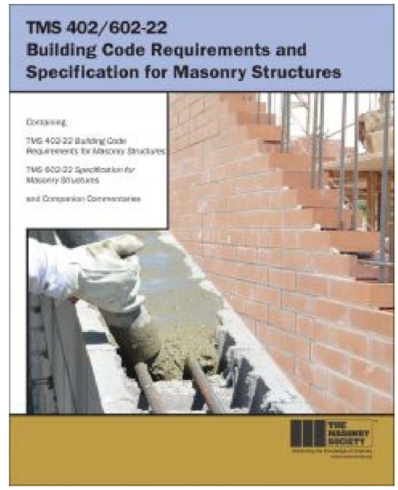
The 2022 TMS 402/602: What is New



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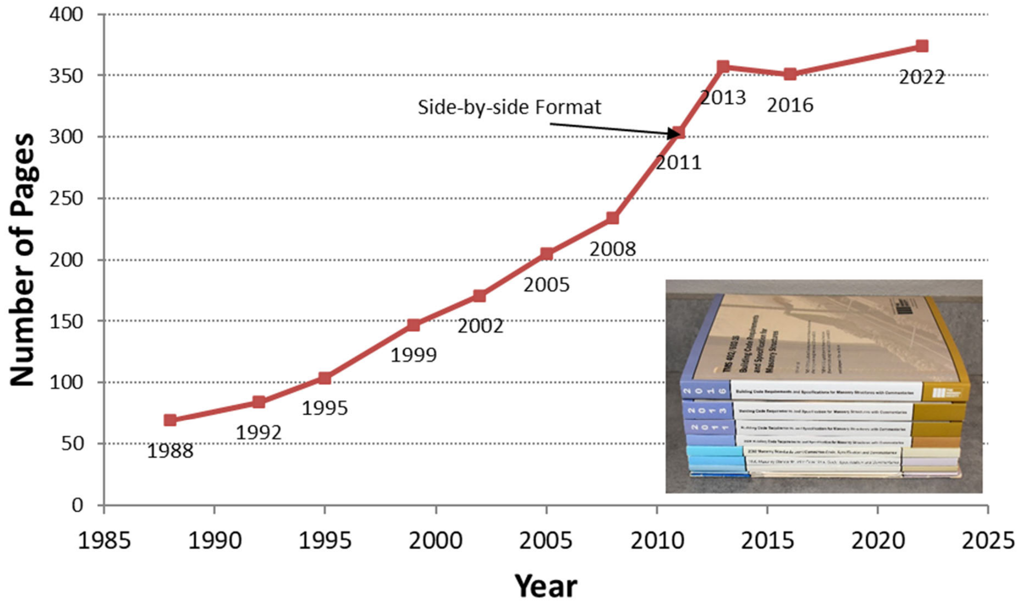
2nd Vice Chair, 2022 TMS 402/602 Code Committee
Vice, Chair, 2028 TMS 402/602 Code Committee

Seminar Description

This seminar will give an overview of the changes to the 2022 TMS 402/602 Building Code Requirements and Specification for Masonry Structures. Some of the major changes are the addition of compression-controlled sections in strength design, the addition of an Appendix for GFRP reinforced masonry, and a complete rewrite of the veneer chapter. An overview of the changes will be provided along with practical examples for implementing the changes.



- Tension and compression-controlled sections in strength design
- Veneer chapter completely rewritten
- New Appendix on GFRP reinforcement

Changes by Chapter

Chapter	Name	Minor	Moderate	Major	Extreme
1	General Requirements	●			
2	Notations/Definitions	●			
3	Quality & Construction		●		
4	General Analysis & Design		●		
5	Structural Members		●		
6	Reinforcement, Metal Accessories & Anchor Bolts			●	
7	Seismic Design Requirements			●	
8	Allowable Stress Design		●		
9	Strength Design			●	
10	Prestressed Masonry	●			
11	Autoclaved Aerated Masonry	●			

Changes by Chapter

Chapter	Name	Minor	Moderate	Major	Extreme
12	Design of Masonry Infills (Prev App B)	●			
13	Veneer (Previously Chapter 12)				●
14	Glass Unit Masonry (Previously Ch 13)	●			
15	Partition Walls (Previously Chapter 14)	●			
App A	Empirical Masonry (Deleted)				●
App B	Masonry Infill (Moved to Chapter 12)				
App C	Limit Design	●			
App D	Glass Fiber Reinforced Polymer Reinf			NEW!	
Spec 1	General	●			
Spec 2	Products			●	
Spec 3	Execution		●		

Chapter 3 - Quality and Construction

- Section 3.2 (Construction Considerations) deleted
  - Minimum grout space already in Specification (Article 3.5)
  - Pipes, conduits and sleeves more appropriately moved to Code Design (4.8)

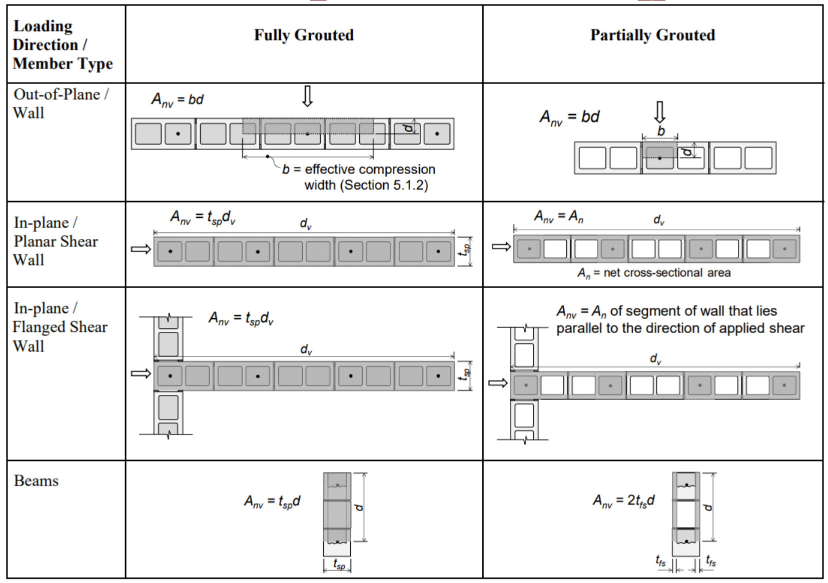
Chapter 4 - General Analysis & Design

- Masonry Compressive Strength limitations moved from Strength Design and AAC Design to General Design, so it now applies to all masonry design (4.3)

Type of Masonry	Specified compressive strength of masonry	Specified compressive strength of grout
Concrete masonry	$f'_m \leq 4{,}000$ psi	$f'_g \geq f'_m \leq 5{,}000$ psi
Clay masonry	$f'_m \leq 6{,}000$ psi	$f'_g \leq 6{,}000$ psi
AAC masonry	$f'_m > 290$ psi	$2{,}000 \text{ psi} \leq f'_g \leq 6{,}000$ psi

Chapter 4 - General Analysis & Design

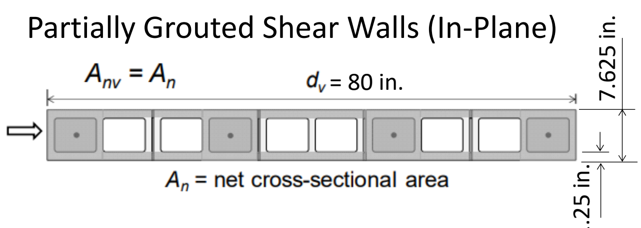
- New table showing Net Shear Area for partially and fully grouted members (including beams) (4.4.5)



Loading Direction / Member Type	Fully Grouted	Partially Grouted
Out-of-Plane / Wall	$A_{nv} = bd$; b = effective compression width (Section 5.1.2)	$A_{nv} = bd$
In-plane / Planar Shear Wall	$A_{nv} = t_{sp}d_v$	$A_{nv} = A_n$; A_n = net cross-sectional area
In-plane / Flanged Shear Wall	$A_{nv} = t_{sp}d_v$	$A_{nv} = A_n$ of segment of wall that lies parallel to the direction of applied shear
Beams	$A_{nv} = t_{sp}d$	$A_{nv} = 2t_{fs}d$

Net Shear Area

Partially Grouted Shear Walls (In-Plane)



$$A_{nv} = 2t_{fs}d_v + n_{cell}(8\text{ in.})(t_{sp} - 2t_{fs})$$

$$= 2(1.25\ in.)(80 in.) + 4(8\ in.)(7.625\ in. - 2\ (1.25\ in.))$$
$$= 364\ in.^2$$

t_{fs} = face shell thickness
t_{sp} = specified wall thickness

$$A_{nv} = t_{eq}d_v$$

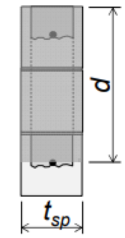
Grout Spacing (in.)	Equivalent Thickness, t_{eq} (in.)	
	8 in.	12 in.
16	5.17	7.28
24	4.28	5.69
32	3.83	4.89
40	3.57	4.41
48	3.39	4.09
72	3.09	3.56

$$A_{nv} = 4.28\ in.(80\ in.) = 342\ in.^2$$

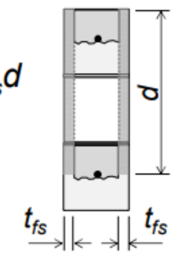
Net Shear Area: Beams

- Clarified A_{nv} for beams is calculated using d, not d_v
- Partially grouted beams are allowed
- Beams need to be fully grouted if shear reinforcement is required

$A_{nv} = t_{sp}d$



$A_{nv} = 2t_{fs}d$



Chapter 4 - General Analysis & Design

- Moved Section 5.2.1.4.1 to General Design Section 4.6 to clarify deflection limitation of $\ell/600$ under D + L for all beams (masonry, steel, concrete) supporting unreinforced masonry.

- Moved Section 3.2 (embedded pipes, conduits and sleeved) to new Section (4.8).

Chapter 5 - Structural Members

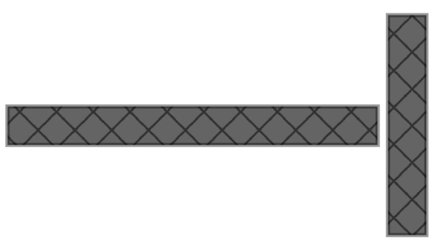
- Reorganized for Clarity

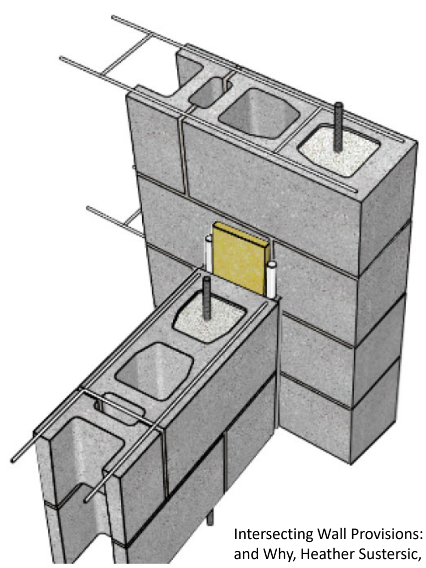
2016 TMS 402	2022 TMS 402
5.1 Masonry Assemblies	5.1 General
5.2 Beams	5.2 Walls
5.3 Columns	5.3 Beams
5.4 Pilasters	5.4 Columns
5.5 Corbels	5.5 Pilasters
	5.6 Corbels

Wall Intersections

- 5.2 Walls
  - 5.2.1 Independent Walls

  No force transfer

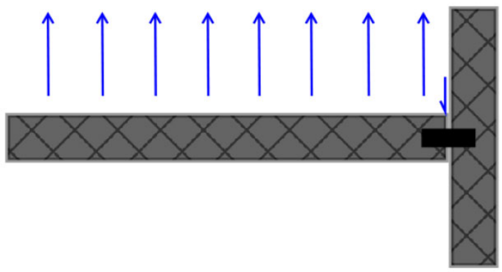


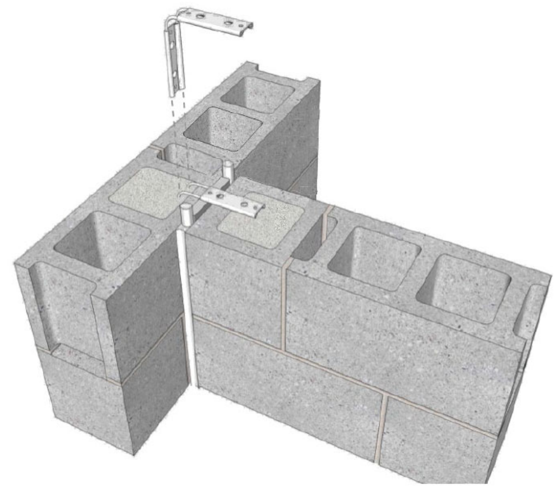


Intersecting Wall Provisions: What Changed in TMS 402-22 and Why, Heather Sustersic, 2022 TMS Annual Meeting

Wall Intersections

- 5.2 Walls
  - 5.2.2 Lateral support for walls without composite action at intersection

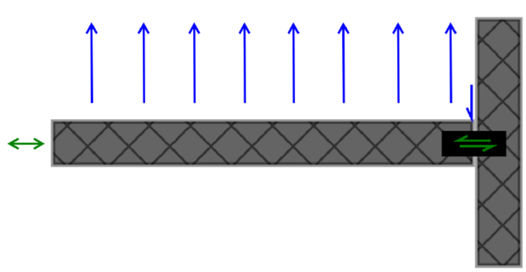


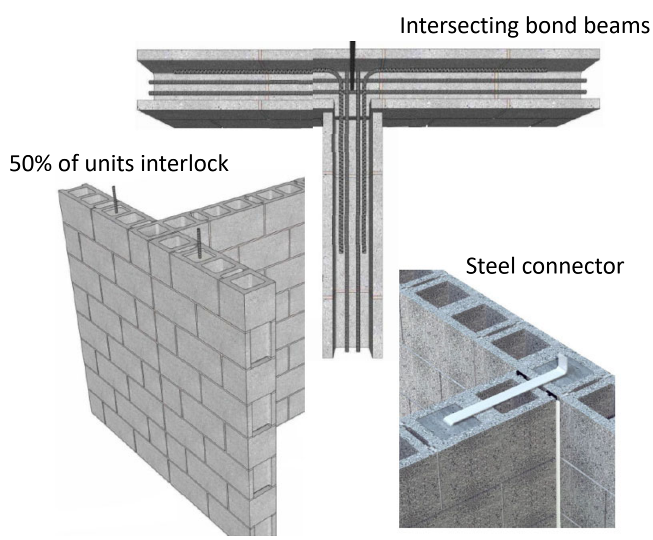


Intersecting Wall Provisions: What Changed in TMS 402-22 and Why, Heather Sustersic, 2022 TMS Annual Meeting

Wall Intersections

- 5.2 Walls
  - 5.2.3 Intersections with composite action





Masonry can be in other than running bond (stack bond) when connected by intersecting bond beams

Intersecting Wall Provisions: What Changed in TMS 402-22 and Why, Heather Sustersic, 2022 TMS Annual Meeting

Chapter 6 - Reinforcement, Metal Access.

- Clarity of Deformed Wire Requirements
- Deformed Bar Size Consistent-ASD/SD
- Size of Reinforcement in Grout
- Hooks for Shear Reinforcement
- Development Length of Hooks

Deformed Wire; Maximum Bar Size

- Clarity of Deformed Wire Requirements (6.1 Various)
- Deformed Bar Size Consistent-ASD/SD (6.1.3.2.1)
  - Maximum bar size No. 11
  - 2016 requirements
    - No. 11 in ASD
    - No 9 in SD

Deformed Wire Properties

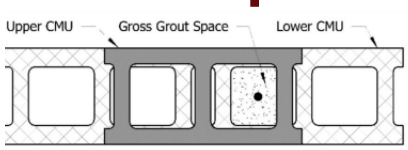
Designation	Nominal Diameter (inch)	Nominal Area, in.^{2}
D 1	0.113	0.010
D 2	0.160	0.020
D 3	0.195	0.030
D4	0.226	0.040
D 5	0.252	0.050
D 6	0.276	0.060
D 7	0.299	0.070
D 11	0.374	0.110
D 20	0.505	0.200
D 31	0.628	0.310

Size of Reinforcement in Grout

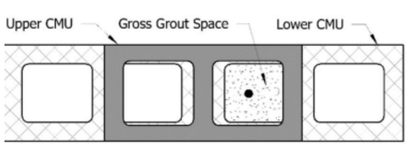
- 2016: Differing requirements in ASD and Strength Design
- 2022: Harmonized to four requirements (Chapter 6).
  - maximum size No. 11 (6.1.3.2.1)
  - one-eighth the least nominal member dimension (6.1.3.2.3).
  - one-third the least dimension of the gross grout space (6.1.3.2.4).
  - 4% of the gross grout space for clay and concrete masonry except 8% at laps (6.1.3.2.5).

Gross grout space: area within the continuous grouted cell, core, bond beam course, or collar joint, considering the effect of unit offset in adjacent courses but neglecting possible mortar protrusions and the presence of perpendicular reinforcement, if any.

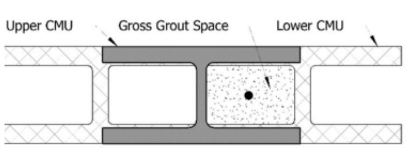
Gross Grout Space



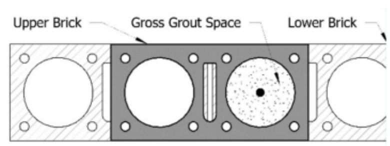
(a) Flanged units laid in one-half running bond



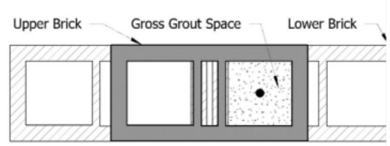
(b) Jamb units laid in one-half running bond



(c) Open-end units laid in one-half running bond



(d) Circular core units laid in one-half running bond



(e) Rectangular core units laid in one-half running bond

Maximum Vertical Reinforcement

Table CC-6.1.3.2.5.2.1: One-Half Running Bond Two-Celled Hollow Concrete of Clay Masonry

Nominal Unit Thickness	Maximum Vertical Reinforcement per Cell		
	Flanged Units	Jamb Units	Open-End Units
6 in.	1 - #6 or 2 - #4	1 - #6 or 2 - #5	1 - #6 or 2 - #5
8 in.	1 - #7 or 2 - #5	1 - #8 or 2 - #6	1 - #8 or 2 - #6
10 in.	1 - #8 or 2 - #6	1 - #9 or 2 - #6	1 - #10 or 2 - #7
12 in.	1 - #9 or 2 - #6	1 - #10 or 2 - #7	1 - #11 or 2 - #8

Table CC-6.1.3.2.5.2.1: Stack Bond Two-Celled Hollow Concrete of Clay Masonry

Nominal Unit Thickness	Maximum Vertical Reinforcement per Cell		
	Flanged Units	Jamb Units	Open-End Units
6 in.	1 - #6 or 2 - #5	1 - #6 or 2 - #5	1 - #6 or 2 - #6
8 in.	1 - #8 or 2 - #6	1 - #8 or 2 - #6	1 - #8 or 2 - #7
10 in.	1 - #9 or 2 - #6	1 - #9 or 2 - #6	1 - #10 or 2 - #8
12 in.	1 - #10 or 2 - #7	1 - #11 or 2 - #8	1 - #11 or 2 - #8

Hooks for Shear Reinforcement

2016 TMS 402

6.1.7.1 *Horizontal shear reinforcement*

6.1.7.1.1 Except at wall intersections, the end of a horizontal reinforcing bar needed to satisfy shear strength requirements of Section 9.3.4.1.2 (Strength Design) shall be bent around the edge vertical reinforcing bar with a 180-degree standard hook.

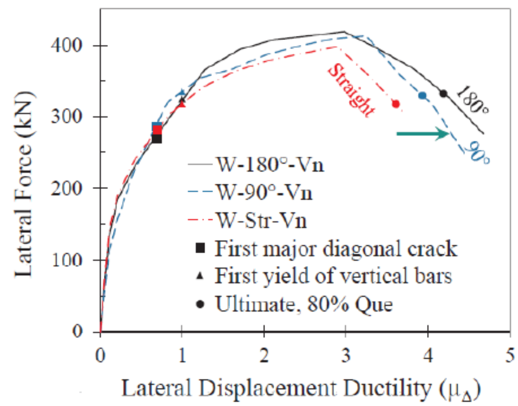
6.1.7.1.2 At wall intersections, horizontal reinforcing bars needed to satisfy shear strength requirements of Section 9.3.4.1.2 (Strength Design) shall be bent around the edge vertical reinforcing bar with a 90- degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.

2022 TMS 402

Hooks for Shear Reinf.: The Research

Hoque (2013): The tests showed no significant difference in strength due to changes in the bond beam anchorage type from straight to 180° hooks.

Rizaee (2015): The results of this research and comparisons to past studies showed no beneficial effect of having 180° hooks at the ends of horizontal rebar over having it straight, having 90° hooks, or having studded ends.



Seif ElDin, H.M., and Galal, K. (2017)

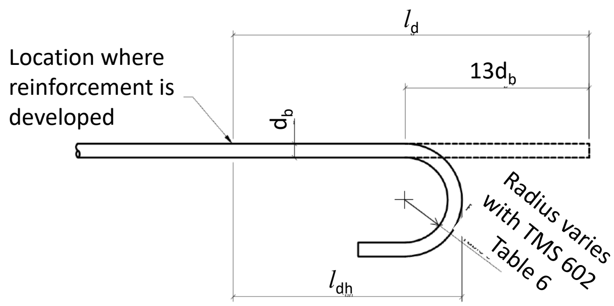
Hooks: Development Length

2016: Equivalent embedment length of $13d_b$

2022: Required development length: $l_{dh} = l_d - \gamma_h d_b$

$\gamma_h = 9.0$ for No. 3 through No. 8 bars

$\gamma_h = 8.0$ for No. 9 through No. 11 bars



Chapter 7 - Seismic Design Requirements

- Exception for Isolating Non-Participating Elements
- Joint Reinforcement Used for Shear
- Shear Capacity Design (Special Reinforced Shear Walls)
- Hooks in Special Reinforced Shear Walls
- Non-participating Element Seismic Steel

Non-Participating Elements

7.3.1 *Nonparticipating elements* — Masonry elements that are not part of the seismic-force-resisting system shall be classified as nonparticipating elements and shall be isolated in their own plane from the seismic-force-resisting system ~~except as required for gravity support~~. Isolation joints and connectors shall be designed to accommodate the design story drift.

Exception was added.

Non-Participating Elements

Exception: Isolation is not required if a deformation compatibility analysis demonstrates that the non-participating element can accommodate the inelastic displacement, Δ, of the structure in a manner complying with the requirements of this Code. Elements supporting gravity loads in addition their self-weight shall be evaluated for gravity load combinations of $(1.2D + 1.0L + 0.15S)$ or $0.9D$, whichever is critical, acting simultaneously with the inelastic displacement and shall have a ductility compatible with the ductility of the lateral force resisting system. The influence of any non-isolated nonparticipating elements on the lateral force resisting system shall be considered in design in accordance with Section 4.1.6 of this code.

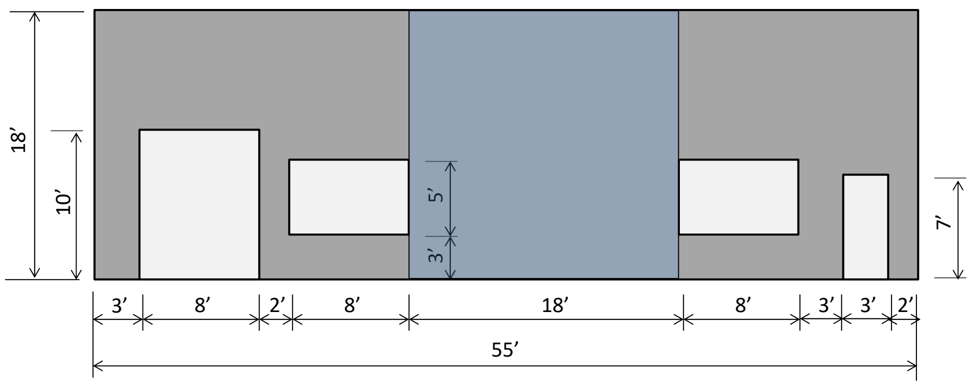
Non-Participating Elements: Exception

My opinion:

- A true deformation analysis is difficult; either isolate or consider as a participating element.
- The exception justifies some common "by inspection" cases which have been routinely used in design.

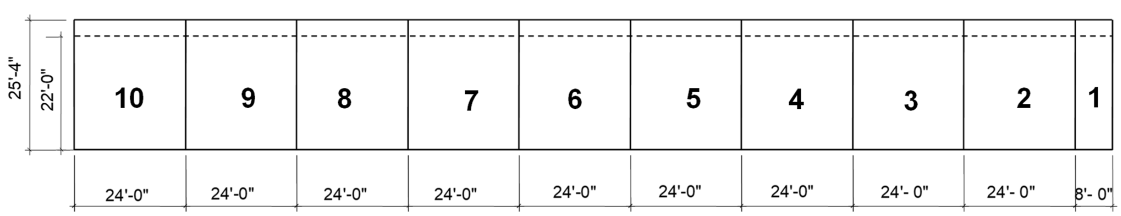
Non-Participating Elements: Exception

Solid portion takes 85% of shear; just consider it



1. Need to provide prescriptive seismic reinforcement everywhere.
2. Need drag strut/collector to get shear to solid portion.

Non-Participating Elements: Exception



North Wall Elevation

Segment 1 takes 0.7% of the load.

Reasonable approximation is to divide in-plane shear equally between segments 2 through 10.

Joint Reinforcement Used for Shear

TMS 402-16

9.3.3.4 *Joint reinforcement used as shear reinforcement*

- Always: (2) 3/16 in. diameter longitudinal wires
- SDC A and B: spacing 16 in. maximum
- SDC C+:
  - Partially Grouted: Spacing 8 in. max
  - Fully Grouted: (4) 3/16 in. diameter longitudinal wires, max 8 in. spacing

TMS 402-22

7.4.1 *Seismic Design Category A*

7.4.1.2.1 *Joint reinforcement used as shear reinforcement*

7.4.3 *Seismic Design Category C+*

7.4.3.2.6 *Joint reinforcement used as shear reinforcement*

On face same as TMS 402-16, but:

- Applies to ASD
- Applies to seismic participating elements only

Joint Reinforcement Used for Shear

TMS 402-16

7.3.2.6 *Special reinforced masonry shear walls*

(b) The maximum spacing of horizontal reinforcement required to resist in-plane shear shall be uniformly distributed, shall be the smaller of one-third the length of the shear wall and one-third the height of the shear wall, and **shall be embedded in grout**.

TMS 402-22

7.3.2.5 *Special reinforced masonry shear walls*

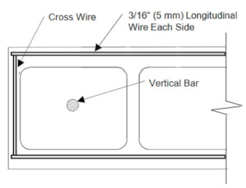
(e) Joint reinforcement and deformed wire placed in mortar required to resist in-plane shear shall be a single piece **without splices** for the length of the wall used for shear design, d_v.

Joint Reinforcement Used for Shear

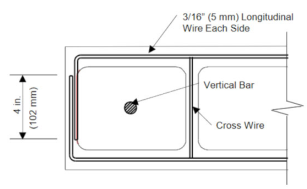
7.3.2.5 *Special reinforced masonry shear walls*

(g) Joint reinforcement used as shear reinforcement shall be anchored in accordance with Section 6.1.8.1.3.1 (a) or (c) when two longitudinal wires are used and Section 6.1.8.1.3.2 when four longitudinal wires are used.

<u>(2) Wires</u>

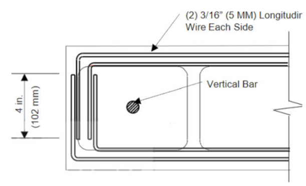


6.1.8.2.1 (a)

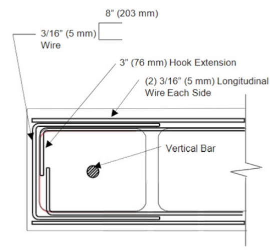


6.1.8.2.1 (c)

<u>(4) Wires</u>



6.1.8.2.2(c)



6.1.8.2.2(a)

Shear Capacity Design: ASD

<u>TMS 402-16</u>

7.3.2.6.1 *Shear capacity design*

7.3.2.6.1.2 When designing special reinforced masonry shear walls in accordance with Section 8.3.5, the calculated shear stress, f_v, or diagonal tension stress resulting from in-plane seismic forces shall be increased by a factor of **1.5**.

<u>TMS 402-22</u>

7.3.2.5.1 *Shear capacity design*

7.3.2.5.1.1 When designing special reinforced masonry shear walls in accordance with Section 8.3.5, the calculated shear stress, f_v, or diagonal tension stress resulting from in-plane seismic forces shall be increased by a factor of **2.0**.

Shear Capacity Design: ASD

TMS 402-16

8.3.5.1.3 The allowable shear stress resisted by the masonry, F_{vm}, shall be calculated using **Equation 8-25 for special reinforced masonry shear** walls and using Equation 8-26 for other masonry:

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.20\frac{P}{A_n}$$

(Equation 8-25)

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.20\frac{P}{A_n}$$

(Equation 8-25)

TMS 402-22

8.3.5.1.3 The allowable shear stress resisted by the masonry, F_{vm}, shall be calculated using Equation 8-23:

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.20\frac{P}{A_n}$$

(Equation 8-23)

Shear Capacity Design: SD

TMS 402-16

7.3.2.6.1 *Shear capacity design*

7.3.2.5.1.1 When designing special reinforced masonry shear walls to resist in-plane forces in accordance with Section 9.3, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed **2.5** times required shear strength, V_u.

TMS 402-22

7.3.2.5.1 *Shear capacity design*

7.3.2.5.1.2 When designing special reinforced masonry shear walls to resist in-plane forces in accordance with Section 9.3, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the design shear strength, ϕV_n, need not exceed **2.0** times required shear strength, V_u.

Hooks in Shear Reinforcement

TMS 402-16

7.3.2.6 *Special reinforced masonry shear walls*

(d) Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.

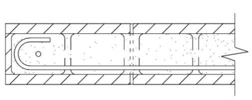
TMS 402-22

7.3.2.5 *Special reinforced masonry shear walls*

(i) When the ratio of V/F_{vm} for masonry designed in accordance with Chapter 8 or when the ratio $V_u/\phi V_{nm}$ for masonry designed in accordance with Chapter 9 exceeds **0.40**, the termination of horizontal reinforcement embedded in grout shall meet one of the following:

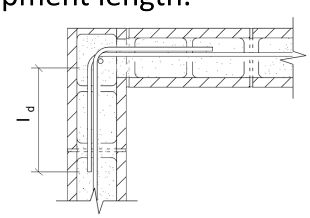
Hooks in Shear Reinforcement

1. Except at wall intersections, the ends of horizontal reinforcement shall be bent around the edge vertical reinforcement with a 180-degree standard hook.



WALL END

2. At wall intersections, horizontal reinforcement shall be bent around the edge vertical reinforcement with a 90-degree standard hook and shall extend horizontally into the intersecting wall a minimum distance at least equal to the development length.



WALL INTERSECTION

Non-Participating Seismic Steel

TMS 402-16

7.4.3 *Seismic Design Category C*

7.4.3.1 *Design of nonparticipating elements* — Nonparticipating masonry elements . . . shall be reinforced in **either the horizontal or vertical direction** . . .

TMS 402-22

7.4.3.1 *Design of nonparticipating elements* — Nonparticipating masonry elements . . . shall be reinforced in **the direction of span** . . .

Horizontal Reinforcement: Joint reinforcement at 16 in. or No. 4 at 48 in.
Vertical Reinforcement: No. 4 at 120 in.

Chapter 8 - Allowable Stress Design

- Masonry Allowable Axial Compressive Force increased from 0.25 to 0.30 (8.3.4.2.1)
- Axial Load Masonry Shear Strength Reduced 0.25 to 0.20 (8.3.5.1.3)
- Beam Shear Force Moved to Chapter 5 to Apply to All Beams

Chapter 8 - Allowable Stress Design

- Masonry Allowable Axial Compressive Force increased from 0.25 to 0.30 (8.3.4.2.1)

$$h/r \le 99 \qquad P_a = (\mathbf{0.30} f'_m A_n + 0.65 A_{st} F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right] \qquad \text{Equation 8-16}$$

$$h/r > 99 \qquad P_a = (\mathbf{0.30} f'_m A_n + 0.65 A_{st} F_s)\left(\frac{70r}{h}\right)^2 \qquad \text{Equation 8-17}$$

- Axial Load Masonry Shear Strength Reduced 0.25 to 0.20 (8.3.5.1.3)

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + \mathbf{0.20}\frac{P}{A_n} \qquad \text{Equation 8-23}$$

Chapter 8 - Allowable Stress Design

- Section 8.3.5.4 (Beam Shear) moved to Section 5.3.1.5

~~8.3.5.4~~ <u>5.3.1.5</u> *Shear* — In cantilever beams, the maximum shear shall be used. In noncantilever beams, the maximum shear shall be used except that sections located within a distance $d/2$ from the face of support shall be permitted to be designed for the same shear as that calculated at a distance $d/2$ from the face of support when the following conditions are met:

(a) support reaction, in direction of applied shear force, introduces compression into the end regions of the beam, and

(b) no concentrated load occurs between face of support and a distance $d/2$ from face.

Chapter 8 & 9 – ASD & SD

- Anchor Bolt Design changed from Yield to Ultimate Strength (8.1.4.3, 9.1.6.3)
- Partially Grouted Shear Wall Factor Decreased 0.75 to 0.70 (8.2.5.1.2, 9.3.3.1.2)
- Shear Friction Factor Changed from 0.6 to 0.75 (8.3.6); Shear Friction Harmonized (9.3.5.5)

Chapter 8 & 9 – Anchor Bolt Strength

TMS 402-16 Commentary:

9.1.6.3 Anchors conforming to A307, Grade A specifications are allowed by the Code, but the ASTM A307, Grade A specification does not specify a yield strength. Use of a yield strength of 37 ksi in the Code design equations for A307 anchors will result in anchor capacities similar to those obtained using the American Institute of Steel Construction provisions.

9.1.6.3.1.1 Steel strength is calculated using the effective tensile stress area of the anchor (that is, including the reduction in area of the anchor shank due to threads).

Anchor Bolt Steel Strength

Anchor bolt steel strength changed from being based on f_y to being based on f_u .

	Strength Design	Allowable Stress Design
Tensile Strength	$B_{ans} = A_b f_u \quad \phi = 0.75$ Equation 9-2	$B_{as} = 0.5 A_b f_u$ Equation 8-2
Shear Strength	$B_{vns} = 0.6 A_b f_u \quad \phi = 0.65$ Equation 9-7	$B_{vs} = 0.25 A_b f_u$ Equation 8-7

The value of f_u shall not be taken greater than the smaller of $1.9f_y$ and 125,000 psi (862 MPa).

Anchor Bolt Steel Strength

TMS 402 6.3.8 Effective Cross-Sectional Area

$$A_b = \frac{\pi}{4}\left(d_o - \frac{0.9743}{n_t}\right)^2$$

d_o = nominal anchor diameter
n_t = number of threads per inch

Bolt size – threads per inch	A_b (in.^{2})
1/2 – 13	0.142
5/8 – 11	0.226
3/4 – 10	0.334
7/8 – 9	0.462
1 – 8	0.606

Chapter 8 & 9 – Grouted Shear Wall Factor

Reinforced Masonry Shear Strength

$$F_v = (F_{vm} + F_{vs})\gamma_g \qquad \text{Equation 8-20}$$

$$V_n = (V_{nm} + V_{ns})\gamma_g \qquad \text{Equation 9-15}$$

γ_g changed from **0.75** to **0.70**
for partially grouted shear walls

Chapter 8 – Shear Friction Strength

Shear Span Ratio	Allowable Shear Friction	
	2016	2022 (8.3.6)
$\frac{M}{Vd_v} \leq 0.5$	$F_f = \frac{\mu(A_{sp}F_s + P)}{A_{nv}}$	
$\frac{M}{Vd_v} \geq 1.0$	$F_f = \frac{0.65(\mathbf{0.6}A_{sp}F_s + P)}{A_{nv}}$	$F_f = \frac{0.65(\mathbf{0.75}A_{sp}F_s + P)}{A_{nv}}$

For $0.5 < \frac{M}{Vd_v} < 1.0$

Linear Interpolation

- $\mu = 1.0 \quad F_f = \frac{\left(0.488+1.024\left(1-\frac{M}{Vd_v}\right)\right)A_{sp}F_s+\left(0.65+0.70\left(1-\frac{M}{Vd_v}\right)\right)P}{A_{nv}}$
- $\mu = 0.7 \quad F_f = \frac{\left(0488+0.424\left(1-\frac{M}{Vd_v}\right)\right)A_{sp}F_s+\left(0.65+0.10\left(1-\frac{M}{Vd_v}\right)\right)P}{A_{nv}}$

Chapter 9 – Shear Friction Strength

Shear Span Ratio	Nominal Shear Friction Strength	
	2016	2022 (9.3.5.5)
$\frac{M_u}{V_u d_v} \leq 0.5$	$V_{nf} = \mu(A_{sp} f_y + P_u)$	
$\frac{M_u}{V_u d_v} \geq 1.0$	$V_{nf} = 0.42 f'_m A_{nc}$	$V_{nf} = 0.65(0.75 A_{sp} f_y + P_u)$

$0.5 < \frac{M_u}{V_u d_v} < 1.0$

Linear Interpolation

$$\mu = 1.0 \quad V_{nf} = \left(0.488 + 1.024\left(1 - \frac{M_u}{V_u d_v}\right)\right) A_{sp} f_y + \left(0.65 + 0.70\left(1 - \frac{M_u}{V_u d_v}\right)\right) P_u$$

$$\mu = 0.7 \quad V_{nf} = \left(0488 + 0.424\left(1 - \frac{M_u}{V_u d_v}\right)\right) A_{sp} f_y + \left(0.65 + 0.10\left(1 - \frac{M_u}{V_u d_v}\right)\right) P_u$$

Chapter 9 - Strength Design

Will Cover Later

- Added Compression Controlled Sections for Combinations of Flexure and Axial Load (9.1.4.4)
- Maximum Reinforcement (Except Beams, Intermediate and Special Reinforced Masonry Shear Walls) Deleted
- Modified Cracked Moment of Inertia Formula (Eq 9-28)

Cracked Moment of Inertia: OOP Loading

2016 Equation 9-30

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

Centered Reinforcement:

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

2022 Equation 9-28

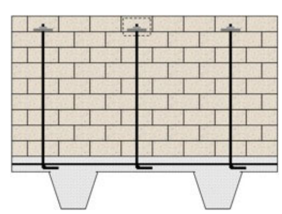
$$I_{cr} = nA_s(d-c)^2 + \frac{nP_u}{f_y}\left(\frac{t_{sp}}{2} - c\right)^2 + \frac{bc^3}{3}$$

Centered Reinforcement:

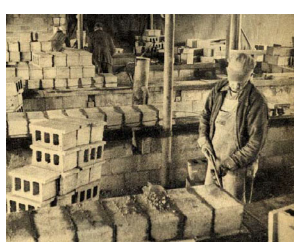
$$\boxed{I_{cr} = n\left(A_s + \frac{P_u}{f_y}\right)(d-c)^2 + \frac{bc^3}{3}}$$

Chapter 10 - Prestressed Masonry

- New equation for laterally restrained and unrestrained walls (10-1)
- Added section for Design of Beams and Lintels (10.6)



Fun Fact: First prestressed bridge in U.S. was a prestressed masonry bridge in Madison County, TN; built in 1951



Blocks being mortared

Prestressing of masonry beams

Prestressed masonry beams lifted into place

Chapter 13-Masonry Veneer

- Completely rewritten in 2022
- Anchored veneer
  - Prescriptive provisions simplified
  - Simplified engineered provision
- Adhered veneer
  - Polymer modified mortar required for prescriptive design
  - Provisions expanded and clarified

Clay Brick with Wood Frame Backing

TMS 402-16

12.2.2.6.1 Anchored veneer with a backing of wood framing shall not exceed 30 ft, or 38 ft at a gable, in height above the location where the veneer is supported.

TMS 402-22

13.1.2.2.2 *Wood light frame backing —* Exterior veneer tied connected to wood light frame construction exceeding 30 ft, or 38 ft at a gable, in height above the vertical support shall be designed and detailed to accommodate differential movement.

Courtesy of Andy Dalrymple

Vertical Differential Movement at a window head, four story building constructed in 2003

Anchored Veneer – Prescriptive Design

Minimum Permitted Design Method

p_{veneer} (psf)	Permitted Design Method	
	SDC A, B, and C	SDC D and higher
≤ 50	Basic Prescriptive	Enhanced Prescriptive
> 50 and ≤ 75	Enhanced Prescriptive	
> 75	Engineered	

Prescriptive Design

	Basic	Enhanced
Maximum tributary area per tie	2.67 ft^2 (24 in. x 16 in.)	1.78 ft^2 (16 in. x 16 in.)
Maximum spacing	24 in.	16 in.

Anchored Veneer – 2016 Engineered Design

12.2.1 *Alternative design of anchored masonry veneer*

The alternative design of anchored veneer, which is permitted under Section 1.3, shall satisfy the following conditions:

(a) Loads shall be distributed through the veneer to the anchors and the backing using principles of mechanics.

(b) Out-of-plane deflection of the backing shall be limited to maintain veneer stability.

(c) The veneer is not subject to the flexural tensile stress provisions of Section 8.2 or the nominal flexural tensile strength provisions of Section 9.1.9.2.

(d) The provisions of Section 12.1 (General Requirements), Section 12.2.2.9 (Veneer not laid in running bond), and Section 12.2.2.10 (Requirements in seismic areas) shall apply.

Stability of Backing (13.2.1.5)

Deemed to comply (Table 13.2.1.5) or a stability analysis with a factor of safety of 1.5

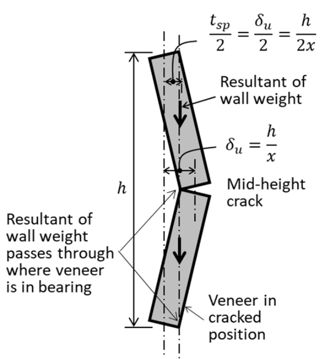


Table 13.2.1.5

h_b/t_{sp}	h_b for $t_{sp} = 3.625$ in.	Maximum Deflection of the Backing for Stability	
		Wind[1], δ_{ser}	Seismic[2], δ_u
67	20.2 ft	h_b / 240	h_b / 100
100	30.2 ft	h_b / 360	h_b / 150
133	40.2 ft	h_b / 480	h_b / 200
167	50.4 ft	h_b / 600	h_b / 250

[1] Under application of 0.42W. Applicable to backing whose stiffness is the same for service level and strength level wind loads. Otherwise evaluate using W and seismic deflection limits.

[2] Under application of strength level seismic load.

Stability of Backing (13.2.1.5)

Examples: (3-⅝ inch thick brick)

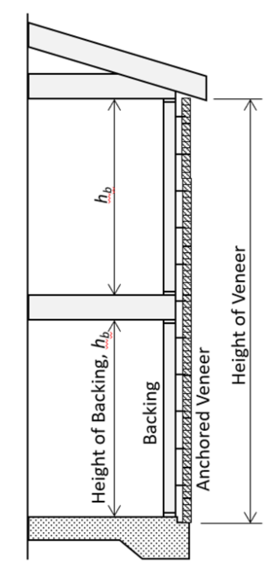
1. Steel stud backing designed for h_b / 360 under service level wind loads.

h_b = 30.2 ft

2. Reinforced CMU backing designed for deflection of $0.007h_b$ under ASD loads. Assume deflection of $0.014h_b$ under strength level loads, or h_b / 71.

From commentary: $\frac{h_b}{t_{sp}} = \frac{2}{3}(71) = 47.3$

h_b = 14.3 ft

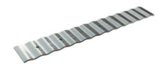


Engineered Design: Tributary Area Method (13.2.3.2)

Tie Stiffness	Tie Force
$k_{tie} \leq 2500$ lb/in.	$2p_uA_t$
2500 lb/in. < $k_{tie} \leq 5000$ lb/in.	$2.5p_uA_t$
5000 lb/in. < $k_{tie} \leq 8000$ lb/in.	$3p_uA_t$
$k_{tie} > 8000$ lb/in.	$4p_uA_t$

Table 13.2.3.1

Veneer Tie	Design Strength	Stiffness
Corrugated	125 lb	500 lb/in.
Adjustable - slotted	330 lb	3000 lb/in.
Adjustable – two leg pintle	210 lb	2500 lb/in.


Corrugated Tie


Adjustable - Slotted


Adjustable – Two Leg Pintle

Tributary Area: Example

Risk Category II Building, Memphis TN, V =105 mph, S_{DS} = 0.6, SDC = D, p_{veneer} = 36.8 psf

- **Prescriptive**: Enhanced due to SDC = D; **16 in. x 16 in.** veneer tie spacing
- Tributary area:
  - Slotted veneer tie: k_{tie} = 3000 lb/in. (factor = 2.5); design strength = 330 lb

    Solve for $A_{trib} = \frac{T}{2.5p_u} = \frac{330\text{ lb}}{2.5(36.8\text{ psf})} = 3.58\text{ ft}^2$

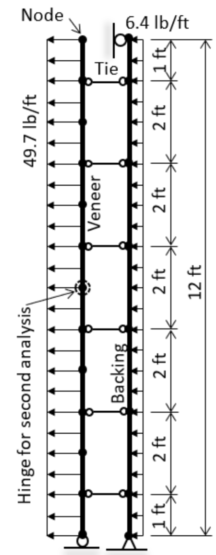
    Use **16 in. x 32 in.** veneer tie spacing ($A_{trib} = 3.56\text{ ft}^2$)
  - Two-leg pintle tie: k_{tie} = 2500 lb/in. (factor = 2.5); design strength = 210 lb

    Solve for $A_{trib} = \frac{T}{2.5p_u} = \frac{210\text{ lb}}{2.5(36.8\text{ psf})} = 2.28\text{ft}^2$

    Use **16 in. x 16 in.** veneer tie spacing ($A_{trib} = 1.78\text{ ft}^2$) For 16 x 24, $A_{trib} = 2.67\text{ ft}^2$

Engineered Design: Modeling Analysis Method (13.2.3.3)

- Backing: Beam elements; simply supported
- Veneer: Beam elements; pinned at bottom, free at top
- Veneer ties: Spring elements (or axial elements)
- Pseudo nonlinear analysis
  - If modulus of rupture is exceeded (veneer cracks), replace with a hinge
  - Rerun model until all tensile stresses in veneer are below modulus of rupture



Modeling Analysis: Example

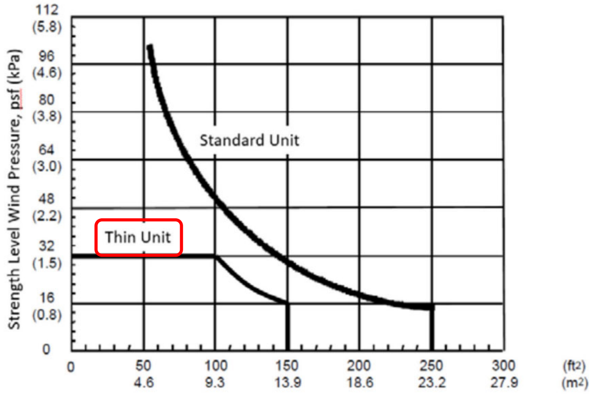
Risk Category II Building, Memphis TN, V =105 mph, S_{DS} = 0.6, SDC = D, p_{veneer} = 36.8 psf

- Modeling Analysis: Try a 16 in. horizontal spacing x 24 in. vertical spacing
  - Two-leg pintle tie: k_{tie} = 2500 lb/in. (factor = 2.5); design strength = 210 lb; backing stiffness assumed to be $h_b/360$ under 0.42W.
  - First run: Maximum veneer moment $2.42M_{cr}$ at midheight; place a hinge there
  - Second run: Maximum veneer moment $0.37M_{cr}$
  - Maximum tie force of 166 lb in ties 1, 3, 4, and 6 from the bottom
  - Use **16 in. horizontal x 24 in. vertical** spacing

Lower veneer tie stiffness and higher backing stiffness result in more uniform distribution of loads to veneer ties.

Chapter 14 - Glass Unit Masonry

- Panel Size Limitations put into table format (Table 14.2)
- Requirements for thin unit glass masonry were added to the strength level wind pressure chart (Figure 14.2) which clarifies the provisions and makes them easier to use.



Appendix D - GFRP Reinforced Masonry

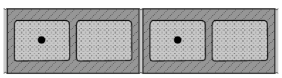
- New Section (Appendix) recognizing Glass Fiber Reinforced Polymer as an acceptable masonry reinforcement in limited applications
  - Limited to Non-Participating elements
  - Limited to SDC C or less

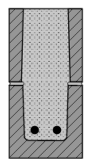
Key benefits:

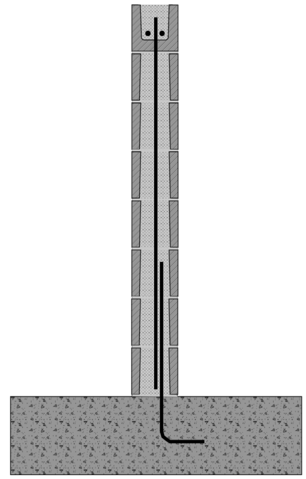
- Non-corrosive
- Thermally neutral
- Non-conductive
- Lightweight
- Improved workability and installation speed

Appendix D Scope and Limitations

- TMS 402/602 Appendix D includes:
  - **G**FRP deformed, *solid bars* in concrete or clay masonry, up to #6
  - Non-bearing walls and lintels in such walls
  - Retaining walls







Appendix D Scope and Limitations

D.1 GENERAL
- **D.1.1** Scope
- **D.1.2** Nonparticipating Elements
- **D.1.3** Reinforcement Materials
- **D.1.4** Strength-Reduction Factors

D.2 MATERIAL PROPERTIES
- **D.2.1** Design Tensile Strength and Strain
- **D.2.2** Modulus of Elasticity

D.3 REINFORCEMENT
- **D.3.1** GFRP Reinforcement
- **D.3.2** Size of GFRP Reinforcement
- **D.3.3** Development
- **D.3.4** Splices
- **D.3.5** Standard Hooks and Bends

D.4 FLEXURAL MEMBERS
- **D.4.1** Compression Reinforcement
- **D.4.2** Nominal Flexural Strength
  - **D.4.2.1** Compression-Controlled Sections
  - **D.4.2.2** Tension-Controlled Sections
- **D.4.3** Lintels
- **D.4.4** Nominal Shear Strength
- **D.4.5** Deflections
  - **D.4.5.1** Effective Moment of Inertia
  - **D.4.5.2** Wall Deflections

D.5 CREEP RUPTURE

GFRP-Reinforced Wall Behavior

Masonry Crushing
Compression-Controlled Section

GFRP Rupture
Tension-Controlled Section

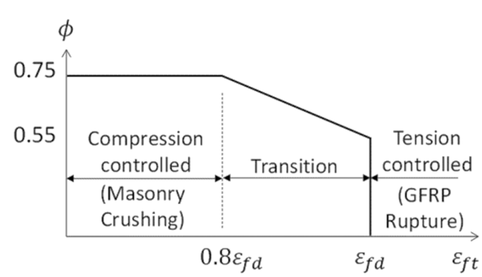
Image Credit: Tumialan, Torres, Quintana, & Nanni, "Masonry Walls Reinforced with FRP Bars Subjected to Out-of-Plane Loading".

Strength-Reduction Factors (D.1.4)

ϕ	Strain	Description
0.55	$\varepsilon_{ft} = \varepsilon_{fd}$	Tension controlled
$1.55 - \frac{\varepsilon_{ft}}{\varepsilon_{fd}}$	$0.80\varepsilon_{fd} < \varepsilon_{ft} < \varepsilon_{fd}$	Transition
0.75	$\varepsilon_{ft} \leq 0.80\varepsilon_{fd}$	Compression controlled

$\varepsilon_{fd} = f_{fd}/E_f$: design tensile strain for GFRP reinforcement

ε_{ft} = tensile strain at failure



Strength Controlled by Masonry

- Analogous to "over-reinforced beam"
  - Steel reinforcement: stress less than yield
- Masonry: equivalent rectangular stress block
- GFRP reinforcement: stress determined from

$$f_f = \sqrt{\frac{\left(E_f \varepsilon_{mu}\right)^2}{4} + \frac{0.64 f'_m}{\rho_f} E_f \varepsilon_{mu}} - \frac{1}{2} E_f \varepsilon_{mu}$$

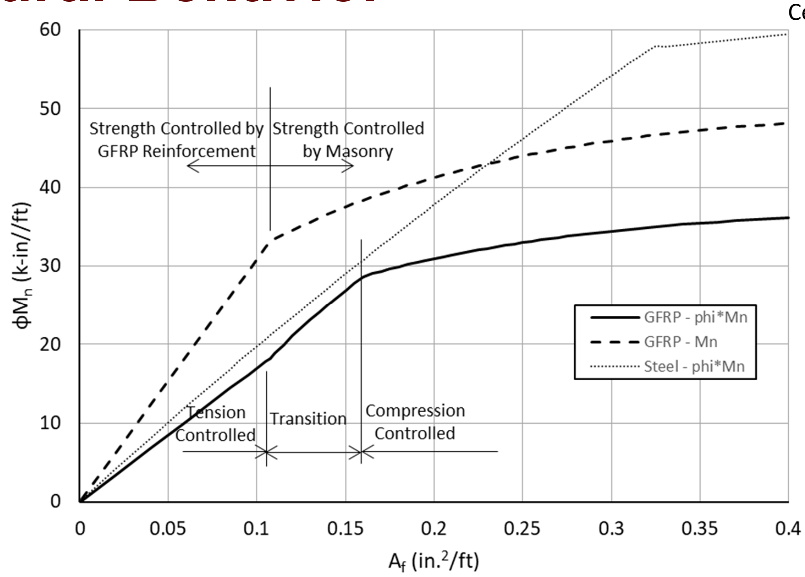
$$\rho_f = \frac{A_f}{bd}$$

Strength Controlled by GFRP Reinforcement

- GFRP reinforcement linear up to failure
- Stress in masonry at GFRP rupture less than ultimate
- Permitted to assume equivalent rectangular stress block:
  - Depth of $a = 0.80 c_{bal}$
  - Stress of $\frac{A_f f_{fd}}{0.80 c_{bal} b} \leq 0.80 f'_m$

Flexural Behavior

8 inch CMU
$f'_m = 2{,}000$ psi
Centered reinforcement



Wall Deflections

Wall deflection limit

$\delta_s \leq 0.01\, h$

Deformed shape of GFRP-reinforced masonry wall

After Failure

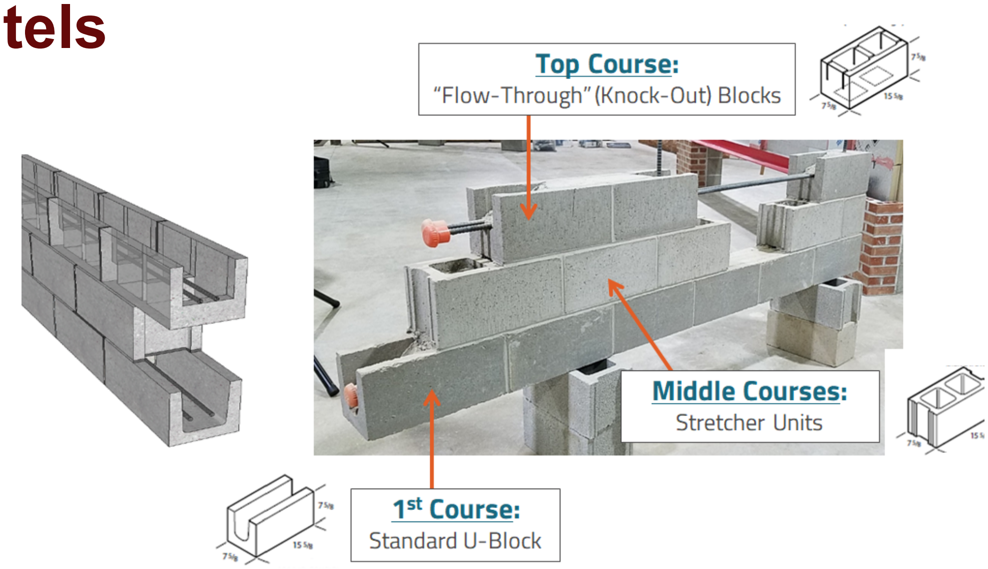
After Removal of Load

Image Credit: Tumialan, Torres, Quintana, & Nanni, "Masonry Walls Reinforced with FRP Bars Subjected to Out-of-Plane Loading".

Lintels

- Masonry has lower compressive strength parallel to bed joint than perpendicular to bed joint
- Stress in equivalent rectangular stress block is $0.80\chi f_m'$
  - $\chi = 0.5$ when grout is not horizontally continuous in compression zone; standard stretcher units with full height web
  - $\chi = 0.7$ when grout is horizontally continuous in compression zone; knock-out bond beam blocks

Lintels



Courtesy of International Masonry Institute

TMS 602 Article 1 - General

- Added periodic inspection requirement for adhered veneer when the height of the veneer exceeds 60 ft above grade plane (Table 4, Item 3.f)
- Added section on GFRP Reinforcing Bars (Article 1.7 F)
  1. Avoid damaging or abrading GFRP bars; do not drag or drop bars.
  2. Store above surface of ground; cover is stored outdoors for more than 2 months to protect from ultra-violet rays.
  3. Prevent exposure to temperatures above 120°F during storage.
  4. Do not use GFRP bars with visible fibers or any cut or defect greater than 0.04 in. deep.

Specification Article 2 - Products

- Preblended mortars (ASTM C1714) added (Article 2.1 A)
- Adhered veneer setting mortar required to be ANSI A118.4 or A118.15 polymer modified mortars (Article 2.1 B)
- Added F1554 bent-bar and headed anchor bolts (Article 2.4 J)
- Requirements were added for adhered veneer system products: cementitious backer units (2.5 G), lath fasteners (2.5 H), weep screeds (2.5 I), and lath (Article 2.5 O)
- Standard reinforcement bends and hook table moved from Code (2016 TMS 402 Table 6.1.8) to Specification (2022 TMS 602 Table 6). Table now includes deformed wire reinforcement in addition to deformed bars.

Specification Article 2 - Products

Stainless Steel Joint Reinforcement (Article 2.4 D.1)

- Commonly available ASTM A580/A580M stainless steel wire does not conform to the minimum yield and tensile strengths required by ASTM A951 (Standard Specification for Joint Reinforcement)
- Fabricated in accordance with ASTM A951 (welded and knurled) with AISI Type 304 or Type 316 stainless steel wire, having a minimum yield strength of 45 ksi and a minimum ultimate tensile strength of 90 ksi.

Courtesy of WireBond

Specification Article 2 - Products

Added a term for (single) wire reinforcement in veneer (Article 2.4 E)

2.4 E. *Veneer wire reinforcement —*

1. Wire with deformations knurled in conformance with ASTM A951. Either:

    (a) ASTM A1064/A1064M wire meeting the minimum mechanical properties of ASTM A951.

    (b) ASTM A580/A580M, AISI Type 304 or Type 316 stainless steel and having a minimum yield strength of 45 ksi and a minimum ultimate tensile strength of 90 ksi.

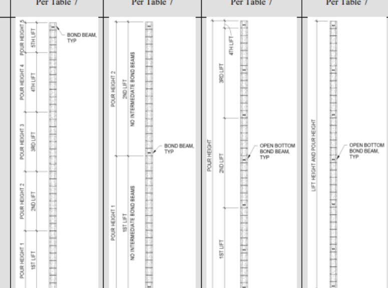
2. Deformed wire that conforms to ASTM A1064/A1064M.
3. Joint reinforcement that conforms to Article 2.4 D.

Specification Article 3 - Execution

- Clarify mechanical splice cover and clear distance between mechanical splices are the same as for reinforcing bars (Article 3.4 B)
- Veneer ties placement tolerance of ± 1 in. added (Article3.4 D)
- Added tolerance for adhered veneer fasteners (fasteners for backer units) of ± 0.25 in. (Article 3.4 F)

Specification Article 3 - Execution

- Added figure to clarify grout pour and lift requirements (Figure SC-20)



Type of Grouting*	Grouting with no cure time limit	Conventional grout with no intermediate bond beams	Conventional grout with intermediate bond beams	Self-consolidating grout with or without intermediate bond beams
TMS 602 Article	3.5 D.1.c 3.5 D.2.b	3.5 D.1.a	3.5 D.1.b	3.5 D.2.a
Lift Limit	5 ft-4 in.	12 ft-8 in.	See Limitation	Pour Height
Pour Height	Per Table 7	Per Table 7	Per Table 7	Per Table 7
Configuration				
Limitations	• Grout slump between 8 and 11 inches • Conventional grout or self-consolidating grout • Lift height is 1-1/2 inches less than pour height for shear key, except at top of wall.	• Masonry cured for at least 4 hours • Grout slump between 10 and 11 inches	• Masonry cured for at least 4 hours • Grout slump between 10 and 11 inches • Lift cannot exceed maximum 12 ft-8 in. • Limit grout lift to the bottom of lowest bond beam that is more than 5 ft-4 in. above bottom of grout lift • Lift height is 1-1/2 inches below the top of block for shear key, except at top of wall	• Masonry cured for at least 4 hours
Cleanouts Required	No	Yes	Yes	Yes

Tension and Compression Controlled Sections

- Motivation
- Provisions
- Design Aids

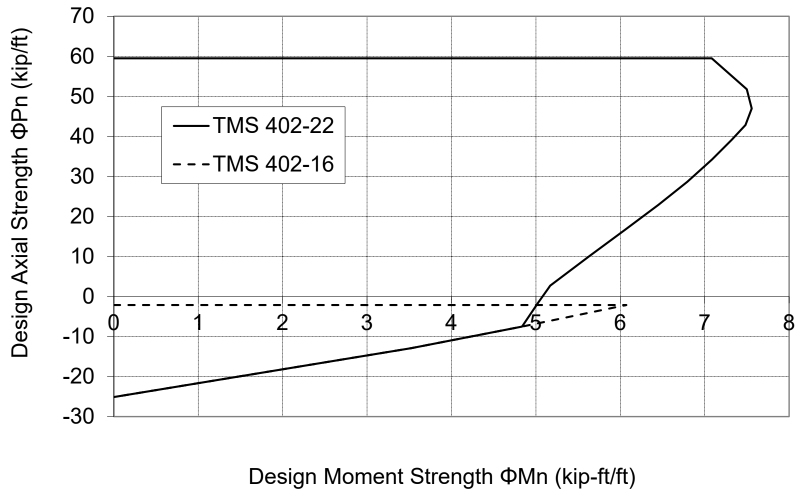
402-16 Maximum Reinforcement (9.3.3.2)

Area of flexural tensile reinforcement ≤ area required to maintain axial equilibrium under the following conditions

- A strain gradient corresponding to ε_{mu} in masonry and $\alpha\varepsilon_y$ in tensile reinforcement
  - α = 1.5 for all except intermediate and special reinforced shear walls with $M_u/(V_u d_v) \geq 1$
- Axial forces from loading combination D + 0.75L + 0.525Q_E
- Compression reinforcement, with or without lateral restraining reinforcement, can be included.

Motivation

8 in. CMU wall, f'_m = 2 ksi, Grade 60 No. 5 at 8 in., out-of-plane loading



Requirements

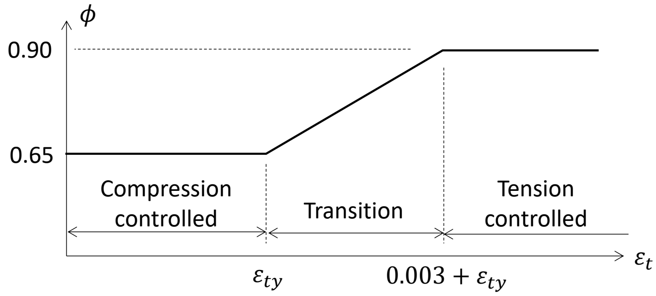
9.1.4.4 *Combinations of flexure and axial load in reinforced masonry* — The value of ϕ for reinforced masonry subjected to flexure, axial load, or combinations thereof shall be in accordance with Table 9.1.4.

9.1.4.4.1 The value of ε_{ty} shall be f_y / E_s. For Grade 60 reinforcement it shall be permitted to take ε_{ty} equal to 0.002.

9.1.4.4.2 In the tension-controlled and transition regions, the value of ϕ for axial load shall be limited so that $\phi P_n \leq 0.65 P_{bal}$, where P_{bal} is determined using a strain gradient corresponding to a strain in the extreme tensile reinforcement equal to ε_{ty} and a maximum strain in the masonry as given by Section 9.3.2(c). {0.0025 for concrete masonry and 0.0035 for clay masonry}

TMS 402 Table 9.1.4

Net Tensile Strain	Classification	Strength-reduction factor, ϕ
$\varepsilon_t \leq \varepsilon_{ty}$	Compression controlled	0.65
$\varepsilon_{ty} < \varepsilon_t < 0.003 + \varepsilon_{ty}$	Transition	$0.65 + 0.25\dfrac{\varepsilon_t - \varepsilon_{ty}}{0.003}$
$\varepsilon_t \geq 0.003 + \varepsilon_{ty}$	Tension controlled	0.90



ε_t = net tensile strain in extreme longitudinal reinforcement

ε_{ty} = value of net tensile strain in extreme layer of longitudinal tension reinforcement used to define a compression-controlled section; yield strain

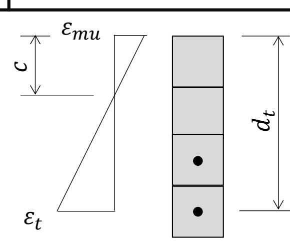
c/d_t Ratio

Useful for drawing interaction diagram

c/d_t ratio		Strength-reduction factor, ϕ
CMU	Clay	
$c/d_t \geq 0.556$	$c/d_t \geq 0.636$	0.65
$0.333 < c/d_t < 0.556$	$0.412 < c/d_t < 0.636$	$0.65 + \dfrac{0.25}{0.003}\left(\left(\dfrac{1}{c/d_t} - 1\right)\varepsilon_{mu} - \varepsilon_{ty}\right)$
$c/d_t \leq 0.333$	$c/d_t \leq 0.412$	0.90

c = depth to neutral axis

d_t = distance from compression surface to furthest longitudinal tension reinforcement

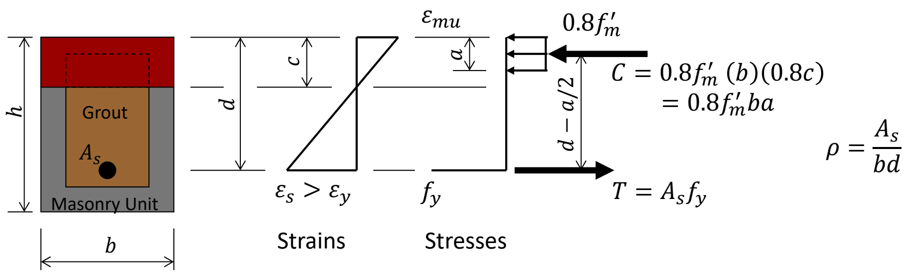


Maximum Reinforcement

- Deleted for out-of-plane, ordinary reinforced shear walls, columns, and pilasters
- Same maximum reinforcement (ductility) requirements for intermediate and special reinforced shear walls
- Beams need to be tension controlled

Beams and Lintels

Flexural Members: Strength Design



$$T = C$$

$$A_s f_y = 0.8f'_m(b)(a)$$

$$a = \frac{A_s f_y}{0.8f'_m b}$$

$$M_n = A_s f_y\left(d - \frac{1}{2}\frac{A_s f_y}{0.8f'_m b}\right)$$

Beams: Strength Design Procedure

1. Determine material properties $(f_y,\ f'_m)$
2. Choose beam dimensions
   A. Thickness: 8 in., 12 in.
   B. Depth: if possible, choose so no shear reinforcement is required

$$d \geq \frac{V_u}{1.8b\sqrt{f'_m}}$$

3. Determine a, depth of compressive stress block

$$a = d - \sqrt{d^2 - \frac{2M_n}{0.8f'_m b}} = d - \sqrt{d^2 - \frac{2M_u}{0.8\phi f'_m b}}$$

4. Solve for $A_{s,reqd}$

$$A_{s,reqd} = \frac{0.8f'_m ba}{f_y}$$

Typical values:
Concrete masonry: $f'_m = 2{,}000$ psi
Clay masonry: $f'_m = 3{,}000$ psi

Beams: Minimum Reinforcement

Minimum reinforcement: (9.3.4.2.2.2, 9.3.4.2.2.3)

- $M_n \geq 1.3M_{cr}$
  - Modulus of rupture: Table 9.1.9.2
  - $M_{cr} = f_r \frac{bh^2}{6}$
- or $A_s \geq (4/3)A_{s,reqd}$

Design Tip:
M_n often not calculated
$M_u \geq 1.17M_{cr}$

Flexural Tension Parallel to Bed Joint	Mortar Type			
	PCL or mortar cement		Masonry Cement	
	M or S	N	M or S	N
Running bond; hollow units; fully grouted	267	200	160	100
Not laid in running bond; continuous grout section parallel to bed joints	335	335	335	335

Beams: Maximum Reinforcement

Maximum reinforcement: (9.3.3.2)
Reinforcement shall NOT exceed area required to maintain equilibrium under a strain gradient of ε_{mu} in masonry and $\alpha\varepsilon_y = 1.5\varepsilon_y$ in reinforcement.

2022:
Beams must be tension controlled.
For Grade 60, strain limit of:
$\varepsilon_{ty} + 0.003 = 2.5\varepsilon_y$

$$\rho_{max} = \frac{0.8(0.8)f'_m}{f_y}\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right)$$

Steel Ratio	Grade 60 steel		Grade 60 steel	
	Clay	CMU	Clay	CMU
ρmax (f'_m in ksi)	$0.00565f'_m$ $0.84\rho_{bal}$	$0.00476f'_m$ $0.82\rho_{bal}$	$0.00439f'_m$ $0.65\rho_{bal}$	$0.00356f'_m$ $0.60\rho_{bal}$
f_y = 60 ksi; f'_m = 2 ksi	0.0113	**0.00952**	0.0088	**0.00711**
f_y = 60 ksi; f'_m = 3 ksi	**0.0169**	0.0144	**0.0132**	0.0197

Beams: Shear Strength

Nominal Shear

$V_n = V_{nm} + V_{ns}$

Conservative approximation

$M_u/(V_u d_v) = 1.0$

$V_{nm} = 2.25 A_{nv}\sqrt{f_m'}$

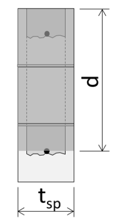
$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y d_v$

$V_n \leq 4A_{nv}\sqrt{f_m'}$

Shear Area (TMS 402-22 Table 4.3.5)

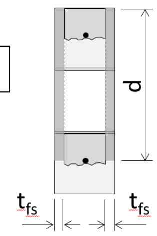
Fully Grouted

$A_{nv} = t_{sp}d$



Partially Grouted

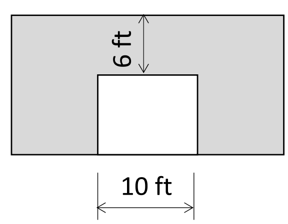
$A_{nv} = 2t_{fs}d$



Example: Beam Design

Given:

- Masonry
  - 8 inch CMU, medium weight (125 pcf)
  - Type S masonry cement mortar
  - f'_m = 2,000 psi
- Reinforcement
  - Grade 60 steel
- Geometry
  - 10 ft opening
  - 6 ft of masonry above opening
- Loads
  - Superimposed dead load of 0.5 kip/ft
  - Live load of 0.5 kip/ft
  - 125 pcf units (81 psf fully grouted; use 45 psf for rest)



Example: Beam Design

Solution:
Length of bearing: Assume to be 8 in.

Span length: L = 10 ft +2(4 in.) =10.67 ft

Depth to reinforcement: $d \approx h - 4$ in.

Approximate weight of beam: $60psf(6ft) = 360\frac{lb}{ft}$

Depth of beam:

$$V_u \sim \frac{w_u L}{2} = \frac{1}{2}\left(1.2\left(0.5\frac{\text{k}}{\text{ft}} + 0.36\frac{\text{k}}{\text{ft}}\right) + 1.6\left(0.5\frac{\text{k}}{\text{ft}}\right)\right)(10.67\text{ft}) = 9.8\text{ k}$$

$$d \geq \frac{V_u}{1.8b\sqrt{f'_m}} = \frac{9{,}800\text{lb}}{1.8(7.625\text{in.})\sqrt{2000\text{psi}}} = 16\text{ in.}$$ Use h = 24 in. deep beam

Example: Beam Design

Factored Load

$$w_u = 1.2D + 1.6L$$

$$w_u = 1.2\left(0.5\frac{\text{k}}{\text{ft}} + 0.081\frac{\text{k}}{\text{ft}^2}(2\text{ft}) + 0.045\frac{\text{k}}{\text{ft}^2}(4\text{ft})\right) + 1.6\left(0.5\frac{\text{k}}{\text{ft}}\right) = 1.81\frac{\text{k}}{\text{ft}}$$

Factored Moment

$$M_u = \frac{w_u L^2}{8} = \frac{\left(1.81\frac{\text{k}}{\text{ft}}\right)(10.67\text{ft})^2}{8} = 25.8\text{k}\cdot\text{ft}$$

Find a
Depth of equivalent rectangular stress block

$$a = d - \sqrt{d^2 - \frac{2M_n}{0.8f'_m b}}$$

$$= 20\text{in.} - \sqrt{(20\text{in.})^2 - \frac{2\left(\frac{25.8\text{k}\cdot\text{ft}}{0.9}\right)\left(\frac{12\text{in.}}{\text{ft}}\right)}{0.8(2.0\text{ksi})(7.625\text{in.})}} = 1.46\text{ in.}$$

Find $A_{s,reqd}$
Req'd area of steel

$$A_{s,reqd} = \frac{0.8f'_m ba}{f_y} = \frac{0.8(2.0\text{ksi})(7.625\text{in.})(1.46\text{in.})}{60\text{ksi}} = 0.29\text{ in.}^2$$

Use 1 - #5 (A_s = 0.31 in^2)

$$\boxed{\left(A_{s,reqd}\right)_{\text{ASD}} = 0.39\text{ in.}^2}$$

Example: Beam Design

Minimum Reinforcement Check: f_r = 160 psi (parallel to bed joints in running bond; fully grouted)

Section modulus $S_n = \frac{bh^2}{6} = \frac{(7.625\text{in.})(24\text{in.})^2}{6} = 732\text{ in.}^3$

Cracking moment $M_{cr} = S_n f_r = 732\text{in.}^3\,(160\text{psi}) = 117.1\text{k}\cdot\text{in.} = 9.76\text{ k}\cdot\text{ft}$

By inspection, $1.17M_{cr} \leq M_u = 25.8\text{ k}\cdot\text{ft}$

d (inch)	2022 $A_{s,max}$ (in.^{2})
4	0.22
12	0.65
20	1.08
28	1.52
36	1.95

$f_m' = 2{,}000$ psi
$b = 7.625$ in.

30% of 2022 ρ_{max}

Maximum Reinforcement Check: 2016 $\rho_{max} = 0.00952$ (2022: $\rho_{max} = 0.00711$)

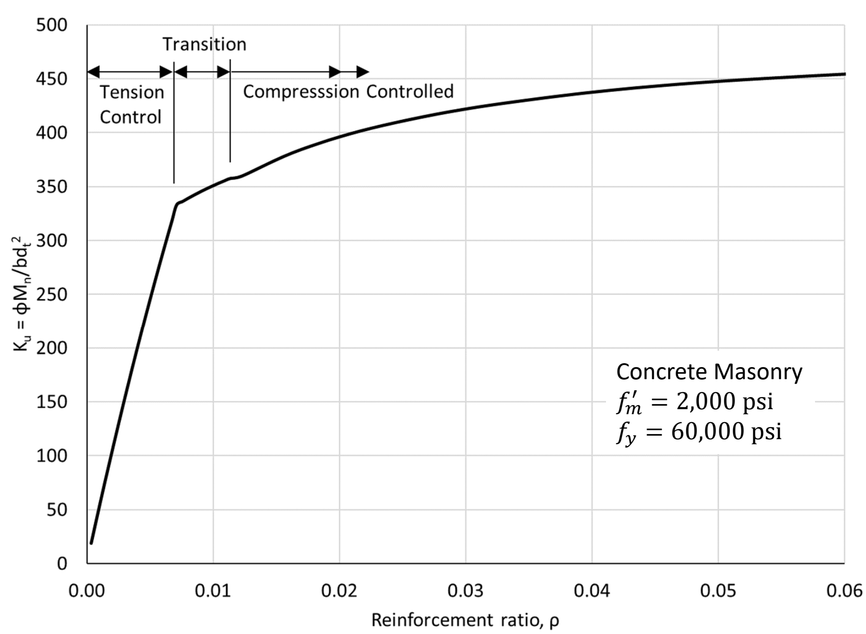
$$\rho = \frac{A_s}{bd} = \frac{0.31\text{in.}^2}{(7.625\text{in.})(20\text{in.})} = 0.00203$$

Non-Load Bearing Walls

Non-Load Bearing Walls: Tension Controlled

Masonry	Concrete		Clay	
f'_m	2,000 psi		3,000 psi	
ρ_t	0.00711		0.01318	
Nominal Wall Thickness	Max. reinforcement to be tension controlled (in.^{2}/ft)			
6 in. Centered reinforcement	0.24	No. 4 @ 16 No. 5 @ 16 No. 6 @ 24	0.44	No. 4 @ 8 No. 5 @ 16 No. 6 @ 16
8 in. Centered reinforcement	0.32	No. 4 @ 8 No. 5 @ 16 No. 6 @ 24	0.60	No. 4 @ 8 No. 5 @ 8 No. 6 @ 16
12 in. Centered reinforcement	0.50	No. 5 @ 8 No. 6 @ 16 No. 7 @ 16		

ρ_t = maximum reinforcement ratio to be tension controlled



Load Bearing Walls

Maximum Axial Load: Tension Controlled

TMS 402-22 Table CC-9.1-1

Masonry Element	Concrete Masonry	Clay Masonry
Fully grouted section with single layer of tension reinforcement	$P_u \leq 0.19 f'_m bd - 0.9 A_s f_y$	$P_u \leq 0.24 f'_m bd - 0.9 A_s f_y$

Maximum Axial Load: Tension Controlled

TMS 402-22 Table CC-9.1-1
Partially grouted wall with a single layer of tension reinforcement subjected to out-of-plane loads

Concrete Masonry	Clay Masonry
$P_u \leq 0.19 f'_m bd - 0.9 A_s f_y$ for $t_{fs} \geq 0.27d$ $P_u \leq 0.72 f'_m (bt_{fs} + (0.27d - t_{fs}) b_w - 0.9 A_s f_y$ for $t_{fs} < 0.27d$ but not greater than $P_u \leq 0.52 f'_m (bt_{fs} + (0.44d - t_{fs}) b_w) - 0.65 A_s f_y$	$P_u \leq 0.24 f'_m bd - 0.9 A_s f_y$ for $t_{fs} \geq 0.33d$ $P_u \leq 0.72 f'_m (bt_{fs} + (0.33d - t_{fs}) b_w - 0.9 A_s f_y$ for $t_{fs} < 0.33d$ but not greater than $P_u \leq 0.52 f'_m (bt_{fs} + (0.50d - t_{fs}) b_w) - 0.65 A_s f_y$

Minimum Axial Load: Compression Controlled

Masonry Element	Concrete Masonry	Clay Masonry
Fully grouted section with single layer of tension reinforcement	$P_u \geq 0.23 f'_m bd - 0.65 A_s f_y$	$P_u \geq 0.26 f'_m bd - 0.65 A_s f_y$

Minimum Axial Load: Compression Controlled

Partially grouted wall with a single layer of tension reinforcement subjected to out-of-plane loads

Concrete Masonry	Clay Masonry
$P_u \geq 0.23 f'_m bd - 0.65 A_s f_y$ for $t_{fs} \geq 0.44d$	$P_u \leq 0.26 f'_m bd - 0.65 A_s f_y$ for $t_{fs} \geq 0.51d$
$P_u \leq 0.52 f'_m (bt_{fs} + (0.44d - t_{fs}) b_w - 0.65 A_s f_y$ for $t_{fs} < 0.44d$	$P_u \leq 0.52 f'_m (bt_{fs} + (0.51d - t_{fs}) b_w - 0.65 A_s f_y$ for $t_{fs} < 0.51d$

Bearing Walls: Tension Controlled

- Maximum Axial Load, P_u, for Section to be **Tension Controlled**, Out-of-Plane Loads
- CMU, f'_m = 2 ksi , Grade 60 Reinforcement

Bar Size	Bar Spacing					
	8 in.	16 in.	24 in.	32 in.	40 in.	48 in.
	8 in. unit, centered reinforcement					
No. 4	1.2 kip/ft	9.3 kip/ft	12.0 kip/ft	13.3 kip/ft	14.1 kip/ft	14.5 kip/ft
No. 5	-7.7 kip/ft	4.8 kip/ft	9.0 kip/ft	11.1 kip/ft	12.4 kip/ft	13.2 kip/ft
No. 6	-18.2 kip/ft	-0.4 kip/ft	5.5 kip/ft	8.5 kip/ft	10.3 kip/ft	11.4 kip/ft
	12 in. unit, centered reinforcement					
No. 4	10.3 kip/ft	16.2 kip/ft	17.1 kip/ft	16.8 kip/ft	16.5 kip/ft	16.4 kip/ft
No. 5	1.4 kip/ft	11.8 kip/ft	15.0 kip/ft	15.1 kip/ft	15.2 kip/ft	15.3 kip/ft
No. 6	-9.1 kip/ft	6.5 kip/ft	11.6 kip/ft	13.2 kip/ft	13.7 kip/ft	14.0 kip/ft
	12 in. unit, offset reinforcement, d = 11.625 in. – 2.5 in. = 9.125 in.					
No. 4	25.4 kip/ft	24.0 kip/ft	23.2 kip/ft	21.3 kip/ft	20.2 kip/ft	19.4 kip/ft
No. 5	16.5 kip/ft	19.5 kip/ft	20.2 kip/ft	19.7 kip/ft	18.9 kip/ft	18.3 kip/ft
No. 6	6.0 kip/ft	14.3 kip/ft	16.7 kip/ft	17.8 kip/ft	17.4 kip/ft	17.1 kip/ft

Bearing Walls: Example

Design a 16 ft – 8 in. high 8 in. CMU bearing wall with an eccentric axial dead load of 700 lb/ft, an eccentric axial roof live load of 300 lb/ft, and an out-of-plane wind load of 30 lb/ft^2. The eccentricity is 2.48 in. There is a 3 ft – 4 in. parapet (total height = 20 ft).

Final Design: No. 4 @ 48 in.
By inspection, maximum axial load is less than 14.5 kip/ft

Bar Size	Bar Spacing					
	8 in.	16 in.	24 in.	32 in.	40 in.	48 in.
	8 in. unit, centered reinforcement					
No. 4	1.2 kip/ft	9.3 kip/ft	12.0 kip/ft	13.3 kip/ft	14.1 kip/ft	14.5 kip/ft
No. 5	-7.7 kip/ft	4.8 kip/ft	9.0 kip/ft	11.1 kip/ft	12.4 kip/ft	13.2 kip/ft
No. 6	-18.2 kip/ft	-0.4 kip/ft	5.5 kip/ft	8.5 kip/ft	10.3 kip/ft	11.4 kip/ft

Bearing Walls:

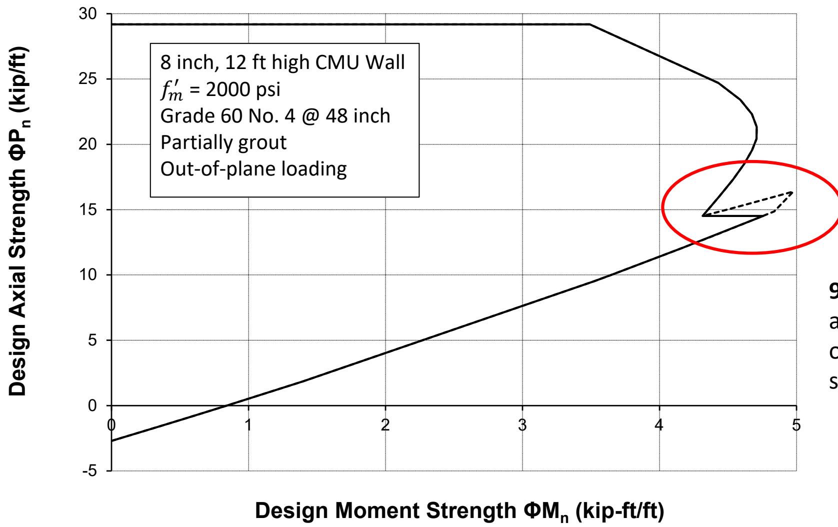
Maximum Axial Load to be **Tension Controlled**
Minimum Axial Load to be **Compression Controlled**

Out-of-Plane Loads
CMU, f'_m = 2 ksi , Grade 60 Reinforcement

Bar Size	Bar Spacing					
	8 in.	16 in.	24 in.	32 in.	40 in.	48 in.
	8 in. unit, centered reinforcement					
No. 4	1.2 kip/ft 9.3 kip/ft	9.3 kip/ft 12.4 kip/ft	12.0 kip/ft 13.5 kip/ft	13.3 kip/ft 14.0 kip/ft	14.1 kip/ft 14.3 kip/ft	14.5 kip/ft 14.5 kip/ft
No. 5	-7.7 kip/ft 2.9 kip/ft	4.8 kip/ft 9.2 kip/ft	9.0 kip/ft 11.3 kip/ft	11.1 kip/ft 12.4 kip/ft	12.4 kip/ft 13.0 kip/ft	13.2 kip/ft 13.5 kip/ft
No. 6	-18.2 kip/ft -4.7 kip/ft	-0.4 kip/ft 5.4 kip/ft	5.5 kip/ft 8.8 kip/ft	8.5 kip/ft 10.5 kip/ft	10.3 kip/ft 11.5 kip/ft	11.4 kip/ft 12.2 kip/ft

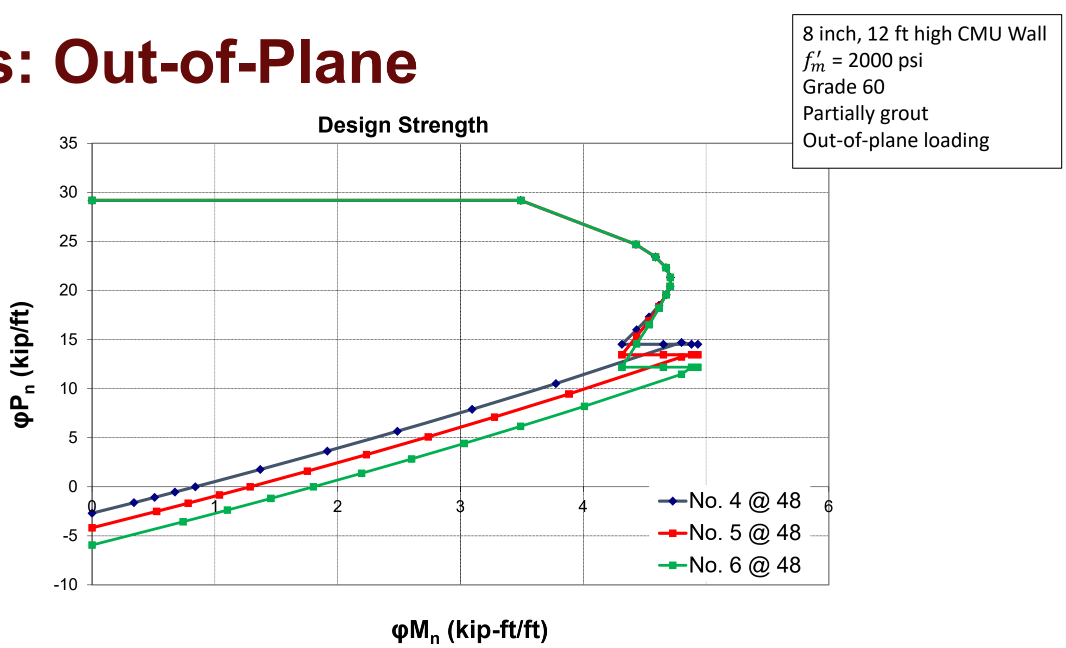
With wider spaced reinforcement, small transition region

Partially Grouted Walls: Out-of-Plane



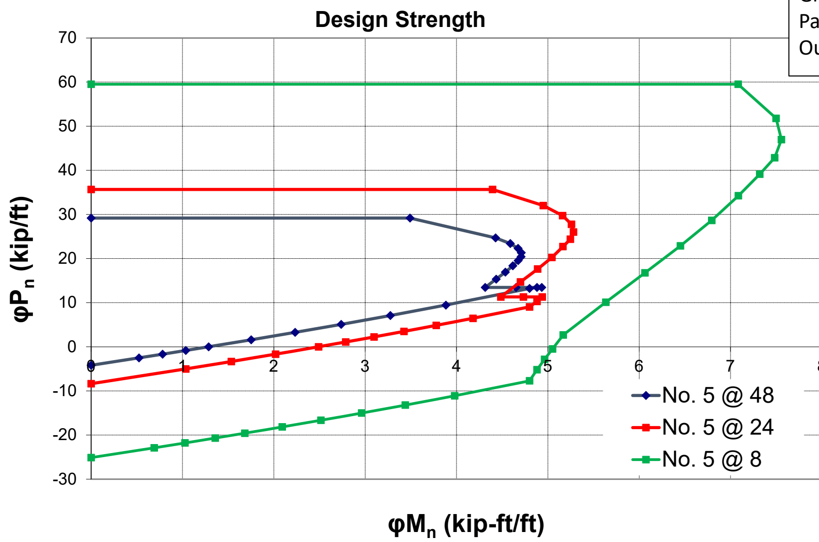
9.1.4.4.2 In the tension-controlled and transition regions, the value of ϕ for axial load shall be limited so that $\phi P_n \leq 0.65 P_{bal}$

Walls: Out-of-Plane



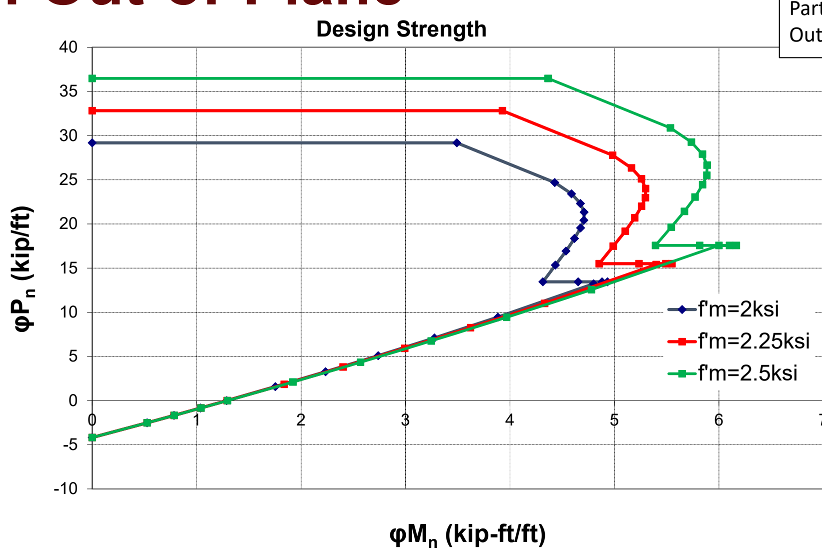
Walls: Out-of-Plane

8 inch, 12 ft high CMU Wall
f'_m = 2000 psi
Grade 60
Partially grout
Out-of-plane loading



Walls: Out-of-Plane

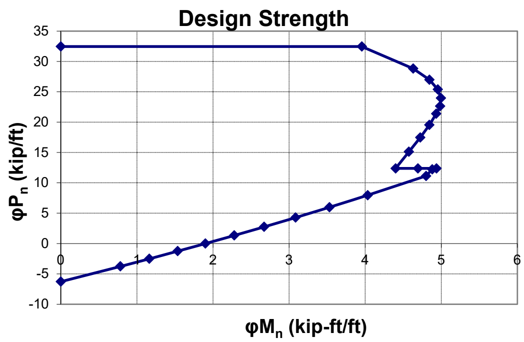
8 inch, 12 ft high CMU Wall
Grade 60, No. 5 @ 48 in.
Partially grout
Out-of-plane loading

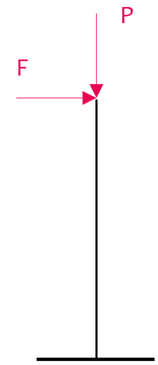


f'_m (ksi)	f_u (ksi)
2	2
2.25	2.6
2.5	3.25

Walls: Design

Design Strength	≥	Factored Load
(Interaction Diagram)		(Second-order; P-delta)





Walls: Slenderness (P-delta) Effects

1. **Slender Wall Method (9.3.4.4.2)**
   a. Second-order moment directly added by P-δ
   b. Usually requires iteration
   c. Difficult for hand calculations for other than simple cases
   d. Basis for second-order analysis in computer programs
   e. Historical method used for masonry design
2. **Second-order Analysis (9.3.4.4.3)**
   a. Added in 2013 TMS 402 Code
   b. Computer analysis
3. **Moment Magnification (9.3.4.4.3)**
   a. Added in 2013 TMS 402 Code
   b. Very general, but a bit conservative

9.3.4.4.2 Slender Wall

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

Loophole!!
Call this a second-order method and there are no restrictions.

Moment:

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

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

P_{uf} = factored floor load

P_{uw} = factored wall load

Deflection:

$M_u \leq M_{cr}$

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

$M_u > M_{cr}$

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

Slender Wall Methods: Simultaneous Equations

$\underline{\boldsymbol{M_u > M_{cr}}}$

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

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

$\underline{\boldsymbol{M_u \leq M_{cr}}}$

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

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

Wall Properties

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

Cracked moment of inertia:

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

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

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

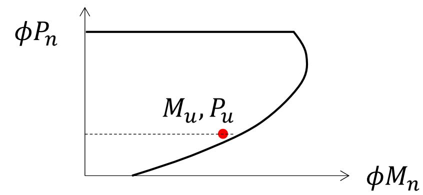
Cracking Moment, M_{cr}

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

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

Interaction Diagram

TMS 402 9.3.4.2 Commentary Equations



Finding point on interaction diagram where $\phi P_n = P_u$.

Depth of stress block, a $$a = \frac{A_s f_y + P_u/\phi}{0.8 f'_m b}$$

Design moment, ϕM_n

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

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

Valid for fully grouted, or partially grouted with a in the face shell.
Valid for tension-controlled and transition (if appropriate ϕ is used).

Walls: Preliminary Design

Tension-controlled

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

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

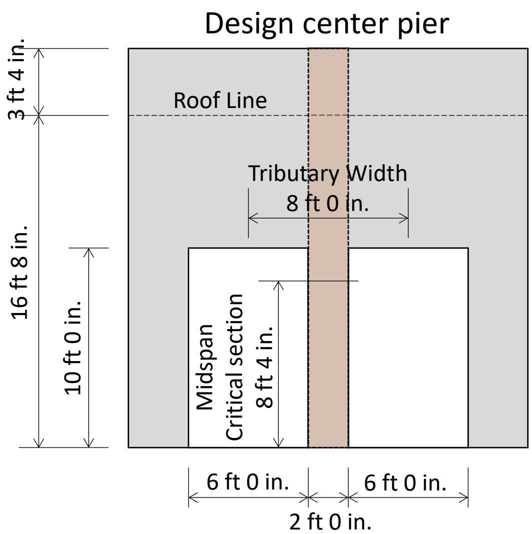
Increase $A_{s,reqd}$ by 10% if $h/t > 30$

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

Compression-controlled and transition

Interaction diagram: trial and error using spreadsheet

Example: Pier in Bearing Wall



Dead load: 700 lb/ft
Roof live load: 300 lb/ft
Eccentricity: 2.48 inch
Out-of-plane wind: 30 psf

Fully grouted wall weight (pier): 81 psf
Other masonry: 45 psf

8 inch CMU
f'_m = 2,000 psi
f_y = 60,000 psi
Type S masonry cement

Try No. 5 @ 8 inch for pier

Example: Axial Loads

Summary of Strength Design Load Combination Axial Forces

Load Combination	P_{uf}	P_{uw}	P_u
0.9D+1.0W	0.9(0.7 k/ft)(8 ft) = 5.04 k	0.9(4.59 k) = 4.13 k	9.17 k
1.2D+1.0W+0.5L$_r$	[1.2(0.7 k/ft) + 0.5(0.3 k/ft)](8 ft) = 7.92 k	1.2(4.59 k) = 5.51 k	13.43 k

P_{uf} =Factored floor load; eccentrically applied load

P_w = Wall load: (0.081 ksf)(2 ft)(8.3 ft + 3.3 ft) + (0.045 ksf)(6 ft)(6.7 ft + 3.3 ft) = 4.59 k

Example: Cracking Moment

Modulus of rupture: TMS 402 Table 9.1.9.1

Fully grouted (Type S masonry cement): 153 psi

Wall properties: determined from NCMA TEK 14-1B Section Properties of CMU Walls

$A_n = 183 \text{ in.}^2 \qquad S_n = 232.6 \text{ in.}^3 \qquad I_n = 886.6 \text{ in.}^4$

Cracking moment, M_{cr}:

Commentary allows inclusion of axial load (9.3.4.4.4)

Use minimum axial load (once wall has cracked, it has cracked)

$$M_{cr} = \frac{(P_u/A_n + f_r)I_n}{t_{sp}/2} = \frac{(9{,}170 \text{ lb}/183 \text{ in.}^2 + 153 \text{ psi})886.6 \text{ in.}^4}{7.625 \text{ in.}/2} = 47.23 \text{ k} \cdot \text{in.} = 3.936 \text{ k} \cdot \text{ft}$$

Example: Cracked Moment of Inertia

Modular ratio, n

$$n = \frac{E_s}{E_m} = \frac{29000\text{ksi}}{1800\text{ksi}} = 16.1$$

Depth to neutral axis, c

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b} = \frac{3(0.31 \text{ in.}^2)(60\text{ksi}) + 9.17 \text{ k}}{0.64(2 \text{ ksi})24 \text{ in.}} = 2.115 \text{ in.}$$

Cracked moment of inertia, I_{cr}

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

$$= 16.1\left(0.93 \text{ in.}^2 + \frac{9.17 \text{ k}}{60 \text{ ksi}}\right)(3.812 \text{ in.} - 2.115 \text{ in.})^2 + \frac{24 \text{ in.}(2.115 \text{ in.})^3}{3} = 125.9\text{in.}^4$$

Example: Factored Moment

Find M_u $\quad P_{uf}e_u$ is the moment at the top support of the wall. It includes eccentric axial load and wind load from parapet.

M_{uf} = factored moment at top of wall

w_{up} = factored out-of-plane load on parapet

h_p = height of parapet

$$M_{uf} = P_{uf}e_u - \frac{w_{up}h_p^2}{2} = 5.04\text{ k}\left(\frac{2.48}{12}\text{ft}\right) - \frac{0.030\text{ ksf}(8\text{ ft})(3.33\text{ ft})^2}{2} = -0.29\text{ k}\cdot\text{ft}$$

Example: Factored Moment

P-delta moment was 12% of first-order moment

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

$$= \frac{\frac{0.030\text{ksf}(8\text{ft})(16.7\text{ft})^2}{8}+\frac{-0.29\text{k}\cdot\text{ft}}{2}+\frac{5(3.94\text{k}\cdot\text{ft})(9.17\text{k})(16.7\text{ft})^2}{48(1800\text{ksi})}\left(\frac{1}{887\text{in.}^4}-\frac{1}{126\text{in.}^4}\right)\left(\frac{144\text{in.}^2}{1\text{ft}^2}\right)}{1-\frac{5(9.17\text{k})(16.7\text{ft})^2}{48(1800\text{ksi})(126\text{in.}^4)}\left(\frac{144\text{in.}^2}{1\text{ft}^2}\right)} = 9.20\text{k}\cdot\text{ft}$$

Whew! We have the factored moment.

Is the (M_u, P_u) combination of $(9.20\text{ k}\cdot\text{ft},\ 9.17\text{ k})$ inside the interaction diagram?

YES! Loaded to 87% of design strength. An efficient design.

Example: Load Combinations

Load Combination	P_u (kip)	M_u (kip-ft)	ϕM_n (kip-ft)	$\frac{M_u}{\phi M_n}$	2nd Order / 1st Order
1.2D + 1.0W + 0.5L_r	13.43	10.01	10.84	0.92	1.18
0.9D + 1.0W	9.17	9.20	10.57	0.87	1.12

Example: Interaction Diagram

Example: Deflections

Load Combination	D+0.6W	0.6D+0.6W
P (kip)	10.19	6.11
c (in.)	2.148	2.015
I_{cr} (in.^{4}/ft)	128.4	119.2
M (k-ft)	5.59	5.16
δ (in.)	0.48	0.41

Deflection Limit: $\delta_s \leq 0.007h = 0.007(16.7 \text{ ft})\left(12\,\frac{\text{in.}}{\text{ft}}\right) = 1.40 \text{ in.}$ **OK**

Example: Design Comparisons

2016 Strength Design: Reinforcement exceeds maximum reinforcement; cannot use SD

2022 Strength Design: No. 5 @ 8 in.; design in transition region

Example: Parametric Study

Change from No. 5 @ 8 in. to No. 4 @ 8 in.

Load Combination	P_u (kip)	M_u (kip-ft)	ϕM_n (kip-ft)	$\frac{M_u}{\phi M_n}$	2nd Order / 1st Order
1.2D + 1.0W + 0.5L_r	13.43 (13.43)	10.82 (10.01)	10.11 (10.84)	1.07 (0.92)	1.27 (1.18)
0.9D + 1.0W	9.17 (9.17)	9.59 (9.20)	9.91 (10.57)	0.97 (0.87)	1.17 (1.12)

(Values in parentheses are No. 5 @ 8 in.)

Example: Parametric Study

No. 4 @ 8 in.
Increase f'_m to 2250 psi; requires a unit strength of 2600 psi

Load Combination	P_u (kip)	M_u (kip-ft)	ϕM_n (kip-ft)	$\frac{M_u}{\phi M_n}$	2nd Order / 1st Order
1.2D + 1.0W + 0.5L_r	13.43 (13.43)	10.67 (10.82)	11.12 (10.11)	0.96 (1.07)	1.26 (1.27)
0.9D + 1.0W	9.17 (9.17)	9.50 (9.59)	10.91 (9.91)	0.87 (0.97)	1.16 (1.17)

(Values in parentheses are No. 4 @ 8 in.; f'_m = 2000 psi)

Example: Unit Strength

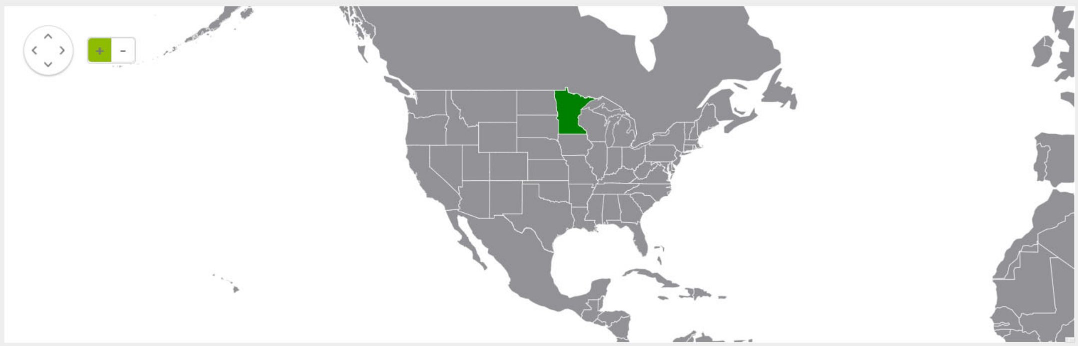
forsei.com/cmudata

CMU BLOCK STRENGTH RESULTS	MINIMUM STRENGTH	AVERAGE STRENGTH	MAXIMUM STRENGTH
6	3030	4136	4807

CHANGE STATES, BLOCK TYPE, AND YEAR RANGE TO UPDATE CMU STATISTICS

States

MN ×



Example: Parametric Study

No. 4 @ 8 in.

f'_m (psi)	E_m (ksi)	I_{cr} (in.^{4})	$E_m I_{cr}$ (k-in^2)
2000	1800	92.0	166,000
2250	2025 (12% increase)	85.5 (7% decrease)	173,000 (4% increase)

Shear Walls

Maximum Reinforcement

<u>TMS 402-16</u>

9.3.3.2 *Maximum area of flexural tensile reinforcement*

Strain gradient of ε_{mu} and $\alpha\varepsilon_y$

<table>
<tr><th rowspan="2">Shear Wall Type</th><th colspan="2">Yield Strain Multiplier, α</th></tr>
<tr><th>$\frac{M_u}{V_u d_v} < 1$</th><th>$\frac{M_u}{V_u d_v} \geq 1$</th></tr>
<tr><td>Ordinary</td><td rowspan="3">1.5</td><td>1.5</td></tr>
<tr><td>Intermediate</td><td>3.0</td></tr>
<tr><td>Special</td><td>4.0</td></tr>
</table>

9.3.6 *Wall design for in-plane loads*

9.3.6.6 The maximum reinforcement requirements of Section 9.3.3.2 shall not apply if a shear wall is designed to satisfy the requirements of 9.3.6.6.1 through 9.3.6.6.5.

9.3.6.6.1 to 9.3.6.6.5 *Special boundary elements*

Maximum Reinforcement

<u>TMS 402-22</u>

9.3.5.6 *Ductility requirements* **—** Intermediate and special reinforced masonry shear walls shall either:

a) comply with the maximum reinforcement requirements of Section 9.3.5.6.1, or

b) comply with alternate ductility provisions of Section 9.3.5.6.2.

9.3.5.6.1 *Maximum area of flexural tensile reinforcement*

Same as TMS 402-16 **9.3.3.2**

9.3.5.6.2 *Alternate approaches to wall ductility*

Same as TMS 402-16 **9.3.6.6.1** through **9.3.6.6.5**.

Ductility: 9.3.5.6.2.1

Boundary element not required if:

- Axial Load

  $P_u \leq 0.10 A_n f'_m$ for geometrically symmetrical sections

  $P_u \leq 0.05 A_n f'_m$ for geometrically **<u>un</u>**symmetrical sections

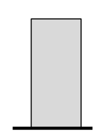
- And either

  $\frac{M_u}{V_u d_v} \leq 1.0$

Elevation

Or

$\frac{M_u}{V_u d_v} \leq 3.0$ and $V_u \leq 3 A_{nv} \sqrt{f'_m}$ $\left(V_n \leq 3.75 A_{nv} \sqrt{f'_m}\right)$



Elevation

Ductility: 9.3.5.6.2.4

Boundary element not required if:

$$\frac{P_u}{A_n} + \frac{M_u}{S_n} < 0.2f'_m$$

– Note: assumes linear elastic behavior and uncracked section properties

– P_u, M_u from strength level load cases

Maximum Reinforcement: 9.3.5.6.1

A_s ≤ area required to maintain axial equilibrium under:

- Strain gradient of:
  - ε_{mu} in masonry
  - $\alpha\varepsilon_y$ in reinforcement
- Axial forces from
  - $D + 0.75L + 0.525Q_E$
- Compression reinforcement, with or without lateral restraining reinforcement, can be included.

Shear Wall Type	Yield Strain Multiplier, α	
	$\frac{M_u}{V_u d_v} < 1$	$\frac{M_u}{V_u d_v} \geq 1$
Intermediate	1.5	3.0
Special		4.0

Maximum Reinforcement: 9.3.5.6.1

- Uniformly distributed, max reinforcement per unit length:

$$\frac{A_s}{d_v} = \frac{0.64 f'_m t_{sp}\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{d_v}}{f_y\left(\frac{\alpha\varepsilon_y - \varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right)}$$

- Concentrated at ends and symmetric, ρ_{max} is:

$$\frac{A_s}{t_{sp} d} = \frac{0.64 f'_m\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{t_{sp} d}}{f_{y-min}\left(\varepsilon_{mu} - \frac{d'}{d}(\varepsilon_{mu} + \alpha\varepsilon_y), \varepsilon_y\right) E_s}$$

- P is from load combination D + 0.75L + 0.525Q_E
- Substitute t_{eq} for t_{sp} for partially grouted walls

Maximum Reinforcement: 9.3.5.6.1

- Determine greatest neutral axis depth for which strain limits can be met:

α	c/d_t, CMU	c/d_t, Clay
3	0.287	0.360
4	0.232	0.297

- Solve for P_n for that neutral axis location, including steel in compression.
- If $P_n \geq$ P from D + 0.75L + 0.525Q_E then OK

Ductility Requirements

Is

$\frac{M_u}{V_u l_w} \leq 1$ OR $\left\{V_u \leq 3A_{nv}\sqrt{f'_m}\right.$ AND $\left.\frac{M_u}{V_u l_w} \leq 3\right\}$

AND

$P_u \leq 0.10 A_g f'_m$: Geometrically symmetrical walls

$P_u \leq 0.05 A_g f'_m$: Geometrically unsymmetrical walls

Yes → Sufficient Ductility

No → Is $\frac{P_u}{A_g} + \frac{M_u}{S_g} < 0.2f'_m$

Yes → Sufficient Ductility

No →

Design for maximum reinforcement.

Area of flexural tensile reinforcement ≤ area required to maintain axial equilibrium under the following conditions

- A strain gradient corresponding to ε_{mu} in masonry and $\alpha\varepsilon_y$ in tensile reinforcement
- Axial forces from loading combination $D + 0.75L + 0.525Q_E$
- Compression reinforcement, with or without lateral restraining reinforcement, can be included.

Is $M_u/(V_u d_v) \geq 1$?

Yes → Intermediate reinforced walls: $\alpha = 3$; Special reinforced walls: $\alpha = 4$

No → $\alpha = 1.5$

Ordinary Reinforced Walls Maximum Axial Load: Tension Controlled

TMS 402-22 Table CC-9.1-1

Masonry Element	Concrete Masonry	Clay Masonry
Fully grouted section with single layer of tension reinforcement	$P_u \leq 0.19 f'_m bd - 0.9A_s f_y$	$P_u \leq 0.24 f'_m bd - 0.9A_s f_y$
Fully grouted shear wall subjected to in-plane loads with uniformly distributed reinforcement	$P_u \leq 0.19 f'_m bd_v - 0.48A_s f_y$	$P_u \leq 0.24 f'_m bd_v - 0.42A_s f_y$

Maximum Axial Load: Tension Controlled

- Maximum Axial Load, P_u, for Section to be **Tension Controlled**, In-Plane Loads
- 8 inch CMU, f'_m = 2 ksi , Grade 60 Reinforcement

Bar Size	Bar Spacing					
	8 in.	16 in.	24 in.	32 in.	40 in.	48 in.
	t_{eq} = 7.63 in.	t_{eq} = 5.17 in.	t_{eq} = 4.28 in.	t_{eq} = 3.83 in.	t_{eq} = 3.57 in.	t_{eq} = 3.39 in.
	8 in. unit					
No. 4	26.1 kip/ft	19.2 kip/ft	16.6 kip/ft	15.3 kip/ft	14.5 kip/ft	14.0 kip/ft
No. 5	21.4 kip/ft	16.8 kip/ft	15.0 kip/ft	14.1 kip/ft	13.6 kip/ft	13.2 kip/ft
No. 6	15.8 kip/ft	14.0 kip/ft	13.2 kip/ft	12.7 kip/ft	12.4 kip/ft	12.3 kip/ft

Shear Wall: Estimate Reinforcement

Following procedure uses a “smeared” reinforcement model to estimate the required reinforcement.

Solve quadratic equation for depth of compression block, a:

$$a = \frac{-B + \sqrt{B^2 - 4AC}}{2A}$$

$$A = (\beta - 1)0.36f'_m t_{sp}$$

$$B = 0.36f'_m t_{sp} d_v - 0.5\beta P_u$$

$$C = -M_u$$

$$\beta = \frac{\varepsilon_y + \varepsilon_{mu}}{0.8\varepsilon_{mu}}$$

For Grade 60 steel:

$\beta = 2.3$ for concrete masonry

$\beta = 2.0$ for clay masonry

Solve for required area of steel per inch of wall length:

$$A^*_{s,req'd} = \frac{0.8f'_m a t_{sp} - P_u/\phi}{f_y(d_v - \beta a)}$$

Method tends to slightly overestimate $A^*_{s,req'd}$, particularly for wider spaced reinforcement.

Example: Ordinary Shear Wall

Given: 10 ft high x 16 ft long 8 in. CMU shear wall; Grade 60 steel, Type S mortar; f'_m = 2000 psi; superimposed dead load of 1 kip/ft. In-plane seismic load of 50 kips. $S_{DS} = 0.5^{-}$ (just less than 0.5)

Required: Design the shear wall; ordinary reinforced shear wall

Solution: Check using 0.9D - E_v + E_h load combination.

- $M_u = (50\text{k})(10\text{ft}) = 500\text{k} \cdot \text{ft}$
- Axial load, P_u
  - Estimate wall weight as 45 psf
    - Wall weight: $45\text{psf}(10\text{ft})(16\text{ft}) = 7.2\text{k}$
  - $D = 1\,\text{k/ft}\,(16\text{ft}) + 7.2\text{k} = 23.2\text{k}$
  - $P_u = (0.9 - 0.2S_{DS})D = 0.80D = 0.80(23.2\text{k}) = 18.6\text{k}$

Example: Ordinary Shear Wall

Estimate reinforcement by solving quadratic equation
Assume $t_{eq} \sim 0.5t_{sp} = 3.8$ in.

$$A = (\beta - 1)0.36f'_m t_{sp} = (2.3 - 1)(0.36)(2\text{ksi})(3.8\text{in.}) = 3.557\,\frac{\text{kip}}{\text{in.}}$$

$$B = 0.36f'_m t_{sp} d_v - 0.5\beta P_u = 0.36(2\text{ksi})(3.8\text{in.})(192\text{in.}) - 0.5(2.3)(18.6\text{k}) = 503.9\text{k}$$

$$C = -M_u = -6000\text{k} \cdot \text{in.}$$

$$a = \frac{-B + \sqrt{B^2 - 4AC}}{2A} = \frac{-503.9\text{k} + \sqrt{(503.9\text{k})^2 - 4\left(3.557\frac{\text{kip}}{\text{in.}}\right)(-6000\text{k}\cdot\text{in.})}}{2\left(3.557\frac{\text{kip}}{\text{in.}}\right)} = 11.0\text{ in.}$$

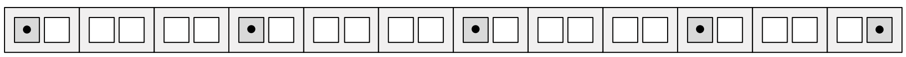
B is approximately the same magnitude of $\sqrt{B^2 - 4AC}$.
Carry extra significant figures to avoid cancellation of terms.

Example: Ordinary Shear Wall

Solve for required area of steel per inch of wall length:

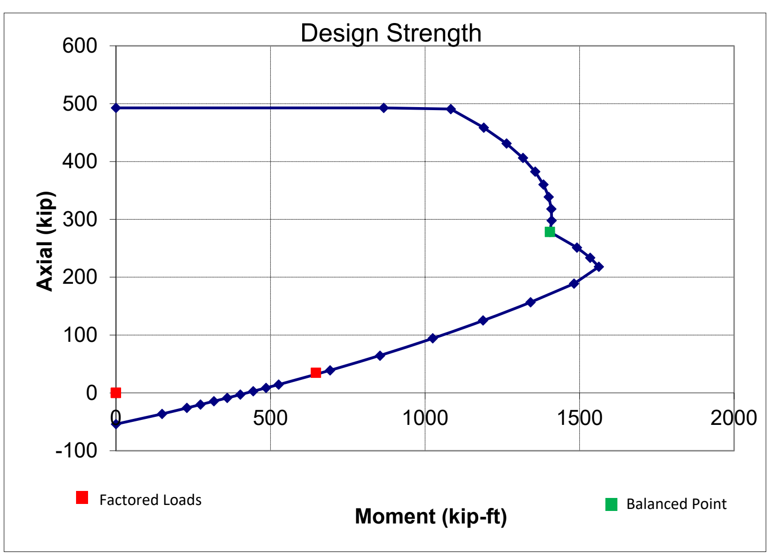
$$A^*_{s,req'd} = \frac{0.8f'_m a t_{sp} - P_u/\phi}{f_y(d_v - \beta a)} = \frac{0.8(2ksi)(11.0in.)(3.8in.) - 18.6k/0.9}{60ksi(192in. - 2.3(11.0in.))} = 0.00462\,\frac{in.^2}{in.} = 0.055\,\frac{in.^2}{ft}$$

Try #4 @ 48 in. (0.050 in.^{2}/ft)



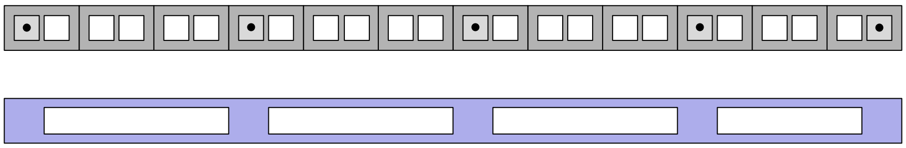
$t_{eq} = 3.39\text{in.}$

Example: Ordinary Shear Wall



Example: Ordinary Shear Wall

Design for shear; Net area is shown below



$$A_{nv} = 2(1.25\text{in.})(192\text{in.}) + 5(8\text{in.})(7.625\text{in.} - 2.5\text{in.}) = 685\text{in.}^2$$

Using equivalent thickness, $A_{nv} = 3.39\text{in.}(192\text{in.}) = 651\text{in.}^2$

Example: Ordinary Shear Wall

Shear Span: $\dfrac{M_u}{V_u d_v} = \dfrac{V_u h}{V_u d_v} = \dfrac{h}{d} = \dfrac{120\text{in.}}{192\text{in.}} = 0.625$

Max Shear:

$$\phi V_{n,max} = \phi\left[\frac{4}{3}\left(5 - 2\frac{M_u}{V_u d_v}\right)A_{nv}\sqrt{f'_m}\right]\gamma_g$$

$$= 0.8\left[\frac{4}{3}(5 - 2(0.625))(685\text{in.}^2)\sqrt{2000\text{psi}}\right]0.70 = 85.8\text{ kip}$$

OK

Masonry Shear:

$$\phi V_{nm} = \phi\left[\left(4 - 1.75\left(\frac{M}{V d_v}\right)\right)A_{nv}\sqrt{f'_m} + 0.25P_u\right]\gamma_g$$

$$= 0.8[(4 - 1.75(0.625))(685\text{in.}^2)\sqrt{2000\text{psi}} + 0.25(18600\text{lb})]0.70 = 52.5\text{ kip}$$

OK

2016: $\phi V_{nm} = 56.2\text{ kip}$

Example: Ordinary Shear Wall

Check shear friction; Assume concrete is unfinished ($\mu = 1.0$)

Flexural reinforcement was 5 - #4 bars; $A_{sp} = 5(0.20\text{in.}^2) = 1.00\text{in.}^2$

$$V_{nf} = \left(0.488 + 1.024\left(1 - \frac{M_u}{V_u d_v}\right)\right) A_{sp} f_y + \left(0.65 + 0.70\left(1 - \frac{M_u}{V_u d_v}\right)\right) P_u$$

$$V_{nf} = \big(0.488 + 1.024(1 - 0.625)\big)(1.00\text{in.}^2)(60\text{ksi}) + \big(0.65 + 0.70(1 - 0.625)\big)18.6\text{k} = 69.3\text{ kip}$$

2016: $V_{nf} = 70.2\text{ kip}$

$\phi V_{nf} = 0.8(69.3\text{k}) = 55.4\text{k}$ **OK**

Thank You

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