- ERRATA -

for

PCI Design Handbook

Second Edition

(MNL-120-78)

Errata

PCI Design Handbook - Precast, Prestressed Concrete Second Edition

(MNL-120-78)

page 2-5 — Section 2.2.7 Strand Placement: add at end of first paragraph, "except as noted."

page 2-7 — Under Section Properties: change weight of untopped unit from 79 psf to 29 psf

page 2-15 — On sketch: add depth dimension of 20"

page 2-51 — On sketch: add ledge dimension of 6"
In table of safe superimposed load, change eccentricities, e, to the following:

18LB20	6.26	18LB44	14.19
18LB24	7.67	18LB48	15.48
18LB28	8.93	18LB52	16.78
18LB32	10.22	18LB56	18.07
18LB36	11.52	18LB60	19.36
18LB40	12.52		

page 2-52 — On sketch, change h_1 to h_2 and h_2 to h_1
In table of safe superimposed load, change eccentricities, e, to the following:

24IT20	6.20	24IT44	13.73
24IT24	7.17	24IT48	15.08
24IT28	8.44	24IT52	16.44
24IT32	9.77	24IT56	17.82
24IT36	11.50	24IT60	19.18
24IT40	12.02		

page 2-53 — In table of safe superimposed load: change third column heading to $\frac{e_e}{e_c}$

page 2-60 — Plots for 6-in. and 8-in. walls are incorrect (see attached)

page 3-10 — Section 3.2.2. Service Load Stresses: for concrete under service loads, items a, b and c, change

f'_{ci} to f'_c

pages 3-12 and 3-13 — Example 3.2.8: change eccentricities, single point depression to

$e_e =$ 4.29 in.

e @ 0.4 ℓ = 11.78 in.

e' = 13.65 − 4.29 = 9.36 in.

correct table of stresses, p.3-13 (see attached)

page 3-15 — Equation 3.3-2: change v_u to v_c

page 3-21 — In Fig. 3.34, under Case 2: change expression to

$$F_h = C_c < T$$

page 3-27 — Section 3.4.3, paragraph 5: change first sentence to "Long-time effects can be substantially reduced by adding non-prestressed reinforcement in prestressed concrete members."

page 3-30 — First column, definition for f_{cds}: delete "compressive"

page 3-36 — Second column, line 14: change "required length" to "required area"

page 3-42 — Second column, line 5: delete expression

$$\frac{b}{4t} \leqslant 1$$

page 3-43 — Equation 3.7-11: change to

$$v'_{tc} = 6\,\lambda\sqrt{f'_c}\,(\gamma_t\ \ -0.60)$$

page 3-45 — First column, under 6: change equation for v'_{tc} to include λ (see correction for page 3-43)

page 3-71 — Fig. 3.9.25: change abscissa scale values for R_v from 0 to 0.1, 0.1 to 0.2, etc., through 0.9 to 1.0.

page 4-19 — Fig. 4.5.1: change expression for max. shear between members to $\left(\frac{2a}{\ell}\right) V_R$

page 4-32 — Example 4.6.5: the moments at top and bottom of column are opposite sign, hence M_1/M_2 is a negative number. To show calculations for moment magnifier, change example to 14 x 14 in column (see attached)

page 4-38 — First column, line 9 from bottom: change "colume" to "column"

(over)

page 4-41 — Section 4.7.2: in definition for K_i, delete "$= K_{si} + K_{fi}$" and add

$$\text{where } \frac{1}{K_i} = \frac{1}{K_{si}} + \frac{1}{K_{fi}}$$

In definition for K_{fi}, add parenthetical (this assumes double curvature in the wall)

page 4-49 and pages 4-50 thru 4-52 — Second column, last line: change expression for $K_s + K_f$ to

$$\frac{1}{K_s} + \frac{1}{K_f} = \frac{3h_s}{EA_w} + \frac{h_s^3}{12EI}$$

This changes all subsequent calculations for Example 4.7.6 (see attached)

page 5-6 — Second column: in definition for f_y change 70,000 to 60,000

page 5-7 — Table 5.6.1 under Item 2: delete "Concrete to steel with headed studs"

(Note: This change implies that Item 4 pertains to all concrete to steel interface conditions, with or without headed studs.)

page 5-20 — Table 5.12.1: for I-section, change expression for Z_s on x-x axis to

$$bt\,(h\text{-}t) + \frac{w}{4}\,(h\text{-}2t)^2$$

for channel section, change expression for Z_s to

$$bt\,(h\text{-}t) + \frac{w}{4}\,(h\text{-}2t)^2$$

for hollow circular section, in columns 2 and 3, change

$t < h$ to $t \ll h$

page 5-31 — Table 5.201.1: add asterisk at beginning of sentence, *Table values — etc.

page 5-33 — Table 5.20.3: in sketch, change notation for inclined bars from A_{vf} to $A_{vf} + A_n$

Change equation for A_n to

$$A_n = \frac{N_u}{\phi f_y}$$

pages 8-19 and 8-20 — Both Tables 8.2.7: change instructions in tables to read

Multiply table values by:

For tension only —

1.4 for top reinforcement

1.33 for "all lightweight" concrete

1.18 for "sand-lightweight" concrete

0.8 for bar spacing 6" or more (3" from member face)

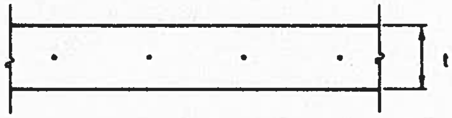
For tension or compression —

$\dfrac{A_{s\,req'd}}{A_{s\,provided}}$ when A_s is greater than required

0.75 for reinforcement in spirals

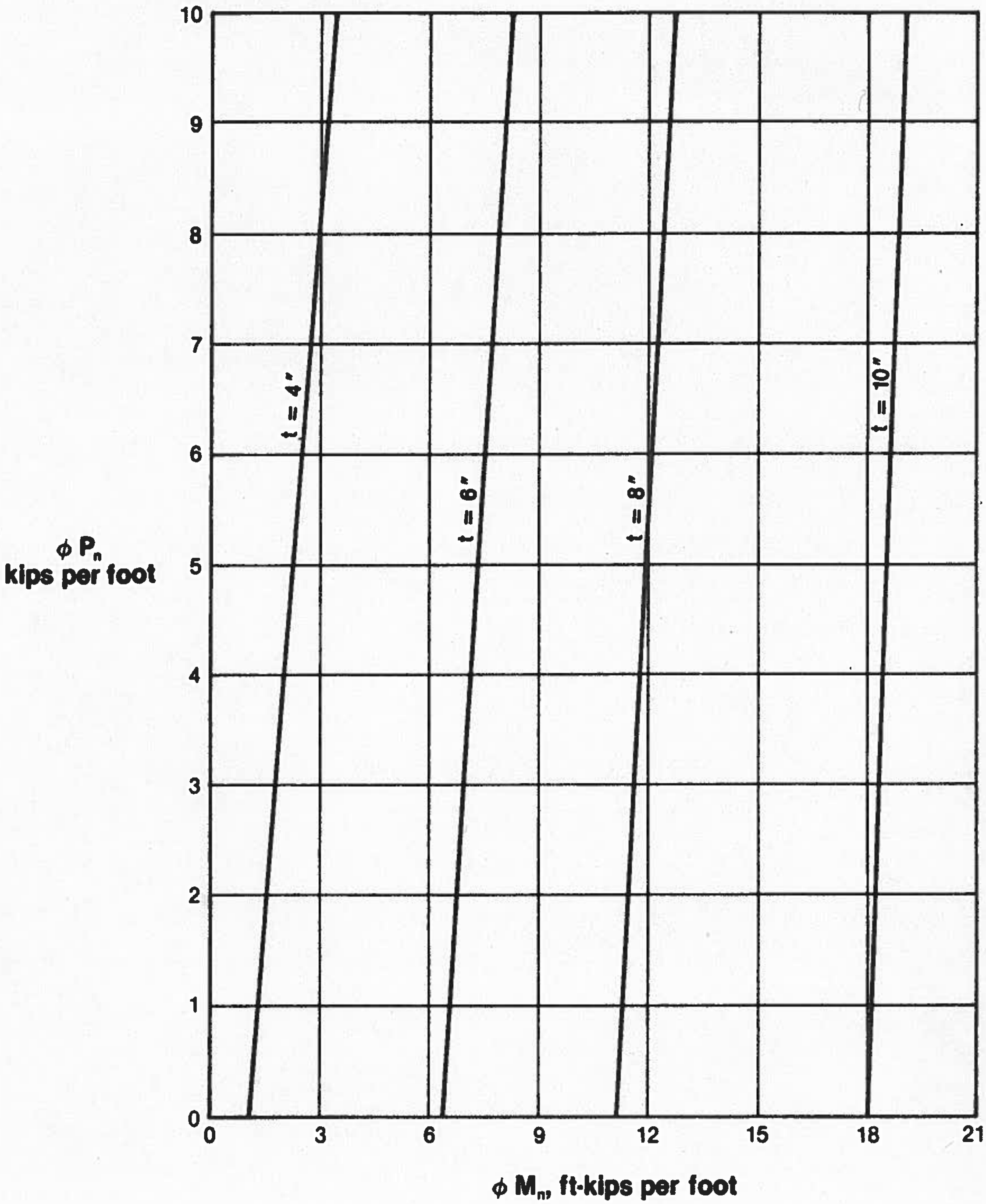
For explanation of some of the PCI Design Handbook material, see "Background and Discussion on PCI Design Handbook Second Edition" by Leslie D. Martin, PCI JOURNAL, January-February 1980, pp.24-41. Reprints available from Prestressed Concrete Institute at $2.50 per copy.

Fig. 2.6.5 Partial interaction curve for prestressed solid wall panels



Curves based on minimum prestress of 250 psi
$f'_c = 5000$ psi
$f'_{pu} = 270$ ksi

t in.	Full interaction curve data ϕP_o	ϕP_{nb}	ϕM_{nb}	ϕM_o
4	140	64	6.2	1.9
6	205	90	13.8	6.4
8	273	121	25.0	11.3
10	341	152	39.4	17.5



$M_d = 0.418(70)^2\ (12/8) = 3072$ in.-kips
$M_{sd} = 0.080(70)^2\ (12/8) = 588$ in.-kips
$M_\ell = 0.280(70)^2\ (12/8) = 2058$ in.-kips

at 0.4ℓ:

$M_d = 3072 \times 0.96 = 2949$ in.-kips
$M_{sd} = 588 \times 0.96 = 564$ in.-kips

$M_\ell = 2058 \times 0.96 = 1976$ in.-kips

Allow $12\sqrt{f'_c}$ final tension

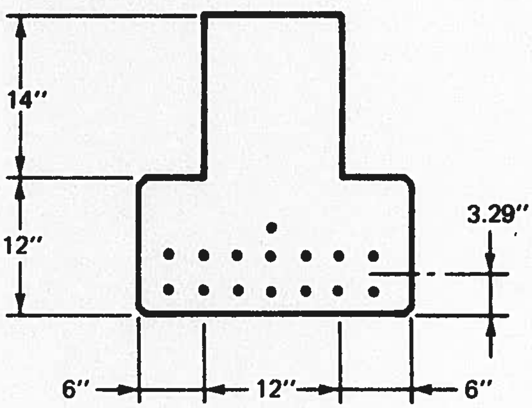
In this example, a release strength of $f'_{ci} = \dfrac{2457}{0.6} =$ 4095 psi should be provided. Also deflection should be checked.

Load	Support at Release $P = P_o$		Midspan at Release $P = P_o$		0.4ℓ at Service load $P = P$	
	f_b	f_t	f_b	f_t	f_b	f_t
P/A	+ 908	+908	+ 908	+ 908	+ 768	+ 768
Pe/Z	+1276	−510	+4059	−1622	+ 2964	−1185
M_d/Z			−2510	+1003	− 2409	+ 963
M_{sd}/Z					− 461	+ 184
M_ℓ/Z					− 1614	+ 645
Stresses	+2184	+398	+2457	+289	− 752	+1375
Allowable Stresses	$0.60\ f'_{ci}$	$0.60\ f'_{ci}$	$0.60\ f'_{ci}$	$0.60\ f'_{ci}$	$12\sqrt{f'_c}$	$0.45\ f'_c$
	+2100	+2100	2100	2100	− 848	2250
	HIGH	OK	HIGH	OK	OK	OK

Example 3.2.9 — Use of Fig. 3.9.8 — Tensile force to be resisted by top reinforcement

Given:

Span = 24 ft
24IT26 as shown



Concrete:

$f'_c = 6000$ psi
$f'_{ci} = 4000$ psi

Prestressing steel

15-1/2" 270K strands

$A_{ps} = 15\ (0.153) = 2.295$ sq in.

Section properties:

A = 456 sq in.
I = 24, 132 in.^{4}
y_b = 10.79 in.
y_t = 15.21 in.
Z_b = 2237 in.^{3}
Z_t = 1587 in.^{3}
wt = 475 plf
e = 7.5 in.

Problem:

Find critical service load stresses

Solution:

Prestress force:

$P_i = 2.295\ (189) = 434$ kips
P_o = (Assume 10% initial loss)
$= 0.90\ (434) = 391$ kips

Moment due to member weight:

at midspan

$M_d = 0.475\ (24)^2\ (12/8) = 410$ in.-kips

at 50 strand diameters (2.08 ft) (transfer point)

$$M_d = \frac{wx}{2}(\ell - x) = \frac{0.475\ (2.08)}{2}$$
$$(24 - 2.08)\ (12) = 130 \text{ in.-kips}$$

(stresses are tabulated on p. 3—14)

Since the tensile stress is high, reinforcement is required to resist the total tensile force.

For 65% fixed, $k_m = 0.40 + 0.65 (0.46 - 0.40) = 0.44$

$M_u = k_m P_u e = 0.44 (121.3) = 53.4$ ft-kips

Maximum restraining force at level 2

$= k_f P_u e / h_s$

$k_f = -0.60 + 0.65 (-0.60 + 0.22) = -0.35$

$F_u = -0.35 (121.3) / 16 = -2.65$ kips (tension)

4.6.6 Slenderness Effects in Compression Members

4.6.6.1 Approximate Evaluation of Slenderness Effects

ACI 318-77 permits an approximate evaluation of slenderness in Section 10.11. Application of this section of the Code for members braced against sidesway is shown in Example 4.6.5, and for unbraced frames in Section 4.6.7. A more rigorous approach, which meets Section 10.10.1 of the Code is discussed briefly in Section 4.6.6.2.

The effective length factor, k, can be determined from the alignment charts, Fig. 4.6.8. For column bases, the value of ψ for use in these charts can be calculated from the rotational stiffness coefficients described in Section 4.6.2, with ψ base $= K_c/K_b$. For most structures, ψ base should not be taken less than 1.0. For column bases which are assumed pinned in the frame analysis, ψ base can be assumed equal to 10 when using Fig. 4.6.8.

Example 4.6.5 Moment magnifier for a column in a braced frame

Given:

The interior column of Example 4.6.4

Column size = 14 x 14 in.

E_c = 4700 ksi

P_u = 624 kips

h_s = 16 ft

Braced frame

Base stiffness coefficient, $K_b = 10.0 \times 10^8$

Problem:

Determine moment magnifier

Solution:

$$K_c = \frac{4E_cI_c}{h_s} = \frac{4\,(4.7 \times 10^6)\,(3201)}{(16 \times 12)}$$
$$= 3.13 \times 10^8$$

From Fig. 4.6.8:

$$\psi_A = \frac{3.13}{10} = 0.31 \text{ use min. of } 1.0$$

$\psi_B = \infty$ (pinned connection)

$k = 0.87$

Slenderness may be neglected when $k\ell_u/r$ is less than $34 - 12M_1/M_2$

$r = 0.3 \times 14 = 4.2$

$k\ell_u/r = 0.87\,(16 \times 12) / 4.2 = 39.8$

From Table 4.6.5:

M_1 = Moment at the base

$= (\% \text{ fixity})\, k_m P_u e$

$= 0.65\,(0.38)\,(39.7) = 9.8$ ft-kips

M_2 = 29.4 ft-kips (see Example 4.6.4)

Note: M_1 and M_2 are opposite direction, therefore:

$M_1/M_2 = -(9.8/29.4) = -0.33$

$34 - 12\,(-0.33) = 38.0 < 39.8$

Therefore slenderness must be considered

$$\beta_d = \frac{\text{Factored dead load moment}}{\text{Factored total load moment}}$$
$$\approx \frac{70\,(14)}{104\,(14)} = 0.67$$

Note: β_d is a factor that takes into account creep due to sustained loads. When the moment to be magnified is caused by short-term loads, such as wind or earthquake, β_d may be considered equal to zero.

Using Eq. 10-10 of ACI 318-77:

$$EI = (E_cI_g/2.5)/(1 + \beta_d)$$
$$= 4700\,(3201)/(2.5 \times 1.67)$$
$$= 3.60 \times 10^6 \text{ k - in}^2.$$

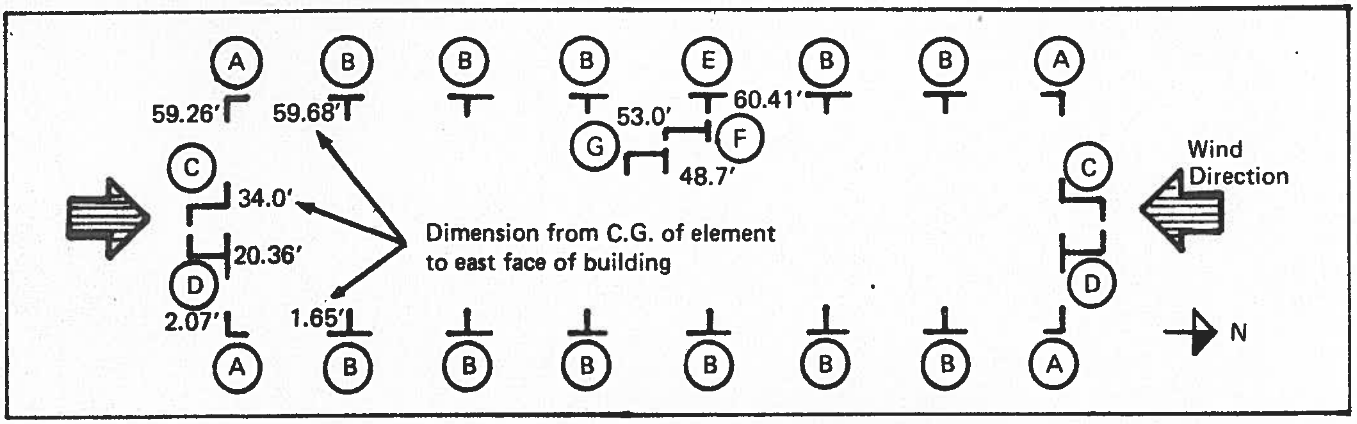
$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2\,(3.60 \times 10^6)}{(0.87 \times 16 \times 12)^2}$$
$$= 1273 \text{ kips}$$

$$C_m = 0.6 + 0.4\,M_1/M_2 = 0.6 + 0.4\,(-0.33)$$
$$= 0.47$$

$$\delta = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} = \frac{0.47}{1 - \dfrac{624}{0.7\,(1273)}} = 1.57$$

Fig. 4.6.7 could also be used for this example.

Fig. 4.7.10 Wind resisting elements for north-south wind



$$= \frac{h_s}{E}\left[\frac{3}{A_w} + \frac{h_s^2}{12I}\right] \quad h_s = 8.0 \text{ ft*}$$

The relative stiffness coefficient for this problem is:

$$\frac{1}{K_r} = \frac{3}{A_w} + \frac{5.33}{I}$$

For element A:

$$\frac{1}{K_r} = \frac{3}{3.11} + \frac{5.33}{12.58} = 1.39$$

$$K_r = \frac{1}{1.39} = 0.72$$

% Distribution to element A (see Table 4.7.1)

$$= \frac{0.72\,(100)}{29.90} = 2.41\%$$

The shears and moments in the north-south direction are shown in Fig. 4.7.11, and the distributions are shown in Table 4.7.2.

To check overturning, consider element B at the first floor. From Fig. 4.7.9 the dead load on the 6′-4″ portion of element B is 1.92 + 3 (2.72) + 0.8 = 10.88 kips/ft. The dead load on the 8′-0″ portion of element B is the weight of the wall = 34.67 x 0.1 = 3.47 kips/ft. The resisting moment is then:

$$\begin{aligned} M_R &= 10.88\,(5.67)\,(4) + 3.47\,(8)\,(4) \\ &= 358 \text{ ft-kips} \times 11 \text{ elements} \\ &= 3938 \text{ ft-kips} \end{aligned}$$

Factor of safety = 3938/966.9 = 4.1 > 1.5 OK

(Note: This conservatively neglects the contribution of the other elements.)

To check for tension, also consider element B:

Total dead weight on the wall

$$\begin{aligned} &= 10.88\,(5.67) + 3.47\,(8) \\ &= 89.45 \text{ kips} \end{aligned}$$

Total wall area

$$\begin{aligned} &= (8.0 + 5.67)\,0.67 \\ &= 9.16 \text{ ft}^2 \end{aligned}$$

$M = 43.3$ ft-kips (see Table 4.7.2)

$$f_{ut} = \frac{1.3M\,(d/2)}{I} - \frac{0.9P}{A}$$

$$= \frac{1.3\,(43.3)\,(4.0)}{28.7} - \frac{0.9\,(89.45)}{9.16}$$

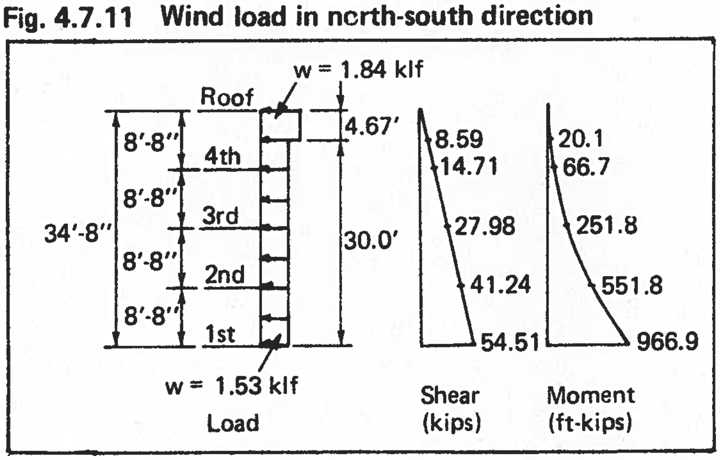
$$= -0.94 \text{ ksf (compression)}$$

No tension connections are required between panels and the foundation. Thus the building is stable under wind loads in the north-south direction.

The connections required to assure that the elements will act in a composite manner as assumed can be designed by considering element A. The unit stress at the interface is determined using the classic equation for horizontal shear:

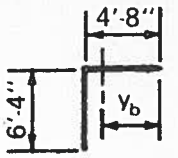
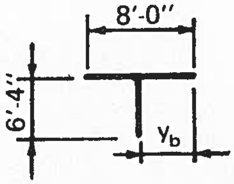
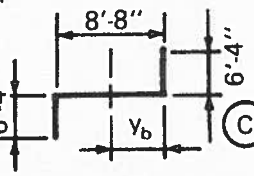
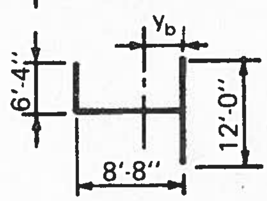
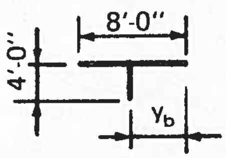
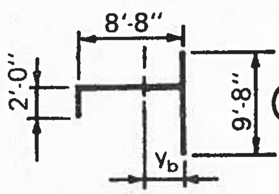
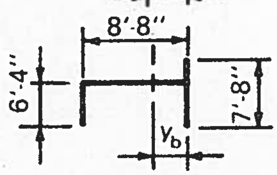
$$v_h = \frac{VQ}{I}$$

Fig. 4.7.11 Wind load in ncrth-south direction



*Either the clear height or the story height can be used to calculate the relative stiffness.

Table 4.7.1 Properties of resisting elements for wind in longitudinal direction

Element	A_w	I	y_b	K_r	No. of elements	nK_r	$\frac{K_r}{\Sigma nK_r}(100)$	$\Sigma\bar{y}$	$K_r(\Sigma\bar{y})$
 A	3.11	12.6	3.43	0.72	4	2.88	2.41	123	89
 B	5.36	28.7	4.0	1.34	11	14.74	4.48	308	413
 C	5.81	158.1	4.34	1.82	2	3.64	6.09	68	124
 D	5.81	205.6	3.45	1.84	2	3.68	6.15	41	75
 E	5.36	29.0	4.0	1.35	1	1.35	4.52	60	81
 F	5.81	114.1	2.72	1.78	1	1.78	5.95	53	94
 G	5.81	171.6	4.09	1.83	1	1.83	6.12	49	90
						$\Sigma_n K_r = 29.90$			$\Sigma = 966$

Center of rigidity = 966 / 29.90 = 32.31 ft from east
Note: The north-south wind load is slightly eccentric by 32.31 – 61.33/2 = 1.65 ft. Torsion due to this eccentricity is neglected in calculating shears and moments in Table 4.7.2.

$$Q = 5.67\,(0.67)\,(1.24 - 0.33) = 3.46\ \text{ft}^3$$

$$v_h = \frac{VQ}{I} = \frac{1.31 \times 3.46}{12.6} = 0.36\ \text{kips/ft}$$

Total shear = 0.36 x 8.0 = 2.88 kips

Connections similar to those shown in Fig. 4.7.8 can be designed using the principles outlined in Part 5 of this Handbook.

Design of floor diaphragm:

Analysis procedures for the floor diaphragm are described in Section 4.5. For this example refer to Fig. 4.7.12:

The factored wind load for a typical floor is:

$$W_u = 1.3 \times 25\ \text{psf} \times 8.67\ \text{ft} = 282\ \text{plf}$$

For wind from the east or west:

$$V_{Ru} = \frac{0.282 \times 26}{2} = 3.67\ \text{kips}$$

Table 4.7.2 Distribution of wind shears and moments (north-south direction)

Element	% Dist.	4th floor Shear 14.71 kips	4th floor Moment 66.7 ft-kips	3rd floor Shear 27.98 kips	3rd floor Moment 251.7 ft-kips	2nd floor Shear 41.24 kips	2nd floor Moment 551.8 ft-kips	1st floor Shear 54.51 kips	1st floor Moment 966.9 ft-kips
A	2.41	0.35	1.6	0.67	6.1	0.99	13.3	1.31	23.3
B	4.48	0.66	3.0	1.25	11.3	1.85	24.7	2.44	43.3
C	6.09	0.90	4.1	1.70	15.3	2.51	33.6	3.32	58.9
D	6.15	0.90	4.1	1.72	15.5	2.54	33.9	3.35	59.5
E	4.52	0.66	3.0	1.26	11.4	1.86	24.9	2.46	43.7
F	5.95	0.88	4.0	1.66	15.0	2.45	32.8	3.24	57.5
G	6.12	0.90	4.1	1.71	15.4	2.52	33.8	3.34	59.2

The relative stiffness and percent distribution for the elements in this table are assumed the same for all stories. The exact values may be slightly different for each story because the values change due to the reduced flange width (see Fig. 4.7.2).

$$C_u = T_u = \frac{M_u}{\ell} = \frac{0.282\,(26)^2}{8\,(56.67)}$$

$= 0.42$ kips

The reaction V_{Ru} is transferred to the shear wall by static friction:

Dead load of floor = 26/2 (64 + 10) x 60 = 57,720

Dead load of wall = 800/2(54) = 21,600

79,320

Static coefficient of friction from Table 5.5.1 (hardboard to concrete) = 0.5. Reduce by factor of 5 as recommended in Sect. 5.5.

$\mu = 0.5/5 = 0.10$

Resisting force = 0.10 (79.3)
= 7.93 > 3.67 OK

The chord tension, T_u, is resisted by the tensile strength of the floor slab. The grout key between slabs must also resist approximately the same force.

Assume area of exterior slab = 218 in.², and the grout key is 3 in. deep. Concrete f'_c = 5000 psi. Use a resisting tensile strength of $3\phi\sqrt{f'_c}$ = 191 psi. Grout key resisting strength is 40 psi (see Sect. 4.5.2).

Resisting tensile strength of slab
= 218 (0.191) = 41.6 kips > 0.67 OK

Resisting strength of grout key
= 26 (12) (3) (0.040)
= 37.4 kips > 0.42 OK

For wind from the north or south:

$$V_{Ru} = \frac{0.282\,(61.33)}{2} = 8.65 \text{ kips}$$

Resisting force in the first joint
= 181.33 (12) (3) (0.04) = 261 kips OK

$$C_u = T_u = \frac{0.282\,(61.33)^2}{8\,(181.33)}$$

= 0.73 kips < 7.93 OK

In this example, only the resistance to wind loading was analyzed. Any other required loading (including "abnormal" loads) must be reviewed for a complete analysis.

Fig. 4.7.12 Diaphragm analysis