Design of Columns and Pilasters & System Behavior

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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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Course Description

Description: Masonry columns and pilasters are often used in masonry buildings to carry large, concentrated loads, and, for pilasters, to support exterior walls subjected to out-of-plane loads. ASD design and detailing required for these elements will be reviewed This session will also look at several sample masonry buildings to illustrate system behavior and overall design methodology.

Learning Objectives

- Introduce masonry column and pilaster elements
- Discuss required detailing of columns and pilasters
- Review the design of masonry columns
- Review the design masonry pilasters

Tonight's Road Map

- Column Design
- Pilaster Design
- More Detailing issues :Volume Change and Jointing

Column Design

Column Design

- TMS 402 Definition (TMS 2.2)
  - Column — A structural member, not built integrally into a wall, designed primarily to resist compressive loads parallel to its longitudinal axis and subject to dimensional limitations.

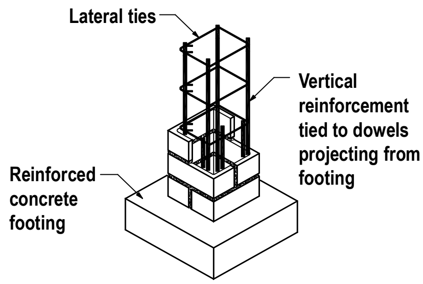
TMS "Strength Design of Masonry"

Column Design

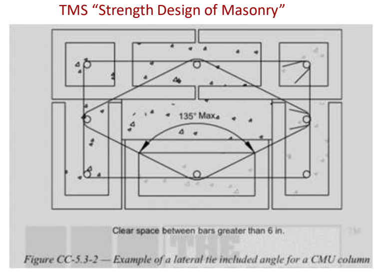
Prescriptive requirements for columns Both ASD and SD Code 5.3 – Columns . . .

Column Design

- TMS 402 Requirements (TMS 5.3)
  - Proportions
    - $h/r \leq 99$
    - 8 in. minimum side dimension
  - Reinforcement
    - Longitudinal max #11 bars
      - 0.0025 A_n minimum
      - 0.04 A_n maximum
      - Other limits such as 6% of grout space , seismic
    - Ties
      - ¼" min diameter
      - Spacing minimum (16$d_{b,long}$, 48 $d_{b,tie}$, least cross-sectional dimension)
      - Support of longitudinal bars



TMS "Strength Design of Masonry"



TMS 402

Column Design

- TMS 402 Requirements (TMS 7.4)
  - SDC C+:

    Participating:

    Where anchor bolts are used to connect horizontal elements to the tops of columns, anchor bolts shall be placed within lateral ties. Lateral ties shall enclose both the vertical bars in the column and the anchor bolts. There shall be a minimum of two No. 4 (M #13) lateral ties provided in the top 5 in. (127 mm) of the column.

  - SDC D+:

    Participating:

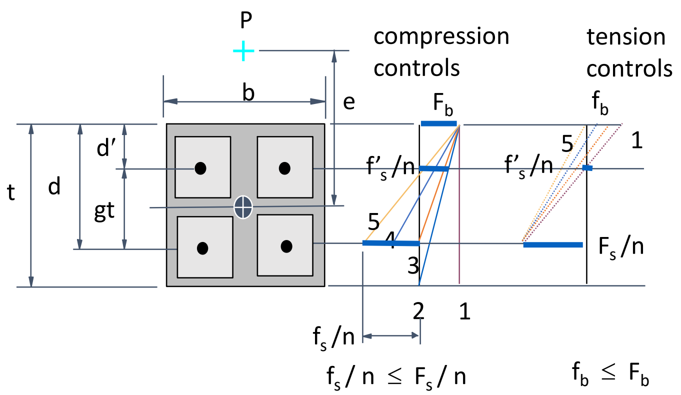
    Lateral ties in masonry columns shall be spaced not more than 8 in. (203 mm) on center and shall be at least 3/8 in. (9.5 mm) diameter.

    Standard hooks for lateral tie anchorage shall be either a 135-degree standard hook or a 180-degree standard hook

    Non-Participating:

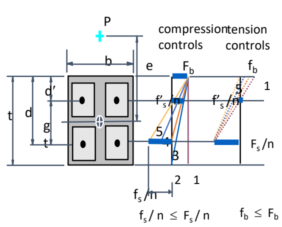
    . . .should also be more heavily tied at the tops and bottoms for more ductile performance and better resistance to shear.

ASD Stress Interaction Diagrams Columns

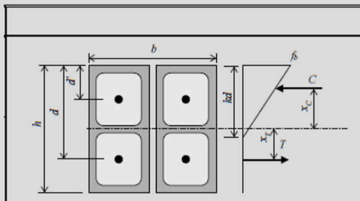
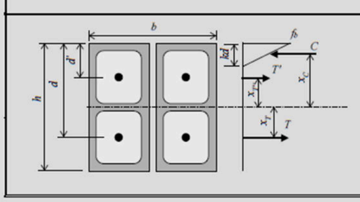


States of Stress - Note the code requires a minimum Eccentricity of 0.1 t - Also Rebar in compression is tied and can be accounted for.

ASD Interaction Diagrams Columns – simplified



Interaction Diagram Equations for Pilaster with Two Layers of Reinforcement

$d' \le kd < d$			
	$P = C - T$ $M = Cx_C + Tx_T$	$C = \frac{1}{2} f_b (kd) b$ $T = A_s f_s$	$x_C = \frac{h}{2} - \frac{kd}{3}$ $x_T = d - \frac{h}{2}$
	If $k > k_{bal}$ $f_b = F_b$ $f_s = F_b n \frac{t_{sp}/2 - kd}{kd}$	If $k \le k_{bal}$ $f_s = F_s$ $f_b = \frac{F_s}{n} \frac{kd}{t_{sp}/2 - kd}$	
$kd < d'$			
	$P = C - T - T'$ $M = Cx_C + Tx_T + T'x_{T'}$	$C = \frac{1}{2} f_b (kd) b$ $T = f_s A_s$ $T' = f_{s'} A'_s$	$x_C = \frac{h}{2} - \frac{kd}{3}$ $x_T = d - \frac{h}{2}$ $x_{T'} = d' - \frac{h}{2}$
	If $k > k_{bal}$ $f_b = F_b$ $f_s = F_b n \frac{t_{sp}/2 - kd}{kd}$	If $k \le k_{bal}$ $f_s = F_s$ $f_b = \frac{F_s}{n} \frac{kd}{t_{sp}/2 - kd}$	$f_{s'} = f_s \frac{d' - kd}{d - kd}$

If you ignore the stress in the compression reinforcing - Use Equations in MDG - Ch 11
Account for steel compression in P_a only - conservative but simpler

Ch. 8.3 in MSJC-ASD Reinforced Masonry

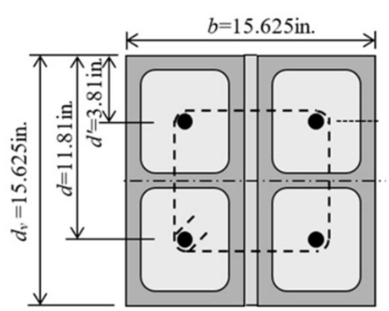
A reminder

Table 11.4.1 Allowable Stresses for Reinforced Masonry

	Equation	TMS 402 Reference
Axial compression	$P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h/r}{140}\right)^2\right]$ $h/r \le 99$ $P_a = (0.25 f'_m A_n + 0.65 A_{st} F_s)\left(\frac{70}{h/r}\right)$ $h/r > 99$ 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)

Column Design - Example

- f'_m = 2000 psi, CMU Column, try 4 #5 rebar
- Ignore rebar in compression except for P_a – 20 ft height



Column Design - Example

- Axial capacity (cont'd)

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{15.625(15.625)^3/12}{(15.625)^2}} = 4.51$$

$$\frac{h}{r} = \frac{12x20}{4.51} = 53.2$$

$$P_a = (0.25f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} \le 99 \quad \text{Eq. 8-18}$$

$$P_a = (0.25(2000)(15.625\ x\ 15.625) + 0.65(4)(.31)(32000))\left[1 - \left(\frac{20x\ 12}{140(4.51)}\right)^2\right] = 126{,}498 \text{ lb}$$

Note that steel gives about ~ 21 kips boost in compression capacity

Column Design - Example

- Note – Must get P and M values for loading for all loading cases
- For each case Check P/M = e is greater than code minimum of 0.1t.

  Or e must be ≥ (0.1) 15.625 = 1.562 in

Develop interaction Diagram - Again ignore compression steel stresses in this as they have only a small impact and it is much less complicated.

Column Design - Example

Spreadsheet for calculating allowable-stress moment-axial force interaction diagram for Column				
Length	15.625	in	**NOTE BASED ON full column**		
f'm	2	ksi	Wall Height, h	20.00	feet
Em	1800	ksi	Radius of Gyration, r	4.51	in
Fb	0.9	ksi	h/r	53.2	
Es	29000	ksi	Reduction Factor, R	0.856	
Fs	32	ksi	Allowable Axial Stress, Fa	428	psi
d	11.81	in	Net Area, An	244.1	in^2
kbalanced	0.311828		Allowable Axial Compr, Pa	126.5	Kips
width	15.625	in			

steel layers are counted from the extreme compression fiber to the extreme tension fiber
distances are measured from the extreme compression fiber

compression in masonry and reinforcement is taken as positive
stress in compressive reinforcement is set to zero and ignored

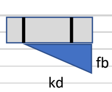
Row of Reinforcement	distance	Area
1	3.81	0.62
2	11.81	0.62

Column Design - Example

$$C_{mas} = 1/2 \times F_b \times kd \times b = 1/2 \times 0.900 \times 11.81 \times 15.625 = 83.0 \text{ kips}$$

$$P = C_{mas} - (A_s \times F_S) = 83.04 = 83.04 \text{ Kips}$$

							Moments are in ft.kips		
							Kip.in	**kips**	
	k	**kd**	**fb**	**Cmas**	**fs(1)**	**fs(2)**	**Moment**	**Axial Force**	
pure axial load							0.00	126.50	0
Points controlled by masonry	1.32	15.59	0.9000	109.61	0.00	0.00	23.90	109.61	286.7548
	1.2	14.17	0.9000	99.65	0.00	0.00	25.65	99.65	307.7594
	1.1	12.99	0.9000	91.34	0.00	0.00	26.51	91.34	318.0714
	1	11.81	0.9000	83.04	0.00	0.00	26.82	83.04	321.8456
	0.9	10.63	0.9000	74.74	0.00	-1.61	26.92	73.74	323.0748
	0.6	7.09	0.9000	49.82	0.00	-9.67	24.63	43.83	295.521
	0.5	5.91	0.9000	41.52	0.00	-14.50	23.22	32.53	278.5846
	0.4	4.72	0.9000	33.22	0.00	-21.75	21.76	19.73	261.0998
	0.38	4.49	0.9000	31.55	0.00	-23.66	21.50	16.89	257.9532
Points controlled by masonry	0.311828	3.68	0.9000	25.89	-0.50	-32.00	20.71	5.74	248.5762
Points controlled by steel	0.311828	3.68	0.9000	25.89	-0.50	-32.00	20.71	5.74	248.5762
	0.3	3.54	0.8512	23.56	-1.03	-32.00	19.42	3.08	232.9959
	0.2	2.36	0.4966	9.16	-4.90	-32.00	10.96	-13.72	131.5112
	0.1	1.18	0.2207	2.04	-7.91	-32.00	6.23	-22.71	74.77534
	0.01	0.12	0.0201	0.02	-10.10	-32.00	4.53	-26.09	54.37947
	0.001	0.01	0.0020	0.00	-10.30	-32.00	4.48	-26.23	53.74751
Pure Tension							0	-39.68	0



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

$$M = 83.04 \times \left(\frac{15.625}{2} - \frac{11.81}{3}\right) - (0) = 321.8 \text{ kip.in}$$

Column Design - Example

$$C_{mas} = 1/2 \times F_b \times kd \times b = 1/2 \times 0.4966 \times 15.625 x 2.36 = 9.16 \text{ kips}$$

	k	kd	fb	Cmas	fs(1)	fs(2)	Moments are in ft.kips		
							Kip.in	kips	
							Moment	Axial Force	
pure axial load							0.00	126.50	0
Points controlled by masonry	1.32	15.59	0.9000	109.61	0.00	0.00	23.90	109.61	286.7548
	1.2	14.17	0.9000	99.65	0.00	0.00	25.65	99.65	307.7594
kd	1.1	12.99	0.9000	91.34	0.00	0.00	26.51	91.34	318.0714
	1	11.81	0.9000	83.04	0.00	0.00	26.82	83.04	321.8456
fb	0.9	10.63	0.9000	74.74	0.00	-1.61	26.92	73.74	323.0748
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	0.5	5.91	0.9000	41.52	0.00	-14.50	23.22	32.53	278.5846
	0.4	4.72	0.9000	33.22	0.00	-21.75	21.76	19.73	261.0998
	0.38	4.49	0.9000	31.55	0.00	-23.66	21.50	16.89	257.9532
Points controlled by masonry	0.311828	3.68	0.9000	25.89	-0.50	-32.00	20.71	5.74	248.5762
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	0.001	0.01	0.0020	0.00	-10.30	-32.00	4.48	-26.23	53.74751
Pure Tension							0	-39.68	0

$$F_b = \frac{\frac{F_{s2}}{n}}{d - kd} \times kd = \frac{\frac{32.0}{16.11}}{11.81 - 2.36} \times 2.36 = 0.4966 \text{ ksi}$$

$$f_{s1} = 4.90 ksi$$

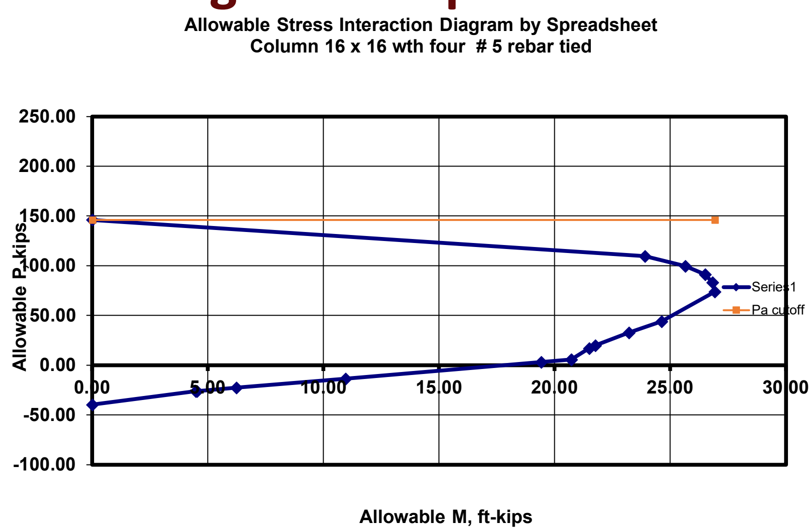
$$P = C_{mas} - (A_s \times F_S) = 9.16 \text{ kips} - (0.62 \times 32) - 0.62(4.90) = -13.7 \text{ kips}$$

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

$$M = 9.16 \times \left(\frac{15.625}{2} - \frac{2.36}{3}\right) + 0.62(-32)(15.625/2 - 11.81) + 0.62(-4.90)(15.625/2 - 3.81)$$

$$= 131.5 kip.\text{in}$$

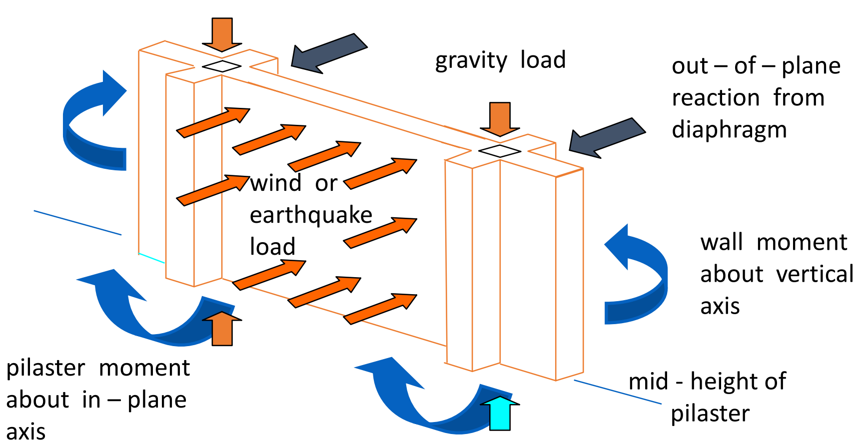
Column Design - Example



. . . Distinction between columns and pilasters -

- Pilasters are thickened sections of masonry walls , are not isolated members , and are not columns
- Pilasters need not be reinforced nor have 4 bars
- If pilaster capacity includes the effect of longitudinal reinforcement , however , that reinforcement must be tied laterally
- MASONRY COLUMNS MUST BE REINFORCED

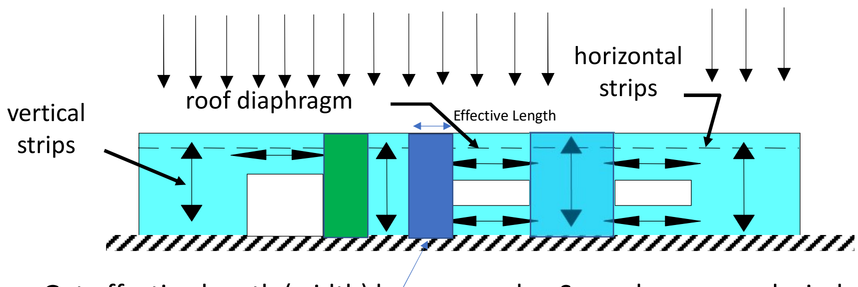
Forces on Pilaster – Wall System



See MDG – Chapter 7

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

MDG 7.5-18 : Examples of Coursing Layouts for Hollow Unit Pilasters

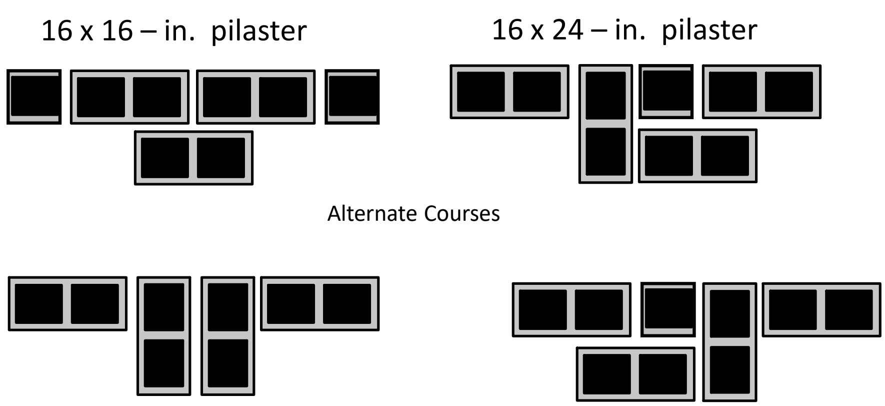


Fig. 7.5-18 Coursing Layout for Hollow Unit Pilasters

Pilaster Design

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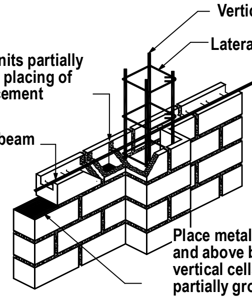
Pilaster Design

- TMS 402 Definition (TMS 2.2)
  - Pilaster - A vertical member, built integrally with a wall, with a portion of its cross section typically projecting from one or both faces of the wall.
- Uses
  - Out-of-plane support for wall spanning horizontally
  - Support concentrated loads
  - Strengthen wall at openings
  - Strengthen end of shear wall

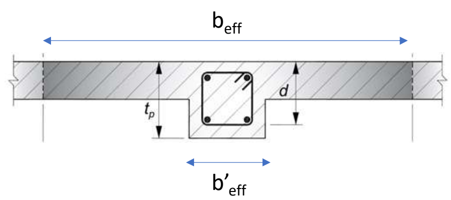


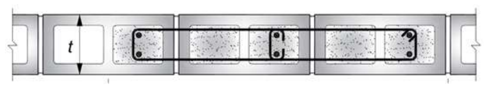
TMS "Strength Design of Masonry"

Figure from RMEH

Pilaster Design

- Types
  - Projecting
    - Treat wall as flange if comply with TMS 402 5.1.1.2.1 and 5.1.1.2.5.
    - Flange width per TMS 5.1.1.2.3.
  - Flush
- Ties only required if:
  - Longitudinal bars used to resist compression
  - Then meet column requirements

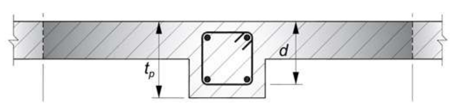


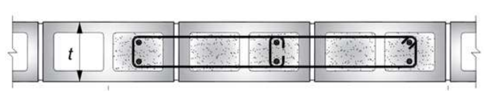


Figures from RMEH

Pilaster Design

- Longitudinal reinforcement
  - No minimum quantity
  - No ρ_{max}
- Can use same techniques as for out-of-plane walls



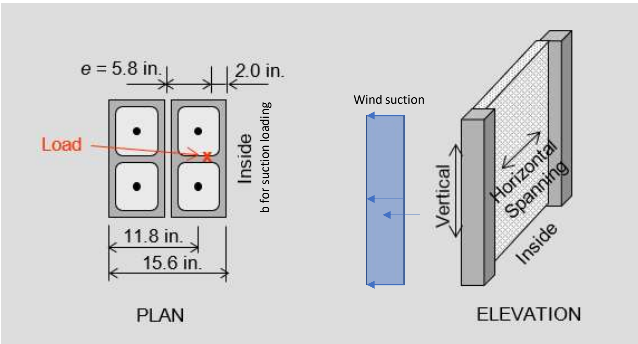


Figures from RMEH

Pilaster Design Example
MDG 2016 – Example 11.4-6

Pilaster Design Example

Design a 24 ft high, 16 in. wide x 16 in. deep CMU pilaster. f'_m = 2,000 psi, grade 60 reinforcement. Dead load = 9.6 kips, Snow load= 9.6 kips, wind uplift = 8.1 kips at an eccentricity of 5.8 in. Wind Load OOP = 26 psf (positive and negative). The pilasters are spaced at 16 ft on center and the wall is assumed to span horizontally between pilasters. The reinforcement is not laterally tied.



Get Loads:

Load combination $0.6D + 0.6W$ usually controls.

Pilaster weight (fully grouted): 75 psf (1.33 ft) (2 wythes) = 200 lb/ft

Out-of-plane wind load on pilaster: 0.6 (26 psf) (16 ft) = 250 lb/ft

Load at top of pilaster, P_f: 0.6 (9,600 lb) – 0.6 (8,100 lb) = 900 lb

Load at mid-height: $P = P_f + P_{wall}$ = 900 lb + 0.6 (200 lb/ft) (12 ft) = 2,340 lb

The maximum moment will occur approximately at the mid-height of the pilaster.

$$M = \frac{wh^2}{8} + \frac{P_f e}{2} = \frac{250\ \text{lb/ft}(24\ \text{ft})^2}{8} + \frac{(900\ \text{lb})(5.8/12\ \text{ft})}{2} = 18{,}200\ \text{lb}-\text{ft} = 218{,}000\ \text{lb}-\text{in.}$$

Generally, the mid-height moment is an adequate approximation. In this example, the difference between the mid-height moment and the maximum moment is less than 0.01%.

Get Trial Area of Steel:

MDG Gives a nice iterative method
I typically assume steel stress governs and look only at moment capacity with an effective moment arm, *jd = 0.9 d*.

For this example $A_{s\ req}$:

$$Asreq = \frac{Mapplied}{jdF_s} = \frac{218{,}000\ \text{lb}-\text{in}}{0.9(11.8)(32{,}000)} = 0.64$$

Try two # 5 bars at 11.8 in. $A_s = 0.62\ \text{in}^2$ just need to be close, place bars on both sides for wind reversal

Set up Spread Sheet – As bars are not tied – Ignore bars in Compression

Spreadsheet for calculating allowable-stress interaction diagram for pilaster									
total depth	15.625	inch							
width	15.625	inch							
f'_m	2,000	psi							
E_m	1,800,000	psi							
F_b	900	psi							
E_s	29,000,000	psi							
F_s	32,000	psi							
d	11.8	inch							
d'	3.8	inch							
$k_{balanced}$	0.311828								
A_s	0.62	in.^2							
$A's$	0.62	in.^2							
h	24	ft						h/r	63.85032

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Pilaster Design - Example

- Axial capacity

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{15.625(15.625)^3/12}{(15.625)^2}} = 4.51$$

$$\frac{h}{r} = \frac{12x24}{4.51} = 63.85$$

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right] \text{ for } \frac{h}{r} \leq 99 \quad \text{Eq. 8-18}$$

$$P_a = (0.25(2000)(15.625\ x\ 15.625))\left[1 - \left(\frac{24x\ 12}{140(4.51)}\right)^2\right] = 96{,}673 \text{ lb}$$

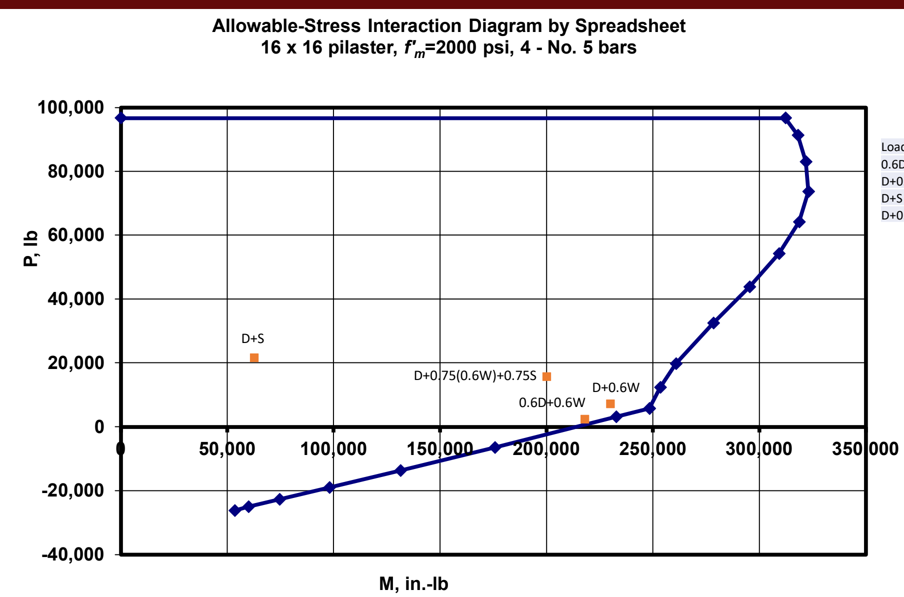
	k	kd	f_b	C	f_s	T	$f_{s'}$	T'	Moment	Axial Force
		(in.)	(psi)	(lb)	(psi)	(lb)	(psi)	(lb)	(lb-in.)	(lb)
Pure compression									0	96,679
Points controlled by masonry	1.165	13.75	900	96,679	0	0	0	0	312,195	96,679
	1.100	12.98	900	91,266	0	0	0	0	318,137	91,266
	1.000	11.80	900	82,969	0	0	0	0	321,850	82,969
	0.900	10.62	900	74,672	1,611	999	0	0	323,019	73,673
	0.800	9.44	900	66,375	3,625	2,248	0	0	318,657	64,128
	0.700	8.26	900	58,078	6,214	3,853	0	0	309,190	54,225
	0.600	7.08	900	49,781	9,667	5,993	0	0	295,331	43,788
	0.500	5.90	900	41,484	14,500	8,990	0	0	278,358	32,494
	0.400	4.72	900	33,188	21,750	13,485	0	0	260,834	19,703
	0.350	4.13	900	29,039	26,929	16,696	0	0	253,465	12,343
	0.312	3.68	900	25,872	32,000	19,840	475	294	248,324	5,738
Points controlled by steel	0.312	3.68	900	25,872	32,000	19,840	475	294	248,324	5,738
	0.300	3.54	851	23,542	32,000	19,840	1,007	625	232,748	3,077
	0.250	2.95	662	15,259	32,000	19,840	3,073	1,906	175,670	-6,487
	0.200	2.36	497	9,155	32,000	19,840	4,881	3,026	131,291	-13,711
	0.150	1.77	351	4,847	32,000	19,840	6,477	4,015	98,006	-19,009
	0.100	1.18	221	2,034	32,000	19,840	7,895	4,895	74,567	-22,700
	0.050	0.59	105	482	32,000	19,840	9,163	5,681	59,986	-25,039
	0.001	0.01	2	0	32,000	19,840	10,283	6,376	53,531	-26,216

For Compression controlling Set f_b = F_b=*900* psi

Set k to a value above k_b, determine f_s and $f_{s'}$ using similar triangles

Get forces , ΣF on section and ΣM about Center line

Below k_b set f_s = F_S get stresses through similar triangles ΣF, ΣM_{CL}



Load Combination	M (lb-in.)	P (lb)
0.6D+0.6W	218,000	2,340
D+0.6W	230,000	7,140
D+S	62,700	21,600
D+0.75(0.6W)+0.75S	200,000	15,600

Pilaster Design Example (flush)
MDG 2016 – Example JHM Box 05 ASD

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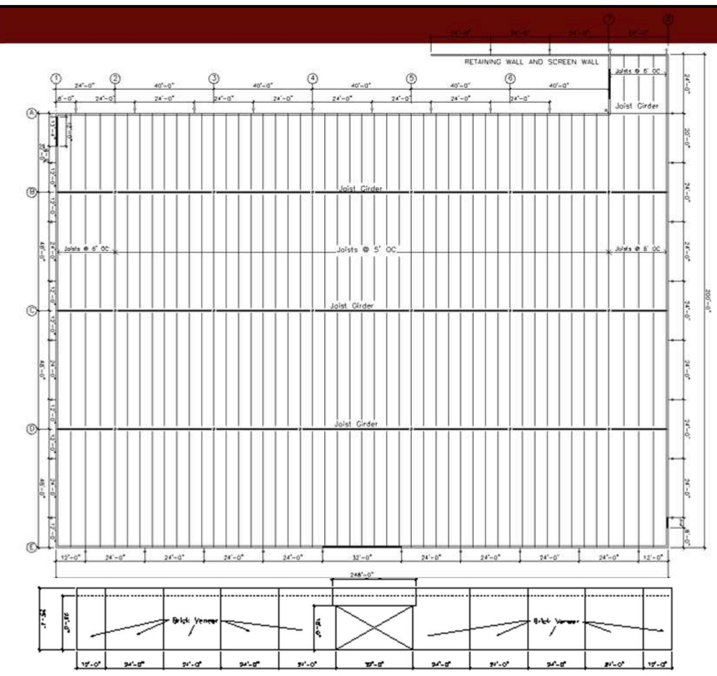

Single Story
Big box Retail

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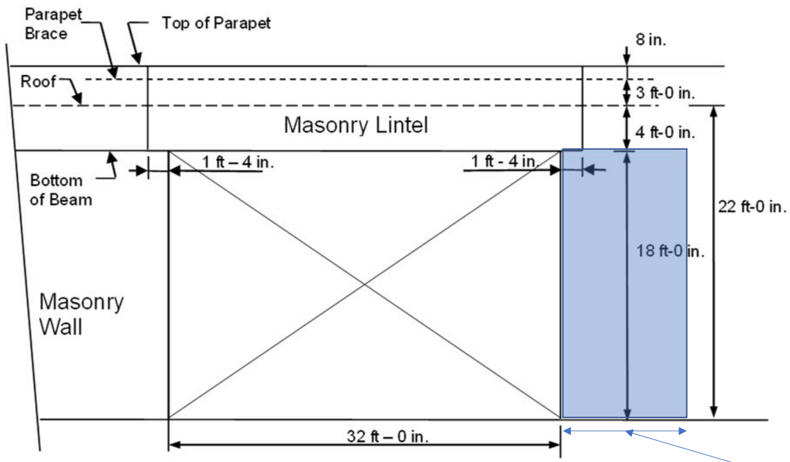
Around Main Door Opening - Use in Wall (flush) Pilaster



Lintel Elevation

Concentrated loads distributed to the critical section at a ratio of 3 vertical to 1 horizontal adjacent to opening but limited to ½ the height of the wall.
Assuming a 16" length to the center of bearing

$L_{effective} = L_{bearing} + 18/3$ ft = 16 in. + 36 in. = 52 in. = 4.33ft

The 4.33 ft long section of wall is resisting the loads.

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Get Loads Based on Effective Length then convert to per foot

The 4.33 ft long section of wall is resisting these loads.

To calculate the effective wall loads per ft of wall, the concentrated loads from the lintel are therefore divided by 4.33 ft before adding to the load effects calculated per ft of wall.
Thus
P_{DL} = 22,600lb/ 4.33ft + 720 lb/ft = 5,900 lb/ft
P_{Lr} = 8,000 lb/ 4.33ft = 1,850lb/ft
P_{WL} = 29,100lb/ 4.33 ft = 6,720 lb/ft (uplift)

D + Lr
P = 5,900 lb/ft + 1,850lb/ft = 7,750 lb /ft
M = 0
D +0 .6W
P= 5,900 lb/ft - (0.6) 6,720 lb/ft= 1,870 lb/ft
M = 0.6(2,650lb-ft /ft) = 1,590lb-ft /ft = 19,100 lb-in./ft ← maximum moment

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Get Loads Based on Effective Length then convert to per foot

D + 0.75 (0.6W) + 0.75 (Lr)

P = 5,900 lb/ft - (0.75)(0.6) 6,720 lb/ft + 0.75(1,850 lb/ft) = 4,260 lb /ft

M =(0.75) 0.6(2,650lb-ft /ft) = 1,190 lb-ft /ft = 14,300 lb-in/ft

0.6D + 0.6W

P = (0.60)5,900 lb/ft - (0.6) 6,720 = -490 lb/ft

M = 0.6(2,650lb-ft /ft) = 1,590 lb-ft /ft =19,100 lb-in. /ft

D + 0.7Ev + 0.7 Eh

P = 5,900 lb/ft + 0.7 (1,240 lb/ft) = 5,900 lb/ft + 870 lb/ft = 6,770 lb/ft

M = 0.7 (2,040) = 1,430lb-ft /ft = 17,200 lb-in./ft

0.6D - 0.7Ev + 0.7 Eh

P = (0.6) 5,900 lb /ft -0.7 (1,240 lb/ft) = 2,670 lb/ft

M = 0.7 (2,040) = 1,430 lb-ft /ft = 17,200lb-in./ft

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Determine axial Capacity per foot

$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

$$= \left(0.25\,(2,000\,\text{psi})(7.63\,\text{in.}\times 12\,\text{in.}) + 0.00\right)\left[1 - \left(\frac{12\ \text{in./in.}\ \times 18\text{ft.}}{140 \times 2.20\ \text{in.}}\right)^2\right]$$

$$= 23,300\,\text{lb /ft of wall} > 7,750\ \text{lb/ft} \qquad \text{Combination 3}$$

Get Estimate of Steel Area

D + 0.6W.

P =1,870 lb/ft   M = 19,100lb-in./ft

A_S = M / $f_s$jd = 19,100lb-in. / (32,000 psi × 0.9× 3.81in.) = 0.17in.^{2}/ft

Try a No.5 bar @ 16 in. o.c. A_s = 0.31 in^2 (12 in./16 in.) = 0.232 in.^{2}/ft

ρ = A_S /bd = 0.232 in.^{2} /(12 in.× 3.81 in.) = 0.005   n = 16.1

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18 ft Wall w/ No. 5 @16. in (Centered)

Total depth, t	7.625	in.	Wall Height, h	18	ft
f'_m	2,000	psi	Radius of Gyration, r	2.20	in.
E_m	1,800,000	psi	h/r	98.2	
F_b	900	psi	Reduction Factor, R	0.508	
E_s	29,000,000	psi	Allowable Axial Stress, F_a	254	psi (TMS 402 Sec. 8.3.4.2.1)
F_s	32,000	psi	Net Area, A_n	91.3	in.^{2}
d	3.8125	in.	Allowable Axial Compr, P_a	23,190	lb
$k_{balanced}$	0.311828				
tensile reinforcement, A_s	0.2325	No. 5 @ 16 in. Centered			
width, b_{eff}	12	in.			

because compression reinforcement is not tied, it is not counted

	k	kd	f_b (psi)	C_{mas} (lb)	f_s (psi)	Axial Force (lb)	Moment (lb-in)	Axial Force w/ stress axial limit
Points controlled by steel	0.01	0.04	20	5	-32,000	-7,435	17	-7,435
	0.05	0.19	105	120	-32,000	-7,320	448	-7,320
	0.1	0.38	221	505	-32,000	-6,935	1,861	-6,935
	0.15	0.57	351	1,203	-32,000	-6,237	4,356	-6,237
	0.24	0.92	627	3,443	-32,000	-3,997	12,078	-3,997
	0.22	0.84	560	2,819	-32,000	-4,621	9,960	-4,621
	0.24	0.92	627	3,443	-32,000	-3,997	12,078	-3,997
	0.3	1.14	851	5,842	-32,000	-1,598	20,044	-1,598
Points controlled by masonry	0.311828	1.19	900	6,420	-32,000	-1,020	21,931	-1,020
	0.4	1.53	900	8,235	-21,750	3,178	27,210	3,178
	0.5	1.91	900	10,294	-14,500	6,923	32,704	6,923
	0.6	2.29	900	12,353	-9,667	10,105	37,675	10,105
change C_{mas} to trapezoid	0.7	2.67	900	14,411	-6,214	12,966	42,123	12,966
when $kd > t$	0.8	3.05	900	16,470	-3,625	15,627	46,047	15,627
Moment needs to be adjusted	0.9	3.43	900	18,529	-1,611	18,154	49,449	18,154
	1.2	4.58	900	24,705	0	24,705	56,513	23,190
	1.4	5.34	900	28,823	0	28,823	58,606	23,190
	1.6	6.10	900	32,940	0	32,940	58,606	23,190
	1.8	6.86	900	37,058	0	37,058	56,513	23,190
	2	7.63	900	41,175	0	41,175	52,327	23,190
Pure compression						45,634	0	23,190

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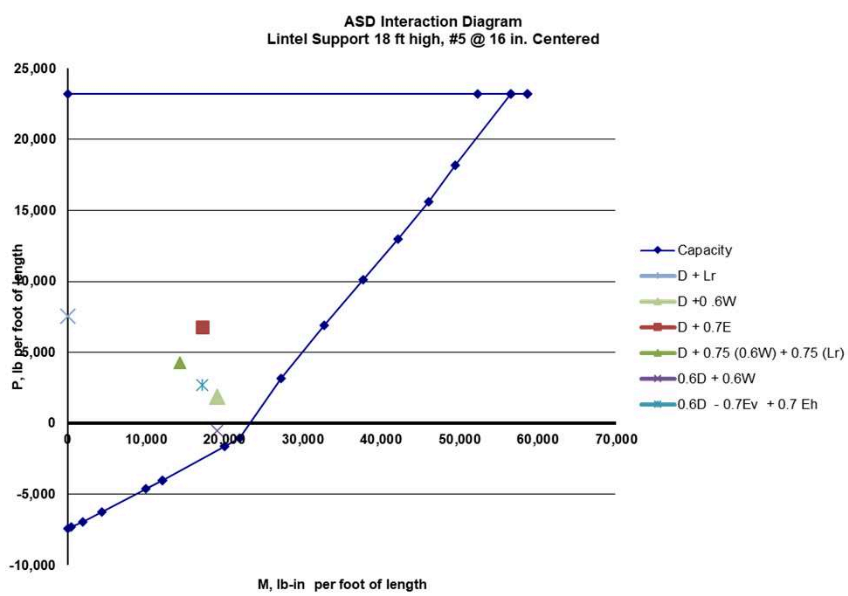

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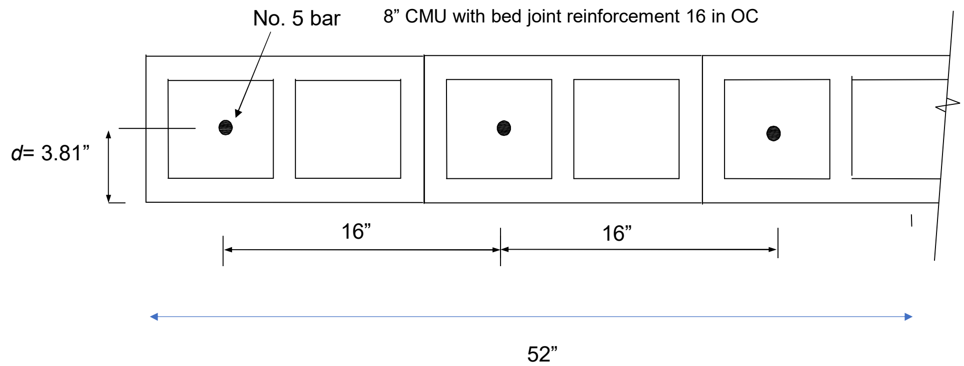
Thus the wall section support shown below will be used to support the door lintel.

Use a Solidly Grouted Section on each Side of the Opening



Volume Change and Jointing

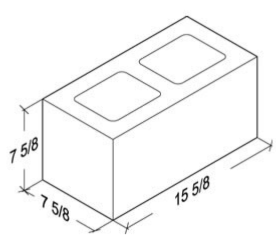
Volume Change and Jointing

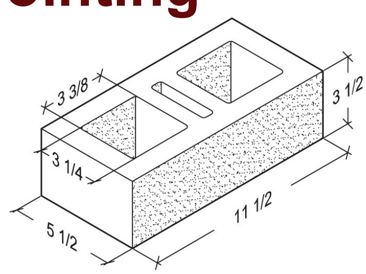
TMS 402/602

4.1.5 *Other effects*

Consideration shall be given to effects of forces and deformations due to prestressing, vibrations, impact, shrinkage, expansion, temperature changes, creep, unequal settlement of supports, and differential movement.

Volume Change and Jointing





Concrete Units:
- Cementitious product
- Shrinks over time

Joint = Control Joint

Clay Units:
- Kiln fired ceramic
- Expands as it gains moisture

Joint = Expansion Joint

Figures from Mutual Materials

Volume Change and Jointing

Concrete Masonry

Comparison to cast-in-place concrete

- Amount of movement (units are preshrunk)
- Amount of reinforcing
  - Cast-in-place wall: 0.0012 to 0.0015 (ACI 318-14, Table 11.6.1)
  - Masonry:
    - Non-participating – may have no horizontal reinforcing
    - Participating:
      - Ordinary ~ 0.00022
      - Special – 0.0007 (0.0015 if not laid in running bond)

Volume Change and Jointing

Concrete Masonry

Resources and Recommendations

- NCMA Technical Notes
  - 10-01A – Control of Cracking
  - 10-02D – Empirical Method
  - 10-03 – Alternative Engineered Method
- Regional Recommendations

Volume Change and Jointing

If Joints Provided

	Maximum Length-to-Height Ratio of Concrete Masonry Panel	Maximum spacing, in. (mm)
Above Grade Concrete Masonry Walls		
Nominal Unit Height: 8 in. (203 mm)[2]	1.5 to 1	25 ft. (7.62 m)
Nominal Unit Height: 4 in. (102 mm)[3]	1.5 to 1	20 ft. (6.10 m)

[1]Adjust spacing as needed where local experience or project conditions warrant.
[2]Include horizontal reinforcement having an equivalent area of not less than 0.025 in.^{2}/ft. (52.9 mm^2/m) of height. See Table 2A.
[3]Include horizontal reinforcement having an equivalent area of not less than 0.034 in. ^{2}/ft. (72.0 mm^2/m) of height. See Table 2B.

Reinforcement size	Maximum spacing, in. (mm)
W1.7 (9 gage) (MW11)[1]	16 (406)
W2.1 (8 gage) (MW13)[1]	16 (406)
W2.8 (3/16 in.) (MW18)[1]	24 (610)
No. 3 (M#10)	48 (129)
No. 4 (M#13)	96 (2,348)
No. 5 (M#16) or larger	144 (3,658)

[1] Minimum two wires per course.

NCMA Technical Note 10-02D

Minimum Reinforcement for No Joints

Table 1—Maximum Spacing of Horizontal Reinforcement to Meet the Criteria $A_s > 0.002A_n$[1]

Wall thickness, in. (mm)	Maximum spacing of horizontal reinforcement, in. (mm) Reinforcement size		
	No.6(M19)	No.5(M16)	No.4(M13)
Ungrouted or partially grouted walls			
6(152)	48(1219)	48(1219)	32(813)
8(203)	48(1219)	40(1016)	24(610)
10(254)	48(1219)	32(813)	16(406)
12(305)	48(1219)	24(610)	8(203)
Fully grouted walls			
6(152)	32(813)	24(610)	16(406)
8(203)	24(610)	16(406)	8(203)
10(254)	16(406)	16(406)	8(203)
12(305)	16(406)	8(203)	8(203)

1. A_s includes cross-sectional area of grout in bond beams

NCMA Technical Note 10-01A

Volume Change and Jointing

Clay Masonry

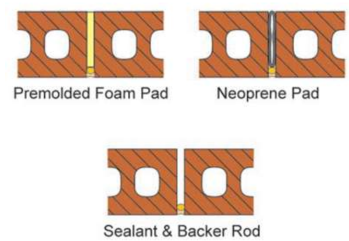
Resources and Recommendations

- BIA Technical Notes
  - 18 – Volume Changes - Analysis and Effects of Movement
  - 18A – Accommodating Expansion of Brickwork
    - Joint spacing should not exceed:
      - 25’ if no openings
      - 20’ with openings

Volume Change and Jointing

Joint Sizing

Previous references provide recommendations on determining the magnitude of movement that needs to be accommodated at the joints.



BIA Technical Note 18A

Must also consider sealant compressability / extensibility. Typical values:

- 50% Compressibility
- 50% to 100% Extensability

Confirm with specified product

This concludes The American Institute of Architects Continuing Education Systems Course

THE MASONRY SOCIETY

The Masonry Society

Mark McGinley
m.mcginley@louisville.edu

CONTINUING EDUCATION AIA