Design of Walls for Axial Load and Out-of-Plane Loads

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Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

2

Course Description

During this session, design of masonry walls loaded with out-of-plane loads and axial loads will be reviewed. Methods to consider secondary bending moments will be examined, including using P-delta provisions, and key points on interaction diagrams will be reviewed. Differences in the strength design provisions and allowable stress design will be briefly discussed.

3

Learning Objectives

- Review the design of walls loaded with out-of-plane with axial loads
- Identify methods to consider secondary bending moment
- Review P-delta provisions for secondary bending moment
- Describe basic differences between allowable stress design and strength design for such walls

4

Determination of Nominal and Design Strength

- Interaction Diagram

Design Assumptions

Strength Design Guide 6.2.3.1; TMS 402 9.3.2

- ε_{mu} = 0.0035 for clay masonry; ε_{mu} = 0.0025 for concrete masonry.
- Reinforcement compression stress does not contribute to strength unless laterally supported according to TMS 402 5.3.1.4.
  - Reinforcement in walls is typically not laterally supported.
- Masonry in tension does not contribute to axial and flexural strength.
- Equivalent rectangular stress block of $0.8f'_m$ over a depth of $0.8c$.

Axial Strength

Strength Design Guide 6.2.2; TMS 402 9.3.4.1.1

$$P_n = 0.80\left[0.80f'_m(A_n - A_{st}) + f_yA_{st}\right]\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{for } \frac{h}{r} \leq 99$$

$$P_n = 0.80[0.80f'_m(A_n - A_{st}) + f_yA_{st}]\left(\frac{70r}{h}\right)^2 \quad \text{for } \frac{h}{r} > 99$$

A_{st} = area of laterally tied steel

7

Interaction Diagrams

Strength Design Guide 6.2.3.2

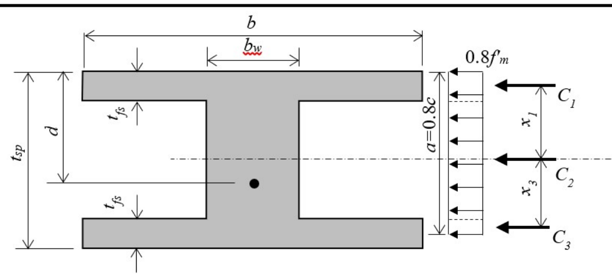
- Assume a value of depth to neutral axis, c.
- Masonry compressive force:
  - For partially grouted walls, the equivalent rectangular stress block will often be in the face shell. Can treat as solid section.
- Reinforcement is often centered, so $d = t_{sp}/2$.
- Wall width is often taken as 1 ft, or the interaction diagram is developed on a per foot basis.
- ϕ = 0.9 for all combinations of flexure and axial load.

8

Interaction Diagram

Strength Design Guide 6.2.3.4

t_{sp} - t_{fs} < 0.8c < t_{sp}

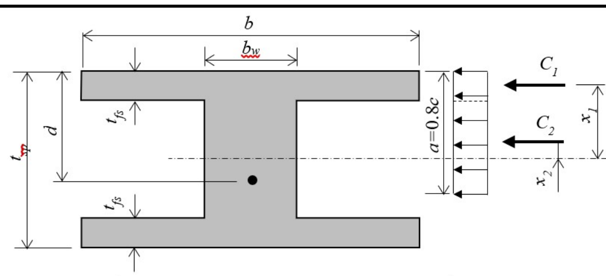


$C_1 = 0.8f'_m b t_{fs}$ $C_2 = 0.8f'_m b_w(t_{sp} - 2t_{fs})$ $C_3 = 0.8f'_m b\left(t_{fs} - (t_{sp} - a)\right)$	$P = C_1 + C_2 + C_3$ $M = C_1 x_1 + C_3 x_3$	$x_1 = \frac{t_{sp}}{2} - \frac{t_{fs}}{2}$ $x_3 = \frac{t_{fs}}{2} - \frac{a}{2}$

Interaction Diagram

Strength Design Guide 6.2.3.4

0.8c ≤ t_{sp} - t_{fs} and c > d

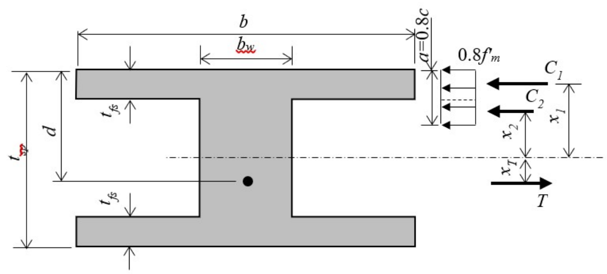


$C_1 = 0.8f'_m b t_{fs}$ $C_2 = 0.8f'_m b_w(a - t_{fs})$	$P = C_1 + C_2$ $M = C_1 x_1 + C_2 x_2$	$x_1 = \frac{t_{sp}}{2} - \frac{t_{fs}}{2}$ $x_2 = \frac{t_{sp}}{2} - \frac{a}{2} - \frac{t_{fs}}{2}$

Interaction Diagram

Strength Design Guide 6.2.3.4

$c \le d$ and $0.8c > t_{fs}$

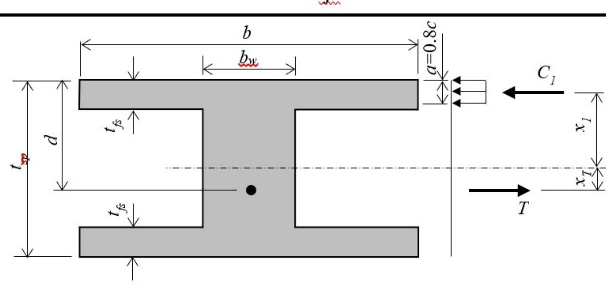


$C_1 = 0.8f'_m b t_{fs}$ $C_2 = 0.8f'_m b_w(a - t_{fs})$ $T = A_s\left[\min\left\{f_y, E_s\varepsilon_{mu}\frac{t_{sp}/2 - c}{c}\right\}\right]$	$P = C_1 + C_2 - T$ $M = C_1x_1 + C_2x_2 + Tx_T$	$x_1 = \frac{t_{sp}}{2} - \frac{t_{fs}}{2}$ $x_2 = \frac{t_{sp}}{2} - \frac{a}{2} - \frac{t_{fs}}{2}$ $x_T = d - \frac{t_{sp}}{2}$

Interaction Diagram

Strength Design Guide 6.2.3.4

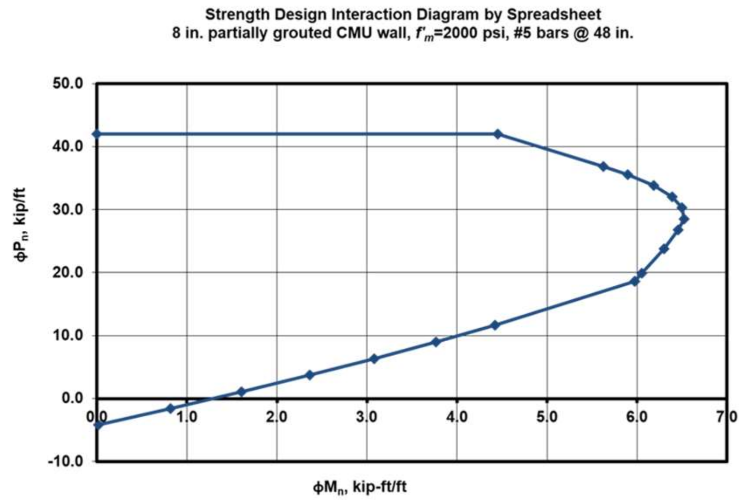
$0.8c \le t_{fs}$



$C_1 = 0.8f'_m ba$ $T = A_s\left[\min\left\{f_y, E_s\varepsilon_{mu}\frac{t_{sp}/2 - c}{c}\right\}\right]$	$P = C_1 - T$ $M = C_1x_1 + Tx_T$	$x_1 = \frac{t_{sp}}{2} - \frac{a}{2}$ $x_T = d - \frac{t_{sp}}{2}$

Interaction Diagram

Strength Design Guide Example 6.2.3.2



13

Interaction Diagram: Below Balanced

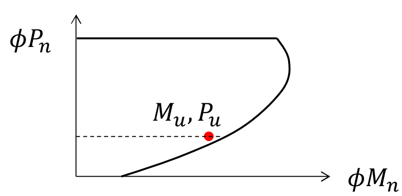
TMS 402 Commentary 9.3.5.2

Depth of stress block, a

$$a = \frac{A_s f_y + P_u/\phi}{0.8 f'_m b}$$

Design moment, ϕM_n

$$\phi M_n = \phi \left(\frac{P_u}{\phi} + A_s f_y\right)\left(d - \frac{a}{2}\right)$$



14

Design

- Estimate Reinforcement
- Maximum Reinforcement

Estimate Wall Thickness and Weight

Strength Design Guide 6.3.3.2

Wall thickness: 8 in. can be used up to ≈ 24 ft in height $(h/t = 36)$

For seismic design, out-of-plane load is function of wall weight

Wall thickness	Partial grout	Full grout
6 inch	35 psf	60 psf
8 inch	45 psf	80 psf
12 inch	65 psf	120 psf

Estimate Reinforcement

Strength Design Guide 6.3.3.2

Centered reinforcement: $A_{s,reqd} \sim \frac{M_u}{0.8 f_y d} - \frac{P_u}{f_y}$

Offset reinforcement: $A_{s,reqd} \sim \frac{M_u}{0.8 f_y d} - \frac{P_u}{2 f_y}$

To account for second-order effects:

- Increase moment by 10% if $h/t \leq 25$
- Increase moment by 20% if $h/t \geq 35$

17

Maximum Reinforcement

Strength Design Guide 6.3.3.4, TMS 402 9.3.3.2

- Strain gradient of ε_{mu} and $\alpha\varepsilon_y$, with α = 1.5 for OOP loading
- P_u determined from $D + 0.75L + 0.525Q_E$ (reduces to just dead load for single story building)

Fully grouted with concentrated tension reinforcement, or partially grouted with neutral axis in face shell

$$\rho = \frac{A_s}{bd} = \frac{0.64 f'_m \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P_u}{bd}}{f_y}$$

Partially grouted walls with concentrated tension reinforcement and neutral axis in web

$$\rho = \frac{0.64 f'_m \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right)\left(\frac{b_w}{b}\right) + 0.8 f'_m t_{fs}\left(\frac{b - b_w}{bd}\right) - \frac{P_u}{bd}}{f_y}$$

18

Maximum Reinforcement

Strength Design Guide Table 6.3.3-6a

Maximum Axial Load from Load Combination $D + 0.75L + 0.525Q_E$ to Meet Maximum Reinforcement Requirements for 8 in. CMU Wall, Centered Grade 60 Reinforcement, f'_m = 2000 psi

Bar Size	Bar spacing					
	8 in.	16 in.	24 in.	32 in.	40 in.	48 in.
No. 4	8.1 kip/ft	16.1 kip/ft	18.7 kip/ft	20.0 kip/ft	20.8 kip/ft	21.4 kip/ft
No. 5		11.1 kip/ft	15.4 kip/ft	17.6 kip/ft	18.8 kip/ft	19.7 kip/ft
No. 6		5.3 kip/ft	11.5 kip/ft	14.6 kip/ft	16.5 kip/ft	17.8 kip/ft
No. 7			6.7 kip/ft	11.0 kip/ft	13.6 kip/ft	15.4 kip/ft

For values not listed, a tension force would be required to meet the maximum reinforcement requirements.

19

Second Order Effects

- Non-Linear Analysis
- Slender Wall Method
- Moment Magnification Method

20

Non-Linear Analysis

Strength Design Guide 6.3.3.3, TMS 402 9.3.5.4.3

- Second-order analysis: typically iterative analysis
- No axial load or h/t limits
- From TMS 402 Equations 9-23 and 9-26

$$I_e = \frac{I_{cr}}{1 - \frac{M_{cr}}{M}\left(1 - \frac{I_{cr}}{I_n}\right)}$$

Slender Wall Method

Strength Design Guide 6.3.3.3, TMS 402 9.3.5.4.2

- Assumes simple support conditions
- Assumes midheight moment is approximately maximum moment
- Assumes uniform load over entire height
- Valid only for the following conditions:
  - $\frac{P_u}{A_n} \leq 0.05 f'_m$ No height limit
  - $\frac{P_u}{A_g} \leq 0.20 f'_m$ Height limited by $\frac{h}{t} \leq 30$

Slender wall method is a valid second-order method, so could be used under TMS 402 9.3.5.4.3 without any limitations.

Slender Wall Method

Strength Design Guide 6.3.3.3, TMS 402 9.3.5.4.2

Moment:

$$M_u = \frac{w_u h^2}{8} + P_{uf}\frac{\varepsilon_u}{2} + P_u \delta_u$$

$$P_u = P_{uw} + P_{uf}$$

P_{uf} = factored floor load

P_{uw} = factored wall load

Deflection:

$$M_u \le M_{cr}$$

$$\delta_u = \frac{5M_u h^2}{48 \;{}_m I_n}$$

$$M_u > M_{cr}$$

$$\delta_u = \frac{5M_{cr} h^2}{48 \;{}_m I_n} + \frac{5(M_u - M_{cr})h^2}{48 E_m I_{cr}}$$

Slender Wall Method

Strength Design Guide 6.3.3.3, TMS 402 9.3.5.4.2

Solve simultaneous linear equations:

$M_u > M_{cr}$

$$M_u = \frac{\frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + \frac{5M_{cr}P_u h^2}{48 \;{}_m}\left(\frac{1}{I_n} - \frac{1}{I_{cr}}\right)}{1 - \frac{5P_u h^2}{48 \;{}_m I_{cr}}}$$

$$\delta_u = \frac{\frac{5h^2}{48 \;{}_m I_{cr}}\left[\frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + M_{cr}\left(\frac{I_{cr}}{I_n} - 1\right)\right]}{1 - \frac{5P_u h^2}{48 \;{}_m I_{cr}}}$$

$M_u \le M_{cr}$

$$M_u = \frac{\frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2}}{1 - \frac{5P_u h^2}{48 \;{}_m I_n}}$$

$$\delta_u = \frac{\frac{5h^2}{48 E_m I_n}\left[\frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2}\right]}{1 - \frac{5P_u h^2}{48 \;{}_m I_n}}$$

Cracking Moment, M_{cr}

Strength Design Guide 6.3.3.3, Table 6.3.3-4; TMS 402 9.3.5.4.2

$$M_{cr} = \frac{(P_u/A_n + f_r)I_n}{t_{sp}/2}$$

Grout Spacing (inch)	Modulus of Rupture (psi)			
	Portland cement/Lime or mortar cement		Masonry cement or air entrained PCL	
	Type M or S	Type N	Type M or S	Type N
Fully Grouted	163	158	153	145
16	124	111	102	88
24	110	95	85	69
32	104	88	77	60
40	100	83	71	54
48	97	80	68	50
Ungrouted	84	64	51	31

Cracked Moment of Inertia, I_{cr}

Strength Design Guide 6.3.3.3, Table 6.3.3-5; TMS 402 9.3.5.4.2

Cracked moment of inertia (fully grouted, or partially grouted wall with neutral axis in face shell):

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d}\right)(d-c)^2 + \frac{bc^3}{3}$$

Modification for non-centered bars;
= 1 for centered bars

Depth to neutral axis: $c = \frac{A_s f_y + P_u}{0.64 f'_m b}$

Moment Magnification Method

Strength Design Guide 6.3.3.3; TMS 402 9.3.5.4.3

Magnified moment: $M_u = \psi M_{u,0}$

Moment magnifier: $\psi = \frac{1}{1-\frac{P_u}{P_e}}$

Buckling load: $P_e = \frac{\pi^2 E_m I_{eff}}{h^2}$

$M_u < M_{cr}$: $I_{eff} = 0.75 I_n$

$M_u \geq M_{cr}$: $I_{eff} = I_{cr}$

Slender Wall Method

First Order Moment

Always Negative

$$M_u = \frac{\frac{w_u h^2}{8} + P_{uf}\frac{e_u}{2} + \frac{5 M_{cr} P_u h^2}{48\, m}\left(\frac{1}{I_n} - \frac{1}{I_{cr}}\right)}{1 - \frac{5 P_u h^2}{48\, m I_{cr}}}$$

$$\frac{5}{48} = 0.104 \sim \frac{1}{\pi^2} = 0.101$$

27

Deflections

Strength Design Guide 6.3.3.3; TMS 402 4.2.2

TMS 402: $\delta_s \leq 0.007h$ under ASD load combinations

IBC 1604.3 Deflection under 0.42 component and cladding wind load (10 yr wind)

- $h/360$ plaster or stucco finishes
- $h/240$ other brittle finishes
- $h/120$ flexible finishes

Compare IBC to TMS:

- TMS deflection limit = $h/143$, or $h/204$ under 10 yr wind
- Walls that meet IBC wind serviceability will generally meet TMS deflection criteria

28

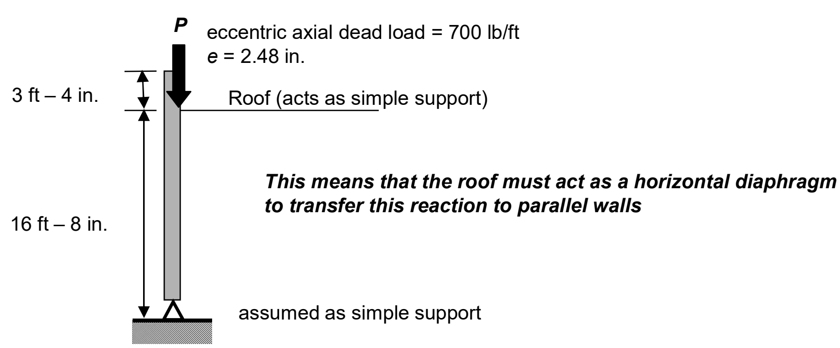
Bearing Wall Example

- Strength Design Guide Example 6.3.3.10

Bearing Wall Design Example

8 in. CMU
Type S masonry cement
Grade 60 steel

Eccentric axial dead load of 700 lb/ft
Eccentric axial roof live load of 300 lb/ft
Out-of-plane wind load of 30 lb/ft^2



Bearing Wall: Design Tips

- Load combination is $0.9D + 1.0W$ typically governs.
- Negative wind pressure typically governs over positive wind pressure.
  - Negative pressure (components and cladding) is generally higher than the positive pressure.
  - Moment from the eccentric axial load is additive with the moment from the negative wind pressure for the typical case of the eccentricity being towards the inside of the building.
- Wind load on parapet will reduce midheight moment.
  - Parapets that are less than 20% of the height of the wall can be neglected when determining the midheight moment, with the impact being less than 8%.
  - Parapet wind load included in example for completeness.

31

Bearing Wall: Estimate Reinforcement

Moment, M_u

$$M_u = \frac{w_u h^2}{8} = \frac{30psf\,(16.67ft)^2 \frac{12\ in.}{ft}}{8} = 12{,}500\,\frac{lb-in.}{ft}$$

Wall weight is estimated as 45 psf

Axial load, P_u

$$P_u = 0.9D = 0.9\left(700\,\frac{lb}{ft} + 45psf\left(3.33ft + \frac{16.67ft}{2}\right)\right) = 1{,}100\,\frac{lb}{ft}$$

Estimate $A_{s,reqd}$

$$A_{s,reqd} \sim \frac{M_u}{0.8 f_y d} - \frac{P_u}{f_y} = \frac{12{,}500\frac{lb-in.}{ft}}{0.8(60{,}000psi)(3.81in.)} - \frac{1{,}100\frac{lb}{ft}}{60{,}000psi} = 0.050\,\frac{in.^2}{ft}$$

Try No. 4 @ 48 in. (0.05 in.^{2}/ft)
actual wall weight = 44 psf

32

Bearing Wall: Properties

Net section properties NCMA TEK 14-1B — A_n = 40.7 in.^{2}/ft; S_n = 87.1 in.^{3}/ft; I_n = 332.0 in.^{4}/ft

Modulus of rupture — Type S masonry cement, 48 inch grout spacing, f_r = 68 psi

Cracking moment — $$M_{cr} = \left(\frac{P_u}{A_n} + f_r\right) S_n = \left(\frac{1{,}100\frac{\text{lb}}{\text{ft}}}{40.7\frac{\text{in.}^2}{\text{ft}}} + 68\text{psi}\right) 87.1\,\frac{\text{in.}^3}{\text{ft}} = 8{,}280\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}}$$

Compressive strength — f'_m = 2000 psi (TMS 602, Table 2)

Modulus of elasticity — $$E_m = 900(f'_m) = 900(2{,}000psi) = 1{,}800{,}000\ psi$$

33

Bearing Wall: I_{cr}

Modular ratio — $$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000\ ps}{1{,}800{,}000\ \text{ksi}} = 16.1$$

Depth to neutral axis — $$c = \frac{A_s f_y + P_u}{0.64 f'_m b} = \frac{0.05\frac{\text{in.}^2}{\text{ft}}(60{,}000\text{ psi}) + 1{,}100\frac{\text{lb}}{\text{ft}}}{0.64(2{,}000\text{ psi})12\frac{\text{in.}}{\text{ft}}} = 0.267\text{ in.}$$

Cracked Moment of Inertia — $$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\right)(d-c)^2 + \frac{bc^3}{3}$$

$$= 16.1\left(0.05\,\frac{\text{in.}^2}{\text{ft}} + \frac{1{,}100\frac{\text{lb}}{\text{ft}}}{60{,}000\text{ psi}}\right)(3.812\text{in.} - 0.267\text{in.})^2 + \frac{12\frac{\text{in.}}{\text{ft}}(0.267\text{in.})^3}{3}$$

$$= 13.9\,\frac{\text{in.}^4}{\text{ft}}$$

34

Bearing Wall: Slender Wall Method

Check applicability of method: Maximum axial load from load combination 1.2D + 1.6L_r

Axial load, P_u $$P_u = 1.2\left(700\frac{lb}{ft} + 44psf\left(3.33ft + \frac{16.67ft}{2}\right)\right) + 1.6\left(700\frac{lb}{ft}\right) = 1{,}940\frac{lb}{ft}$$

$$\frac{h}{t} = \frac{16.67ft}{7.625in.}12\frac{in.}{ft} = 26.2 \leq 30$$ OK, the $0.05f'_m$ stress limitation does not apply.

$$0.20f'_mA_g = 0.20(2{,}000psi)(7.625in.)\left(12\frac{in.}{ft}\right) = 36{,}600\frac{lb}{ft} \geq 1{,}940\frac{lb}{ft}$$ OK, method is applicable.

35

Bearing Wall: Moment at Top

TMS 402 assumes simple span.

Replace $P_{uf}e_u$ with moment at top of wall, M_{uf}

$$M_{uf} = P_{uf}e_u - \frac{w_uh_p^2}{2} = 0.9\left(700\,\frac{\text{lb}}{\text{ft}}\right)(2.48\text{ in.}) - \frac{30\text{ psf}(3.33\text{ft})^2\left(12\frac{\text{in.}}{\text{ft}}\right)}{2} = -437\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}}$$

36

Bearing Wall: Factored Moment

$$M_u = \frac{\frac{w_u h^2}{8} + \frac{M_{uf}}{2} + \frac{5M_{cr}P_u h^2}{48E_m}\left(\frac{1}{I_n} - \frac{1}{I_{cr}}\right)}{1 - \frac{5P_u h^2}{48E_m I_{cr}}}$$

$$= \frac{\frac{30\text{psf}(16.67\text{ft})^2\left(12\frac{\text{in.}}{\text{ft}}\right)}{8} + \frac{-437\frac{\text{lb}\cdot\text{in.}}{\text{ft}}}{2} + \frac{5\left(8{,}260\frac{\text{lb}\cdot\text{in.}}{\text{ft}}\right)\left(1090\frac{\text{lb}}{\text{ft}}\right)(16.67\text{ft})^2\left(12\frac{\text{in.}}{\text{ft}}\right)^2}{48(1{,}800{,}000\text{psi})}\left(\frac{1}{332\frac{\text{in.}^4}{\text{ft}}} - \frac{1}{13.9\frac{\text{in.}^4}{\text{ft}}}\right)}{1 - \frac{5\left(1090\frac{\text{lb}}{\text{ft}}\right)(16.67\text{ft})^2\left(12\frac{\text{in.}}{\text{ft}}\right)}{48(1{,}800{,}000\text{psi})\left(13.9\frac{\text{in.}^4}{\text{ft}}\right)}} = 13{,}300\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}}$$

Moment magnifier method resulted in M_u 14,900 lb-in./ft, or 12% greater

37

Bearing Wall: Check Capacity

Depth of stress block, a

$$a = \frac{A_s f_y + P_u/\phi}{0.8 f'_m b} = \frac{0.05\frac{\text{in.}^2}{\text{ft}}(60{,}000\text{psi}) + 1{,}090\frac{\text{lb}}{\text{ft}}/0.9}{0.8(2{,}000\text{psi})\left(12\frac{\text{in.}}{\text{ft}}\right)} = 0.219\text{in.}$$

Design moment, ϕM_n

$$\phi M_n = \phi\left(\frac{P_u}{\phi} + A_s f_y\right)\left(d - \frac{a}{2}\right)$$

$$= 0.9\left(\frac{1{,}090\frac{\text{lb}}{\text{ft}}}{0.9} + 0.05\,\frac{\text{in.}^2}{\text{ft}}\,(60{,}000\text{pksi})\right)\left(3.812\text{in.} - \frac{0.219\text{in.}}{2}\right)$$

$$= 0.9(15{,}600\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}}) = 14{,}000\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}}$$

Check capacity

$$M_u = 13{,}300\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}} \le 14{,}000\,\frac{\text{lb}\cdot\text{in.}}{\text{ft}} = \phi M_n$$

With factored moment being 95% of the design moment, this is an efficient design.

38

Bearing Wall: Load Combinations

Load Combination	M_u (lb·in./ft)	P_u (lb/ft)	ϕM_n (lb·in./ft)	$M_u / \phi M_n$
0.9D + 1.0W	13,300	1,090	14,000	0.95
1.2D + 1.6L_r +0.5W	7,500	1,940	17,100	0.44
1.2D + 0.5L +1.0W	14,300	1,610	15,900	0.90

Bearing Wall: Maximum Reinforcement

$$P = D + 0.75L + 0.525Q_E = 700\,\frac{lb}{ft}$$

From previous table, maximum axial load for No. 4 @ 48 in. is 21.4 kip/ft

Bearing Wall: Deflections

Deflections are checked using ASD load combinations.
A quick check can be made using SD Load Combinations, TMS 402 Equation 9-26.

$$\delta_u = \frac{5M_{cr}h^2}{48 \quad {}_mI_n} + \frac{5(M_u - M_{cr})h^2}{48E_mI_{cr}}$$

$$= \frac{5\left(8{,}260\frac{lb\cdot i}{ft}\right)(16.67ft)^2\left(12\frac{in.}{ft}\right)^2}{48(1{,}800{,}000psi)\left(332\frac{in.^4}{ft}\right)} + \frac{5\left(13{,}30 \quad \frac{lb\cdot in.}{ft} - 8{,}260\frac{lb\cdot i}{ft}\right)(16.67ft)^2\left(12\frac{in.}{ft}\right)^2}{48(1{,}800{,}000psi)\left(13.9\frac{in.^4}{ft}\right)} = 0.90in.$$

Allowable Deflection $\quad 0.007h = 0.007(16.67ft)12\frac{in.}{ft} = 1.40in.$ $\quad$ OK

When checking deflections, typically the load combination $D + 0.6W$ results in the largest deflection.

Example: Seismic Loads

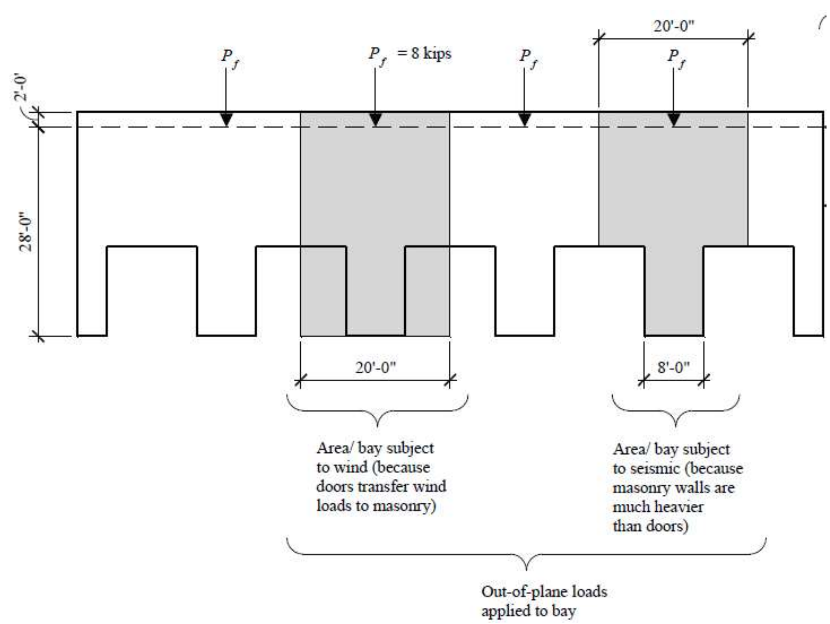
- Strength Design Guide Example 6.3.3.12

Seismic Loads

Warehouse building from FEMA P-1051

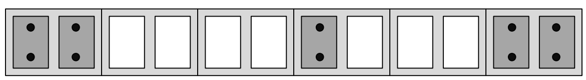
- Type S Portland cement/lime mortar
- f'_m = 2000 psi
- Grade 60 reinforcement
- S_{DS} = 1.43 and I_e = 1.0
- Roof dead load of 400lb/ft at 3.5 in. from inside face of wall
- Because of the height of the wall, 12 in. CMU will be used, resulting in an h/t=28
- Two layers of reinforcement will be used, with a 2 in. cover (1.25 in. face shell, 0.25 in. taper, and 0.5 in. for coarse grout)

After 2015 NEHRP Recommended Seismic Provisions:
Design Examples *FEMA P-1051 / July 2016*



Seismic Loads

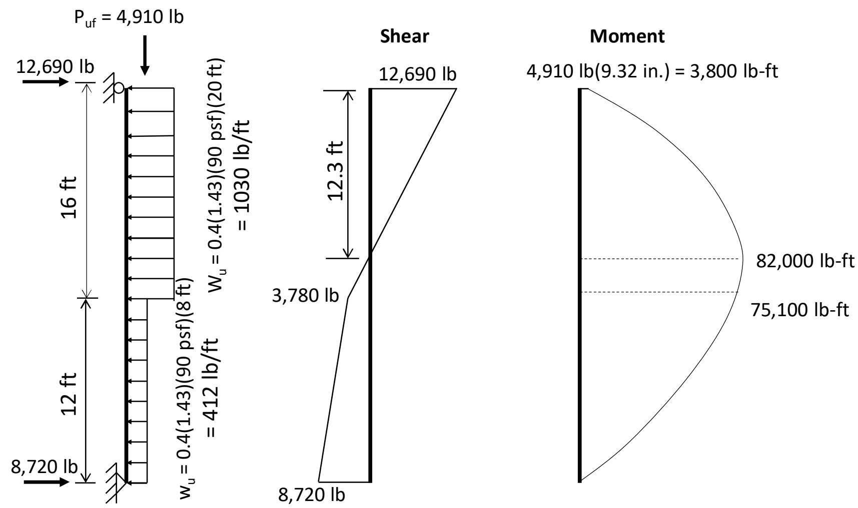
- Due to an anticipated higher level of reinforcement (and hence more grouted cells) and grouted bond beams, assume a wall weight 90 psf.
- The out-of-plane seismic force ASCE 7-16 Section 12.11.1
  - $w_u = 0.4S_{DS}I_e w_{wall} = 0.4(1.43)(1)(2)(90psf) = 51.5psf$
- Estimate the required reinforcement based on a uniform load of 51.5psf(16ft) = 824lb/ft
  - Try a No. 6 bar to determine d (11.625in.-2in.-0.75in./2=9.25in.). Since the axial load is small, ignore the axial load in the estimate of the reinforcement.
  - $A_{s,reqd} \sim \frac{M_u}{0.8f_y d} = \frac{824\frac{lb}{ft}(28ft)^2/8}{0.8(60{,}000psi)\left(9.25in.\frac{1f}{12in.}\right)} = 2.2in.^2$
  - Try 5 - #6 bars ($A_s = 5(0.44in.^2\ ) = 2.2\ in.^2$)



Loads

- Check load combination $0.9D - E_v + E_h$.
- Axial load at the top of the wall:
  - $P_{uf} = (0.9 - 0.2S_{DS})D = \big(0.9 - 0.2(1.43)\big)\left(400\frac{lb}{ft}\right)(20ft) = 4{,}910lb$
- Load, shear, and moment diagrams are shown in the following. Following FEMA 1051, the weight of the overhead doors is neglected. For some types of doors, the weight could be 7-10 psf, which could affect the design.
- Factored axial load at location of maximum moment
  - $P_u = P_{uf} + P_{uw} = 4{,}910lb + \big(0.9 - 0.2(1.43)\big)(90psf)(20ft)(12.3ft) = 18{,}500lb$

Shear and Moment Diagrams



Cracked Moment of Inertia

Depth to neutral axis, c

$$c = \frac{A_s f_y + P_u}{0.64\, f'_m b} = \frac{5(0.44 in.^2)(60{,}000 psi) + 18{,}500 lb}{0.64(2{,}000 psi)(96 in.)} = 1.22 in.$$

c is in face shell

Cracked moment of inertia, I_{cr}

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d}\right)(d-c)^2 + \frac{bc^3}{3}$$

$$= 16.11\left(2.20 in.^2 + \frac{18{,}500 lb}{60{,}000 psi}\frac{11.625 in.}{2(9.25 in.)}\right)(9.25 in. - 1.22 in.)^2 + \frac{96 in.(1.22 in.)^3}{3}$$

$$= 2{,}540 in.^4$$

Magnified Moment

Buckling load, P_e

$$P_e = \frac{\pi^2 E_m I_{eff}}{h^2} = \frac{\pi^2 (1{,}800{,}000 psi)(2{,}540 in.^4)}{\left(28 ft \frac{12 in.}{ft}\right)^2} = 400{,}000 lb$$

Moment magnifier, ψ

$$\psi = \frac{1}{1 - \frac{P_u}{P_e}} = \frac{1}{1 - \frac{18{,}500 lb}{400{,}000 lb}} = 1.05$$

Factored moment, M_u

$$M_u = \psi M_{u,0} = 1.05(82{,}000 lb \cdot ft) = 86{,}000 lb \cdot ft$$

49

Check Capacity

Depth of stress block, a

$$a = \frac{A_s f_y + P_u/\phi}{0.8 f'_m b} = \frac{2.2 in.^2 (60{,}000 psi) + 1\ \ ,500 l\ \ /\ \ .9}{0.8(2{,}000 ps\ \ )(96 in.)} = 0.993 in.$$

Nominal moment, M_n

$$M_n = \left(\frac{P_u}{\phi} + A_s f_y\right)\left(\frac{t_{sp} - a}{2}\right) + A_s f_y \left(d - \frac{t_{sp}}{2}\right)$$
$$= \left(\frac{18{,}500 lb}{0.9} + 2.20 in.^2 (60{,}000 psi)\right)\left(\frac{11.625 in. - 0.993 in.}{2}\right) + 2.20 in.^2 (60{,}000 psi)\left(9.25 in. - \frac{11.625 in.}{2}\right)$$
$$= 105{,}400\ lb \cdot in.$$

If second layer of reinforcement had been included, $M_n = 111{,}300\ lb \cdot in.$, a 6% increase.

Check capacity

$$\phi M_n = 0.9(105{,}400\ lb \cdot ft) = 94{,}800\ lb \cdot ft \quad > \quad M_u = 86{,}000\ lb \cdot ft$$

Check other load combinations

For **1.2D + E_v + E_h**, ψ = 1.12 M_u = 95,100 lb-ft ϕM_n = 105,000 lb-ft

50

Maximum Reinforcement

Strength Design Guide, Example 6.3.3.9: Good Structural Design Tip:

- Maximum axial load > 25 kip/ft with 12 in. CMU and two layers of reinforcement

For pier, maximum axial load > 25kip/ft(8ft) = 200 kip/ft

- Maximum reinforcement requirements met by inspection

Deflections

Quick check of deflections:

- Use ASD load of 0.7(1030 lb/ft) = 721 lb/ft   (OOP load above opening)
- Use cracked moment of inertia of 2540 in.^{4}
- Use moment magnifier of 1.05

$$\delta = \psi \frac{5wh^4}{384EI} = 1.05 \frac{5\left(721\frac{lb}{ft}\right)(28ft)^4 1728\frac{in.^3}{ft^3}}{384(1{,}800{,}000\ psi)(2540\ in.^4)} = 2.30\ in.$$

Allowable deflection: $0.007(28ft)\left(12\frac{in.}{ft}\right) = 2.35in.$

Detailed Calculations in Strength Design Guide

Comparison to ASD

Allowable Stress Design

- No second-order analysis required
- Allowable tension stress controls
  - Wind load: approximately the same reinforcement
  - Seismic load: the 0.7 factor for seismic in ASD causes SD to often require slightly less reinforcement
- Allowable masonry stress controls
  - ASD is inefficient, with SD requiring significantly less reinforcement

ASD vs. SD

- Bearing wall design
  - SD: $M_u/\phi M_n = 0.95$
  - ASD: $M/M_{all} = 0.90$
- Seismic example
  - Ignoring second layer or reinforcement
    - SD: $M_u/\phi M_n = 0.91$
    - ASD: $M/M_{all} = 1.02$
  - Including second layer of reinforcement
    - SD: $M_u/\phi M_n = 0.86$
    - ASD: $M/M_{all} = 1.02$ $(kd = 2.83in. > 2.38in. = d')$

55

This concludes The American Institute of Architects Continuing Education Systems Course

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The Masonry Society