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

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W. Mark McGinley, Ph. D., P.E., FASTM, FTMS
Professor and Endowed Chair,
University of Louisville , Department of Civil Engineering

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

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Course Description

During this session, allowable stress design of masonry walls loaded with out-of-plane loads and axial loads will be reviewed. Differences in the Allowable Stress design provisions and strength design procedures will be briefly discussed, especially the secondary bending moments.

Learning Objectives:

1. Review the design of walls loaded with out-of-plane with axial loads, including a brief overview of unreinforced masonry design.
2. Describe basic differences between allowable stress design and strength design for such walls
3. Development of ASD Interaction diagrams will be presented.
4. Provide examples of masonry walls for common thicknesses, reinforcing and load and loads.

Learning Objectives

- Describe basic differences between allowable stress design and strength design for Out of plane loading on walls
- Review the ASD design of walls loaded with out-of-plane with axial loads

Combined loading Out of Plane Loading on Masonry Walls - ASD

- For unreinforced Masonry
- Interaction Diagram - Only method allowed by Code

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

Ch. 8.2 in MSJC-ASD URM Masonry

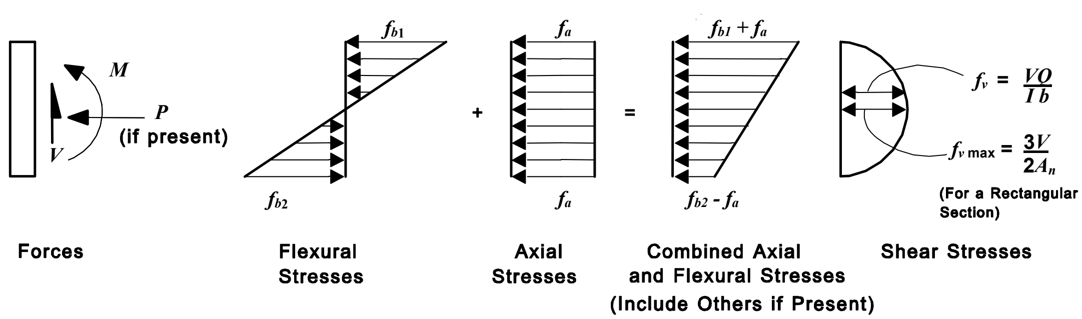


Figure 11.3-2 Combined Flexural Stress and Shear Stress Distribution in Uncracked Section, Unreinforced Masonry

- From MDG 2016 Chapter 11

Ch. 8.2 in MSJC-ASD URM Masonry

Assumptions (Stresses on net section) – $f_a = \frac{P}{A_n}$, $f_b = \frac{M}{S_n}$

- Net flexural tension stress limited - Table 8.2.1.4 $f_t \leq F_t$

Table 8.2.4.2 — Allowable flexural tensile stresses for clay and concrete masonry, psi (kPa)

Direction of flexural tensile stress and masonry type	Mortar types			
	Portland cement/lime or mortar cement		Masonry cement or air entrained portland cement/lime	
	M or S	N	M or S	N
Normal to bed joints				
Solid units	53 (366)	40 (276)	32 (221)	20 (138)
Hollow units[1]				
Ungrouted	33 (228)	25 (172)	20 (138)	12 (83)
Fully grouted	65 (448)	63 (434)	61 (420)	58 (400)
Parallel to bed joints in running bond				
Solid units	106 (731)	80 (552)	64 (441)	40 (276)
Hollow units				
Ungrouted and partially grouted	66 (455)	50 (345)	40 (276)	25 (172)
Fully grouted	106 (731)	80 (552)	64 (441)	40 (276)
Parallel to bed joints in masonry not laid in running bond				
Continuous grout section parallel to bed joints	133 (917)	133 (917)	133 (917)	133 (917)
Other	0 (0)	0 (0)	0 (0)	0 (0)

Ch. 8.2 in MSJC-ASD URM Masonry

- Compression stress limited $f_a \leq F_a$, $f_b \leq 1/3 f_m$

$$F_a = (0.25f'_m)\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} \leq 99$$

$$F_a = (0.25f'_m)\left(\frac{70r}{h}\right)^2 \text{ for } \frac{h}{r} > 99 \text{ and}$$

$$P \leq P_e = \left[\frac{\pi^2 E_m I_n}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3\right]$$

- Force unity equation $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$
- Shear , $f_v = \frac{VQ}{I_n b} \leq 1.5\sqrt{f'_m}$, 120 psi, or 37 psi +0.45 $\frac{N_V}{A_n}$, or 60 psi +0.45 $\frac{N_V}{A_n}$, or 15 psi

Ch. 8.3 in MSJC-ASD Reinforced Masonry

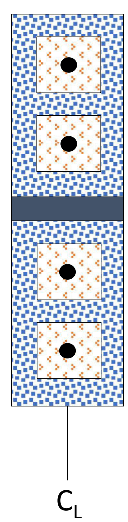
Table 11.4.1 Allowable Stresses for Reinforced Masonry

`	Equation	TMS 402 Reference
Axial compression	$P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h/r}{140}\right)^2\right] \quad h/r \le 99$ $P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left(\frac{70}{h/r}\right)^2 \quad h/r > 99$ A_{st} = 0 unless the reinforcement in compression is tied in compliance with TMS 402 Section 5.3.1.4	Equation (8-18) Equation (8-19)
Flexural compression	$F_b = 0.45 f'_m$	Section 8.3.4.2.2
Flexural tension	F_s = 20,000 psi (Grade 40 or 50 reinforcement) F_s = 32,000 psi (Grade 60 reinforcement) F_s = 30,000 psi (wire joint reinforcement)	Section 8.3.3.1 Section 8.3.3.2
Shear	$F_v = (F_{vm} + F_{vs})\gamma_g$ $F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}$ $F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right)$ Special shear walls: $F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}$ $F_v \le \begin{cases} \left(3\sqrt{f'_m}\right)\gamma_g & M/(Vd_v) \le 0.25 \\ \left(2\sqrt{f'_m}\right)\gamma_g & M/(Vd_v) \ge 1.0 \\ \left(\frac{2}{3}\left(5 - 2\frac{M}{Vd_v}\right)\sqrt{f'_m}\right)\gamma_g & 0.25 < M/(Vd_v) < 1.0 \end{cases}$ Take M/Vd_v as a positive number, need not be > 1.0 γ_g = 0.75 for partially grouted shear walls and 1.0 otherwise.	Equation (8-22) Equation (8-26) Equation (8-27) Equation (8-25) Equation (8-23) Equation (8-24) Linear interpolation Section 8.3.5.1.2 (c)

Allowable Stress Interaction Diagrams - OOP

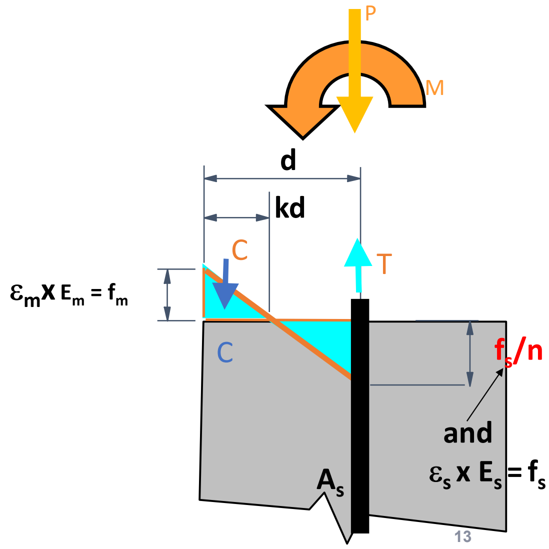
<u>Assume single reinforced</u>

- Out-of-plane flexure
- Grout and masonry the same
- Solid grouted
- Steel in center

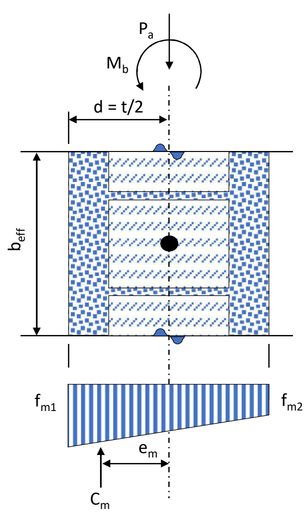


Allowable Stress Interaction Diagrams Walls – Singly Reinforced

- Allowable – stress interaction diagram
- Linear elastic theory – tension in masonry it is ignored, plane sections remain plane
- Limit combined compression stress to $F_b = 0.45F'_m$
- $P \leq P_a$
- d usually = t/2 – no compression steel since not tied, ignore in compression
- Assume a kd value and limit stresses



Allowable Stress Interaction Diagrams OOP



Assume stress gradient range A:

All section in compression
Kd>thickness of wall

Get equivalent force-couple about center line

$$P_a = 0.5(f_{m1} + f_{m2})A_n$$

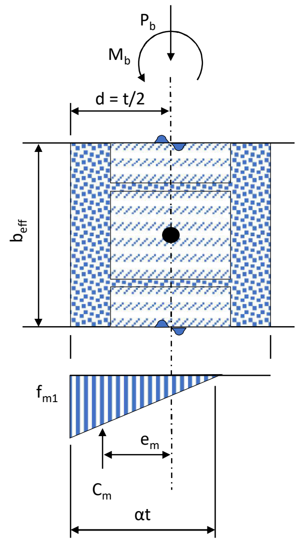
$M_a = (f_{m1} - f_{m2})/2(S),\ S = bt^2/6$

Note at limit – f_{m1} and $f_{m2} \leq F_b$ (set $f_{m1} = F_b$)

Note much of this is from Masonry Course notes by Dan Abrams

Also P_a cut off

Allowable Stress Interaction Diagrams OOP



Assume stress gradient range B:

Not all section in compression, but no tension in steel

Get equivalent force-couple about center line

$P_b = C_m = 0.5(f_{m1})\alpha t b$

$M_b = e_m \times C_m$

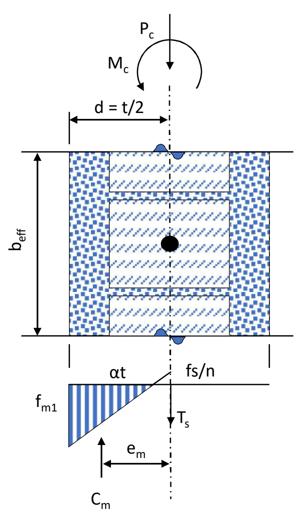
$e_m = d - \frac{\alpha t}{3} = t/2 - \frac{\alpha t}{3} = t\left(\frac{1}{2} - \alpha/3\right)$

Note that $\alpha t = kd$

This is valid until steel goes into tension

Set $f_{m1} = F_b$ at limit

Allowable Stress Interaction Diagrams OOP



Assume stress gradient range C:

Section in compression, tension in steel

Get equivalent force-couple about center line

$e_m = d - \frac{\alpha t}{3} = t/2 - \frac{\alpha t}{3} = t\left(\frac{1}{2} - \alpha/3\right)$

$C_m = 0.5(f_{m1})\alpha t b$

$P_c = C_m - Ts$ and $T_s = A_s \times f_s$

From similar triangles on stress diagram

$f_s/n = ([d - \alpha t]/\alpha t) f_{m1}$

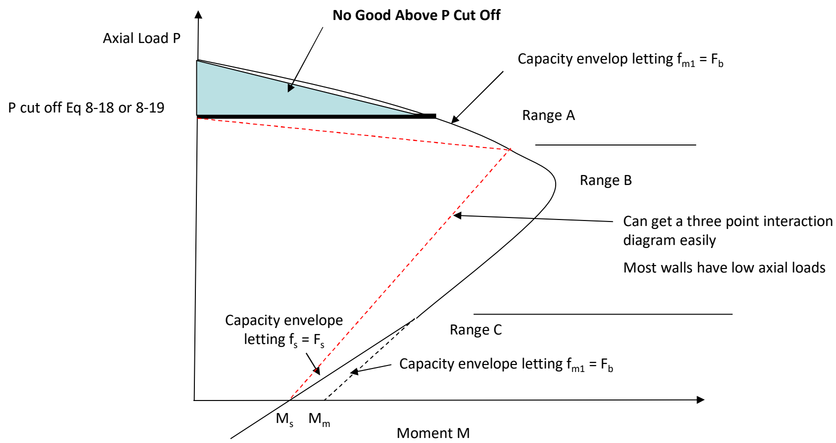
$M_b = e_m \times C_m - T_s(d - t/2)$; note that $d = t/2$ usually, so second term goes to zero

At limit $f_s = F_s$ and $f_{m1} \leq F_b$ or $f_{m1} = F_b$ and $f_s \leq F_s$ and the other governs – balance point when both occur.

Note that $\alpha t = kd$

$$k_b = \frac{n}{n + F_s / F_b}$$

Allowable Stress Interaction Diagrams OOP Singly Reinforced

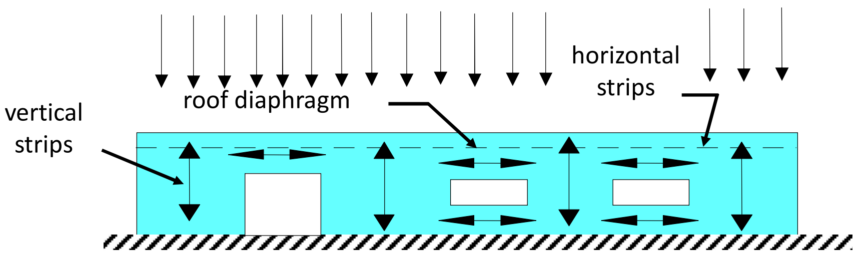


ASD Load Combinations – ASCE 7-16

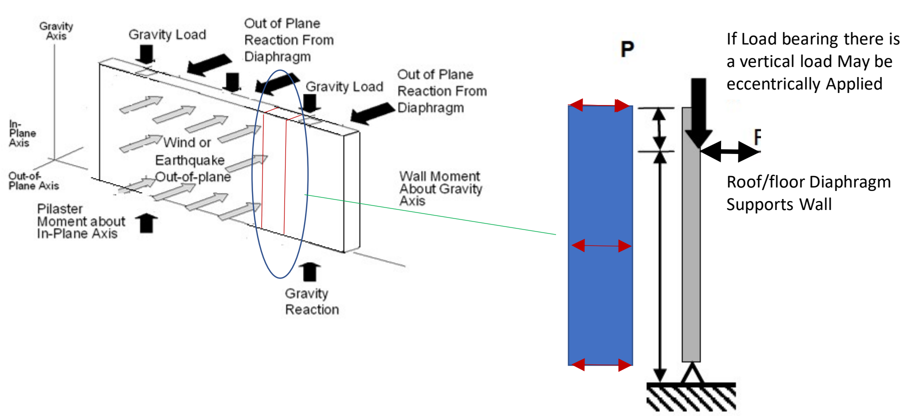
- $D + F$
- $D + H + F + L$
- $D + H + F + (L_r \text{ or } S \text{ or } R)$
- $D + H + F + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$
- $D + H + F + (0.6W \text{ or } 0.7E)$
- $D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
- $D + H + F + 0.75(0.7E) + 0.75L + 0.75(S)$
- $0.6D + 0.6W + H$
- $0.6(D + F) + H + 0.7E$
- No increase for E or W any more with Stress Recalibration – even with alternative load cases

Rational Design of Load – Bearing Walls – Also must account for holes in wall and span direction

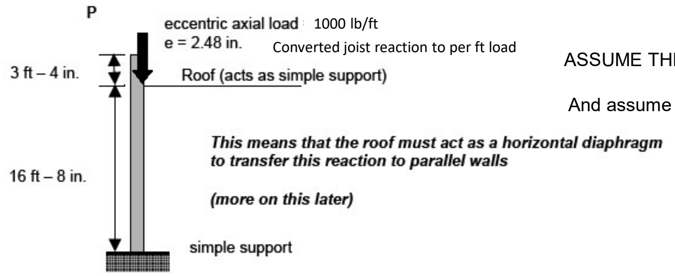
- gravity and out – of – plane loads are resisted by combinations of horizontal and vertical strips



ALLOWABLE STRESS (ASD) DESIGN URM WALLS – OUT OF PLANE AND IN- PLANE VERTICAL LOADS

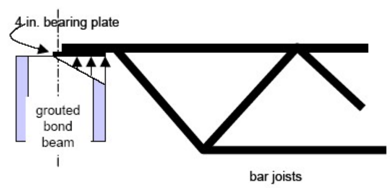


ALLOWABLE STRESS (ASD) DESIGN WALLS – OUT OF PLANE AND IN- PLANE VERTICAL LOADS



ASSUME THE AXIAL LOAD IS 700 lb/ft DEAD LOAD and the rest LIVE LOAD
And assume Strength level Wind load is 30 psf

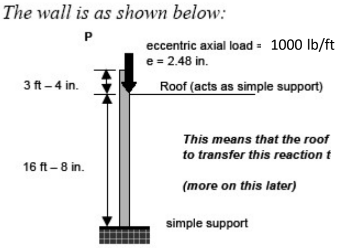
e found by assuming the bearing conditions shown



Then the eccentricity of the applied load with respect to the centerline of the wall is

$$e = \frac{t}{2} - \frac{plate}{3} = \frac{7.63\ in.}{2} - \frac{4\ in.}{3} = 2.48\ in.$$

The wall is as shown below:



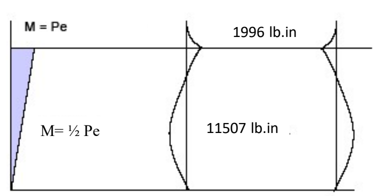
Due to wind only, the moment at the base of the parapet (roof level) is

M_{wind}= 30 plf x (3.33)^2/2 x 12in/ft = 1996 lb.in

The maximum moment is close to that occurring at mid-height. The moment from wind load is the superposition of one-half moment at the upper support due to wind load on the parapet only, plus the midspan moment in a simply supported beam with that same wind load:

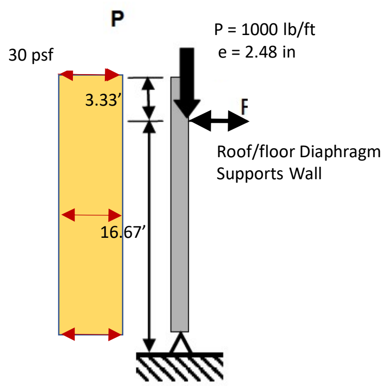
M_{wind} = (- 1996 lb.in/2 + (30 plf x (16.67)^2/8) x 12in/ft)
= 11,507 lb.in

M at top due to axial load
M_{DL} = 700 x 2.48" = 1736 lb.in,
M_{LL} = 300 x 2.48" = 744 lb.in
at mid-height ½ these values



ASD Interaction Diagram Walls – Singly Reinforced Example Problem 1 - MDG

Check adequacy of a solidly grouted 8" CMU – f'_m = 2000 psi



- If P = all dead load
- Check .6D + .6W at mid-height
- Would also have check other load combos
- 0.6D + .6W at mid-height often governs

- Per foot of wall P at mid-height including weight of wall
- P= 728 lb/ft (0.6 D)

  and M = 7,421 lb.in/ft (0.6D+0.6W)

- You would need to look at other load cases and at the top of the wall.

ASD Interaction Diagram

Set up

Spreadsheet

Guess at amount of steel

How?

I use first trial

A_s About $=M_{max}/(0.9dF_s)$

More axial force, less steel

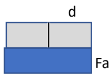
Try # 5 at 48 in. OC

0.068 by equation

0.066 in^2 / ft Provided

Spreadsheet for calculating allowable-stress M-N diagram for solid masonry wall							
16.67 Ft Wall w/ No. 5 @ 48in (Centered)				NOTE BASED ON 1 ft of Wall and Not EFFECTI			
total depth, t	7.625			Wall Height, h		16.67	feet
f'm, fprimem	2000			Radius of Gyration, r		2.20	in
Em	1800000				h/r	90.9	
Fb	900.00			Reduction Factor, R		0.578	
Es	29000000			Allowable Axial Stress, Fa		289	psi
Fs	32000			Net Area, An		365.7	in^2
d				le Axial Compr, Pa		26429	lb
kbalanced	0.311828						
tensile reinforcement, As/beff	0.31	#5 @ 48 Centered					
width, beff	48						
because compression reinforcement is not tied, it is not counted							
	k	**kd**	**fb**	**Cmas**	**fs**	**Axial Force**	**Moment**
			(psi)	**(lb)**	**(psi)**	**(lb)**	**(lb-in)**
Points controlled by steel	0.01	0.04	20	18	-32000	-2475	17
	0.05	0.19	105	478	-32000	-2360	448
	0.1	0.38	221	2019	-32000	-1975	1861
	0.15	0.57	351	4811	-32000	-1277	4356
	0.2	0.76	497	9087	-32000	-208	8084
	0.25	0.95	662	15145	-32000	1306	13232
	0.24	0.92	627	13774	-32000	963	12078
	0.3	1.14	851	23366	-32000	3362	20044
Points controlled by masonry	0.311828	1.19	900	25679	-32000	3940	21931
	0.4	1.53	900	32940	-21750	6549	27210
	0.5	1.91	900	41175	-14500	9170	32704
	0.6	2.29	900	49410	-9667	11603	37675
	0.8	3.05	900	65880	-3625	16189	46047
	1	3.81	900	82350	0	20588	52327
	1.2	4.58	900	98820	0	24705	56513
	1.4	5.34	900	115290	0	28823	58606
	1.6	6.10	900	131760	0	32940	58606
	1.8	6.86	900	148230	0	37058	56513
	2	7.63	900	164700	0	41175	52327
Pure compression			900	329400	0	329400	0
Axial Force Limits						26429	0
						26429	58606

ASD Interaction Diagram



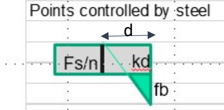
Spreadsheet for calculating allowable-stress M-N diagram for solid masonry wall				
16.67 Ft Wall w/ No. 5 @ 48in (Centered)		NOTE BASED ON 1 ft of Wall and Not EFFECT		
total depth, t	7.625	Wall Height, h	16.67	feet
f'm, fprimem	2000	Radius of Gyration, r	2.20	in
Em	1800000	h/r	90.9	
Fb	900.00	Reduction Factor, R	0.578	
Es	29000000	Allowable Axial Stress, Fa	289	psi
Fs	32000			
d	3.8125	Allowable Axial Compr, Pa	26429	lb
kbalanced	0.311828			
tensile reinforcement, As/beff	0.31 #5 @ 48 Centered			
width, beff	48			

h/r < 99 so $R = \left[1 - \left(\frac{h}{140r}\right)^2\right] = \left[1 - \left(\frac{16.67x\ 12}{140(2.2)}\right)^2\right] = 0.578$

8- 21 - $P_a = 0.25 f'_m\, R\, An = 0.25(2000)(0.578)(7.625$ x 48)/4ft =26429 lb

Em = 900(2000) = 1,800,000 , n= Es/Em = 29,000/1800 =16.11

ASD Interaction Diagram



k	kd	fb (psi)	Cmas (lb)	fs (psi)	Axial Force (lb)	Moment (lb-in)
0.01	0.04	20	18	-32000	-2475	17
0.05	0.19	105	478	-32000	-2360	448
0.1	0.38	221	2019	-32000	-1975	1861
0.15	0.57	351	4811	-32000	-1277	4356
0.2	0.76	497	9087	-32000	-208	8084
0.25	0.95	662	15145	-32000	1306	13232
0.24	0.92	627	13774	-32000	963	12078
0.3	1.14	851	23366	-32000	3362	20044

$$f_b = \frac{\frac{F_s}{n}}{d - kd} \times kd = \frac{\frac{32{,}000}{16.11}}{3.81 - 0.38} \times 0.38 = 221\ \text{psi}$$

$$C_{mas} = 1/2 \times fb \times kd \times b = 1/2 \times 221 \times 0.38 \times 48 = 2{,}019\ \text{lb}$$

$$P = C_{mas} - (A_s \times F_S) = 2019 - (0.31 \times 32{,}000) = -7{,}901\ \text{lb}$$

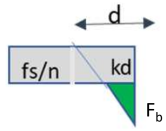
For b_{eff} for 1 ft of wall = $1/4 \times -7901 = -1975$ lb per foot of wall

$$M = C_{mas} \times e_m - T_s(d_s - \frac{t}{2}) =$$

$$M = 2019 \times \left(\frac{7.625}{2} - \frac{0.38}{3}\right) - T_s(0) = 7{,}437\ \text{lb.in}$$

For b_{eff} for 1 ft of wall = $1/4 \times 7437 = 1{,}860$ lb.in per foot of wall

ASD Interaction Diagram



	k	kd	fb (psi)	Cmas (lb)	fs (psi)	Axial Force (lb)	Moment (lb-in)
kb	0.311828	1.19	900	25679	-32000	3940	21931
	0.4	1.53	900	32940	-21750	6549	27210
	0.5	1.91	900	41175	-14500	9170	32704
	0.6	2.29	900	49410	-9667	11603	37675
	0.8	3.05	900	65880	-3625	16189	46047
	1	3.81	900	82350	0	20588	52327
	1.2	4.58	900	98820	0	24705	56513
	1.4	5.34	900	115290	0	28823	58606
	1.6	6.10	900	131760	0	32940	58606
	1.8	6.86	900	148230	0	37058	56513

$$F_b = \frac{\frac{f_s}{n}}{d-kd} \times kd = \frac{\frac{f_s}{16.11}}{3.81-1.53} \times 1.53 = 900\text{ psi}$$

$$f_s = 21{,}750\,psi$$

$$C_{mas} = 1/2 \times F_b \times kd \times b = 1/2 \times 900 \times 1.53 \times 48 = 32{,}940\text{ lb}$$

$$P = C_{mas} - (A_s \times F_S) = 32{,}940 - (0.31 \times 21{,}750) = 26{,}198\text{ lb}$$

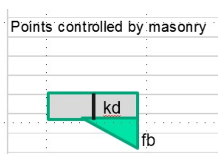
For b_{eff} for 1 ft of wall $= 1/4 \times 26{,}198 = 6549$ lb per foot of wall

$$M = C_{mas} \times e_m - T_s(d_s - \frac{t}{2}) =$$

$$M = 32940 \times \left(\frac{7.625}{2} - \frac{1.53}{3}\right) - T_s(0) = 108{,}702\text{ lb.in}$$

For b_{eff} for 1 ft of wall $= 1/4 \times 108{,}702 = 27{,}200$ lb.in per foot of wall

ASD Interaction Diagram



k	kd	fb (psi)	Cmas (lb)	fs (psi)	Axial Force (lb)	Moment (lb-in)
0.311828	1.19	900	25679	-32000	3940	21931
0.4	1.53	900	32940	-21750	6549	27210
0.5	1.91	900	41175	-14500	9170	32704
0.6	2.29	900	49410	-9667	11603	37675
0.8	3.05	900	65880	-3625	16189	46047
1	3.81	900	82350	0	20588	52327
1.2	4.58	900	98820	0	24705	56513
1.4	5.34	900	115290	0	28823	58606
1.6	6.10	900	131760	0	32940	58606
1.8	6.86	900	148230	0	37058	56513

$$C_{mas} = 1/2 \times F_b \times kd \times b = 1/2 \times 900 \times 3.81 \times 48 = 82{,}350\text{ lb}$$

$$P = C_{mas} - (A_s \times F_S) = 82{,}350 - (0.31 \times 0) = 82{,}350\text{ lb}$$

For b_{eff} for 1 ft of wall $= 1/4 \times 82{,}350 = 20{,}587$ lb per foot of wall

$$M = C_{mas} \times e_m - T_s(d_s - \frac{t}{2}) =$$

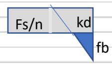
$$M = 82{,}350 \times \left(\frac{7.625}{2} - \frac{3.81}{3}\right) - T_s(0) = 209{,}306\text{ lb.in}$$

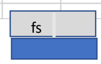
For b_{eff} for 1 ft of wall $= 1/4 \times 209306 = 52{,}326$ lb.in per foot of wall

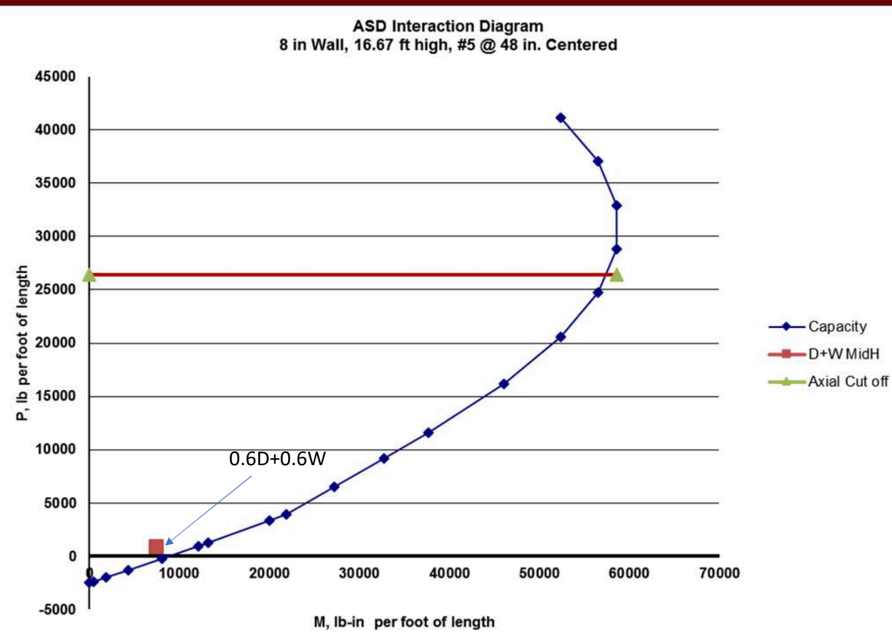
Note that Equations not valid after Kd =2

ASD Interaction Diagram

Spreadsheet for calculating allowable-stress M-N diagram for solid masonry wall							
16.67 Ft Wall w/ No. 5 @ 48in (Centered)				NOTE BASED ON 1 ft of Wall and Not EFFECT			
total depth, t	7.625				Wall Height, h	16.67	feet
f'm, fprimem	2000				Radius of Gyration, r	2.20	in
Em	1800000				h/r	90.9	
Fb	900.00				Reduction Factor, R	0.578	
Es	29000000				Allowable Axial Stress, Fa	289	psi
Fs	32000				Net Area, An	365.7	in^2
d	3.8125				Allowable Axial Compr, Pa	26429	lb
kbalanced	0.311828						
tensile reinforcement, As/beff	0.31	#5 @ 48 Centered					
width, beff	48						
because compression reinforcement is not tied, it is not counted							
	k	**kd**	**fb**	**Cmas**	**fs**	**Axial Force**	**Moment**
			(psi)	**(lb)**	**(psi)**	**(lb)**	**(lb-in)**
Points controlled by steel	0.01	0.04	20	18	-32000	-2475	17
	0.05	0.19	105	478	-32000	-2360	448
	0.1	0.38	221	2019	-32000	-1975	1861
	0.15	0.57	351	4811	-32000	-1277	4356
	0.2	0.76	497	9087	-32000	-208	8084
	0.25	0.95	662	15145	-32000	1306	13232
	0.24	0.92	627	13774	-32000	963	12078
	0.3	1.14	851	23366	-32000	3362	20044
Points controlled by masonry	0.311828	1.19	900	25679	-32000	3940	21931
	0.4	1.53	900	32940	-21750	6549	27210
	0.5	1.91	900	41175	-14500	9170	32704
	0.6	2.29	900	49410	-9667	11603	37675
	0.8	3.05	900	65880	-3625	16189	46047
	1	3.81	900	82350	0	20588	52327
	1.2	4.58	900	98820	0	24705	56513
	1.4	5.34	900	115290	0	28823	58606
	1.6	6.10	900	131760	0	32940	58606
	1.8	6.86	900	148230	0	37058	56513
	2	7.63	900	164700	0	41175	52327
Pure compression			900	329400	0	329400	0
Axial Force Limits						26429	0
						26429	58606







Interaction Diagrams

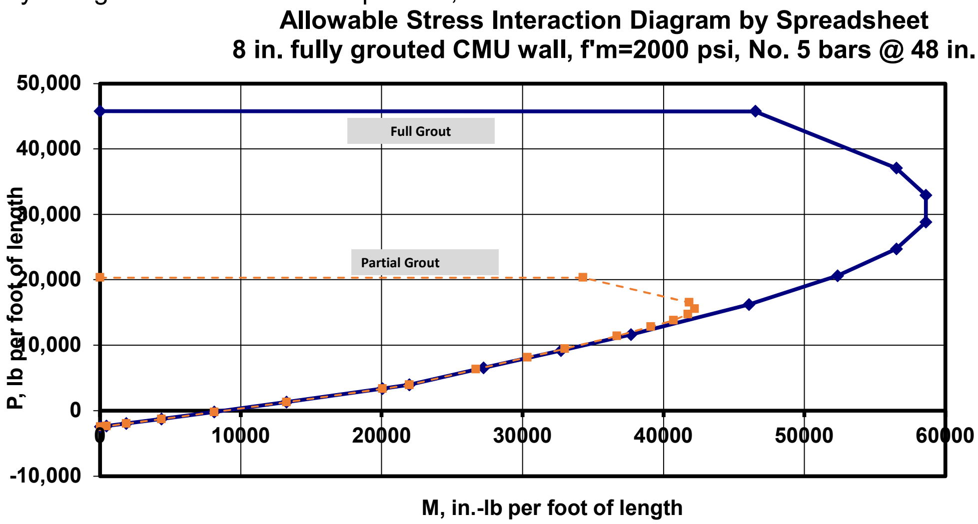
Masonry Design Guide 2016 – Chapter 11
Impact of Partial Grouting

When constructing the interaction diagrams, we assumed solid grouted walls. What happens when the wall is partially grouted?

- Typically, the width of a grouted cell and adjoining webs can be assumed to be 8 in. Some designers will use a slightly greater width but 8 in. is convenient. Thus, the the effective width in 12 inches for a bar at 48 inches is (8 in./48 in.)(12 in./1 ft) = 2 in/ft.
- When the kd is in the face shell of the wall, there is no difference between solid and grouted walls
- The following diagram shows the impacts that partially grouted walls have on the interaction diagram.

Interaction Diagram

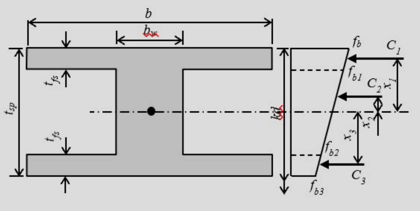
Masonry Designers Guide 2016 Chapter 11 , h = 0



Interaction Diagram – Useful Formulas MDG 2016 Ch 11

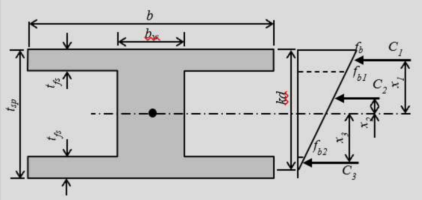
Interaction Diagram Equations for Partially Grouted Walls Loaded Out-of-Plane

$kd > t_{sp}$



$P = C_1 + C_2 + C_3$ $M = C_1x_1 + C_2x_2 + C_3x_3$	$C_1 = \frac{f_b + f_{b1}}{2} bt_{fs}$ $C_2 = \frac{f_b + f_{b1}}{2} b_w(t_{sp} - 2t_{fs})$ $C_3 = \frac{f_{b2} + f_{b3}}{2} bt_{fs}$	$x_1 = \frac{t_{sp}}{2} - \frac{f_b + 2f_{b1}}{f_b + f_{b1}} \frac{t_{fs}}{3}$ $x_2 = \frac{t_{sp}}{2} - t_{fs} - \frac{f_{b1} + 2f_{b2}}{f_{b1} + f_{b2}} \frac{t_{sp} - 2t_{fs}}{3}$ $x_3 = \frac{t_{sp}}{2} - (t_{sp} - t_{fs}) - \frac{f_{b2} + 2f_{b3}}{f_{b2} + f_{b3}} \frac{t_{fs}}{3}$
$f_b = F_b$ $f_{b1} = \frac{kd - t_{fs}}{kd} f_b$	$f_{b2} = \frac{kd - (t_{sp} - t_{fs})}{kd} f_b$ $f_{b3} = \frac{kd - t_{sp}}{kd} f_b$	

$(t_{sp}-t_{fs}) \le kd < t_{sp}$

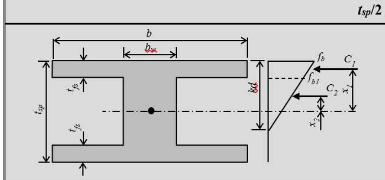


$P = C_1 + C_2 + C_3$ $M = C_1x_1 + C_2x_2 + C_3x_3$	$C_1 = \frac{f_b + f_{b1}}{2} bt_{fs}$ $C_2 = \frac{f_b + f_{b1}}{2} b_w(t_{sp} - 2t_{fs})$ $C_3 = \frac{f_{b2}}{2} b(kd - (t_{sp} - t_{fs}))$	$x_1 = \frac{t_{sp}}{2} - \frac{f_b + 2f_{b1}}{f_b + f_{b1}} \frac{t_{fs}}{3}$ $x_2 = \frac{t_{sp}}{2} - t_{fs} - \frac{f_{b1} + 2f_{b2}}{f_{b1} + f_{b2}} \frac{t_{sp} - 2t_{fs}}{3}$ $x_3 = \frac{t_{sp}}{2} - (t_{sp} - t_{fs}) - \frac{kd - (t_{sp} - t_{fs})}{3}$
$f_b = F_b$ $f_{b1} = \frac{kd - t_{fs}}{kd} f_b$	$f_{b2} = \frac{kd - (t_{sp} - t_{fs})}{kd} f_b$ $f_{b3} = 0$	

Interaction Diagram – Useful Formulas MDG 2016 Ch 11

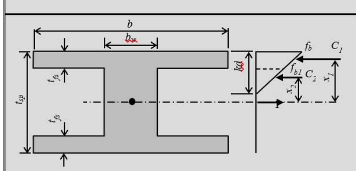
Interaction Diagram Equations for Partially Grouted Walls Loaded Out-of-Plane, cont.

$t_{sp}/2 \le kd < (t_{sp}-t_{fs})$



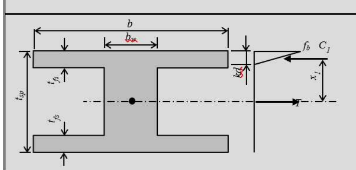
$P = C_1 + C_2$ $M = C_1x_1 + C_2x_2$	$C_1 = \frac{f_b + f_{b1}}{2} bt_{fs}$ $C_2 = \frac{f_{b1}}{2} b_w(kd - t_{fs})$	$x_1 = \frac{t_{sp}}{2} - \frac{f_b + 2f_{b1}}{f_b + f_{b1}} \frac{t_{fs}}{3}$ $x_2 = \frac{t_{sp}}{2} - t_{fs} - \frac{kd - t_{fs}}{3}$
$f_b = F_b$ $f_{b1} = \frac{kd - t_{fs}}{kd} f_b$	$f_{b2} = 0$	

$t_{fs} \le kd < t_{sp}/2$



$P = C_1 + C_2 - T$ $M = C_1x_1 + C_2x_2$	$C_1 = \frac{f_b + f_{b1}}{2} bt_{fs}$ $C_2 = \frac{f_{b1}}{2} b_w(kd - t_{fs})$ $T = f_sA_s$	$x_1 = \frac{t_{sp}}{2} - \frac{f_b + 2f_{b1}}{f_b + f_{b1}} \frac{t_{fs}}{3}$ $x_2 = \frac{t_{sp}}{2} - t_{fs} - \frac{kd - t_{fs}}{3}$
If $k > k_{bal}$ $f_b = F_b$ $f_s = F_bn\frac{t_{sp}/2 - kd}{kd}$	If $k \le k_{bal}$ $f_s = F_s$ $f_b = \frac{F_s}{n} \frac{kd}{t_{sp}/2 - kd}$	$f_{b1} = \frac{kd - t_{fs}}{kd} f_b$

$kd < t_{fs}$



$P = C_1 - T$ $M = C_1x_1$	$C_1 = \frac{f_b}{2} b(kd)$ $T = f_sA_s$	$x_1 = \frac{t_{sp}}{2} - \frac{kd}{3}$
If $k > k_{bal}$ $f_b = F_b$ $f_s = F_bn\frac{t_{sp}/2 - kd}{kd}$	If $k \le k_{bal}$ $f_s = F_s$ $f_b = \frac{F_s}{n} \frac{kd}{t_{sp}/2 - kd}$	$f_{b1} = 0$

Interaction Diagrams

Masonry Design Guide 2016 – Chapter 11
Impact of height

The interaction diagram can be constructed neglecting slenderness effects (*h* = 0). Slenderness effects will reduce the maximum axial load, or put a cap on the interaction diagram. The maximum axial load is shown for different wall heights. An average radius of gyration of *r* = 2.66 in. partially grouted , as given in NCMA TEK 14-1B, is used to calculate slenderness effects

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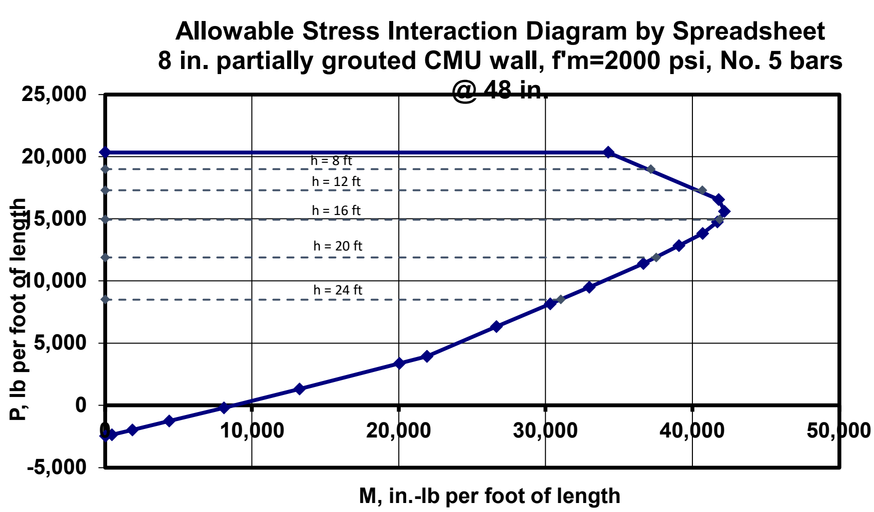
Interaction Diagram

Masonry Designers Guide 2016 Chapter 11



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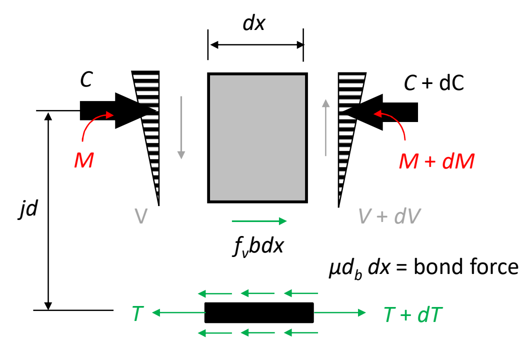
ASD – Reinforced Masonry – Shear



Reinforced Masonry Walls OOP – Shear

Rarely governs wall design OOP

- Shear stress is computed as:

$$f_v = \frac{V}{A_{nv}} \qquad \text{(8-24)}$$

- Allowable shear stresses

$$F_v = (F_{vm} + F_{vs})\gamma_g \qquad \text{(8-25)}$$

$\gamma_g = 0.75$ for partially grouted shear walls, 1.0 otherwise.

Shear Stresses

- Allowable shear stress resisted by the masonry
  - Special reinforced masonry shear walls

$$F_{vm} = \left(\frac{1}{4}\right)\left[4 - 1.75\left(\frac{M}{Vd_v}\right)\right]\sqrt{f'_m} + 0.25\frac{P}{A_n} \quad \text{(8-28)}$$

  - All other masonry

$$F_{vm} = \left(\frac{1}{2}\right)\left[4 - 1.75\left(\frac{M}{Vd_v}\right)\right]\sqrt{f'_m} + 0.25\frac{P}{A_n} \quad \text{(8-29)}$$

M/Vd_v is positive and need not exceed 1.0.

Cut offs

- Allowable shear stress limits:
  - $M/Vd_v \leq 0.25$

$$F_v \leq \left(3\sqrt{f'_m}\right)\gamma_g \quad \text{(8-26)}$$

  - $M/Vd_v \geq 1$

$$F_v \leq \left(2\sqrt{f'_m}\right)\gamma_g \quad \text{(8-27)}$$

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Out of Plane Shear Capacity

- $F_v = (F_{vm} + F_{vs})\gamma_g$ for a Solid grouted 8" CMU wall no shear reinforcing (almost impossible to do in a masonry wall loaded out of plane)
- $F_v = (F_{vm} + 0)1.0$

Ignore Axial Force for now-top of wall critical.

$$F_{vm} = \left(\frac{1}{2}\right)[4 - 1.75(1.0)]\sqrt{2000} + 0 = 1.125\sqrt{f'_m}$$

= 50.3 psi this is less than the 2 $\sqrt{f'_m}$ cut off

Shear capacity per foot = 7.625 x 12 x 50.3 = 4603 lb / ft of wall – this is order of magnitudes lower that shear loads on typical walls

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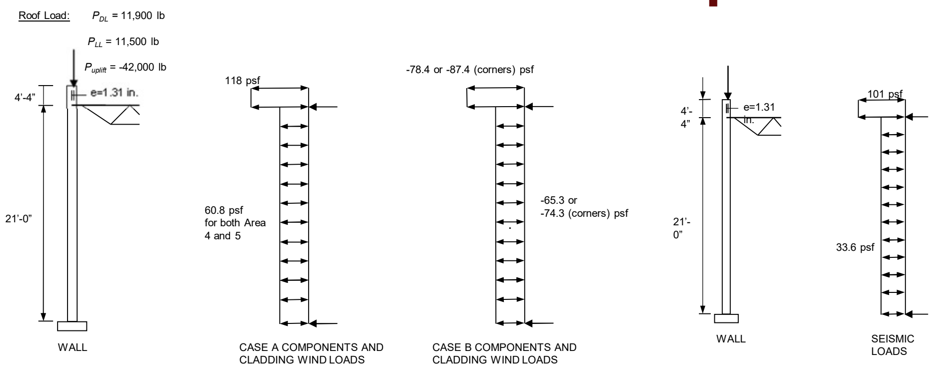
Walls Out of Plane Example 2 – MDG 2016 Box 02

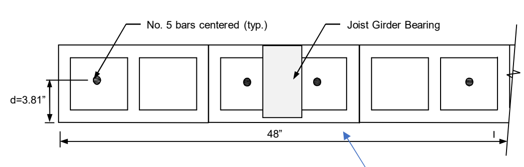


TRY 8" CMU – UNDER JOIST REACTIONS, FULLY GROUTED

Often want to concentrate bars under loads - Uplift Often Governs –
Try an effective width under each reaction as 6 t = 6 x 8 = 48 inches limit of bars effective width – distribution of concentrated load limited 2 to 1 (say at mid height) OK for this as well

Walls Out of Plane Example 2 – MDG 2016 Box 02

Spreadsheet for calculating allowable-stress M-N diagram for solid masonry wall				
21 Ft Wall w/ 4 - No. 5 (Centered)				
total depth, t	7.625	Wall Height, h	21.00	feet
f'm, fprimem	2000	Radius of Gyration, r	2.20	in
Em	1800000	h/r	114.5	
Fb	900	Reduction Factor, R	0.373	
Es	29000000	Allowable Axial Stress, Fa	187	psi (MSJC8.3.4.2.1)
Fs	32000	Net Area, An	91.5	in^2
d	3.8125	Allowable Axial Compr, Pa	17086	lb
kbalanced	0.311828			
tensile reinforcement, As	0.31	Four #5 bars centered below joist seat for 4 ft width		
width, beff	12			

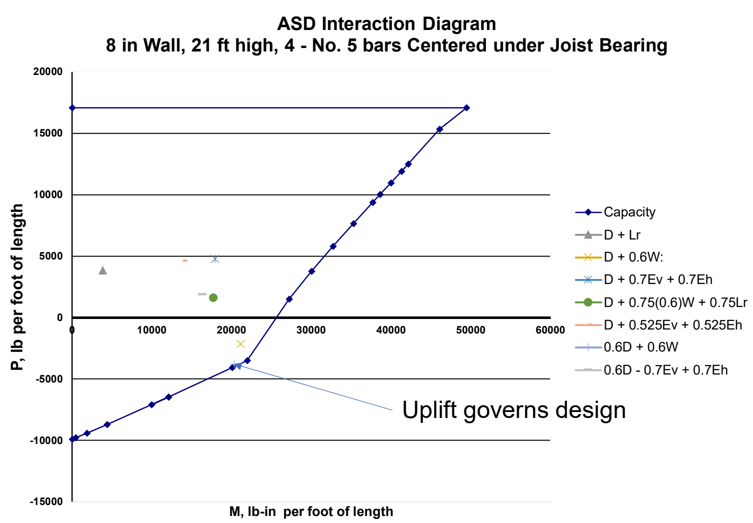


Try an effective with under each reaction as 6 t = 6 x 8 = 48 inches limit of bars effective width – distribution of concentrated load limited 2 to 1 (say at mid height) OK for this as well – this give A_s=0.31 in^2 per foot of wall

Walls Out of Plane Example 2 – MDG 2016 Box 02

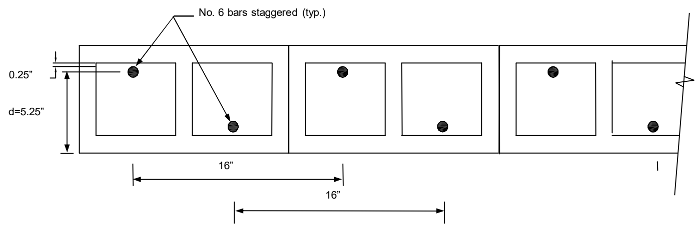
because compression reinforcement is not tied, it is not counted								
	k	kd	fb	Cmas	fs	Axial Force	Moment	Axial Force
			(psi)	(lb)	(psi)	(lb)	(in.-lb)	w/ stress axial limit
Points controlled by steel	0.01	0.04	20	5	-32000	-9915	17	-9915
	0.05	0.19	105	120	-32000	-9800	448	-9800
	0.1	0.38	221	505	-32000	-9415	1861	-9415
	0.15	0.57	351	1203	-32000	-8717	4356	-8717
	0.24	0.92	627	3443	-32000	-6477	12078	-6477
	0.22	0.84	560	2819	-32000	-7101	9960	-7101
	0.24	0.92	627	3443	-32000	-6477	12078	-6477
	0.3	1.14	851	5842	-32000	-4078	20044	-4078
Points controlled by masonry	0.311828	1.19	900	6420	-32000	-3500	21931	-3500
	0.4	1.53	900	8235	-21750	1493	27210	1493
	0.45	1.72	900	9264	-17722	3770	30022	3770
	0.5	1.91	900	10294	-14500	5799	32704	5799
Must change Cmas to trapezoid	0.55	2.10	900	11323	-11864	7645	35255	7645
when kd>t	0.6	2.29	900	12353	-9667	9356	37675	9356
Moment needs to be adjusted	0.62	2.36	900	12764	-8887	10009	38607	10009
	0.65	2.48	900	13382	-7808	10961	39964	10961
	0.68	2.59	900	14000	-6824	11884	41275	11884
	0.7	2.67	900	14411	-6214	12485	42123	12485
	0.8	3.05	900	16470	-3625	15346	46047	15346
	0.9	3.43	900	18529	-1611	18029	49449	17086
Pure compression						45595	0	17086

Walls Out of Plane Example 2 – MDG 2016 Box 02



Walls Out of Plane - Staggard Bars

Often rebar size or moment capacity can be significantly increased by using the highest depth (d) practice
For 8" CMU wall and #6 bar , d can be up to 7.625 – 2 -0 .25 = 5.375 for fine grout or 5.175 for coarse grout



The two layers of bars is for load reversal

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Serviceability – Walls Out-of-Plane

- Deflection Limits IBC says for For Masonry Use TMS 402 or Table 1604.3
  - Exterior walls, under 10 year wind load - 0.42 times C&C Wind loads
    - With plaster or stucco finishes: $l/360$
    - With other brittle finishes: $l/240$
    - With flexible finishes: $l/120$
  - Interior partitions, under 5 psf interior live load
    - With plaster or stucco finishes: $l/360$
    - With other brittle finishes: $l/240$
    - With flexible finishes: $l/120$
- TMS 402 no limits for OOP Walls – Use average section properties – uncrack is OK but more accurate may be determined with cracked allowances. (4.3.2)

Serviceability – Walls Out-of-Plane See Tek Note 14-01B

3a: Horizontal Section Properties (Masonry Spanning Vertically)									
			Net cross-sectional properties[A]			Average cross-sectional properties[B]			
Unit	Grout spacing (in.)	Mortar bedding	A_n (in.^{2}/ft)	I_n (in.^{4}/ft)	S_n (in.^{3}/ft)	A_{avg} (in.^{2}/ft)	I_{avg} (in.^{4}/ft)	S_{avg} (in.^{3}/ft)	r_{avg} (in.)
Hollow	No grout	Face shell	30.0	308.7	81.0	41.5	334.0	87.6	2.84
Hollow	No grout	Full	41.5	334.0	87.6	41.5	334.0	87.6	2.84
100% solid/solidly grouted		Full	91.5	443.3	116.3	91.5	443.3	116.3	2.20
Hollow	16	Face shell	62.0	378.6	99.3	65.8	387.1	101.5	2.43
Hollow	24	Face shell	51.3	355.3	93.2	57.7	369.4	96.9	2.53
Hollow	32	Face shell	46.0	343.7	90.1	53.7	360.5	94.6	2.59
Hollow	40	Face shell	42.8	336.7	88.3	51.2	355.2	93.2	2.63
Hollow	48	Face shell	40.7	332.0	87.1	49.6	351.7	92.2	2.66
Hollow	72	Face shell	37.1	324.3	85.0	46.9	345.8	90.7	2.71
Hollow	96	Face shell	35.3	320.4	84.0	45.6	342.8	89.9	2.74
Hollow	120	Face shell	34.3	318.0	83.4	44.8	341.0	89.5	2.76
3b: Vertical Section Properties (Masonry Spanning Horizontally)									
Hollow	No grout	Face shell	30.0	308.7	81.0	40.5	330.1	86.6	2.86
Hollow	No grout	Full	30.0	308.7	81.0	41.5	334.0	87.6	2.84
100% solid/solidly grouted		Full	91.5	443.3	116.3	91.5	443.3	116.3	2.20
Hollow	16	Face shell	60.8	376.0	98.6	71.2	397.4	104.2	2.36
Hollow	24	Face shell	50.5	353.6	92.7	61.0	374.9	98.3	2.48
Hollow	32	Face shell	45.4	342.4	89.8	55.8	363.7	95.4	2.55
Hollow	40	Face shell	42.3	335.6	88.0	52.8	357.0	93.6	2.60
Hollow	48	Face shell	40.3	331.1	86.9	50.7	352.5	92.5	2.64
Hollow	96	Face shell	35.1	319.9	83.9	45.6	341.3	89.5	2.74
Hollow	120	Face shell	34.1	317.7	83.3	44.6	339.0	88.9	2.76

Table 3–8-inch (203-mm) Single Wythe Walls, 1¼ in. (32 mm) Face Shells (standard)

Deflections

Quick check of deflections:

- Use wind load of 0.42 (-74.3 lb/ft), 1 ft design width = 31.2 lb/ft
- This is over a 21 ft height and ignoring parapet -
- Use the uncracked moment of inertia solid grouted in.^{4}

$$\Delta = \frac{5wh^4}{384EI} = \frac{5\left(31.2\frac{lb}{ft}\right)(21ft)^4 1728\frac{in.^3}{ft^3}}{384(2000\ x\ 900\ psi)(443.3\ in.^4)} = 0.171\ in.$$

Allowable deflection: $\frac{(21ft)\left(12\frac{in.}{ft}\right)}{360} = 0.7in.$

You could use crack sections but not required.

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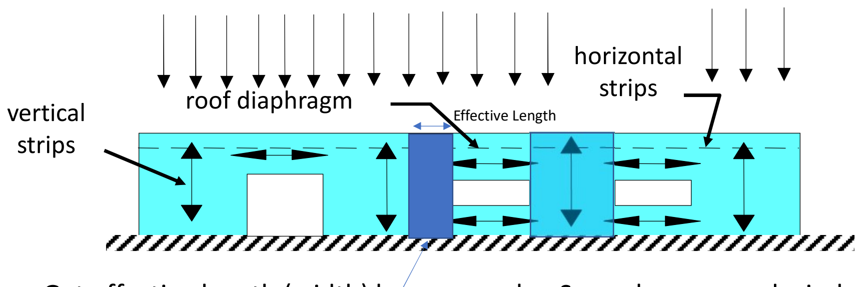
Other Issues on OOP and Bearing Walls

- gravity and out – of – plane loads are resisted by combinations of horizontal and vertical strips



Get effective length (width) large enough – Space bars around window to grab more masonry if needed – Gives more bars and reduces effective M and P

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This concludes The American Institute of Architects Continuing Education Systems Course

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Mark McGinley
m.mcginley@louisville.edu

CONTINUING EDUCATION AIA