STRUCTURAL WALL LOADS AND ANCHORAGE

OUT-OF-PLANE WALL DESIGN LOADS

ASCE7-10 REF UNO

Exterior Walls: Design for wind loads in addition to seismic loads

Interior Walls: Design for 5 psf minimum live load — IBC 1607.14

Seismic Design Category > A

$F_{Pwall} = 0.4S_{DS}I_eW_p$ — 12.11.1

$\geq 0.10W_p$ — 12.11.1

Design walls for bending between anchors when anchors spaced more than 4 feet apart — 12.11.2.1

OUT-OF-PLANE WALL DESIGN ANCHORAGE LOADS

Seismic Design category A

$F_{Panchor} = 0.2W_p$ — 1.4.5

$\geq 5\ psf$ — 1.4.5

Seismic Design Category > A

$F_{Panchor} = 0.4S_{DS}k_aI_eW_p$ — 12.11-1

$\geq 0.20k_aI_eW_p$ — 12.11.2.1

where

k_a 1.0 for rigid diaphragms — 12.11.2.1

$1.0 + \frac{L_f}{100}$ for flexible diaphragms — 12.11-2

≤ 2.0 — 12.11.2.1

Where Lf is the span in feet of the flexible diaphragm that provides support for the wall

Rigid diaphragms defined by IBC Section 202 as being diaphragms in which the diaphragm deflection is less than or equal to twice the story drift. IBC Section 202 defines flexible diaphragms per ASCE 7-10 Section 12.3.1.1.

If the diaphragm is not flexible and the anchor is not located at the roof, the anchor force may be multiplied by: — 12.11.2.1

$$\frac{1+\frac{2z}{h}}{3}$$

Increase $F_{Panchor}$ by 1.4 for the design of steel elements other than reinforcing steel and anchor bolts — 12.11.2.2.2

Pilasters: Calculate anchorage force based on wall area supported. Does not reduce anchorage forces between pilasters — 12.11.2.2.7

NON-STRUCTURAL WALL LOADS AND ANCHORAGE

OUT-OF-PLANE WALL DESIGN LOADS

ASCE 7-10 REF UNO

Exterior Walls: Design for wind loads in addition to seismic loads

Interior Walls: Design for 5 psf minimum live load — IBC 1607.14

Seismic Design Category > A

$$F_{Pwall} = \frac{0.4 a_p S_{DS} I_e W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right) \quad 13.3\text{-}1$$

$$\le 1.6 S_{DS} I_P W_p \quad 13.3\text{-}2$$

$$\ge 0.3 S_{DS} I_P W_p \quad 13.3\text{-}3$$

where

a_p	1.0 walls supported top and bottom	Table 13.5-1
	2.5 cantilevered walls and parapets	
R_p	2.5	

OUT-OF-PLANE WALL DESIGN ANCHORAGE LOADS

Seismic Design Category > A

$$F_{Pwall} = \frac{0.4 a_p S_{DS} I_e W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right) \quad 13.3\text{-}1$$

$$\le 1.6 S_{DS} I_P W_p \quad 13.3\text{-}2$$

$$\ge 0.3 S_{DS} I_P W_p \quad 13.3\text{-}3$$

where

a_p	1.0 body of wall panel connections	Table 13.5-1
	1.25 fasteners of the connecting system	
R_p	2.5 body of wall panel connections	
	1.0 fasteners of the connecting system	

Anchors in masonry shall be designed to be governed by tensile or shear strength of a ductile steel element or design for 2.5 times the factored load. — 13.4.2.2

Note: If anchors are attached to flexible diaphragm, design forces are in accordance with ASCE 7 Section 12.11.2. See hand-out for structural walls. — Table 13.5-1 Footnote b

EXAMPLE 8: Shear wall design ("Flexure Controlled")

(Neglect "relatively small" Axial Load once verified $f_a << f_b$)

Applied axial loads and wall self dead load are negligible compared to shear and flexural loads. Therefore this is a flexural problem.

Given

12" Solid-grouted CMU Wall $P_r := 0$ $E_{vr} := 0$

$b := 11.625in$

Special Reinforced Shear Wall

M >>> P

Special Inspection Provided

$e > \frac{L}{2}$

$f'_m := 2000psi$ SDC 'D'

$F_s := 32000\,psi$ Grade 60 steel

$h := 14ft$ $S_{DS} := .51$

Typical "beam" problem

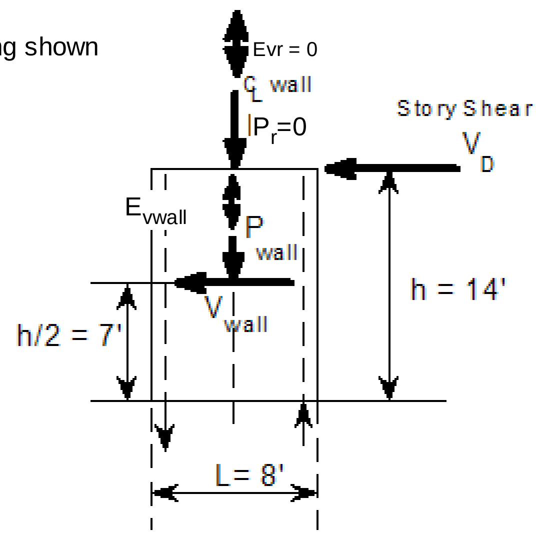
$L_{wall} := 8ft$

"Beam" is vertical cantilever

$V_D := 43.5kip$ ASD story shear from LFRS analysis applied to shear wall from diaphragm shear transfer at roof level.

Required

Design wall for loading shown



2012 IBC 1605.3.1 Load Cases

D+L	(16-8)
D+0.6W	(16-12)
D+0.7E	(16-12)
D+0.525W+0.75 L	(16-13)
D+0.56E+0.75L	(16-14)
0.6D+0.6W	(16-15)
0.6D+0.75E	(16-16)

Plus Snow Load Combinations

$$.P_{wall} = (14ft)\cdot(8ft)\cdot\frac{(124psf)}{1000\frac{lb}{kip}} = 13.9kip$$

Main Force Resisting System

$C_s = S_{DS}/(R/I)$

$V_{wall} := 3kip$ = (Seismic lateral coefficient)(W_{wall})= (Cs)(W)/1.4

From overall LFRS Analysis

R=5 ASCE 7-05 Table 12.2.-1.A.7

$\pm E_v = 0.2(S_{DS})D/1.4$

ASCE 7-05 12.4.2.2. EQN (12.4-4)

$$E_{vwall} = (0.2\cdot0.51)\cdot(13.9kip)\cdot0.7 = \pm1\,kip$$

<u>Design Loads (Sum moments and forces at base of wall centerline)</u>

$M := V_D \cdot h + V_{wall} \cdot \frac{h}{2}$ $\boxed{M = (43.5kip)(14ft) + (3kip)\left(\frac{14ft}{2}\right)}$ $M = 630 \cdot kip \cdot ft$

Mmax Compression

$V := V_D + V_{wall}$ $\boxed{V = (43.5kip) + (3kip)}$ $V = 46.5 \cdot kip$

Total shear

<u>Check C, T, and P Relationship: e=M/P</u>

Max $e := \frac{M}{(0.6\,P_{wall} - E_{vwall})}$ $e = \frac{(630kip \cdot ft) \cdot}{[(0.6))13.9kip - 1.0kip]} = 85.8ft$

*IBC EQN (16-21) 0.9DL+E/1.4

Max $f_a = \frac{(13.9kip + 1.0) \cdot 1000\frac{lb}{kip}}{(11.625in)(8ft) \cdot \left(12\frac{in}{ft}\right)} = 13.33psi$

e is much greater than lwall/2.

flexural controls problem when f_b is much greater than f_a

*D +L+ E (16-20)

*Upward or Downward Ev depends on the check

D + L (16-16)

<u>Trial Flexural Reinforcement</u>

Neglect non-jamb tension reinforcing and use estimate of location for centroid of shear wall "chord" or "jamb" reinforcing

Assume $j := 0.9$

$d := L_{wall} - 16in = \boxed{8ft - 1.33ft = 6.67\,ft} = 80$ in See sketches below.

Assume vertical jamb/chord reinforcing in last four cells of wall.

$A_s := \frac{M}{F_s \cdot j \cdot d}$ $\boxed{A_s = \frac{(630kip \cdot ft)\left(\frac{12in}{ft}\right)}{(32\,ksi)(0.90)(80in)}}$ $A_s = 3.29 \cdot in^2$

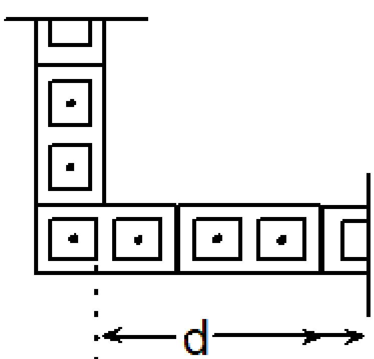
EQN 4 From the formula sheet

Try 8-#6 (2 bars each face in last 4 cells)

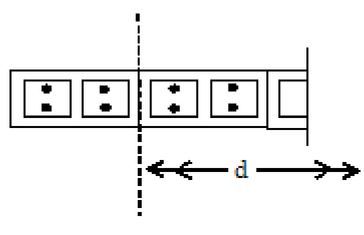
$A_s := 8 \cdot Area_6$ $\boxed{A_s = 8 \cdot \left(0.44in^2\right) = 3.52in^2}$

Jamb reinforcing = Chord reinforcing = Boundary reinforcing

Examples:



or



12" wall at end

'd' can be 88" or 80" as shown above

8" wall at corner

<u>Plan Views</u>

Check Maximum Flexural Compression:

$\rho := \frac{A_s}{b \cdot d}$ $\boxed{\rho = \frac{3.52in^2}{(11.625in)(80in)} = 0.0038}$

$E_s = 2.9 \times 10^4 \cdot ksi$

$E_m := 900 \cdot f'_m$ MSJC 1.8.2.2.1

$n := \frac{E_s}{E_m}$ $\boxed{n = \frac{29000ksi}{(900)(2.0ksi)}}$ $n = 16.1$

$\rho \cdot n = 0.061$

$k := \sqrt{(\rho \cdot n)^2 + 2(\rho \cdot n)} - \rho \cdot n$ $\boxed{k = \sqrt{0.061^2 + 2(0.061)} - 0.061}$ EQN 1 From the formula sheet

$k = 0.29$

$j := 1 - \frac{k}{3}$ $\boxed{j = 1 - \frac{0.33}{3}}$ $j = 0.9$ EQN 2 From the formula sheet

$k \cdot d = (0.29)(80in) = 23.4in < d = 80in$ So, tension reinforcing is active.

Note: If partially grouted, check if kd is in solid grouted portion ∴ not a T-beam

$f_b = \frac{2M}{j \cdot k \cdot b \cdot d^2}$ Flexural EQN 6 From the formula sheet

$\boxed{f_b = \frac{2(630kip \cdot ft)\left(12\frac{in}{ft}\right)\left(1000\frac{lbf}{kip}\right)}{(0.9)(0.29)(11.625in)(80in)^2}}$ $f_b = 779 \cdot psi$

$\boxed{F_b := 0.45\ (2000psi) = 900\ psi}$ $F_b = 900\ psi$ $F_b > f_b$ OK

$f_b <$ F_b Therefore: Flexural Compression OK

Check Tension Stress:

$f_s := \frac{M}{A_s \cdot j \cdot d}$ $\boxed{f_s = \frac{(630kip \cdot ft)\left(12\frac{in}{ft}\right)\left(\frac{1000lbf}{kip}\right)}{\left(3.52in^2\right)(0.9)(80in)}}$ EQN 4 From the formula sheet

$f_s = 29.8 \cdot ksi$

$f_s \le$ F_s $F_s =$ 32 ksi $> f_s$ OK

Note that stresses due to combined compression and flexure need not be checked because the section is controlled by tension stress in the and the small amount of compression present will only be beneficial

Check Maximum Reinforcing Percentage

MSJC 2.3.4.4

Special reinforced shear wall. Yes $\longrightarrow$ Proceed

$$\frac{M}{V \cdot d} \geq 1.0 \qquad \text{Yes} \longrightarrow \text{Proceed}$$

Check if:

$$P > 0.05 \cdot f'm \cdot A_n = (0.05)(2.0ksi)(11.625in)(8ft)\left(12\frac{in}{ft}\right) = 111.6kip$$

solid grouted

$$P = Pwall + Evwall = 13.9kip + 1.0kip = 14.9kip \leq 111.6kip \qquad f_a \leq 0.05 \cdot f'm$$

not heavily loaded

Rho Max Check Not Required. Maximum tension steel is not required to be checked.
But, to demonstrate:

The following is not required but will calculate to demonstrate.

$$\rho_{max} := \frac{n \cdot f'_m}{2 \cdot f_y \cdot \left(n + \frac{f_y}{f'_m}\right)}$$

EQN 2-23

ρ = tension steel in jamb calculation = 8 - #6 = 3.52 in^2

$$\rho = \frac{3.52in^2}{(11.625in)(80in)} = 0.0038$$

$$\rho_{max} = \frac{(16.1)(2.0ksi)}{2(60ksi)\left(16.1 + \frac{60ksi}{2.0ksi}\right)} = 0.0058$$

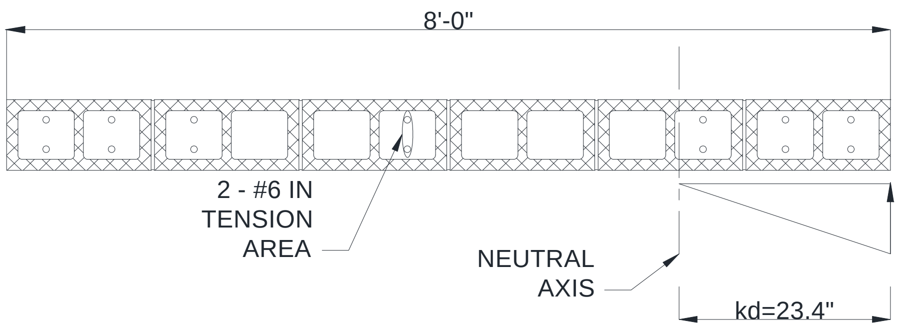
$0.0038 < 0.0058$ OK

Chord Steel:
Not over- reinforced.
BUT:
Must check total of all steel in tension area beyond neutral axis.

BUT, Check all reinforcement in tension zone to neutral axis
=jamb steel and steel in cracked area:
OOP Wall design requires #6 @ 32" EF Need to add 2-#6

$$\frac{3.52in^2 + 0.88in^2}{11.625in \cdot 80in} = 0.0047 < 0.0058$$

therefore total tension reinforcing is adequate



Check Shear

Allowable Shear Stress:

$$\boxed{\frac{M}{V \cdot d} = \frac{(630\text{kip}\cdot\text{ft})\left(12\,\frac{\text{in}}{\text{ft}}\right)}{(46.5\text{kip})(80\text{in})} = 2.03}$$

Note: there is no IBC1.5 factor on V in the equation

MSJC 2.3.6

More accurate check: M/(Vd)

Design shear stress:

$$f_v = \frac{1.5V}{b \cdot h} = \frac{1.5(46.5\text{kips})\left(10^3\right)}{(11.625\text{in})(96\text{in})} = 62.5 \text{ psi}$$

MSJC 1.18.3.2.6.1.2
1.5 factor for Special Reinforced Masonry Shear Walls

For a special reinforced masonry wall, allowable shear stress in masonry is determined by Eq 2-29 (Note that M/Vd need not be taken greater than 1):

Use M/Vd = 1

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] = \frac{1}{4}\left[\left(4.0 - 1.75(1)\right)\sqrt{2000}\right] = 25.2\ psi$$

$F_{vm} < f_v$ Shear reinforcing will be required.

Check whether maximum allowable shear stress is being exceeded. Since M/Vd is1.0, use MSJC Equation 2-27:

$$F_{v.max} = 2\sqrt{f'_m} = 2\sqrt{2000} = 89.4\ psi > f_v = 62.5\ psi \text{ OK}$$

Determine required shear reinforcing

$$F_{vm} + F_{vs} \geq f_v$$

$$F_{vs} = f_v - F_{vm} = 0.5\left(\frac{A_v F_s d}{A_n s}\right)$$

Rearranging terms

$$\frac{A_v}{s} = \frac{2(f_v - F_{vm})A_n}{F_s d} = \frac{2(62.5 - 25.2)(11.625)(96)}{(32{,}000)(80)} = 0.032\frac{in^2}{in}$$

A_v is 0.52 in^2 for a 16” reinforcing spacing. Provide (1) #5 each face.

(2)#5 @ 16” $\frac{A_v}{s} = \frac{(2)(0.31)}{16} = 0.039\frac{in^2}{in}$ OK

Shear Steel Detailing Requirements:

For Seismic Design Category D, masonry shear walls must comply with the requirements of special reinforced masonry shear walls. — MSJC 1.18.3.2.6

(a)* S_v and $S_H \le \frac{1}{3} \cdot L = \left(\frac{1}{3}\right)(8ft)\left(12\frac{in}{ft}\right) = 32in$ — MSJC 1.18.3.2.6(a)

$OR \le \frac{1}{3} \cdot H = \left(\frac{1}{3}\right)(14ft)\left(12\frac{in}{ft}\right) = 56in$ — S_v= vertical bar spacing

$OR \le 48in$ — MSJC 1.18.3.2.6(b)

$S_H = 16in < 32in \qquad S_v \le 32in$ — S_h = horizontal bar spacing

Regardless of O.O.P. wall design, (a) will require vertical bars and dowels to be at maximum 32" o.c.

(b) A_{sv} (from O.O.P. analysis) $\geq \left(\frac{1}{3}\right)\left(A_{sh\_reqd}\right)$

$A_{sh\_reqd} =$ .032 in²/ in = 0.384 in²/ft as calculated above

(Required, Not provided)

Horizontal Reinforcing

$A_{sv} \ge \left(\frac{1}{3}\right)A_{sh} = \left(\frac{1}{3}\right)\frac{0.384\ in^2}{ft} = 0.13\frac{in^2}{ft}$ — MSJC 1.18.3.2.6(c)1

#4 EF @ 32" = (2)(0.2)/ (32/12) = 0.15 in²/ft

Use #4 EF @ 32in as minimum but

O.O.P. wall design will probably require more reinforcing but vertical spacing may not exceed 32" o.c. per (a) above. — MSJC 1.18.3.2.6(c)

After OOP Design Complete
Check:

$$\frac{A_{sh}}{bt} = \frac{2 \cdot 0.31in^2}{(11.625in \cdot 16in)} = 0.0033$$

Min Reinforcing
MSJC 1.18.3.2.6(c)

$$\frac{Asv}{b \cdot t} > 0.0007$$

$$\frac{Asv}{b \cdot t} + \frac{Ash}{b \cdot t} \geq 0.002$$

Already provided by A_{sh} alone

(c) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook:

MSJC 1.18.3.2.6(d)

Standard Hooks:

MSJC 1.16.5
+ code commentary
Fig.1.16-1

(a) 180º

(b) 90º

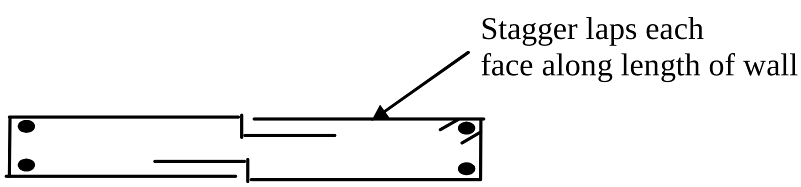
But, lateral tie anchorage shall be either:

(1) 180º
OR

(2) 135º

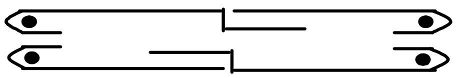
MSJC 1.18.4.4.2.3

Closed ties:



Follow column detailing in compression zone

OR



ETC

EXAMPLE 8: Shear wall design ("Flexure Controlled")

(Neglect "relatively small" Axial Load once verified $f_a << f_b$)

Applied axial loads and wall self dead load are negligible compared to shear and flexural loads. Therefore this is a flexural problem.

Given

12" Solid-grouted CMU Wall $P_r := 0$ $E_{vr} := 0$

$b := 11.625\text{in}$

Special Reinforced Shear Wall

M >>> P

Special Inspection Provided

$e > \frac{L}{2}$

$f'_m := 2000\text{psi}$ SDC 'D'

$F_y = 60{,}000$ psi Grade 60 steel

$h := 14\text{ft}$ $S_{DS} := .51$

Typical "beam" problem

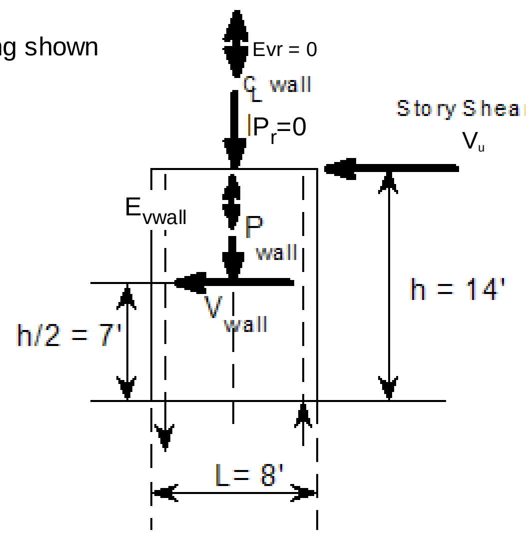
$L_{wall} := 8\text{ft}$

"Beam" is vertical cantilever

$V_u = 61$ kip story shear from LFRS analysis applied to shear wall from diaphragm shear transfer at roof level.

Required

Design wall for loading shown



$P_{wall} = (14\text{ft})\cdot(8\text{ft})\cdot\frac{(124\text{psf})}{1000\frac{\text{lb}}{\text{kip}}} = 13.9\text{kip}$

Main Force Resisting System

$C_s = S_{DS}/(R/I)$

$V_{u,wall} = 4$ kip = (Seismic lateral coefficient)(W_{wall})= (Cs)(W)

From overall LFRS Analysis

R=5 ASCE 7-05 Table 12.2.-1.A.7

$\pm E_v = 0.2(S_{DS})D$

ASCE 7-05 12.4.2.2. EQN (12.4-4)

$E_{vwall} = (0.2\cdot 0.51)\cdot(13.9\text{kip}) = \pm 1\text{ kip}$

STRENGTH DESIGN

Design Loads (Sum moments and forces at base of wall centerline)

$M_u := V_D \cdot h + V_{wall} \cdot \frac{h}{2}$ $\quad M_u = (61\ \text{kip})(14\text{ft}) + (4\,\text{kip})\left(\frac{14\text{ft}}{2}\right)$ $\quad M = 882 \cdot \text{kip} \cdot \text{ft}$

$V_u = V_u + V_{u,wall}$ $\quad V_u = (61\ \text{kip}) + (4\ \text{kip})$ $\quad V = 65 \cdot \text{kip}$

Check C, T, and P Relationship: e=M/P

Max $e := \frac{M_u}{(0.9\,P_{wall} - E_{vwall})}$ $\quad e = \frac{(882\,\text{kip} \cdot \text{ft}) \cdot}{[(0.9))13.9\text{kip} - 1.0\text{kip}]} = 76.6\ \text{ft}$

e is much greater than lwall/2.

flexural controls problem

Trial Flexural Reinforcement Neglect non-jamb tension reinforcing and use estimate of location for centroid of shear wall "chord" or "jamb" reinforcing

Assume vertical jamb/chord reinforcing in last four cells of wall.

Estimate reinforcing, including beneficial effect of compression:

$$A_s = \frac{M_u}{0.9F_y d} - \frac{P_u}{2F_y} = \frac{882(12)}{0.9(60)(80)} - \frac{(12.5)}{(2)60}$$

EQN 101 From the formula sheet

$$A_s = 2.45 - 0.10 = 2.35 in^2$$

This is somewhat unconservative since this estimate does not account for the depth of the compression block.

Therefore, try (6) #6 This will only require three cells of reinforcing; revise d to 84"

Jamb reinforcing = Chord reinforcing = Boundary reinforcing

$$\rho = \frac{A_s}{bd} = \frac{(6)\,(0.44)}{(11.625)(80)} = 0.0028$$

Before proceeding, check ρ_{max} per MSJC Section 3.3.3.5.1. This is applicable when $M_u/V_u d \geq 1$. For a special shear wall, 4 times the yield strain must be able to be achieved:

$$\frac{M_u}{V_u d} = \frac{882(12)}{(65)(80)} = 2.0$$

ρ_{max} must be checked

STRENGTH DESIGN

$$\rho_{max} = \frac{0.64f'_m\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+4\varepsilon_y}\right)}{f_y}$$

MSJC 3.3.3.5.3

$$\rho_{max} = \frac{0.64(2)\left(\frac{0.0025}{0.0025+(4)0.0021}\right)}{60} = 0.049$$

$$A_{s,max} = \rho_{max}bd = (0.049)(11.625)(80) = 4.55\ in^2$$

Note that this must account for all vertical reinforcing that is in tension, plus the effect of the compression load from the D + 0.75L + 0.525QE case.

MSJC 3.3.3.5.1 (d)

For this wall, ρ_{max} is okay by inspection.

CHECK FLEXURAL CAPACITY OF WALL

$$\phi M_n = \phi\rho f_y A_s bd^2\left(1-\frac{0.625\rho f_y}{f'_m}\right)$$

EQN 102

$$\phi M_n = (0.9)(0.0028)(60)(11.625)(80)^2\left(1-\frac{0.625(0.0028)(60)}{2}\right)$$

$$\phi M_n = 10{,}800\ k-in = 900\ k-ft > M_u = 882\ k-ft$$

STRENGTH DESIGN

DESIGN WALL FOR SHEAR

Since this is a special reinforced shear wall, design for 2.5Vu:

For exam, designing for 2.5 Vu is the faster of the two options available for ensuring ductility as required by MSJC 1.18.3.2.6.1.1

$$V_{u,special} = 2.5V_u = 2.5(65) = 162\ kips$$

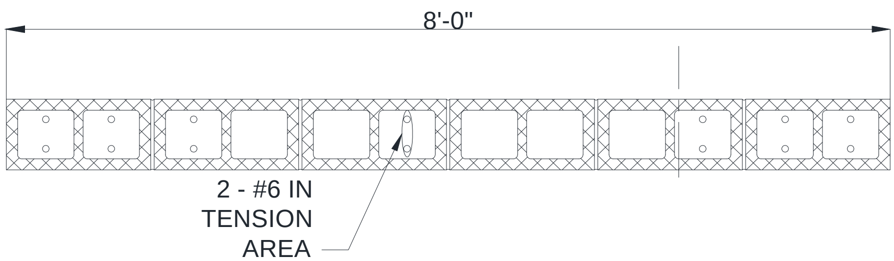
First check to ensure that $\phi V_{n,max}$ is not exceeded. Since $M_u/V_ud > 1$, use MSJC Equation 3-22:

$$\phi V_{n.max} = \phi 4\sqrt{f'_m}A_{nv} = (0.8)4\sqrt{2000}(11.625)(96) = 160 kips$$

In this case, the fast approach didn't work.

Since this is exceeded (slightly), determine the shear associated with 1.25 times the nominal flexural strength. Must consider all vertical reinforcing that could contribute to the flexural strength of the wall; add in (2) #6 at mid length of the wall. This results in d = 76

MSJC 1.18.3.2.6.1.1



$$\rho = \frac{A_s}{bd} = \frac{(8)(0.44)}{(11.625)(76)} = 0.0040$$

$$M_n = (0.0040)(60)(11.625)(76)^2\left(1 - \frac{0.625(0.0040)(60)}{2}\right)$$

$$M_n = 1{,}242\ k - ft$$

$$V_{u,special} = V_u\frac{1.25M_n}{M_u} = 65\frac{1.25(1{,}242)}{882} = 114\ kips$$

Since this is < $\phi V_{n,max}$, the design may proceed.

Determine nominal capacity of masonry, noting that M_u/V_ud need not exceed 1:

$$\phi V_{nm} = \phi\left(4.0 - 1.75\left(\frac{M_u}{V_ud}\right)\right)A_{nv}\sqrt{f'_m}$$

MSJC Eqn 3-23

$$\phi V_{nm} = 0.8\big(4.0 - 1.75(1)\big)(11.625)(96)\sqrt{2000} = 89.8\ kips < V_{u,special} = 133\ kips$$

Determine required shear reinforcing

STRENGTH DESIGN

$\phi V_{nm} + \phi V_{ns} \geq V_u$

$\phi V_{ns} \geq V_u - \phi V_{nm} = \phi 0.5\left(\frac{A_v}{s}\right) f_y d_v$ — MSJC Eqn 3-24

Rearranging terms:

$$\frac{A_v}{s} = \frac{2(V_u - \phi V_{nm})}{\phi f_y d_v} = \frac{2(133 - 89.8)}{(0.8)(60)(96)} = 0.019\frac{in^2}{in}$$

If provide (1) #5 each face, these could be spaced at 32" on center.

(2)#5 @ 32" $\frac{A_v}{s} = \frac{(2)(0.31)}{32} = 0.019\frac{in^2}{in}$ OK

Shear Steel Detailing Requirements:

For Seismic Design Category D, masonry shear walls must comply with the requirements of special reinforced masonry shear walls. — MSJC 1.18.3.2.6

(a)* S_v and $S_H \leq \frac{1}{3}\cdot L = \left(\frac{1}{3}\right)(8ft)\left(12\frac{in}{ft}\right) = 32in$ — MSJC 1.18.3.2.6(a)

$OR \leq \frac{1}{3}\cdot H = \left(\frac{1}{3}\right)(14ft)\left(12\frac{in}{ft}\right) = 56in$ — S_v= vertical bar spacing

$OR \leq 48in$ — MSJC 1.18.3.2.6(b)

$S_H = 32in < 32in \quad S_v \leq 32in$ — S_h = horizontal bar spacing

Regardless of O.O.P. wall design, (a) will require vertical bars and dowels to be at maximum 32" o.c.

(b) A_{sv} (from O.O.P. analysis) $\geq \left(\frac{1}{3}\right)(A_{sh\_reqd})$

$A_{sh\_reqd} = 0.23\ in^2$

(Required, Not provided)

Horizontal Reinforcing

$A_{sv} \geq \left(\frac{1}{3}\right)A_{sh} = \left(\frac{1}{3}\right)\frac{0.23\,in^2}{ft} = 0.08\ \frac{in^2}{ft}$ — MSJC 1.18.3.2.6(c)1

At 32" = 2.67' max spacing, As = 0.21 in²

Use #4 EF @ 32in as minimum but

O.O.P. wall design will probably require more reinforcing but vertical spacing may not exceed 32" o.c. per (a) above. — MSJC 1.18.3.2.6(c)

STRENGTH DESIGN

After OOP Design Complete
Check:

Min Reinforcing
MSJC 1.18.3.2.6(c)

$$\frac{A_{sh}}{bt} = \frac{2 \cdot 0.31 in^2}{(11.625 in\ 32 in)} = 0.0017$$

$$\frac{Asv}{b \cdot t} > 0.0007$$

$$\frac{Asv}{b \cdot t} + \frac{Ash}{b \cdot t} \geq 0.002$$

Will be achieved by minimum vertical reinforcing

(c) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook:

MSJC 1.18.3.2.6(d)

Standard Hooks:

MSJC 1.16.5
+ code commentary Fig.1.16-1

(a) 180º

(b) 90º

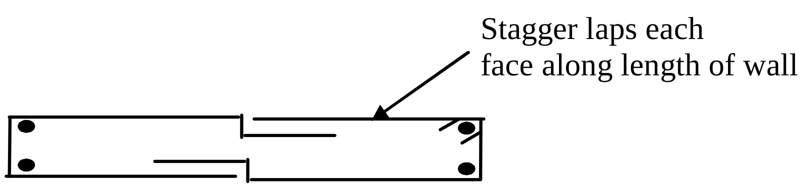
But, lateral tie anchorage shall be either:

(1) 180º
OR

(2) 135º

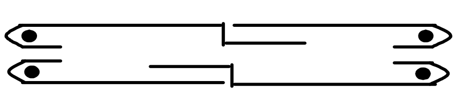
MSJC 1.18.4.4.2.3

Closed ties:



Follow column detailing in compression zone

OR



ETC

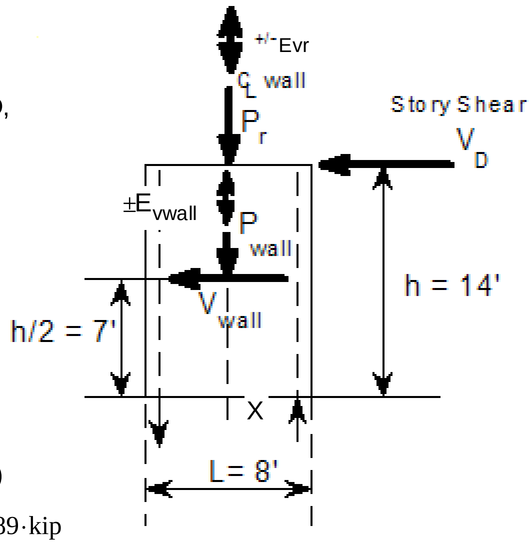
EXAMPLE 9: Shear Wall Design (Large axial load, Medium moment)

Given

SDC D,

S_{DS}=0.50

Solid Grouted 12" CMU, at 124 PSF



Forces due to seismic loads in SDC 'D'

$b := 11.625in \quad S_{DS} := 0.51$

$f'_m := 1500psi \quad h := 14ft$

$F_s := 24ksi \quad L := 8ft$

$V_R := 14kip$ (Seismic)

$P_{roof} := 210kip$ (Dead Load)

$P_{wall} := 0.124ksf \cdot 8ft \cdot 14ft = 13.89 \cdot kip$

Pw is entire height x length of wall weight

$$\pm E_{vr} = \frac{0.2 \cdot S_{DS} \cdot P_{roof}}{1.4} = \frac{0.2 \cdot (0.51) \cdot 210kip}{(1.4)} = 15.3kip$$

$$\pm E_{vwall} = \frac{0.2 \cdot S_{DS} \cdot P_{wall}}{1.4} = \frac{0.2 \cdot (0.51) \cdot 13.9kip}{(1.4)} = \square\ 1.01kip$$

Note: This example is based on using ASD Basic load combinations. For the ASD basic load combinations, 0.7E should be used instead E/1.4.

$V_W := 2kip$ From MLFR analysis

$$A := b \cdot L = (11.625in)(96in)$$

$$M := V_R \cdot h + V_W \cdot \left(\frac{h}{2}\right) = \boxed{(14kip)(14ft) + (2kip)\left(\frac{14ft}{2}\right) = 210 \cdot kip \cdot ft}$$

$$V := V_R + V_W = 16kip$$

$$P_{(DL)} = \Sigma P_{DL} = 210kip + 13.9kip = 223.9kip$$

$$P_{max(DL+0.7E)} = \Sigma P_{DL} = 210kip + 13.9kip + 15.3kip + 1.01kip = 240.2kip$$

$$P_{min(0.9DL-0.7E)} = \Sigma P_{DL} = 0.6 \cdot 210kip + 0.6 \cdot 13.9kip - 1.01kip - 15.3kip = 118.0kip$$

Check for uncracked section (using 0.90DL ±E/1.4))

Min $f_a := \frac{P}{A} = \boxed{\frac{(118.0\,kip)\left(1000\,\frac{lbf}{kip}\right)}{(11.625 \cdot in) \cdot (96 \cdot in)} = 106 \cdot psi}$

$$S := \frac{b \cdot L^2}{6} = \boxed{\frac{(11.625in)(96in)^2}{6} = 17856 \cdot in^3}$$

USE P_{min} to check P/A > M/S

$$f_b := \frac{M}{S} = \frac{(210kip\cdot ft)\left(1000\frac{lbf}{kip}\right)\left(12\frac{in}{ft}\right)}{\left(17856in^3\right)} = 141psi$$

$f_a < f_b$ Therefore: Section is cracked

For compression check, use full DL + LL

T = 0
kd = L

Check for Tension in Reinforcing

$$e := \frac{M}{P_{min}} = \frac{(210kip\cdot ft)}{118.0\ kip} = 1.78\ ft$$

$$M = V\cdot h + V\cdot\left(\frac{h}{2}\right)$$

If no tension reinforcing, compression block would be 3 x 1.78 = 5.33'. Reinforcing will be in tension.

Provide minimum vertical reinforcing for special reinforced wall. Target ρ = 0.0013 with a maximum spacing of 32" given 96" wall length.

Provide (2) #5 @ 32" oc; ρ = 0.0017

C=P
T=0

Check Compression in Masonry --- Use P_{max}

Use Full DL

$$r := \frac{d}{\sqrt{12}} = \frac{11.625in}{\sqrt{12}} = 3.36\cdot in \quad \text{(solid grouted)}$$

$$\frac{h'}{r} = \frac{(14ft)\left(12\frac{in}{ft}\right)}{3.36in} = 50 < 99$$

R and r are based on slenderness, out of plane, k = 1.0 (propped at top) MSJC EQN 2-20

MSJC 2.3.4.2.1

$$F_a := 0.25f'_m\cdot\left[1-\left(\frac{h'}{140r}\right)^2\right] \quad \textbf{OR} \quad = \quad 0.25f'_m\cdot[R] \quad \text{where } R := \left[1-\left(\frac{h'}{140r}\right)^2\right]$$

$$F_a = (0.25)(1500psi)\left[1-\left[\frac{(14ft)\left(12\frac{in}{ft}\right)}{(140)(3.36in)}\right]^2\right] = 375psi\cdot[R] = 375psi\cdot[0_{87}] = 327psi$$

[R] = 0.87

Check axial DL + 0.7E

$$P_{max} := 240.2kip$$

$$f_a := \frac{240kip\cdot 1000\frac{lbf}{kip}}{\left(11.625in\cdot 8ft\cdot 12\frac{in}{ft}\right)} = 215.1\ psi$$

$215\cdot psi = f_a <$ $F_a =$ $327\cdot psi$ OK

Check Combined Axial Load and Flexure

Construct Interaction Diagram

Given large axial load, we likely only need upper half of interaction diagram.

We already have M=0, P= (327)(11.625)(96) = 365 kips

Determine M_{bal}, P_{bal}.

Neutral axis at balanced condition, k_b:

$k_b = \frac{F_b}{F_b + \frac{F_s}{n}} = \frac{0.675}{0.675 + \frac{32}{21.5}} = 0.31$ where

$F_b = 0.45f'_m = 0.45(1{,}500) = 675$ and $n = \frac{E_s}{E_m} = \frac{29.000{,}000}{900f'_m} = 21.5$

Thus

$$k_b = \frac{F_b}{F_b + \frac{F_s}{n}} = \frac{0.675}{0.675 + \frac{32}{21.5}} = 0.31$$

Once the kb is known, Pbal and Mbal can be determined:

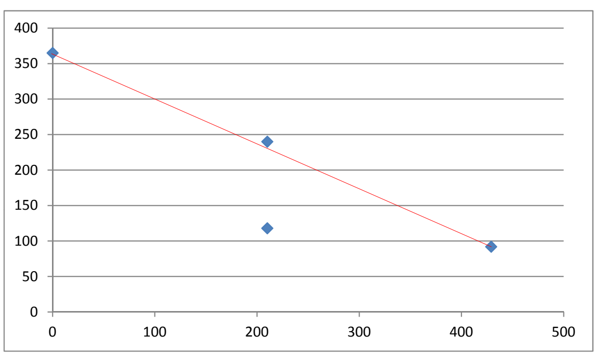
$$P_{bal} = \frac{F_b}{2} bkd - A_s F_s = \frac{0.675}{2}(11.625)(0.31)(92) - (2)(0.31)(32) = 92\ kips$$

$$M_{bal} = \frac{F_b}{2} bkd\left(\frac{h}{2} - \frac{kd}{3}\right) + A_s F_s\left(d - \frac{h}{2}\right)$$

$$M_{bal} = \frac{0.675}{2}(11.625)(0.31)(92)\left(\frac{96}{2} - \frac{0.31(92)}{3}\right) + (2)(0.31)(32)\left(92 - \frac{96}{2}\right) = 5145\ k - in$$

$$M_{bal} = 429\ k - ft$$

Constructing the interaction diagram we find:



Since P_{max} is slightly outside the interaction diagram, add another point to interaction diagram.

Choose k = 0.5. Strain in steel will be limited by the maximum stress in the masonry

$$f_s = \varepsilon_s E_s = \frac{1-k}{k} n F_b \leq F_s$$

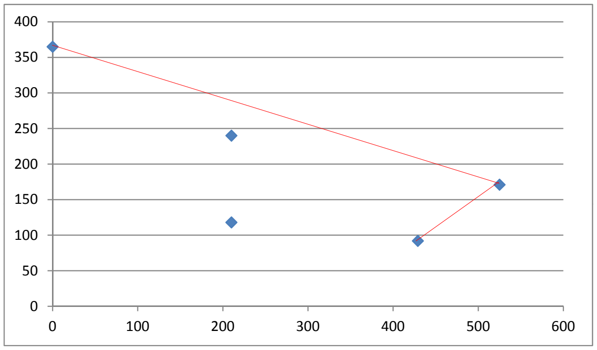
$$f_s = \frac{1-0.5}{0.5}(21.5)675 \leq 14{,}500\ psi$$

$$P = \frac{F_b}{2} bkd - A_s F_s = \frac{0.675}{2}(11.625)(0.5)(92) - (2)(0.31)(14.5) = 171\ kips$$

$$M = \frac{F_b}{2} bkd \left(\frac{h}{2} - \frac{kd}{3}\right) + A_s F_s \left(d - \frac{h}{2}\right)$$

$$M = \frac{0.675}{2}(11.625)(0.5)(92)\left(\frac{96}{2} - \frac{0.5(92)}{3}\right) + (2)(0.31)(14.5)\left(92 - \frac{96}{2}\right)$$

$$M = 6{,}290\ k - in = 525\ k - ft$$



Therefore the section is adequate.

Check Shear

$$\frac{M}{Vd} = \frac{210(12)}{16(92)} = 1.71$$

$$V_{special} = 1.5(V) = 1.5(16) = 24\ kips$$

$$f_v = \frac{V_{special}}{bh} = \frac{24{,}000}{(11.625)(96)} = 32.2\ psi$$

For a special reinforced masonry wall, allowable shear stress in masonry is determined by Eq 2-29 (Note that M/Vd need not be taken greater than 1):

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}$$

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75(1)\right)\sqrt{1500}\right] + 0.25\frac{118}{(11.625)96} = 36.1\ psi$$

$F_{vm} > f_v$ Shear reinforcing is not required.

Verify that the maximum allowable shear stress is not being exceeded. Since M/Vd is1.0, use MSJC Equation 2-27:

$F_{v.max} = 2\sqrt{f'_m} = 2\sqrt{1500} = 77.4\ psi > f_v = 32.2\ psi$ OK

Use minimum horizontal wall reinforcement

Horizontal Reinforcing

$$A_{s\_min} = 0.0007\cdot(11.625\cdot in)\cdot\left(\frac{12\cdot in}{ft}\right) = 0.098\cdot\frac{in^2}{ft}$$

For 96" long special wall, max spacing is 32"

Use: #4 @ 32" ea face horz $\left(A_s := 0.15\ \frac{in^2}{ft} \right)$

Vertical Reinforcing :

1. As Required by OOP Design for Special Shear Wall.

2. $\frac{Av + Ah}{b\cdot t} \geq 0.002$

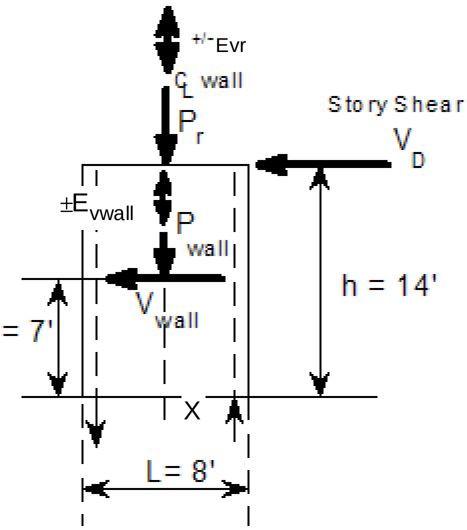
EXAMPLE 9: Shear Wall Design (Large axial load, Medium moment)

Given

SDC D,

S_{DS}=0.50

Solid Grouted 12" CMU, at 124 PSF



$b := 11.625in$ $S_{DS} := 0.51$

$f'_m := 1500psi$ $h := 14ft$

$F_s :=$ 60 ksi $L := 8ft$

$V_{u,R} := 20\,kip$ (Seismic)

$P_{roof} :=$ 210kip (Dead Load)
100 kip (Live Load)

$P_{wall} := 0.124ksf \cdot 8ft \cdot 14ft = 13.89 \cdot kip$

Forces due to seismic loads in SDC 'D'

Pw is entire height x length of wall weight

$$\pm E_{vr} = 0.2 \cdot S_{DS} \cdot P_{roof} = 0.2 \cdot (0.51) \cdot 210kip = 21.4\,kip$$

$$\pm E_{vwall} = 0.2 \cdot S_{DS} \cdot P_{wall} = 0.2 \cdot (0.51) \cdot 13.9kip = 1.4\ kip$$

$V_{u,w} := 3\,kip$ From MLFR analysis

Note: This example is based on using ASD Basic load combinations. For the ASD basic load combinations, 0.7E should be used instead E/1.4.

$$A := b \cdot L = (11.625in)(96in)$$

$$M_u := V_{u,R} \cdot h + V_{u,w} \cdot \left(\frac{h}{2}\right) = \boxed{(20kip)(14ft) + (3kip)\left(\frac{14ft}{2}\right) = 301\ kip \cdot ft}$$

$$V_u := V_{u,R} + V_{u,w} = 23\ kip$$

$$P_{(DL)} = \Sigma P_{DL} = 210kip + 13.9kip = \cdot 223.9kip$$

$P_{1.2D + 1.0E + 0.5L}$ = 1.2(210 + 13.9) + 1.0(21.4 + 1.4) + 0.5 (100)= 341 kips

$P_{0.9D - 1.0E}$ = 0.9(210 + 13.9) - 1.0(21.4 + 1.4) = 179 kips

$P_{1.2D+1.6L}$ = 1.2(210+13.9) + 1.6(100) = 429 kips

Check Compression in Masonry

Compute axial capacity; ignore reinforcing as it is unconfined:

$$\phi P_n = \phi 0.80\left[0.80 f'_m (A_n)\right]\left[1-\left(\frac{h}{140r}\right)^2\right]$$ EQN 3-18

$$\phi P_n = (0.9)0.80[0.8(1.5)(11.625)(96)]\left[1-\left(\frac{50}{140}\right)^2\right]$$

$$\phi P_n = 1{,}050\ kips > \ P_u = 429\ kips$$

Estimate Reinforcing

Since axial forces are large, assume minimum reinforcing will be adequate.

Provide minimum vertical reinforcing for special reinforced wall. Target ρ = 0.0013 with a maximum spacing of 32" given 96" wall length.

Provide (2) #5 @ 32" oc; ρ = 0.0017

Before proceeding, check ρ_{max} per MSJC Section 3.3.3.5.1. This is applicable when $M_u/V_u d \geq 1$. Shear and moment are both at a maximum at the base of wall, therefore:

$$\frac{M_u}{V_u d} = \frac{301(12)}{(23)(92)} = 1.71$$

ρ_{max} must be checked. For special wall must be able to achieve 4 times the yield strain when subject to axial load from D + 0.75L + 0.525 QE = (210 + 13.9) + 0.75(100) + 0.525(21.4 + 1.4) = 311 kips. 3.3.3.5.2.1(d)

The check must consider all reinforcing which will be intension, but can also consider any reinforcing in compression – confinement is not required. (4) #5 will be in tension, (2) #5 will be in compression. Remaining (2) #5 will be near neutral axis.

Neutral axis depth when four times the yield strain is reached in the extreme tension reinforcing:

$$c = \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right)d = \left(\frac{0.0025}{0.0025 + (4)0.0021}\right)d = 0.22d$$

Stress in compression steel:

$$F_s = \frac{c - d'}{c}\varepsilon_{mu} E_s \leq F_y$$

$$F_s = \frac{0.22(92) - 4}{0.22(92)} 0.0025(29{,}000) = 58.2 \leq 60$$

Available compression capacity

$$C = 0.64 f'_m bc + A_s F_s = 0.64(1.5)(11.625)(0.22)(92) + (2)(0.31(58.2) \\ = 262\ kips$$

Note that ρ_{max} cannot be achieved due to large compression load.

Increase f'_m to 2,500 psi.

$$C = 0.64f'_m bc + A_s F_s = 0.64(2.5)(11.625)(0.22)(92) + (2)(0.31(58.2)$$
$$= 412\ kips$$

This must balance the axial load plus the yielding tension reinforcing:

$P + A_s F_y = 311 + (4)(0.31)(60) = 385\ kips$ OK

Complete the problem using f'_m = 2,500 psi.

Recompute the axial capacity of the wall:

$$\phi P_n = (0.9)0.80[0.8(2.5)(11.625)(96)]\left[1 - \left(\frac{50}{140}\right)^2\right]$$

$$\phi P_n = 1{,}400\ kips > \ P_u = 429\ kips$$

Check Combined Axial Load and Flexure

Construct Interaction Diagram

Given large axial load, we likely only need upper half of interaction diagram.

We already have ϕM_n=0, ϕP_n = 1,400 kips

Determine $\phi M_{n,bal}$, $\phi P_{n,bal}$.

Determine neutral axis depth at balanced condition:

$$c_{bal} = \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_y} d = \frac{0.0025}{0.0025 + 0.0021} 92 = 50\ in$$

Therefore

$$\phi P_{n,bal} = \phi\left(0.64 f'_m bc_{bal} - A_s F_y\right)$$

$$\phi P_{n,bal} = 0.9(0.64(2.5)(11.625)(50) - (2)(0.31)(60)) = 803\ kips$$

$$\phi M_{n,bal} = \phi\left[0.64 f'_m bc_{bal}\left(\frac{h}{2} - 0.4c_{bal}\right) + A_s F_y\left(d - \frac{h}{2}\right)\right]$$

$$\phi M_{n,bal} = 0.9\left[0.64(2.5)(11.625)(50)\left(\frac{96}{2} - 0.4(50)\right) + (2)(0.31)(60)\left(92 - \frac{96}{2}\right)\right]$$

$$\phi M_{n,bal} = 24{,}900\ k - in = 2.075\ k - ft$$

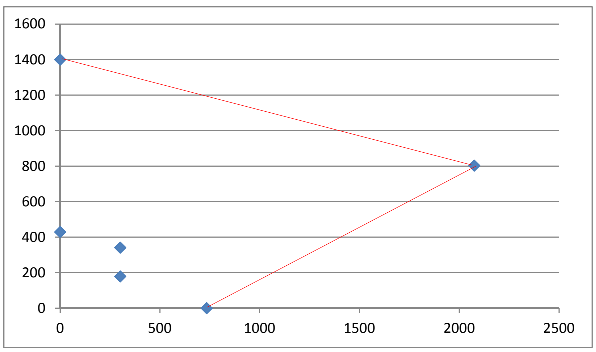
Note that since our nominal axial capcities are high relative to the demand, we will need to add ϕM_n, $\phi P_n = 0$.

$$\phi M_n = \phi \rho f_y b d^2 \left(1 - \frac{0.625 \rho f_y}{f'_m}\right)$$

$$\phi M_n = (0.9)(0.0017)(60)(11.625)(92)^2 \left(1 - \frac{0.625(0.0017)(60)}{2.5}\right)$$

$$\phi M_n = 8{,}800\ k - in = 734\ k - ft$$

Constructing the interaction diagram we find:



The section is adequate.

Check Shear

Since this is a special reinforced shear wall, design for 2.5Vu:

$$V_{u,special} = 2.5V_u = 2.5(23) = 57.5\ kips$$

First check to ensure that $\phi V_{n,max}$ is not exceeded. Since $M_u/V_u d > 1$, use MSJC Equation 3-22:

$$\phi V_{n.max} = \phi 4\sqrt{f'_m} A_{nv} = (0.8)4\sqrt{2500}(11.625)(96) = 178\ kips$$

Determine nominal capacity of masonry, noting that $M_u/V_u d$ need not exceed 1:

$$\phi V_{nm} = \phi\left(4.0 - 1.75\left(\frac{M_u}{V_u d}\right)\right) A_{nv}\sqrt{f'_m}$$

$$\phi V_{nm} = 0.8\big(4.0 - 1.75(1)\big)(11.625)(96)\sqrt{2500} = 100\ kips < V_{u,special} = 57.5\ kips$$

Only minimum shear reinforcing is required.

Use minimum horizontal wall reinforcement

Horizontal Reinforcing

$$A_{s\_min} = 0.0007\cdot(11.625\cdot in)\cdot\left(\frac{12\cdot in}{ft}\right) = 0.098\cdot\frac{in^2}{ft}$$

For 96" long special wall, max spacing is 32"

Use: #4 @ 32" ea face horz $\left(A_s := 0.15\ \frac{in^2}{ft} \right)$

Vertical Reinforcing :

1. As Required by OOP Design for Special Shear Wall.

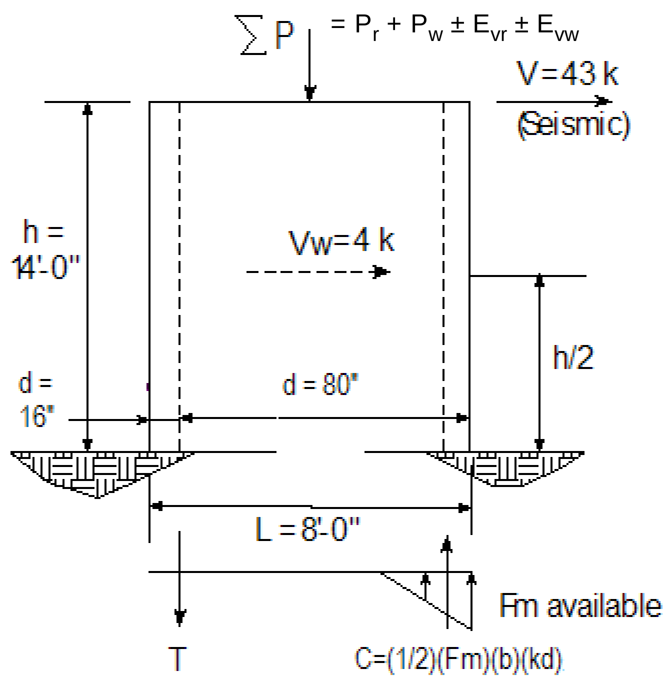
2. $\frac{Av + Ah}{b\cdot t} \geq 0.002$

EXAMPLE 10: CMU Shear Wall Design

(Similar to example 8 but with larger axial load)

Given:

12" CMU wall with medium levels of both axial and shear loads



$h := 14.0ft$

$k := 1.0$ Pinned-Pinned Condition

$h' := k \cdot h$ $h' = 14\,ft$

$L := 8ft$ Total length of wall

$d := L - 16in$

$d = 80 \cdot in$

$f'_m := 2500psi$

$E_m := 900 \cdot f'_m$

$E_m = 2.25 \times 10^6 \cdot psi$

Grade 60 Steel $F_s =$ 32 ksi

$E_s := 29 \times 10^3 ksi$ $E_m := 900 \cdot f'_m = 2.25 \times 10^6 ksi$

$E_m := 2250ksi$ $n := \dfrac{E_s}{E_m}$ $n = 13$

$t := 11.625in$ Wall thickness

$b := 12 \cdot in$ 1 ft width of wall

Running Bond

Solid grouted

SDC D

$\Sigma\ P := 160kip$ Axial load at wall center line (also includes weight of entire wall height and downward effects $P = P_R + E_{VR} + P_{WALL} + E_{VW}$)

$V := 43kip$ Seismic shear at top of wall from LFRS analysis

$V_w := 4kip$ Shear wall seismic force due to self weight

Approximate assumptions since E_v would not have a 0.6 reduction

$M := V \cdot h' + V_w \cdot \left(\frac{h'}{2}\right)$ $\boxed{M = 43kip \cdot (14ft) + (4kip)\left(\frac{14ft}{2}\right)}$ Bending moment

$M = 630 \cdot ft \cdot kip$

Check C, T, and P Relationships:

$e := \frac{M}{P} = \frac{(630ft \cdot kip)}{(0.6)\ (160kip)} = 6.56\ ft$ $e > L/6$ — 1605.3.2 EQN (16-21)

$\frac{L}{6} = \frac{8ft}{6} = 1.33\ ft$ $\frac{L}{2} - \frac{d}{3} = \frac{8ft}{2} - \frac{6.67ft}{3} = 1.78ft$ $e > \frac{L}{2} - \frac{d}{3}$ Tension reinf. required — 0.9D +E/1.4

OR Check for uncracked section:

$A := L \cdot t$ $\boxed{A = (8ft) \cdot \left(12\frac{in}{ft}\right) \cdot (11.625ft)}$ Cross sectional area of wall

$A = 7.75\,ft^2$ $= 1116\ in^2$

$f_a := \frac{0.6\ P}{A}$ $\boxed{f_a = \frac{0.6 \cdot 160kip}{1116in^2}}$ Axial stress — P_{min}

$f_a = 86 \cdot psi$

$S := \frac{t \cdot L^2}{6}$ $\boxed{S = \frac{(11.625 \cdot in) \cdot (8 \cdot ft)^2 \cdot \left(12 \cdot \frac{in}{ft}\right)^2}{6}}$ Section modulus — S_{gross}

$S = 17856 \cdot in^3$

$f_b := \frac{M}{S}$ $\boxed{f_b = \frac{630 \cdot ft \cdot kip \cdot \left(12 \cdot \frac{in}{ft}\right)}{17856in^3}}$ Bending stress

$f_b = 423 \cdot psi$

$f_b >> f_a$, therefore section is <u>cracked</u>.

$F_b := 0.45f'_m$ $F_b = 1125psi$ Allowable bending stress — MSJC 2.3.3.2.2

$r := 3.34in$ Radius of gyration for 12" solid grouted wall — Table 10(b)

$\frac{h'}{r} = 50$ 50 < 99, Therefore use equation (2-17ws) for determining allowable axial stress

$F_a := 0.25 \cdot f'_m \cdot \left[1 - \left(\frac{h'}{140 \cdot r}\right)^2\right]$ $\boxed{F_a = 0.25 \cdot (2500psi) \cdot \left[1 - \left[\frac{(14ft)\left(12\frac{in}{ft}\right)}{(140)(3.34)}\right]^2\right]}$

$F_a = (625\text{psi})[0_{87]}$ $\quad F_a = 544 \cdot \text{psi}$

$P_a = F_a bh = (544)(11.625)(96) = 607$ kips

Estimate Reinforcing

Given that neither the moment nor the axial load appears to dominate behavior, estimate reinforcing assuming section is controlled:

$$A_s = \frac{M}{0.9F_s d} - \frac{P}{2F_y} = \frac{630(12)}{0.9(32)(80)} - \frac{(0.6)(160)}{2(60)}$$

$$A_s = 3.28 - 0.80 = 0.95\ in^2$$

Provide (8) #5 @ end of wall, d = 80.

Check Maximum Reinforcing

Applies if M/Vd ≥ 1 and P > 0.05 $f'_m A_n$

$$\frac{M}{Vd} = \frac{630(12)}{(47)(80)} = 2.01$$

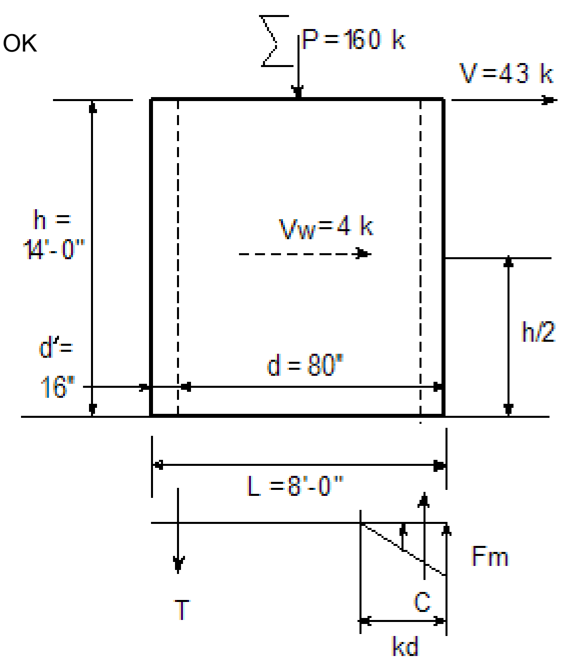
$$0.05f'_m A_n = 0.05(2{,}500)(11.625)(96) = 139\ kips < 160\ kips$$

Must check limit

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'_m}\right)} = \frac{13(2.5)}{2(60)\left(13 + \frac{60}{2.5}\right)} = 0.0073$$

Need to consider all possible tension steel - assume that a total of (10) #5 are in tension, effective d = 76"

$\rho = \frac{A_s}{bd} = \frac{(10)(0.31)}{(11.625)(76)} = 0.0035$ OK



Check Combined Axial Load and Flexure

Construct Interaction Diagram

Given large axial load, we likely only need upper half of interaction diagram.

We already have M=0, P= 607 kips

Determine M_{bal}, P_{bal}.

Neutral axis at balanced condition, k_b:

$$k_b = \frac{F_b}{F_b + \frac{F_s}{n}} = \frac{1.125}{1.125 + \frac{32}{13}} = 0.31$$

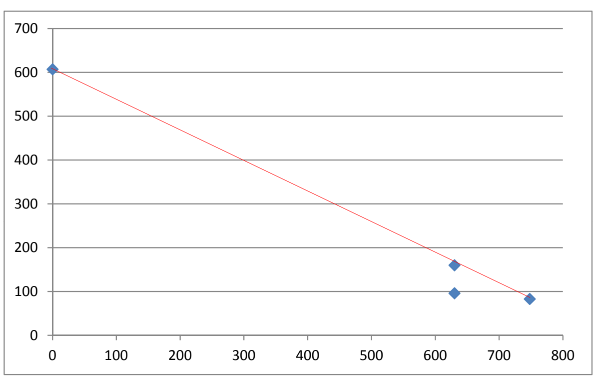
Once the kb is known, Pbal and Mbal can be determined:

$$P_{bal} = \frac{F_b}{2} bk_{bal}d - A_s F_s = \frac{1.125}{2}(11.625)(0.31)(80) - (8)(0.31)(32) = 83\ kips$$

$$M_{bal} = \frac{F_b}{2} bk_{bal}d\left(\frac{h}{2} - \frac{k_{bal}d}{3}\right) + A_s F_s\left(d - \frac{h}{2}\right)$$

$$M_{bal} = \frac{1.125}{2}(11.625)(0.31)(80)\left(\frac{96}{2} - \frac{0.31(80)}{3}\right) + (8)(0.31)(32)\left(80 - \frac{96}{2}\right) = 8983\ k - in$$

$$M_{bal} = 748\ k - ft$$



Shear Design

$$V_{special} = 1.5(V) = 1.5(47) = 70.5\ kips$$

$$f_v = \frac{V_{special}}{bh} = \frac{70{,}500}{(11.625)(96)} = 63.2\ psi$$

For a special reinforced masonry wall, allowable shear stress in masonry is determined by Eq 2-29 (Note that M/Vd need not be taken greater than 1):

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}$$

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75(1)\right)\sqrt{2500}\right] + 0.25\frac{(0.6)160{,}000}{(11.625)96} = 49.6\ psi$$

$F_{vm} < f_v$ Shear reinforcing is required.

Verify that the maximum allowable shear stress is not being exceeded. Since M/Vd is1.0, use MSJC Equation 2-27:

$F_{v.max} = 2\sqrt{f'_m} = 2\sqrt{2500} = 100\ psi > f_v = 63.2\ psi$ OK

Determine required shear reinforcing

$$F_{vm} + F_{vs} \geq f_v$$

$$F_{vs} = f_v - F_{vm} = 0.5\left(\frac{A_v F_s d}{A_n s}\right)$$

Rearranging terms

$$\frac{A_v}{s} = \frac{2(f_v - F_{vm})A_n}{F_s d} = \frac{2(63.2 - 49.6)(11.625)(96)}{(32{,}000)(80)} = 0.012\frac{in^2}{in}$$

A_v is 0.38 in^2 for a 32” reinforcing spacing. Provide (1) #4 each face.

(2)#4 @ 32” $\frac{A_v}{s} = \frac{(2)(0.20)}{32} = 0.0125\frac{in^2}{in}$ OK