Design of Masonry Shear Walls

May 10, 2018

Richard Bennett, PhD, PE
Professor, Civil and Environmental Engineering
The University of Tennessee

THE MASONRY SOCIETY

The Masonry Society

AIA Provider: 50119857

The Masonry Society is a registered Provider with the American Institute of Architects Continuing Education Systems. Credit earned on completion of this program will be reported to CES Records for AIA members. Certificates of completion for non-AIA members are available upon request.

This program is registered with AIA/CES for continuing professional education. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the AIA of any material of construction or any method or manner of handling, using, distributing or dealing in any material or product.

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

Course Description

The design of reinforced masonry shear walls using both the Allowable Stress Design method and the Strength Design method will be covered in this webinar. The requirements for both the design for overturning and the design for shear are explained. The prescriptive seismic detailing requirements for each of three reinforced masonry shear walls types, ordinary, intermediate, and special, are explained, with particular attention to special reinforced shear walls. The design process is illustrated with examples.

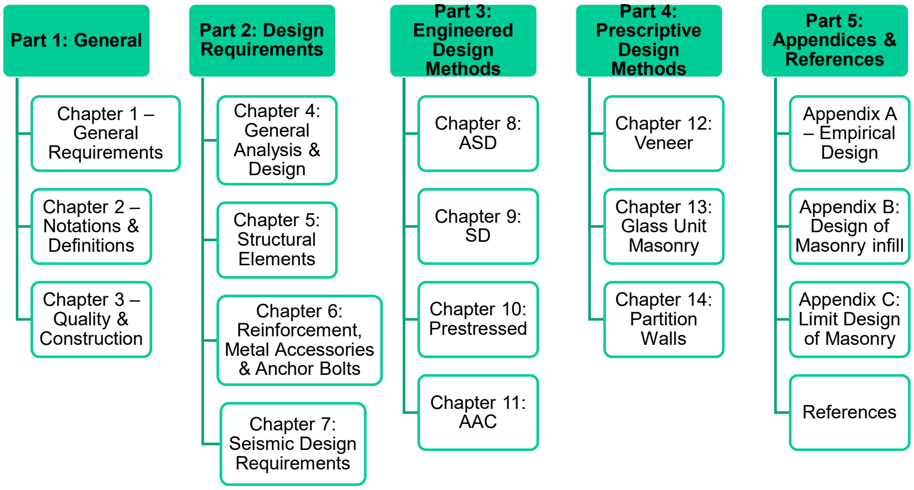
Learning Objectives

- Understand the three different types of reinforced masonry shear walls and the requirements for each type.
- Understand seismic design requirements for shear walls.
- Understand the design process for shear walls using the Allowable Stress Design method.
- Understand the design process for shear walls using the Strength Design method.

Recent Changes to TMS 402

- 2013
  - Reorganization
  - Partially Grouted Shear Wall Factor
  - Unit Strength Table
- 2016
  - Anchor Bolt
    - Shear Strength
    - Tension and Shear Interaction
  - Shear Friction

TMS 402 Reorganization



- **Part 1: General**
  - Chapter 1 – General Requirements
  - Chapter 2 – Notations & Definitions
  - Chapter 3 – Quality & Construction
- **Part 2: Design Requirements**
  - Chapter 4: General Analysis & Design
  - Chapter 5: Structural Elements
  - Chapter 6: Reinforcement, Metal Accessories & Anchor Bolts
  - Chapter 7: Seismic Design Requirements
- **Part 3: Engineered Design Methods**
  - Chapter 8: ASD
  - Chapter 9: SD
  - Chapter 10: Prestressed
  - Chapter 11: AAC
- **Part 4: Prescriptive Design Methods**
  - Chapter 12: Veneer
  - Chapter 13: Glass Unit Masonry
  - Chapter 14: Partition Walls
- **Part 5: Appendices & References**
  - Appendix A – Empirical Design
  - Appendix B: Design of Masonry infill
  - Appendix C: Limit Design of Masonry
  - References

Partially Grouted Shear Wall Factor

$V_n = (V_{nm} + V_{ns})\gamma_g$ 9.3.4.1.2, Equation (9-21)

γ_g = 0.75 for partially grouted shear walls
= and 1.0 otherwise

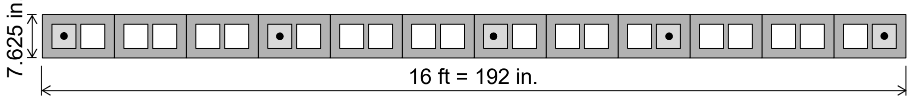
	$\frac{V_{experimental}}{V_{nominal}}$	
	Mean	St Dev
Fully grouted (Davis et al, 2010)	1.16	0.17
Partially grouted (Minaie et al, 2010)	0.90	0.26

$$\frac{0.90}{1.16} = 0.776$$

Partially Grouted Shear Wall Factor

Methods to calculate shear strength of partially grouted shear walls (Minaie et al, 2010)	$\frac{V_{experimental}}{V_{nominal}}$	
	Mean	St Dev
2008 MSJC Code	0.90	0.26
Multiply shear strength by A_n/A_g	1.53	0.43
Using just face shells	1.77	0.78

Calculate net area, A_{nv}, including grouted cells.



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

Table 2, TMS 602, CMU Unit Strength Table

Net area compressive strength of concrete masonry, psi	Net area compressive strength of ASTM C90 concrete masonry units, psi (MPa)	
	Type M or S Mortar	Type N Mortar
1,700	---	1,900
1,900	1,900	2,350
2,000	2,000	2,650
2,250	2,600	3,400
2,500	3,250	4,350
2,750	3,900	----

---

f'_m = 2000 psi vs. f'_m = 1500 psi

- Small effect on overturning (flexural reinforcement)
- Significant effect on maximum reinforcement
- 13% decrease in development and splice length
- 15% increase in masonry shear strength

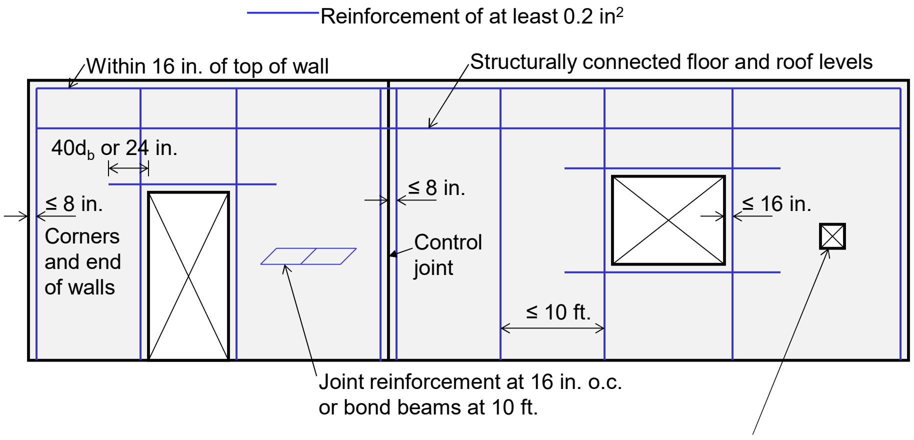
Shear Wall Types

- Plain
  - Ordinary
  - Detailed
- Reinforced
  - Ordinary
  - Intermediate
  - Special

Shear Wall Types

Shear Wall Type	**Minimum Reinforcement**	**Seismic Design Category**
Ordinary Plain	none	A, B
Detailed Plain	vertical reinforcement = 0.2 in.^{2} at corners, within 16 in. of openings, within 8 in. of movement joints, maximum spacing 10 ft; horizontal reinforcement W1.7 @ 16 in. or #4 in bond beams @ 10 ft	A, B
Ordinary Reinforced	same as above	A, B, C
Intermediate Reinforced	same as above, but vertical reinforcement @ 4 ft	A, B, C
Special Reinforced	same as above, but horizontal reinforcement @ 4 ft, and $\rho_v + \rho_h \geq 0.002$, and ρ_v and $\rho_h \geq 0.0007$	any

Ordinary Wall: Reinforcement



Joint reinforcement at 16 in. o.c. or bond beams at 10 ft.

Reinforcement not required at openings smaller than 16 in. in either vertical or horizontal direction

Special Walls: Reinforcement

- ❑ Maximum spacing of vertical reinforcement smallest of:
  - one-third length of wall
  - one-third height of wall
  - 48 in. for running bond; 24 in. not laid in running bond
- ❑ Maximum spacing of horizontal reinforcement required to resist shear same as above
- ❑ $\rho_v + \rho_h \geq 0.002$, and ρ_v and $\rho_h \geq 0.0007$
- ❑ Shear reinforcing anchored around vertical bars with a standard hook

Shear Capacity Design

- Allowable Stress Design
  - Calculated shear stress increased by 1.5
  - Allowable shear stress due to masonry approximately 1/2
- Strength Design
  - Design shear strength, ϕV_n, greater than shear corresponding to 1.25 times nominal flexural strength, M_n
  - Except V_n need not be greater than $2.5V_u$.
  - Normal design: ϕV_n has to be greater than V_u. Thus, V_n has to be greater than $V_u/\phi = V_u/0.8 = 1.25V_u$. This requirement doubles the shear.

SDC D Material Requirements

- Type S or Type M cement-lime mortar or mortar cement mortar
- 2013: Masonry cement mortar permitted for fully grouted members

Allowable Stress Design

Allowable Stress Design

- Tension
  - Grade 60 32,000 psi
  - Wire joint reinforcement 30,000 psi
- Stress in masonry from axial load plus bending:
  - $0.45f'_m$ (if allowable masonry stress controls, reinforcement is not being used efficiently)
- Axial
  - $P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left[1 - \left(\frac{h}{140r}\right)^2\right]$ for $\frac{h}{r} \leq 99$
  - $P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left(\frac{70r}{h}\right)^2$ for $\frac{h}{r} > 99$

Allowable Stress Design

- Shear stress is computed as:
  - $f_v = \frac{V}{A_{nv}}$
- Allowable shear stress
  - $F_v = (F_{vm} + F_{vs})\gamma_g$
  - $\gamma_g = 0.75$ for partially grouted shear walls, 1.0 otherwise
- Allowable stress limit
  - $M/(Vd_v) \leq 0.25$ $\quad F_v = \left(3\sqrt{f'_m}\right)\gamma_g$
  - $M/(Vd_v) \geq 1.0$ $\quad F_v = \left(2\sqrt{f'_m}\right)\gamma_g$
  - $0.25 < M/(Vd_v) < 1.0$ $\quad F_v = \left(\frac{2}{3}\left(5 - 2\frac{M}{Vd_v}\right)\right)\gamma_g$

Allowable Stress Design

- Allowable masonry shear stress

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

  - Special reinforced walls:

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

- Allowable reinforcement shear stress
  - $F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right)$
  - Shear reinforcement is placed parallel the direction of the applied force at a maximum spacing of $d/2$ or 48 in.
  - One - third of A_v is required perpendicular to the applied force at a spacing of no more than 8 ft.

Shear Friction

Shear Span Ratio	Allowable Shear Friction
$\frac{M}{Vd_v} \leq 0.5$	$F_f = \frac{\mu\left(A_{sp}F_s + P\right)}{A_{nv}}$
$0.5 < \frac{M}{Vd_v} < 1.0$ Linear interpolation	$F_f = \frac{\left(0.39 + \frac{\mu - 0.39}{0.5}\left(1 - \frac{M}{Vd_v}\right)\right)A_{sp}F_s + \left(0.65 + \frac{\mu - 0.65}{0.5}\left(1 - \frac{M}{Vd_v}\right)\right)P}{A_{nv}}$
$\frac{M}{Vd_v} \geq 1.0$	$F_f = \frac{0.65\left(0.6A_{sp}F_s + P\right)}{A_{nv}}$

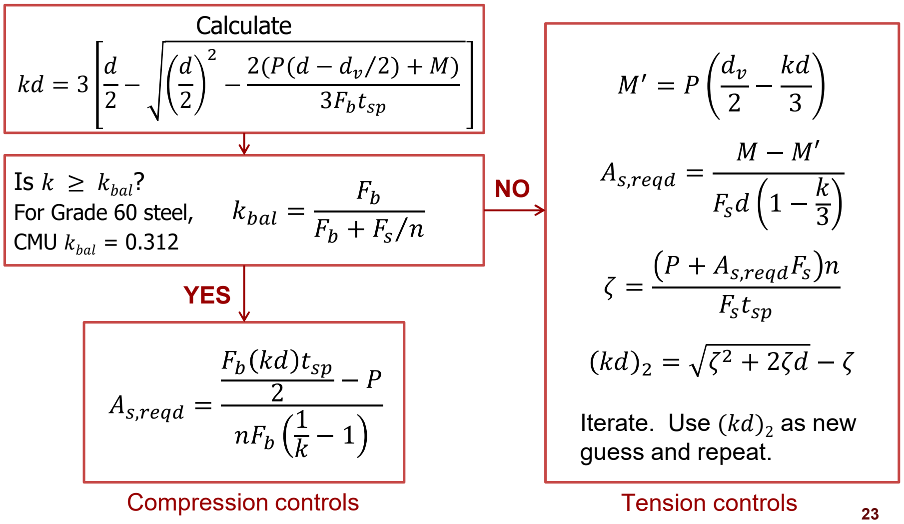
A_{sp} = cross-sectional area of reinforcement within the net shear area, perpendicular to and crossing the horizontal shear plane

Shear Friction

- μ = 1.0 for masonry on concrete with unfinished surface, or concrete with a surface that has been intentionally roughened
    - UBC (1997) required concrete abutting structural masonry to be roughened to a full amplitude of 1/16 inch.
- μ = 0.70 for all other conditions
- For $0.5 < \frac{M}{Vd_v} < 1.0$
    - $\mu = 1.0 \quad F_f = \frac{\left(0.39+1.22\left(1-\frac{M}{Vd_v}\right)\right)A_{sp}F_s+\left(0.65+0.70\left(1-\frac{M}{Vd_v}\right)\right)P}{A_{nv}}$
    - $\mu = 0.7 \quad F_f = \frac{\left(0.39+0.62\left(1-\frac{M}{Vd_v}\right)\right)A_{sp}F_s+\left(0.65+0.10\left(1-\frac{M}{Vd_v}\right)\right)P}{A_{nv}}$

Special reinforced shear walls: The 1.5 multiplier should not be applied to V when calculating the $M/(Vd_v)$ ratio, or for shear friction design.

Design Procedure: Single Layer of Reinforcement



Calculate

$$kd = 3\left[\frac{d}{2} - \sqrt{\left(\frac{d}{2}\right)^2 - \frac{2(P(d - d_v/2) + M)}{3F_b t_{sp}}}\right]$$

Is $k \geq k_{bal}$?
For Grade 60 steel,
CMU k_{bal} = 0.312

$$k_{bal} = \frac{F_b}{F_b + F_s/n}$$

YES

$$A_{s,reqd} = \frac{\frac{F_b(kd)t_{sp}}{2} - P}{nF_b\left(\frac{1}{k} - 1\right)}$$

Compression controls

NO

$$M' = P\left(\frac{d_v}{2} - \frac{kd}{3}\right)$$

$$A_{s,reqd} = \frac{M - M'}{F_s d\left(1 - \frac{k}{3}\right)}$$

$$\zeta = \frac{(P + A_{s,reqd}F_s)n}{F_s t_{sp}}$$

$$(kd)_2 = \sqrt{\zeta^2 + 2\zeta d} - \zeta$$

Iterate. Use $(kd)_2$ as new guess and repeat.

Tension controls

Design Procedure: Single Layer of Reinforcement

If $k < k_{bal}$ tension controls; determine kd from cubic equation.

$$\frac{t_{sp}F_s}{6n}[kd]^3 - \frac{t_{sp}dF_s}{2n}[kd]^2 - \left(P\left(d - \frac{d_v}{2}\right) + M\right)[kd] + \left(P\left(d - \frac{d_v}{2}\right) + M\right)d = 0$$

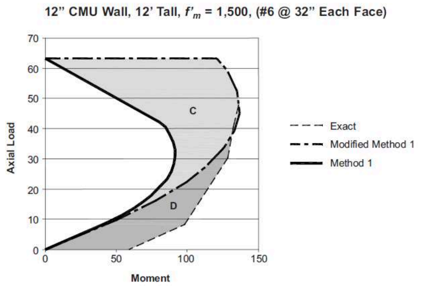
$$A_{s,reqd} = \frac{1}{2}(kd)t_{sp}\left(\frac{1}{n}\frac{kd}{d - kd}\right) - \frac{P}{F_s}$$

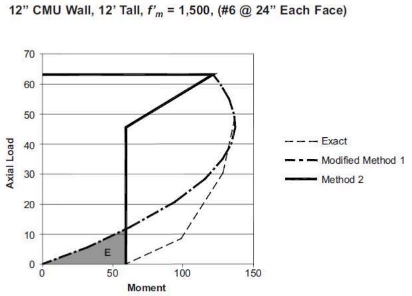
Determination of k_{bal}:

$$k_{bal} = \frac{F_b}{F_b + \frac{F_s}{n}} = \frac{F_b}{F_b + \frac{F_s}{\frac{E_s}{E_m}}} = \frac{0.45f'_m}{0.45f'_m + \frac{32\text{ksi}}{\frac{29000\text{ksi}}{900f'_m}}} = \frac{0.45}{0.45 + \frac{32}{\frac{29000}{900}}} = 0.312$$

For clay masonry, $E_m = 700f'_m$, $k_{bal} = 0.368$

Design Procedures





Method 1: $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$

Modified Method 1: $f_a + f_b \leq F_b$

Method 2: $F'_b = F_b - f_a$

Reinforced Masonry Engineering Handbook, 7th edition

Example: ASD-1

Given: 2 ft long, 8 ft high CMU wall; Type S masonry cement mortar; Grade 60 steel; fully grouted. P = 5 kips; M = 18 k-ft (216 k-in.)

Required: Required amount of steel

Solution: Choose/determine material properties.

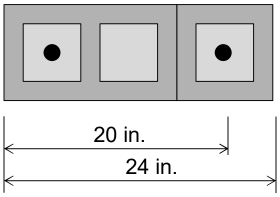
f'_m = 2000 psi

E_m = 900f'_m = 1,800,000 psi

F_b = 0.45f'_m = 900 psi

F_s = 32,000 psi

$n = E_s/E_m$ = 29,000,000 / 1,800,000 = 16.11



Example: ASD-1

Assume masonry compression controls. Determine kd.

$$kd = 3\left[\frac{d}{2} - \sqrt{\left(\frac{d}{2}\right)^2 - \frac{2(P(d - d_v/2) + M)}{3F_b t_{sp}}}\right]$$

$$= 3\left[\frac{20\text{in.}}{2} - \sqrt{\left(\frac{20\text{in.}}{2}\right)^2 - \frac{2(5\text{k}(20\text{in.} - 24\text{in.}/2) + 216\text{k}\cdot\text{in.})}{3(0.9\text{ksi})(7.625\text{in.})}}\right] = 4.00\text{in.}$$

Compare to k_{bal}

$$k = \frac{kd}{d} = \frac{4.00\text{in.}}{20\text{in.}} = 0.200 < 0.312 = k_{bal}$$

Steel tension stress controls; iterate with kd = 4.00 in. as initial guess.

Example: ASD-1

$$M' = P\left(\frac{d_v}{2} - \frac{kd}{3}\right) = 5\text{k}\left(\frac{24\text{in.}}{2} - \frac{4.00\text{in.}}{3}\right) = 53.33\text{k}\cdot\text{in.}$$

$$A_{s,reqd} = \frac{M - M'}{F_s d\left(1 - \frac{k}{3}\right)} = \frac{216\text{k}\cdot\text{in.} - 53.3\text{k}\cdot\text{in.}}{32\text{ksi}(20\text{in.})\left(1 - \frac{0.200}{3}\right)} = 0.272\text{in.}^2$$

$$\zeta = \frac{(P + A_{s,reqd}F_s)n}{F_s t_{sp}} = \frac{(5\text{k} + 0.272\text{in.}^2\,(32\text{ksi})\,)16.11}{32\text{ksi}(7.625\text{in.})} = 0.9055\text{in.}$$

$$(kd)_2 = \sqrt{\zeta^2 + 2\zeta d} - \zeta = \sqrt{(0.9055\text{in.})^2 + 2(0.9055\text{in.})20\text{in.}} - 0.9055\text{in.}$$

$$= 5.181\text{in.}$$

Iterate. Use kd = 5.181in. as new guess and repeat until convergence.

Example: ASD-1

Equation / Value	Iteration 1	Iteration 2	Iteration 3
kd (in.)	4.00	5.181	5.228
k	0.200	0.259	0.261
$M' = P\left(\frac{d_v}{2} - \frac{kd}{3}\right)$ (k-in.)	53.33	51.36	51.29
$A_{s,reqd} = \frac{M-M'}{F_s d\left(1-\frac{k}{3}\right)}$ (in^2)	0.272	0.281	0.282
$\zeta = \frac{(P+A_{s,reqd}F_s)n}{F_s t_{sp}}$ (in.)	0.9055	0.9250	0.9258
$(kd)_2 = \sqrt{\zeta^2 + 2\zeta d} - \zeta$ (in.)	5.181	5.228	5.230

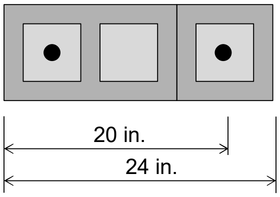
$A_{s,reqd}$ = 0.28in.^{2} Use 1 - #5 each face

Example: ASD-1

Given: 2 ft long, 8 ft high CMU pier; Type S masonry cement mortar; Grade 60 steel; fully grouted. P = 5 kips; M = 18 k-ft; V = 4.5 kips

Required: Check shear

Solution: Material properties stay the same



$$f_v = \frac{V}{A_{nv}} = \frac{4.5\text{k}}{24\text{in.}\,(7.625\text{in.})} = \frac{4.5\text{k}}{183\text{in.}^2} = 24.6\text{psi}$$

Example: ASD-1

Determine allowable shear stress due to masonry, F_{vm}

$\frac{M}{Vd_v} = \frac{18\text{k}\cdot\text{ft}}{4.5\text{k}(2\text{ft})} = 2.0$ Use $M/(Vd_v)$ = 1.0

$$F_v = (F_{vm})\gamma = F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}$$

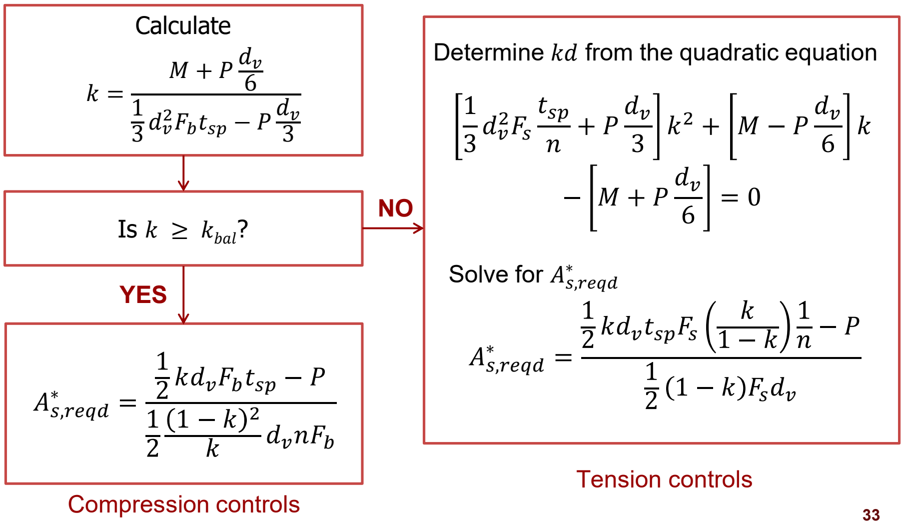
$$= \frac{1}{2}\left[(4.0 - 1.75(1.0))\sqrt{2000\text{psi}}\right] + 0.25\frac{5000\text{lb}}{183\text{in.}^2} = 57.1\text{ psi}$$

f_v = 24.6 psi < F_{vm} = 57.1 psi **OK**

Design: Distributed Reinforcement

- Design method similar to single layer of reinforcement
  - Based on uniformly distributed reinforcement, A_s^*
  - Tends to overestimate reinforcement by 10-15% for wider spaced reinforcement
  - Use specified thickness, even for partial grout
- Interaction diagram to check capacity
- Spacing of intermediate reinforcing bars often controlled by out-of-plane loading

Design: Distributed Reinforcement



Calculate

$$k = \frac{M + P\frac{d_v}{6}}{\frac{1}{3}d_v^2 F_b t_{sp} - P\frac{d_v}{3}}$$

Is $k \geq k_{bal}$?

YES

$$A^*_{s,reqd} = \frac{\frac{1}{2}k d_v F_b t_{sp} - P}{\frac{1}{2}\frac{(1-k)^2}{k} d_v n F_b}$$

Compression controls

NO

Determine kd from the quadratic equation

$$\left[\frac{1}{3}d_v^2 F_s \frac{t_{sp}}{n} + P\frac{d_v}{3}\right]k^2 + \left[M - P\frac{d_v}{6}\right]k - \left[M + P\frac{d_v}{6}\right] = 0$$

Solve for $A^*_{s,reqd}$

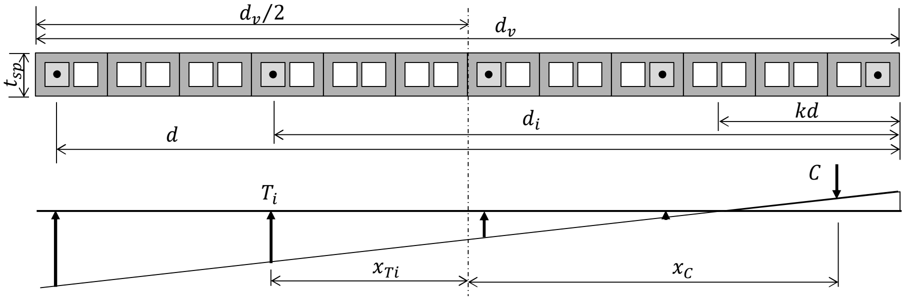
$$A^*_{s,reqd} = \frac{\frac{1}{2}k d_v t_{sp} F_s\left(\frac{k}{1-k}\right)\frac{1}{n} - P}{\frac{1}{2}(1-k)F_s d_v}$$

Tension controls

Distributed Reinforcement

Spacing (inches)	Steel Area, A_s^* (in.^{2}/ft)			
	#3	#4	#5	#6
8	0.16	0.30	0.46	0.66
16	0.082	0.15	0.23	0.33
24	0.055	0.10	0.16	0.22
32	0.041	0.075	0.12	0.16
40	0.033	0.060	0.093	0.13
48	0.028	0.050	0.078	0.11
56	0.024	0.043	0.066	0.094
64	0.021	0.038	0.058	0.082
72	0.018	0.033	0.052	0.073
80	0.016	0.030	0.046	0.066
88	0.015	0.027	0.042	0.060
96	0.014	0.025	0.039	0.055
104	0.013	0.023	0.036	0.051
112	0.012	0.021	0.033	0.047
120	0.011	0.020	0.031	0.044

Interaction Diagram



$$P = C - \sum_{d_i > kd} T_i$$

$$M = Cx_C + \sum_{d_i > kd} T_i x_{Ti}$$

Interaction Diagram

Stress	Force	Moment Arm
If $k > k_{bal}$ $f_b = F_b$ $f_s = F_b n \frac{d - kd}{kd}$ If $k \leq k_{bal}$ $f_s = F_s$ $f_b = \frac{F_s}{n} \frac{kd}{d - kd}$ $f_{si} = f_s \frac{d_i - kd}{d - kd}$	$C = \frac{1}{2} f_b (kd) t_{eq}$ $T_i = A_{si} f_{si}$	$x_C = \frac{d_v}{2} - \frac{kd}{3}$ $x_{T_i} = d_i - \frac{d_v}{2}$

Partial Grout: Equivalent Thickness

Grout Spacing (in.)	Equivalent Thickness, t_{eq} (in.)	
	8 in. CMU	12 in. CMU
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
96	2.94	3.30
120	2.86	3.13

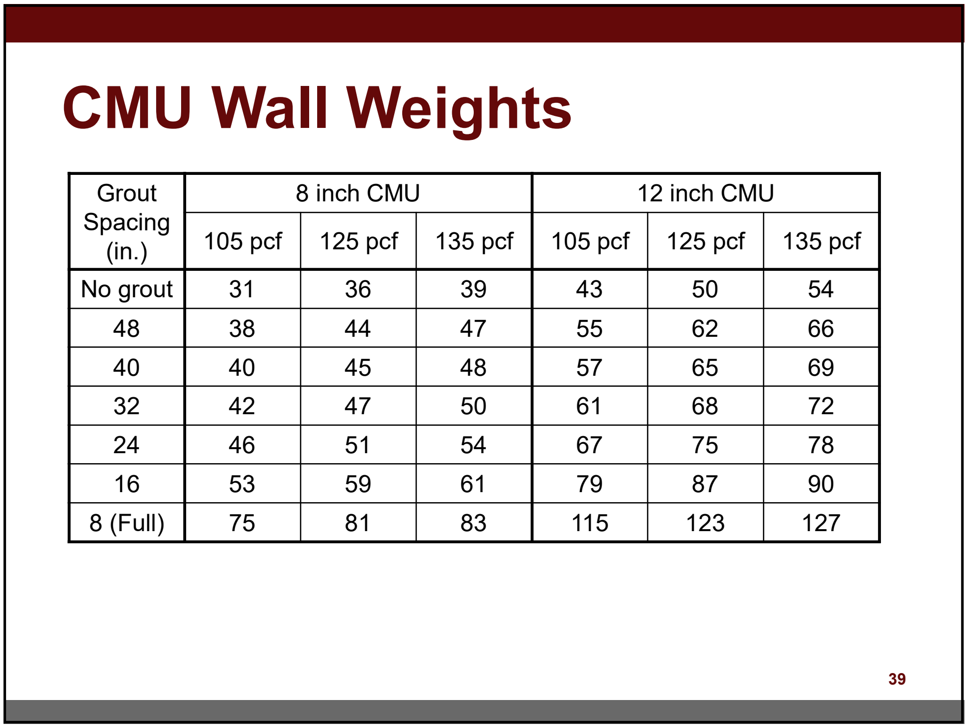
Example: ASD-2

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

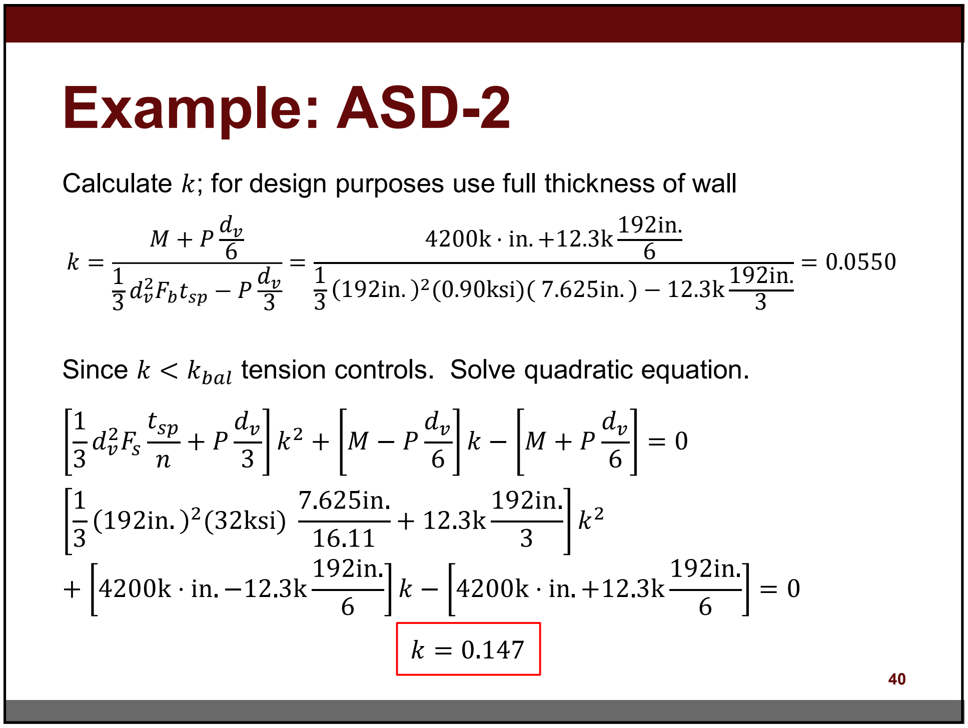
Solution: Check using 0.6D+0.7E load combination.

- $M = 0.7(50\text{k})(10\text{ft}) = 350\text{k}\cdot\text{ft} = 4200\text{k}\cdot\text{ft}$
- Axial load, P
    - Need to know weight of wall to determine P.
    - Need to know reinforcement spacing to determine wall weight
    - Based on next page, estimate wall weight as 45 psf
        - Wall weight: $45\text{psf}(10\text{ft})(16\text{ft}) = 7.2\text{k}$
    - $D = 1\,\text{k/ft}\,(16\text{ft}) + 7.2\text{k} = 23.2\text{k}$
    - $P = (0.6 - 0.7(0.2)S_{DS})D = 0.53D = 0.53(23.2\text{k}) = 12.3\text{k}$



CMU Wall Weights

Grout Spacing (in.)	8 inch CMU			12 inch CMU		
	105 pcf	125 pcf	135 pcf	105 pcf	125 pcf	135 pcf
No grout	31	36	39	43	50	54
48	38	44	47	55	62	66
40	40	45	48	57	65	69
32	42	47	50	61	68	72
24	46	51	54	67	75	78
16	53	59	61	79	87	90
8 (Full)	75	81	83	115	123	127



Example: ASD-2

Calculate k; for design purposes use full thickness of wall

$$k = \frac{M + P\frac{d_v}{6}}{\frac{1}{3}d_v^2 F_b t_{sp} - P\frac{d_v}{3}} = \frac{4200\text{k}\cdot\text{in.} + 12.3\text{k}\frac{192\text{in.}}{6}}{\frac{1}{3}(192\text{in.})^2(0.90\text{ksi})(7.625\text{in.}) - 12.3\text{k}\frac{192\text{in.}}{3}} = 0.0550$$

Since $k < k_{bal}$ tension controls. Solve quadratic equation.

$$\left[\frac{1}{3}d_v^2 F_s \frac{t_{sp}}{n} + P\frac{d_v}{3}\right]k^2 + \left[M - P\frac{d_v}{6}\right]k - \left[M + P\frac{d_v}{6}\right] = 0$$

$$\left[\frac{1}{3}(192\text{in.})^2(32\text{ksi})\,\frac{7.625\text{in.}}{16.11} + 12.3\text{k}\frac{192\text{in.}}{3}\right]k^2$$

$$+\left[4200\text{k}\cdot\text{in.} - 12.3\text{k}\frac{192\text{in.}}{6}\right]k - \left[4200\text{k}\cdot\text{in.} + 12.3\text{k}\frac{192\text{in.}}{6}\right] = 0$$

$$\boxed{k = 0.147}$$

Example: ASD-2

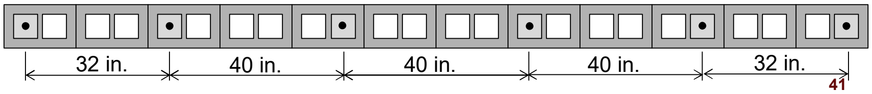
Calculate required area of reinforcement

$$A^*_{s,reqd} = \frac{\frac{1}{2}kd_v t_{sp} F_s \left(\frac{k}{1-k}\right)\frac{1}{n} - P}{\frac{1}{2}(1-k)F_s d_v}$$

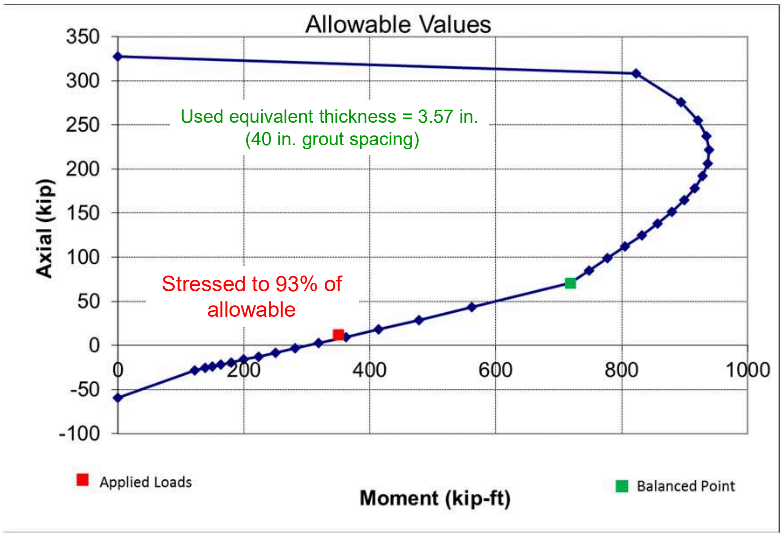
$$= \frac{\frac{1}{2}(0.147)(192\text{in.})(7.625\text{in.})(32\text{ksi})\left(\frac{0.147}{1-0.147}\right)\frac{1}{16.11} - 12.3\text{k}}{\frac{1}{2}(1-0.147)(32\text{ksi})(192\text{in.})}$$

$$= 0.00934\frac{\text{in.}^2}{\text{in.}} = 0.112\frac{\text{in.}^2}{\text{ft}}$$

Try #5 @ 32 in. ($0.120\text{in.}^2/\text{ft}$)
Due to module; use 40 in. ($0.093\text{in.}^2/\text{ft}$) for interior bars



Example: ASD-2

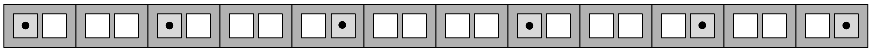


Used equivalent thickness = 3.57 in.
(40 in. grout spacing)

Stressed to 93% of allowable

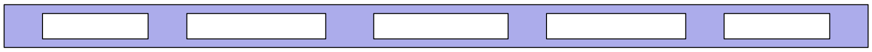
Example: ASD-2

Given: 10 ft high x 16 ft long 8 in. CMU shear wall; Grade 60 steel, f'_m =2000psi; #5 at 32in. ends; #5 @ 40in. interior; superimposed dead load of 1 kip/ft. In-plane seismic load of 50 kips.



Required: Design for shear

Solution: Net area is shown below



$$A_{nv} = 2(1.25\text{in.})(192\text{in.}) + 6(8\text{in.})(7.625\text{in.} - 2.5\text{in.}) = 726\text{in.}^2$$

Example: ASD-2

Shear Stress: $$f_v = \frac{V}{A_{nv}} = \frac{0.7(50\text{k})}{726\text{in.}^2} = 48.2\text{psi}$$

Shear Span: $$\frac{M}{Vd_v} = \frac{Vh}{Vd_v} = \frac{h}{d} = \frac{120\text{in.}}{192\text{in.}} = 0.625$$

Max Shear: $$F_{v,max} = \left[\frac{2}{3}\left(5 - 2\frac{M}{Vd_v}\right)\sqrt{f'_m}\right]\gamma_g$$
$$= \left[\frac{2}{3}(5 - 2(0.625))\sqrt{2000\text{psi}}\right]0.75 = 83.8\text{psi}$$

OK

Masonry Shear: $$F_v = (F_{vm})\gamma_g = \left[\frac{1}{2}\left[\left(4 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n}\right]\gamma_g$$
$$= \left[\frac{1}{2}\left[(4 - 1.75(0.625))\sqrt{2000\text{psi}}\right] + 0.25\frac{12300\text{lb}}{726\text{in.}^2}\right]0.75$$
$$= 51.9\text{psi}$$

OK

Example: ASD-3

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. $S_{DS} = 0.5$

Required: Design the shear wall; **special reinforced shear wall**

Solution:

- Flexural reinforcement remains the same (although ASCE 7 allows a load factor of 0.9 for dead load for special shear walls)
- Design for 1.5V, or 1.5(0.7)(50 kips) = 52.5 kips (Section 7.3.2.6.1.2)
- $f_v = 52.5\text{k}/726\text{in.}^2 = 72.3\text{psi}$
- Maximum $F_v = 83.8\text{psi}$ **OK**

Example: ASD-3

Masonry Stress:

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

$$= \frac{1}{4}\left[\left(4 - 1.75(0.625)\right)\sqrt{2000\text{psi}}\right] + 0.25\frac{12300\text{lb}}{726\text{in.}^2} = 36.7\text{psi}$$

Required Steel Stress:

$$F_{vs,reqd} = \frac{f_v}{\gamma_g} - F_{vm} = \frac{72.3\text{psi}}{0.75} - 36.7\text{psi} = 59.7\text{psi}$$

Determine spacing:
Use #5 bars

$$F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right) \quad \Rightarrow \quad s = \frac{0.5 A_v F_s d_v}{F_{vs,reqd} A_{nv}}$$

$$s = \frac{0.5(0.31\text{in.}^2)(32000\text{psi})(192\text{in.})}{(59.7\text{psi})(726\text{in.}^2)} = 22.0\text{in.}$$

Use #5 at 16 in. o.c.

Alternate is 2-#4 at 24 in.
(s=28.3 in.)

Example: ASD-3

Due to closely spaced horizontal reinforcement, fully grout wall.

$A_{nv} = 1464\text{in.}^2$

$f_v = 35.6\text{psi}$

$D = 1\,\text{k/ft}\,(16\text{ft}) + 0.081\text{ksf}(10\text{ft})(16\text{ft}) \; = 29.0\text{k}$

$P = 0.53D = 0.53(29.0\text{k}) = 15.3\text{k}$

$F_{vm} = 35.1\text{psi}$

$\gamma = 1.0$

$F_{vs,reqd} = 0.5\text{psi}$

Horizontal reinforcement will be controlled by prescriptive reinforcement

---

Example: ASD-3

Section 8.3.4.4 *Maximum Reinforcement*

Requirements only apply to special reinforced shear walls

No need to check maximum reinforcement since only need to check if:

- $M/(Vd_v) \; \geq 1$ and $M/(Vd_v) \; = \; 0.625$
- $P \; > \; 0.05f'_m A_n$
    - $0.05(2000\text{psi})(1464\text{in.}^2\,) = 146\text{kips}$
    - Assume a live load of 1 k/ft
    - $P = (1.2 + 0.7(0.2)S_{DS}) + L = (1.2 + 0.7(0.2)0.5)29\text{k} + 16\text{k} = 52.8\text{k}$

OK

Example: ASD-3

If we needed to check maximum reinforcing, do as follows.

$$\rho_{max} = \frac{nf'_m}{2f_y\left(n + \frac{f_y}{f'm}\right)} = \frac{16.1(2\text{ksi})}{2(60\text{ksi})\left(16.1 + \frac{60\text{ksi}}{2\text{ksi}}\right)} = 0.00582$$

For distributed reinforcement, the reinforcement ratio is obtained as the total area of tension reinforcement divided by *bd*. Assume 5 out of 6 bars in tension.

$$\rho = \frac{A_s}{bd} = \frac{5(0.31\text{in.}^2)}{7.625\text{in.}(188\text{in.})} = 0.00108$$

OK

Example: ASD-3

Maximum Spacing Requirements (7.3.2.6)

maximum spacing smallest of one-third length, one-third height, 48 in.

$$s_{max} = \min\left\{\frac{192in.}{3}, \frac{120in.}{3}, 48in.\right\} = \min\{64in., 40in., 48in.\} = 40in.$$

Prescriptive Reinforcement Requirements (7.3.2.6)

- $\rho \geq 0.0007$ in each direction
- $\rho_v + \rho_h \geq 0.002$

Vertical: $\rho_v = \frac{6(0.31\text{in.}^2)}{1464in.^2} = 0.00127$ **OK**

Horizontal: Try #5 @ 40 in.

$\rho_h = \frac{3(0.31\text{in.}^2)}{120\text{in.}(7.625\text{in.})} = 0.00102$ **OK**

Total: $\rho_v + \rho_h = 0.00127 + 0.00102 = 0.00229$ **OK**

Example: ASD-3

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. $S_{DS} = 0.5$

Required: Check shear friction

Solution:

- Assume concrete is unfinished ($\mu = 1.0$)
- Flexural reinforcement was 6 - #5 bars; $A_{sp} = 6(0.31\text{in.}^2) = 1.86\text{in.}^2$
- $P = 15.3\text{k}$
- $M/(Vd_v) = 0.625$
- $A_{nv} = 1464\text{in.}^2$
- $f_v = 35.6\text{psi}$

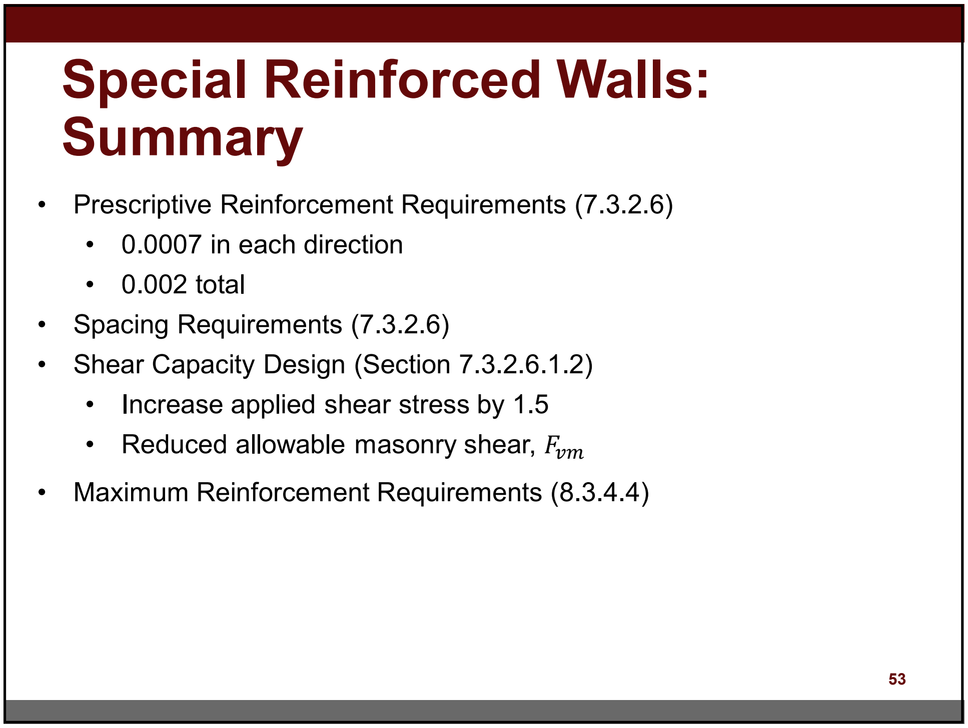
Example: ASD-3

$$F_f = \frac{\left(0.39+1.22\left(1-\frac{M}{Vd_v}\right)\right)A_{sp}F_s+\left(0.65+0.70\left(1-\frac{M}{Vd_v}\right)\right)P}{A_{nv}}$$

$$= \frac{\big(0.39 + 1.22(1 - 0.625)\big)(1.86\text{in.}^2)(32000\text{psi}) + \big(0.65 + 0.70(1 - 0.625)\big)15300\text{lb}}{1464\text{in.}^2}$$

$$= 44.0\text{ psi}$$

$f_v = 35.6\text{psi}$ **OK**



Special Reinforced Walls: Summary

- Prescriptive Reinforcement Requirements (7.3.2.6)
  - 0.0007 in each direction
  - 0.002 total
- Spacing Requirements (7.3.2.6)
- Shear Capacity Design (Section 7.3.2.6.1.2)
  - Increase applied shear stress by 1.5
  - Reduced allowable masonry shear, F_{vm}
- Maximum Reinforcement Requirements (8.3.4.4)

Strength Design

Strength Design

- Strength-reduction factors
  - Combinations of flexure and axial: $\phi = 0.9$
  - Shear: $\phi = 0.8$
- Nominal axial capacity

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

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

Strength Design Assumptions

- continuity between reinforcement and grout
- equilibrium
- ε_{mu} = 0.0035 for clay masonry, ε_{mu} = 0.0025 for concrete masonry
- plane sections remain plane
- elasto–plastic stress–strain curve for reinforcement
- tensile strength of masonry is neglected
- equivalent rectangular compressive stress block of stress $0.80 f'_m$ and depth of $0.80c$

Strength Design

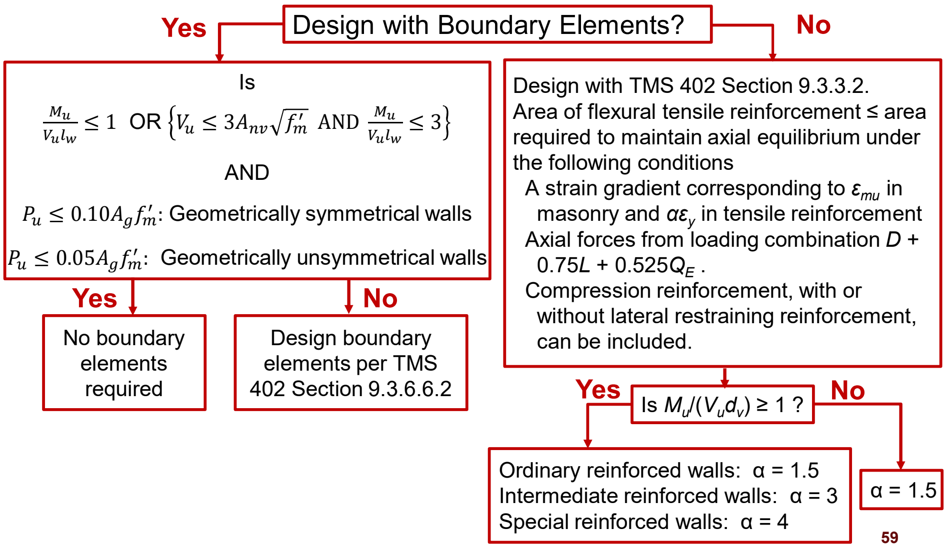
- Nominal shear strength
  - $V_n = (V_{nm} + V_{ns})\gamma_g$
  - $\gamma_g = 0.75$ for partially grouted shear walls, 1.0 otherwise
- Nominal strength limit

$M_u/(V_u d_v) \leq 0.25$	$V_n = \left(6A_{nv}\sqrt{f'_m}\right)\gamma_g$
$M_u/(V_u d_v) \geq 1.0$	$V_n = \left(4A_{nv}\sqrt{f'_m}\right)\gamma_g$
$0.25 < M_u/(V_u d_v) < 1.0$	$V_n = \left(\frac{4}{3}\left(5 - 2\frac{M_u}{V_u d_v}\right)A_{nv}\right)\gamma_g$

Strength Design

- Nominal masonry shear strength
  - $V_{nm} = \left[4 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25P_u$
  - $M_u/(V_u d_V)$ is positive and need not exceed 1.0.
- Nominal reinforcement shear strength:
  - $V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y d_v$
  - shear reinforcement bent around the edge vertical reinforcing bar with a 180° standard hook.
  - wall intersections: bent around the edge vertical bar with a 90° standard hook and extend horizontally into intersecting wall at least the development length.

Maximum Reinforcement



Design with Boundary Elements?

Yes

Is

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

AND

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

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

Yes

No boundary elements required

No

Design boundary elements per TMS 402 Section 9.3.6.6.2

No

Design with TMS 402 Section 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
- Axial forces from loading combination D + $0.75L + 0.525Q_E$.
- Compression reinforcement, with or without lateral restraining reinforcement, can be included.

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

Yes

Ordinary reinforced walls: α = 1.5
Intermediate reinforced walls: α = 3
Special reinforced walls: α = 4

No

α = 1.5

Shear Friction

Shear Span Ratio	Allowable Shear Friction
$\frac{M_u}{V_u d_v} \le 0.5$	$V_{nf} = \mu\left(A_{sp} f_y + P_u\right) \ge 0$
$0.5 < \frac{M_u}{V_u d_v} < 1.0$	Linear interpolation
$\frac{M_u}{V_u d_v} \ge 1.0$	$V_{nf} = 0.42 f'_m A_{nc}$

A_{sp} = cross-sectional area of reinforcement within the net shear area, perpendicular to and crossing the horizontal shear plane

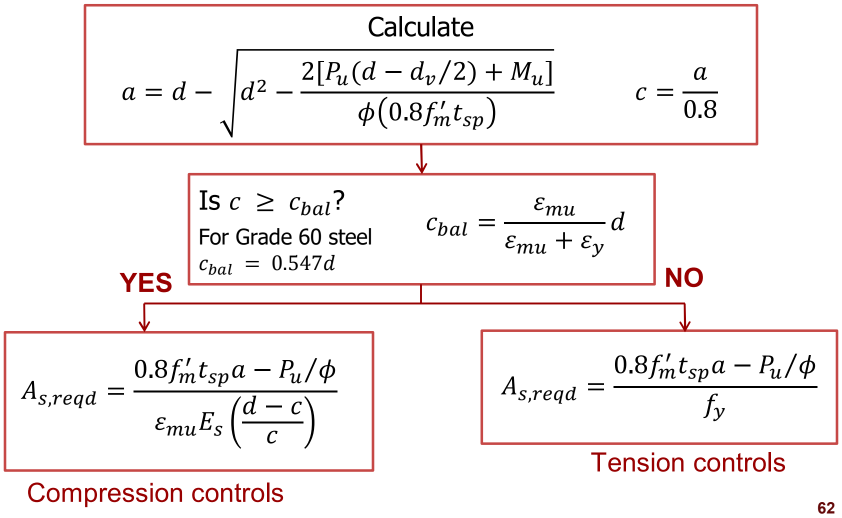
A_{nc} = net cross-sectional area between the neutral axis of bending and the fiber of maximum compressive strain calculated at the nominal moment capacity of the section

Shear Friction

- μ = 1.0 for masonry on concrete with unfinished surface, or concrete with a surface that has been intentionally roughened
  - UBC (1997) required concrete abutting structural masonry to be roughened to a full amplitude of 1/16 inch.
- μ = 0.70 for all other conditions

Special reinforced shear walls: The shear capacity provisions only apply to the nominal shear strength, V_n, and not to the nominal shear friction strength, V_{nf}, or when calculating the $M_u/(V_u d_v)$ ratio.

Design Procedure: Single Layer of Reinforcement



Calculate

$$a = d - \sqrt{d^2 - \frac{2[P_u(d - d_v/2) + M_u]}{\phi(0.8f'_m t_{sp})}} \qquad c = \frac{a}{0.8}$$

Is $c \geq c_{bal}$?

For Grade 60 steel

$c_{bal} = 0.547d$

$$c_{bal} = \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_y} d$$

YES

$$A_{s,reqd} = \frac{0.8f'_m t_{sp} a - P_u/\phi}{\varepsilon_{mu} E_s \left(\frac{d - c}{c}\right)}$$

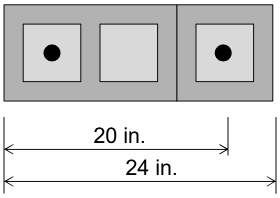
Compression controls

NO

$$A_{s,reqd} = \frac{0.8f'_m t_{sp} a - P_u/\phi}{f_y}$$

Tension controls

Example: SD-1



Given: 2 ft long, 8 ft high CMU pier; Type S masonry cement mortar; Grade 60 steel; fully grouted. P_u = 8 kips; M_u = 28 k-ft

Required: Required amount of steel

Solution: Choose/determine material properties.

f'_m = 2000 psi

f_y = 60,000 psi

$$c_{bal} = \frac{\varepsilon_{mu}}{\varepsilon_{mu} + \varepsilon_y} d = \frac{0.0025}{0.0025 + 0.00207} 20\text{in.} = 0.547(20\text{in.}) = 10.94\text{in.}$$

Example: SD-1

a, depth of stress block

$$a = d - \sqrt{d^2 - \frac{2[P_u(d - d_v/2) + M_u]}{\phi(0.8 f'_m t_{sp})}}$$

$$= 20\text{in.} - \sqrt{(20\text{in.})^2 - \frac{2[8.0\text{k}(20\text{in.} - 24\text{in.}/2) + 336\text{k} \cdot \text{in.}]}{0.9(0.8)(2\text{ksi})(7.625\text{in.})}} = 1.91\text{in.}$$

c, depth of neutral axis

$$c = \frac{a}{0.8} = \frac{1.91\text{in.}}{0.8} = 2.39\text{in.} < 10.94\text{in.} = c_{bal}$$

Tension controls

$A_{s,reqd}$, area of steel

$$A_{s,reqd} = \frac{0.8 f'_m t_{sp} a - P_u/\phi}{f_y}$$

$$= \frac{0.8(2\text{ksi})(7.625\text{in.})(1.91\text{in.}) - 8.0\text{k}/0.9}{60\text{ksi}} = 0.24\text{in.}^2$$

$A_{s,reqd}$ = 0.24in^2 Use 1 - #5 each face

Example: SD-1

Check maximum reinforcement:

Check using load combination D + 0.75L + 0.525Q$_E$.
Assume axial force is from 0.9D, so P_u for maximum reinforcement is 24k/0.9 = 26.7 kips.

Fully grouted with equal tension and compression reinforcement

$$\rho = \frac{A_s}{bd} = \frac{0.64\ {}'_{m}\left(\frac{\varepsilon_{mu}}{\varepsilon_{mu}+\alpha\varepsilon_y}\right)-\frac{P_u}{bd}}{f_y-\min\left\{\varepsilon_{mu}-\frac{d'}{d}(\varepsilon_{mu}+\alpha\varepsilon_y),\varepsilon_y\right\}E_s}$$

$$= \frac{0.64(2\text{ksi})\left(\frac{0.0025}{0.0025+1.5(0.00207)}\right)-\frac{8.0\text{k}}{7.625\text{in.}(20\text{in.})}}{60\text{ksi}-\min\left\{0.0025-\frac{4\text{in.}}{20\text{in.}}(0.0025+1.5(0.00207)),0.00207\right\}29000\text{ksi}}$$

$$= 0.0259$$

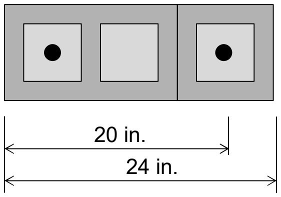
$$A_{s,max} = \rho bd = 0.029(7.625\text{in.})(20\text{in.}) = 3.95\text{in.}^2$$ **OK**

Example: SD-1

Given: 2 ft long, 8 ft high CMU pier; Type S masonry cement mortar; Grade 60 steel; fully grouted. P$_u$ = 8 kips; M$_u$ = 28 k-ft; V$_u$ = 7 kips

Required: Check shear

Solution: Material properties stay the same



Shear span ratio

$$\frac{M_u}{V_u d_v} = \frac{28.8\text{k}\cdot\text{ft}}{7.2\text{k}(2\text{ft})} = 2$$

Use $M_u/(V_u d_v)$ = 1.0

Example: SD-1

Determine design shear strength due to masonry, ϕV_{nm}

$$\phi V_{nm} = \phi\left(4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right) A_{nv}\sqrt{f'_m} + 0.25P_u$$

$$= 0.8\big(4.0 - 1.75(1.0)\big)[(7.625\text{in.})(24\text{in.})]\sqrt{2000\text{psi}} + 0.25(8000\text{lb})$$

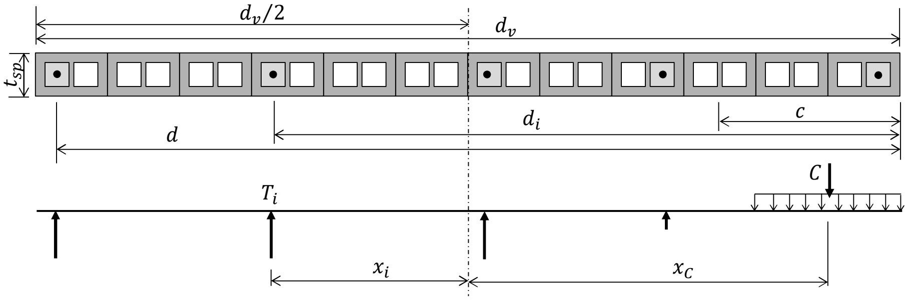
$$= 16700\text{lb}$$

$\phi V_{nm} = 16.7\text{k} \geq V_u = 7.0\text{k}$ **OK**

Design: Distributed Reinforcement

- ❑ Modify design method for single layer of reinforcement
  - Use $d = 0.9d_v$
  - Determine distributed reinforcement, $A_s^* = A_s/0.65d_v$
  - Use specified thickness, even for partial grout
- ❑ Interaction diagram to check capacity
- ❑ Spacing of intermediate reinforcing bars often controlled by out-of-plane loading

Interaction Diagram



$$P_n = C - \sum_{d_i > kd} T_i$$

$$M_n = C x_C + \sum_{d_i > kd} T_i x_i$$

Interaction Diagram

Strain and Stress	Force	Moment Arm
$\varepsilon_s = \varepsilon_{mu} \dfrac{d - c}{c}$ $\varepsilon_{si} = \varepsilon_s \dfrac{d_i - c}{d - c}$ $f_{si} = \min\{E_s \varepsilon_{si}, f_y\}$	$C = 0.8 f'_m (0.8c) t_{net}$ $T_i = A_{si} f_{si}$	$x_C = \dfrac{d_v}{2} - \dfrac{0.8c}{2}$ $x_{T_i} = d_i - \dfrac{d_v}{2}$

Example: SD-2

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+1.0E load combination.

- $M_u = (50\text{k})(10\text{ft}) = 500\text{k}\cdot\text{ft}$
- Axial load, P_u
  - Need to know weight of wall to determine P.
  - Need to know reinforcement spacing to determine wall weight
  - Based on next page, 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: SD-2

Estimate d

$$d = 0.9d_v = 0.9(192\text{in.}) = 173\text{in.}$$

a, depth of stress block

$$a = d - \sqrt{d^2 - \frac{2[P_u(d - d_v/2) + M_u]}{\phi(0.8f'_m t_{sp})}}$$

$$= 173\text{in.} - \sqrt{(173\text{in.})^2 - \frac{2[18.6\text{k}(173\text{in.} - 192\text{in.}/2) + 6000\text{k}\cdot\text{in.}]}{0.9(0.8)(2000\text{psi})(7.625\text{in.})}} = 3.96\text{in.}$$

$A_{s,reqd}$, area of steel

$$A_{s,reqd} = \frac{0.8f'_m t_{sp} a - P_u/\phi}{f_y}$$

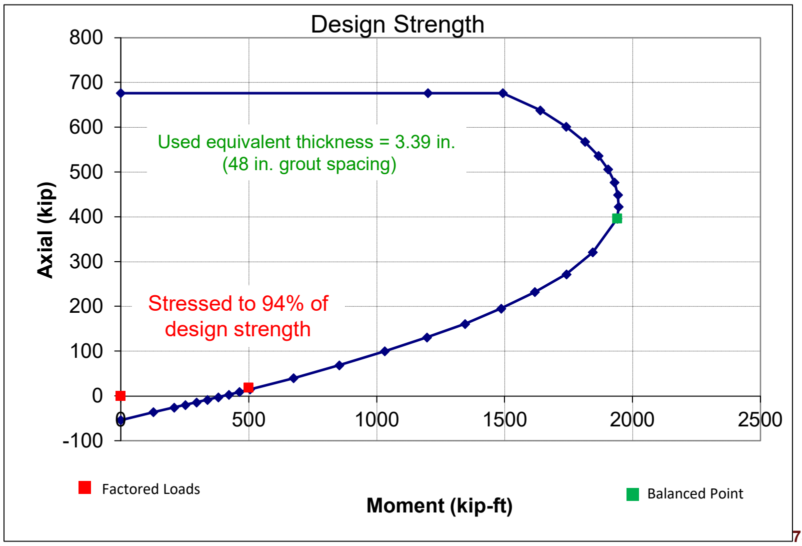
$$= \frac{0.8(2\text{ksi})(7.625\text{in.})(3.96\text{in.}) - 18.6\text{k}/0.9}{60\text{ksi}} = 0.460\text{in.}^2$$

$A^*_{s,reqd}$, dist. steel

$$A^*_{s,reqd} = \frac{A_{s,reqd}}{0.65d_v} = \frac{0.460\text{in.}^2}{0.65(192\text{in.})}\,\frac{12\text{in.}}{\text{ft}} = 0.044\ \text{in.}^2/\text{ft}$$

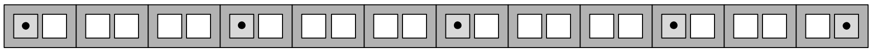
Try #4 @ 48 in. (0.050in.^{2}/ft)

Example: SD-2



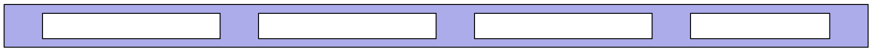
Example: SD-2

<u>Given:</u> 10 ft high x 16 ft long 8 in. CMU shear wall; Grade 60 steel, f'_m =2000 psi; #4 at 48in.; superimposed dead load of 1 kip/ft. In-plane seismic load of 50 kips.



<u>Required:</u> Design for shear

<u>Solution:</u> Net area is shown below



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

Example: SD-2

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

Max Shear:

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

$$= 0.8\left[\frac{4}{3}(5 - 2(0.625))(685\text{in.}^2)\sqrt{2000\text{psi}}\right]0.75 = 91.9\text{kip}$$

OK

Masonry Shear:

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

$$= 0.8[(4 - 1.75(0.625))(685\text{in.}^2)\sqrt{2000\text{psi}} + 0.25(18600\text{lb})]0.75$$
$$= 56.2\text{kip}$$

OK

Example: SD-2

Section 9.3.3.2 Maximum Reinforcement

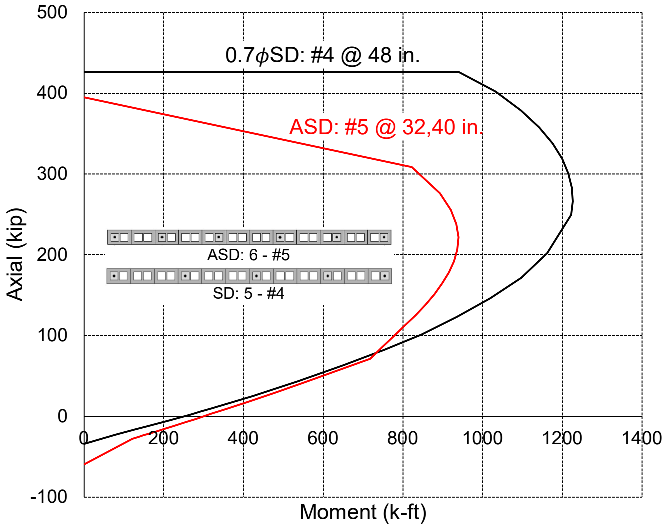
Since $M_u/(V_u d_v) < 1$, strain gradient is based on $1.5\varepsilon_y$.

Strain	c/d, CMU	c/d, Clay
$1.5\varepsilon_y$	0.446	0.530
$3\varepsilon_y$	0.287	0.360
$4\varepsilon_y$	0.232	0.297

c = 0.446(188in.) = 83.8 in.

- Calculate axial force based on c = 83.8 in.
- Include compression reinforcement
- ϕP_n = 323 kips
- Assume a live load of 1 k/ft
- $D + 0.75L + 0.525Q_E$ = (1k/ft + 0.75(1k/ft))16ft = 28 kips **OK**

Example: ASD vs. SD



Example: SD-3

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. $S_{DS} = 0.5$

Required: Design the shear wall; **special reinforced shear wall**

Solution:

- Flexural reinforcement remains the same (#4 @ 48 in.)
- Design for 2.0V_u, or 2(50 kips) = 100 kips (Section 7.3.2.6.1.2)
- Maximum V_u = 91.9 kips
- Spacing of reinforcement for special walls ≤ (1/3)height = 40 in.
- By trial and error, decrease spacing of reinforcement to 32 in.
  - A_{nv} = 767 in.^{2}
  - $V_{u,max}$ = 102.9 kip

Example: SD-3

Masonry Shear:

$$V_{nm} = \left(4 - 1.75\left(\frac{M}{Vd_v}\right)\right) A_{nv}\sqrt{f'_m} + 0.25P_u$$
$$= \left(4 - 1.75(0.625)\right)(767\text{in.}^2)\sqrt{2000\text{psi}} + 0.25(18600\text{lb})$$
$$= 99.7\text{kip}$$

Required Steel Strength:

$$\phi V_n = \phi(V_{nm} + V_{ns})\gamma_g$$
$$V_{ns,reqd} = \frac{V_u}{\phi\gamma_g} - V_{nm} = \frac{100\text{k}}{0.8(0.75)} - 99.7\text{k} = 67.0\text{k}$$

Determine spacing:
Use #5 bars

$$V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y d_v \quad \Rightarrow \quad s = \frac{0.5A_v f_y d_v}{V_{ns,reqd}}$$
$$s = \frac{0.5(0.31\text{in.}^2)(60\text{ksi})(192\text{in.})}{67.0\text{k}} = 26.7\text{in.}$$

Use #5 at 24 in. o.c.

Example: SD-3

- c = 0.446(188in.) = 83.8 in.
- Calculate axial force based on c = 83.8 in. (#4@ 32 in.)
- Since $M_u/(V_u d_v) \leq 1$, α = 1.5
- Include compression reinforcement
- ϕP_n = 364 kips
- Assume a live load of 1 k/ft
- $D + 0.75L + 0.525Q_E$ = (1k/ft + 0.75(1k/ft))16ft = 28 kips **OK**

Example: SD-3

- Prescriptive Reinforcement Requirements (7.3.2.6)
  - 0.0007 in each direction
  - 0.002 total

- Vertical: 7(0.20in^2)/[192in(7.625in)] = 0.00096
- Horizontal: 5(0.31in^2)/[120in(7.625in)] = 0.00169
- Total = 0.000960 + 0.00169 = 0.00267 **OK**

Example: SD-3

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. $S_{DS} = 0.5$

Required: Check shear friction

Solution:

- Assume concrete is unfinished ($\mu = 1.0$)
- Flexural reinforcement was 7 - #4 bars; $A_{sp} = 7(0.20\text{in.}^2) = 1.40\text{in.}^2$
- $P_u = 18.6\text{k}$
- $M_u/(V_u d_v) = 0.625$
- $A_{nv} = 767\text{in.}^2$
- $t_{eq} = 3.83\text{in.}$

Example: SD-3

$\frac{M_u}{V_u d_v} \leq 0.5$ $\quad V_{nf} = \mu\left(A_{sp} f_y + P_u\right) = 1.0\left((1.40\text{in.}^2)60\text{ksi} + 18.6\text{k}\right) = 102.6\text{k}$

$\frac{M_u}{V_u d_v} \geq 1.0$ $\quad V_{nf} = 0.42 f'_m A_{nc} = 0.42(2\text{ksi})(3.83\text{in.})(18.1\text{in.}) = 58.2\text{k}$

c = 18.1 in. from interaction diagram

Interpolate $\quad V_{nf} = 91.5\text{k} \quad \phi V_{nf} = 0.8(91.5\text{k}) = 73.2\text{k}$ **OK**

$$y_2 = \frac{(x_2 - x_1)(y_3 - y_1)}{(x_3 - x_1)} + y_1 = \frac{(0.625 - 0.5)(58.2\text{k} - 10\;\;.6\text{k})}{(1.0 - 0.5)} + 102.6\text{k} = 91.5\text{k}$$

This concludes The American Institute of Architects
Continuing Education Systems Course

The Masonry Society

Richard Bennett
rmbennett@utk.edu