S.E. Exam Review: Masonry Design

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   - Code documents, standards, terms, units, mortar, loads, analysis methods, basic behavior, URM

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3. Reinforced Masonry Walls Combined Axial and Flexure – Analysis and Design (Slides 45 – 57)

4. Shear, Shear Wall Analysis and Design, Seismic Details (Slides 58 – 91)
NCEES Guide

1. Vertical Forces Exam – Friday Breadth
   - Masonry, 3 out of 40 questions: flexural members, compression members, bearing walls, detailing

2. Vertical Forces Exam – Friday Depth
   - 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

Ch. 1 - General Requirements

Ch. 2 - Allowable Stress Design

Ch. 3 - Strength Design

Ch. 4 - Prestressed Masonry

Ch. 5 - Empirical Design

Ch. 6 - Veneer

Ch. 7 - Glass Block

Ch. 8 - AAC

MSJC TMS 602

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

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
Concrete masonry units (Table 2)

<table>
<thead>
<tr>
<th>Net area compressive strength of clay masonry units, psi (MPa)</th>
<th>Net area compressive strength of clay masonry units, psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000 (6.90)</td>
<td>1,700 (11.72)</td>
</tr>
<tr>
<td>1,500 (10.34)</td>
<td>3,500 (23.10)</td>
</tr>
<tr>
<td>2,000 (13.79)</td>
<td>4,950 (34.13)</td>
</tr>
<tr>
<td>2,500 (17.24)</td>
<td>6,600 (45.51)</td>
</tr>
<tr>
<td>3,000 (20.69)</td>
<td>8,250 (56.88)</td>
</tr>
<tr>
<td>3,500 (24.13)</td>
<td>9,900 (68.26)</td>
</tr>
<tr>
<td>4,000 (27.58)</td>
<td>11,500 (79.29)</td>
</tr>
</tbody>
</table>

Type S mortar $f_{m}$ can be taken as 2,000 psi

Concrete masonry unit compressive strength $\geq 2,000$

ASD Load Combinations – IBC 2015/ASCE 7-10

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

Unreinforced masonry: masonry resists flexural tension, reinforcement is neglected

Reinforced masonry: masonry in flexural tension neglected, reinforcement resists all tension

Design Methods

ASD – applied stresses service loads ≤ allowed stresses
  \[ f \leq F \text{ Ch 8} \]

Strength – Ch 9

Factored load effects ≤ factored resistance
  \[ aL \leq \phi R \]
Allowable Stresses (ASD) Depend On - VERT. & LATERAL

- Specified masonry compressive strength, $f'_{\text{m}}$
- 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. ($l_{\text{eff}}$ 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, (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
Assumed stress distribution in multiwythe noncomposite walls

**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)

<table>
<thead>
<tr>
<th>Direction of flexural tensile stress and masonry type</th>
<th>Mortar types</th>
<th>Portland cement/lime or mortar cement</th>
<th>Masonry cement or air entrained portland cement/lime</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M or S</td>
<td>N</td>
<td>M or S</td>
</tr>
<tr>
<td>Normal to bed joints</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Solid units</td>
<td>53 (366)</td>
<td>40 (276)</td>
<td>32 (221)</td>
</tr>
<tr>
<td>Hollow units</td>
<td>33 (228)</td>
<td>25 (172)</td>
<td>20 (138)</td>
</tr>
<tr>
<td>Ungrouted</td>
<td>65 (448)</td>
<td>63 (434)</td>
<td>61 (420)</td>
</tr>
<tr>
<td>Fully grouted</td>
<td></td>
<td></td>
<td>58 (400)</td>
</tr>
<tr>
<td>Parallel to bed joints in running bond</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Solid units</td>
<td>106 (731)</td>
<td>80 (552)</td>
<td>64 (441)</td>
</tr>
<tr>
<td>Hollow units</td>
<td>66 (455)</td>
<td>50 (345)</td>
<td>40 (276)</td>
</tr>
<tr>
<td>Ungrouted and partially grouted</td>
<td>106 (731)</td>
<td>80 (552)</td>
<td>64 (441)</td>
</tr>
<tr>
<td>Fully grouted</td>
<td></td>
<td></td>
<td>40 (276)</td>
</tr>
<tr>
<td>Parallel to bed joints in masonry</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Continuous grout section parallel to bed joints</td>
<td>133 (917)</td>
<td>133 (917)</td>
<td>133 (917)</td>
</tr>
<tr>
<td>Other</td>
<td>0 (0)</td>
<td>0 (0)</td>
<td>0 (0)</td>
</tr>
</tbody>
</table>

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

- Compression stress limited  \( f_a \leq F_a, \quad f_b \leq 1/3 f_m \)

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

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

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

- Force unity equation \( \frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \)

- Shear , \( f_v = \frac{VQ}{l_{nb}} \leq 1.5 \sqrt{f'_m}, \) 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

<table>
<thead>
<tr>
<th>Grade</th>
<th>Stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade 40 or 50</td>
<td>20,000</td>
</tr>
<tr>
<td>Grade 60</td>
<td>32,000</td>
</tr>
<tr>
<td>Wire joint reinforcement</td>
<td>30,000</td>
</tr>
</tbody>
</table>

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 $\Sigma$(masonry + tied compressive reinforcement)
- Max. compressive stress in masonry from axial load & bending $\leq (0.45)f'_m$
- 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 \( \frac{1}{4} \) in dia.

b) Vertical spacing of ties \( \leq 16 d_b, 48 d_{ies} \), 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 \(<135^\circ\). Can be in mortar.

d) \( \frac{1}{2} \) 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

\[ n = \frac{E_s}{E_m} \] and from equil.

\[ M_s = A_s f_s j d \quad \text{(at the limit)} \]
\[ M_m = \frac{1}{2} b j k d^2 f_m \quad \text{(at limit)} \]

\[ k = \sqrt{(\rho n)^2 + 2(\rho n) - \rho n} \]
\[ j = 1 - k/3 \]
\[ \rho = \frac{A_s}{bd} \]

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 \( h \sim 32 \)
- \( A_{req} = \frac{M}{F_s j d} = \frac{493,300}{(32,000 \times 0.9 \times 27.8)} = 0.62 \text{ in}^2 \)
- Try 2 – #5 rebar \( As = 0.62 \text{ in}^2 \)
- \( n = \frac{E_s}{E_m} = \frac{29,000,000}{(900)(2,000)} = 16.11 \)

\( j = 0.9 \)
Lintel Design - VERT

Check design

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

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

\[ M_s = A_s \times F_s \times j \times d = 0.62 \times 32,000 \times 0.912 \times 27.8 = 503 \text{ kip. in} > 493.3 \]

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

or

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

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

\[ M_m = 636.5 \text{ kips} \cdot \text{in} > 503 \quad \text{– steel stress governs} \]

- 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 \( f'_m = 1,500 \text{ psi} \), Grade 60 rebar

\[ .6D + 0.6W \text{ governs at mid-height – start with } 1 \text{ ft design width of wall} \]

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

\[ M_{max} = 455.6 \text{ lb} \cdot \text{ft per ft of wall} \]
Design Masonry Wall Flexure Out-of-Plane - LATERAL

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

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

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

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

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

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

\[ n = 290,000,000 / (900(1,500)) = 21.48 \]

Design Masonry Wall Flexure Out-of-Plane LATERAL

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

\[ M_s = A_s j d F_s = 0.31(0.921)(3.81) (32,000)/12 = 2,901 \text{ lb} \cdot \text{ft} \]

\[ F_b = 0.45 f_m' = 0.45(1,500) = 675 \text{ psi} \]

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

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

If \( K_d > \) 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

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
Assume stress gradient range B:

**Not all section in compression, but no tension in steel**

Get equivalent force-couple about center line

\[ P_b = C_m = 0.5(f_{m1})atb \]
\[ M_b = e_m \times C_m \]

\[ e_m = d - \frac{at}{3} = \frac{t}{2} - \frac{at}{3} = t \left( \frac{1}{2} - \frac{\alpha}{3} \right) \]

Note that \( at = kd \)

This is valid until steel goes into tension

Set \( f_{m1} = F_b \) at limit

---

Assume stress gradient range C:

**Section in compression, tension in steel**

Get equivalent force-couple about center line

\[ e_m = d - \frac{at}{3} = \frac{t}{2} - \frac{at}{3} = t \left( \frac{1}{2} - \frac{\alpha}{3} \right) \]

\[ C_m = 0.5(f_{m1})atb \]

\[ P_c = C_m - T_s \text{ and } T_s = A_s \times f_s \]

From similar triangles on stress diagram

\[ f_s/n = ([d - at]/at)f_{m1} \]

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

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 \( at = kd \)
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.

<table>
<thead>
<tr>
<th>P (kip)</th>
<th>M (k*in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D + 0.75L + 0.75(0.6W) at mid-height</td>
<td>2.072</td>
</tr>
<tr>
<td>D + L at top</td>
<td>2</td>
</tr>
<tr>
<td>0.6D + 0.6W at mid-height</td>
<td>0.642667</td>
</tr>
</tbody>
</table>
ASD Interaction Diagram - VERT. & LATERAL

Spreadsheet for calculating allowable-stress M-N diagram for solid masonry wall – center rebar

<table>
<thead>
<tr>
<th>16.67 ft. wall w/ No. 5 at 16 in. (centered)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>total depth, t</td>
<td>7.625 in.</td>
</tr>
<tr>
<td>$f_{m,r}$</td>
<td>1,500 psi</td>
</tr>
<tr>
<td>$E_m$</td>
<td>1,350,000 psi</td>
</tr>
<tr>
<td>Fb</td>
<td>675.00 psi</td>
</tr>
<tr>
<td>Es</td>
<td>29,000,000 psi</td>
</tr>
<tr>
<td>Fs</td>
<td>32,000 psi</td>
</tr>
<tr>
<td>$d$</td>
<td>3.81 in.</td>
</tr>
<tr>
<td>$K_{balanced}$</td>
<td>0.311828</td>
</tr>
<tr>
<td>tensile reinf., $A_s/b_{eff}$</td>
<td>0.31 #5 @ 16 centered</td>
</tr>
<tr>
<td>width, $b_{eff}$</td>
<td>16 in.</td>
</tr>
<tr>
<td>Wall Height, $h$</td>
<td>16.67 ft.</td>
</tr>
<tr>
<td>Radius of Gyration, $r$</td>
<td>90.9 in.</td>
</tr>
<tr>
<td>$f_{m}^{'}$</td>
<td>1,500 psi</td>
</tr>
<tr>
<td>$h/r$</td>
<td>90.9</td>
</tr>
<tr>
<td>Reduction Factor, $R$</td>
<td>0.578</td>
</tr>
<tr>
<td>Allowable Axial Stress, $F_a$</td>
<td>217 psi</td>
</tr>
<tr>
<td>Net Area, $A_n$</td>
<td>121.7 in.²</td>
</tr>
<tr>
<td>Allowable Axial Compr, $P_a$</td>
<td>26384 lb/ft</td>
</tr>
</tbody>
</table>

Note: axial force is per foot of wall

Note: moment equation not valid after $k > 2$

<table>
<thead>
<tr>
<th>RANGE C</th>
<th>Points controlled by steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>k</td>
<td>$K_d$ (at)</td>
</tr>
<tr>
<td>---------</td>
<td>------------</td>
</tr>
<tr>
<td>0.01</td>
<td>0.04</td>
</tr>
<tr>
<td>0.05</td>
<td>0.14</td>
</tr>
<tr>
<td>0.15</td>
<td>0.57</td>
</tr>
<tr>
<td>0.24</td>
<td>0.91</td>
</tr>
<tr>
<td>0.22</td>
<td>0.84</td>
</tr>
<tr>
<td>0.11</td>
<td>0.55</td>
</tr>
<tr>
<td>0.03</td>
<td>1.14</td>
</tr>
<tr>
<td>0.011628</td>
<td>1.11</td>
</tr>
<tr>
<td>0.04</td>
<td>1.52</td>
</tr>
<tr>
<td>0.01</td>
<td>0.91</td>
</tr>
<tr>
<td>0.06</td>
<td>2.23</td>
</tr>
<tr>
<td>0.07</td>
<td>2.61</td>
</tr>
</tbody>
</table>

Points controlled by masonry

<table>
<thead>
<tr>
<th>RANGE B</th>
<th>Pure compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>k</td>
<td>$f_s/n [\pi (r - d)] = f_s$</td>
</tr>
<tr>
<td>---------</td>
<td>------------------</td>
</tr>
<tr>
<td>0.03</td>
<td>$K_{balanced}$</td>
</tr>
<tr>
<td>0.05</td>
<td>0.8</td>
</tr>
<tr>
<td>0.09</td>
<td>3.43</td>
</tr>
<tr>
<td>1.1</td>
<td>4.15</td>
</tr>
<tr>
<td>1.3</td>
<td>4.98</td>
</tr>
<tr>
<td>1.5</td>
<td>5.72</td>
</tr>
<tr>
<td>1.7</td>
<td>6.48</td>
</tr>
<tr>
<td>2</td>
<td>7.62</td>
</tr>
</tbody>
</table>

Note: moment equation not valid after $k > 2$
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).
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 $M_s = M_{\text{max,Applied}}$.** 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.

$$j \frac{d}{d}$$

$$M \quad \text{V} \quad \text{dx}$$

$$f, b \text{dx}$$

$$f_v = \frac{V}{b \cdot j d} \approx \frac{V}{b d} = \text{applied shear stress}$$

$$F_v = \text{varies with type of element}$$
Reinforced Masonry Shear Stresses - VERT. & LATERAL

Shear stress is computed as:

\[ f_v = \frac{V}{A_{nv}} \]  \hspace{1cm} (8-24)

Allowable shear stresses

\[ F_v = (F_{vm} + F_{vs})\gamma_g \]  \hspace{1cm} (8-25)

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

Shear Stress Cutoffs - VERT. & LATERAL

Allowable shear stress limits:

\( M/Vd_v \leq 0.25 \)

\[ F_v = (3\sqrt{f_m^r})\gamma_g \]  \hspace{1cm} (8-26)

\( M/Vd_v \geq 1 \)

\[ F_v = (2\sqrt{f_m^r})\gamma_g \]  \hspace{1cm} (8-27)

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

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

\( M/Vd_v \) 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 \( F_{vm} \)
  \[ F_{vs} = 0.5 \left( \frac{A_v F_{sdv}}{A_{nvS}} \right) \]  
  \( (8-30) \)

  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.
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.

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

Segments get load w.r.t. relative \( k \).

- For cantilevered shear wall segments
  \[
  k_c = E_m t \left[ 4 \left( \frac{h'}{h_w} \right)^3 + 3 \left( \frac{h'}{h_w} \right) \right]^{-1}
  \]

- For fixed-fixed shear wall segments
  \[
  k_c = E_m t \left[ \left( \frac{h'}{h_w} \right)^3 + 3 \left( \frac{h'}{h_w} \right) \right]^{-1}
  \]
### Table 18.1-2 DPC Box Building Shear Load on Wall Segments on the West Wall

<table>
<thead>
<tr>
<th>Segment</th>
<th>H</th>
<th>L</th>
<th>Ri</th>
<th>Vi from Diaphragm (lb)</th>
<th>Vi wt (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22</td>
<td>12</td>
<td>0.332</td>
<td>4.99</td>
<td>4.05</td>
</tr>
<tr>
<td>2</td>
<td>22</td>
<td>24</td>
<td>1.715</td>
<td>25.80</td>
<td>8.1</td>
</tr>
<tr>
<td>3</td>
<td>22</td>
<td>24</td>
<td>1.715</td>
<td>25.80</td>
<td>8.1</td>
</tr>
<tr>
<td>4</td>
<td>22</td>
<td>24</td>
<td>1.715</td>
<td>25.80</td>
<td>8.1</td>
</tr>
<tr>
<td>5</td>
<td>22</td>
<td>24</td>
<td>1.715</td>
<td>25.80</td>
<td>8.1</td>
</tr>
<tr>
<td>6</td>
<td>22</td>
<td>24</td>
<td>1.715</td>
<td>25.80</td>
<td>8.1</td>
</tr>
<tr>
<td>7</td>
<td>22</td>
<td>24</td>
<td>1.715</td>
<td>25.80</td>
<td>8.1</td>
</tr>
<tr>
<td>8</td>
<td>22</td>
<td>6.67</td>
<td>0.065</td>
<td>0.98</td>
<td>2.25</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td>10.687</td>
<td>160.800</td>
<td></td>
</tr>
</tbody>
</table>

Segment 2 designed in later example
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 $\alpha L$ 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) = $\sum (T_i \times (d_i - L/2) + c_m \times (L/2 - \alpha L/3))$

Reinforced Masonry Shear Walls – ASD - LATERAL

Flexure only $P = 0$ on diagram

$V$ = base shear

$M = \text{over turning moment}$

$P = \text{self weight only, ignore}$

Multiple rebar locations
Reinforced Masonry Shear Walls – ASD - LATERAL

(P = 0) Can use the singly reinforced equations

\[ V = base \ shear \]

\[ M = over \ turning \ moment \]

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 \( k^* = (\rho^* n)^2 + 2\rho^* n \)^{1/2} – \( np^* \)
- 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 ($\sum$ about $C$) = $\sum(T_i \times (d_i - k^d d^*/3))$

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

\[ V_{\text{diaphragm}} = 25,800 \text{ lb} \text{ acting 22 ft above foundation} \]
\[ V_{\text{pier}} = 8,100 \text{ lb} \text{ acting 12.7 ft above foundation} \]

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: \[ V_{\text{pier}} = 0.2S_{D,S}D = (-0.2)(1.11)(48,600) = -10,800 \text{ lb} \]

ASD Load Combination: \( 0.6D + 0.7E \)

\[ P = 0.6 \times 48,600 + 0.7 \times -0.2(1.11)(48,600) = 21,600 \text{ lb} \]
\[ M = 0.6 \times 0 + 0.7 \times (25,800 \text{ lb} \times 22 \text{ ft} + 8,100 \text{ lb} \times 12.7 \text{ ft}) \]
\[ = 469,000 \text{ lb} \cdot \text{ft} = 5,630,000 \text{ lb in.} \]
\[ V = 0.6 \times 0 + 0.7 \times (25,800 \text{ lb} + 8,100 \text{ lb}) = 23,700 \text{ lb} \]

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, $M$.

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

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

Shear Wall Example 2 - LATERAL

Try three No. 5 bars, $A_s = 3 \times 0.31 \text{ in}^2 = 0.93 \text{ in}^2$. Calculate $j$ and $k$:

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

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

$$k^* = \sqrt{2n\rho^* + (n\rho^*)^2} - n\rho^* = \sqrt{2 \times 0.000950 + (0.000950)^2} - 0.000950 = 0.129$$

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

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

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

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

Check masonry compression stresses

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

$$f_b = \frac{M}{0.5k^*bd^*} = \frac{5,630,000}{0.5(0.957)(0.129)(7.63)(276)^2} = 157 \text{ psi} \leq 675 \text{ psi}$$
Shear Wall Example 2 - LATERAL

Check shear stress.

Assume no shear reinforcing, and thus:

\[ F_v = F_{vm} + F_{vs} = F_{vm} \]

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

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

\[ = 48.3 \text{ psi} \]

\[ F_v = \gamma_g F_{vm} = (0.75)48.3 = 36.2 \leq \text{(conservatively)} = 2\sqrt{f_m'} \]

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

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

<table>
<thead>
<tr>
<th>SW Type</th>
<th>Minimum Reinforcement</th>
<th>SDC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empirically Designed</td>
<td>None – drift limits and connection force</td>
<td>A</td>
</tr>
<tr>
<td>Ordinary Plain</td>
<td>None – same as A</td>
<td>A, B</td>
</tr>
<tr>
<td>Detailed Plain</td>
<td>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</td>
<td>A, B</td>
</tr>
<tr>
<td>Ordinary Reinforced</td>
<td>Same as above</td>
<td>A, B, C</td>
</tr>
<tr>
<td>Intermediate Reinforced</td>
<td>Same as above, but vertical reinforcement @ 4 ft</td>
<td>A, B, C</td>
</tr>
<tr>
<td>Special Reinforced</td>
<td>Same as above, but horizontal reinforcement @ 4 ft, and $\rho = 0.002$ – no stack bond</td>
<td>any</td>
</tr>
</tbody>
</table>
Minimum reinforcement for detailed plain shear walls and SDC C - LATERAL

- #4 bar (min) within 8 in. of corners & ends of walls
- #4 bar (min) at roof connectors
  As per IBC or ASCE 7
  IBC – 4 ft usual
- #4 bar (min) within 16 in. of top of parapet
- #4 bar (min) @ diaphragms
  continuous through control joint
- #4 bar (min) within 8 in. of all control joints
- #4 bars around openings
- 24 in. or 40 db past opening
- #4 bars @ 10 ft oc & within 16 in. of openings
- #4 bars @ 10 ft oc or 2 leg W1.7 joint reinforcement @ 16 in. oc

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 $\rho_v$ 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

- Roof connectors @ 48 in. max oc
- Roof diaphragm
- #4 bar (min) within 16 in. of top of parapet
- Top of Parapet
- #4 bar (min) @ diaphragms continuous through control joint
- #4 bar (min) within 8 in. of all control joints
- #4 bar (min) @ smallest of 4 ft, L/3, or H/3 (int. SW just 4 ft)
- #4 bar within 8 in. of corners & ends of walls
- #4 bars around openings
- 24 in. or 40 db past opening
- #4 bars min @ smallest of 4 ft, L/3, or H/3 (int. SW just 4 ft)
- Hook to vert.

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.
Thank you for your attention!

Any questions?