11 SMRF

- Joint Shear
- Anchorage
- Column Shear
- $\sum M_{nc} \geq \frac{6}{5} \sum M_{nb}$ (Check one floor below roof)

Columns

SEE JOINT SECTION

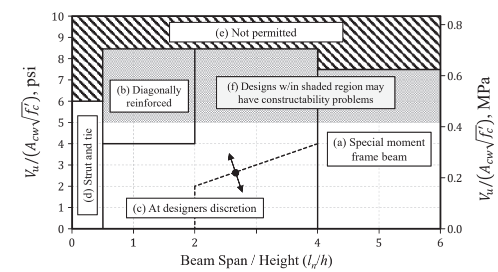
Beams

Shear

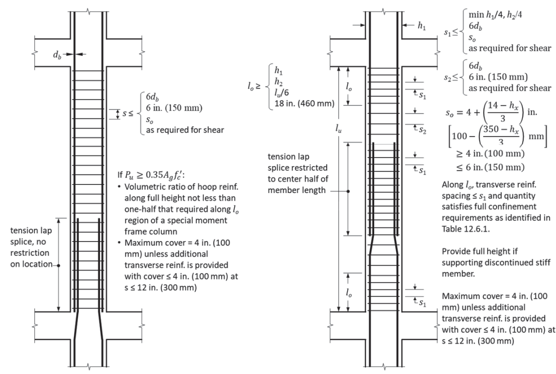
$V_u = f(M_p r)$

$V_c = 0$ in plastic hinge regions.

Coupling beam design space



- Longitudinal reinforcement satisfies $0.01 \leq A_{st}/A_g \leq 0.06$.
- Transverse reinforcement is spirals, circular hoops, or rectilinear hoops and crossties, designed to resist shear corresponding to M_{pr}.
- Rectilinear hoops and crossties engage at least corner and alternate longitudinal bars, with no unsupported bar more than 6 in. (150 mm) clear from a supported bar, and with spacing of legs h_x within cross section not exceeding 14 in. (350 mm) on center. Where $P_u \geq 0.3A_g f_c'$ or $f_c' \geq$ 10,000 psi (70 MPa), every longitudinal bar around the perimeter of the column core shall have lateral support provided by the corner of a hoop or by a seismic hook, with h_x not exceeding 8 in. (200 mm).



(a) Demands do not exceed design strengths

(b) Demands exceed design strengths

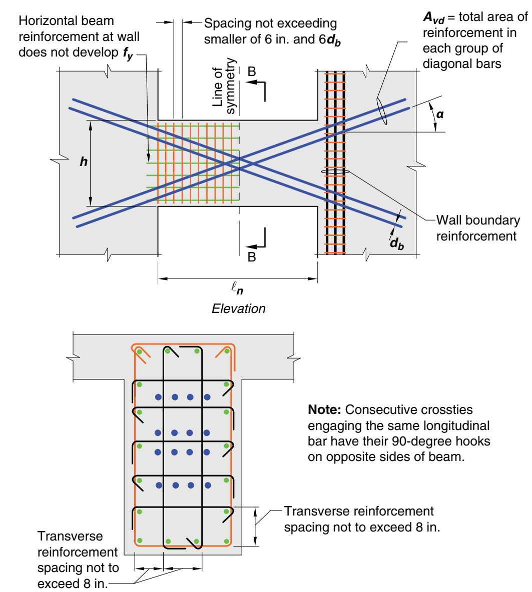


Fig. R18.10.7b—Full confinement of diagonally reinforced concrete beam section in coupling beams with diagonally oriented reinforcement. Wall boundary reinforcement shown on one side only for clarity.

18.10.8 *Wall piers*

18.10.8.1 Wall piers shall satisfy the special moment frame requirements for columns of 18.7.4, 18.7.5, and 18.7.6, with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $(\ell_w/b_w) > 2.5$ shall satisfy (a) through (f):

(a) Design shear force shall be calculated in accordance with 18.7.6.1 with joint faces taken as the top and bottom of the clear height of the wall pier. If the general building code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force

R18.10.8 *Wall piers*

Door and window placements in structural walls sometimes lead to narrow vertical wall segments that are considered to be wall piers. The dimensions defining wall piers are given in Chapter 2. Shear failures of wall piers have been observed in previous earthquakes. The intent of this section is to provide sufficient shear strength to wall piers such that inelastic response, if it occurs, will be primarily in flexure. The provisions apply to wall piers designated as part of the seismic-force-resisting system. Provisions for wall piers not designated as part of the seismic-force-resisting system are given in 18.14. The effect of all vertical wall segments on the

13 Gravity Framing

Columns

Confinement

if $P_u \geq 0.35 A_g f_c'$:

- Support all bars with 135^o hook.
- $\frac{A_{sh}}{sb_c} \geq 0.3..., 0.09..., 0.2...$

Shear

$V_u = f(M_{pr}(P_u))$, often $\frac{2M_{pr}}{l_w}$

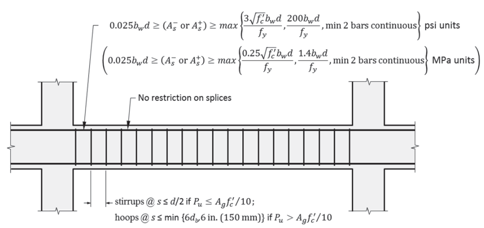
Beams

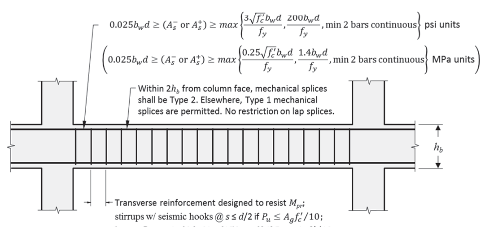
FIGURE FROM NOTES PAGE 200

Slabs

$V_n = 4\lambda_s\sqrt{f_c'}b_o d$

DRIFT CAPACITY





14 Diaphragms

18.14.5 **Slab-Column Connections**

18.14.5.2 The shear reinforcement requirements of 18.14.5.1 need not be satisfied if (a) or (b) is met:

(a) $\Delta_x/h_{sx} \leq 0.005$ for nonprestressed slabs
(b) $\Delta_x/h_{sx} \leq 0.01$ for unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1

18.14.5.3 Required slab shear reinforcement shall provide $v_s \geq 3.5\sqrt{f_c'}$ at the slab critical section and shall extend at least four times the slab thickness from the face of the support adjacent to the slab critical section.

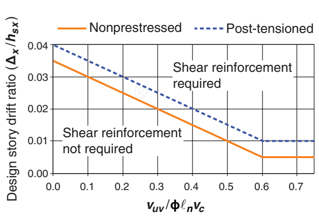


Fig. R18.14.5.1—Illustration of the criteria of 18.14.5.1.

12 Walls

δ_c Wall displacement capacity at top of wall

Resistance Factors

Axial/Moment:
$\phi = 0.65 - 0.9$

Shear:
$\phi = 0.75$, For V_u amplified by Ω_v
$\phi = 0.60$ if $V_n < V(M_n)$
for squat walls, take $\phi = 0.6$

Coupling beam shear:
$\phi = 0.85$ diagonally reinforced
$\phi = 0.75$ otherwise

Lateral stability

$\lambda_c = \frac{c}{b}\frac{l_w}{b} > 40$

$b \geq \sqrt{0.025 c l_w}$ **for SBEs**.

Preliminary Proportioning for V_b

assume $l_{be} = 0.2 l_w$

Slender Walls w/ C. Section

Distributed rebar

$\rho_l, \quad \rho_t = \frac{2A_{si}}{ts} \geq 0.0025$ if $V_u \geq \lambda A_{cv}\sqrt{f_c'}$

Boundary Elements

SEE NOTES PAGE 163 B.E. FRACTURE

18.10.3.1 The design shear force V_e shall be calculated by:

$$V_e = \Omega_v \omega_v V_u \leq 3V_u \qquad (18.10.3.1)$$

18.10.3.1.1 V_u is the shear force obtained from code lateral load analysis with factored load combinations.

Table 18.10.3.1.2—Overstrength factor Ω_v at critical section

Condition	Ω_v	
$h_{wcs}/\ell_w > 1.5$	Greater of	M_{pr}/M_u[1]
		1.5[2]
$h_{wcs}/\ell_w \leq 1.5$	1.0	

[1] For the load combination producing the largest value of Ω_v.
[2] Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.

18.10.3.1.3 For walls with $h_{wcs}/\ell_w < 2.0$, ω_v shall be taken as 1.0. Otherwise, ω_v shall be calculated as:

$$\omega_v = 0.9 + \frac{n_s}{10} \quad n_s \leq 6$$
$$\omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 \quad n_s > 6 \qquad (18.10.3.1.3)$$

where n_s shall not be taken less than the quantity $0.007h_{wcs}$.

18.10.4 *Shear strength*

18.10.4.1 V_n shall be calculated by:

$$V_n = (\alpha_c \lambda \sqrt{f_c'} + \rho_t f_{yt}) A_{cv} \qquad (18.10.4.1)$$

where:
$\alpha_c = 3$ for $h_w/\ell_w \leq 1.5$
$\alpha_c = 2$ for $h_w/\ell_w \geq 2.0$
It shall be permitted to linearly interpolate the value of α_c between 3 and 2 for $1.5 < h_w/\ell_w < 2.0$.

18.10.4.2 In 18.10.4.1, the value of ratio h_w/ℓ_w used to calculate V_n for segments of a wall shall be the greater of the ratios for the entire wall and the segment of wall considered.

18.10.4.3 Walls shall have distributed shear reinforcement in two orthogonal directions in the plane of the wall. If h_w/ℓ_w does not exceed 2.0, reinforcement ratio ρ_ℓ shall be at least the reinforcement ratio ρ_t.

18.10.4.4 For all vertical wall segments sharing a common lateral force, V_n shall not be taken greater than $8\sqrt{f_c'}A_{cv}$. For any one of the individual vertical wall segments, V_n shall not be taken greater than $10\sqrt{f_c'}A_{cw}$, where A_{cw} is the area of concrete section of the individual vertical wall segment considered.

18.10.4.5 For horizontal wall segments and coupling beams, V_n shall not be taken greater than $10\sqrt{f_c'}A_{cw}$, where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam.

SBE

Required if
$c \geq \frac{l_w}{900(\delta_u/h_w)}$ or $\sigma \geq \frac{f_c'}{5}$

use given graph to determine c.

for min. height of $h_{be} = \max(l_w, \frac{M_{u,cs}}{4V_{u,cs}})$

- $A_{sh} \geq 0.09 s b_c \frac{f_c'}{f_y} \geq 0.3\left(\frac{A_g}{A_{ch}} - 1\right) s b_c \frac{f_c'}{f_y}$

Openings

Tie region: $A_s = T_u/\phi f_y$
Strut region: $P_u \leq \phi P_o$

$\phi = 0.65$ in wall piers
$\phi = 0.60$ for wall shear
$\phi = 0.75$ otherwise

13.12 Coupled Walls

Coupling beams

$V_n = 2A_{vd} f_y \sin\alpha \leq 10\sqrt{f_c'}A_{cw}$

- ϕ for A_{tr} is 0.75

Shear

$V_u = \frac{M_n[P_u]}{M_u}\omega_v V_{code}$

Detailing

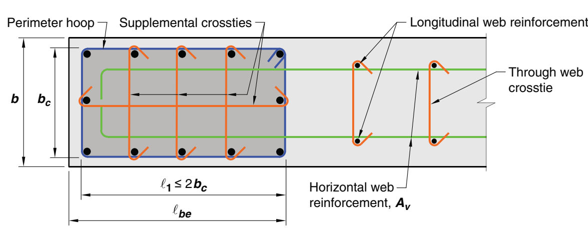
long. bar fracture:
$A_s f_y \geq A_{be} f_r \therefore \rho_l \geq 6\sqrt{f_c'}/f_y$

Cut-offs:
$0.8 l_w$ or l_d above next floor

No-splice zone:
min($20'$, $h_{1^{st} floor}$)

Wall Piers

$P_{uc} = 1.2P_{Dc} + 0.5P_{Lc} + n_s P_E$



(a) Perimeter hoop with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement

14 Diaphragms

Moment

$M_u = \frac{wl^2}{8} \leq \phi M_n = 0.9(A_s f_y 0.9d)$

$T_u = C_u = \frac{M_u}{jd}$

$T_u \leq Tn = 0.9A_s fy$

$C_u \leq \phi\alpha P_{no} = (0.65)(0.8)(A_s f_y + 0.85A_c f_c')$

Collectors

Shear friction

$\Omega_o V_u \leq \underbrace{\phi}_{0.60}\underbrace{\mu}_{1.4} A_{sf} f_y$

- Give A_{sf} as in^2/ft

Openings

Comp. Zones

- Confine if $\frac{C}{A} \geq 0.2f_c'$

18.12.7 *Reinforcement*

18.12.7.4 Type 2 splices are required where mechanical splices on Grade 60 reinforcement are used to transfer forces between the diaphragm and the vertical elements of the seismic-force-resisting system. Grade 80 and Grade 100 reinforcement shall not be mechanically spliced for this application.

18.12.7.5 Longitudinal reinforcement for collectors shall be proportioned such that the average tensile stress over length (a) or (b) does not exceed ϕf_y where the value of f_y is limited to 60,000 psi.

(a) Length between the end of a collector and location at which transfer of load to a vertical element begins
(b) Length between two vertical elements

18.12.7.6 Collector elements with compressive stresses exceeding $0.2f_c'$ at any section shall have transverse reinforcement satisfying 18.7.5.2(a) through (e) and 18.7.5.3, except the spacing limit of 18.7.5.3(a) shall be one-third of the least dimension of the collector. The amount of transverse reinforcement shall be in accordance with Table 18.12.7.6. specified transverse reinforcement discontinued where compressive stress is less than $0.15f_c'$.
If design forces have been amplified to account for the overstrength $0.2f_c'$ shall be increased to $0.5f_c'$, and $0.15f_c'$ increased to $0.4f_c'$.

Table 18.12.7.6—Transverse reinforcement for collector elements

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	$0.09\frac{f_c'}{f_{yt}}$		(a)
ρ_s for spiral or circular hoop	Greater of:	$0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$	(b)
		$0.12\frac{f_c'}{f_{yt}}$	(c)

18.12.3 *Seismic load path*

18.12.3.1 All diaphragms and their connections shall be designed and detailed to provide for transfer of forces to collector elements and to the vertical elements of the seismic-force-resisting system.

18.12.3.2 Elements of a structural diaphragm system that are subjected primarily to axial forces and used to transfer

diaphragm shear or flexural forces around openings or other discontinuities shall satisfy the requirements for collectors in 18.12.7.6 and 18.12.7.7.

R18.12.3.2 This provision applies to strut-like elements that occur around openings, diaphragm edges, or other

discontinuities in diaphragms.

R18.12.2 *Design forces*

R18.12.2.1 For collector elements, the general building code in the United States specifies load combinations that amplify earthquake forces by a factor Ω_o. The forces amplified by Ω_o are also used for the local diaphragm shear forces resulting from the transfer of collector forces, and for local diaphragm flexural moments resulting from any eccentricity of collector forces.

R18.12.7 *Reinforcement*

R18.12.7.1 Minimum reinforcement ratios for diaphragms correspond to the required amount of temperature and shrinkage reinforcement (refer to 24.4).

R18.12.7.6 In documents such as ICBO 1997, collector elements of diaphragms are designed for forces amplified by a factor Ω_o to account for the overstrength in the vertical elements of the seismic-force-resisting systems.
R18.12.7.7 This section is intended to reduce the possibility of bar buckling and provide adequate bar development conditions in the vicinity of splices and anchorage zones.

18.12.9 *Shear strength*

18.12.9.1 V_n of diaphragms shall not exceed:

$$V_n = A_{cv}\left(2\lambda\sqrt{f_c'} + \rho_t f_y\right) \qquad (18.12.9.1)$$

18.12.9.2 V_n of diaphragms shall not exceed $8\sqrt{f_c'}A_{cv}$.

18.12.9.3 Above joints between precast elements in noncomposite and composite cast-in-place topping slab diaphragms, V_n shall not exceed:

$$V_n = A_{vf} f_y \mu \qquad (18.12.9.3)$$

where A_{vf} is the total area of shear friction reinforcement within the topping slab, including both distributed and boundary reinforcement, that is oriented perpendicular to joints in the precast system and coefficient of friction, μ, is 1.0λ, where λ is given in 19.2.4. At least one-half of A_{vf} shall be uniformly distributed along the length of the potential shear plane.

R18.12.9 *Shear strength*

A_{cv} refers to the gross area of the diaphragm, but may not exceed the thickness times the width of the diaphragm. This corresponds to the gross area of the effective deep beam that forms the diaphragm.
Distributed slab reinforcement ρ_t used to calculate shear strength of a diaphragm in Eq. (18.12.9.1) is positioned perpendicular to the diaphragm flexural reinforcement.
In addition to satisfying 18.12.9.1 and 18.12.9.2, cast-in-place topping slab diaphragms must also satisfy 18.12.9.3 and 18.12.9.4.
The coefficient of friction, μ, in the shear friction model is taken equal to 1.0 for normalweight concrete due to the presence of these shrinkage cracks.
Provision 18.12.9.4 limits the maximum shear that may be transmitted by shear friction within a topping slab diaphragm.