Engineering Notes for Design with Concrete Block Masonry

# MASONRY CHRONICLES

Summer 2021

Gilbert/Kivett Residence
Honor Award Winner of the 2017 CMACN Design Awards in Residential Design
Corsini Stark Architects, LLP
Steven King Architectural Photography

## DESIGN OF REINFORCED MASONRY SITE WALLS FOR SEISMIC LOADS

This paper covers the design of reinforced masonry site walls according to TMS 402 and ASCE 7-16. Only the design due to out-of-plane seismic loads is considered, and only cantilever walls are covered. The strength design method of TMS 402 is used.

The paper is divided into three parts: The first part is a design procedure to determine the required reinforcing in the wall. The second part is examples. The third part is appendices which provide additional background information.

Concrete Masonry Association of California and Nevada

## Part 1: Design Procedure

The out-of-plane loading is determined using the following procedure. Background on the procedure is given in Appendix A.

1. Determine $S_{DS}$ for the site. This can be done using the ATC hazard web site, https://hazards.atcouncil.org/#/, or an equivalent hazard tool.

2. Determine $I_e$, the importance factor of the wall. Per ASCE 7-16, the importance factor would typically be 1.0. The importance factor would be 1.5 if the wall were required to function for life-safety purposes after an earthquake or the wall were attached to a Risk Category IV structure and it is needed for continued operation of the facility or its failure could impair the continued operation of the facility.

3. Determine $w$, the weight of the wall. The following table can be used to estimate the weight of the wall for preliminary design. Specific wall weights are given in NCMA TEK 14-13B, Concrete Masonry Wall Weights, which is available at https://ncma.org/resource/concrete-masonry-wall-weights/.

| Wall Thickness (inch) | Wall Weight (psf) | |
|---|---|---|
| | Fully Grouted | Partially Grouted |
| 8 | 81 | 50 |
| 10 | 103 | 60 |
| 12 | 125 | 70 |

4. Calculate $M_u$, the factored moment using the following equation, where $h$ is the height of the wall and $w$ is the weight of the wall. Typically, $w$ is in units of psf, $h$ is in units of feet, and $M_u$ is in units of lb-ft/ft length of wall.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2$$

5. Select the required reinforcement from the following tables. Except as noted, the tables are for $f'_m = 2000\ psi$ and Grade 60 reinforcement. For other conditions that are not covered in the tables, determine the required reinforcement using procedures found in sources such as the Design of Reinforced Masonry Structures (CMACN), Reinforced Masonry Engineering Handbook (MIA) or Strength Design of Masonry (TMS).

   Notes on design tables:

   A. In a few cases a slightly higher $f'_m$ is required to meet the maximum reinforcement requirements of TMS 402 Section 9.3.3.2. In those cases, the required $f'_m$ is given along with the required unit strength, $f_u$ (if using Unit Strength Method), to meet this $f'_m$ requirement using

Table 2 of TMS 602. If the required $f'_m$ is greater than 3000 psi, no value is given. When walls are designed using an $f'_m$ of 2,500 psi or greater, CMACN recommends using the Prism test method to verify the design $f'_m$. In many instances "off the shelf" CMU may be used rather than higher strength units that may need to be special ordered. In California and Nevada, there is a significant cost increase from "off the shelf" CMU and CMU that will meet the strength requirements for $f'_m$ 3,000 psi using Unit Strength Method. The additional cost per CMU is 70 to 80 percent higher than "standard" strength units.

B. The equivalent rectangular stress block is typically in the face shell, so the results apply to both fully grouted and partially grouted sections. Exceptions are #7 @ 16 in. for centered bars (all block sizes), #5, #6, and #7 @ 16 in. and #7 @ 24 in. for two layers of reinforcement (all block sizes). The design moment strength would be slightly greater for full grout, but not significantly so.

C. With two layers of reinforcement, the reinforcement is assumed to be centered 2.5 in. from the face of the block.

D. With two layers of reinforcement, both layers are considered if they are both in tension. The reinforcement is assumed not to have any strength in compression consistent with TMS 402.

E. TMS 402 Section 7.4.4.1 requires a minimum reinforcement of #4 @ 48 in. for Seismic Design Category D and higher, so the table is limited to a maximum 48 in. spacing. When using Chapter 21A of the California Building Code, minimum reinforcement requirements are #5 @ 24 inches.

**8 inch CMU, Reinforcement Centered in Wall**

| Reinforcement Spacing (in.) | Design Moment Strength, $\phi M_n$ (kip-ft/ft) | | | |
|---|---|---|---|---|
| | #4 | #5 | #6 | #7 |
| 8 | 4.51 | 6.56<br>$f'_m$ = 2150 psi<br>$f_u$ = 2350 psi | 9.28<br>$f'_m$ = 3000 psi<br>$f_u$ = 4500 psi | |
| 16 | 2.42 | 3.61 | 4.90 | 6.40<br>$f'_m$ = 2150 psi<br>$f_u$ = 2350 psi |
| 24 | 1.65 | 2.49 | 3.43 | 4.51 |
| 32 | 1.25 | 1.90 | 2.64 | 3.50 |
| 40 | 1.00 | 1.53 | 2.14 | 2.86 |
| 48 | 0.84 | 1.29 | 1.80 | 2.42 |

### 10 inch CMU, Reinforcement Centered in Wall

| Reinforcement Spacing (in.) | Design Moment Strength, $\phi M_n$ (kip-ft/ft) | | | |
|---|---|---|---|---|
| | #4 | #5 | #6 | #7 |
| 8 | 5.86 | 8.55 | 11.74<br>$f'_m$ = 2400 psi<br>$f_u$ = 3000 psi | |
| 16 | 3.09 | 4.65 | 6.38 | 8.30 |
| 24 | 2.10 | 3.19 | 4.42 | 5.86 |
| 32 | 1.58 | 2.42 | 3.38 | 4.52 |
| 40 | 1.27 | 1.95 | 2.74 | 3.67 |
| 48 | 1.07 | 1.64 | 2.30 | 3.09 |

### 12 inch CMU, Reinforcement Centered in Wall

| Reinforcement Spacing (in.) | Design Moment Strength, $\phi M_n$ (kip-ft/ft) | | | |
|---|---|---|---|---|
| | #4 | #5 | #6 | #7 |
| 8 | 7.21 | 10.64 | 14.20 | 19.32<br>$f'_m$ = 2700 psi<br>$f_u$ = 3800 psi |
| 16 | 3.77 | 5.70 | 7.87 | 10.33 |
| 24 | 2.55 | 3.89 | 5.41 | 7.21 |
| 32 | 1.92 | 2.95 | 4.12 | 5.53 |
| 40 | 1.54 | 2.37 | 3.33 | 4.48 |
| 48 | 1.29 | 1.98 | 2.79 | 3.77 |

### 10 inch CMU, Two Layers of Reinforcement

| Reinforcement Spacing (in.) | Design Moment Strength, $\phi M_n$ (kip-ft/ft) | | | |
|---|---|---|---|---|
| | #4 | #5 | #6 | #7 |
| 8 | 9.89 | 13.75 | 18.10 | 23.16 |
| 16 | 5.86 | 8.22 | 10.48 | 13.15 |
| 24 | 4.05 | 6.04 | 7.90 | 9.75 |
| 32 | 3.09 | 4.66 | 6.38 | 8.03 |
| 40 | 2.50 | 3.79 | 5.23 | 6.85 |
| 48 | 2.10 | 3.19 | 4.42 | 5.86 |

### 12 inch CMU, Two Layers of Reinforcement

| Reinforcement Spacing (in.) | Design Moment Strength, $\phi M_n$ (kip-ft/ft) | | | |
|---|---|---|---|---|
| | #4 | #5 | #6 | #7 |
| 8 | 12.59 | 17.93 | 24.04 | 31.26 |
| 16 | 7.21 | 10.31 | 13.45 | 17.20 |
| 24 | 4.95 | 7.43 | 9.88 | 12.45 |
| 32 | 3.77 | 5.70 | 7.87 | 10.05 |
| 40 | 3.04 | 4.62 | 6.42 | 8.47 |
| 48 | 2.55 | 3.89 | 5.41 | 7.21 |

4. CMACN Summer 2021 Issue of Masonry Chronicles

**Design Tip:** Effect of $f'_m$:

The effect of the specified compressive strength of the masonry, $f'_m$, on the design moment strength is minimal, with the impact of a higher $f'_m$ on the design moment strength being shown in the following table for 10 in. CMU with a #6 centered bar. An increased $f'_m$ has little effect for small amounts of reinforcing, but has a greater effect for larger amounts of reinforcing, although still less than a 10% impact. Therefore, increasing $f'_m$ usually has little benefit. Typically the only reason to use a higher $f'_m$ is to meet the maximum reinforcement provisions.

| Reinforcement Spacing (in.) | Design Moment Strength, $\phi M_n$ (kip-ft/ft) [Percent increase over $f'_m$ = 2000 psi] | | |
|---|---|---|---|
| | $f'_m$ = 2000 psi | $f'_m$ = 2500 psi | $f'_m$ = 3000 psi |
| 8 | 11.23 | 11.84 [5.4%] | 12.25 [9.1%] |
| 16 | 6.38 | 6.53 [2.4%] | 6.64 [4.0%] |
| 24 | 4.42 | 4.49 [1.5%] | 4.54 [2.6%] |
| 32 | 3.38 | 3.42 [1.1%] | 3.45 [1.9%] |
| 40 | 2.74 | 2.76 [0.9%] | 2.78 [1.5%] |
| 48 | 2.30 | 2.31 [0.7%] | 2.33 [1.2%] |

Evergreen Valley College
Automotive Technology Building
Honor Award Winner of the 2017 CMACN
Design Awards in Education Design
Architect: Lionakis
Photography: Tim Maloney, Technical Imagery Studios

## Part 2: Design Examples

Example 1A:

Determine the required reinforcement for an 8 ft high CMU wall with $S_{DS} = 1.32$ and $I_e = 1.00$. This value of $S_{DS}$ would be typical of Los Angeles or San Francisco.

From Step 3 of the Design Procedure, estimate the wall weight. Assume a partially grouted 8 in. wall; estimate $w = 50\,psf$.

From Step 4, determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(1.32)(1.0)(50psf)(8ft)^2 = 2{,}250\,\frac{lb\cdot ft}{ft} = 2.25\,\frac{kip\cdot ft}{ft}$$

From Step 5, possible reinforcement would be as follows. The wall weight is for 125 pcf units.

- 8 in. CMU with #4 @ 16 in., $\phi M_n = 2.42\frac{k\cdot ft}{ft}$, $w = 60\,psf$
- 8 in. CMU with #5 @ 24 in., $\phi M_n = 2.49\frac{k\cdot ft}{ft}$, $w = 52\,psf$
- 8 in. CMU with #6 @ 32 in., $\phi M_n = 2.64\frac{k\cdot ft}{ft}$, $w = 48\,psf$

The design could be refined by noting that #6 @ 32 in. has 17% greater capacity than needed, and the wall weight is slightly smaller than assumed. Try #6 @ 40 in. The factored moment is directly proportional to $w$, so the factored moment would be reduced due to the lighter wall weight than assumed. For 40 in. spacing, the wall weight is 46 psf for 125 pcf units. The new factored moment is:

$$M_u = 2.25\,\frac{kip\cdot ft}{ft}\,\frac{46psf}{50psf} = 2.07\,\frac{kip\cdot ft}{ft}$$

The design strength for #6 @ 40 in. is $\phi M_n = 2.14\frac{k\cdot ft}{ft}$.

The final design is 8 in. CMU with #6 Grade 60 reinforcement @ 40 in., $f'_m = 2000\,psi$.

Example 1B:

Repeat Example 1A, except for a wall height of 10 ft.

Assume a partially grouted 8 in. wall; estimate $w = 50\,psf$. Determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(1.32)(1.0)(50psf)(10ft)^2 = 3{,}520\,\frac{lb\cdot ft}{ft} = 3.52\,\frac{kip\cdot ft}{ft}$$

Try 8 in. CMU with #7 Grade 60 reinforcement @ 32 in. The design moment is $\phi M_n = 3.50\frac{k\cdot ft}{ft}$, which is less than the factored moment, but the wall weight is only 48 psf, which reduces the factored moment to $M_u = 3.38\frac{kip\cdot ft}{ft}$.

Use 8 in. CMU with #7 Grade 60 reinforcement @ 32 in., $f'_m = 2000\,psi$.

Example 1C:

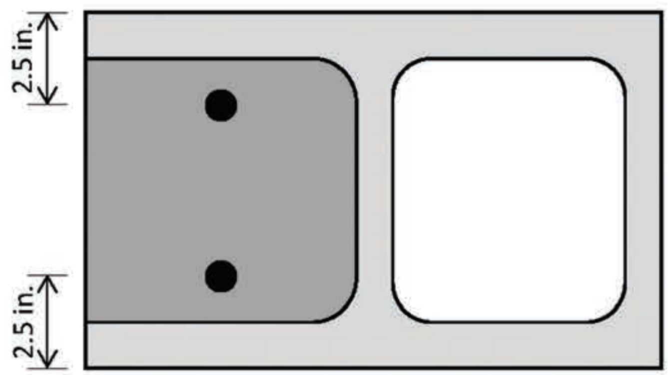
Repeat Example 1A, except for a wall height of 12 ft.

Assume a partially grouted 10 in. wall; estimate $w = 60\,psf$. Determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(1.32)(1.0)(60psf)(12ft)^2 = 6{,}080\,\frac{lb\cdot ft}{ft} = 6.08\,\frac{kip\cdot ft}{ft}$$

Try two layers of #6 @ 32 in., $\phi M_n = 6.38\,\frac{k\cdot ft}{ft}$, $w = 58\,psf$, $f_m' = 2000\,psi$. Since the actual wall weight is slightly smaller than the assumed wall weight, this design is adequate.

Use 10 in. CMU with two layers of #6 Grade 60 reinforcement @ 32 in., each bar is 2.5 in. from the face. The reinforcement configuration is shown below.




If 10 in. CMU were not readily available, 12 in. CMU could be used with the same #6 @ 32 in. reinforcement. The wall would be heavier, with $w = 66\,psf$, increasing the factored moment to $M_u = 6.69\,\frac{kip\cdot ft}{ft}$. However, the reinforcement is at a greater depth, increasing the design moment to $\phi M_n = 7.87\,\frac{k\cdot ft}{ft}$. A refined design would be #6 @ 40 in., with $w = 62\,psf$, $M_u = 6.28\,\frac{kip\cdot ft}{ft}$, and $\phi M_n = 6.42\,\frac{k\cdot ft}{ft}$.

Example 1D:

Repeat Example 1C, except for an importance factor of $I_e = 1.5$.

Assume a partially grouted 12 in. wall; estimate $w = 70\,psf$. Determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(1.32)(1.5)(70psf)(12ft)^2 = 10{,}640\,\frac{lb\cdot ft}{ft} = 10.64\,\frac{kip\cdot ft}{ft}$$

Try 2-#7 @ 32 in. The design moment is $\phi M_n = 10.05\frac{k \cdot ft}{ft}$, which is less than the factored moment, but the wall weight is only 66 psf, which reduces the factored moment to $M_u = 10.04\frac{kip \cdot ft}{ft}$. Use 12 in. CMU with two layers of #7 Grade 60 reinforcement @ 32 in., each bar is 2.5 in. from the face, $f'_m = 2000\ psi$.

Example 2A:

Determine the required reinforcement for an 8 ft high CMU wall with $S_{DS} = 0.51$ and $I_e = 1.00$. This value of $S_{DS}$ would be typical of Sacramento.

From Step 3 of the Design Procedure, estimate the wall weight. Assume a partially grouted 8 in. wall; estimate $w = 50\ psf$.

From Step 4, determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(0.51)(1.0)(50psf)(8ft)^2 = 870\frac{lb \cdot ft}{ft} = 0.87\frac{kip \cdot ft}{ft}$$

Try 8 in. CMU with #4 @ 48 in., $\phi M_n = 0.84\frac{k \cdot ft}{ft}$. For 125 pcf units, $w = 44\ psf$, which reduces the factored moment to $M_u = 0.77\frac{kip \cdot ft}{ft}$.

Use 8 in. CMU with #4 Grade 60 reinforcement @ 48 in., $f'_m = 2000\ psi$. This is the minimum prescriptive reinforcement required by TMS 402 for Seismic Design Category D.

Example 2B:

Repeat Example 2A, except for a wall height of 10 ft.

Assume a partially grouted 8 in. wall; estimate $w = 50\ psf$. Determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(0.51)(1.0)(50psf)(10ft)^2 = 1{,}360\frac{lb \cdot ft}{ft} = 1.36\frac{kip \cdot ft}{ft}$$

Try 8 in. CMU with #5 @ 48 in., $\phi M_n = 1.29\frac{k \cdot ft}{ft}$. The wall weight is $w = 44\ psf$, which reduces the factored moment to $M_u = 1.20\frac{kip \cdot ft}{ft}$.

Use 8 in. CMU with #5 Grade 60 reinforcement @ 48 in., $f'_m = 2000\ psi$.

Example 2C:

Repeat Example 2A, except for a wall height of 12 ft.

Assume a partially grouted 8 in. wall; estimate $w = 50\ psf$. Determine the factored moment, $M_u$.

$$M_u = \frac{8}{15} S_{DS} I_e w h^2 = \frac{8}{15}(0.51)(1.0)(50psf)(12ft)^2 = 1{,}960\frac{lb \cdot ft}{ft} = 1.96\frac{kip \cdot ft}{ft}$$

Try 8 in. CMU with #6 @ 48 in., $\phi M_n = 1.80\frac{k\cdot ft}{ft}$. The wall weight is $w = 44\,psf$, which reduces the factored moment to $M_u = 1.72\frac{kip\cdot ft}{ft}$.

Use 8 in. CMU with #6 Grade 60 reinforcement @ 48 in., $f'_m = 2000\,psi$.

Example 2D:

Repeat Example 2C, except for an importance factor of $I_e = 1.5$.

Assume a partially grouted 8 in. wall; estimate $w = 50\,psf$. Determine the factored moment, $M_u$.

$$M_u = \frac{8}{15}S_{DS}I_pW_ph^2 = \frac{8}{15}(0.51)(1.5)(50psf)(12ft)^2 = 2{,}940\frac{lb\cdot ft}{ft} = 2.94\frac{kip\cdot ft}{ft}$$

Try 8 in. CMU with #7 Grade 60 reinforcement @ 40 in., $\phi M_n = 2.86\frac{k\cdot ft}{ft}$. The wall weight is $w = 46\,psf$, which reduces the factored moment to $M_u = 2.70\frac{kip\cdot ft}{ft}$.

Use 8 in. CMU with #7 Grade 60 reinforcement @ 40 in., $f'_m = 2000\,psi$.


Encinitas Fire Station 2
Merit Award Winner of the 2017 CMACN
Design Awards in Public/Civic Design
domusstudio architecture
Jim Brady, Brady Architectural Photography

## Part 3: Appendices

### Appendix A: Determination of Factored Moment and Shear

The out-of-plane loading is determined from ASCE 7-16 Chapter 15 Seismic Design Forces for Nonbuilding Structures.

1. Section 15.6.8.2 requires walls or fences to be designed to resist earthquake ground motions in accordance with Section 15.4.

2. Per Section 15.4.1, part 1.b, non-building structures not similar to buildings, the $R$ value shall be determined from Table 15.4-2 for the entry "ground-supported cantilever walls or fences", and is 1.25.

3. Section 15.4.1 directs to Section 12.8 to determine the seismic forces, with some modifications to the minimum value of $C_s$.

4. From Section 12.8.1, the base shear is $V = C_s W$. Often for site walls, the period of the wall need not be determined if the peak of the design spectra is used, which for taller walls yield a conservative design. Take the value of $C_s$ as $C_s = S_{DS}/(R/I_e)$, Eq. 12.8-2.

5. Section 15.4.1 part 4 requires the vertical distribution of the lateral seismic forces to be determined using the requirements of Section 12.8.3, assuming the equivalent lateral force procedure is being used. Section 12.8.3 is intended for discrete masses at multiple levels over the height of the structure. For vertically distributed masses, the masses can be lumped as $w\Delta h$, where $w$ is the distributed wall weight and $\Delta h$ is a discretized wall height. In the limit, as $\Delta h$ goes to 0, the base moment is obtained as (derivation given below):

$$M_u = \frac{k+1}{k+2} C_s w h^2$$

   Where $k$ is per Section 12.8.3.

   a. For $k = 1$, the distributed load is an inverted triangle, with the resultant base shear applied at (2/3)$h$. This results in a base moment of $M_u = \frac{2}{3} C_s w h^2$.

   b. For the maximum of $k = 2$ applicable to tall walls with a period of 2.5 seconds or larger, the distributed load increases parabolically over the height of the wall, with a resultant base shear applied at (3/4)$h$. This results in a base moment of $M_u = \frac{3}{4} C_s w h^2$, or 12.5% greater than with $k = 1$. It would be overly conservative to use the maximum value of $k = 2$ and design walls at the peak of the design spectra using equation 12.8-2 since for longer period walls the design forces will decrease at a much greater rate than the effects of the force distribution by increasing $k$ from 1 to 2.

6. Putting step 4 and step 5 together, the moment at the base of a cantilever wall can be obtained as:

$$M_u = \frac{2}{3}\frac{S_{DS}}{R/I_e} w h^2 = \frac{2}{3}\frac{S_{DS}}{1.25/I_e} w h^2 = \frac{8}{15} S_{DS} I_e w h^2$$

## Derivation of Factored Moment

The distributed load on the cantilever would be:

$$f_x = c_{vx}V = c_{vx}C_S wh$$

where $f_x$ would have units of force/length (lb/ft), $c_{vx}$ would have units of 1/length (1/ft), $C_s$ is unitless, $w$ has units of force/length (lb/ft), and $h$ has units of length. Lower case notation is used to indicate a distributed parameter.

The force, $f_x$, would be proportional to the mass (assumed to be uniform, $w$) and the displacement. The displacement, using ASCE 7-16, would be proportional to $x^k$, where $x$ is the height above the base and $k$ varies between 1 and 2 based on the period. Therefore $f_x = Cx^k$, where $C$ is a constant to be determined.

The total force has to be equal to the base shear, so:

$$\int_0^h f_x dx = C_s wh$$

$$\int_0^h Cx^k dx = C_s wh$$

$$C\frac{x^{k+1}}{k+1}\bigg|_0^h = C\frac{h^{k+1}}{k+1} = C_s wh$$

$$C = C_s wh\frac{k+1}{h^{k+1}}$$

Substituting into the first equation:

$$f_x = C_s wh\frac{k+1}{h^{k+1}}x^k = c_{vx}C_s wh$$

Therefore:

$$c_{vx} = \frac{k+1}{h^{k+1}}x^k$$

The base moment would be obtained as:

$$M_u = \int_0^h f_x x dx = \int_0^h c_{vx}C_s whx dx = \int_0^h \frac{k+1}{h^{k+1}}x^k C_s whx dx$$

$$M_u = C_s wh\frac{k+1}{h^{k+1}}\int_0^h x^{k+1}dx = C_s wh\frac{k+1}{h^{k+1}}\frac{h^{k+2}}{k+2} = C_s wh\frac{k+1}{k+2}h = \frac{k+1}{k+2}C_s wh^2$$

## Appendix B: Shear Design

The design shear strength of the wall under out-of-plane loads can be determined as a lower bound of TMS 402 Equation 9-20:

$$\phi V_n = 0.8(2.25)bd\sqrt{f'_m} = 1.8bd\sqrt{f'_m}$$

For example, for an 8 in. CMU wall with centered reinforcement, the design shear strength is:

$$\phi V_n = 1.8bd\sqrt{f'_m} = 1.8\left(12\frac{in.}{ft}\right)(3.81in.)\sqrt{2000psi} = 3{,}680\frac{lb}{ft}$$

The table below gives the design shear strength for $f'_m$ = 2000 psi for various configurations.

| Reinforcement Spacing | Design Shear Strength, $\phi V_n$ (kip/ft) | | | | |
|---|---|---|---|---|---|
| | 8 in. CMU Centered Bars | 10 in. CMU Centered Bars | 12 in. CMU Centered Bars | 10 in. CMU 2 layers | 12 in. CMU 2 layers |
| 8 | 3.68 | 4.65 | 5.61 | 6.88 | 8.81 |
| 16 | 1.84 | 2.32 | 2.81 | 3.44 | 4.41 |
| 24 | 1.23 | 1.55 | 1.87 | 2.29 | 2.94 |
| 32 | 0.92 | 1.16 | 1.40 | 1.72 | 2.20 |
| 40 | 0.74 | 0.93 | 1.12 | 1.38 | 1.76 |
| 48 | 0.61 | 0.77 | 0.94 | 1.15 | 1.47 |

The following table shows the factored shear load, the design shear strength, and the ratio of the factored shear load to the design shear strength for each of the example problems. It can be seen that the factored shear is always much less than the design shear. For this reason, out-of-plane shear generally does not need to be checked but is okay by inspection.

| Example | Factored Shear, $V_u$ (kip/ft) | Design Shear, $\phi V_n$ (kip/ft) | $V_u/\phi V_n$ |
|---|---|---|---|
| 1A: 8 in. CMU; #6 @ 40 in. centered | 0.39 | 0.74 | 0.53 |
| 1B: 8 in. CMU; #7 @ 32 in. centered | 0.53 | 0.92 | 0.57 |
| 1C: 10 in. CMU; #6 @ 32 in. two layers | 0.73 | 1.72 | 0.43 |
| 1D: 12 in. CMU; #7 @ 32 in. two layers | 1.25 | 2.20 | 0.57 |
| 2A: 8 in. CMU; #4 @ 48 in. centered | 0.14 | 0.61 | 0.23 |
| 2B: 8 in. CMU; #5 @ 48 in. centered | 0.18 | 0.61 | 0.29 |
| 2C: 8 in. CMU; #6 @ 48 in. centered | 0.22 | 0.61 | 0.35 |
| 2D: 8 in. CMU; #7 @ 40 in. centered | 0.34 | 0.74 | 0.46 |

## Appendix C: Second-Order Moments

Technically TMS 402 Section 9.3.5.4.1 requires the inclusion of second-order moments for all walls. The TMS 402 equations 9-23 through 9-26 are for simply supported walls with a uniform load. Similar equations can be derived for a cantilever wall with an inverted triangular load.

The deflection at the top of a cantilever, $\delta_u$, for an inverted triangular load (triangular base at the free end) is, where $w_u$ is the distributed load at the top of the wall:

$$\delta_u = \frac{11}{120}\frac{w_u h^4}{EI}$$

The first-order moment, $M_{u,0}$ at the base of the cantilever is:

$$M_{u,0} = \frac{1}{3} w_u h^2$$

Substituting this into the deflection equation:

$$\delta_u = \frac{11}{40}\frac{M_u h^2}{EI}$$

Two equations can now be written that are equivalent to TMS 402 Equations 9-23 and 9-26.

$$M_u = \frac{1}{3} w_u h^2 + \frac{P_u}{2}\delta_u$$

$$\delta_u = \frac{11}{40}\frac{M_{cr} h^2}{E_m I_n} + \frac{11}{40}\frac{(M_u - M_{cr})h^2}{E_m I_{cr}}$$

where $P_u$ is the factored wall weight (half assumed to be acting at the top of the wall), $I_n$ is the net moment of inertia, $I_{cr}$ is the cracked moment of inertia, and $M_{cr}$ is the cracking moment. For this particular case, $w_u = 1.6 S_{DS} w I_e$. The modulus of elasticity of the masonry, $E_m$, is $900 f'_m$ for concrete masonry. The cracked moment of inertia is obtained as:

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

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b}$$

where $n$ is the ratio of the modulus of elasticity of the reinforcement to the modulus of elasticity of the masonry.

The factored axial force per ASCE 7-16 Section 2.3.6 would be $P_u = (0.9 - 0.2 S_{DS})D$.

Solving the simultaneous linear equations for $M_u$ and $\delta_u$, the factored moment can be obtained as:

$$M_u\left[1 - \frac{P_u}{2}\frac{11}{40}\frac{h^2}{E_m I_{cr}}\right] = \frac{1}{3} w_u h^2 + \frac{P_u}{2}\frac{11}{40}\frac{M_{cr} h^2}{E_m}\left(\frac{1}{I_n} - \frac{1}{I_{cr}}\right)$$

The following table shows the first-order factored moment or the factored moment ignoring second-order effects, the factored moment including second-order effects, and the ratio. For purposes of determining the cracking moment, Type S masonry cement mortar was assumed. If Portland-cement lime mortar were used, the cracking moment would be larger and the second-order moment smaller.

| Example | Factored Moment, $M_u$ (kip-ft/ft) | | Ratio |
|---|---|---|---|
| | First-order | Second-order | |
| 1A: 8 in. CMU; #6 @ 40 in. centered | 2.073 | 2.083 | 1.005 |
| 1B: 8 in. CMU; #7 @ 32 in. centered | 3.379 | 3.407 | 1.008 |
| 1C: 10 in. CMU; #6 @ 32 in. two layers | 5.880 | 5.911 | 1.005 |
| 1D: 12 in. CMU; #7 @ 32 in. two layers | 10.036 | 10.065 | 1.003 |
| 2A: 8 in. CMU; #4 @ 48 in. centered | 0.766 | 0.770 | 1.006 |
| 2B: 8 in. CMU; #5 @ 48 in. centered | 1.197 | 1.213 | 1.013 |
| 2C: 8 in. CMU; #6 @ 48 in. centered | 1.723 | 1.761 | 1.022 |
| 2D: 8 in. CMU; #7 @ 40 in. centered | 2.703 | 2.754 | 1.019 |

The increase in the factored moment due to second-order effects is minimal. Appendix D will show that the increase in design moment strength due to including the axial load is always greater than this.

The following is an example calculation for Example 1A.

$$P_u = (0.9 - 0.2S_{DS})D = \big(0.9 - 0.2(1.32)\big)(46psf)(8ft) = 234\frac{lb}{ft} = 0.234\frac{k}{ft}$$

$$A_s = \frac{0.44in.^2}{40in.}\frac{12in.}{ft} = 0.132\frac{in.^2}{ft}$$

$$c = \frac{A_s f_y + P_u}{0.64 f'_m b} = \frac{0.132\frac{in.^2}{ft}(60ksi) + 0.234\frac{k}{ft}}{0.64(2ksi)\left(12\frac{in.}{ft}\right)} = 0.531\ in.$$

$$n = \frac{E_s}{E_m} = \frac{E_s}{900 f'_m} = \frac{29000ksi}{900(2ksi)} = \frac{29000ksi}{1800ksi} = 16.11$$

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

$$= 16.11\left(0.132\frac{in.^2}{ft} + \frac{0.234\frac{k}{ft}}{60ksi}\frac{7.625in.}{2(3.812in.)}\right)(3.812in. - 0.531in.)^2 + \frac{12\frac{in.}{ft}(0.531in.)^3}{3} = 24.2\frac{in.^4}{ft}$$

$I_n = 336.7\frac{in.^4}{ft}$ from NCMA TEK 14-1B Section Properties of Concrete Masonry Walls

Determine modulus of rupture using interpolation between 153 psi (full grout) and 51 psi (ungrouted).

$$f_r = 153psi\frac{1\ cell\ grouted}{5\ cells} + 51psi\frac{4\ cells\ ungrouted}{5\ cells} = 71.4\ psi$$

$A_n = 42.8\frac{in.^2}{ft}$ from NCMA TEK 14-1B Section Properties of Concrete Masonry Walls

$$M_{cr} = \frac{(P_u/A_n + f_r)I_n}{t_{sp}/2} = \frac{\left(0.234\frac{k}{ft}/42.8\frac{in.^2}{ft}+0.0714ksi\right)336.7\frac{in.^4}{ft}}{7.625in./2}\frac{1ft}{12in.} = 0.566\frac{k{\cdot}ft}{ft}$$

$$w_u = 1.6S_{DS}W_pI_P = 1.6(1.32)(46psf)(1.0) = 97.2psf = 0.0972\ ksf$$

Solve for $M_u$ from $M_u\left[1-\frac{P_u}{2}\frac{11}{40}\frac{h^2}{E_mI_{cr}}\right] = \frac{1}{3}w_uh^2 + \frac{P_u}{2}\frac{11}{40}\frac{M_{cr}h^2}{E_m}\left(\frac{1}{I_n}-\frac{1}{I_{cr}}\right)$

$$\left[1-\frac{P_u}{2}\frac{11}{40}\frac{h^2}{E_mI_{cr}}\right] = \left[1-\frac{0.234\frac{k}{ft}}{2}\frac{11}{40}\frac{(96in.)^2}{(1800ksi)24.2\frac{in.^4}{ft}}\right] = 0.9932$$

$$\frac{1}{3}w_uh^2 + \frac{P_u}{2}\frac{11}{40}\frac{M_{cr}h^2}{E_m}\left(\frac{1}{I_n}-\frac{1}{I_{cr}}\right)$$

$$= \frac{1}{3}(0.0972ksf)(8ft)^2 + \frac{0.234\frac{k}{ft}}{2}\frac{11}{40}\frac{0.566\frac{k{\cdot}ft}{ft}(96in.)^2}{1800ksi}\left(\frac{1}{336.7\frac{in.^4}{ft}}-\frac{1}{24.2\frac{in.^4}{ft}}\right) = 2.069\frac{k{\cdot}ft}{ft}$$

$0.993M_u = 2.069\frac{k{\cdot}ft}{ft}$ or $M_u = \frac{2.069\frac{k{\cdot}ft}{ft}}{0.993} = 2.083\frac{k{\cdot}ft}{ft}$

## <u>Appendix D: Inclusion of Wall Weight</u>

The above calculations neglect the small axial force from the wall weight. This is a reasonable approximation for design, but is slightly conservative. The effect of including the wall weight is examined in this appendix.

The factored axial force per ASCE 7-16 Section 2.3.6 would be $P_u = (0.9 - 0.2S_{DS})D$.

The following table shows the design moment strength ignoring the axial load from wall weight, the design moment strength including the axial load from wall weight, and the increase in design moment strength when including the axial load from the wall weight for each of the example problems.

| Example | Design Moment, $\phi M_n$ (kip-ft/ft) | | Ratio |
|---|---|---|---|
| | Ignoring Axial Load | Including Axial Load | |
| 1A: 8 in. CMU; #6 @ 40 in. centered | 2.14 | 2.21 | 1.031 |
| 1B: 8 in. CMU; #7 @ 32 in. centered | 3.50 | 3.58 | 1.023 |
| 1C: 10 in. CMU; #6 @ 32 in. two layers | 6.38 | 6.52 | 1.022 |
| 1D: 12 in. CMU; #7 @ 32 in. two layers | 10.05 | 10.22 | 1.016 |
| 2A: 8 in. CMU; #4 @ 48 in. centered | 0.84 | 0.93 | 1.102 |
| 2B: 8 in. CMU; #5 @ 48 in. centered | 1.29 | 1.39 | 1.082 |
| 2C: 8 in. CMU; #6 @ 48 in. centered | 1.80 | 1.92 | 1.067 |
| 2D: 8 in. CMU; #7 @ 40 in. centered | 2.86 | 2.98 | 1.042 |

For all examples, the increase in the design moment strength including axial load is greater than the increase in factored moment including second-order effects. Thus, ignoring second-order effects, but also ignoring the wall weight in determining the nominal moment capacity, results in a very reasonable, and just slightly conservative, design.

The following provides formulas for determining the design moment strength. Numerical values are for no axial load, as the axial load is height dependent.

A. Fully grouted wall or a partially grouted wall with the equivalent rectangular stress block in the face shell and centered reinforcement. 8 in. CMU with #6 @ 48 in.

$$a = \frac{A_s f_y + P_u/\phi}{0.8 f'_m b} = \frac{0.11\frac{in.^2}{ft}(60ksi)}{0.8(2ksi)\left(12\frac{in.}{ft}\right)} = 0.344\ in.$$

This is less than the face shell thickness of 1.25 in.

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

$$= 0.9\left(0.11\frac{in.^2}{ft}\right)(60ksi)\left(3.812in. - \frac{0.344in.}{2}\right) = 21.6\frac{k\cdot in.}{ft} = 1.80\frac{k\cdot ft}{ft}$$

B. Partially grouted wall with the equivalent rectangular stress block in the web. 10 in. CMU with #7 @ 16 in.

Masonry force in face shell: $C_{m,fs} = 0.8 f'_m t_{fs} b = 0.8(2ksi)(1.25in.)\left(12\frac{in.}{ft}\right) = 24\frac{kip}{ft}$

Masonry force in web: $C_{m,web} = A_s f_y + P_u/\phi - C_{m,fs} = 0.45\frac{in.^2}{ft}(60ksi) - 24\frac{kip}{ft} = 3.0\frac{kip}{ft}$

Depth of stress block: $a = \frac{C_{m,web}}{0.8 f'_m b_w} + t_{fs} = \frac{3.0\frac{kip}{ft}}{0.8(2ksi)\left(6.0\frac{in.}{ft}\right)} + 1.25in. = 1.562\ in.$

$$\phi M_n = \phi\left[C_{m,fs}\left(d - \frac{t_{fs}}{2}\right) + C_{m,web}\left(d - t_{fs} - \frac{a - t_{fs}}{2}\right)\right]$$

$$= 0.9\left[24\frac{kip}{ft}\left(4.812in. - \frac{1.25in.}{2}\right) + 3.0\frac{kip}{ft}\left(4.812in. - 1.25in. - \frac{1.562in. - 1.25in.}{2}\right)\right]$$

$$= 99.6\frac{k\cdot in.}{ft} = 8.30\frac{k\cdot ft}{ft}$$

If the wall were fully grouted, the design moment would only increase to $8.32\frac{k\cdot ft}{ft}$, or an insignificant change. Partial grouting reduced the weight from 106 psf to 74 psf, a 30% decrease in weight and load, for 125 pcf units.

C. Fully grouted wall or a partially grouted wall with the equivalent rectangular stress block in the face shell and two layers of reinforcement. 10 in. CMU with #6 @ 48 in.

   i. If the distance to the layer of steel nearest the compression face, $d'$, is as follows than the second layer of reinforcement has yielded.

   ii. $d' > \left(\frac{\varepsilon_y}{\varepsilon_{mu}} + 1\right)\frac{a}{0.8} = \left(\frac{0.00207}{0.0025} + 1\right)\frac{a}{0.8} = 2.28\,a$

$$a = \frac{A_s f_y + A_s' f_y + P_u/\phi}{0.8 f_m' b} = \frac{0.11\frac{in.^2}{ft}(60ksi) + 0.11\frac{in.^2}{ft}(60ksi)}{0.8(2ksi)\left(12\frac{in.}{ft}\right)} = 0.688\,in.$$

Compressive stress block is in face shell and

$$d' = 2.5in. > 2.28a = 2.28(0.688in.) = 1.57\,in.$$

Therefore second layer of steel has yielded.

$$\phi M_n = \phi\left[0.8 f_m' ab\left(\frac{t_{sp}}{2} - \frac{a}{2}\right) + A_s f_y\left(d - \frac{t_{sp}}{2}\right) - A_s' f_y\left(\frac{t_{sp}}{2} - d'\right)\right]$$

If the reinforcement is symmetric:

$$\phi M_n = \phi\left[0.8 f_m' ab\left(\frac{t_{sp}}{2} - \frac{a}{2}\right)\right]$$
$$= 0.9(0.8)(2ksi)(0.688in.)\left(12\frac{in.}{ft}\right)\left(\frac{9.625in.}{2} - \frac{0.688in.}{2}\right) = 53.1\frac{k\cdot in.}{ft} = 4.42\frac{k\cdot ft}{ft}$$

If the second layer had not been included, the design moment would have been $\phi M_n = 3.44\frac{k\cdot ft}{ft}$, or 22% less.

   iii. If the distance to the layer of steel nearest the compression face, $d'$, is such that $d' < 2.28a$ but $d' > c = a/0.8$ the second layer of steel is in tension but has not yielded. The value of $a$ would be obtained from the following quadratic equation. This is illustrated for 10 in. CMU with #6 @ 24 in.

$$f_m' ba^2 - 1.25\left(A_s f_y + P_u/\phi - A_s' E_s \varepsilon_{mu}\right)a - A_s' E_s \varepsilon_{mu} d' = 0$$

$$2ksi\left(12\frac{in.}{ft}\right)a^2 - 1.25\left(0.22\frac{in.^2}{ft}(60ksi) - \left(0.22\frac{in.^2}{ft}\right)(29000ksi)(0.0025)\right)a -$$
$$\left(0.22\frac{in.^2}{ft}\right)(29000ksi)(0.0025)(2.5in.) = 0$$

$a = 1.219\,in.$ $\qquad c = \frac{a}{0.8} = \frac{1.219in.}{0.8} = 1.524\,in.$

Compressive stress block is in face shell. The stress in the second layer of steel is:

$$f_s' = \frac{d'-c}{c} E_s \varepsilon_{mu} = \frac{2.5in.-1.524in.}{1.524in.} 29000ksi(0.0025) = 46.4\,ksi$$

$$\phi M_n = \phi\left[0.8f_m' ab\left(\frac{t_{sp}}{2} - \frac{a}{2}\right) + A_s f_y\left(d - \frac{t_{sp}}{2}\right) - A_s' f_s'\left(\frac{t_{sp}}{2} - d'\right)\right]$$

$$= 0.9\left[0.8(2ksi)(1.219in.)\left(12\frac{in.}{ft}\right)\left(\frac{9.625in.}{2} - \frac{1.219in.}{2}\right) + \right.$$
$$\left. 0.22\frac{in.^2}{ft}(60ksi)\left(7.125in. - \frac{9.625in.}{2}\right) - 0.22\frac{in.^2}{ft}(46.4ksi)\left(\frac{9.625in.}{2} - 2.5in.\right)\right]$$

$$= 94.8\frac{k\cdot in.}{ft} = 7.90\frac{k\cdot ft}{ft}$$

If the second layer had not been included, the design moment would have been $\phi M_n = 6.71\frac{k\cdot ft}{ft}$, or 15% less.

iii. If $d' > c = a/0.8$, then the second layer of reinforcement is in compression and would not be included. The value of $a$ and the design moment strength would be determined based on just one layer of reinforcement.

D. Partially grouted wall with the equivalent rectangular stress block in the web and two layers of reinforcement. There are three possibilities for the second layer of reinforcement: the reinforcement is in compression in which case it is not considered, the reinforcement is in tension but has not yielded, and the reinforcement is in tension and has yielded. Although closed form equations could be derived for each case, a general equation will be used and it will be solved iteratively. The value of $a$ is obtained from the following equation.

$$0.8f_m' t_{fs} b + 0.8f_m' b_w(a - t_{fs}) = A_s f_y + A_s' \max\left\{0, \min\left\{f_y, \frac{d'-a/0.8}{a/0.8} E_s \varepsilon_{mu}\right\}\right\} + P_u/\phi$$

The design moment is obtained from:

$$\phi M_n = \phi\left[C_{m,fs}\left(\frac{t_{sp}}{2} - \frac{t_{fs}}{2}\right) + C_{m,web}\left(\frac{t_{sp}}{2} - t_{fs} - \frac{a-t_{fs}}{2}\right) 0.8f_m' ab\left(\frac{t_{sp}}{2} - \frac{a}{2}\right) + A_s f_y\left(d - \frac{t_{sp}}{2}\right) - \right.$$
$$\left. A_s' f_s'\left(\frac{t_{sp}}{2} - d'\right)\right]$$

where

$$C_{m,fs} = 0.8f_m' t_{fs} b$$

$$C_{m,web} = A_s f_y + A_s' \max\left\{0, \min\left\{f_y, \frac{d'-a/0.8}{a/0.8} E_s \varepsilon_{mu}\right\}\right\} + P_u/\phi - C_{m,fs}$$

## References

*Building Code Requirements for Masonry Structures*, TMS 402-16, The Masonry Society, 2016.

*Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE 7-16, American Society of Civil Engineers, 2016.

*Design of Reinforced Masonry Structures*, Concrete Masonry Association of California and Nevada, 2015.

*Reinforced Masonry Engineering Handbook*, Masonry Institute of America, 8th edition, 2017.

*Strength Design of Masonry*, The Masonry Society, 2020.

**This issue of Masonry Chronicles was written by:**

***Richard M. Bennett, PhD, PE***, is a professor of Civil and Environmental Engineering, and the Director of Engineering Fundamentals, at the University of Tennessee, Knoxville. He received his PhD from the University of Illinois, Urbana-Champaign. He served as Chairman of the Flexural and Axial Loads Subcommittee of the TMS 402/602 Code Committee from 2004 to 2010. From 2010 to 2013, he was the Vice-Chair of the Main committee, and the Chair of the 2016 TMS 402/602 Committee.

and

***Kurt Siggard***, Executive Director
Concrete Masonry Association of California and Nevada (CMACN)

Yares Residence
Grand Award Winner of the 2017 CMACN Design Awards in Residential Design
o2 architecture
David Blank Photo

Concrete Masonry Association
of California and Nevada
6060 Sunrise Vista Drive, Suite 1990
Citrus Heights, CA 95610
(916) 722-1700
info@cmacn.org
www.cmacn.org

Presort
Standard
U.S. Postage
PAID
Premit No. 604
Sacramento, CA

CHANGE SERVICE REQUESTED

# Why Masonry?

## www.whymasonry.org



## CMACN Producer Members

Producer Members are an individual, partnership, or corporation, which is actively engaged in the manufacture and sale of concrete masonry units (CMUs).

- Air Vol Block, Inc.
- Angelus Block Company, Inc.
- Basalite Concrete Products, LLC
- Calstone Company, Inc.
- CEMEX
- ORCO Block & Hardscape
- RCP Block & Brick, Inc.
- Oldcastle

Please go to *www.cmacn.org* for additional information on the latest design tools, previous “Masonry Chronicles,” additional publications, links to our Members, links to other associated organizations, and much more.