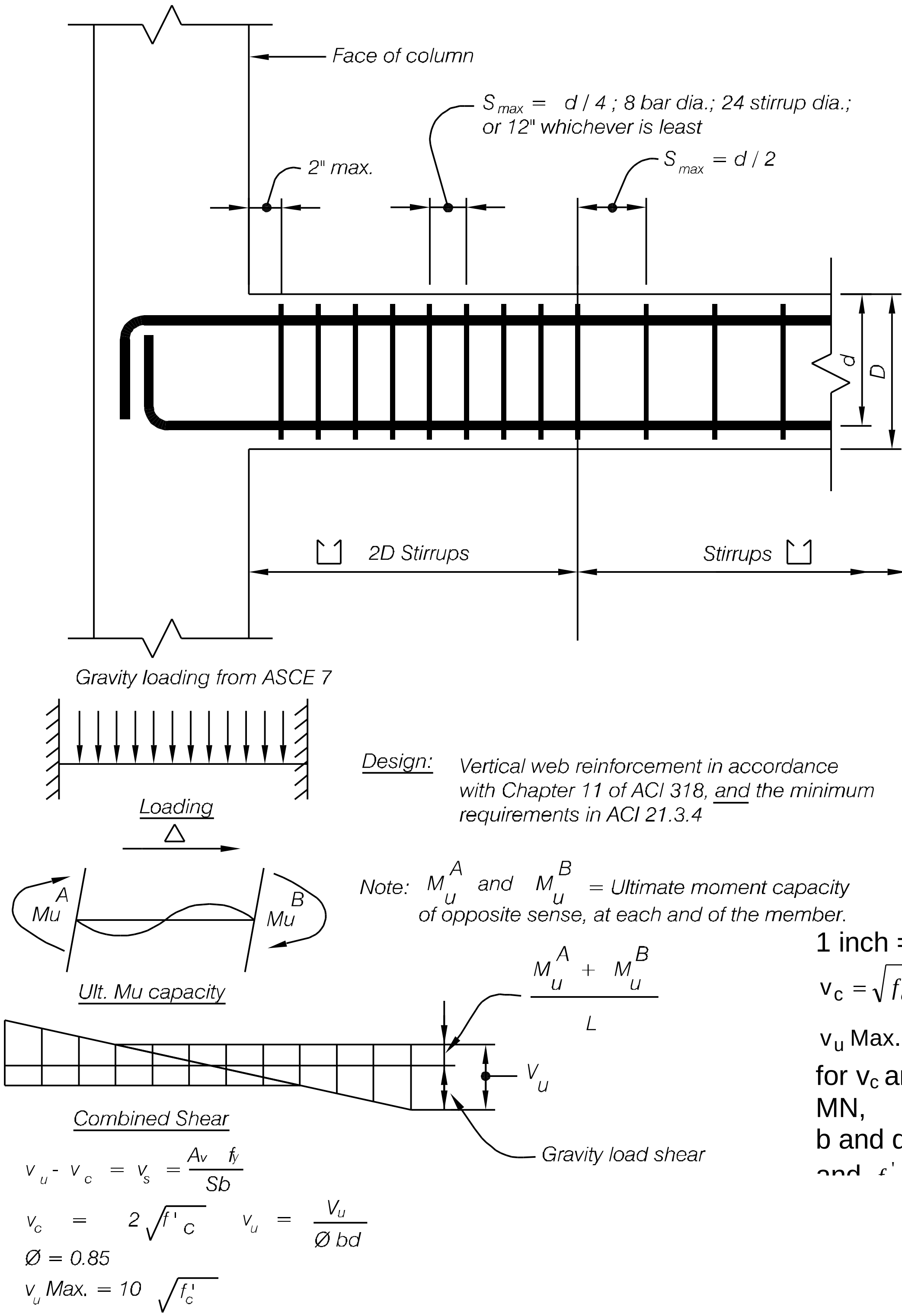


1 inch = 25mm

$v_c = \sqrt{f'_c}/6$ and

v_u Max. $= 0.83\sqrt{f'_c}$

for v_c and v_u in MN,

b and d in mm,

and f'_c in MPa

Figure 7-31 Intermediate Moment Frame Girder Web reinforcement

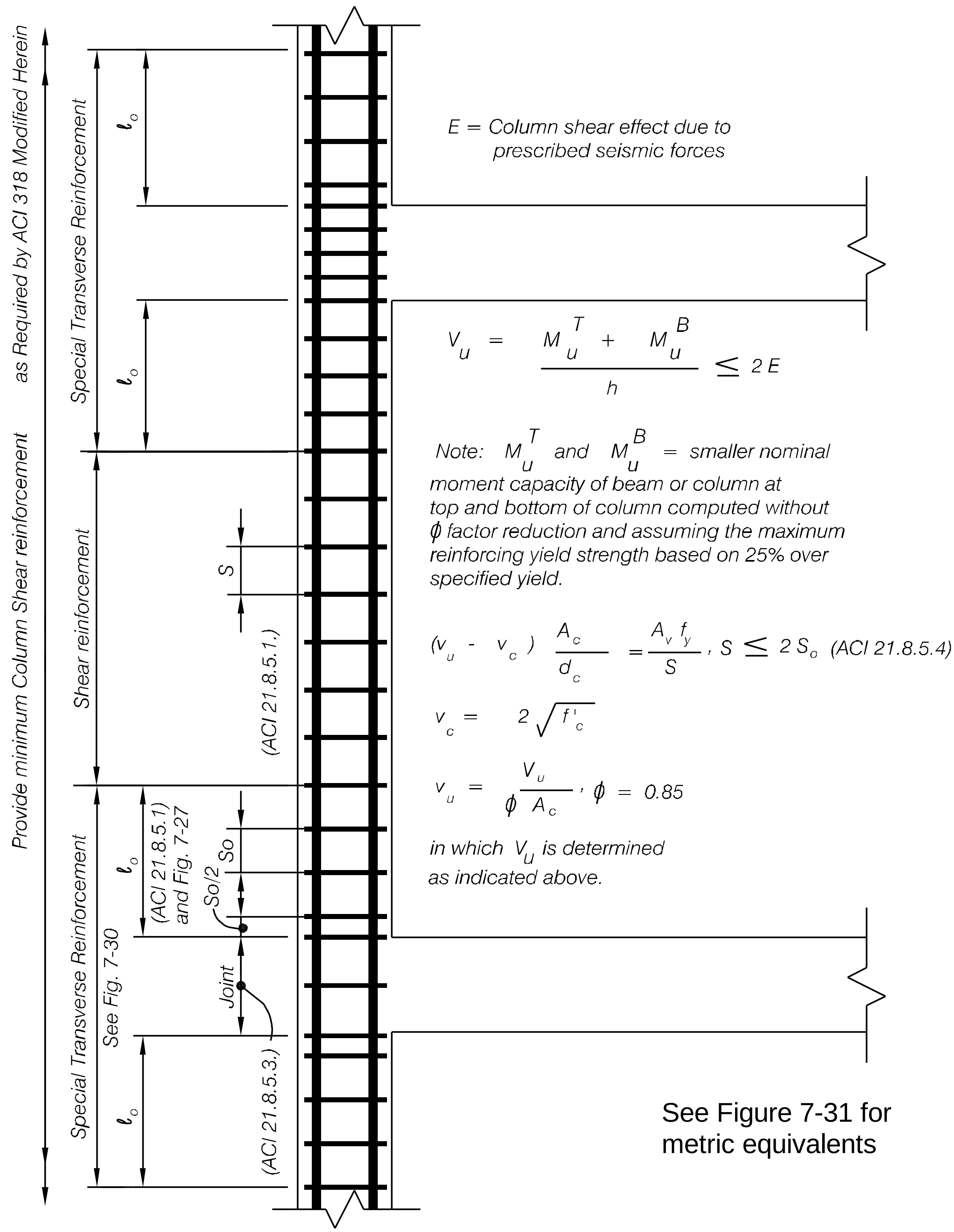


Figure 7-32 Intermediate Moment Frame transverse reinforcement

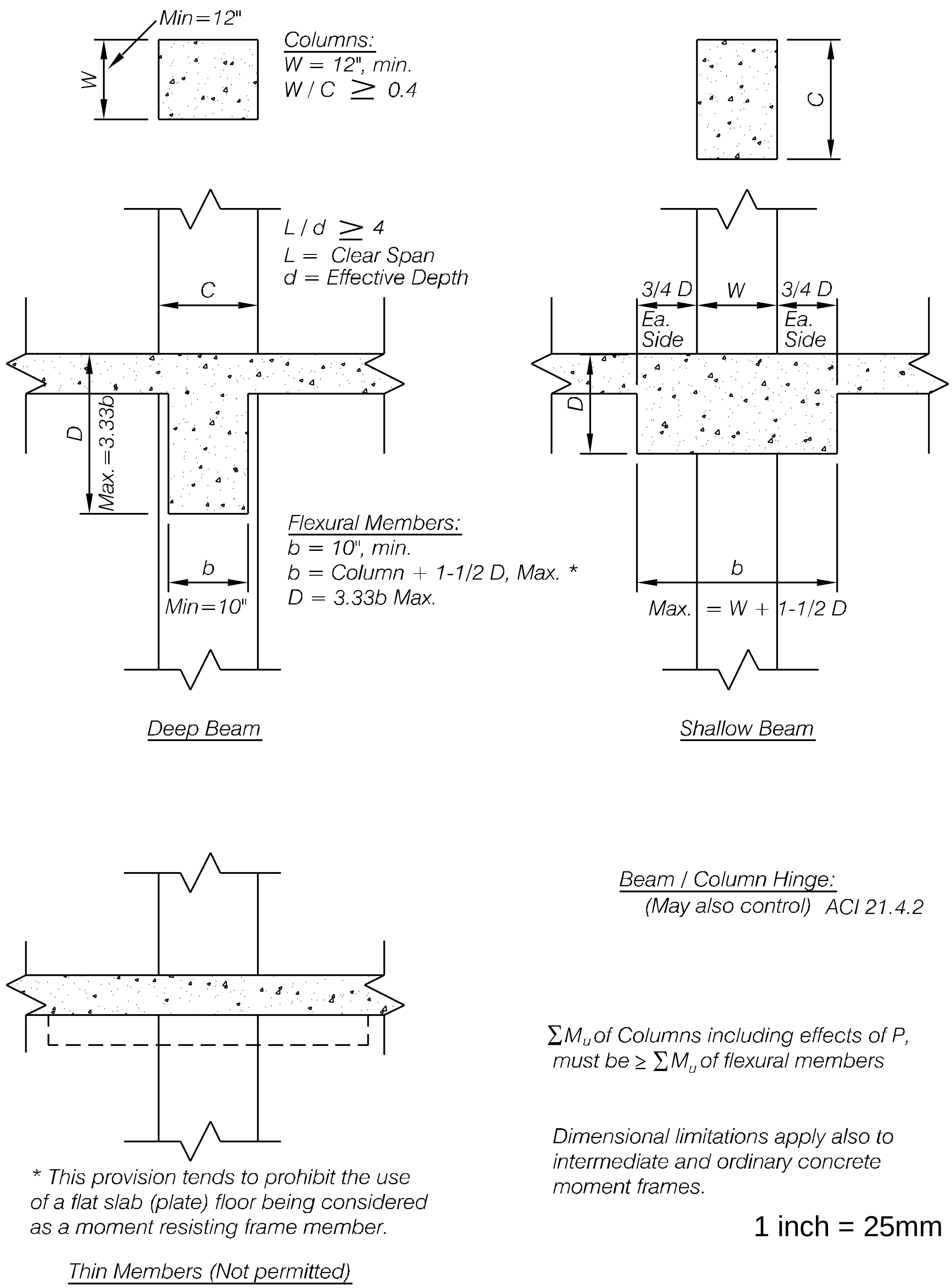
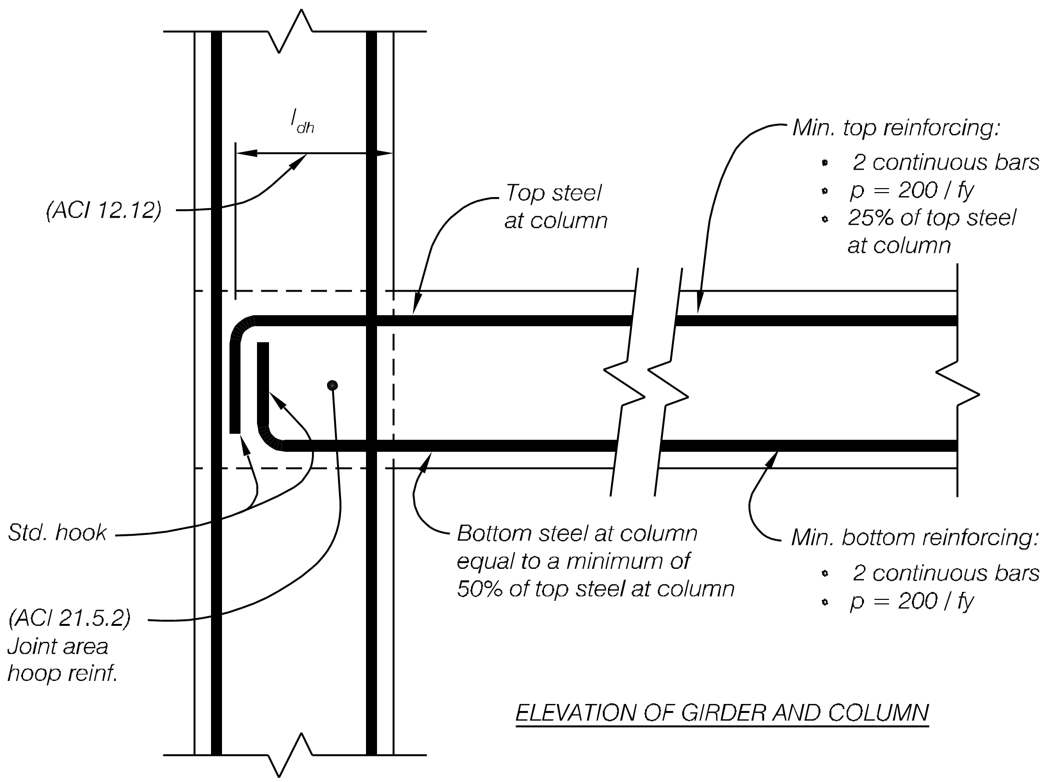


Figure 7-33 Special Concrete Moment Frame - limitations on dimensions.



FLEXURAL MEMBER:

$f'c$ = 3,000 p.s.i. min. at 28 days
fy = 40 ksi or 60 ksi
Reinforcement ratio $p = A_s / bd$ or $p' = A'_s / bd$: $p = 0.025$ max.

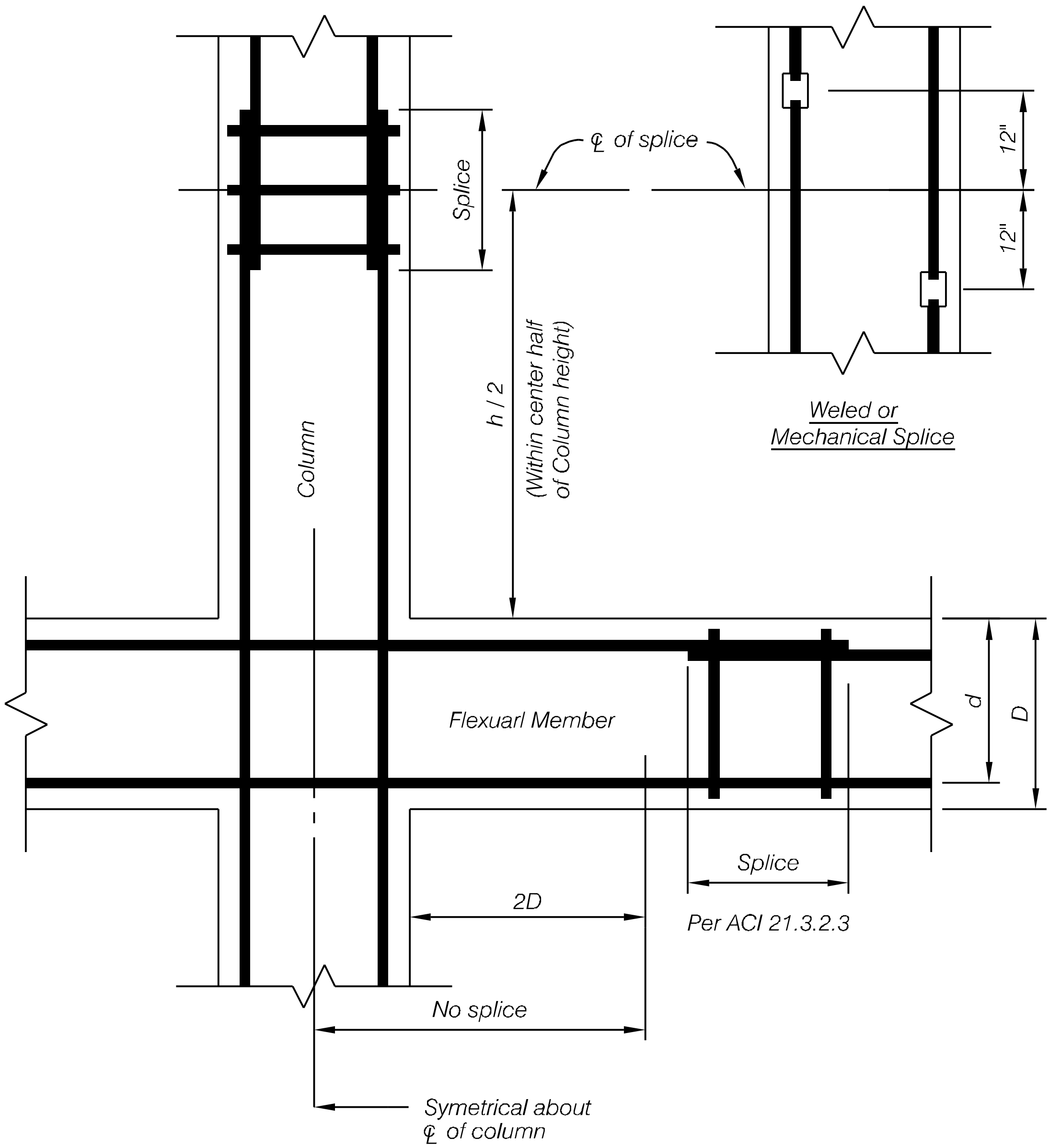
COLUMN:

f'_c = 3,000 p.s.i. min. at 28 days
f_y = 40 ksi or 60 ksi
Reinforcement ratio, p (for tied columns)

$$0.01 \leq p \leq 0.06$$

1 ksi = 6.89

Figure 7-34 Special Concrete Moment Frame longitudinal reinforcement



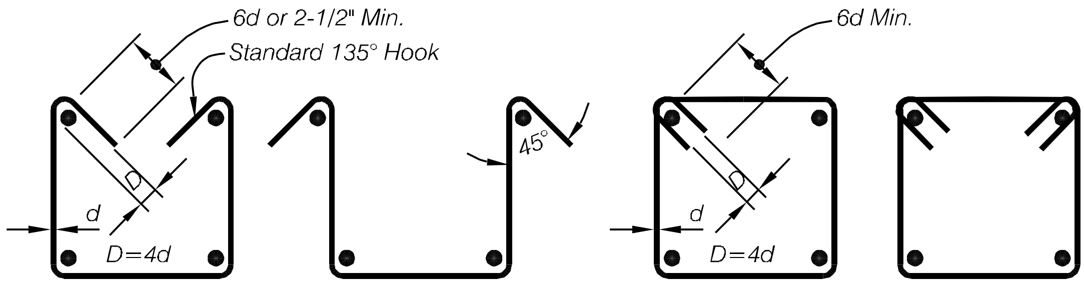
Column:

l_d Is the development length. See ACI 318-95 Sect. 12.2

At any level, not more than alternate bars will be welded or mechanical spliced. Min. distance between two adjacent bar splices = 24".

1 inch = 25mm

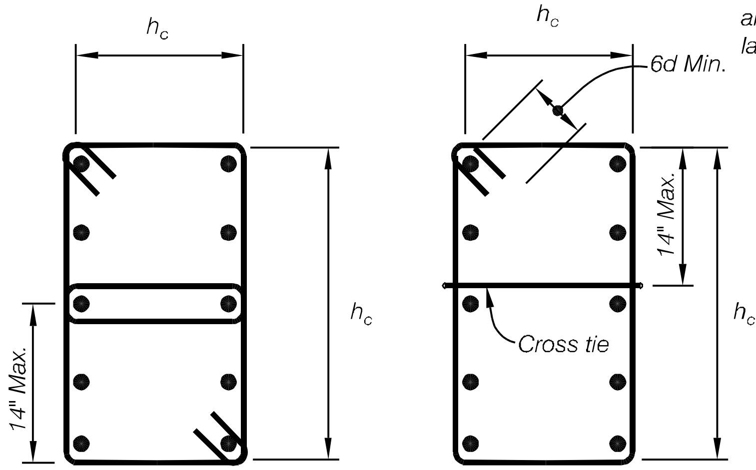
Figure 7-35 Special Moment Frame splices in reinforcement

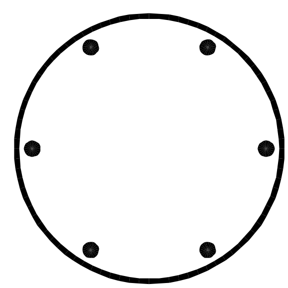


STIRRUPS   STIRRUPS - TIES

USED IN BEAMS

Min. hoop and tie size is #3 for longitudinal bars #10 and smaller, and #4 for longitudinal bars #11 or larger.





HOOP OR COLUMN TIE   SPIRAL

USED IN COLUMNS

Spiral Ratio:

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

Whichever is greater.

Functions	Stirrups	Stirrup-Ties	Column Ties	Hoops	Spirals
Shear Reinforcement and "Caging"	•	•	•	•	•
Restrain Longitudinal Steel from Buckling		•	•	•	•
Confine Concrete				•	•

Hoop Requirements -Total Tie Area:

$$A_{sh} = 0.3\left(\frac{sh_c}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$

Formula 21 - 3

Formula 21 - 4

$$A_{sh} = 0.09\ sh_c \frac{f'_c}{f_{yh}}$$, whichever is greater.

Provide hoops or spirals in columns where special transverse reinforcement is required. (ACI 21.4.4)

See Figure 7-30 for metric

Figure 7-36 Special Moment Frame - transverse reinforcement

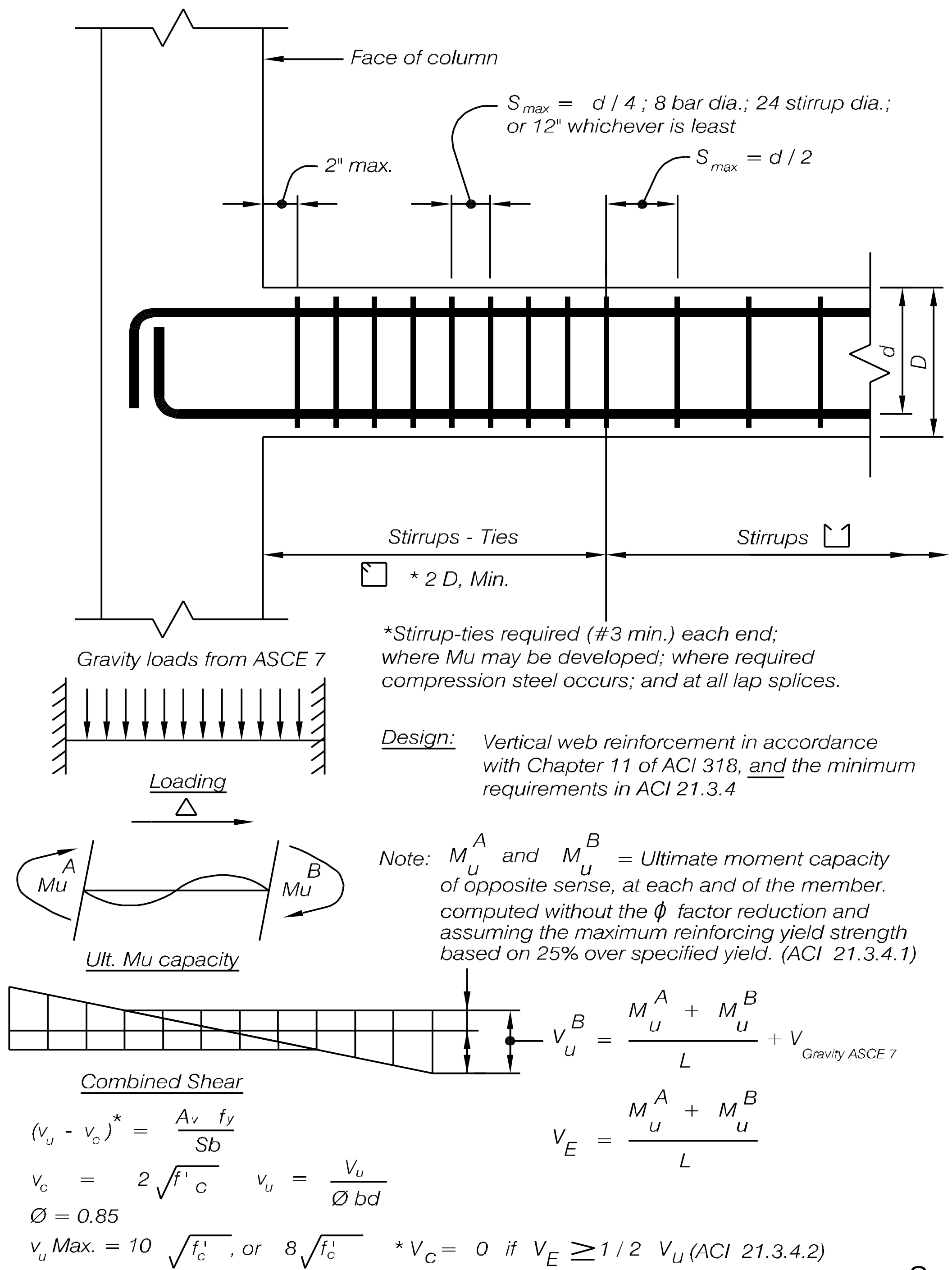


Figure 7-37 Special Moment Frame Girder Web reinforcement

See Figure 7-31 for

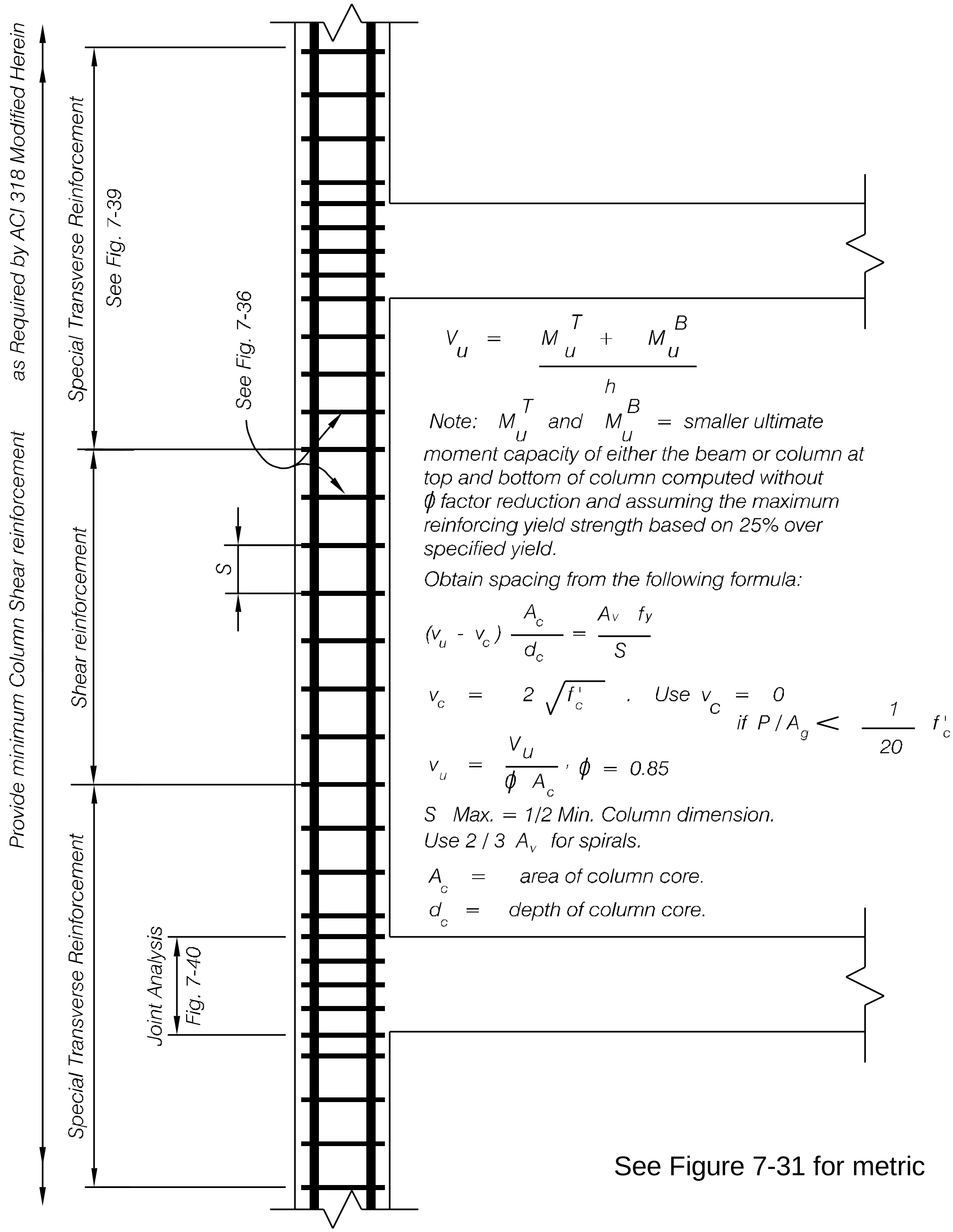
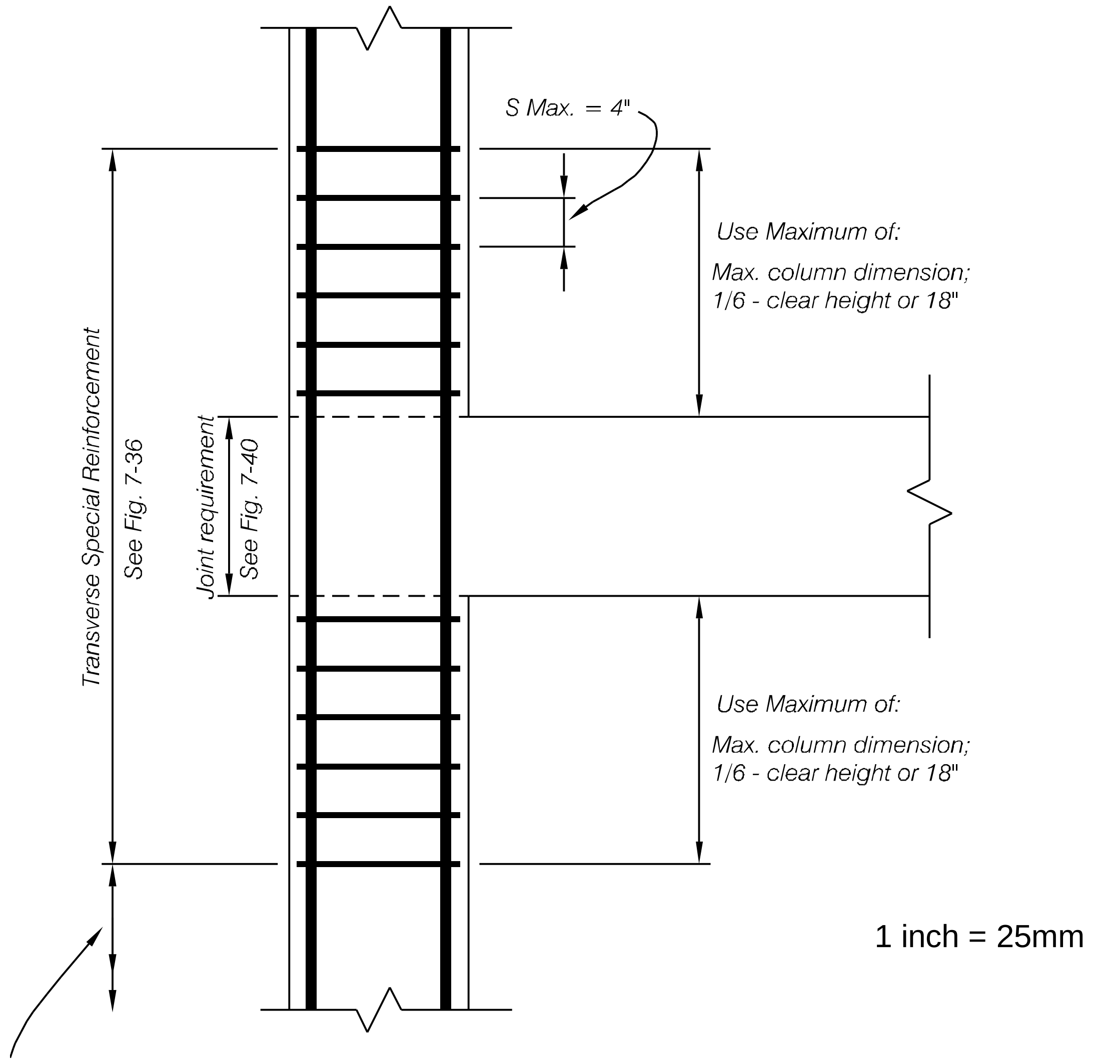
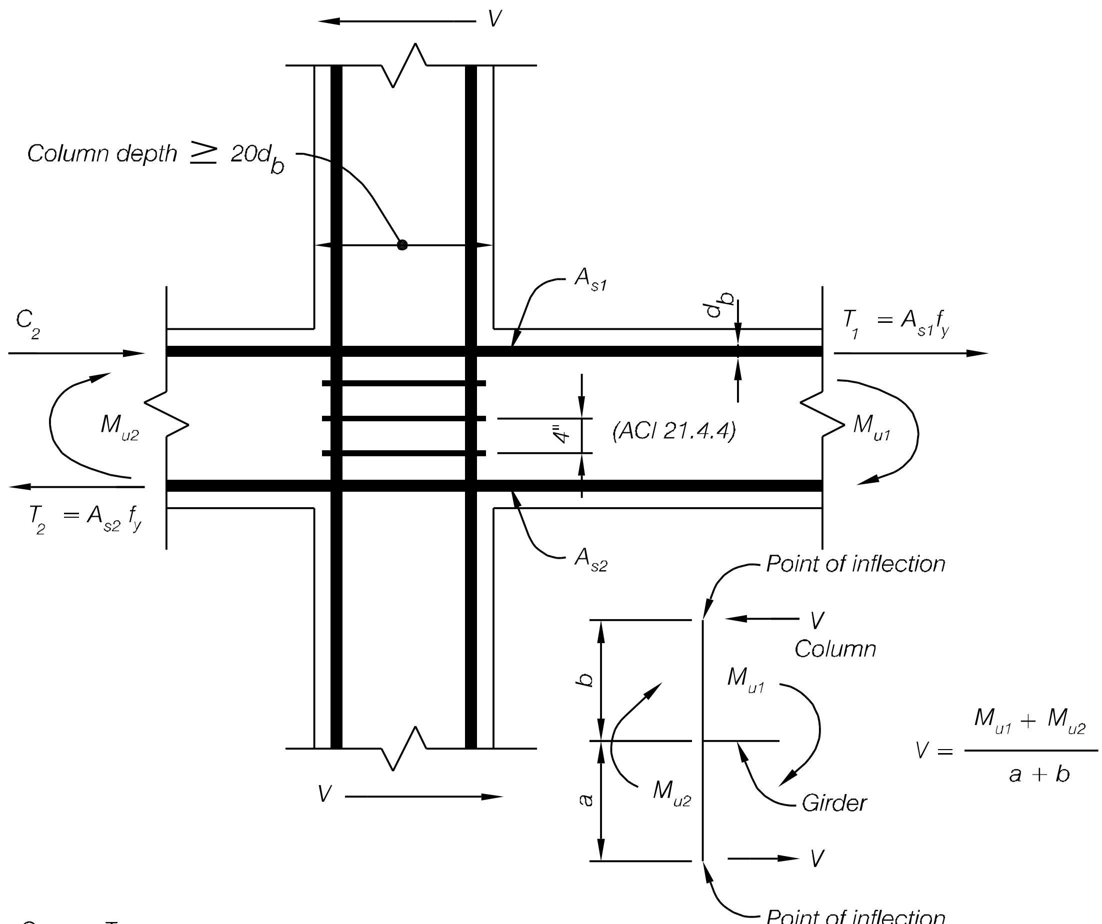


Figure 7-38 Special Moment Frame transverse reinforcement



At any section where the ultimate capacity of the column (P_u) is less than sum of the beam shears (ΣV_u) computed by $Vu = \dfrac{Mu^A + Mu^A}{L} + V_{Grav\ ASCE\ 7}$ for all the beams. Above the level under consideration, confining reinforcement shall be provided (See Fig. 7-36). This confining reinforcement is also required when point of contra-flexure not in middle half of column (ACI 21.4.4.5) for columns supporting discontinued stiff members, such as walls.

Figure 7-39 Special Moment Frame - special transverse reinforcement.



$C_2 = T_2$

$V_u = T_1 + C_2 - V$

$v_u = \dfrac{V_u}{A_J}$ Where A_J is defined in ACI 21.5.3.

Modified to concur with FEMA 302 and ACI 318-95

$S = $ 4" max. for non-confined joints (ACI 21.5.2.1)

only 1/2 the special transverse reinforcement is required for confined joints where girders frame into all four sides. (ACI 21.5.2.2)

Note: The intersection of the orthogonal beam steel and the column steel, along with the required joint confinement hoop steel frequently results in congestion of bars. A careful study of the bar layouts should be made during design and represented on the construction documents.

Figure 7-40 Special Frame - girder column joint analysis.

Conditions			m factors[3]					
			Component Type					
			Primary				Secondary	
			Performance Level					
			IO	SE	LS	CP	LS	CP
i. Beams controlled by flexure[1]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤ 0.0	C	≤ 3	2	4	6	7	6	10
≤ 0.0	C	≥ 6	2	2.5	3	4	3	5
≥ 0.5	C	≤ 3	2	2.5	3	4	3	5
≥ 0.5	C	≥ 6	2	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	2.5	3	4	3	5
≤ 0.0	NC	≥ 6	1	1.5	2	3	2	4
≥ 0.5	NC	≤ 3	2	2.5	3	3	3	4
≥ 0.5	NC	≥ 6	1	1.5	2	2	2	3
ii. Beams controlled by shear[1]								
Stirrup spacing ≤ $d/2$			–	–	–	–	3	4
Stirrup spacing > $d/2$			–	–	–	–	2	3
iii. Beams controlled by inadequate development or splicing along the span[1]								
Stirrup spacing ≤ $d/2$			–	–	–	–	3	4
Stirrup spacing > $d/2$			–	–	–	–	2	3
iv. Beams controlled by inadequate embedment into beam-column joint[1]								
			2	2	2	3	3	4

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

Table 7-14: Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

Conditions			m factors[4]					
			Component Type					
			Primary				Secondary	
			Performance Level					
			IO	SE	LS	CP	LS	CP
i. Columns controlled by flexure[1]								
$\frac{P}{A_g f_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f_c}}$						
≤ 0.1	C	≤ 3	2	2.5	3	4	3	4
≤ 0.1	C	≥ 6	2	2.5	3	3	3	3
≥ 0.4	C	≤ 3	1	1.5	2	2	2	2
≥ 0.4	C	≥ 6	1	1	1	2	1	2
≤ 0.1	NC	≤ 3	2	2	2	3	2	3
≤ 0.1	NC	≥ 6	2	2	2	2	2	2
≥ 0.4	NC	≤ 3	1	1	1	2	1	2
≥ 0.4	NC	≥ 6	1	1	1	1	1	1
ii. Columns controlled by shear[1,3]								
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f_c} \leq 0.1$			–	–	–	–	2	3
Other cases			–	–	–	–	1	1
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]								
Hoop spacing ≤ $d/2$			–	–	–	–	3	4
Hoop spacing > $d/2$			–	–	–	–	2	3
iv. Columns with axial loads exceeding $0.70P_o$[1,3]								
Conforming reinforcement over the entire length			1	1	1	2	2	2
All other cases			–	–	–	–	1	1

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

Table 7-15: Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Columns

Conditions			m factors[4]					
			Component Type					
			Primary[5]				Secondary	
			Performance Level					
			IO	SE	LS	CP	LS	CP
i. Interior joints								
$\frac{P}{A_g f_c}$ 2	Trans. Reinf.[1]	$\frac{V}{V_n}$ 3						
≤ 0.1	C	≤ 1.2	–	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	–	2	3
≥ 0.4	NC	≥ 1.5	–	–	–	–	2	3
ii. Other joints								
$\frac{P}{A_g f_c}$ 2	Trans. Reinf.[1]	$\frac{V}{V_n}$ 3						
≤ 0.1	C	≤ 1.2	–	–	–	–	3	4
≤ 0.1	C	≥ 1.5	–	–	–	–	2	3
≥ 0.4	C	≤ 1.2	–	–	–	–	3	4
≥ 0.4	C	≥ 1.5	–	–	–	–	2	3
≤ 0.1	NC	≤ 1.2	–	–	–	–	2	3
≤ 0.1	NC	≥ 1.5	–	–	–	–	2	3
≥ 0.4	NC	≤ 1.2	–	–	–	–	1	1
≥ 0.4	NC	≥ 1.5	–	–	–	–	1	1

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures as described in Chapter 5.
3. This is the ratio of the design shear force to the shear strength for the joint.
4. Linear interpolation between values listed in the table is permitted.
5. All interior joints are force-controlled, and no *m* factors apply.

Table 7-16: Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beam-Column Joints

		m factors					
		Component Type					
		Primary				Secondary	
		Performance Level					
Conditions		**IO**	**SE**	**LS**	**CP**	**LS**	**CP**
i. Slabs controlled by flexure, and slab-column connections[1]							
$\frac{V_g}{V_o}$[2]	Continuity Reinforcement[3]						
≤ 0.2	Yes	2	2	2	3	3	4
≥ 0.4	Yes	1	1	1	1	2	3
≤ 0.2	No	2	2	2	3	2	3
≥ 0.4	No	1	1	1	1	1	1
ii. Slabs controlled by inadequate development or splicing along the span[1]							
		–	–	–	–	3	4
iii. Slabs controlled by inadequate embedment into slab-column joint[1]							
		2	2	2	3	3	4

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318; V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."

Table 7-17: Numerical Acceptance Criteria for Linear Procedures—Two-Way Slabs and Slab-Column Connections

			Modeling Parameters[3]			Acceptance Criteria[3]					
						Plastic Rotation Angle, radians					
						Component Type					
			Plastic Rotation Angle, radians		Residual Strength Ratio	Primary				Secondary	
						Performance Level					
Conditions			a	b	c	IO	SE	LS	CP	LS	CP
i. Beams controlled by flexure[1]											
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$									
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.005	0.013	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.008	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.008	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.008	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0	0.003	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.008	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0	0.003	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]											
Stirrup spacing ≤ $d/2$			0.0	0.02	0.2	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > $d/2$			0.0	0.01	0.2	0.0	0.0	0.0	0.0	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]											
Stirrup spacing ≤ $d/2$			0.0	0.02	0.0	0.0	0.0	0.0	0.0	0.01	0.02
Stirrup spacing > $d/2$			0.0	0.01	0.0	0.0	0.0	0.0	0.0	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]											
			0.015	0.03	0.2	0.01	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic region, closed stirrups are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table is permitted.

Table 7-18: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[4]			Acceptance Criteria[4]					
						Plastic Rotation Angle, radians					
						Component Type					
			Plastic Rotation Angle, radians		Residual Strength Ratio	Primary				Secondary	
						Performance Level					
Conditions			**a**	**b**	**c**	**IO**	**SE**	**LS**	**CP**	**LS**	**CP**
i. Columns controlled by flexure[1]											
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$									
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.008	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.015	0.025	0.2	0.005	0.008	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.0	0.003	0.005	0.015	0.010	0.025
≥ 0.4	C	≥ 6	0.01	0.015	0.2	0.0	0.003	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.01	0.015	0.2	0.005	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	–	0.005	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.005	0.005	–	0.0	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0
ii. Columns controlled by shear[1,3]											
Hoop spacing ≤ $d/2$, or $\frac{P}{A_g f'_c} \leq 0.1$			0.0	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
Other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]											
Hoop spacing ≤ $d/2$			0.01	0.02	0.4	1	1	1	1	0.01	0.02
Hoop spacing > $d/2$			0.0	0.01	0.2	1	1	1	1	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1,3]											
Conforming reinforcement over the entire length			0.015	0.025	0.02	0.0	0.003	0.005	0.001	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A component is conforming if, within the flexural plastic hinge region, closed hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the stirrups (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, hoops must not be lap spliced in the cover concrete, and hoops must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4. Linear interpolation between values listed in the table is permitted.

Table 7-19: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

columns; Table 7-20 for beam/column joints, and Table 7-21 for slab/column frames.

(4) Expected strength of deformation-controlled components shall be the nominal flexural strengths determined in accordance with Chapter 9 of FEMA 302 with 1.25 f_y in lieu of f_y for the contribution of the reinforcement.

(5) The lower-bound strength for force-controlled component shall be taken as the applicable nominal strength, Q_m, times the appropriate capacity reduction factor, N, in accordance with ACI 318.

7-5. Steel Moment-Resisting Frames.

a. General.

(1) Function. Steel moment-resisting frames have functions and behavior similar to those of concrete moment frames, as discussed in Paragraph 7-4a(1).

(2) Frame types. FEMA 302 prescribes three types of steel moment frames: Ordinary Moment Frames (OMFs), Intermediate Moment Frames (IMFs), and Special Moment Frames (SMFs). Restrictions regarding the use of these frames are summarized in Table 7-1, which also provides the appropriate R value for each classification. Design of steel moment frames shall be in accordance with the provisions of AISC Seismic Provisions for Structural Steel Buildings.

b. Ordinary Moment Frames (OMFs).

(1) General. OMFs are expected to withstand limited inelastic deformations in their members when subjected to the forces resulting from the ground motions of the design earthquakes in combination with other loads. OMFs shall have a design strength, as provided in the AISC Seismic Provisions, to resist load combinations 4-1 through 4-2 of that document.

(2) Beam-to-column connections shall be made by welds or high-strength bolts. Connections shall be fully restrained or partially restrained (Type PR).

(a) Fully restrained connections. The required flexural strength, M_u, of each beam-to-column connection considered to be part of the lateral-force-resisting system shall be at least equal to $1.1R_yM_p$ of the beam or column, whichever is weaker. For pre-engineered steel structures, M_p is permitted to be taken as the critical buckling moment of the beam section. Welded joints in the connection shall be made with filler metal rated to have a Charpy V-notch toughness of 20 ft-lbs (27N-m) at a temperature of 0°F, as determined by ASTM A673. Except for connections of beams to end plates for use in pre-engineered metal structures, welded joints shall be complete penetration welds. At the bottom flange of welds, weld backing shall be removed, the root inspected and repaired, and a reinforcing fillet added. At the top flange welds, backing shall be removed and repaired or shall be attached by means of a continuous fillet weld on the edge away from the complete penetration weld. Alternately, only connections having a demonstrated inelastic rotation capability of at least 0.01 radian, based on tests as described in Paragraph 7-5c, shall be permitted to be

			Modeling Parameters[4]			Acceptance Criteria[4]					
						Plastic Rotation Angle, radians					
						Component Type					
						Primary				Secondary	
			Shear Angle, radians		Residual Strength Ratio	Performance Level					
Conditions			d	e	c	IO	SE	LS	CP	LS	CP
i. Interior joints											
$\frac{P}{A_g f_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]									
≤ 0.1	C	≤ 1.2	0.015	0.03	0.2	0.0	0.0	0.0	0.0	0.02	0.03
≤ 0.1	C	≥ 1.5	0.015	0.03	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≤ 1.2	0.015	0.025	0.2	0.0	0.0	0.0	0.0	0.015	0.025
≥ 0.4	C	≥ 1.5	0.015	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≤ 1.2	0.005	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≤ 0.1	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≤ 1.2	0.005	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≥ 0.4	NC	≥ 1.5	0.005	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
ii. Other joints											
$\frac{P}{A_g f_c}$ [2]	Trans. Reinf.[1]	$\frac{V}{V_n}$ [3]									
≤ 0.1	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≤ 0.1	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≥ 0.4	C	≤ 1.2	0.01	0.02	0.2	0.0	0.0	0.0	0.0	0.015	0.02
≥ 0.4	C	≥ 1.5	0.01	0.015	0.2	0.0	0.0	0.0	0.0	0.01	0.015
≤ 0.1	NC	≤ 1.2	0.005	0.01	0.2	0.0	0.0	0.0	0.0	0.005	0.01
≤ 0.1	NC	≥ 1.5	0.005	0.01	0.2	0.0	0.0	0.0	0.0	0.005	0.01
≥ 0.4	NC	≤ 1.2	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0
≥ 0.4	NC	≥ 1.5	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0

1. Under the heading "Transverse Reinforcement," "C" and "NC" are abbreviations for conforming and nonconforming details, respectively. A joint is conforming if closed hoops are spaced at ≤ $h_c/3$ within the joint. Otherwise, the component is considered nonconforming. Also, to qualify as conforming details under ii, hoops must not be lap spliced in the cover concrete, and must have hooks embedded in the core or other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
2. This is the ratio of the design axial force on the column above the joint to the product of the gross cross-sectional area of the joint and the concrete compressive strength. The design axial force is to be calculated using limit analysis procedures, as described in Chapter 5.
3. This is the ratio of the design shear force to the shear strength for the joint.
4. Linear interpolation between values listed in the table is permitted.

Table 7-20: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beam-Column Joints

		Modeling Parameters[4]			Acceptance Criteria[4]					
					Plastic Rotation Angle, radians					
					Component Type					
					Primary				Secondary	
		Plastic Rotation Angle, radians		Residual Strength Ratio	Performance Level					
Conditions		a	b	c	IO	SE	LS	CP	LS	CP
i. Slabs controlled by flexure, and slab-column connections[1]										
$\frac{V_g}{V_o}$ 2	Continuity Reinforcement[3]									
≤ 0.2	Yes	0.02	0.05	0.2	0.01	0.008	0.015	0.02	0.03	0.05
≥ 0.4	Yes	0.0	0.04	0.2	0.0	0.0	0.0	0.0	0.03	0.04
≤ 0.2	No	0.02	0.02	–	0.01	0.008	0.015	0.02	0.015	0.02
≥ 0.4	No	0.0	0.0	–	0.0	0.0	0.0	0.0	0.0	0.0
ii. Slabs controlled by inadequate development or splicing along the span[1]										
		0.0	0.02	0.0	0.0	0.0	0.0	0.0	0.01	0.02
iii. Slabs controlled by inadequate embedment into slab-column joint[1]										
		0.015	0.03	0.2	0.01	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
2. V_g = the gravity shear acting on the slab critical section as defined by ACI 318; V_o = the direct punching shear strength as defined by ACI 318.
3. Under the heading "Continuity Reinforcement," assume "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, assume "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, assume "No."
4. Interpolation between values shown in the table is permitted.

Table 7-21: Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-Way Slabs and Slab-Column Connections

used. Such connections shall be constructed using the same configurations, materials, processes, and quality control as was used in the tested connections. Member sizes used shall be similar to those tested. A typical pre-Northridge Earthquake fully restrained moment connection is shown in Figure 7-41. This connection is permitted by FEMA 302, provided it meets the requirements of this paragraph for ordinary moment frames, and the further requirements of Paragraph 7-5c for intermediate moment frames or 7-5d for special moment frames.

(b) Partially restrained connections shall be used, provided that the following requirements are met:

1. The strength requirements of Paragraph 7-5b(1) are met.

2. The nominal bending strength of the connection is at least equal to $0.5M_p$ of the connected beams.

3. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at an interstory drift calculated from the design story drift,), as determined in Section 5.3.8.1 of FEMA 302.

4. The additional drift and lower strength of the partially restrained connections is considered in the design, including the effects on overall frame stability.

Partially restrained connections are described in detail in Section A2 of AISC "Design Specifications for Structural Steel Buildings." A partially restrained connection using split wide-flange beam sections is shown in Figure 7-42.

(c) Required shear strength. The required shear strength V_u of a beam-to-column connection shall be determined as a minimum using the load combination $1.2D + 0.5L + 0.2S$, plus the shear resulting from M_u as defined in Paragraph 7-5b(2)(a) for fully restrained connections, on each end of the beam. For partially restrained connections, V_u shall be determined from the load combination above plus the shear resulting from the maximum end moments that the partially restrained connections are capable of resisting.

(d) Continuity of column-flange stiffener plates. Where fully restrained connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, continuity or column-flange stiffener plates shall be provided to transmit beam-flange forces to the column web or webs. Such plates shall have a minimum thickness of one-half that of the beam flange or beam-flange connection plate. The connections of the plates to the column flanges shall have a design strength equal to the design strength of the contact area of the plate with the column flange. The connection of the plate to the column web shall have a design shear strength equal to the lesser of the following:

1. The design strength of the connections of the plate to the column flanges, or

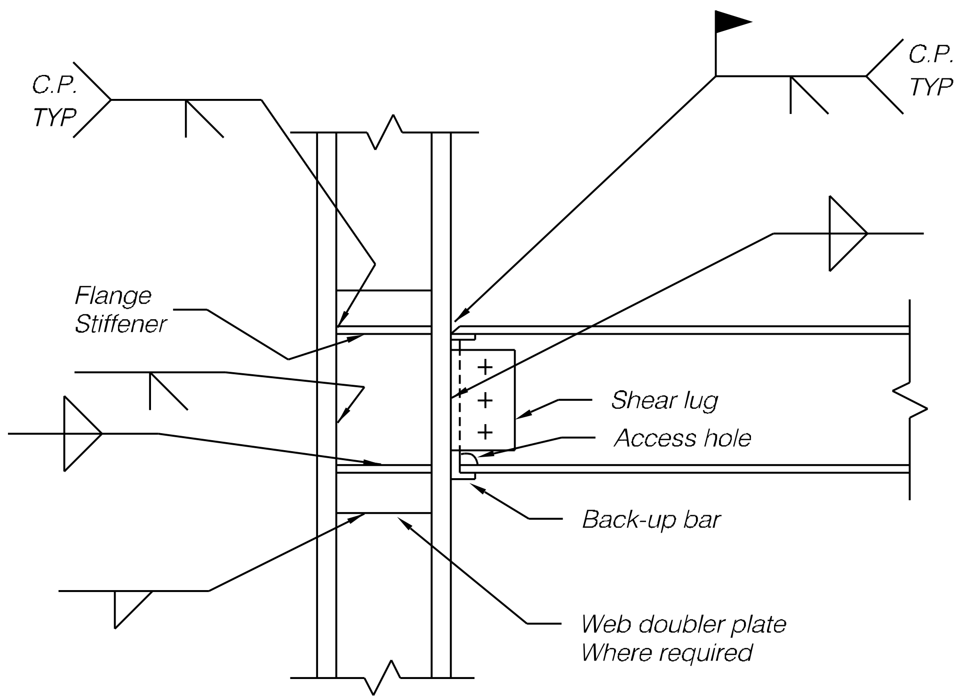


Figure 7-41 Typical pre-Northridge fully restrained moment connection

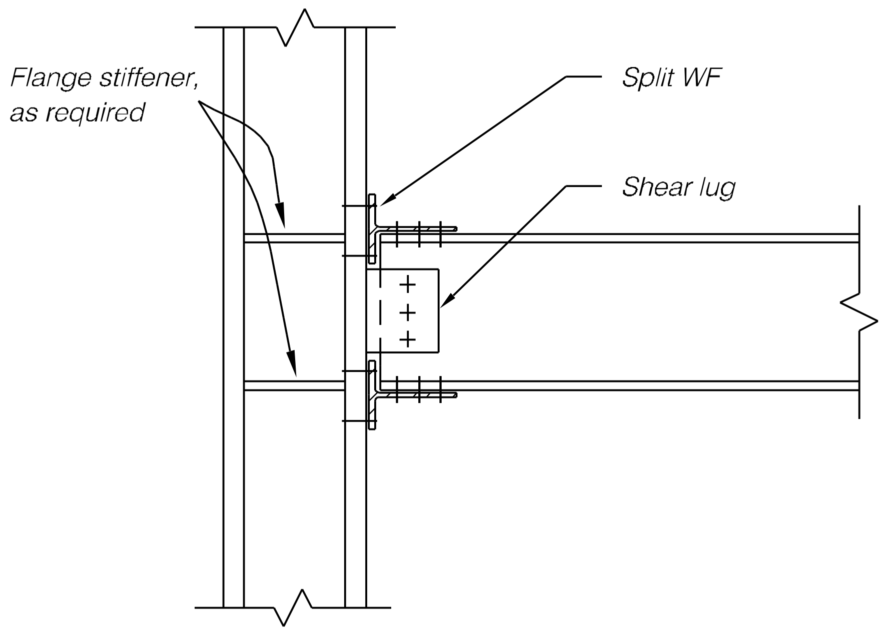


Figure 7-42 Typical partially restrained moment conection

2. The design shear strength of the contact area of the plate with the column web.

Continuity plates are not required if tested connections demonstrate that the intended inelastic rotation capacity can be achieved without their use. Partial penetration welds of the plates to the column flanges shall not be used.

c. Intermediate Moment Frames (IMFs). Intermediate moment frames are expected to withstand moderate inelastic deformations when subjected to the forces resulting from the motions of the design earthquake in combination with other loads. Intermediate moment frames shall conform to the AISC Seismic Provisions, Section 12, Requirements for Special Moment Frames, except as follows:

(1) Beam-to-column connections. Beam-to-column connection design shall be based on cyclic test results demonstrating inelastic rotation capacity of at least 0.020 radian. Inelastic rotation is defined as the total angle change between the column face at the connection and a line connecting the beam inflection point to the column face, less that part of the angle change occurring prior to yield of the beam. Qualifying test results shall consist of cyclic tests and shall be based on one of the following:

(a) Tests reported in research, or documented tests performed for other projects, which can be demonstrated to simulate project conditions.

(b) Tests conducted specifically for the project and representative of project member sizes, material strengths, connection configurations, and matching connection processes. At least two tests of each subassemblage type shall be performed successfully to qualify a connection for use. Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal stresses consistent with tested assemblies, and which considers potentially adverse effects of larger material and weld thickness and variations in material properties. Extrapolation of test results shall be limited to similar combinations of member sizes. Connections shall be constructed using materials, configurations, processes, and quality control and assurance methods that match as closely as is feasible those of the tested connections. Tests that utilize beams with tested F_y more than 10 percent lower than F_{ye} shall not be used.

(2) Connection flexural strength. The test results and analysis shall demonstrate a connection flexural strength, determined at the column face, at least equal to the nominal plastic moment, M_p, of the tested beam at the required inelastic rotation.

Exception: When beam flange buckling rather than connection strength limits the moment strength of the beam, and when connections using a reduced beam flange are used, then the limit shall be $0.8M_p$ of the tested beam. Figure 7-43 illustrates a fully restrained moment connection with haunches provided at the ends of the beam. This connection is designed such that the plastic hinge mechanism forms in the beam at the end of the haunch rather than at the column connection. If the beam size is based on strength considerations, haunches may

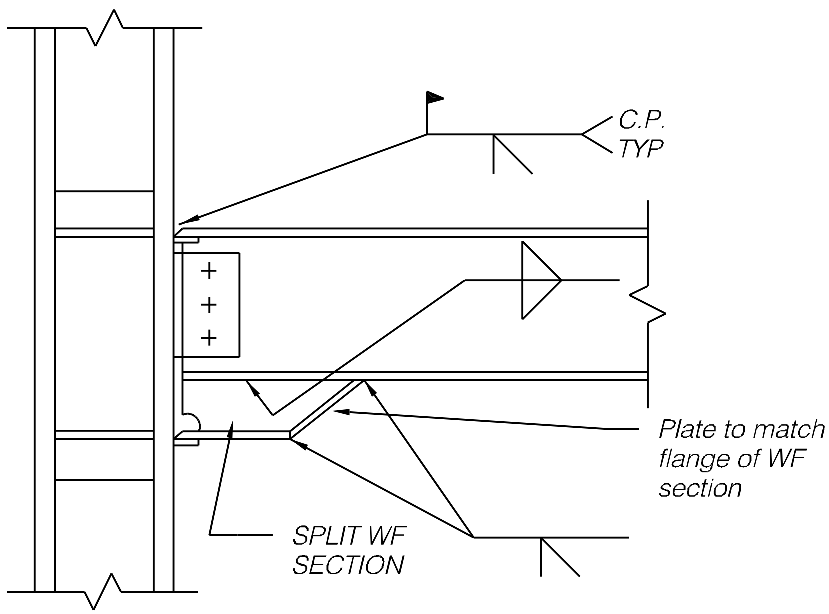


Figure 7-43 Typical post-Northridge fully restrained moment connection

permit selection of a shallower, and more economical, beam section. However, the beam depth and the length and depth of the haunch must be carefully selected to assure that the plastic hinge will occur at the end of the haunch, and not at the column face, for the combined seismic and factored gravity load moments. If the beam size is based on stiffness to control drift, the haunches may not contribute adequate stiffness to permit reduction in the size of the beam.

Exception: Connections that accommodate the required rotations within the connection itself and maintain the minimum required strength of Paragraph 7-5b(2)(a) are permitted to be used provided that the additional drift due to the connection deformation can be accommodated by the structure as demonstrated by rational analysis. Such rational analysis shall include consideration of overall frame stability, including the P-delta effect.

(3) Connection shear strength. The required shear strength, V_u, of a beam-to-column shall be determined using the load combination $1.2D + 0.5L + 0.2S$ plus the shear resulting from applying $1.1R_yF_yZ$ in the opposite sense on each end of the beam. Alternately, V_u shall be justified by rational analysis. The required shear strength is not required to exceed the shear resulting from the load combinations prescribed by Equations 4-6 and 4-7.

(4) Panel zone shear strength. The required shear strength, V_u, of the panel zone shall be the shear force determined by applying load combinations prescribed above to the moment-connected beam or beams in the plane of the frame at the column. V_u is not required to exceed the shear force determined from $0.8\Sigma M_p$ of the beams framing into the column flanges at the connection.

(5) Width-thickness ratios. Beams shall comply with λ_p in the AISC Design Specifications Table B5-1. When the ratio in Equation 7-3 is less than or equal to 1.25, columns shall comply with λ_p in Table I-9-1 of the AISC Seismic Provisions; otherwise, columns shall comply with Table B5-1 of the AISC Design Specifications.

(6) Continuity plates. Continuity plates shall be provided to match the tested connections. When tested connections do not include continuity plates, neither columns with thinner flanges nor beams with thicker or wider flanges shall be considered to be qualified by the test.

(7) Column/beam moment ratio. At any beam-to-column connection, the following strong column/weak beam relationship shall be satisfied:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} \geq 1.0 \qquad (7\text{-}3)$$

Where:

ΣM^*_{pc} = the moment at the intersection of the beam and column center-line determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g, from the top and bottom of the beam moment connection (including haunches where used). It shall be permitted to take G M^*_{pc} as G Z_c (F_{yc} - P_{uc}/A_g).

ΣM^*_{pb} = the moment at the intersection of the beam and column center-line determined by projecting the beam maximum developed moments from the column face thereto. Maximum developed moments shall be determined from test results as required by Paragraph 7-5b(2)(a) or by rational analysis based on the tests. Alternately, the maximum developed moment may be taken as 1.1 R_y M_p + M_v where M_v is the additional moment due to the shear amplification from the location of the plastic hinge to the column centerline. When connections with reduced beam sections are used, M_{pb} may be taken as 1.1 R_y F_y z + M_v, where z is the minimum plastic section modulus at the reduced section.

A_g = gross area of column, in^2 (mm^2).

F_{yc} = specified minimum yield strength of column ksi (Mpa).

P_{uc} = required axial strength in column, kips (kN).

Z_c = plastic section modulus of a column, in^3 (mm^3).

Ry = ratio of the expected yield strength, F_{ye}, to the minimum specified yield strength, F_y.

These requirements do not apply in any of the following cases, provided that the columns conform to the above minimum width-thickness ratios.

(a) Column with $P_{uc} < 0.3F_{yc}A_g$ for all load combinations:

1. Which are used in the top story of a multistory structure with a period greater than 0.7 seconds, or

2. Where the sum of their resistance is less than 20 percent of the shear in a story and is less than 33 percent of the shear on each of the column lines within that story. A column line is defined for the purpose of this exception as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

(b) Columns in any story that have a ratio of design shear strength to design force 50 percent greater than the story above.

(c) Any column not included in the design to resist the required seismic shears, but included in the design to resist axial overturning forces.

(8) Lateral support at beams. Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral support shall not exceed 3,600r_y/F_y (689.5 r_y/F_y for F_y in MPa). In addition, lateral supports shall be placed at concentrated loads where an analysis indicates a

hinge will be formed during inelastic deformation of the intermediate moment frame.

d. Special Moment Frames (SMFs). Special moment frames are expected to withstand significant inelastic deformation when subjected to the forces resulting from the motions of the design earthquake in combination with other loads. Special moment frames shall conform to all of the requirements for IMFs, except:

(1) Cyclic test results of the beam/column connection must demonstrate inelastic rotation capacity of at least 0.03 radian. The second exceptions in Paragraph 7-5c(2) shall not apply to SMFs.

(2) Circular sections shall have an outside-wall-diameter-to-thickness ratio not exceeding $1300/F_y$ ($250/F_y$ for F_y in MPa). Rectangular tubes shall have an out-to-out width-to-wall thickness b/t not exceeding $110/F_y$ ($21/F_y$ for F_y in MPa).

e. Special Truss Moment Frames (STMFs).

(1) General. Special truss moment frames, as shown in Figure 7-44, shall be designed so that when subjected to earthquake loading, yielding will occur in specially designed segments of the truss girders which are part of the lateral-force-resisting system. Such trusses shall be limited to span length between column not to exceed 60 feet (18m), and overall depth not to exceed 6 feet (1.8m). The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. Special truss moment frames shall have a design strength to resist the applicable load combinations of the AISC Seismic Provisions as modified by the following added requirements.

(2) Special segment. Each horizontal truss that is part of the moment frame shall have a special segment located within the middle one-half length of the truss. The length of the special segment shall range from 0.1 to 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall be limited to a maximum of 1.5 and a minimum of 0.67. All panels within a special segment shall be either Vierendeel or X braced, not a combination thereof. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points of crossing. The interconnection shall have a design strength adequate to resist a force at least equal to 0.25 times the diagonal member nominal tensile strength. Bolted connections shall not be used for web members within the special segment. Splicing of chord members shall not be permitted within the special segment, nor within ½ panel length from the ends of the special segment. Axial forces in diagonal web members due to factored dead plus live loads acting within the special segment shall not exceed $0.03\ F_yA_g$.

(3) Special segment nominal strength. In the fully yielded state, the special segment shall develop vertical nominal shear strength through the nominal flexural strength of the chord

members and through the nominal axial tensile and compressive strengths of the diagonal web members. The top and bottom chord members in the special segment shall be made of identical sections and in the fully yielded state shall provide at least 25 percent of the required vertical shear strength. The required axial strength in the chord members shall not exceed $0.45\,\phi F_y A_g$ where $f = 0.9$. Diagonal members in any panel of the special segment shall be made of identical sections. The end connections of diagonal members in the special segment shall have a design strength at

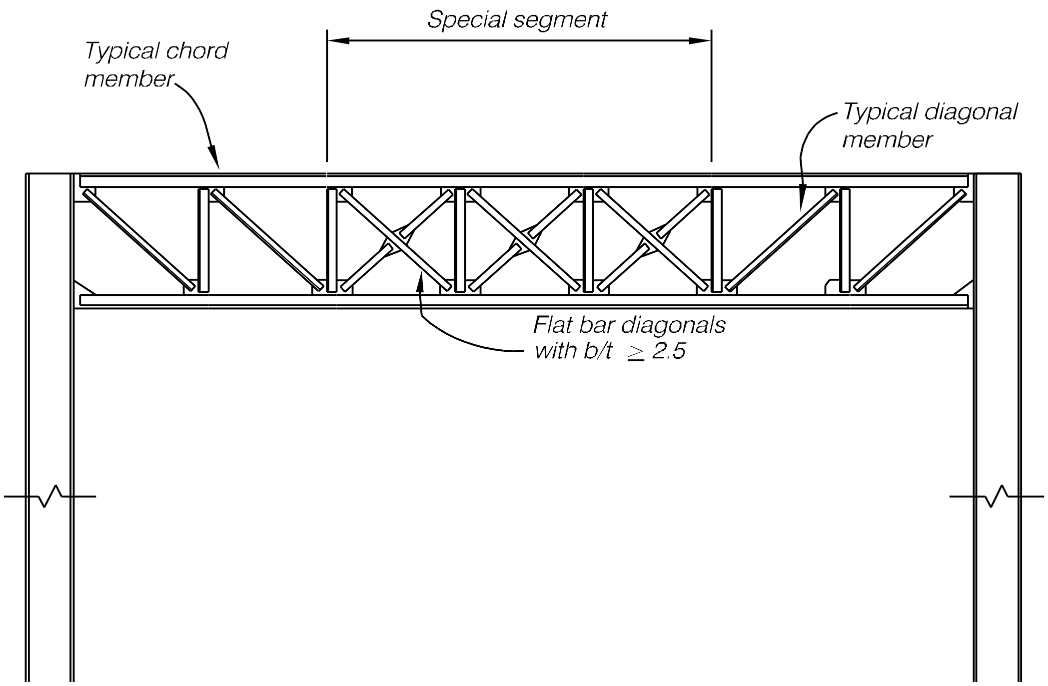


Figure 7-44 Special truss moment frame

least equal to the expected nominal axial tension strength of the web member, $R_yF_yA_g$.

(4) Non-special segment nominal strength. All members and connections of special truss moment frames, except those members identified as special segments, shall have a design strength to resist the factored gravity loads and the lateral loads necessary to develop the expected vertical nominal shear strength in all special segments, $V_{ne,}$ given by the following formula:

$$V_{ne} = \frac{3.4R_yM_{nc}}{L_s} + 0.07\ EI\left(\frac{L-L_s}{L_s^3}\right) + R_y\left(P_{nt} + 0.3P_{nc}\right)\sin\ a \tag{7-4}$$

where:

R_y = defined in the AISC Design Specification;

M_{nc} = nominal flexural strength of the chord member of the special segment (kips-in.) (kN-m);

EI = flexural elastic stiffness of the chord members of the special segment (kip-in^2) (MPa);

L = span length of the truss (in.) (mm);

L_s = 0.9 times the length of the special segment (in.) (mm);

P_{nt} = nominal axial tension strength of diagonal members of the special segment (kips) (kN);

P_{nc} = nominal axial compression strength of the diagonal members of the special segment (kips) (kN);

″ = angle of diagonal members with the horizontal plane.

(5) Compactness. Diagonal web members of the special segment shall be made of flat bars. The width-thickness ratio of such flat bars shall not exceed 2.5. The width-thickness ratio of chord members shall not exceed the limiting l_p values from Table B5.1 of the AISC Design Specification. The width-thickness ratio of angles, and flanges and webs of tee sections used for chord members in the special segment, shall not exceed $52/\sqrt{F_y}$ $(137/\sqrt{F_y}$ for F_y in MPa.

(6) Lateral bracing. Top and bottom chords of the trusses shall be laterally braced at the ends of special segments, and at intervals not to exceed $L_{p,}$ according to Section F1.1 of the AISC Design Specification, along the entire length of the truss. Each lateral brace at the ends of, and within, the special segment shall have a design strength to resist at least 5 percent of the required compressive axial strength, P_{nc}, of the largest adjoining chord member. Lateral braces outside of the special segment shall have at least 2.5 percent of the required P_{nc} of the
largest adjoining chord members.

f. Acceptance Criteria.

(1) Response modification factors, R, for Performance Objective 1A for moment frames in

various structural systems are provided in Table 7-1.

(2) Modification factors, *m*, for enhanced performance objectives for beams, columns, and connections in fully restrained moment frames are provided in Table 7-12, and for partially restrained moment frames in Table 7-13.

(3) Modeling parameters and numerical acceptance criteria for nonlinear procedures are provided in Table 7-22 for fully restrained moment frames, and in Table 7-23 for partially restrained moment frames.

(4) Expected strength of columns, beams, and other deformation-controlled components shall be determined using the expected yield strength, F_{ye}, as defined in the AISC Seismic Provisions and the plastic section modulus, *Z*, where applicable.

(5) The lower bound strength of connections and other force-controlled components shall be determined in accordance with the nominal strength and *N* factors prescribed by AISC Seismic Provisions.

7-6. Dual Systems.

a. General.

(1) Combinations of structural systems. The connotation of dual systems is sometimes erroneously interpreted to mean different systems in each orthogonal direction of structural framing. To clarify this point, FEMA 302 describes that condition as "combinations of structural systems." In addition to the above interpretation, these combinations could also include different systems in the same vertical plane (e.g., a two-story building with steel moment frames in the second story and a concrete shear wall system in the first story). In the first case above, FEMA 302 permits the use of the appropriate *R* factor pertaining to the structural system in each orthogonal direction. For the second case, the FEMA provision states: "The response modification coefficient, *R*, in the direction under consideration at any story shall not exceed the lowest response modification factor, *R*, for the seismic-force-resisting system in the same direction considered above that story, excluding penthouses. For other than dual systems where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of *R* used in that direction shall not be greater than the least value of any of the systems utilized in the same direction. If a system other than a dual system with a response modification coefficient, *R*, with a value of less than 5 is used as part of the seismic-force-resisting system in any direction of the structure, the lowest such value shall be used for the entire structure. The system overstrength factor, Ω_o, in the direction under consideration at any story, shall not be less than the largest value of this factor for the seismic-force-resisting system in the same direction considered above that story."

Exceptions:

(a) Supported structural systems with a weight equal to or less than 10 percent of the weight of the structure.

Component/Action	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits					
				Primary				Secondary	
	d	*e*	*c*	IO	SE	LS	CP	LS	CP
Beams[1]:									
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	5	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	5	7	0.2	1	2	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation									
Columns[2]: For $P/P_{ye} < 0.20$									
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	10	12	0.6	2	5	7	9	10	12
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$			0.2	1	2	3	4	4	5
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation									

1. Add θ_y from Equations 6-6 or 6-7 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = $0.072\ (1 - 1.7\ P/P_{ye})$
4. Deformation = $0.100\ (1 - 1.7\ P/P_{ye})$
5. Deformation = $0.042\ (1 - 1.7\ P/P_{ye})$
6. Deformation = $0.060\ (1 - 1.7\ P/P_{ye})$
7. $0.043 - 0.0009\ d_b$
8. $0.035 - 0.0008\ d_b$
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 7-22: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames

Component/Action	$\frac{\Delta}{\Delta_y}$		Residual Strength Ratio	Plastic Rotation, Deformation Limits					
				Primary				Secondar	
	d	e	c	IO	SE	LS	CP	LS	C
For $0.2 \le P/P_{ye} \le 0.50$[9]									
a. $\frac{b}{2t_f} < \frac{52}{\sqrt{F_{ye}}}$	—[3]	—[4]	0.2	0.04	0.05	—[5]	—[6]	0.019	0.(
b. $\frac{b}{2t_f} > \frac{95}{\sqrt{F_{ye}}}$	2	2.5	0.2	1	1.3	1.5	1.8	1.8	2
c. For $\frac{52}{\sqrt{F_{ye}}} \le \frac{b}{2t_f} \le \frac{95}{\sqrt{F_{ye}}}$ use linear interpolation									
	Plastic Rotation								
	a	b							
Panel Zones	0.052	0.081	0.800	0.004	0.015	0.025	0.043	0.055	0.(
Connections									
For full penetration flange weld, bolted or welded web: beam deformation limits									
a. No panel zone yield	—[7]	—[7]	0.200	0.008	—[8]	—[8]	—[8]	0.017	0.(
b. Panel zone yield	0.009	0.017	0.400	0.003	0.004	0.005	0.007	0.010	0.(

1. Add θ_y from Equations 6-6 or 6-7 to plastic end rotation to estimate chord rotation.
2. Columns in moment or braced frames need only be designed for the maximum force that can be delivered.
3. Deformation = $0.072 (1 - 1.7\, P/P_{ye})$
4. Deformation = $0.100 (1 - 1.7\, P/P_{ye})$
5. Deformation = $0.042 (1 - 1.7\, P/P_{ye})$
6. Deformation = $0.060 (1 - 1.7\, P/P_{ye})$
7. $0.043 - 0.0009\, d_b$
8. $0.035 - 0.0008\, d_b$
9. If $P/P_{ye} > 0.5$, assume column to be force-controlled.

Table 7-22: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Fully Restrained (FR) Moment Frames (Continued)

	Plastic Rotation[1] a	Plastic Rotation[1] b	Residual Force Ratio c	Joint Rotation Primary IO	Joint Rotation Primary SE	Joint Rotation Primary LS	Joint Rotation Primary CP	Joint Rotation Secondary LS	Joint Rotation Secondary CP
Top and Bottom Clip Angles[1]									
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.014	0.020	0.030	0.030	0.040
b. Angle flexure	0.042	0.084	0.200	0.010	0.018	0.025	0.035	0.035	0.070
c. Bolt tension	0.016	0.025	1.000	0.005	0.007	0.008	0.013	0.020	0.020
Top and Bottom T-Stub[1]									
a. Rivet or bolt shear[2]	0.036	0.048	0.200	0.008	0.014	0.020	0.030	0.030	0.040
b. T-stub flexure	0.042	0.084	0.200	0.010	0.018	0.025	0.035	0.035	0.070
c. Rivet or bolt tension	0.016	0.024	0.800	0.005	0.007	0.008	0.013	0.020	0.020
Composite Top Angle Bottom[1]									
a. Deck reinforcement	0.018	0.035	0.800	0.005	0.008	0.010	0.015	0.020	0.030
b. Local yield column flange	0.036	0.042	0.400	0.008	0.014	0.020	0.030	0.025	0.035
c. Bottom angle yield	0.036	0.042	0.200	0.008	0.014	0.020	0.030	0.025	0.035
d. Connectors in tension	0.015	0.022	0.800	0.005	0.007	0.008	0.013	0.013	0.018
e. Connections in shear[2]	0.022	0.027	0.200	0.005	0.009	0.013	0.018	0.018	0.023
Flange Plates Welded to Column Bolted or Welded to Beam[2]									
a. Flange plate net section or shear in connectors	0.030	0.030	0.800	0.008	0.014	0.020	0.025	0.020	0.025
b. Weld or connector tension	0.012	0.018	0.800	0.003	0.006	0.008	0.010	0.010	0.015
End Plate Bolted to Column Welded to Beam									
a. End plate yield	0.042	0.042	0.800	0.010	0.019	0.028	0.035	0.035	0.035
b. Yield of bolts	0.018	0.024	0.800	0.008	0.009	0.010	0.015	0.020	0.020
c. Fracture of weld	0.012	0.018	0.800	0.003	0.006	0.008	0.010	0.015	0.015

1. If $d_b > 18$, multiply deformations by $18/d_b$. Assumed to have web plate to carry shear. Without shear connection, this may not be downgraded to a secondary member.
2. For high-strength bolts, divide rotations by 2.

Table 7-23: Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—Partially Restrained (PR) Moment Frames

(b) Detached one- and two-family dwelling of light-frame construction.

(2) Dual systems in the FEMA 302 provisions are defined as moment frames with either braced frames or shear walls that jointly resist lateral forces along the same line of force. The lateral forces are distributed to the various structural components in accordance with their relative rigidities, but the moment frames are designed to be capable of resisting at least 25 percent of the design forces. The moment frame shall be part of an essentially complete space frame system providing support for vertical loads. These dual systems are described further in the following paragraphs.

b. Moment Frame/Shear Wall Systems. As limited by this document, these dual systems shall consist of either structural steel or reinforced concrete moment frames resisting lateral forces jointly with either reinforced concrete or reinforced masonry shear walls. Appropriate *R* factors and other design coefficients for other systems are provided in Table 7-1.

c. Moment Frames/Bracing Systems. As defined by this document, these systems shall consist of steel moment frames with selected braced bays so that lateral forces are resisted partly by moment frame action and partly by braced frame action. The bracing system can consist of either concentrically or eccentrically braced frames. *R* factors and other design coefficients for these various systems are provided in Table 7-1. The use of concrete moment frames with either concrete or steel bracing is not prescribed by this document, as the detailing requirements are very demanding, and the performance of these systems has not been satisfactory.

d. Acceptance Criteria.

(1) Response modification factors, *R*, for Performance Objective 1A for various dual systems are provided in Table 7-1.

(2) Modification factors, *m*, for enhanced performance objectives, and modeling parameters and numerical criteria for nonlinear procedures are prescribed in the following paragraphs;

Reinforced concrete shear walls….…para. 7-2f(3)

Precast concrete shear walls………...para. 7-2g(5)

Unreinforced masonry shear walls…para. 7-2h(5)

Concentric braced frames…………..para. 7-3b(9)

Eccentric braced frames…………….para. 7-3c(5)

Steel moment resisting frames………para. 7-5f.

7-7. Diaphragms.

a. General.

(1) Function. Floors and roofs, acting as diaphragms, are the horizontal resisting elements in a structure. Diaphragms are subject to lateral forces due to their own weight plus the tributary weight of walls connected to them. The diaphragms distribute the lateral forces to the vertical elements: the shear walls or frames, which resist the lateral forces and transfer them to lower levels of the building and finally to the ground. If floors or roofs cannot be made strong enough, their diaphragm function can be accomplished by horizontal bracing. In an industrial building, horizontal bracing can be the only resisting element. Where there is a horizontal offset between resisting vertical elements above and below, the diaphragm transfers lateral forces between the elements.

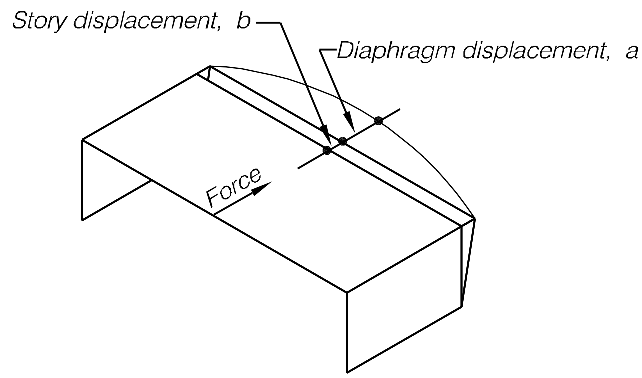
(2) Diaphragms. Usually the roof and floors of the structure perform the function of distributing lateral forces to the vertical-resisting elements (such as walls and frames). These elements, called diaphragms, make use of their inherent strength and rigidity, supplemented, when needed, by chords and collectors. A diaphragm is analogous to a plate girder laid in a horizontal plane (or inclined plane, in the case of a roof). The floor or roof deck functions as the girder web, resisting shear; the joists or beams function as web stiffeners; and the chords (peripheral beams or integral reinforcement) function as flanges, resisting flexural stresses (Figure 7-46). A diaphragm may be constructed of any material of which a structural floor or roof is made. Some materials, such as cast-in-place reinforced concrete and structural steel, have well-established properties and present no problems for diaphragm design once the loading and reaction system is known. Other materials, such as wood sheathing and metal deck, have properties that are well-established for vertical loads, but not so well established for lateral loads. For these materials, tests have been required to demonstrate their ability to resist lateral forces. Moreover, where a diaphragm is made up of units such as sheets of plywood or metal deck, or precast concrete units, the characteristics of the diaphragm are, to a large degree, dependent upon the connections that join one unit to another and to the supporting members.

(3) Horizontal bracing. A horizontal bracing system may also be used as a diaphragm to transfer the horizontal forces to the vertical-resisting elements. A horizontal bracing system may be of any approved material. A common system that is not recommended is the rod or angle tension-only bracing used in older

industrial buildings. The general layout of a bracing system and the sizing of members must be determined for each case in order to meet the requirements for load resistance and deformation control. The bracing system will be fully developed in both directions so that the bracing diagonals and chord members form complete horizontal trusses between vertical-resisting elements (Figure 7-47). Horizontal bracing systems will be designed using diaphragm design principles.

b. Diaphragm Flexibility.

(1) Diaphragm classification. Floor diaphragms shall be classified as either flexible, stiff, or rigid, as indicated in Figure 7-45. Diaphragms shall be considered flexible when the maximum lateral deformation of the diaphragm along its length is more than twice the average interstory drift of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm may be used in lieu of the basement story.



Deflection Ratio	Flexibility
a > 2b	Flexible
2b > a > 1/2b	Stiff
a < 1/2b	Rigid

Figure 7-45 Diaphragm flexibility

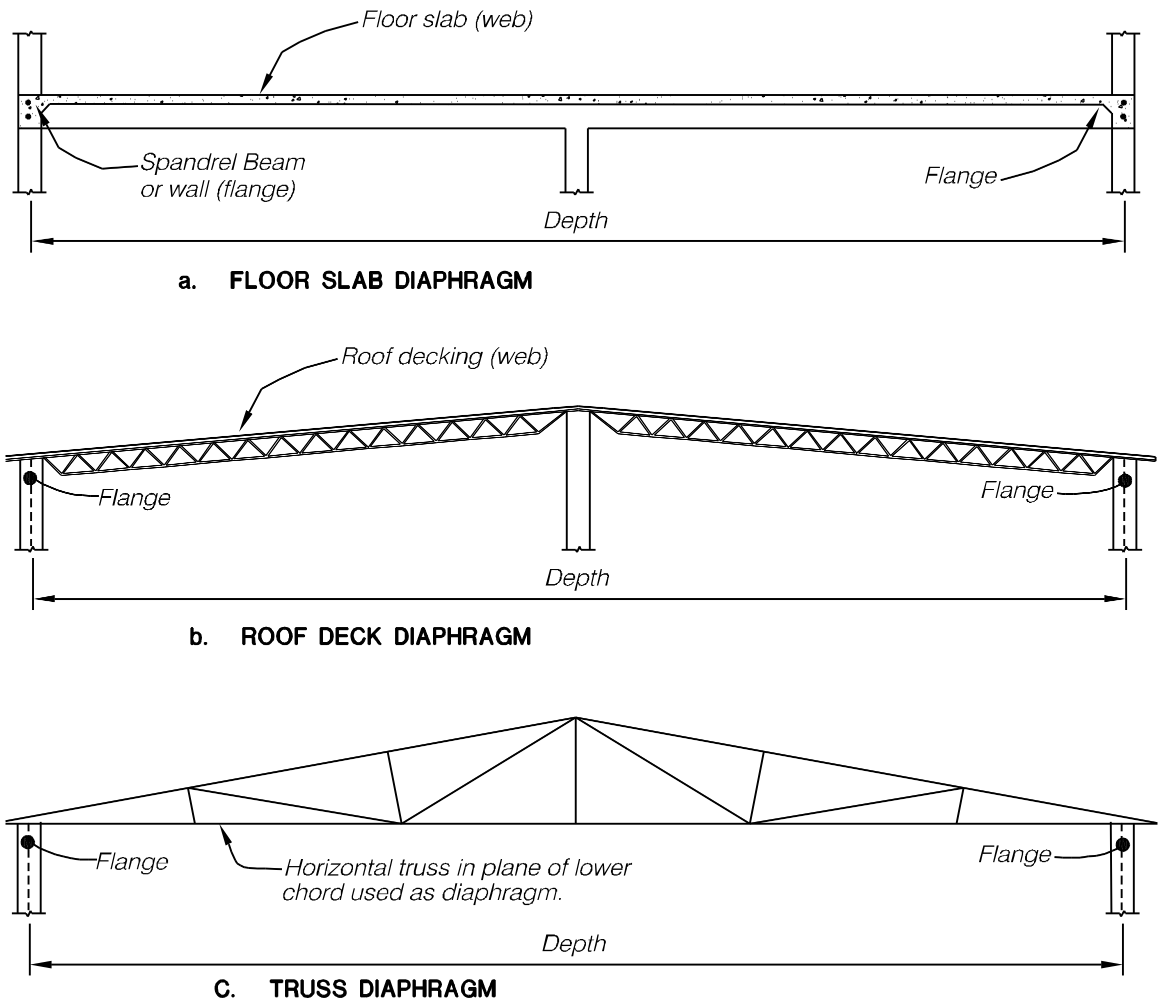


Figure 7-46 Diaphragms.

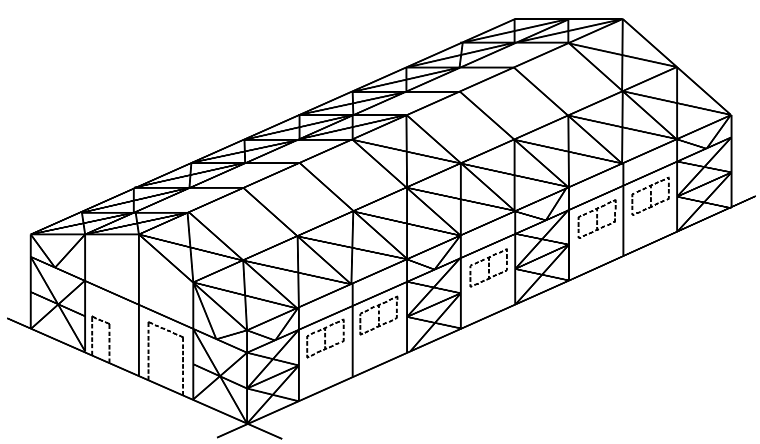


Figure 7-47 Bracing an industrial building.