ASDIP Concrete

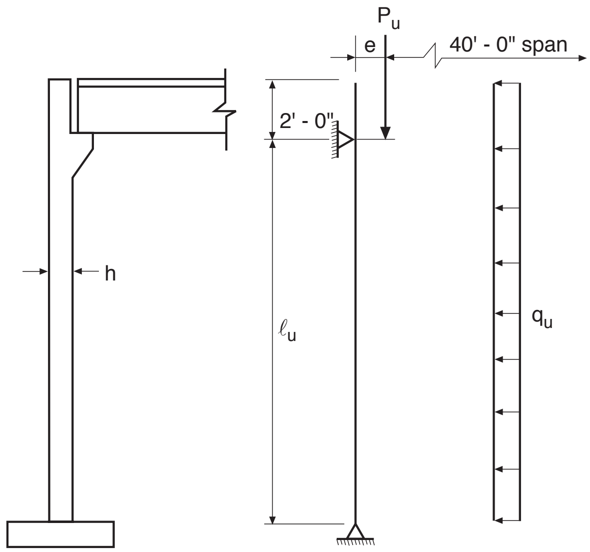
STRUCTURAL ENGINEERING SOFTWARE

Bearing Wall Example

PCA Notes on ACI 318

Example 21.1—Design of Tilt-up Wall Panel by Chapter 10 (14.4)

Design of the wall shown is required. The wall is restrained at the top edge, and the roof load is supported through 4 in. tee stems spaced at 4 ft on center.



Design data:

Roof dead load = 50 psf
Roof live load = 20 psf
Wind load = 20 psf
Unsupported length of wall ℓ_u = 16 ft
Effective length factor k = 1.0 (pinned-pinned end condition)
Concrete f'_c = 4000 psi (w_c = 150 pcf)
Reinforcing steel f_y = 60,000 psi
Assume non-sway condition.

Calculations and Discussion	Code Reference

1. Trial wall selection

   Try h = 6.5 in. with assumed e = 6.75 in.

   Try a single layer of No. 4 @ 12 in. vertical reinforcement (A_s = 0.20 in.^{2}/ft) at centerline of wall

   For a 1-ft wide design strip:

   $$\rho_\ell = \frac{A_s}{bh} = \frac{0.20}{(12 \times 6.5)} = 0.0026 > 0.0012 \quad \text{O.K.}$$ *14.3.2 (a)*

2. Effective wall length for roof reaction *14.2.4*

   Bearing width + 4 (wall thickness) = 4 + 4 (6.5) = 30 in. = 2.5 ft (governs)

   Center-to-center distance between stems = 4 ft

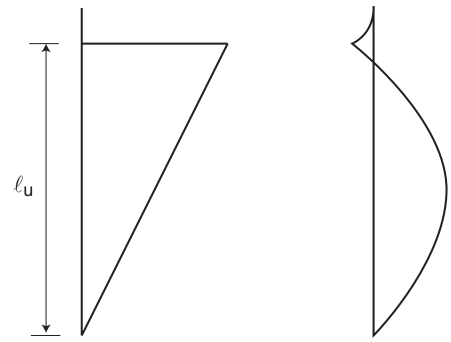
3. Roof loading per foot width of wall

$$\text{Dead load} = \left[50 \times \left(\frac{4}{2.5}\right)\right] \times \frac{40}{2} = 1600 \text{ plf}$$

$$\text{Live load} = \left[20 \times \left(\frac{4}{2.5}\right)\right] \times \frac{40}{2} = 640 \text{ plf}$$

Wall dead load at mid-height

$$= \frac{6.5}{12} \times \left(\frac{16}{2} + 2\right) \times 150 = 813 \text{ plf}$$



Moment Diagrams

4. Factored load combinations

Load comb. 1: $U = 1.2D + 0.5L_r$ *Eq. (9-2)*

$P_u = 1.2\,(1.6 + 0.81) + 0.5\,(0.64) = 2.9 + 0.3 = 3.2$ kips

$M_u = 1.2\,(1.6 \times 6.75) + 0.5\,(0.64 \times 6.75) = 15.1$ in.-kips at top

$\beta_d = 2.9/3.2 = 0.91$

Load comb. 2: $U = 1.2D + 1.6L_r + 0.8W$ *Eq. (9-3)*

$P_u = 1.2\,(1.6 + 0.81) + 1.6\,(0.64) + 0 = 2.9 + 1.0 = 3.9$ kips

$M_u \geq 1.2\,(1.6 \times 6.75)/2 + 1.6\,(0.64 \times 6.75)/2 + 0.8\,(0.02 \times 16^2 \times 12/8)$

$= 16.1$ in.-kips at midspan

$M_u \geq 1.2\,(1.6 \times 6.75) + 1.6\,(0.64 \times 6.75) + 0.8\,(0.02 \times 2^2 \times 12/2)$

$= 19.9$ in.-kips at top

$\beta_d = 2.9/3.9 = 0.74$

Load comb. 3: $U = 1.2D + 1.6W + 0.5L_r$ *Eq. (9-4)*

$P_u = 1.2\,(1.6 + 0.81) + 0 + 0.5\,(0.64) = 3.2$ kips

$M_u \geq 1.2\,(1.6 \times 6.75)/2 + 1.6\,(0.02 \times 16^2 \times 12/8) + 0.5\,(0.64 \times 6.75)/2$

$= 19.8$ in.-kips at midspan

$M_u \geq 1.2\,(1.6 \times 6.75) + 1.6\,(0.02 \times 2^2 \times 12/2) + 0.5\,(0.64 \times 6.75)$

$= 15.9$ in.-kips at midspan

$= 19.8$ in.-kips

$\beta_d = 2.9/3.2 = 0.91$

Load comb. 4: $U = 0.9D + 1.6W$ *Eq. (9-6)*

$P_u = 0.9\,(1.6 + 0.81) + 0 = 2.2$ kips

$M_u \geq 0.9\,(1.6 \times 6.75)/2 + 1.6\,(0.02 \times 16^2 \times 12/8) = 17.1$ in.-kips at midspan

$M_u \geq 0.9\,(1.6 \times 6.75) + 1.6\,(0.02 \times 2^2 \times 12/2) = 10.5$ in.-kips at top

$= 17.1$ in.-kips

$\beta_d = 2.2/2.2 = 1.0$

5. Check wall slenderness

$$\frac{k\ell_u}{r} = \frac{1.0\,(16 \times 12)}{(0.3 \times 6.5)} = 98.5$$

where $r = 0.3h$ — 10.10.1.2

6. Calculate magnified moments for non-sway case — 10.10.6

$$M_c = \delta_{ns} M_2$$ Eq. (10-11)

$$\delta_{ns} = \frac{C_m}{1 - \left(\dfrac{P_u}{0.75P_c}\right)} \geq 1$$ Eq. (10-12)

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2}$$ Eq. (10-13)

$$EI = \frac{E_c I_g}{\beta}\left(0.5 - \frac{e}{h}\right) \geq 0.1\,\frac{E_c I_g}{\beta}$$

$$\leq 0.4\,\frac{E_c I_g}{\beta}$$ Eq. (1)

$$\frac{e}{h} = \frac{6.75}{6.5} = 1.04 > 0.5$$

Thus, $EI = 0.1\left(\dfrac{E_c I_g}{\beta}\right)$

$E_c = 57{,}000\sqrt{4000} = 3.605 \times 10^6$ psi — 8.5.1

$$I_g = \frac{12 \times 6.5^3}{12} = 274.6 \text{ in.}^4$$

$$\beta = 0.9 + 0.5\,\beta^2{}_d - 12\rho \geq 1.0$$

$$= 0.9 + 0.5\,\beta^2{}_d - 12\,(0.0026)$$

$$= 0.869 + 0.5\,\beta^2{}_d \geq 1.0$$

$$EI = \frac{0.1 \times 3.605 \times 10^6 \times 274.6}{\beta} = \frac{99 \times 10^6}{\beta} \text{ lb-in.}^2$$

$$P_c = \frac{\pi^2 \times 99 \times 10^6}{\beta(16 \times 12)^2 \times 1000} = \frac{26.5}{\beta} \text{ kips}$$

C_m = 1.0 for members with transverse loads between supports *10.10.6.4*

Determine magnified moment M_c for each load case.

Load Comb.	P_u (kips)	$M_2 = M_u$ (in.-kips)	β_d	β	EI (lb-in.^{2})	P_c (kips)	δ_{ns}	M_c (in.-kips)
1	3.2	15.1	0.91	1.28	77 x 10^6	20.7	1.26	19.1
2	3.9	19.9	0.74	1.14	87 x 10^6	23.2	1.29	25.7
3	3.2	19.8	0.91	1.28	77 x 10^6	20.7	1.26	25.0
4	2.2	17.1	1.00	1.37	72 x 10^6	19.4	1.18	20.2

Note: M_2 must at least = $P_u(0.6 + 0.03h) = 3.9[0.6 + 0.03\ (6.5)] = 3.1$ in.-kips *10.10.6.5*

7. Check design strength vs. required strength

Assume that the section is tension-controlled for each load combination, i.e., $\varepsilon_t \geq 0.005$ and $\phi = 0.90$. *10.3.4* *9.3.2*

The following table contains a summary of the strain compatibility analysis for each load combination, based on the assumption above:

Load Comb.	$P_n = P_u/\phi$ (kips)	a (in.)	c (in.)	ε_t (in./in.)
1	3.6	0.38	0.44	0.0190
2	4.4	0.39	0.46	0.0180
3	3.6	0.38	0.44	0.0190
4	2.4	0.35	0.47	0.0206

For example, the strain in the reinforcement ε_t is computed for load combination No. 2 as follows:

$$A_{se,w} = A_s + \left(P_n / f_y\right)(h/2d) = 0.20 + \left(\frac{4.3}{60}\right)\left(\frac{6.5}{(2)(3.25)}\right) = 0.27 \text{ in.}^2$$

$$a = \frac{A_{se,w} f_y}{0.85 f'_c \ell_w} = \frac{(0.27)(60)}{0.85(4)(12)}$$

a = 0.39 in.

$c = a/\beta_1 = 0.39/0.85 = 0.46$ in. *10.2.7.1* *10.2.7.3*

$$\varepsilon_t = \frac{0.003}{c}(d - c)$$ *10.2.2*

$$= \frac{0.003}{0.47}(3.25 - 0.47)$$

$= 0.0180 > 0.0050 \rightarrow$ tension-controlled section *10.3.4*

Note that the strain in the reinforcement for each of the load combinations is greater than 0.0050, so that the assumption of tension-controlled sections ($\phi = 0.90$) is correct.

For each load combination, the required nominal strength will be compared to the computed design strength. The results are tabulated below.

Load Comb.	Required Nominal Strength		Design Strength M_n (in.-kips)
	$P_n = P_u/\phi$ (kips)	$M_n = M_c/\phi$ (in.-kips)	
1	3.6	21.2	47
2	4.4	28.5	49
3	3.6	27.8	47
4	2.4	22.4	44

For example, the design strength M_n is computed for load combination No. 2 as follows:

$$M_n = 0.85\, f'_c ba \left(\frac{h}{2} - \frac{a}{2}\right) - A_s f_y \left(\frac{h}{2} - d_t\right)$$

$$= 0.85(4)(12)(0.39)\left(\frac{6.5}{2} - \frac{0.39}{2}\right) - 0.2(60)\left(\frac{6.5}{2} - 3.25\right)$$

$$= 49 \text{ in.-kips}$$

The wall is adequate with the No. 4 @ 12 in. since the design strength is greater than the required nominal strength for all load combinations.

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ASDIP Concrete 2.0.0 — **CONCRETE WALL DESIGN** — www.asdipsoft.com

GEOMETRY

Wall Thickness .... 6.5 in = KLu/30
Use 1-#4 @ 12.0 in Vertical Bars , d' = 3.3 in
Vertical ρ = 0.0026 > ρ min = 0.0012 OK
Use 1-#4 @ 12.0 in Horizontal Bars
Horizontal ρ = 0.0026 > ρ min = 0.0020 OK

SLENDERNESS

Unsupported Height Lu .... 16.00 ft
Effective Height K-factor 1.00
Parapet Height .... 2.00 ft
Lateral Stability .... *Non-sway Wall*
Slenderness Ratio = 102 > Limit = 22, Slender

UNFACTORED LOADS (Elastic First-Order Analysis)

	Dead	Live	RLive	Snow	Wind		Ecc.	Spacing	
Uniform Load w	0.0	0.0	0.0	0.0		k/ft ,	2.0		in
Concentrated P	4.0	0.0	1.6	0.0	0.0	kip ,	6.8	48.0	in
Pressure on Wall					20.0	psf			
Pressure on Parapet ...					20.0	psf			

AMPLIFIED FACTORED LOADS (Non-sway Wall)

	Load Combination	Pu (kip)	M1 (k-in)	M2 (k-in)	M1/ M2	Cm	δns *	Mc (k-in)	
1	1.4D	3.4	7.6	15.2	0.50	1.00	1.30	19.8	OK
2	1.2D+1.6L+0.5Lr	3.2	7.6	15.2	0.50	1.00	1.26	19.2	OK
3	1.2D+1.6L+0.5S	2.9	6.5	13.1	0.50	1.00	1.25	16.3	OK
4	1.2D+0.5L+1.6Lr	3.9	10.0	20.0	0.50	1.00	1.29	25.8	OK
5	1.2D+0.5L+1.6S	2.9	6.5	13.1	0.50	1.00	1.25	16.3	OK
6	1.2D+1.6Lr+0.8W	3.9	16.0	19.6	0.81	1.00	1.29	25.3	OK
7	1.2D+1.6S+0.8W	2.9	12.5	12.7	0.98	1.00	1.25	15.8	OK
8	1.2D+0.5L+0.5Lr+1.6W	3.2	14.5	19.5	0.74	1.00	1.26	24.6	OK
9	1.2D+0.5L+0.5S+1.6W	2.9	12.3	18.4	0.67	1.00	1.25	23.0	OK
10	1.2D+0.5L+0.2S+1.0E	2.9	6.5	13.1	0.50	1.00	1.25	16.3	OK
11	0.9D+1.6W	2.2	9.0	16.8	0.54	1.00	1.18	19.8	OK
12	0.9D+1.0E	2.2	4.9	9.8	0.50	1.00	1.18	11.5	OK

* Per ACI 10.10.2.1, δns cannot be greater than 1.4

DESIGN CODES

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

MATERIALS

Concrete Strength f'c .... 4.0 ksi
Rebar Steel Strength fy ...... 60.0 ksi
Compression Strain Limit 0.003

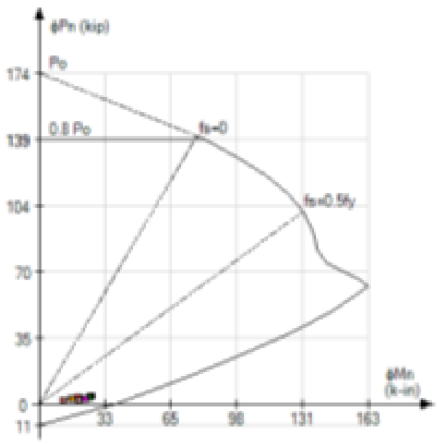
CONCRETE WALL DESIGN

INTERACTION DIAGRAM

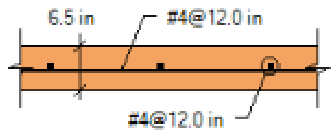
Condition	k = c/d	Steel Fs (kip)	Steel Ms (k-in)	Conc Fc (kip)	Conc Mc (k-in)	Pn (kip)	Mn (k-in)
Pure Compression	Inf.	-11.3	0.0	256.4	0.0	174.0	0.0
Max. Usable Axial	0.99	-0.2	0.0	214.4	126.3	214.2	126.3
Zero Steel Stress	1.00	0.0	0.0	217.2	120.4	217.2	120.4
Steel Stress = 0.5 fy	0.74	-6.0	0.0	161.5	201.2	155.5	201.2
Balanced Condition	0.59	-12.0	0.0	128.5	212.7	116.5	212.7
Steel Strain = 0.005	0.38	-12.0	0.0	81.4	182.3	69.4	182.3
Pure Bending	0.06	0.0	0.0	0.0	42.3	0.0	42.3
Pure Tension	0.00	10.8	0.0	0.0	0.0	10.8	0.0

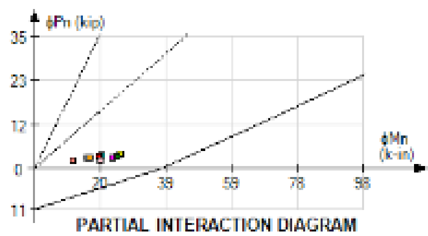
WALL STRENGTH

Comb	Pu (kip)	Mu (k-in)	k = c/d	Steel Fs (kip)	Steel Ms (k-in)	Conc Fc (kip)	Conc Mc (k-in)	ϕ Factor	ϕPn (kip)	ϕMn (k-in)	
1	3.4	19.8	0.08	-13.3	0.0	17.0	51.7	0.90	3.4	46.5	OK
2	3.2	19.2	0.08	-13.3	0.0	16.9	51.2	0.90	3.2	46.1	OK
3	2.9	16.3	0.08	-13.3	0.0	16.5	50.3	0.90	2.9	45.3	OK
4	3.9	25.8	0.08	-13.2	0.0	17.6	53.2	0.90	3.9	47.9	OK
5	2.9	16.3	0.08	-13.3	0.0	16.5	50.3	0.90	2.9	45.3	OK
6	3.9	25.3	0.08	-13.2	0.0	17.6	53.2	0.90	3.9	47.9	OK
7	2.9	15.8	0.08	-13.3	0.0	16.5	50.3	0.90	2.9	45.3	OK
8	3.2	24.6	0.08	-13.3	0.0	16.9	51.2	0.90	3.2	46.1	OK
9	2.9	23.0	0.08	-13.3	0.0	16.5	50.3	0.90	2.9	45.3	OK
10	2.9	16.3	0.08	-13.3	0.0	16.5	50.3	0.90	2.9	45.3	OK
11	2.2	19.8	0.07	-13.4	0.0	15.8	48.3	0.90	2.2	43.5	OK
12	2.2	11.5	0.07	-13.4	0.0	15.8	48.3	0.90	2.2	43.5	OK



INTERACTION DIAGRAM





PARTIAL INTERACTION DIAGRAM