S.E. Exam Review: Masonry Design

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ASCE | KNOWLEDGE & LEARNING

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NCEES Guide

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   - Masonry, 3 out of 40 questions: **flexural members**, **compression members**, **bearing walls**, **detailing**
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   - 4 - 1 hour problems, will include a masonry structure
3. Lateral Forces Exam – Saturday Breadth
   - Masonry, 3 out of 40 questions: **flexural-compression members**, slender walls, ordinary or intermediate **shear walls**, special shear walls, anchorages, attachments
4. Vertical Forces Exam – Saturday Depth
   - 4 - 1 hour problems, may include a masonry structure

Any masonry design experience? – Courses or Design

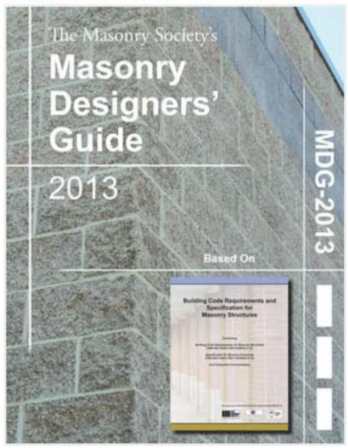
Masonry Experience – feedback from sites “via chat”

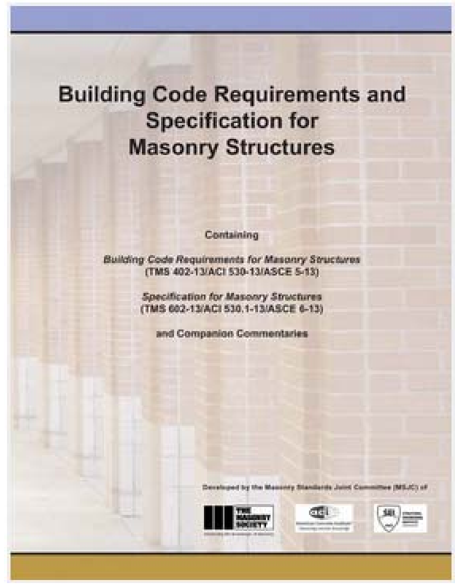
Most at site have:

A. Little or none
B. A short course and/or a little design
C. Design simple buildings/elements
D. Design masonry routinely
E. Design masonry in sleep – a masonry wiz, etc.

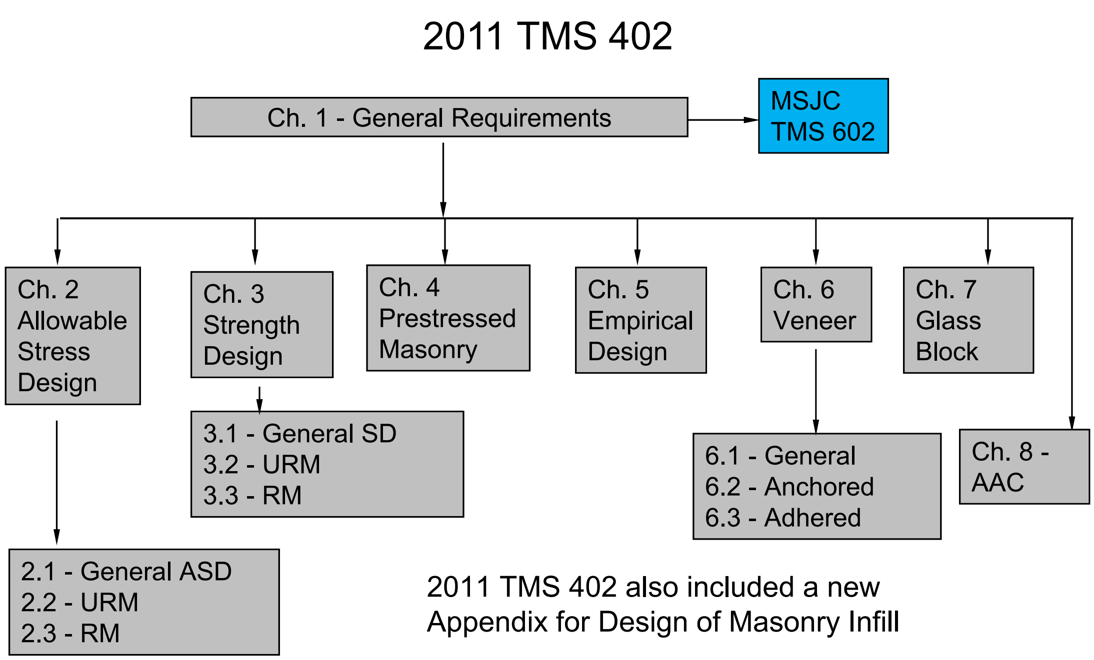
NCEES uses 2013 Code (TMS402/ACI 530/ASCE 5-13) and Specification – VERT. & LATERAL

Assume you have access to MSJC – recommend MDG 2013



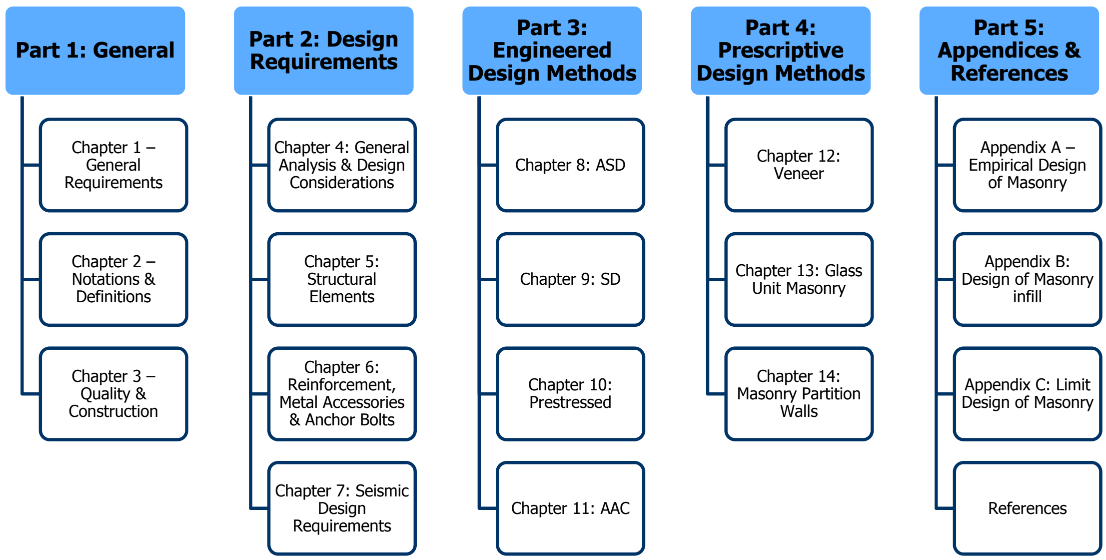


TMS 402 2011 vs TMS 402 2013 VERT. & LATERAL

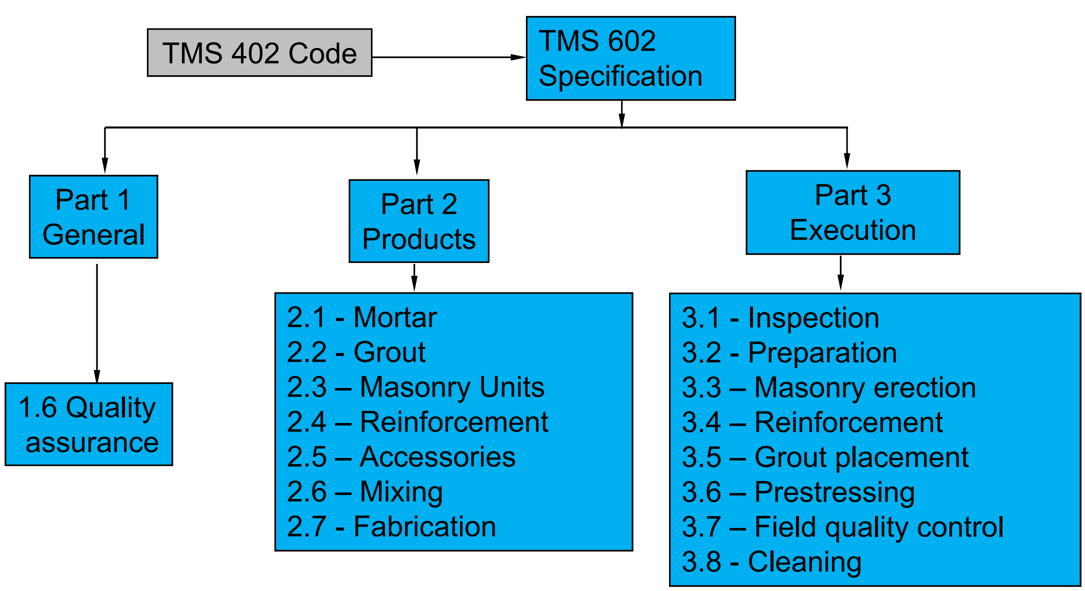


2011 TMS 402 also included a new Appendix for Design of Masonry Infill

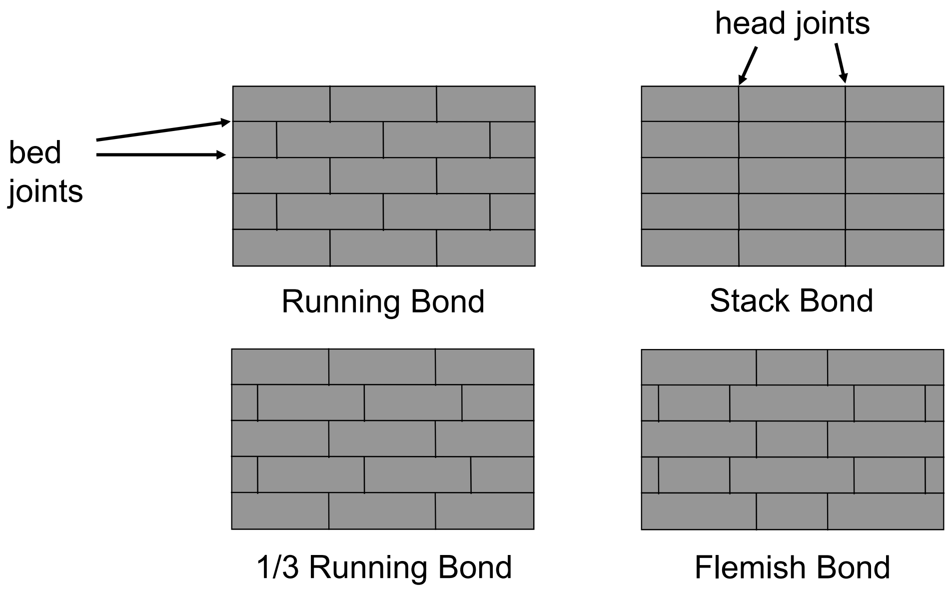
2013 TMS 402 (Reorganized) – VERT. & LATERAL



2013 TMS 602 – VERT. & LATERAL

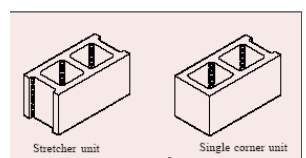


Bond Patterns VERT. & LATERAL



Concrete Masonry Units VERT. & LATERAL

- Concrete masonry units (CMU) usually hollow & 8 x 16 x (8 or 10 or 12)
  - specified by ASTM C 90
  - minimum specified compressive strength (net area) of 1900 psi (Ave). New ASTM – 2000 psi (2011)
  - net area is about 55% of gross area
  - nominal versus specified versus actual dimensions
  - Type I and Type II designations no longer exist
- Also, Concrete Brick – ASTM C 55
- Most masonry modules 8”
- See TEK NOTE 2-1A www.ncma.org (online resources)



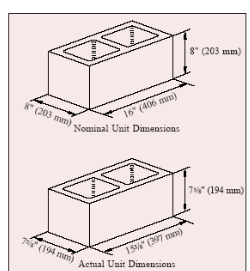
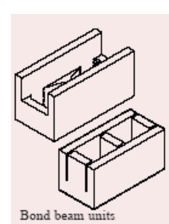
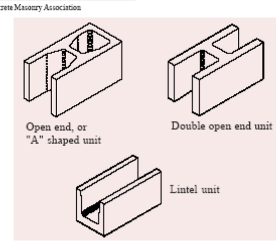


Figure 1—Nominal and Actual Unit Dimensions

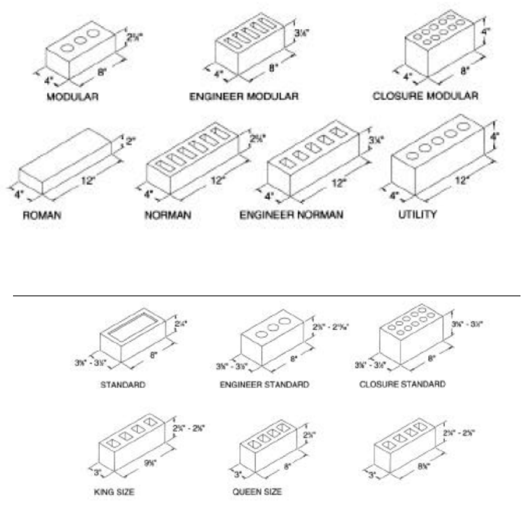
TEK 2-1A © 2002 National Concrete Masonry Association





Clay Masonry Units VERT. & LATERAL

- ASTM C 62 or C 216 or C 652 (hollow)
- Usually solid, with small core holes for manufacturing purposes
- If cores occupy ≤ 25% of net area, units can be considered 100% solid
- Bia tech Note 10 B – see www.bia.org



Non-Modular Brick Sizes

Masonry Mortar VERT. & LATERAL

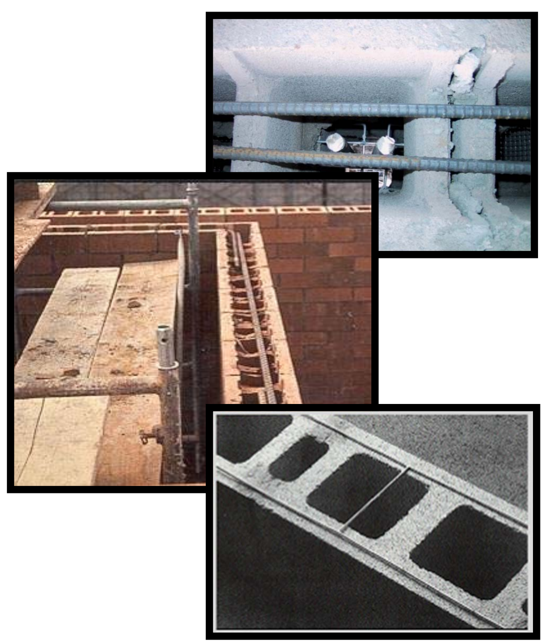
- ASTM C 270 – mortar for unit masonry
- Three systems
  - Portland-cement-lime mortar (PCL)
  - Masonry cement mortar
  - Mortar cement mortar
  - Two ways to spec – proportion and property
  - Then have 4 types

Masonry Mortar VERT. & LATERAL

- Mortar Type (MaSoN wOrK)
- Going from Type K to Type M – more Portland cement; higher compressive and tensile bond strengths, stiffer.
- **Types N and S are specified for modern masonry construction.**

Reinforcement – Code Ch. 6 VERT. & LATERAL

- Reinforcing bars in grout; joint reinforcement (ties) embedded in mortar
- Usually center placement of reinforcement
- Protection – in code
- Hooks – in code



Role of f'_m VERT. & LATERAL

- Concrete
  - Designer states assumed value of f'_c
  - Compliance is verified by compression tests on cylinders cast in the field and cured under ideal conditions
- Masonry
  - Designer states assumed value of f'_m
  - Compliance is verified by "Unit Strength Method" or by "Prism Test Method"

Verify Compliance with Specified f'_m VERT. & LATERAL

- *Unit strength method (Spec 1.4 B 2)*
  - *Compressive strengths from unit manufacturer*
  - *ASTM C 270 mortar*
  - *Grout meeting ASTM C 476 - min. f'_m - 2,000 psi*
- Prism test method (Spec 1.4 B 3)
  - Pro: can permit optimization of materials
  - Con: requires testing, qualified testing lab, and procedures in case of non-complying results

Unit Strength Method VERT. & LATERAL

Table 1 — Compressive strength of masonry based on the compressive strength of clay masonry units and type of mortar used in construction

Net area compressive strength of clay masonry, psi (MPa)	Net area compressive strength of clay masonry units, psi (MPa)	
	Type M or S mortar	Type N mortar
1,000 (6.90)	1,700 (11.72)	2,100 (14.48)
1,500 (10.34)	3,350 (23.10)	4,150 (28.61)
2,000 (13.79)	4,950 (34.13)	6,200 (42.75)
2,500 (17.24)	6,600 (45.51)	8,250 (56.88)
3,000 (20.69)	8,250 (56.88)	10,300 (71.02)
3,500 (24.13)	9,900 (68.26)	—
4,000 (27.58)	11,500 (79.29)	—

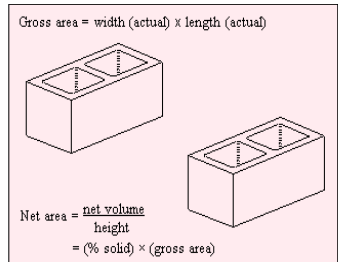


Figure 1—Gross and Net Areas

Table 2 — Compressive strength of masonry based on the compressive strength of concrete masonry units and type of mortar used in construction

Net area compressive strength of concrete masonry, psi (MPa)	Net area compressive strength of concrete masonry units, psi (MPa)	
	Type M or S mortar	Type N mortar
1,700 (11.72)	---	1,900 (13.10)
1,900 (13.10)	1,900 (13.10)	2,350 (14.82)
2,000 (13.79)	2,000 (13.79)	2,650 (18.27)
2,250 (15.51)	2,600 (17.93)	3,400 (23.44)
2,500 (17.24)	3,250 (22.41)	4,350 (28.96)
2,750 (18.96)	3,900 (26.89)	-----
3,000 (20.69)	4,500 (31.03)	-----

[1]For units of less than 4 in. (102 mm) nominal height, use 85 percent of the values listed.

Concrete masonry units (Table 2)

unit compressive strength ≥ 2,000

Type S mortar

f'_m can be taken as 2,000 psi

ASD Load Combinations – IBC 2015/ASCE 7-10 VERT. & LATERAL

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

General Structural Analysis and Design VERT. & LATERAL

- Unreinforced vs. Reinforced masonry
- Unreinforced masonry: masonry resists flexural tension, reinforcement is neglected
- Reinforced masonry: masonry in flexural tension neglected, reinforcement resists all tension

General Structural Analysis and Design VERT. & LATERAL

Design Methods

- ASD – applied stresses service loads ≤ allowed stresses
  - f ≤ F Ch 8
- Strength – Ch 9
- Factored load effects ≤ factored resistance
  - $\alpha L \leq \phi R$

Allowable Stresses (ASD) Depend On - VERT. & LATERAL

- Specified masonry compressive strength, f'_m
  - Compressive strength of masonry units
  - Mortar type
- Bond pattern
- Unit type – hollow or solid
- Extent of grouting
- Slenderness
- Type of stress – flexure, tension, compression, shear, etc.

General Analysis Considerations VERT. & LATERAL

- Load distribution and deformation – elastic analysis based on uncracked sections, except beam defl. (I_{eff} was in Commentary now in Section 5.2 for beams)
- Member stresses and actions – calculated on minimum critical sections (reinforced – cracked). Section 4.3
- Member stiffness calculated based on average sections.
- For CMU – See Tek Note 14-1B Section Properties (www.ncma.org)

Material Properties Code – 4.2 VERT. & LATERAL

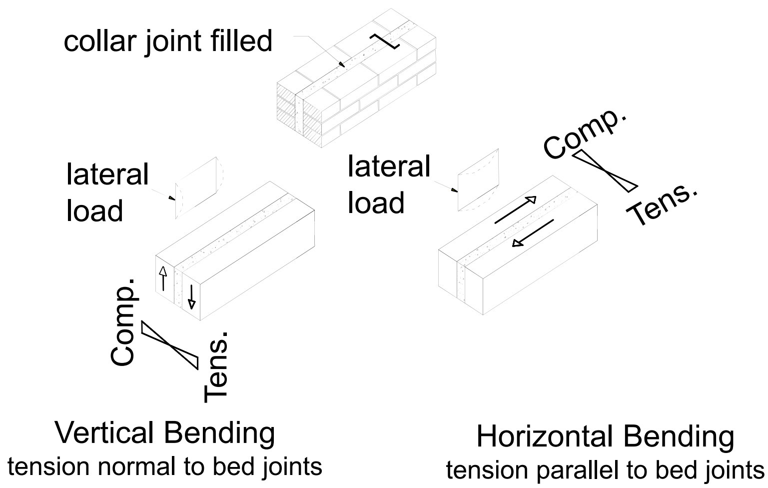
- Chord modulus of elasticity
  - 700 f'_m for clay masonry
  - 900 f'_m for concrete masonry
- Thermal expansion coefficients for clay and concrete masonry
- Moisture expansion coefficient for clay masonry
- Creep coefficients for clay and concrete masonry

Composite vs. Noncomposite Construction VERT. & LATERAL

- Masonry can have more than one wythe (thickness)
- Multiwythe walls may be designed for:
  - Composite action or noncomposite action
- Composite action requires that collar joints be:
  - Crossed by connecting headers, or filled with mortar or grout and connected by ties
- Code 5.1.4.2 and 8.1.4.2 (ASD) limits shear stresses on collar joints or headers – 5 psi for mortar, 13 psi for grout, $(\text{Header strength})^{1/2}$

Stresses – Composite Action, Code Commentary Fig. CC-5.1-6 VERT. & LATERAL

Assumed stress distribution in multiwythe composite walls

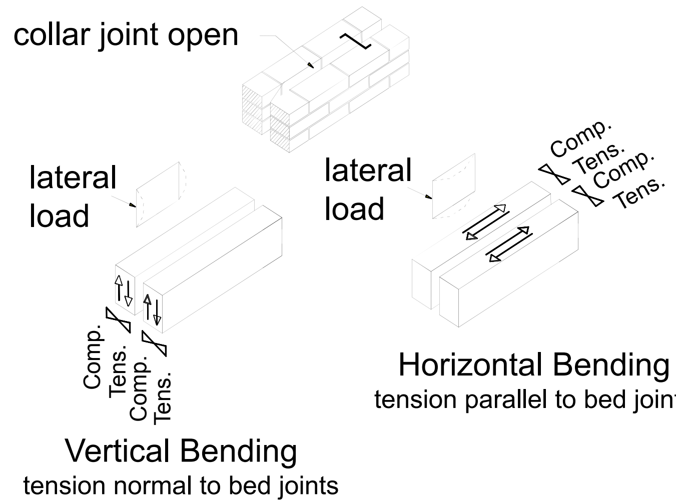


If Not a Composite Multiwythe Masonry Wall VERT. & LATERAL

- Horizontal in-plane loads and gravity loads resisted to wythe applied to only
- Weak-axis bending moments are distributed to each wythe in proportion to flexural stiffness

Stresses with Noncomposite Action, Code Commentary Fig. CC-5.1-8 VERT. & LATERAL

Assumed stress distribution in multiwythe noncomposite walls



Ch. 8.2 in MSJC-ASD URM Masonry VERT. & LATERAL

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

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

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

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

Ch. 8.2 in MSJC-ASD URM Masonry VERT. & LATERAL

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

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

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

- $P \leq P_e = \left[\frac{\pi^2 E_m I_n}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3\right]$
- Force unity equation $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$
- Shear , $f_v = \frac{VQ}{I_n b} \leq 1.5\sqrt{f'_m}$, 120 psi, or 37 psi +0.45 $\frac{N_V}{A_n}$, or 60 psi +0.45 $\frac{N_V}{A_n}$, or 15 psi

Ch. 8.3 in MSJC-ASD Reinforced Masonry VERT. & LATERAL

Assumptions

- Masonry in flexural tension is cracked
- Reinforcing steel is needed to resist tension
- Linear elastic theory
- No min. required steel area except columns
- Wire joint reinforcement can be used as flexural reinforcement
- No unity or interaction equation – use interaction curves

Allowable Stresses Steel VERT. & LATERAL

Tension

Grade 40 or 50	20,000 psi
Grade 60	32,000 psi
Wire joint reinforcement	30,000 psi

Compression

- Only reinforcement that is laterally tied (Section 5.3.1.4) can be used to resist compression
- Allowable compressive stress = allowable tension stress if tied.

Allowable Axial Compression VERT. & LATERAL

- ASD reinforced allowable compressive capacity is expressed in terms of force rather than stress
- Allowable capacity Σ(masonry + tied compressive reinforcement)
- Max. compressive stress in masonry from axial load & bending ≤ $(0.45)f'_m$
- Axial compressive stress must not exceed allowable axial stress from Code 8.2.4.1

Allowable Axial Compressive Capacity - VERT. & LATERAL

Code equations (8-21) and (8-22) slenderness reduction factors are the same as unreinforced masonry.

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

$$P_a = (0.25f'_m A_n + 0.65A_{st}F_s)\left(\frac{70r}{h}\right)^2 \text{ for } \frac{h}{r} > 99$$

Axial Compression in Bars Can be Accounted for Only If Tied As: - VERT. & LATERAL

Code 5.3.1.4:

a) Longitudinal reinforcement – enclosed by lateral ties at least ¼ in dia.

b) Vertical spacing of ties ≤ 16 d_b, 48 d_{ties}, or least cross-sectional dimension of the member.

c) Lateral ties are required to enclose bar, max. 6 in along tie between bars, have splices and included angle of <135º. Can be in mortar.

d) ½ spacing at top and bottom.

e) terminated within 3" of beams

Amount of Masonry Effective Around Each Bar Is Limited by Code VERT. & LATERAL

For running-bond masonry, or masonry with bond beams spaced no more than 48 in. center-to-center, the width of masonry in compression per bar for stress calculations less than or = to:

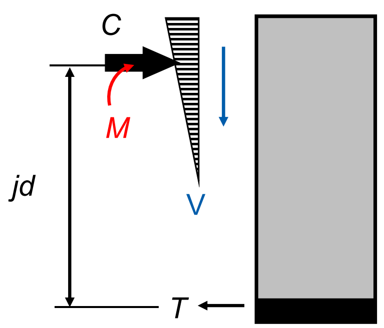
- Center-to-center bar spacing
- Six times the wall thickness (nominal)
- 72 in.

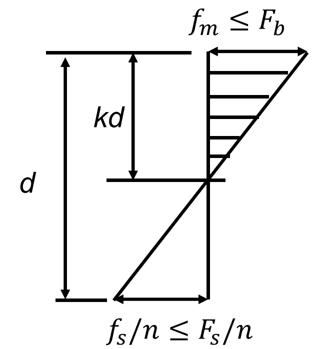
Now in Code 5.1.2

Code Section 5.2 Beams Only - VERT

- Span = clear span plus depth ≤ than distance between support centers
- Minimum bearing length = 4 in.
- Lateral support on beam compression face at a maximum spacing of 32 times the beam thickness (nominal) or $120b^2/d$ (smaller of these).
- Must meet deflection limits of Code 5.2.1.4 – gives I_{eff} and lets you ignore deflection for Span ≤ 8d

ASD Reinforced Masonry – Singly Reinforced VERT. & LATERAL





$n = E_s/E_m$ and from equil.

$M_s = A_s f_s jd$ (at the limit) $= A_s F_s jd$

$M_m = \frac{1}{2} bjkd^2 f_m$ (at limit) $= \frac{1}{2} bjkd^2 F_b$

$k = \sqrt{(\rho n)^2 + 2(\rho n)} - \rho n$

$j = 1 - k/3$

$\rho = \frac{A_s}{bd}$

Example Design Masonry for Flexure (ASD) - VERT

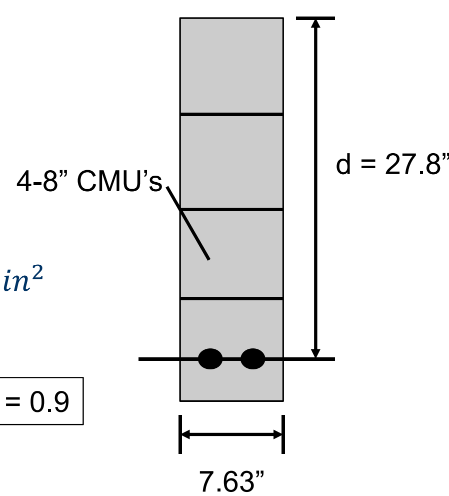
Given a lintel over a door in a 8 CMU wall:

Max moment = 493.3 kip.in V = 8 kips

Assume by tests f'_m = 2,000 psi

- Assume that the beam is sized for shear
- Try four courses h ~ 32
- $As_{req} = M/F_s jd = \frac{493{,}300}{(32{,}000 \times 0.9 \times 27.8)} = 0.62\ in^2$
- Try 2 – #5 rebar As = 0.62 in^2
- $n = E_s/E_m = \frac{29{,}000{,}000}{(900)(2{,}000)} = 16.11$

Guess j = 0.9



Lintel Design - VERT

Check design

$\rho = \frac{A_s}{bd} = \frac{0.62}{7.63(27.8)} = 0.00292$

$k = [(n\rho)^2 + 2(n\rho)]^{1/2} - n\rho = 0.2635,\ j = 1 - 0.2635/3 = 0.912$

$M_s = A_s \times F_S \times j \times d \quad = 0.62 \times 32{,}000 \quad \times 0.912 \times 27.8 = 503\ kip.in > 493.3$

$f_m = \frac{M}{0.5jkbd^2} = \frac{493.3 \times 1{,}000}{0.5(0.912)0.2635(7.63)(27.8)^2} = 692.2\text{ psi} \leq 0.45(2{,}000) = 900\text{ psi}$

or

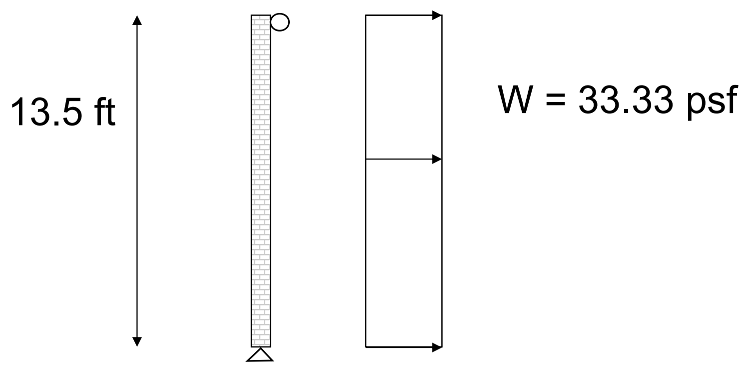
$M_m = \frac{1}{2} F_b \times b \times k \times j \times d^2$

$M_m = \frac{1}{2} 900 \times 7.63 \times 0.263 \times 0.912 \times (27.8)^2$

$M_m = 636.5\ kips \cdot in > 503$ – steel stress governs

- Problem types – could ask you to calculate moment capacity, select the number of #5 bars needed to resist load, etc.

Design Masonry Wall Flexure Out-of-Plane - LATERAL



Try a reinforced 8" CMU f'_m = 1,500 psi, Grade 60 rebar

.6D +0.6W governs at mid-height – **start with** 1 ft design width of wall

$M_{max} = 0.6 \times 33.33(13.5)^2/8$

$M_{max} = 455.6\text{ lb} \cdot \text{ft}$ per ft of wall

Design Masonry Wall Flexure Out-of-Plane - LATERAL

Per foot of wall – assume $j = 0.9$ and $d = t/2 = 7.625/2 = 3.81$"

$As_{req} = M/F_s jd = 455.6 \times 12/(32{,}000 \times 0.9 \times 3.81) = 0.050 \text{ in}^2$

Try #5 rebar at 56 OC " $A_s = 0.31 \text{ in}^2$

$A_s/\text{ft} \sim 0.066 \text{ in}^2/\text{ft}$ (based on 56/12)

Check section – effective width = 6t = 48, or s = 56 or 72

$\rho = A_s/bd = 0.31/48(3.81) = 0.001695,$

$n = 290{,}00{,}000/\left(900(1{,}500)\right) = 21.48$

Design Masonry Wall Flexure Out-of-Plane LATERAL

$k = [(n\rho)^2 + 2(n\rho)]^{1/2} - n\rho = 0.236, \ j = 1 - k/3 = 0.921$

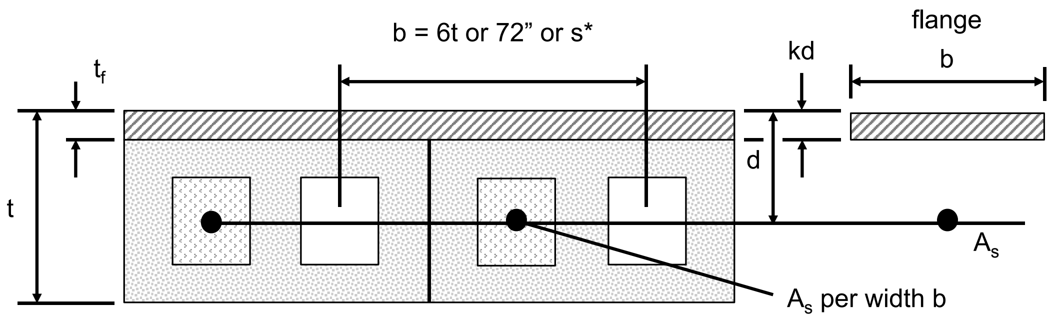
$M_s = A_s jdF_s = 0.31(0.921)(3.81)\,(32{,}000)/12 = 2{,}901 \text{ lb} \cdot \text{ft}$

$F_b = 0.45 f'_m = 0.45(1{,}500) = 675 \text{ psi}$

$M_m = 1/2\,(0.921)(0.236)48(3.81)^2\,(675)/12 = 4{,}259 \text{ lb} \cdot \text{ft}$

M_s governs since 2,901 is less than 4,259 and is greater than the applied moment $= 455.6 \times 56/12 = 2{,}126 \text{ lb} \cdot \text{ft}$

Design Masonry Wall Flexure Out-of-Plane - LATERAL



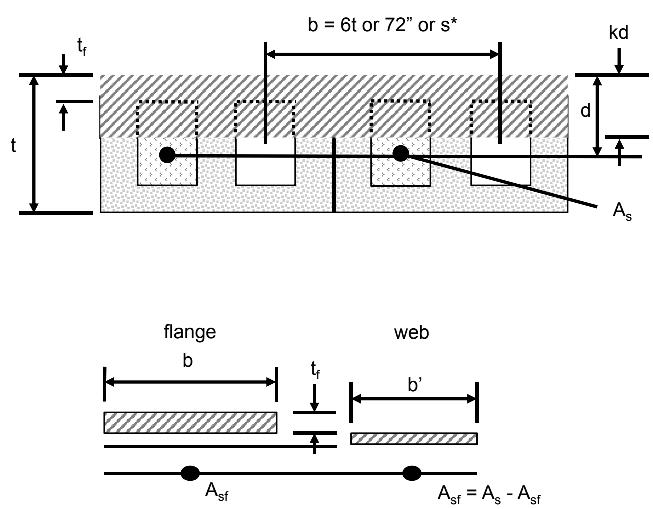
Check depth of Neutral Axis:

$kd = 0.236(3.81) = 0.90$ the face shell of a 8 CMU is 1.25"

So compression stresses are in face shell and partial grouting is possible without recalculation.

Use #5 at 56" OC

Design Masonry Wall Flexure Out-of-Plane – Partial Grouting - LATERAL



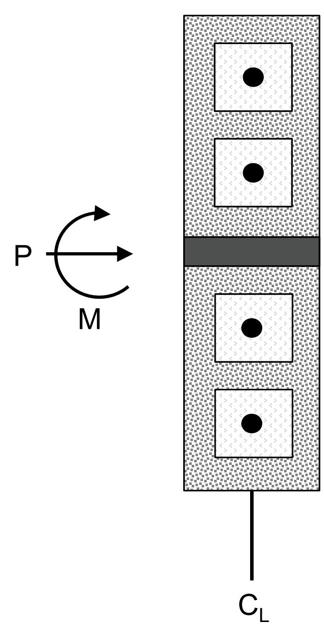
If Kd > face-shell for partial grouted section, you would need to sum the moment produced by each couple or just limit the moment to the flange stresses.

Allowable Stress Interaction Diagrams – Flexural-Compression Members - VERT. & LATERAL

- To design reinforced walls under combined loading, must construct interaction diagram
- Stress is proportional to strain; assume plane sections remain plane; vary stress (stress) gradient to maximum limits and position of neutral axis and back calculate combinations of P and M that would generate this stress distribution

Allowable Stress Interaction Diagrams VERT. & LATERAL

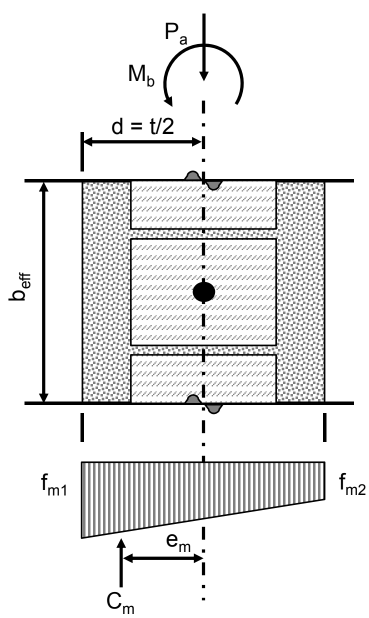
- Assume single reinforced
- Out-of-plane flexure
- Grout and masonry the same
- Solid grouted
- Steel in center



Allowable Stress Interaction Diagrams Walls – Singly Reinforced VERT. & LATERAL

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

Allowable Stress Interaction Diagrams Walls – Singly Reinforced - VERT. & LATERAL



Assume stress gradient range A:

All sections in compression

Get equivalent force-couple about center line

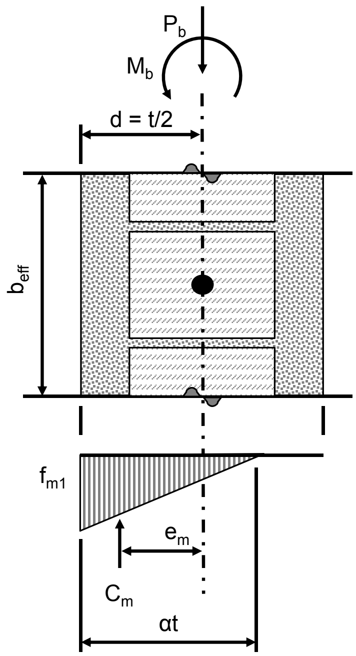
$$P_a = 0.5(f_{m1} + f_{m2})A_n$$

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

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

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

Allowable Stress Interaction Diagrams Walls – Singly Reinforced VERT. & LATERAL



Assume stress gradient range B:

Not all section in compression, but no tension in steel

Get equivalent force-couple about center line

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

$M_b = e_m \times C_m$

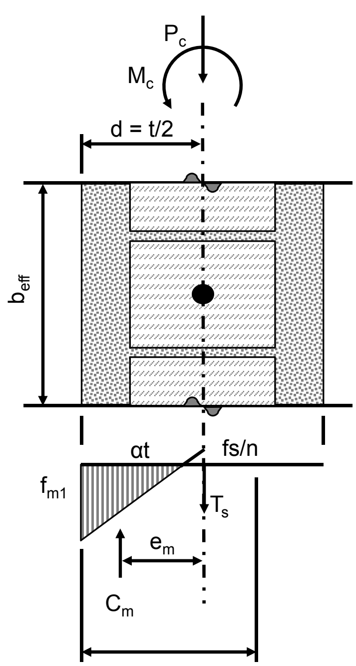
$e_m = d - \frac{\alpha t}{3} = t/2 - \frac{\alpha t}{3} = t\left(\frac{1}{2} - \alpha/3\right)$

Note that $\alpha t = kd$

This is valid until steel goes into tension

Set $f_{m1} = F_b$ at limit

Allowable Stress Interaction Diagrams Walls – Singly Reinforced VERT. & LATERAL



Assume stress gradient range C:

Section in compression, tension in steel

Get equivalent force-couple about center line

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

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

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

From similar triangles on stress diagram

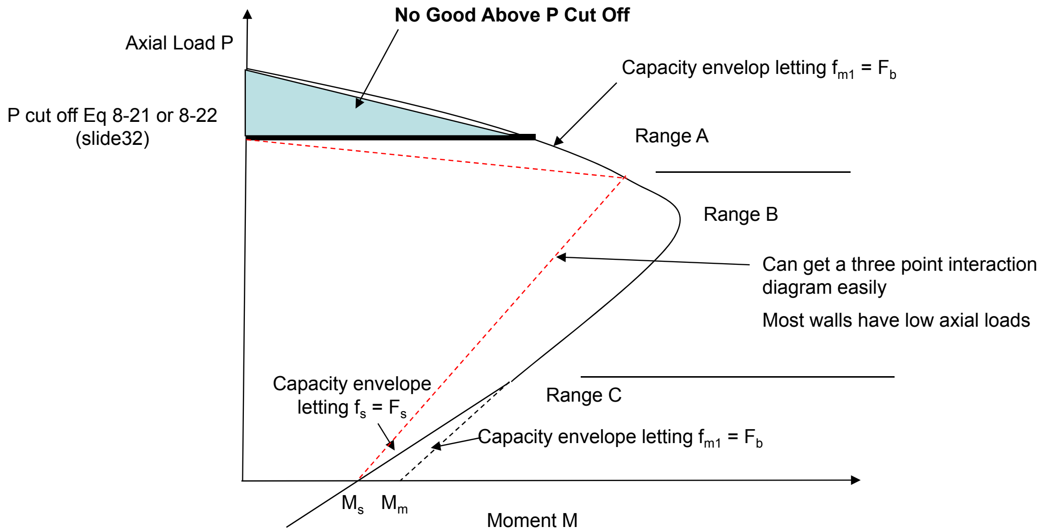
$f_s/n = ([d - \alpha t]/\alpha t) f_{m1}$

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

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

Note that $\alpha t = kd$

Allowable Stress Interaction Diagrams Walls – Singly Reinforced VERT. & LATERAL



ASD Interaction Diagram Walls – Singly Reinforced Example - VERT. & LATERAL

Construct the interaction diagram for a solidly grouted 8" CMU wall, f'_m = 1,500 psi, with height 16.67 ft and grade 60 #5 rebar at 16" OC. Also, see if the wall is adequate for the loads below. Assume pinned top and bottom of the wall.

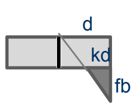
	P (kip)	M (k*in)
D + 0.75L + 0.75(0.6W) at mid-height	2.072	9.204
D + L at top	2	5.5
0.6D + 0.6W at mid-height	0.642667	13.33

ASD Interaction Diagram - VERT. & LATERAL

Spreadsheet for calculating allowable-stress M-N diagram for solid masonry wall – center rebar						
16.67 ft. wall w/ No. 5 at 16 in. (centered)						
total depth, t	7.625	in.	Wall Height, h	16.67	feet	
f'_m,	1,500	psi	Radius of Gyration, r	2.20	in.	
Em	1,350,000	psi	h/r	90.9		
Fb	675.00	psi	Reduction Factor, R	0.578		
Es	29,000,000	psi	Allowable Axial Stress, Fa	217	psi	
Fs	32,000	psi	Net Area, An	121.7	in.^{2}	
d	3.81	in.	Allowable Axial Compr, P_a	26384	lb/ft	(MSJC 8.3.4.2.1)
$K_{balanced}$	0.311828			Note: axial force is per foot of wall		
tensile reinf., A_s/b_{eff}	0.31	#5 @ 16 centered				
width, beff	16	in.				

ASD Interaction Diagram - VERT. & LATERAL

		k	**Kd (αt)**	**fb (psi)**	**Cmas (lb)**	**fs (psi)**	**Axial Force (lb)/ft**	**Moment (lb-in)/ft**	**Axial Force w/ force limit/ft**
RANGE C	Points controlled by steel	0.01	0.04	15	5	-32,000	-7,437	-6	-7,437
		0.05	0.19	78	119	-32,000	-7,350	317	-7,350
		0.1	0.38	166	504	-32,000	-7,062	1,376	-7,062
		0.15	0.57	263	1,202	-32,000	-6,539	3,246	-6,539
		0.24	0.91	470	3,441	-32,000	-4,859	9,034	-4,859
		0.22	0.84	420	2,817	-32,000	-5,327	7,447	-5,327
		0.24	0.91	470	3,441	-32,000	-4,859	9,034	-4,859
		0.3	1.14	638	5,838	-32,000	-3,062	15,006	-3,062
	K balanced	0.311828	1.19	675	6,416	-32,000	-2,628	16,420	-2,628
		0.4	1.52	675	8,230	-21,750	1,115	20,383	1,115
		0.5	1.91	675	10,287	-14,500	4,344	24,507	4,344
		0.6	2.29	675	12,344	-9,667	7,011	28,237	7,011
		0.7	2.67	675	14,402	-6,214	9,357	31,574	9,357
	Points controlled by masonry	0.8	3.05	675	16,459	-3,625	11,502	34,519	11,502
		0.9	3.43	675	18,517	-1,611	13,513	37,072	13,513
		1.1	4.19	675	22,631	0	16,974	41,000	16,974
		1.3	4.95	675	26,746	0	20,060	43,359	20,060
		1.5	5.72	675	30,861	0	23,146	44,151	23,146
		1.7	6.48	675	34,976	0	26,232	43,374	26,232
RANGE B		2	7.62	675	41,148	0	30,861	39,271	30,861
RANGE A	Pure compression			675	82,350	0	82,350	0	82,350
	Axial Force Limits						26,384	0	26,384
							26,384	44,151	26,384

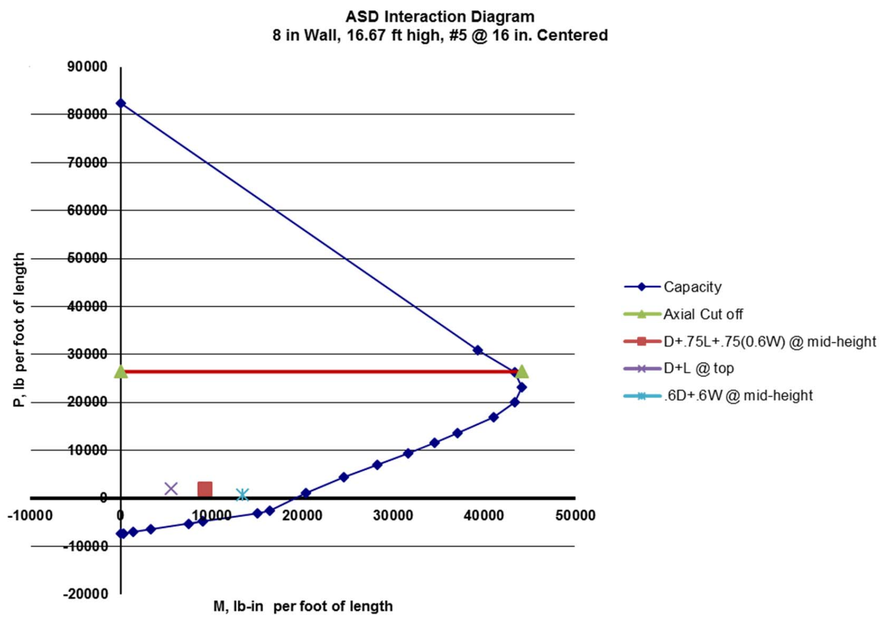


$f_s/n\,[\alpha t/([d-\alpha t])] = f_b$

$32{,}000/21.48\,[0.1(3.81/[3.81-0.1(3.81)])]$

Note: moment equation not valid after k > 2

ASD Interaction Diagram Walls – Singly Reinforced Example - VERT. & LATERAL



What would happen to previous problem if I increased the steel size? - VERT. & LATERAL

Possible Answers:

A. Nothing

B. The entire capacity curve would shift up and to the right.

C. The moment capacity governed by steel stress would increase, but this would not increase the wall capacity.

D. The lower section of the curve would shift to the right (increase M).

Possible Wall Breadth Exam Questions - VERT. & LATERAL

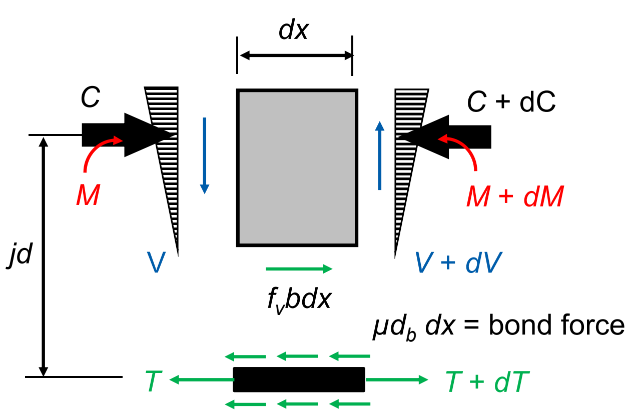
a) Given a non load bearing wall with out-of-plane loading (wall size and f'_m). Size rebar placed in center of wall. **Assume steel governs, j = 0.9, and set $M_s = M_{max.Applied}$. Find A_s. Check M_m. Iterate if needed.**

b) Given a wall configuration – size of units, rebar location and size, etc. Find max moment capacity. **Get smaller of M_m or M_s.**

c) Given a wall configuration – size of units, rebar location and size, etc. Find axial load capacity. **Eq. 8-21 or 8-22 (slide 32)**

ASD – Reinforced Masonry – Shear - VERT. & LATERAL

No shear reinforcing masonry resists all shear.



$f_v = \frac{V}{bjd} \approx \frac{V}{bd}$ = applied shear stress

F_v = varies with type of element

Reinforced Masonry Shear Stresses - VERT. & LATERAL

- Shear stress is computed as:

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

- Allowable shear stresses

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

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

Shear Stress Cutoffs - VERT. & LATERAL

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

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

  - $M/Vd_v \geq 1$

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

  - Can linear interpolate between limits

$$F_v = \left(\frac{2}{3}\left(5 - 2\frac{M}{Vd_v}\right)\right)\gamma_g$$

Shear Stresses - VERT. & LATERAL

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

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

  - All other masonry

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

M/Vd_v is positive and need not exceed 1.0.

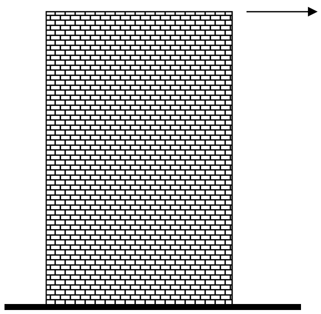
If Shear Reinforcement Present VERT. & LATERAL

- If allowable shear stress in the masonry is exceeded, then:
  - Design shear reinforcement using Equation 8-30 and add F_{vs} to F_{vm}

$$F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} S}\right) \qquad (8\text{-}30)$$

  - 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.

Look at Shear Wall Design - LATERAL



To check wall segments under in-plane loads, must first:

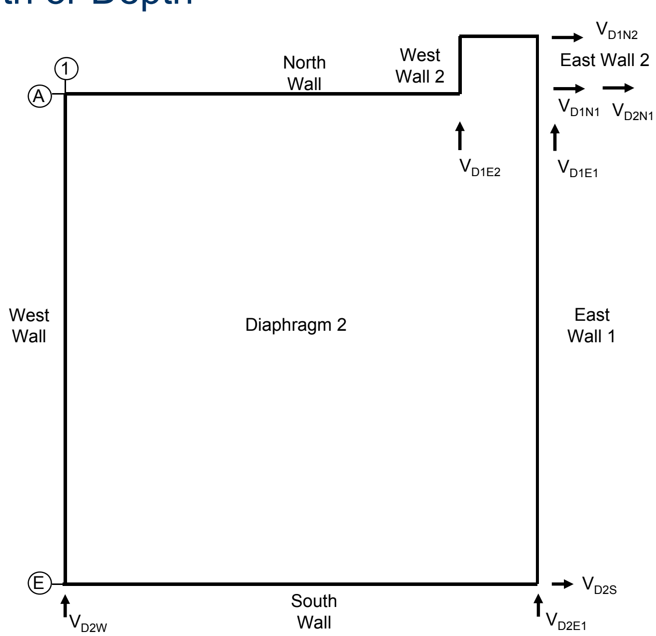
- Distribute load to shear wall lines – either by trib. width or rigid diaphragm analysis.
- Distribute line load to each segment w.r.t. relative rigidity.

Shear Wall in a Single Story Building – Shear Wall Example 1 - LATERAL

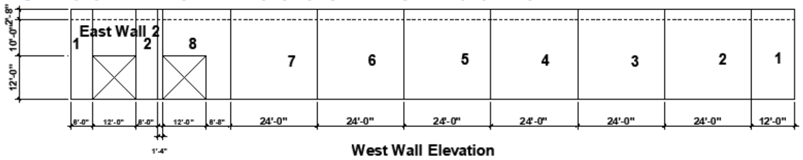
Lateral Loads Breadth or Depth

Plan of typical big box single story flexible diaphragm

See MDG for load determination and distribution to shear wall lines – Flex Diaphragm – SDC-D



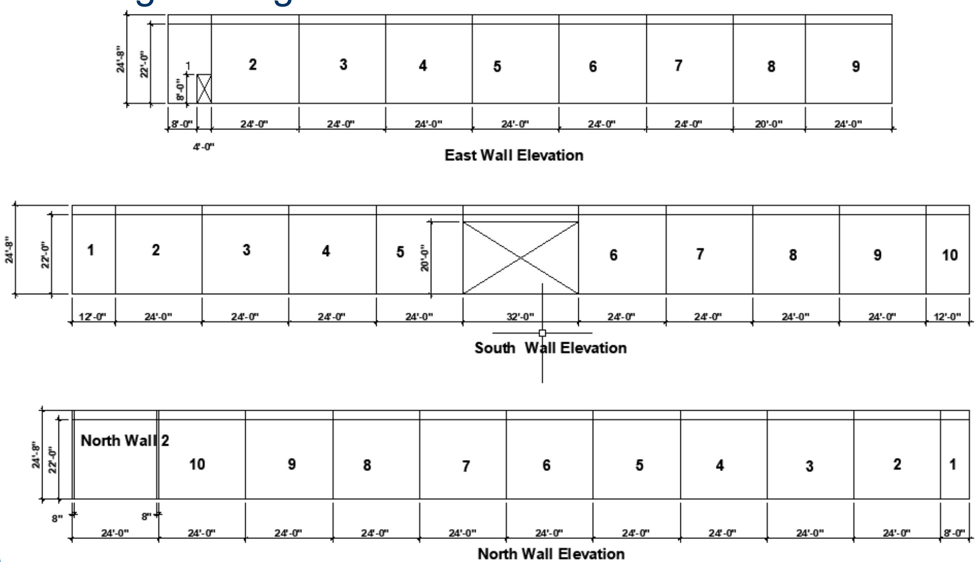
Shear Wall Loads Distribution - LATERAL



160.8 kips

Diaphragm shear due to seismic

Segments get load w.r.t. relative k



Shear Wall Loads Distribution - LATERAL

Segments get load w.r.t. relative k.

- For cantilevered shear wall segments

$$k_c = E_m t \left[4\left(\frac{h'}{l_w}\right)^3 + 3\left(\frac{h'}{l_w}\right) \right]^{-1}$$

- For fixed-fixed shear wall segments

$$k_c = E_m t \left[\left(\frac{h'}{l_w}\right)^3 + 3\left(\frac{h'}{l_w}\right) \right]^{-1}$$

Shear Wall Load Distribution - LATERAL

Table 18.1-2 DPC Box Building Shear Load on Wall Segments on the West Wall					
DPC Box West Wall (Grid 1)				Vd = 160.8 kips	
Segment	H	L	Ri	Vi from Diaphragm (lb)	Vi wt (lb)
1	22	12	0.332	4.99	4.05
2	22	24	1.715	25.80	8.1
3	22	24	1.715	25.80	8.1
4	22	24	1.715	25.80	8.1
5	22	24	1.715	25.80	8.1
6	22	24	1.715	25.80	8.1
7	22	24	1.715	25.80	8.1
8	22	6.67	0.065	0.98	2.25
		Sum	**10.687**	**160.800**	

Shear Wall Load Distribution - LATERAL

Table 18.1-2 DPC Box Building Shear Load on Wall Segments on the West Wall					
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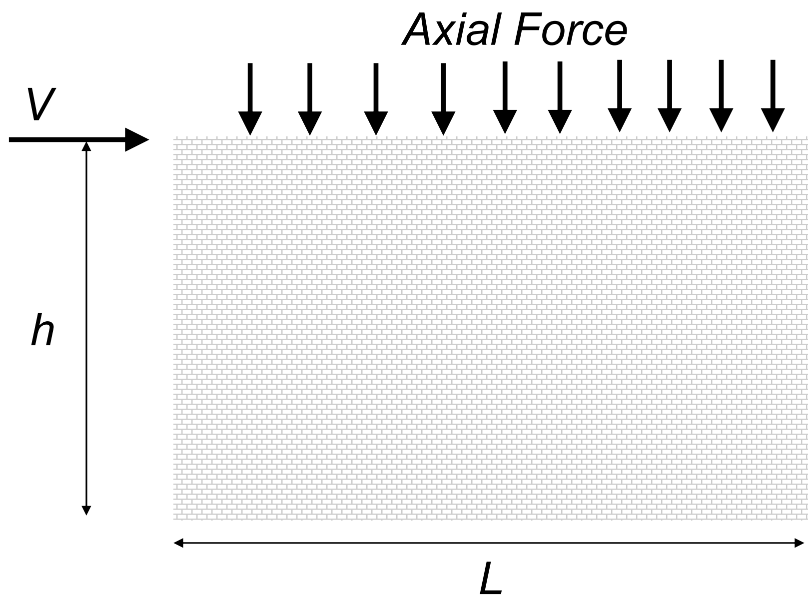
5	22	24	1.715	25.80	8.1
6	22	24	1.715	25.80	8.1
7	22	24	1.715	25.80	8.1
8	22	6.67	0.065	0.98	2.25
		Sum	**10.687**	**160.800**	

$$R_i = 10 \times \left[4\left(\frac{22}{12}\right)^3 + 3\left(\frac{22}{12}\right)\right]^{-1}$$

$$V_i = 160.8 \times \left[\left(\frac{1.715}{10.687}\right)\right]$$

Segment 2 designed in later example

Design of Reinforced Masonry (ASD) in Plane Loading (Shear Walls) - LATERAL



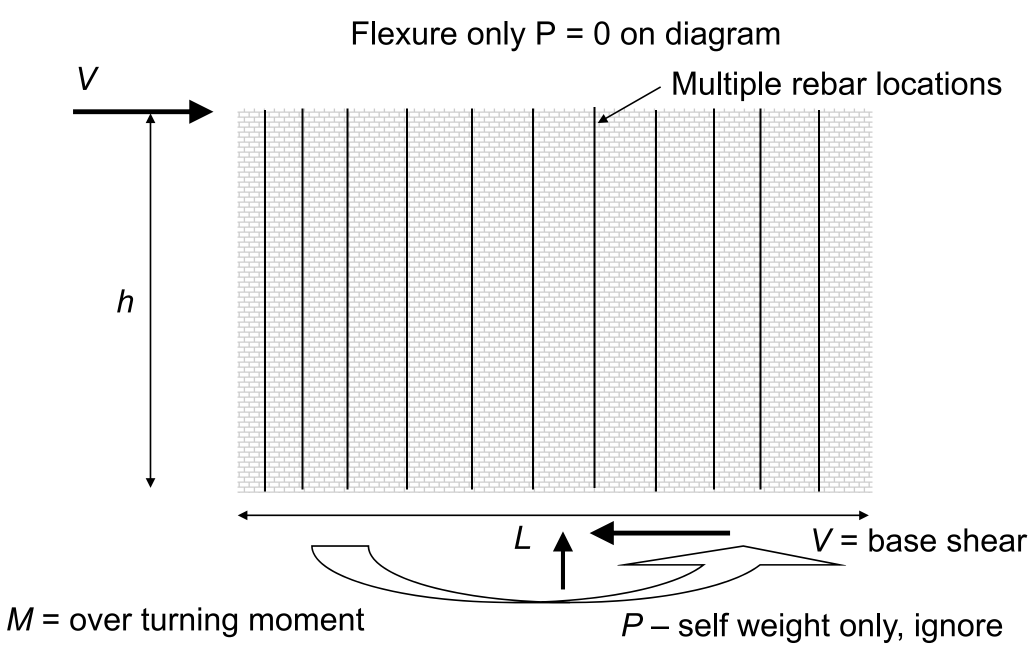
ASD Design of Reinforced Masonry – In Plane Loading (Shear Walls) - LATERAL

- Still use interaction diagrams
- Axial load is still dealt with as out of plane (M = 0)
- In plane load produces moment and thus moment capacity is dealt with slightly differently

P-M Diagrams ASD-In Plane - LATERAL

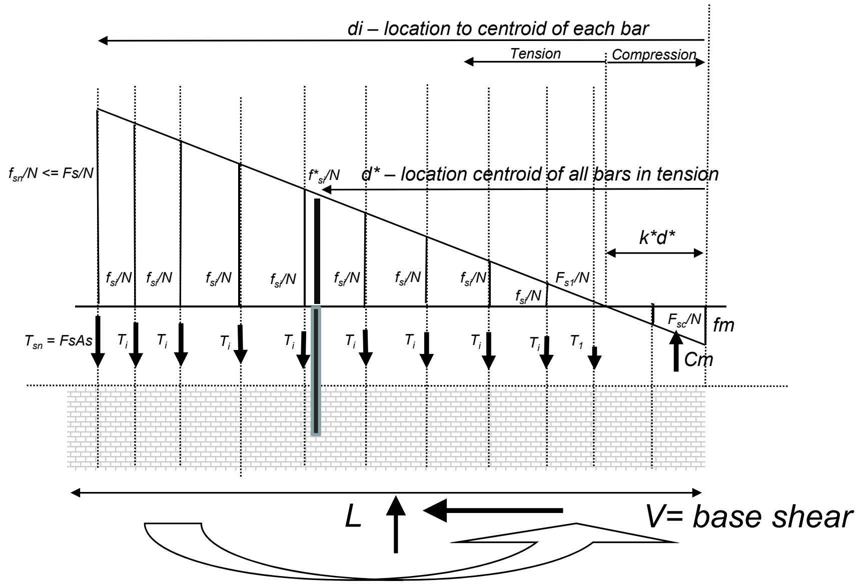
- Initially assume $f_m = F_b$ and neutral axis
- Then same as out-of-plane, but area and S are based on length = d and t = b. Use OOP equations in range A and B.
- Adjust αL as before until rebars start to go into tension. Note that $\alpha L = kd$
- Determine f_{si} from similar triangles & get T_i
- Check extreme $f_{si} = \frac{f_s}{n} \leq F_s$ and $f_m \leq F_b$
- $C_m = \alpha L \times b \times \frac{1}{2} F_b$ (or f_m when $f_{sn} = F_s$)
- M capacity $(\sum about\ center) = \sum\left(T_i \times (d_i - L/2) + C_m \times (L/2 - \alpha L/3)\right)$

Reinforced Masonry Shear Walls – ASD - LATERAL



Reinforced Masonry Shear Walls – ASD - LATERAL

(P = 0) Can use the singly reinforced equations



Moment Only ASD in Plane - LATERAL

- To locate neutral axis, guess how many bars on tension side – A_s*
- Find d* (centroid of tension bars) and $\rho^* = A_s/bd^*$
- Get $k^* = \left((\rho^* n)^2 + 2\rho^* n\right)^{1/2} - n\rho^*$
- Unless tied, ignore compression in steel.

Moment Only ASD in Plane LATERAL

- Check k*d* to ensure assume tension bars correct – iterate if not
- Determine f_{si} from similar triangles and then $T_i = (f_{si} \times A_i)$
- M capacity $(\sum \text{about } C) = \sum\left(T_i \times (d_i - k^* d^*/3)\right)$

Shear Wall Example 2 - LATERAL

Geometry

Typical wall element:

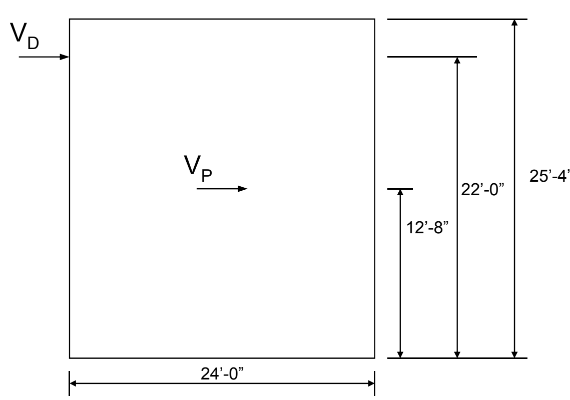
25 ft – 4 in. total height

3 ft – 4 in. parapet

24 ft length between control joints

8 in. CMU grouted solid: 80 psf dead

$f'_m = 1{,}500$ psi



Shear Wall Example 2 - LATERAL

West wall seismic load condition

$V_{\text{diaphragm}} = 25{,}800\ \text{lb}$ acting 22 ft above foundation

$V_{\text{pier}} = 8{,}100\ \text{lb}$ acting 12.7 ft above foundation

8 in. CMU grouted solid (maximum possible dead load)

$P_{\text{base}} = 80\ \ \text{lb/ft}^2 \times 253\ \text{ft} \times 24\ \text{ft} = 48{,}600\ \text{lb}$

Vertical seismic: $V_{\text{pier}} = 0.2S_{DS}D = (-0.2)(1.11)(48{,}600) = -10.800\ \text{lb}$

ASD Load Combination: $0.6D + 0.7E$

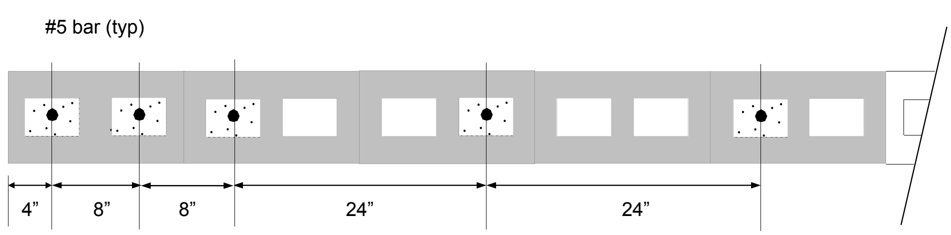
$P = 0.6 \times 48{,}600 + 0.7 \times -0.2(1.11)(48{,}600) = 21{,}600\ \text{lb}$

$M = 0.6 \times 0 + 0.7 \times (25{,}800\ \text{lb} \times 22\ \text{ft} + 8{,}100\ \text{lb} \times 12.7\ \text{ft})$

$= 469{,}000\ \text{lb} \cdot \text{ft} = 5{,}630{,}000\ \text{lb in.}$

$V = 0.6 \times 0 + 0.7 \times (25{,}800\ \text{lb} + 8{,}100\ \text{lb}) = 23{,}700\ \text{lb}$

Shear Wall Example 2 - LATERAL



Assume the rebar in the wall are as shown

- Axial load is negligible – ignore
- To simplify, assume that only three end bars are effective (only lap these to foundation)

Shear Wall Example 2 - LATERAL

For the 24 ft long wall panel between control joints subjected to in-plane loading, the flexural depth, *d**, is the wall length less the distance to the centroid of the vertical steel at the ends of the wall.

$$d^* = \ell - 12 \text{ in} = (24 \text{ ft} \times 12 \text{ in/ft}) - 12 \text{ in} = 276 \text{ in}$$

We are using three #5 bars, but if needed, an estimate of A_s can be determined by assuming $j = 0.9$ and applied moment, M.

$$A_{sreq} = \frac{M}{F_s j d^*} = \frac{5{,}630{,}000 \text{ lb−in}}{32{,}000 \text{ psi} \times 0.9 \times 276 \text{ in}} = 0.71 \text{ in}^2$$

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{900(1{,}500)} = 21.48 = 21.5$$

Shear Wall Example 2 - LATERAL

Try three No. 5 bars, $A_s = 3 \times 0.31\, in^2 = 0.93\, in^2$.

$\rho^* = \frac{A_s}{bd^*} = \frac{0.93 \text{ in}^2}{7.63 \text{ in} \times 276 \text{ in}} = 0.000442$

Calculate j and k:

$n\rho^* = 21.5 \times 0.000442$

$k^* = \sqrt{2n\rho^* + (n\rho^*)^2} - n\rho^* = \sqrt{2 \times 0.00950 + (0.00950)^2} - 0.00950 = 0.129$

$j = 1 - \frac{k}{3} = 1 - \frac{0.129}{3} = 0.957$ and $k^* d^* = 35.6$ in. Don't need to check since other bars not lapped

You need to get the stress at the centroid based on the extreme bar $f_s = F_s$

$\frac{276 - k*d*}{284 - k*d*} \times 32{,}000 = 30{,}970$ psi. Should get third bar stress then $\sum$ moments but $M \approx$

$M \approx 0.93 \text{ in}^2 \times 30{,}970 \text{ lb/in}^2 \times 0.957 \times 276 \text{ in} = 7{,}608{,}000 \text{ lb} \cdot \text{in} > 5{,}630{,}000 \text{ lb} \cdot \text{in}$ OK

Check masonry compression stresses

$F_b = 0.45 f'_m = 0.45 \times 1{,}500 \text{ psi} = 675 \text{ psi}$

$f_b = \frac{M}{0.5 j k^* b d^{*2}} = \frac{5{,}630{,}000}{0.5(0.957)0.129(7.63)(276)^2} = 157 \text{ psi} \leq 675 \text{ psi}$

Shear Wall Example 2 - LATERAL

Check shear stress.

Assume no shear reinforcing, and thus:

$$F_v = F_{vm} + F_{vs} = F_{vm}$$

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

$$= \left(\frac{1}{2}\right)\left[4 - 1.75\left(\frac{5{,}630{,}000}{23{,}700\times(276)}\right)\right]\sqrt{1{,}500} + 0.25\frac{0}{A_n}$$

$$= 48.3 \text{ psi}$$

$$F_v = \gamma_g F_{vm} = (0.75)48.3 = 36.2 \le (\text{conservatively}) = 2\sqrt{f'_m}$$

$$= 2\sqrt{1{,}500} = 77.5 \text{ psi OK}$$

Shear Wall Example 2 - LATERAL

Check shear stress.

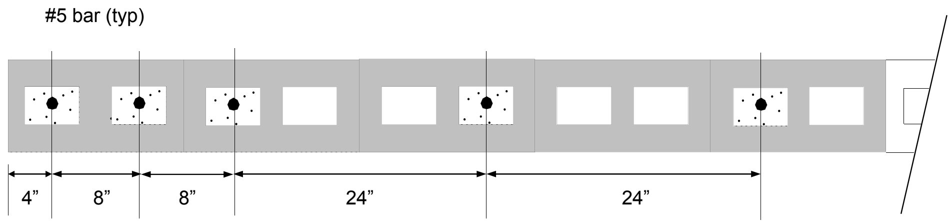
Conservatively assume just face shell bedded areas resist shear.

$$f_v = \frac{V}{A_{nv}} = \frac{23{,}700 \text{ lb}}{280(1.25)2} = 33.9 \text{ psi} < 36.2 \text{ psi OK}$$

Shear Wall Example 2 - LATERAL

So, the final design:

Can use the #5 at the ends of the wall, ignoring any bars that will likely be there for out-of-plane loading.



Possible Breadth Exam Problems - LATERAL

a) Given a diaphragm shear line load, determine the critical shear and overturning moment on a shear wall segment. SW Ex1.

b) Given a shear wall segment size and rebar config., find max. diaphragm shear at top of wall. SW Ex2 – just back calculate V after setting applied stresses = allowable stress values. Look at both shear and flexure, take lowest resulting V.

c) Given a SW segment loading, wall size, and rebar location, select size of bars needed. SW Ex 2. Flexure only – assume M_s governs. Find A_s, check M_m.

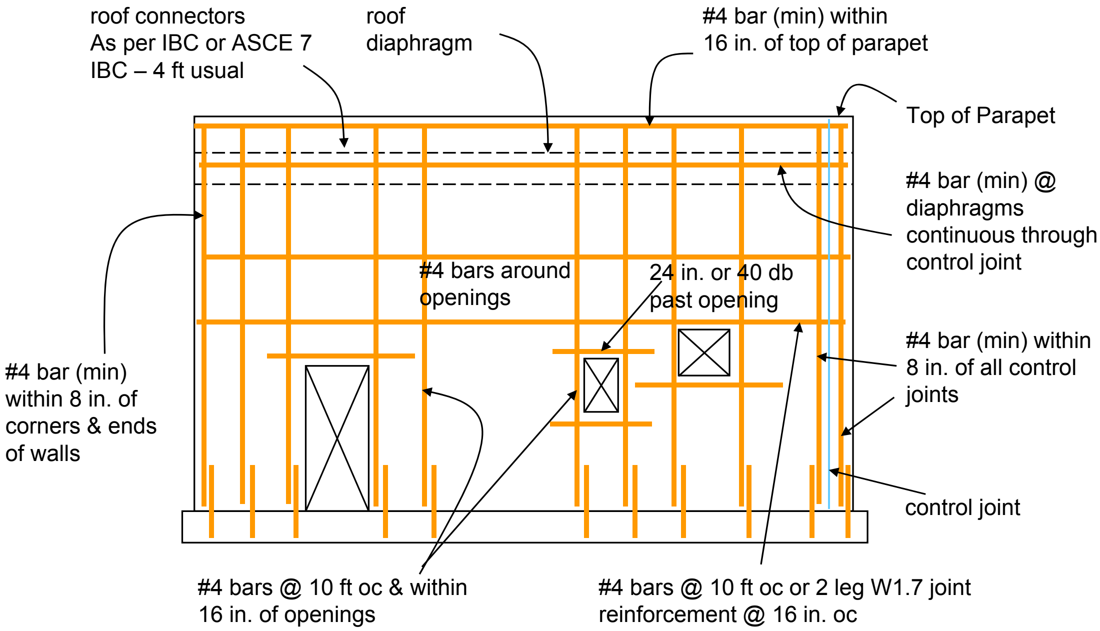
Seismic Detailing Code – Ch. 7 - LATERAL

- Define Seismic Design Category ASCE 7
- SDC determines
  - Required types of shear walls
  - Prescriptive reinforcement for other masonry elements (non-participating walls must be isolated)
  - Type of design allowed for lat. force resisting system – note, for special shear walls, $V_{\text{capacity}} \geq 1.5 \times V_{\text{applied}}$.

Minimum Reinf., SW Types, etc. – Cumulative - LATERAL

SW Type	Minimum Reinforcement	SDC
Empirically Designed	None – drift limits and connection force	A
Ordinary Plain	None – same as A	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 ρ = 0.002 – no stack bond	any

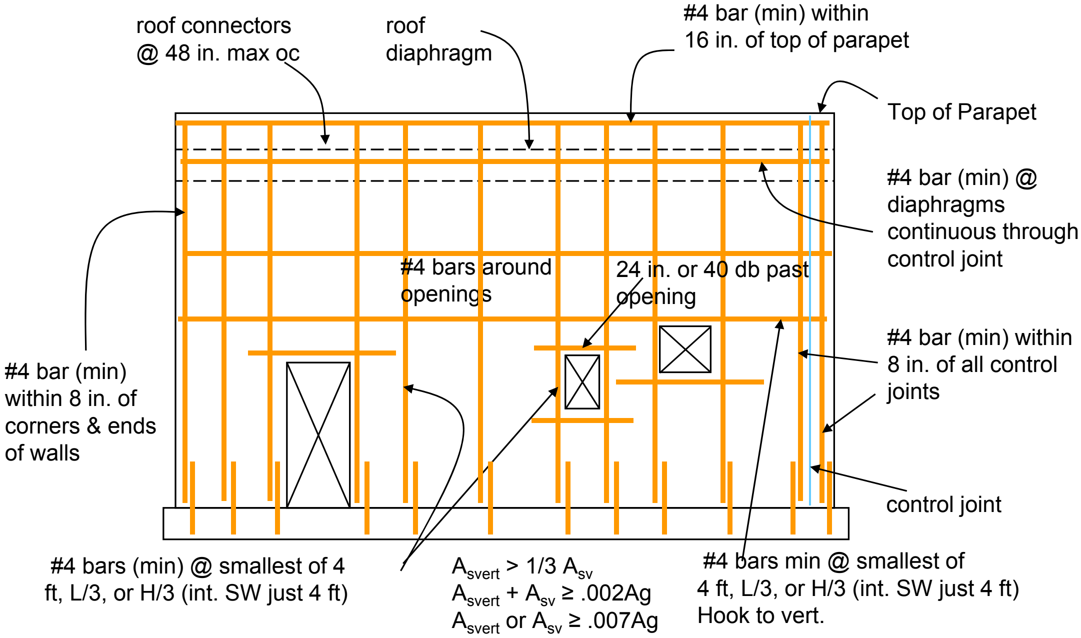
Minimum reinforcement for detailed plain shear walls and SDC C - LATERAL



MSJC 7.4 - LATERAL

- Seismic Design Category D
  - Masonry – part of lateral force-resisting system must be reinforced so that $\rho_v + \rho_h \geq 0.002$, and ρ_v and $\rho_h \geq 0.0007$
  - Type N mortar & masonry cement mortars are prohibited in the lateral force-resisting system, except for fully grouted.
  - Shear walls must meet minimum prescriptive requirements for reinforcement and connections (special reinforced)
  - Other walls must meet minimum prescriptive requirements for horizontal and vertical reinforcement

Minimum Reinforcement for Special Reinforced Shear Walls – Running Bond - LATERAL



Possible Breadth Exam Problems - LATERAL

a) Select type of shear wall for a given SDC

b) Define prescriptive detailing requirements needed for a specific shear wall type and SDC.

Conclusion

Thank you for your attention!

Any questions?