Multiple Layers of Reinforcement:

- Design equations for single layer of reinforcement can be used for preliminary steel estimates.
- Spacing of intermediate reinforcing bars often controlled by out-of-plane loading.
- Shear area for a partially grouted shear wall is area of face shells and grouted cells.

Example

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 (from ASCE 7-10) of 90 kips. $S_{DS} = 0.4$

Required: Design the shear wall; ordinary reinforced shear wall

Solution: Check using 0.9D+1.0E load combination.

- Need to know weight of wall to determine $P_u$.
- Need to know reinforcement spacing to determine wall weight.
- Estimate $P_u$ by ignoring wall weight and seismic vertical force
  - $P_u = 0.9(1k/ft)(16ft) = 14.4k$
  - $M_u = (90k)(10ft) = 900k$-ft
Example

Estimate required reinforcement:
1. Use equations for single layer of reinforcement \( (d - d_v - 4 = 188 \text{ in.}) \):
   - \( a = 6.0 \text{ in.}, \) \( A_s = 0.95 \text{ in}^2 \)  
     This is about 3.1 #5 bars; try 2-#5 at end and #5 @ 64 in.
2. Use spreadsheet/computer program and interaction diagram:

![Interaction Diagram]

overstressed by 3.3%; may work

Example

Weight of wall: \( 36 \text{psf}(10\text{ft})(16\text{ft}) = 5760 \text{ lb} \)
Lightweight units, grout at 64 in. o.c. \( 36 \text{ psf} \) (estimated)

\[
P_u = [0.9 - 0.2(S_{DS})]D = [0.9 - 0.2(0.4)](1\text{k/ft}(16\text{ft})+5.8\text{k}) = 17.8 \text{ kips}
\]

With slightly higher axial force
\[
M_n = 995 \text{ k-ft}
\]
\[
\phi M_n \text{ has increased to 896 k-ft}
\]
Wall weight may be slightly greater due to bond beams
Example

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

$$A_{nv} = M_u / (V_u d_v) =$$

Maximum $V_n$  $\phi V_n =$

Example

Top of wall (critical location):  $P_u = (0.9 - 0.2 S_{DS})D = 0.82 \times (1 \, \text{k/ft})(16 \, \text{ft}) = 13.1 \, \text{kips}$

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

$$V_n = \frac{V_u}{\phi} = (V_{nm} + V_{ns}) f_y \implies V_{ns} =$$

Use #5 bars in bond beams. Determine spacing.

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

ASD:  $s \leq \min(d/2, 48 \, \text{in.}) = \min(94 \, \text{in.}, 48 \, \text{in.}) = 48 \, \text{in.}$  Code 8.3.5.2.1

In strength design, this provision only applies to beams (9.3.4.2.3 (e))

Suggest that minimum spacing also be applied to shear walls.

Use #5 bars at 16 inches
Joint Reinforcement

- 9.1.9.3.2 Maximum specified yield strength of 85,000 psi
- 9.3.3.1 (b) Minimum diameter of 3/16 in. diameter.
- 9.3.3.2.3 Anchor around edge reinforcing bar in the edge cell
  - by bar placement between adjacent crosswires, or
  - 90° bend in longitudinal wires with 3-in. bend extensions in mortar or grout.
- 9.3.3.7 Seismic Requirements
  - Seismic Design Categories (SDC) A and B
    - At least two 3/16 in. wires; Maximum spacing of 16 in.
  - SDC C, D, E, and F; partially grouted walls
    - At least two 3/16 in. wires; Maximum spacing of 8 in.
  - SDC C, D, E, and F; fully grouted walls
    - At least four 3/16 in. wires; Maximum spacing of 8 in.

Equivalent Joint Reinforcement Options:

<table>
<thead>
<tr>
<th>Joint Reinforcement</th>
<th>Equivalent Bar Reinforcement</th>
<th>Replaces this Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 - 3/16 in. wires at 16 in.</td>
<td>0.0472 in²/ft</td>
<td>#4 @ 56 in.; #5 @ 80 in.</td>
</tr>
<tr>
<td>2 – 3/16 in. wires at 8 in.</td>
<td>0.0945 in²/ft</td>
<td>#4 @ 32 in.; #5 @ 40 in.</td>
</tr>
<tr>
<td>4 – 3/16 in. wires at 8 in.</td>
<td>0.189 in²/ft</td>
<td>#4 @ 16 in.; #5 @ 24 in.</td>
</tr>
</tbody>
</table>

Bar reinforcement yield stress = 60 ksi
Joint reinforcement yield stress = 70 ksi

- Instead of bond beams in example, could use 4 – 3/16 in. wires at 8 in.
  - but no manufacturer makes this
- Smaller distributed reinforcement generally results in better behavior.
Example: Special Wall

Given: 10 ft high x 16 ft long 8 in. CMU shear wall; Grade 60 steel, Type S mortar; $f'_{m} = 2000$psi; superimposed dead load of 1 kip/ft. In-plane seismic load (from ASCE 7-10) of 90 kips. $S_{DS} = 0.4$

Required: Design the shear wall; special reinforced shear wall

Solution: Check using 0.9D+1.0E load combination.

- Shear capacity design provisions (Section 7.3.2.6.1.1)
  - $\phi V_n \geq$ shear corresponding to $1.25M_n$.
    - Minimum increase is $1.25/0.9 = 1.39$
    - $M_n = 995$ k-ft; $1.25M_n = 1243$ k-ft; design for shear of 124 kips
  - $V_n$ need not exceed $2.5V_u$
    - Normal design $V_n \geq V_d/\phi = V_d/0.8 = 1.25V_u$
    - Increases shear by a factor of 2
  - Fully grout wall (Max $\phi V_n$ was 91.9 kips)

\[ A_{nv} = 7.625 \times 192in. = 1464in^2 \]
**Example: Special Wall**

- Design for shear of 124 kips
- But wait, spacing of #5’s was decreased to 48 inches
  - \( M_n = 1124 \text{ k-ft}, 1.25M_n = 1405 \text{ k-ft}, \) Design for 140 kips
- But wait, wall is fully grouted. Wall weight has increased to 75 psf
  - For \( P_u = 23.0k, \) fully grouted, \( M_n = 1167 \text{ k-ft}, 1.25M_n = 1459 \text{ k-ft} \)
  - Design for 146 kips
- But wait, Section 7.3.2.6 has maximum spacing requirements:
  - 1/3 length of wall = (192 in.)/3 = 64 in.
  - 1/3 height of wall = (120 in.)/3 = 40 in.
  - 48 in. for running bond; 24 in. for not laid in running bond
  - Decrease spacing of vertical reinforcement from 48 in. to 40 in.
- For 2-#5 at end, and #5 @ 40in., \( M_n = 1269 \text{ k-ft} \) \( (P_u = 23.0k) \)
  - 1.25\( M_n = 1586 \text{ k-ft}; \) Design for 159 kips
- Bottom line: any change in wall will change \( M_n, \) which will change design requirement; often easier to just use \( 2.5V_u. \)

---

**Example: Special Wall**

\[
\phi V_{nm} = \phi \left[ 4.0 - 1.75 \left( \frac{M_u}{V_ud_v} \right) A_{nv} \sqrt{f_m} + 0.25P_u \right]
\]

\[
= 0.8 \left[ 4.0 - 1.75(0.625)(1464in^2) \sqrt{2000psi} \frac{1\text{kip}}{1000\text{lb}} + 0.25(13.1k) \right] = 154.8kips
\]

\[
V_{ns} = \frac{V_u - \phi V_{nm}}{\phi} = \frac{158.6k - 154.8k}{0.8} = 4.7k
\]

Use #5 bars in bond beams.

Determine spacing.

\[
V_{ns} = 0.5 \left( \frac{A_v}{s} \right) f_yd_v \quad \Rightarrow \quad s = \frac{0.5A_vf_yd_v}{V_{ns}} = \frac{0.5(0.31in)(60ksi)(192in)}{4.7k} = 383in
\]

Use maximum spacing of 1/3(height) = 40 in.

Section 7.3.2.6(d): Shear reinforcement shall be anchored around vertical reinforcing bars with a standard hook.
Example: Special Wall

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

- Vertical: $8(0.31\text{in}^2)/1464\text{in}^2 = 0.00169$ (spacing of 40 in.)
- Horizontal: $3(0.31\text{in}^2)/[120\text{in}(7.625\text{in})] = 0.00102$
- Total = $0.00169 + 0.00102 = 0.00271$ OK

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Example: Special Wall

Section 9.3.3.5 Maximum Reinforcement
Since $M_d/(V_d d_y) < 1$, strain gradient is based on $1.5\varepsilon_y$.

<table>
<thead>
<tr>
<th>Strain</th>
<th>$c/d$, CMU</th>
<th>$c/d$, Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1.5\varepsilon_y$</td>
<td>0.446</td>
<td>0.530</td>
</tr>
<tr>
<td>$3\varepsilon_y$</td>
<td>0.287</td>
<td>0.360</td>
</tr>
<tr>
<td>$4\varepsilon_y$</td>
<td>0.232</td>
<td>0.297</td>
</tr>
</tbody>
</table>

c = $0.446(188\text{in.}) = 83.8$ in.

- Calculate axial force based on $c = 83.8$ in.
- Include compression reinforcement
- $\phi P_n = 732$ kips
- Assume a live load of 1 k/ft
- $D + 0.75L + 0.525Q_E = (1\text{k/ft} + 0.75(1\text{k/ft}))16\text{ft} = 28$ kips OK
Example: Special Wall

- Section 9.3.6.5: Maximum reinforcement provisions of 9.3.3.5 do not apply if designed by this section (boundary elements)
- Special boundary elements not required if:

\[
P_u \leq 0.1f_{\text{m}}^\prime A_g \quad \text{geometrically symmetrical sections}
\]
\[
P_u \leq 0.05f_{\text{m}}^\prime A_g \quad \text{geometrically unsymmetrical sections}
\]

AND

\[
\frac{M_u}{V_u d_v} \leq 1 \quad \text{OR} \quad V_u \leq 3A_g \sqrt{f_{\text{m}}^\prime} \quad \text{AND} \quad \frac{M_u}{V_u d_v} \leq 3
\]

For our wall, \( \frac{M_u}{V_u d_v} < 1 \)  
\( P_u < 0.1f_{\text{m}}'A_g = 0.1(2.0\text{ksi})(1464\text{in}^2) = 293 \text{kips} \)

Special 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 7.3.2.6.1.1)
  - \( \phi V_n \geq \text{shear corresponding to } 1.25M_n \)
  - \( V_n \text{ need not exceed } 2.5V_u \)
- Maximum Reinforcement Requirements (9.3.3.5; 9.3.6.5)
Example: T-Wall

Given: 10 ft high x 16 ft long 8 in. CMU shear wall; Grade 60 steel, Type S mortar; $f_m=2000\text{psi}$; superimposed dead load of 1 kip/ft. In-plane seismic load (from ASCE 7-10) of 90 kips. $S_{DS} = 0.4$; intersecting wall on one side.

Required: Design the shear wall; ordinary reinforced shear wall

Solution: Check using 0.9D+1.0E load combination.

Effective flange width:
- $6t = 6(8\text{in.}) = 48\text{ in. compression}$
- $0.75h = 0.75(120\text{in.}) = 90\text{ in. tension}$

Example: T-Wall

Flange in tension: Approximate as a single layer of reinforcement. Use design procedure for single layer of reinforcement with $P_u = 18.1\text{k}$, $M_u = 900\text{k-ft.}$, $b=7.625\text{in.}$, $l_w=200\text{in.}$ (16 ft + 8 in. flange); $d=196\text{in.}$

Required reinforcement: $a = 5.9\text{in.}$, $A_s = 0.87\text{in}^2$ or 3 - #5.

Flange in compression: Reinforcement will be approximately the same as for a non-flanged wall. Increase in compression area will only slightly reduce required steel.

Axial load on just the web creates a moment with a small tension in the flange and compression in the web.
Trial Design

Tension: 3 bars total; assume spacing of 48 in. for OOP
90 in. flange
Compression: 3 grouted cells within flange

Flange compression area:
\[(48+48+7.62)(2.5) + 3(8)(7.625-2.5) = 382\text{in}^2\]
Equivalent thickness = \(382\text{in}^2/7.625\text{in.} = 50.1\text{in.}\)

Check maximum reinforcement with flange in tension:
\[\frac{c}{d} = 0.446 (\alpha = 1.5)\]
\[c = 0.446(196\text{in.}) = 87.4 \text{in.}\]
\[\phi P_n = 285.3 \text{ kips}\]
\[P_u = D + 0.75L\]
\[= (6.1 + 16) + 0.75(16)\]
\[= 34.1 \text{ kips}\]

Perhaps should include steel just outside effective tension flange:
\[\phi P_n = 251.8 \text{ kips}\]
Example: T-Wall

Shear strength is based on the flange, and similar to previous example. Use #5 @ 40 in.

Check shear at interface. Check using intersecting bond beams.

Reinforcement from #5@40in.
0.093in²/ft close to 0.1in²/ft

Shear at interface: Set equal to tension force in reinforcement in flange.

\[ V = T = \frac{f_y A_{sf}}{12in.} = 60000psi(3)(0.31in^2) = 55800 \text{ lb} \]

\[ A_{nv} = 2.5in(120in) + 3(8in)(7.62in - 2.5in) = 423in^2 \]

\[ \phi V_{nm} = \phi \left[ 4.0 - 1.75 \left( \frac{M_u}{V_u d_v} \right) \right] A_{nv} \sqrt{f_m'} + 0.25P_u = \phi (2.25) A_{nv} \sqrt{f_m'} \]

\[ = 0.8 \left( 2.25 \right) \left( 423in^2 \right) \sqrt{\frac{2000 \text{ psi}}{25.0}} = 34050 \text{ lb} \]

If used \( \gamma = 0.75 \), \( \phi V_{nm} = 25,500 \text{ lbs} \)
Example: T-Wall

Need to fully grout interface: \( A_{nv} = 915\text{in}^2; \) \( \phi V_n = 73600\text{ lb} \)

Additional cells to be grouted

Suggest adding shear straps between each bond beam (16 in. o.c.)

Suggest using bond beam units and cutting out half of face-shell in flange blocks to get good grout at interface.