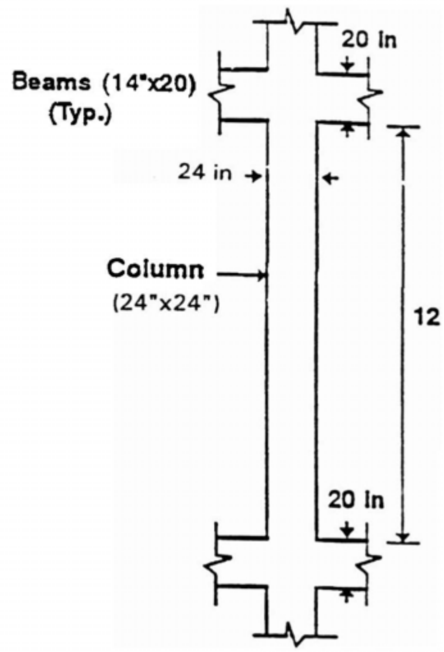
The column shown has a 24 in square cross-section, and forms part of a special moment column. Design the column for longitudinal and confinement reinforcement. Assume that the slenderness effects are negligible, and the framing beams are the same as that given in SEISMIC DESIGN EXAMPLE 1.

Given:

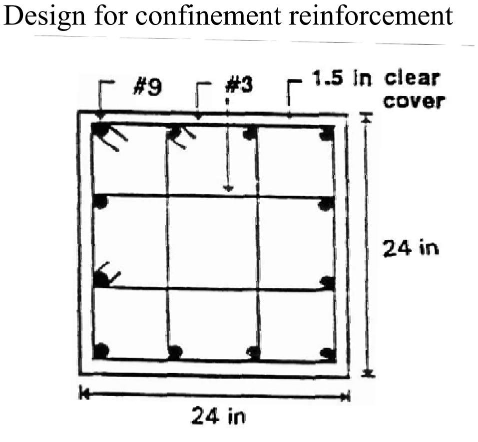
f'_c = 4,000 psi
f_y = 60,000 psi
Clear cover: 1.5 in
Slenderness is negligible
Column is bent in double curvature

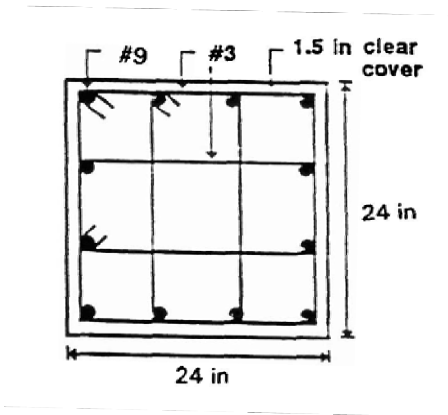
i) Design forces for sidesway to right:
ϕP_n= 1079 kip
$(\phi M_n)_{top}$= 390 ft-kip
$(\phi M_n)_{bot.}$= 353 ft-kip

ii) Design forces for sidesway to left:
ϕP_n= 910 kip
$(\phi M_n)_{top}$= 367 ft-kip
$(\phi M_n)_{bot.}$= 236 ft-kip



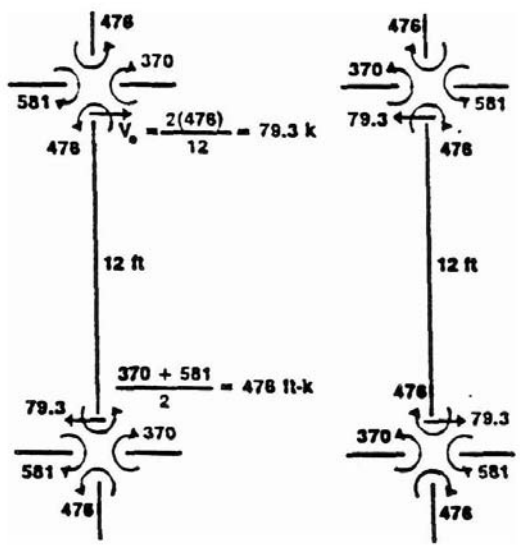
Procedure	Calculation	ACI 318-05 Section	Design Aid
Determine column size	Given: h = b = 24 in		
Check if slenderness effects may be neglected.	Given : Slenderness effects are negligible.	10.12.2	
Check the level of axial compression	$A_g f'_c/10$ = (576)(4000)/[(10)(1000)] = 230 kips ϕP_n = 1079 kips > 230 kips. Therefore, the requirements of section 21.4 apply.	21.4.1	
Check geometric constraints. Note that the beam reinforcement is continuous over the support (#9 bars with b_d = 1.128 in).	h = b = 24 in > 12 in O.K. 24/24 = 1.0 > 0.4 O.K. h = 24 in > 20 b_d = 20(1.128) = 22.6 in O.K.	21.4.1.1 21.4.1.2 21.2.1.4	Seismic 5
Determine longitudinal reinforcement. First select the appropriate interaction diagram. Estimate γ for a column section of 24 in, cover of 1.5 in, and assumed bar sizes of #3 ties and #9 longitudinal bars.	γ = [24 - 2(1.5 + 0.375) - 1.128)] / 24 γ = 0.80 Square cross-section. If equal area of reinforcement is to be provided on four sides, select Columns 3.2.3 interaction diagrams.		Columns 3.2.3

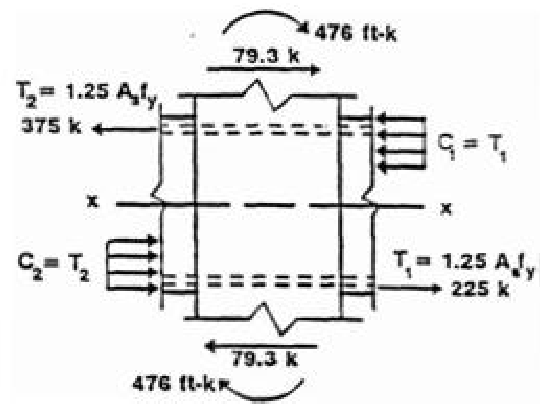
Procedure	Calculation	ACI 318-05 Section	Design Aid
Select the critical design loads and compute; $K_n = P_n /(f'_c A_g)$ and $R_n = M_n / (f'_c A_g h)$ Obtain reinforcement ratio ρ from interaction diagrams. Select longitudinal reinforcement	ϕP_n = 1079 kips; P_n =1079/0.65 =1660 kips ϕM_n = 390 ft-k; M_n = 390/0.65 = 600 ft-k $A_g = (24)(24) = 576\ in^2$ $P_n / (f'_c A_g)$ = 1660 /[(4)(576)] = 0.72 $M_n / (f'_c A_g h)$ = (600 x 12) / [(4)(576)(24)] = 0.13 for γ = 0.80; ρ = 0.019 $A_s = A_g \rho$ A_s = 576 x 0.019 $A_s = 10.94\ in^2$ (req'd); use 12 #9 bars $A_s = 12 \times 1.00 = 12.0\ in^2$ (provided)	10.2 10.3	Columns 3.2.3
Check the limits of reinforcement ratio ρ	$(\rho)_{prov.}$ = 12.0/576 = 0.021 0.01 < 0.021 < 0.006 O.K.	21.4.3.1	
Check flexural strengths of columns and beams at each joint. Determine column strength M_{nc} from interaction diagram Determine strengths of the adjoining beams from SEISMIC DESIGN EXAMPLE 1	$\sum M_{nc} \geq \frac{6}{5} \sum M_{nb}$ For sidesway to right: ϕP_n = 1079 kips; P_n =1079/0.65=1660 kips for ρ = 0.021 and $P_n / f'_c A_g$ = 0.72; $M_{nc} /(f'_c A_g h)$ = 0.14 when γ = 0.80 M_{nc}=(0.14)(4)(576)(24)/12= 645 ft-kips From SEISMIC DESIGN EXAMPLE 1: M^+_{nb} = 163 ft-kips M^-_{nb} = 301 ft-kips (2)(645)=1290>(6/5)(163+301)=557 O.K. For sidesway to left: ϕP_n = 910 kips; P_n = 910/0.65= 1400 kips for ρ = 0.021 and $P_n / f'_c A_g$ = 0.61; $M_n /(f'_c A_g h)$ = 0.16 when γ = 0.80 M_n=(0.16)(4)(576)(24)/12= 737 ft-kips From SEISMIC DESIGN EXAMPLE 1: M^+_{nb} = 163 ft-kips M^-_{nb} = 301 ft-kips (2)(327)=1474>(6/5)(163+301)=557 O.K.	21.4.2.2	Columns 3.2.3 Columns 3.2.3
Design for confinement reinforcement 	$A_g = (24)^2 = 576\ in^2$ $A_{ch} = [24 - 2(1.5)]^2 = 441\ in^2$ A_g/A_{ch} = 1.31 $\rho_c = 0.0062 = A_{sh}/(s b_c)$; Try #3 overlapping hoops as shown, $A_{sh} = 4(0.11) = 0.44\ in^2$; b_c = 20.6 in s = 0.44/[(0.0062)(20.6)] = 3.45 in	21.4.4.1	Seismic 7

Procedure	Calculation	ACI 318-05 Section	Design Aid
Check for maximum spacing of hoops. 	$s < 24/4 = 6$ in O.K. $s < 6(b_d)_{long.} = 6(1.125) = 6.75$ in O.K. $s < s_0 = 4 + (14 – h_x)/3 = 4 + (14 – 10.3)/3 = 5.23$ in O.K. spacing of hoop legs, $h_x < 14$ in O.K. use #3 overlapping hoops @ 3.5 in spacing. $\ell_0 \geq h = 24$ **in** $\ell_0 \geq \ell_c / 6 = 24$ **in** $\ell_0 \geq 18$ **in** Provide hoops over 24 in (2 ft) top and bottom, measured from the joint face. <u>Note:</u> Because the strong-column weak-beam requirement of 21.4.2.2 was met, the confinement reinforcement need not be provided throughout the entire column length. Also, the contribution of the column to lateral strength and stiffness of the structure can be considered.	21.4.4.2 21.4.4.3 21.4.4.4 21.4.2.3 21.4.2.1	Seismic 5 Seismic 5

SEISMIC DESIGN EXAMPLE 4 – Shear strength of a monolithic beam-column joint.

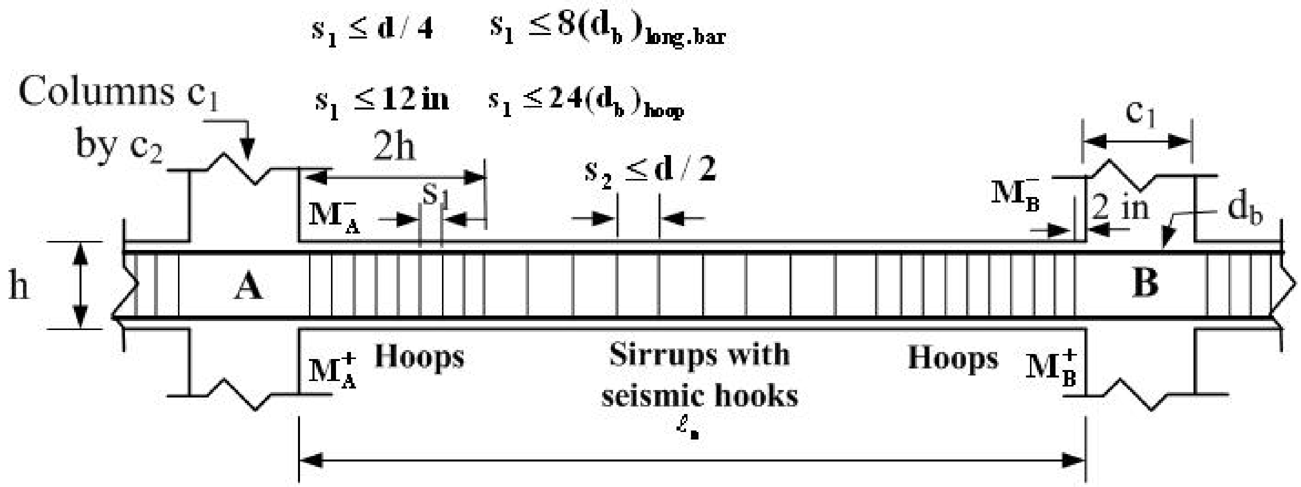
Consider a special moment frame and check the shear strength of an interior beam-column joint. The columns have a 24 in square cross-section, and a 12 ft clear height. The maximum probable moment strength of columns is $(M_{pr})_{col.}$= 520 ft-kips. The framing beams have the same geometry and reinforcement as those given in SEISMIC DESIGN EXAMPLE 2. f'_c = 4,000 psi; f_y = 60,000 psi.

Procedure	Calculation	ACI 318-05 Section	Design Aid
Compute column shear force V_e associated with the formation of plastic hinges at the ends of columns, i.e. when probable moment strengths, $(M_{pr})_{col.}$ are developed.	$V_e = 2(M_{pr})_{col.} / 12$ V_e = 2(520) / 12 = 86.7 kips	21.4.5.1	Seismic 4
Note that column shear need not exceed that associated with formation of plastic hinges at the ends of the framing beams. Compute V_e when probable moment strengths are developed at the ends of the beams.	From SEISMIC DESIGN EXAMPLE 2; M^-_{pr} = 581 ft-k and M^+_{pr} = 370 ft-k  Sidesway to right Sidesway to left V_e = 79.3 kips < 86.7 kips Therefore, use V_e= 79.3 kips.	21.4.5.1	Seismic 4

Procedure	Calculation	ACI 318-05 Section	Design Aid
Compute joint shear, when stress in flexural tension reinforcement of the framing beams is $1.25f_y$. Note that, in this case, because the framing beams have symmetrical reinforcement arrangements, the joint shear associated with sidesway to left would be the same as that computed for sidesway to right.	 Sidesway to right $V_{x\text{-}x} = T_2 + C_1 - V_e$ $V_{x\text{-}x} = 375 + 225 - 79.3 = 520.7$ kips	21.5.1.1	Seismic 8
Compute shear strength of the joint. The joint is confined externally by four framing beams, each covering the entire face of the joint.	$\mathbf{V_c = 20\sqrt{f'_c}A_j}$ $A_j = (24)(24) = 576$ in^2 $\boldsymbol{\phi}\mathbf{V_c = (0.75)20\sqrt{4000}(576)/1000}$ $\phi V_c = 546$ kips > 520.7 kips O.K.	21.5.3	
Note that transverse reinforcement equal to at least one half the amount required for column confinement, has to be provided, with absolute maximum tie spacing relaxed to 6 in.		21.5.2.2	

6. Seismic Design Aids

Seismic 1 – Flexural design requirements for beams of special moment frames



Top, as well as bottom reinforcement, each;

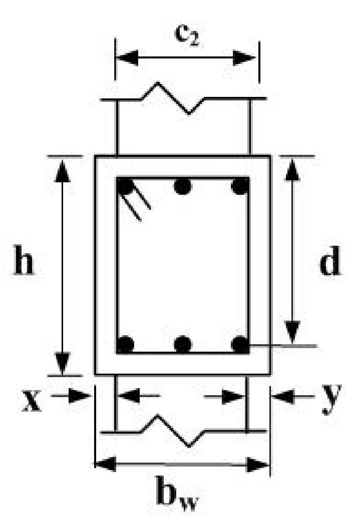
- **Shall not be less than:**
  - $A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d$
  - $200 b_w d / f_y$
  - **Two continuous bars**
- **Shall not exceed:** $\rho = 0.025$

Minimum flexural capacities;

- $M_A^+ \ge 0.5 M_A^-$
- $M_B^+ \ge 0.5 M_B^-$
- **If the largest end moment is M_{max};**
  at any section along the beam:

  $M^+ \ge 0.25 M_{max}$

  $M^- \ge 0.25 M_{max}$



$b_w \ge 0.3h$ $\quad$ $b_w \le c_2 + x + y$

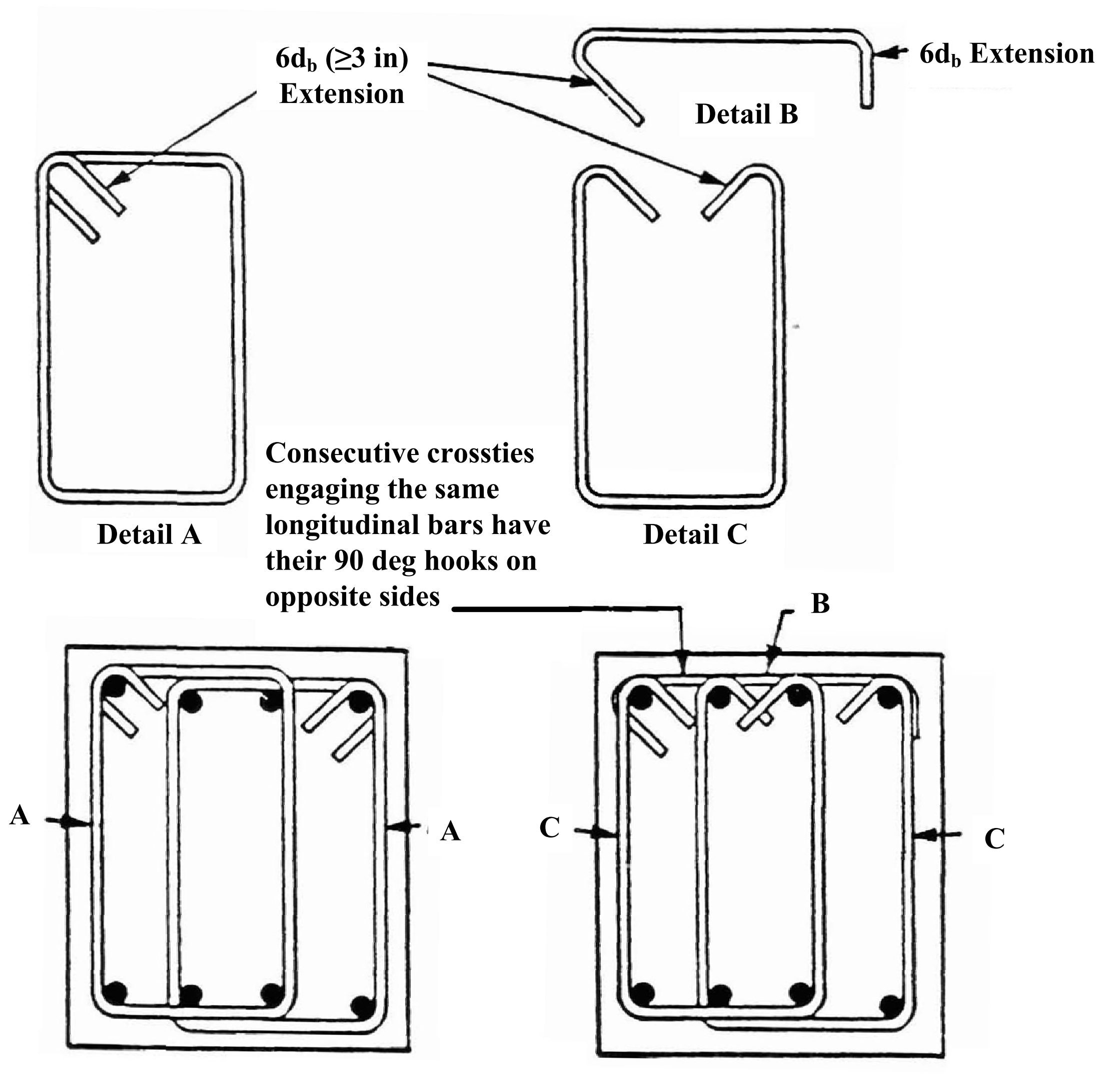
$b_w \ge 10$ in $\quad$ $x \le 3/4h$

$\ell_n \ge 4d$ $\quad$ $y \le 3/4h$

If beam reinforcement is continuous over the support:

$c_1 \ge 20 d_b$ (NWC) $\quad$ $c_1 \ge 26 d_b$ (LWC)

Seismic 2 – Details of transverse reinforcement for beams of special frames



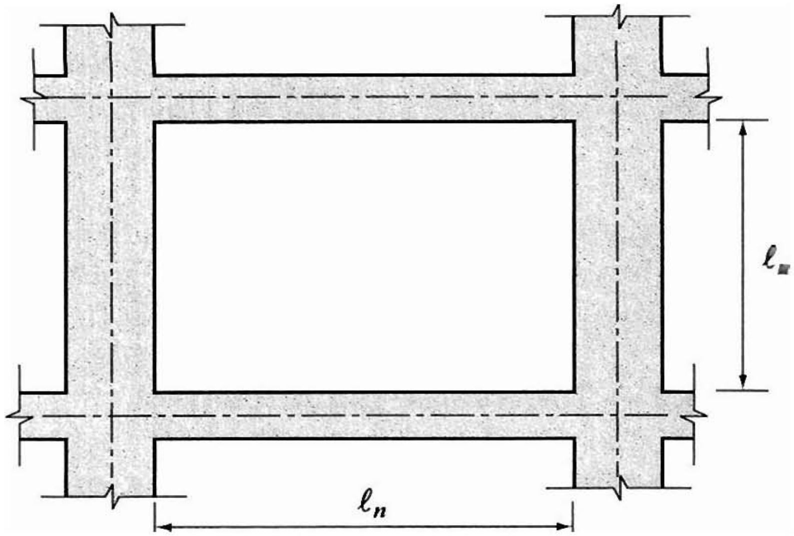
Overlapping Hoops for Beams

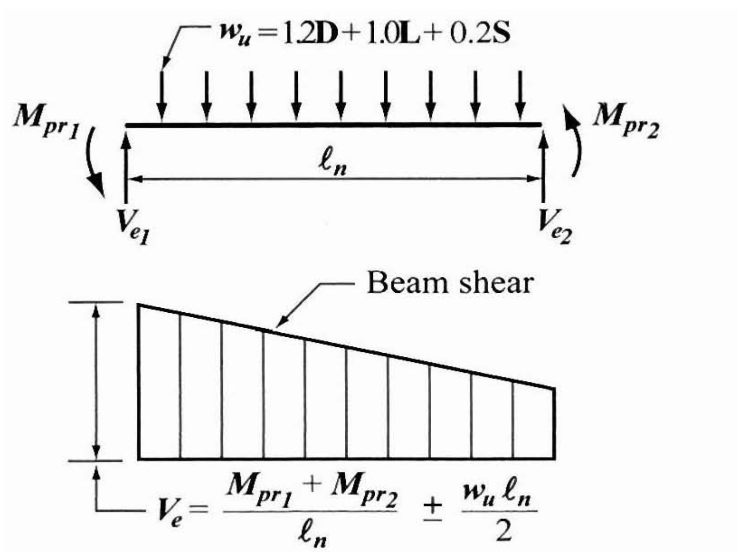
Seismic 3 - Probable moment resistance for beams
f_y = 60,000;   1,25 f_y = 75,000 psi

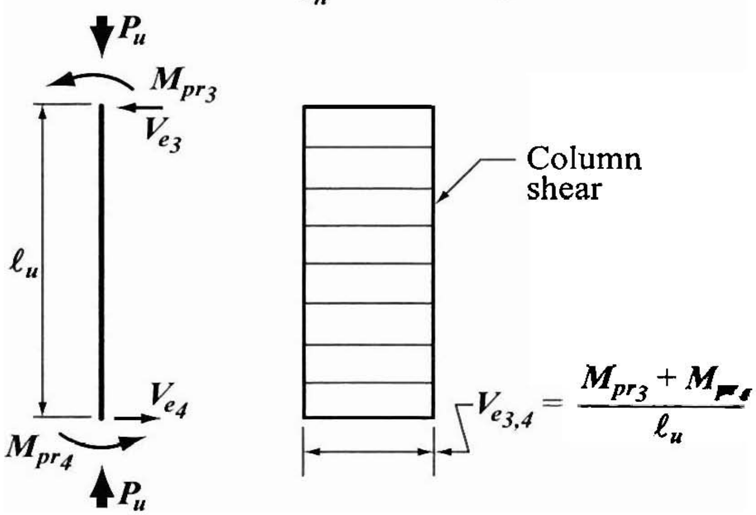
$M_{pr} = K_{pr}\, bd^2 / 1200$ ft-kips   **$\rho = A_s / bd$**

f'_c (psi):	3000	4000	5000	6000	7000	8000	9000	10000
ρ	K_{pr} (psi)							
0.004	282	287	289	291	292	293	294	295
0.005	347	354	358	361	363	365	366	367
0.006	410	420	426	430	433	435	437	438
0.007	471	484	493	498	502	505	507	509
0.008	529	547	558	565	570	574	576	579
0.009	586	608	621	630	637	642	645	648
0.010	640	667	684	695	703	709	713	717
0.011	692	725	745	758	768	775	781	785
0.012	741	781	805	821	832	840	847	852
0.013	789	835	863	882	895	905	913	919
0.014	834	888	920	942	957	969	978	985
0.015	877	939	976	1001	1019	1032	1042	1051
0.016	918	988	1031	1059	1079	1094	1106	1115
0.017	956	1036	1084	1116	1138	1156	1169	1179
0.018	993	1082	1136	1171	1197	1216	1231	1243
0.019	1027	1126	1186	1226	1254	1276	1292	1306
0.020	1059	1169	1235	1280	1311	1335	1353	1368
0.021	1089	1210	1283	1332	1367	1393	1413	1429
0.022	1116	1250	1330	1383	1421	1450	1472	1490
0.023	1142	1288	1375	1433	1475	1506	1531	1550
0.024	1165	1324	1419	1482	1528	1562	1588	1609
0.025	1186	1358	1462	1530	1580	1617	1645	1668

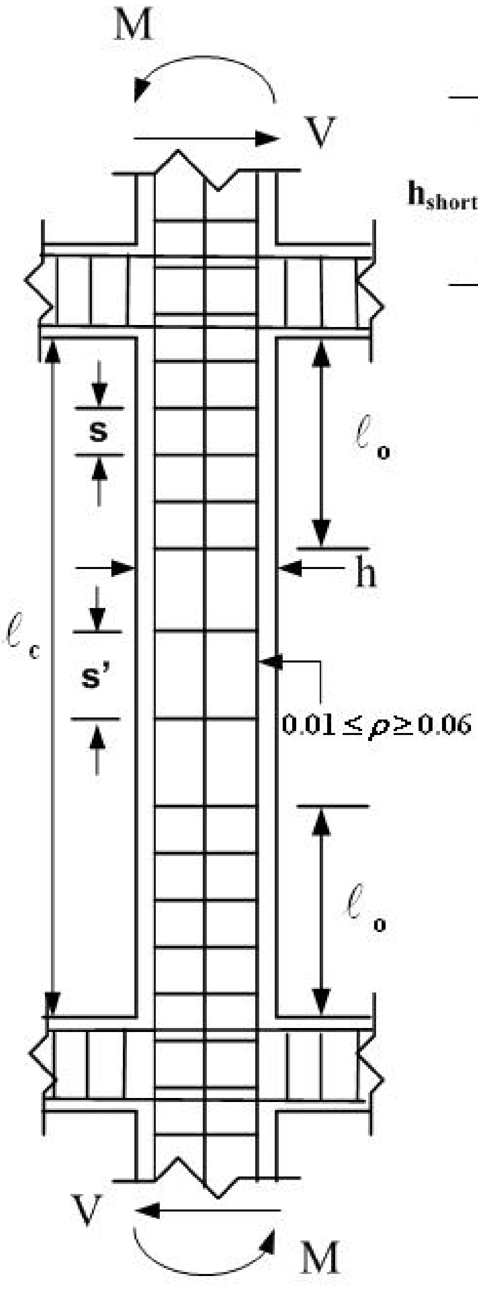
Seismic 4 – Seismic design shear in beams and columns of special frames

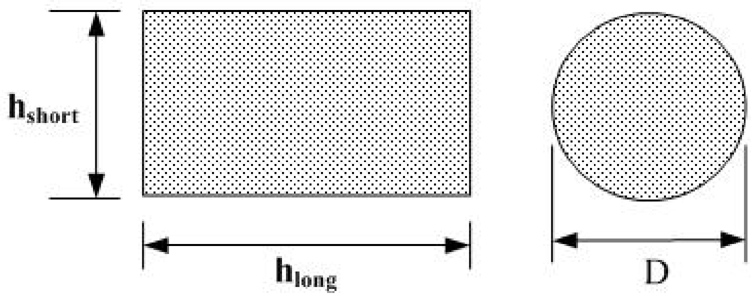






Seismic 5 – Design requirements for columns of special moment frames





$h_{short} \ge 12\,in$ $D \ge 12\,in$

If beam reinforcement is continuous over the support:

$\frac{h_{short}}{h_{long}} \ge 0.4$

$h \ge 20d_b$ (NWC)

$h \ge 26d_b$ (LWC)

$s \le h/4$ $s' \ge 6\,in$

$s \le 6\,(b_d)_{long.}$ $s' \ge 6(b_d)_{long.}$

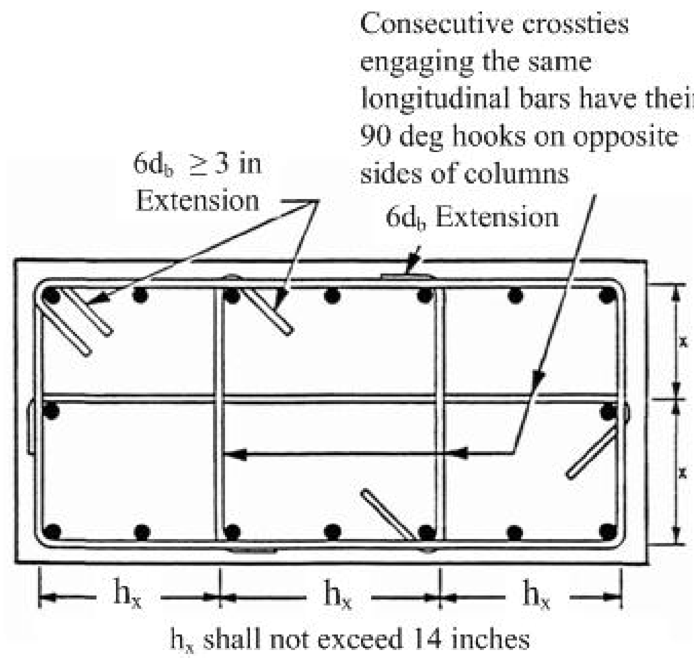
$s \le s_0$ but need not be less than 4 in

Where; $s_0 = 4 + \left(\frac{14 - h_x}{3}\right) \le 6\,in$

$\ell_o \ge h$

$\ell_o \ge \ell_c / 6$

$\ell_o \ge 18\,in$



Transverse Reinforcement for Columns

Seismic 6 – Volumetric ratio of spiral reinforcement (ρ_s) for concrete confinement

$$\rho_s \geq 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}} \quad \text{but} \quad \rho_s \geq 0.12\frac{f'_c}{f_{yh}}$$

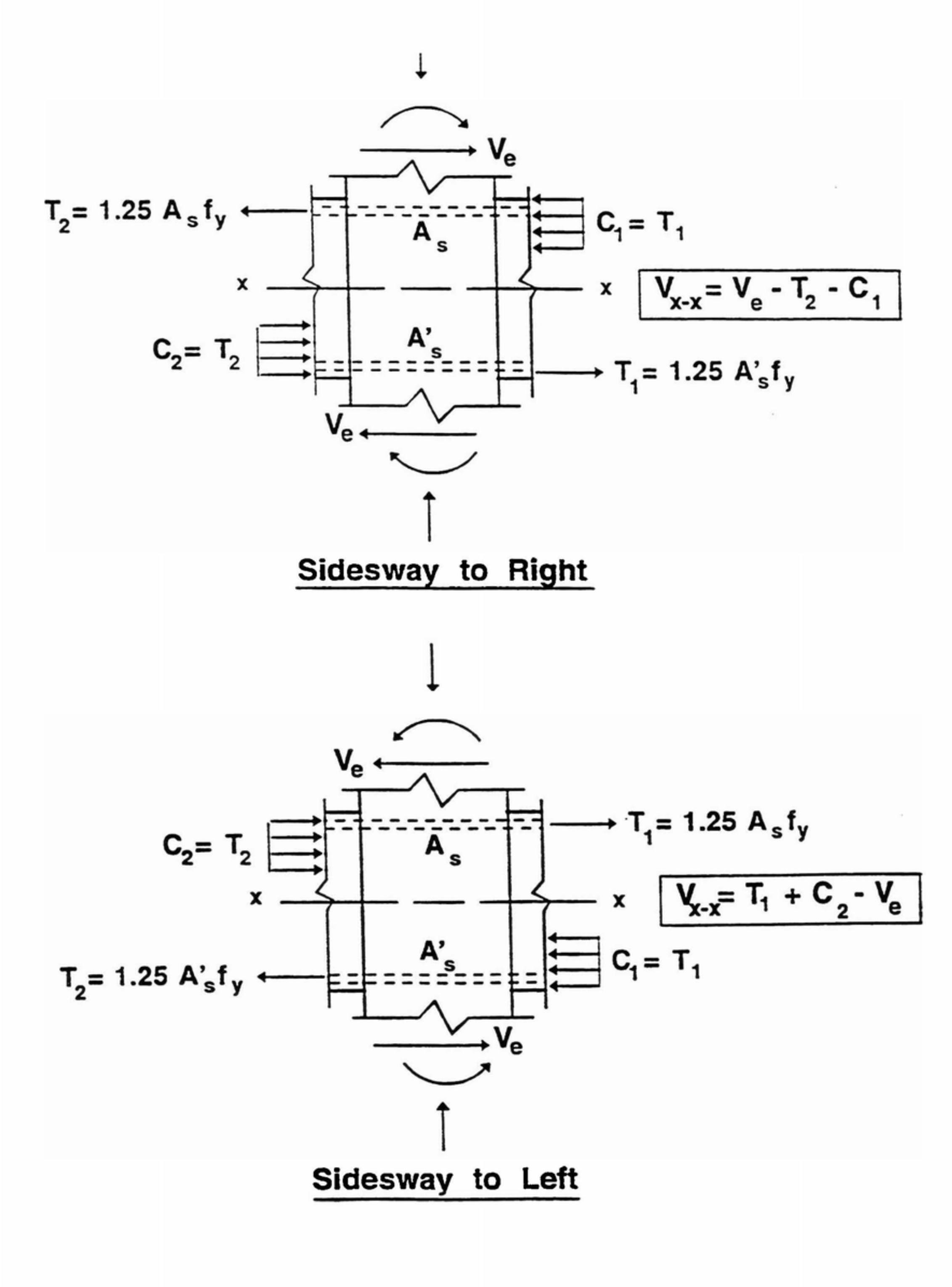
f'c (psi):	3000	4000	5000	6000	7000	8000	9000	10000
A_g/A_c				ρ_s				
1.1	0.006	0.008	0.010	0.012	0.014	0.016	0.018	0.020
1.2	0.006	0.008	0.010	0.012	0.014	0.016	0.018	0.020
1.3	0.007	0.009	0.011	0.014	0.016	0.018	0.020	0.023
1.4	0.009	0.012	0.015	0.018	0.021	0.024	0.027	0.030
1.5	0.011	0.015	0.019	0.023	0.026	0.030	0.034	0.038
1.6	0.014	0.018	0.023	0.027	0.032	0.036	0.041	0.045
1.7	0.016	0.021	0.026	0.032	0.037	0.042	0.047	0.053
1.8	0.018	0.024	0.030	0.036	0.042	0.048	0.054	0.060
1.9	0.020	0.027	0.034	0.041	0.047	0.054	0.061	0.068
2.0	0.023	0.030	0.038	0.045	0.053	0.060	0.068	0.075
2.1	0.025	0.033	0.041	0.050	0.058	0.066	0.074	0.083
2.2	0.027	0.036	0.045	0.054	0.063	0.072	0.081	0.090
2.3	0.029	0.039	0.049	0.059	0.068	0.078	0.088	0.098
2.4	0.032	0.042	0.053	0.063	0.074	0.084	0.095	0.105
2.5	0.034	0.045	0.056	0.068	0.079	0.090	0.101	0.113

Seismic 7 – Area ratio of rectilinear confinement reinforcement (ρ_c) for concrete

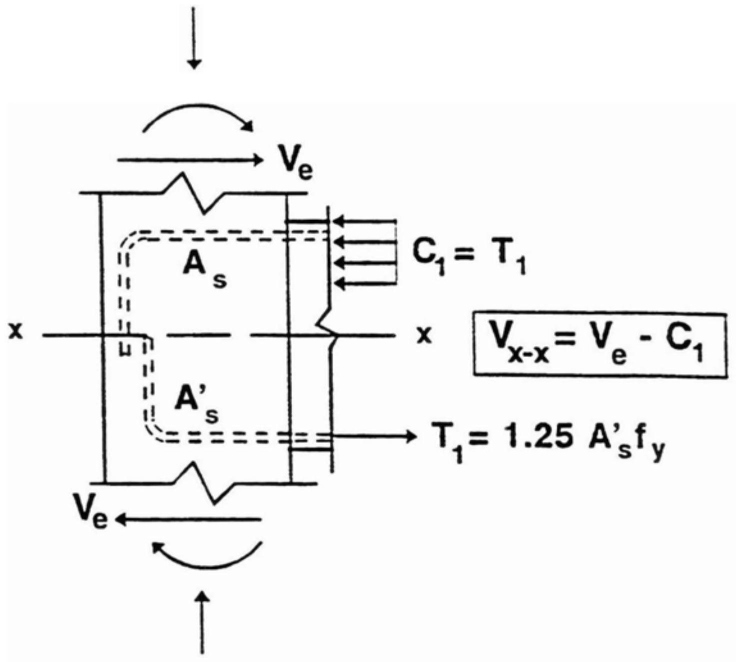
$$\rho_c = \frac{A_{sh}}{sb_c} \geq 0.3\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \qquad \textbf{but} \qquad \rho_c \geq 0.09\,\frac{f_c'}{f_{yh}}$$

f'_c (psi):	3000	4000	5000	6000	7000	8000	9000	10000
A_g/A_c				ρ_s				
1.1	0.005	0.006	0.008	0.009	0.011	0.012	0.014	0.015
1.2	0.005	0.006	0.008	0.009	0.011	0.012	0.014	0.015
1.3	0.005	0.006	0.008	0.009	0.011	0.012	0.014	0.015
1.4	0.006	0.008	0.010	0.012	0.014	0.016	0.018	0.020
1.5	0.008	0.010	0.013	0.015	0.018	0.020	0.023	0.025
1.6	0.009	0.012	0.015	0.018	0.021	0.024	0.027	0.030
1.7	0.011	0.014	0.018	0.021	0.025	0.028	0.032	0.035
1.8	0.012	0.016	0.020	0.024	0.028	0.032	0.036	0.040
1.9	0.014	0.018	0.023	0.027	0.032	0.036	0.041	0.045
2.0	0.015	0.020	0.025	0.030	0.035	0.040	0.045	0.050
2.1	0.017	0.022	0.028	0.033	0.039	0.044	0.050	0.055
2.2	0.018	0.024	0.030	0.036	0.042	0.048	0.054	0.060
2.3	0.020	0.026	0.033	0.039	0.046	0.052	0.059	0.065
2.4	0.021	0.028	0.035	0.042	0.049	0.056	0.063	0.070
2.5	0.023	0.030	0.038	0.045	0.053	0.060	0.068	0.075

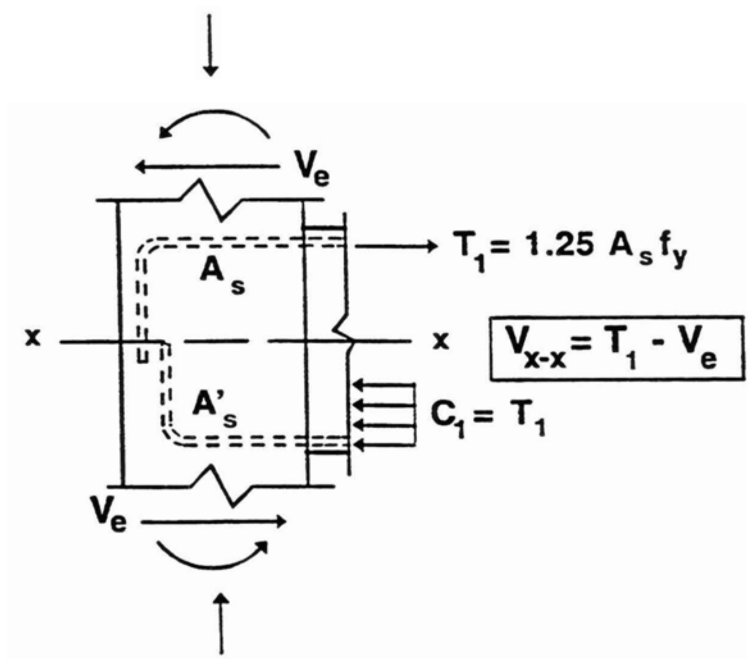
Seismic 8 – Joint shear, $V_{x\text{-}x}$ in an interior beam-column joint



Seismic 9 – Joint shear, $V_{x\text{-}x}$ in an exterior beam-column joint

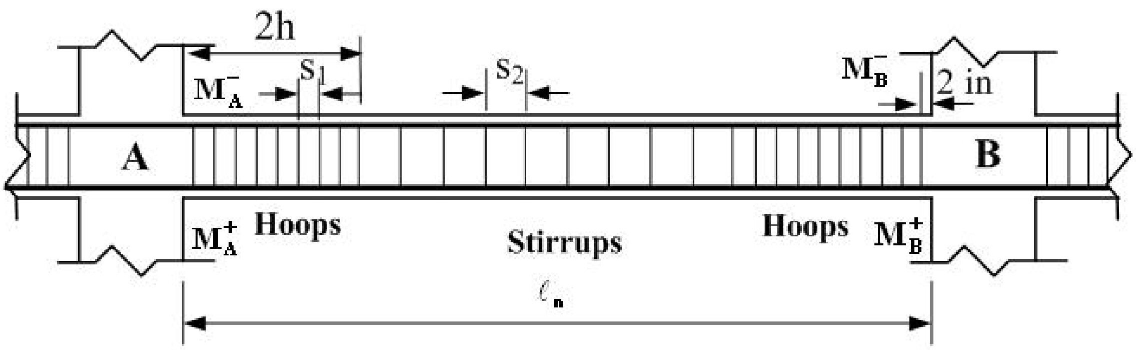


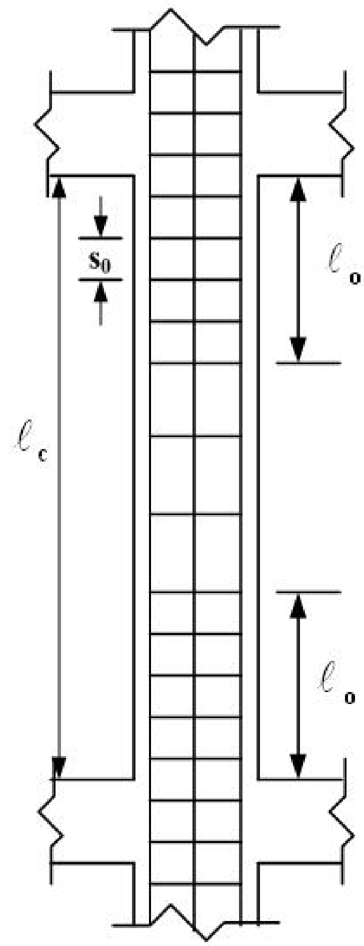
Sidesway to Right



Sidesway to Left

Seismic 10 – Design requirements for beams and columns of intermediate moment frames





The spacing of transverse reinforcement in columns, outside ℓ_o will be designed following the requirements of Sec. 7.10 and 11.5.5.1 of ACI 318-05, as for ordinary building columns.

Minimum flexural capacities;

- $M_A^+ \geq \frac{1}{3} M_A^-$
- $M_B^+ \geq \frac{1}{3} M_B^-$
- **If the largest end moment is M_{max}; at any section along the beam:**

$$M^+ \geq \frac{1}{5} M_{max}$$

$$M^- \geq \frac{1}{5} M_{max}$$

Hoop/stirrup spacing;

$s_1 \leq d/4$ $\quad s_1 \leq 8(d_b)_{long.bar}$

$s_1 \leq 12\,in$ $\quad s_1 \leq 24(d_b)_{hoop}$

$s_2 \leq d/2$

$s_0 \leq 8(d_b)_{long\,bar}$ $\quad s_0 \leq 12\,in$

$s_0 \leq$ **smallest dimension of column section**

Potential plastic hinge length;

$\ell_o \geq \ell_c / 6$ $\quad \ell_o \geq 18\,in$

$\ell_0 \geq$ **maximum dimension of column section**

Seismic 11 – Seismic design shear in beams and columns of intermediate frames