ASDIP Concrete

STRUCTURAL ENGINEERING SOFTWARE

Sway Column Example

PCA Notes on ACI 318

Example 11.2—Slenderness Effects for Columns in a Sway Frame

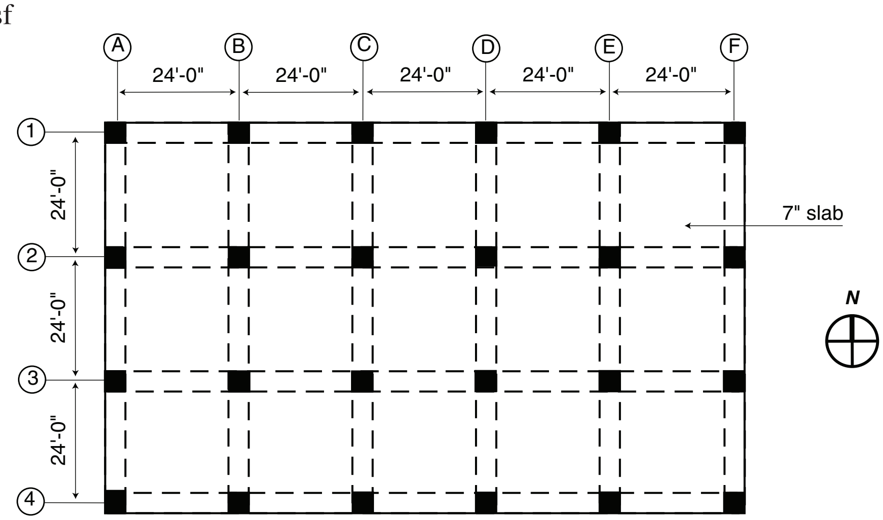
Design columns C1 and C2 in the first story of the 12-story office building shown below. The clear height of the first story is 13 ft-4 in., and is 10 ft-4 in. for all of the other stories. Assume that the lateral load effects on the building are caused by wind, and that the dead loads are the only sustained loads. Other pertinent design data for the building are as follows:

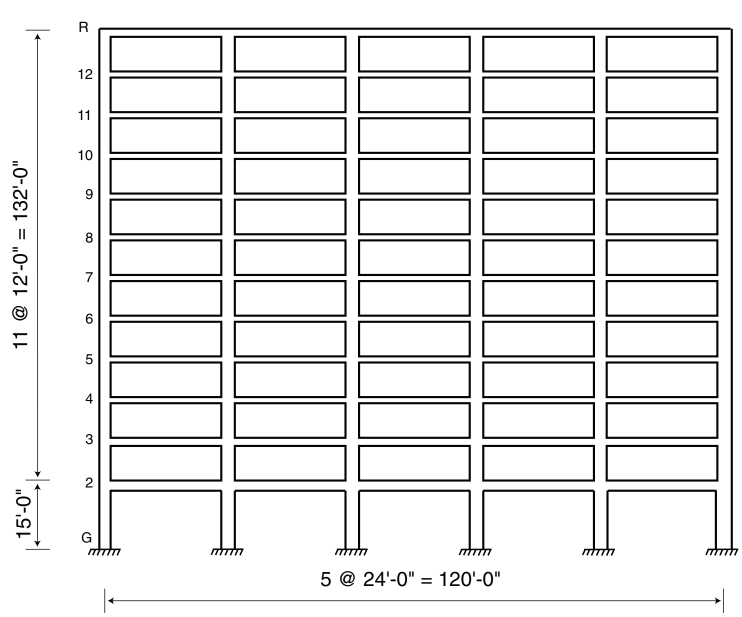
Material properties:

Concrete: = 6000 psi for columns in the bottom two stories (w_c = 150 pcf)
= 4000 psi elsewhere (w_c = 150 pcf)
Reinforcement: f_y = 60 ksi

Beams: 24 × 20 in.
Exterior columns: 22 × 22 in.
Interior columns: 24 × 24 in.

Superimposed dead load = 30 psf
Roof live load = 30 psf
Floor live load = 50 psf
Wind loads computed according to ASCE 7





1. Factored axial loads and bending moments for columns C1 and C2 in the first story

Since this is a symmetrical frame, the gravity loads will not cause appreciable sidesway.

Column C1

	No.	Load Case / Load Combination	Axial Load (kips)	Bending moment (ft-kips) Top	Bottom	M_1	M_2	M_{1ns}	M_{2ns}	M_{1s}	M_{2s}
		Dead (D)	622.4	34.8	17.6						
		Live (L)*	73.9	15.4	7.7						
		Roof live load (L_r)	8.6	0.0	0.0						
		Wind (W) (N-S)	-48.3	17.1	138.0						
		Wind (W) (S-N)	48.3	-17.1	-138.0						
9-1	1	1.4D	871.4	48.7	24.6	24.6	48.7	24.6	48.7	– – –	– – –
9-2	2	1.2D + 1.6L + 0.5L_r	869.4	66.4	33.4	33.4	66.4	33.4	66.4	– – –	– – –
9-3	3	1.2D + 0.5L + 1.6L_r	797.6	49.5	25.0	25.0	49.5	25.0	49.5	– – –	– – –
	4	1.2D + 1.6L_r + 0.8W	722.0	55.4	131.5	55.4	131.5	41.8	21.1	13.7	110.4
	5	1.2D + 1.6L_r - 0.8W	799.3	28.1	-89.3	28.1	-89.3	41.8	21.1	-13.7	-110.4
9-4	6	1.2D + 0.5L + 0.5L_r + 1.6W	710.9	76.8	245.8	76.8	245.8	49.5	25.0	27.4	220.8
	7	1.2D + 0.5L + 0.5L_r - 1.6W	865.4	22.1	-195.8	22.1	-195.8	49.5	25.0	-27.4	-220.8
9-6	8	0.9D + 1.6W	482.9	58.7	236.6	58.7	236.6	31.3	15.8	27.4	220.8
	9	0.9D - 1.6W	637.4	4.0	-205.0	4.0	-205.0	31.3	15.8	-27.4	-220.8

*includes live load reduction per ASCE 7

Column C2

	No.	Load Case / Load Combination	Axial Load (kips)	Bending moment (ft-kips) Top	Bottom	M_1	M_2	M_{1ns}	M_{2ns}	M_{1s}	M_{2s}
		Dead (D)	1,087.6	-2.0	-1.0						
		Live (L)*	134.5	-15.6	-7.8						
		Roof live load $(L)_r$	17.3	0.0	0.0						
		Wind (W) (N-S)	-0.3	43.5	205.0						
		Wind (W) (S-N)	0.3	-43.5	-205.0						
9-1	1	1.4D	1,522.6	-2.8	-1.4	-1.4	-2.8	-1.4	-2.8	– – –	– – –
9-2	2	1.2D + 1.6L + 0.5L_r	1,529.0	-27.4	-13.7	-13.7	-27.4	-13.7	-27.4	– – –	– – –
9-3	3	1.2D + 0.5L + 1.6L_r	1,400.1	-10.2	-5.1	-5.1	-10.2	-5.1	-10.2	– – –	0.0
	4	1.2D + 1.6L_r+ 0.8W	1,332.6	32.4	162.8	32.4	162.8	-2.4	-1.2	34.8	164.0
	5	1.2D + 1.6L_r - 0.8W	1,333.0	-37.2	-165.2	-37.2	-165.2	-2.4	-1.2	-34.8	-164.0
9-4	6	1.2D + 0.5L + 0.5L_r + 1.6W	1,380.5	59.4	322.9	59.4	322.9	-10.2	-5.1	69.6	328.0
	7	1.2D + 0.5L + 0.5L_r - 1.6W	1,381.5	-79.8	-333.1	-79.8	-333.1	-10.2	-5.1	-69.6	-328.0
9-6	8	0.9D + 1.6W	978.4	67.8	327.1	67.8	327.1	-1.8	-0.9	69.6	328.0
	9	0.9D - 1.6W	979.3	-71.4	-328.9	-71.4	-328.9	-1.8	-0.9	-69.6	-328.0

*includes live load reduction per ASCE 7

2. Determine if the frame at the first story is nonsway or sway

The results from an elastic first-order analysis using the section properties prescribed in 10.10.4.1 are as follows:

ΣP_u = total vertical load in the first story corresponding to the lateral loading case for which ΣP_u is greatest

The total building loads are: D = 17,895 kips, L = 1991 kips, Lr = 270 kips. The maximum ΣP_u is from Eq. (9-4):

ΣP_u = (1.2 3 17,895) + (0.5 3 1991) + (0.5 3 270) + 0 = 22,605 kips

V_{us} = factored story shear in the first story corresponding to the wind loads

= 1.6 3 302.6 = 484.2 kips — *Eq. (9-4), (9-6)*

Δ_o = first-order relative deflection between the top and bottom of the first story due to V_u
= 1.6 × (0.28 – 0) = 0.45 in.

$$\text{Stability index } Q = \frac{\Sigma P_u \Delta_o}{V_{us} \ell_c} = \frac{22{,}605 \times 0.45}{484.2 \times \left[(15\times12)-(20/2)\right]} = 0.12 > 0.05$$ *Eq. (10-10)*

Since Q > 0.05, the frame at the first story level is considered sway. — *10.10.5.2*

3. Design of column C1

a. Determine if slenderness effects must be considered.

Determine k from alignment chart in R10.12.1.

$$I_{col} = 0.7 \left(\frac{22^4}{12}\right) = 13{,}665 \text{ in.4}$$ *10.10.4.1*

$$E_c = 57{,}000 \frac{\sqrt{6000}}{1000} = 4{,}415 \text{ ksi}$$ *8.5.1*

For the column below level 2:

$$\frac{E_c I}{\ell_c} = \frac{4{,}415 \times 13{,}665}{(15 \times 12)-10} = 355 \times 10^3 \text{ in.-kips}$$

For the column above level 2:

$$\frac{E_c I}{\ell_c} = \frac{4{,}415 \times 13{,}665}{12 \times 12} = 419 \times 10^3 \text{ in.-kips}$$

$$I_{beam} = 0.35 \left(\frac{24 \times 20^3}{12}\right) = 5{,}600 \text{ in.}^4$$ *10.10.4.1*

For the beam: $$\frac{E_c I}{\ell_c} = \frac{57\sqrt{4{,}000}\times 5{,}600}{24\times12} = 70 \times 10^3 \text{ in.-kips}$$

$$\psi_A = \frac{\Sigma E_c I/\ell_c}{\Sigma E_c I/\ell} = \frac{355+419}{70} = 11.1$$

Assume ψ_B = 1.0 (column essentially fixed at base)

From the alignment chart (Fig. R10.10.1.1(b)), k = 1.9.

$$\frac{k\ell_u}{r} = \frac{1.9 \times 13.33 \times 12}{0.3 \times 22} = 46 > 22$$ 10.10.1

Thus, slenderness effects must be considered.

b. Determine total moment M_2 (including slenderness effects) and the design load combinations, using the approximate analysis of 10.10.7.

The following table summarizes magnified moment computations for column C1 for all load combinations, followed by detailed calculations for combinations no. 4 and 5 to illustrate the procedure.

No.	Load Combination	ΣP_u (kips)	Δ_o (in.)	V_{us} (kips)	Q	δ_s	M_{2ns} (ft-kips)	M_{2s} (ft-kips)	M_2 (ft-kips)
1	1.4D	25,053					48.7		48.7
2	$1.2D+1.6L+0.5L_r$	24,795					66.4		66.4
3	$1.2D+0.5L+1.6L_r$	22,903					49.5		49.5
4	$1.2D+1.6L_r+0.8W$	21,908	0.28	302.6	0.12	1.14	21.1	110.4	147.0
5	$1.2D+1.6L_r-0.8W$	21,908	0.28	302.6	0.12	1.14	21.1	-110.4	-104.8
6	$1.2D+0.5L+0.5L_r+1.6W$	22,605	0.28	484.2	0.08	1.08	25.0	220.8	264.2
7	$1.2D+0.5L+0.5L_r-1.6W$	22,605	0.45	484.2	0.12	1.14	25.0	-220.8	-226.8
8	0.9D+1.6W	16,106	0.45	484.2	0.09	1.10	15.8	220.8	257.9
9	0.9D-1.6W	16,106	0.45	484.2	0.09	1.10	15.8	-220.8	-226.2

$$M_2 = M_{2ns} + \delta_s M_{2s}$$ Eq. (10-18)

$$\delta_s M_{2s} = \frac{M_{2s}}{I - Q} \geq M_{2s}$$ Eq. (10-20)

For load combinations no. 4 and 5:

$U = 1.2D + 1.6L_r \pm 0.8W$

$\Sigma P_u = (1.2 \times 17{,}895) + (1.6 \times 270) \pm 0 = 21{,}906$ kips

$\Delta_o = 0.8 \times (0.28\text{-}0) = 0.22$ in.

$V_{us} = 0.8 \times 302.6 = 240.1$ kips

$\ell_c = (15 \times 12) - (20/2) = 170$ in.

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us}\ell_c} = \frac{21{,}906 \times 0.22}{240.1 \times 170} = 0.12$$

$$\delta_s = \frac{1}{1-Q} = \frac{1}{1-0.12} = 1.14$$

- For sidesway from north to south (load combination no. 4):

  $\delta_s M_{2s} = 1.14 \times 110.4 = 125.9$ ft-kips

  $M_2 = M_{2ns} + \delta_s M_{2s} = 21.1 + 125.9 = 147.0$ ft-kips

  $P_u = 722.0$ kips

- For sidesway from south to north (load combination no. 5):

  $M_{2s} = 0.8 \times 138.0 = 110.4$ ft-kips

  $M_{2su} = 1.2 \times 17.6 + 1.6 \times 0 = 21.1$ ft-kips

  $\delta_s M_{2s} = 1.14 \times (-110.4) = -125.9$ ft-kips

  $M_2 = 21.1 - 125.9 = -104.8$ ft-kips

  $P_u = 799.3$ kips

c. For comparison purposes, recompute $\delta_s M_{2s}$ using the magnified moment method outlined in 10.10.7.4

$$\delta_s M_{2s} = \frac{M_{2S}}{1 - \dfrac{\Sigma P_u}{0.75 \Sigma Pc}} = M_{2s}$$ *Eq. (10-21)*

The critical load P_c is calculated from Eq. (10-13) using k from 10.10.7.2 and EI from Eq. (10-14) or (10-15). Since the reinforcement is not known as of yet, use Eq. (10-15) to determine EI.

For each of the 12 exterior columns along column lines 1 and 4 (i.e., the columns with one beam framing into them in the direction of analysis), k was determined in part 3(a) above to be 1.9.

$$EI = \frac{0.4 E_c I}{1+\beta_{dns}} = \frac{0.4 \times 4415 \times 22^4}{12(1+0)} = 34.5 \times 10^6 \text{ in.}^2\text{-kips}$$ *Eq. (10-15)*

$\beta_{ds} = 0$ *10.10.4.2*

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2 \times 34.5 \times 10^6}{(1.9 \times 13.33 \times 12)^2} = 3{,}686 \text{ kips}$$ *Eq. (10-13)*

For each of the exterior columns A2, A3, F2, and F3, (i.e., the columns with two beams framing into them in the direction of analysis):

$$\psi_A = \frac{355 + 419}{2 \times 70} = 5.5$$

$$\psi_n = 1.0$$

From the alignment chart, k = 1.75.

$$P_c = \frac{\pi^2 \times 34.5 \times 10^6}{(1.75 \times 13.33 \times 12)^2} = 4{,}345 \text{ kips}$$ *Eq. (10-13)*

For each of the 8 interior columns:

$$I_{col} = 0.7\left(\frac{24^4}{12}\right) = 19{,}354 \text{ in.}^4$$ *10.10.4.1*

For the column below level 2:

$$\frac{E_c I}{\ell_c} = \frac{4{,}415 \times 19{,}354}{(15 \times 12) - 10} = 503 \times 10^3 \text{ in.-kips}$$

For the column above level 2:

$$\frac{E_c I}{\ell_c} = \frac{4{,}415 \times 19{,}354}{12 \times 12} = 593 \times 10^3 \text{ in.-kips}$$

$$\psi_A = \frac{503 + 593}{2 \times 70} = 7.8$$

$$\psi_A = 1.0$$

From the alignment chart, k = 1.82.

$$EI = 0.4 \times 4{,}415 \times \frac{24^2}{12} = 48.8 \times 10^6 \text{ in.-kips}$$

Eq. (10-13)

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2} = \frac{\pi^2 \times 48.8 \times 10^6}{(1.82 \times 13.33 \times 12)^2} = 5{,}683 \text{ kips}$$

Therefore,

$$\Sigma P_c = 12(3{,}686) + 4(4{,}345) + 8(5{,}683) = 107{,}076 \text{ kips}$$

The following table summarizes magnified moment computations for column C1 using 10.10.7.4 for all load conditions. The table is followed by detailed calculations for combinations no. 4 and 5 to illustrate the procedure.

No.	Load Combination	ΣP_u (kips)	δ_s (in.)	M_{2ns} (ft-kips)	M_{2s} (ft-kips)	M_2 (ft-kips)
1	1.4D	25,053	– – –	48.7	– – –	48.7
2	1.2D + 1.6L + 1.6L_r	24,795	– – –	66.4	– – –	66.4
3	1.2D + 0.5L + 1.6L_r	22,903	– – –	49.5	– – –	49.5
4	1.2D + 1.6L_r + 0.8W	21,908	1.38	21.1	110.4	173.5
5	1.2D + 1.6L_r - 0.8W	21,908	1.38	21.1	-110.4	-131.3
6	1.2D + 0.5L + 0.5L_r + 1.6W	22,605	1.39	25.0	220.8	331.9
7	1.2D + 0.5L + 0.5L_r - 1.6W	22,605	1.39	25.0	-220.8	-281.9
8	0.9D + 1.6W	16,106	1.25	15.8	220.8	292.0
9	0.9D - 1.6W	16,106	1.25	15.8	-220.8	-260.3

For load combinations No. 4 and 5:

$$U = 1.2D + 1.6L_r \pm 0.8W$$

$$\delta_s = \frac{1}{1 - \dfrac{\Sigma P_u}{0.75 \Sigma P_c}} = \frac{1}{1 - \dfrac{21{,}908}{0.75 \times 107{,}076}} = 1.38$$

- For sidesway from north to south (load combination no. 4):

  $\delta_s M_{2s} = 1.38 \times 110.4 = 152.4$ ft-kips

  $M_2 = 21.1 + 152.4 = 173.5$ ft-kips

  $P_u = 722.0$ kips

- For sidesway from south to north (load combination no. 5):

  $\delta_s M_{2s} = 1.38 \times (-110.4) = -152.4$ ft-kips

  $M_2 = 21.1 - 152.4 = -131.3$ ft-kips

  $P_u = 799.3$ kips

A summary of the magnified moments for column C1 for all load combinations is provided in the following table.

No.	Load Combination	P_u (kips)	10.10.7.3		10.10.7.4	
			δ_s	M_2 (ft-kips)	δ_s	M_2 (ft-kips)
1	1.4D	871.4	– – –	48.7	– – –	48.7
2	1.2D + 1.6L + 0.5L_r	869.4	– – –	66.4	– – –	66.4
3	1.2D + 0.5L + 1.6L_r	797.6	– – –	49.5	– – –	49.5
4	1.2D + 1.6L_r + 0.8W	722.0	1.14	147.0	1.38	173.5
5	1.2D + 1.6L_r - 0.8W	799.3	1.14	-104.8	1.38	-131.3
6	1.2D + 0.5L + 0.5L_r + 1.6W	710.9	1.14	276.7	1.39	331.9
7	1.2D + 0.5L + 0.5L_r - 1.6W	865.4	1.14	-226.8	1.39	-281.9
8	0.9D + 1.6W	482.9	1.10	257.9	1.25	292.0
9	0.9D - 1.6W	637.4	1.10	-226.2	1.25	-260.3

d. Determine required reinforcement.

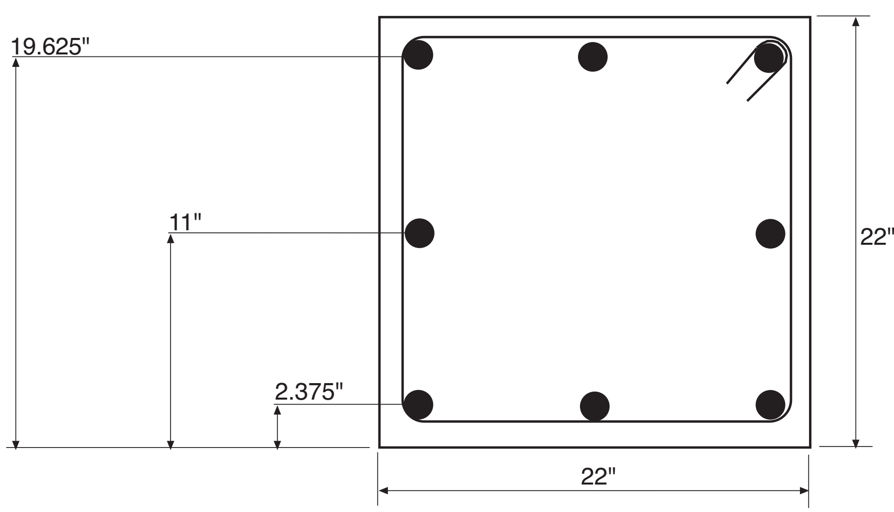
For the 22 × 22 in. column, try 8-No. 8 bars. Determine maximum allowable axial compressive force, $\phi P_{n,max}$:

$$\phi P_{n,max} = 0.80\phi\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$ *Eq. (10-2)*

$$= (0.80 \times 0.65)[(0.85 \times 6)(22^2 - 6.32) + (60 \times 6.32)]$$

$$= 1{,}464.0 \text{ kips} > \text{maximum } P_u = 871.4 \text{ kips} \quad \text{O.K.}$$

The following table contains results from a strain compatibility analysis, where compressive strains are taken as positive (see Parts 6 and 7). Use $M_u = M_2$ from the approximate method in 10.10.7.



No.	P_u (kips)	M_u (ft-kips)	c (in.)	ε_t	ϕ	ϕP_n (kips)	ϕM_n (ft-kips)
1	871.4	48.7	14.85	-0.00096	0.65	871.4	459.4
2	869.4	66.4	14.82	-0.00097	0.65	869.4	459.7
3	797.6	49.5	13.75	-0.00128	0.65	797.6	468.2
4	722.0	147.0	12.75	-0.00162	0.65	722.0	474.1
5	799.3	-104.8	13.78	-0.00127	0.65	799.3	468.0
6	710.9	276.7	12.61	-0.00167	0.65	710.9	474.8
7	865.4	-226.8	14.76	-0.00099	0.65	865.4	460.2
8	482.9	257.9	7.36	-0.00500	0.90	482.9	557.2
9	637.4	-226.2	11.68	-0.00204	0.65	637.4	478.8

Therefore, since $\phi M_n > M_u$ for all $\phi P_n = P_u$, use a 22 × 22 in. column with 8-No. 8 bars (rg = 1.3%). The same reinforcement is also adequate for the load combinations from the magnified moment method of 10.10.7.

4. Design of column C2

a. Determine if slenderness effects must be considered.

In part 3(c), k was determined to be 1.82 for the interior columns. Therefore,

$$\frac{k\ell_u}{r} = \frac{1.82 \times 13.33 \times 12}{0.3 \times 24} = 40.4 > 22$$ 10.10.1

Slenderness effects must be considered.

b. Determine total moment M_2 (including slenderness effects) and the design load combinations, using the approximate analysis of 10.10.7.

The following table summarizes magnified moment computation for column C2 for all load combinations, followed by detailed calculations for combinations no. 4 and 5 to illustrate the procedure.

No.	Load Combination	ΣP_u (kips)	Δ_o (in.)	V_{us} (kips)	Q	δ_s	M_{2ns} (ft-kips)	M_{2s} (ft-kips)	M_2 (ft-kips)
1	1.4D	25,053	-	-	-	-	2.8	-	2.8
2	$1.2D+1.6L+0.5L_r$	24,795	-	-	-	-	27.4	-	27.4
3	$1.2D+0.5L+1.6L_r$	22,903	-	-	-	-	10.2	-	10.2
4	$1.2D+1.6L_r+0.8W$	21,908	0.28	302.6	0.12	1.14	-1.2	164.0	185.0
5	$1.2D+1.6L_r-0.8W$	21,908	0.28	302.6	0.12	1.14	-1.2	-164.0	-187.4
6	$1.2D+0.5L+0.5L_r+1.6W$	22,605	0.45	484.2	0.12	1.14	-5.1	328.0	368.9
7	$1.2D+0.5L+0.5L_r-1.6W$	22,605	0.45	484.2	0.12	1.14	-5.1	-328.0	-379.1
8	0.9D+1.6W	16,106	0.45	484.2	0.09	1.10	-0.9	328.0	358.6
9	0.9D-1.6W	16,106	0.45	484.2	0.09	1.10	-0.9	-328.0	-360.4

$$M_2 = M_{2ns} + M_{2s}$$ Eq. (10-19)

$$\delta_s M_{2s} = \frac{M_{2s}}{I - Q} \geq M_{2s}$$ Eq. (10-20)

For load combinations no. 4 and 5:

$U = 1.2D + 1.6L_r \pm 0.8W$

From part 3(b), δ_s was determined to be 1.14.

- For sidesway from north to south (load combination no. 4):

  $M_{2s} = 0.8 \times 205.0 = 164.0$ ft-kips

  $M_{2ns} = 1.2(-1.0) + 1.6 \times 0 = 1.2$ ft-kips

  $\delta_s M_{2s} = 1.14 \times 164 = 187.0$ ft-kips

  $M_2 = M_{2ns} + \delta_s M_{2s} = -1.2 + 187.0 = 185.8$ ft-kips

  $P_u = 1{,}332.6$ kips

- For sidesway from south to north (load combination no. 5):

  $\delta_s M_{2s} = 1.14 \times (-164) = -187.0$ ft-kips

  $M_2 = -1.2 - 187.0 = -188.2$ ft-kips

  $P_u = 1{,}333.0$ kips

c. For comparison purposes, recompute using the magnified moment method outlined in 10.10.7.4. Use the values of computed in part 3(c).

No.	Load Combination	ΣP_u (kips)	δ_s (in.)	M_{2ns} (ft-kips)	M_{2s} (ft-kips)	M_2 (ft-kips)
1	1.4D	25,053	– – –	-2.8	– – –	-2.8
2	$1.2D + 1.6L + 0.5L_r$	24,795	– – –	-27.4	– – –	-27.4
3	$1.2D + 0.5L + 1.6L_r$	22,903	– – –	-10.2	– – –	-10.2
4	$1.2D + 1.6L_r + 0.8W$	21,908	1.38	-1.2	164.0	225.1
5	$1.2D + 1.6L_r - 0.8W$	21,908	1.38	-1.2	-164.0	-227.5
6	$1.2D + 0.5L + 0.5L_r + 1.6W$	22,605	1.39	-5.1	328.0	451.4
7	$1.2D + 0.5L + 0.5L_r - 1.6W$	22,605	1.39	-5.1	-328.0	-461.6
8	0.9D + 1.6W	16,106	1.25	-0.9	328.0	409.4
9	0.9D - 1.6W	16,106	1.25	-0.9	-328.0	-411.2

$U = 1.2D + 1.6L_r \pm 0.8W$

$\delta_s = 1.38$ from part 3(c)

- For sidesway from north to south (load combination no. 4):

$\delta_s M_{2s} = 1.38 \times 164.0 = 226.3$ ft-kips

$M_2 = -1.2 + 226.3 = 225.1$ ft-kips

$P_u = 1{,}332.6$ kips

- For sidesway from south to north (load combination no. 5):

  $\delta_s M_{2s} = 1.38 \times (-164.0) = -226.3$ ft-kips

  $M_2 = -1.2 - 226.3 = -227.5$ ft-kips

  $P_u = 1{,}333.0$ kips

A summary of the magnified moments for column C2 under all load combinations is provided in the following table.

No.	Load Combination	P_u (kips)	10.10.7.3		10.10.7.4	
			δ_s	M_2 (ft-kips)	δ_s	M_2 (ft-kips)
1	1.4D	1,522.6	– – –	-2.8	– – –	-2.8
2	$1.2D + 1.6L + 0.5L_r$	1,529.0	– – –	-27.4	– – –	-27.4
3	$1.2D + 0.5L + 1.6L_r$	1,400.1	– – –	-10.2	– – –	-10.2
4	$1.2D + 1.6L_r + 0.8W$	1,332.6	1.14	185.8	1.38	225.1
5	$1.2D + 1.6L_r - 0.8W$	1,333.0	1.14	-188.2	1.38	-227.5
6	$1.2D + 0.5L + 0.5L_r + 1.6W$	1,380.5	1.14	368.8	1.39	451.4
7	$1.2D + 0.5L + 0.5L_r - 1.6W$	1,381.5	1.14	-379.0	1.39	-461.6
8	0.9D + 1.6W	978.4	1.10	358.6	1.25	409.4
9	0.9D - 1.6W	979.3	1.10	-360.4	1.25	-411.2

d. Determine required reinforcement.

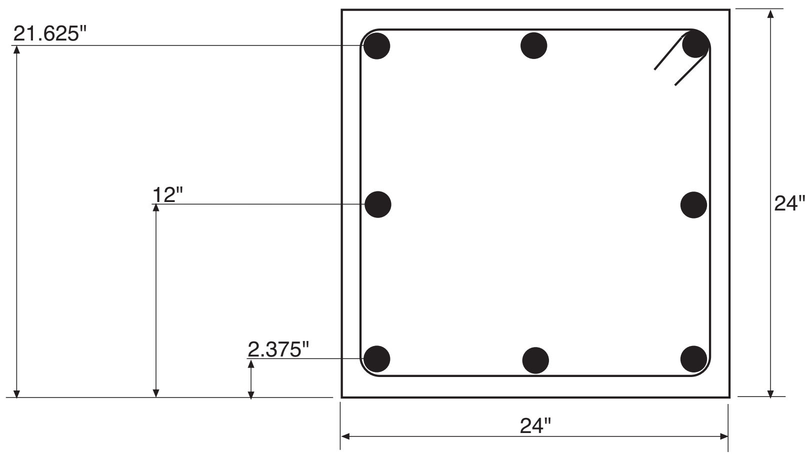
For the 24 × 24 in. column, try 8-No. 8 bars. Determine maximum allowable axial compressive force, $\phi P_{n,max}$:

$$\phi P_{n,max} = 0.80\phi\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad \text{Eq. (10-2)}$$

$$= (0.80 \times 0.65)[(0.85 \times 6)(24^2 - 6.32) + (60 \times 6.32)]$$

$$= 1{,}708 \text{ kips} > \text{maximum } P_u = 1{,}529.0 \text{ kips O.K.}$$

The following table contains results from a strain compatibility analysis, where compressive strains are taken as positive (see Parts 6 and 7). Use $M_u = M_2$ from the approximate method in 10.10.7.



No.	P_u (kips)	M_u (ft-kips)	c (in.)	ε_t	ϕ	ϕP_n (kips)	ϕM_n (ft-kips)
1	1,522.6	-2.8	23.30	0.00022	0.65	1,522.6	438.1
2	1,529.0	-27.4	23.39	0.00023	0.65	1,529.0	435.3
3	1,400.1	-10.2	21.49	-0.00002	0.65	1,400.1	489.7
4	1,332.6	185.8	20.50	-0.00016	0.65	1,332.6	513.3
5	1,333.0	-188.2	20.51	-0.00016	0.65	1,333.0	513.1
6	1,380.5	368.8	21.20	-0.00006	0.65	1,380.5	496.9
7	1,381.5	-379.0	21.22	-0.00005	0.65	1,381.5	496.4
8	978.4	358.6	15.52	-0.00118	0.65	978.4	587.1
9	979.3	-360.4	15.46	-0.00120	0.65	979.3	587.5

Therefore, since $\phi M_n > M_u$ for all $\phi P_n = P_u$, use a 24 × 24 in. column with 8-No. 8 bars (ρ_g = 1.1%).

	Project:	Page # ___
	Engineer:	2/13/2013
	Descrip: Sway Column Example	

ASDIP Concrete 2.0.0 **CONCRETE COLUMN DESIGN** *www.asdipsoft.com*

GEOMETRY

Width .... 22.0 in , Height ... 22.0 in

Use 8-#8 Longitudinal Bars , ρ = 0.013 OK

Use Ties #3 Transverse Reinf. , d' = 2.4 in

Concrete Clear Cover .... 1.5 in

SLENDERNESS

Clear Member Length Lu	13.30	ft
Effective Length Kx-factor	1.90	
Effective Length Ky-factor	1.00	
Lateral Stability	*Sway Column*	

Slenderness Ratio = 48 > Limit = 22, Slender

UNFACTORED LOADS (Elastic First-Order Analysis)

	Dead	Live	RLive	Snow	Wind	Seismic	
Axial Force P	622.4	73.9	8.6	0.0	-48.3	0.0	kip
Mx at Bottom	34.8	15.4	0.0	0.0	17.1	0.0	k-ft
Mx at Top	-17.6	-7.7	0.0	0.0	-138.0	0.0	k-ft
Story Axial Force ΣP ..	17895.0	1991.0	270.0	0.0			kip
Story Shear Force Vsy					302.6	0.0	kip

AMPLIFIED FACTORED LOADS (Sway Column by Approx. 2nd-Order Method)

	Load Combination	Pu (kip)	ΣPu (kip)	M2x (k-ft)	Vusy (kip)	Qy	δsx *	Mcx ** (k-ft)	
①	1.4D	871.4	25053	48.7	0.0	0.000	1.00	48.7	OK
②	1.2D+1.6L+0.5Lr	869.4	24795	66.4	0.0	0.000	1.00	66.4	OK
③	1.2D+1.6L+0.5S	865.1	24660	66.4	0.0	0.000	1.00	66.4	OK
④	1.2D+0.5L+1.6Lr	797.6	22902	49.5	0.0	0.000	1.00	49.5	OK
⑤	1.2D+0.5L+1.6S	783.8	22470	49.5	0.0	0.000	1.00	49.5	OK
⑥	1.2D+1.6Lr+0.8W	722.0	21906	131.5	242.1	0.118	1.13	146.3	OK
⑦	1.2D+1.6S+0.8W	708.2	21474	131.5	242.1	0.116	1.13	146.0	OK
⑧	1.2D+0.5L+0.5Lr+1.6W	710.9	22605	245.8	484.2	0.122	1.14	276.4	OK
⑨	1.2D+0.5L+0.5S+1.6W	706.6	22470	245.8	484.2	0.121	1.14	276.2	OK
⑩	1.2D+0.5L+0.2S+1.0E	783.8	22470	49.5	0.0	0.000	1.00	49.5	OK
⑪	0.9D+1.6W	482.9	16106	236.6	484.2	0.087	1.10	257.6	OK
⑫	0.9D+1.0E	560.2	16106	31.3	0.0	0.000	1.00	31.3	OK

* If δs > 1.5 use the Approx. Magnifier Method

** Mc cannot be greater than 1.4 M2

DESIGN CODES

Concrete Design	ACI 318-11
Load Combinations	ASCE 7-05

MATERIALS

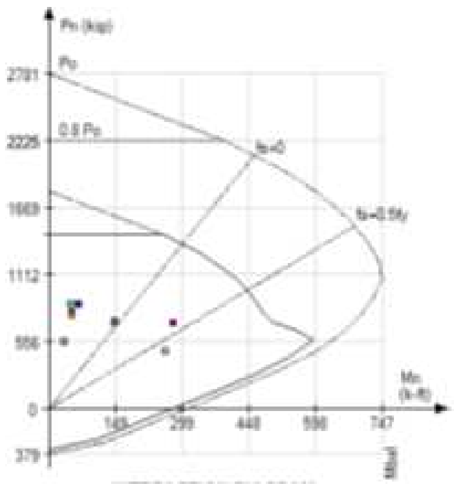
Concrete Strength f'c	6.0	ksi
Rebar Steel Strength fy	60.0	ksi
Compression Strain Limit	0.003	

INTERACTION DIAGRAM

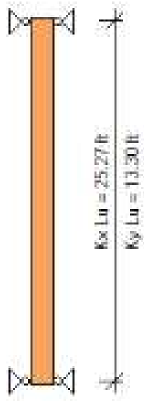
Condition	k = c/d	Steel Fs (kip)	Steel Ms (k-ft)	Conc Fc (kip)	Conc Mc (k-ft)	Pn (kip)	Mn (k-ft)
Pure Compression	Inf.	347.4	0.0	2433.6	0.0	2781.0	0.0
Max. Usable Axial	1.06	197.8	86.0	2027.0	304.5	2224.8	390.5
Zero Steel Stress	1.00	183.3	93.3	1919.4	362.9	2102.6	456.2
Steel Stress = 0.5 fy	0.74	87.8	144.3	1427.2	536.1	1515.0	680.4
Balanced Condition	0.59	-6.6	195.3	1136.0	552.1	1129.4	747.4
Steel Strain = 0.005	0.38	-83.2	193.5	719.6	463.4	636.5	656.9
Pure Bending	0.12	0.0	110.5	0.0	197.3	0.0	307.8
Pure Tension	0.00	379.2	0.0	0.0	0.0	379.2	0.0

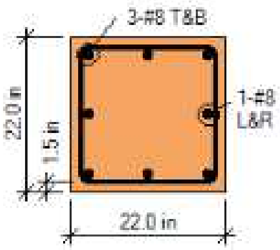
COLUMN STRENGTH

Comb	Pu (kip)	Mux (k-ft)	k = c/d	Steel Fs (kip)	Steel Ms (k-ft)	Conc Fc (kip)	Conc Mc (k-ft)	ϕ Factor	ϕPn (kip)	ϕMn (k-ft)	
①	871.4	48.7	0.67	49.0	165.2	1291.5	551.6	0.65	871.4	465.9	OK
②	869.4	66.4	0.67	48.3	165.6	1289.2	551.7	0.65	869.4	466.2	OK
③	865.1	66.4	0.67	46.7	166.4	1284.2	552.0	0.65	865.1	467.0	OK
④	797.6	49.5	0.63	20.4	180.6	1206.6	554.1	0.65	797.6	477.6	OK
⑤	783.8	49.5	0.62	14.8	183.7	1191.1	554.0	0.65	783.8	479.5	OK
⑥	722.0	146.3	0.57	-11.2	195.3	1090.7	548.8	0.67	722.0	497.6	OK
⑦	708.2	146.0	0.54	-17.1	195.2	1043.3	543.7	0.69	708.2	510.0	OK
⑧	710.9	276.4	0.55	-15.9	195.2	1052.0	544.8	0.69	710.8	507.7	OK
⑨	706.6	276.2	0.54	-17.8	195.2	1037.8	543.0	0.69	706.5	511.4	OK
⑩	783.8	49.5	0.62	14.8	183.7	1191.1	554.0	0.65	783.8	479.5	OK
⑪	482.9	257.6	0.34	-112.6	188.4	649.1	435.3	0.90	482.8	561.3	OK
⑫	560.2	31.3	0.37	-87.1	192.8	709.5	459.5	0.90	560.1	587.1	OK



INTERACTION DIAGRAM





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