UNREINFORCED MASONRY EXAMPLES

This program is intended as a preliminary design tool for design professionals who are experienced and competent in masonry design. This program is not intended to replace sound engineering knowledge, experience, and judgment. Users of this program must determine the validity of the results. The International Masonry Institute assumes no responsibility for the use or application of this program.

EXAMPLE A:

Given:

- 8 inch medium weight hollow CMU
- Type N masonry cement
- 12 ft vertical span (simple support)
- Design load is 5 psf out-of-plane
- Seismic Design Category A
- Use the 2012 IBC

Required: Determine if the wall is adequate.

Solution:
Load Combination A (0.6D + w_L) will control.
The maximum moment will occur at mid-height of the wall.

$$M = \frac{w_L h^2}{8} = \frac{5 psf (12 ft)^2}{8} = 90 ft - lb / ft$$

The axial force (wall weight is 36 psf) is:

$$P = 0.6 w_{wall} \frac{h}{2} = 0.6 (36 psf) \frac{12 ft}{2} = 129.6 lb / ft$$

Calculate the flexural tension stress.

$$-\frac{P}{A_n} + \frac{12M}{S_n} = -\frac{129.6 lb / ft}{30.0 in^2 / ft} + \frac{12(90 ft - lb / ft)}{81.0 in^3 / ft} = 9.01 psi$$

The allowable stress, F_t, is 12 psi, so the wall is adequate.

If the 2009 IBC had been used, the allowable flexural tension stress is 9 psi, so the wall would not be adequate. Many designers would say that 9.01 psi is close enough to 9 psi that the wall would be OK. The program rounds to the nearest 0.1 psi, so 9.01 psi would round to 9.0 psi, and the program would indicate the wall is adequate. If Type S masonry cement were used, the allowable flexural tension stress would be 15 psi, and the wall would be adequate.

The reaction at the top of the wall, R_{top}, is:

$$R_{top} = \frac{w_L h}{2} = \frac{5 psf (12 ft)}{2} = 30 lb / ft$$

A sufficient anchorage would need to be provided at the top of the wall to carry this out-of-plane reaction force.

EXAMPLE B:

Given:

- 8 inch medium weight hollow CMU
- Type N masonry cement
- 12 ft vertical span (simple support)
- Design load is a horizontal load of 50 lb/ft at a height of 3'-6" from the floor
- Seismic Design Category A
- Use the 2012 IBC

Required: Determine if the wall is adequate.

Solution:

Load Combination D (0.6D + H_L) will control.

The maximum moment will occur at the location of the horizontal load.

$$M = H_L \frac{h_L}{h}(h - h_L) = 50 lb/ft \frac{3.5 ft}{12 ft}(12 ft - 3.5 ft) = 124.0 ft - lb/ft$$

The axial force (wall weight is 36 psf) is:

$$P = 0.6 w_{wall}(h - h_L) = 0.6(36 psf)(12 ft - 3.5 ft) = 183.6 lb/ft$$

Calculate the flexural tension stress.

$$-\frac{P}{A_n} + \frac{12M}{S_n} = -\frac{183.6 lb/ft}{30.0 in^2/ft} + \frac{12(124 ft - lb/ft)}{81.0 in^3/ft} = 12.2 psi$$

The allowable stress, F_t, is 12 psi, so the wall is not adequate.

Using a Type S masonry cement would increase the allowable stress to 20 psi, and the wall would be OK.

The 50 lb/ft at 3'-6" above the floor is a typical handrail load. The IBC and ASCE 7 are unclear as to whether a handrail load should be combined with a 5 psf out-of-plane load. Our interpretation is that the 5 psf out-of-plane load is a minimum load, and that it does not need to be combined with the handrail load.

The reaction at the top of the wall, R_{top}, is:

$$R_{top} = \frac{H_L h_L}{h} = \frac{50 lb/ft(3.5 ft)}{12 ft} = 14.6 lb/ft$$

This force is smaller than the 30 lb/ft anchorage force from a 5 psf uniform lateral load (see Example A). A sufficient anchorage would need to be provided at the top of the wall to carry 30 lb/ft.

EXAMPLE C:

Given:

- 8 inch medium weight hollow CMU
- Type N masonry cement
- 12 ft vertical span (simple support)
- Design load is a vertical load of 40 lb/ft at an eccentricity of 4 inches outside the wall
- Seismic Design Category A
- Use the 2012 IBC

Required: Determine if the wall is adequate.

Solution:
Load Combination B (0.6D + P_L) and C (D + P_L) will be checked.
The maximum moment will be Pe.

$$M = Pe = 40lb/ft\left(\frac{7.625in}{2} + 4in\right)\frac{1ft}{12in} = 26.0\,ft - lb/ft$$

The eccentric load is assumed to occur at the top of the wall; hence the wall weight is 0. The axial force will just be the applied vertical load.
Calculate the flexural tension stress.

$$-\frac{P}{A_n} + \frac{12M}{S_n} = -\frac{40lb/ft}{30.0in^2/ft} + \frac{12(26\,ft - lb/ft)}{81.0in^3/ft} = 2.5\,psi$$

The allowable stress, F_t, is 12 psi, so the wall is not adequate.

Although tension almost always controls with unreinforced masonry, the unity equation will be checked for completeness. The value of r is 3.21 in., making h/r = 144in/3.21in. = 44.8

$$f_a = \frac{P}{A_n} = \frac{40lb/ft}{30.0in^2\,ft} = 1.3\,psi$$

$$F_a = \frac{f'_m}{4}\left(1 - 140\left(\frac{h}{r}\right)^2\right) = \frac{1350\,psi}{4}\left(1 - 140\left(\frac{144in.}{3.21in.}\right)^2\right) = 302.8\,psi$$

$$f_b = \frac{12M}{S_n} = \frac{12(26\,ft - lb/ft)}{81.0in^3\,ft} = 3.8\,psi$$

$$F_b = \frac{f'_m}{3} = \frac{1350\,psi}{3} = 450\,psi$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{1.3\,psi}{302.8\,psi} + \frac{3.8\,psi}{450\,psi} = 0.013 \le 1.0 \qquad \text{OK}$$

The TMS 402 Code also requires that when using allowable stress design the following equations be checked:

$$P \le (1/4)P_e$$

$$P_e = \frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3$$

For this wall, $r = \sqrt{I/A}$=3.21in. With e = 7.81in., 1-0.577e/r = -0.405. Thus, the wall does not meet this code requirement. However, given the relatively light load, many designers would accept the wall as adequate.

The eccentric vertical load is typical of a large monitor that is attached to the wall. The IBC and ASCE 7 are unclear as to whether this load should be combined with a 5 psf out-of-plane load. Our interpretation is that the 5 psf out-of-plane load is a minimum load, and that it does not need to be combined with the handrail load. The 5 psf out-of-plane load should also be checked (see Example A), and would control in this case.

EXAMPLE D:

Given:

- 8 inch medium weight hollow CMU
- Type N masonry cement
- 12 ft horizontal span (simple support)
- Design load is 5 psf out-of-plane
- Seismic Design Category A
- Use the 2012 IBC

Required: Determine if the wall is adequate.

Solution:

Load Combination A (0.6D + w_L) will control.

The maximum moment will occur at mid-length of the wall (h is used for length).

$$M = \frac{w_L h^2}{8} = \frac{5psf\left(12\,ft\right)^2}{8} = 90\,ft - lb/\,ft$$

There is no axial force on the wall.

Calculate the flexural tension stress.

$$\frac{12M}{S_n} = \frac{12\left(90\,ft - lb/\,ft\right)}{81.0in^3/\,ft} = 13.3psi$$

The allowable stress, F_t, is 25 psi, so the wall is adequate.

By trial and error changing of the wall length, it can be determined that the maximum horizontal span for this wall is 16 ft for a 5 psf out-of-plane load.

EXAMPLE E:

<u>Given:</u>

- 8 inch medium weight hollow CMU
- Type N masonry cement
- 5 ft high cantilever
- Design load is 5 psf out-of-plane
- Seismic Design Category A
- Use the 2012 IBC

<u>Required:</u> Determine if the wall is adequate.

<u>Solution:</u>
Load Combination A (0.6D + w_L) will control.
The maximum moment will occur at the base of the wall.

$$M = \frac{w_L h^2}{2} = \frac{5psf\,(5\,ft)^2}{2} = 62.5\,ft - lb/\,ft$$

The axial force (wall weight is 36 psf) is:

$$P = 0.6 w_{wall} h = 0.6(36\,psf)5\,ft = 108lb/\,ft$$

Calculate the flexural tension stress.

$$-\frac{P}{A_n} + \frac{12M}{S_n} = -\frac{108lb/\,ft}{30.0in^2/\,ft} + \frac{12(62.5\,ft - lb/\,ft)}{81.0in^3/\,ft} = 5.66\,psi$$

The allowable stress, F_t, is 12 psi, so the wall is adequate.

EXAMPLE F:

Given:
- 8 inch medium weight hollow CMU
- Type N Portland cement lime mortar
- 12 ft vertical span (simple support)
- Minimum design load is 5 psf out-of-plane
- S_{DS} = 0.5; Seismic Design Category C
- Use the 2012 IBC

Required: Determine if the wall is adequate.

Solution:
Check Load Combination G (0.6D + 0.7E). Wall weight is 36 psf, and R = 1.5 for unreinforced masonry.

The seismic load is $w_E = \dfrac{1.2S_{DS}W_p}{\left(\dfrac{R_p}{I_p}\right)} = \dfrac{1.2(0.5)36\,psf}{\left(\dfrac{1.5}{1.0}\right)} = 14.4\,psf$

Since 0.7(14.4psf) = 10.1 psf, seismic will control.
The maximum moment will occur at mid-height of the wall.

$$M = \frac{0.7w_E h^2}{8} = \frac{0.7(14.4\,psf)(12\,ft)^2}{8} = 181\,ft-lb/\,ft$$

The axial force is:

$$P = [0.6 - 0.7(0.2)(S_{DS})]w_{wall}\frac{h}{2} = [0.6 - 0.7(0.2)(0.5)](36\,psf)\frac{12\,ft}{2} = 114.5lb/\,ft$$

Calculate the flexural tension stress.

$$-\frac{P}{A_n} + \frac{12M}{S_n} = -\frac{114.5lb/\,ft}{30.0in^2/\,ft} + \frac{12(181\,ft-lb/\,ft)}{81.0in^3/\,ft} = 23.1psi$$

The allowable stress, F_t, is 25 psi, so the wall is adequate.

Partition walls in SDC are required to have prescriptive seismic reinforcement in either the horizontal OR vertical direction in accordance with the following:

(a) Horizontal reinforcement — Two longitudinal wires of W1.7 (9 gage) bed joint reinforcement spaced not more than 16 in. on center, or No. 4 bars spaced not more than 48 in. on center. Horizontal reinforcement needs to be provided within 16 in. of the top and bottom of the wall.

(b) Vertical reinforcement —No. 4 bars spaced not more than 120 in. on center. Vertical reinforcement needs to be provided within 16 in. of the ends of the wall.

It is recommended that W1.7 (9 gage) bed joint reinforcement at 16 in. (every other course) be provided to meet the prescriptive seismic requirements.

Due to the permitted one-third stress increase in the 2009 IBC (2008 MSJC), the allowable flexural tension stress would be the same, and this wall would be adequate using the 2009 IBC.

The reaction at the top of wall, R_t, is:

$$R_t = 0.7 w_E \frac{h}{2} = 0.7(14.4 psf)\frac{12\,ft}{2} = 60.5 lb/ft$$

Sufficient anchors need to be provided at the top of wall to carry this force.

If this partition wall were part of an egress stairway, the importance factor would be 1.5, which increases the seismic load, w_E, to 21.6 psf, the moment to 272 ft-lb/ft, and the flexural tensile stress to 36.5 psi. Unreinforced, ungrouted masonry will no longer work, even with Type S Portland cement lime mortar.

VERIFICATION OF STRUCTURAL ANALYSIS

The following examples are verification of the structural analysis of the simply supported vertical wall under various loading conditions. The loads are not necessarily representative of typical partition loads. The maximum moment and the location of the maximum moment are checked for Load Combination L. All of the other load combinations are subsets of this. For each of the four cases of Load Combination L, the results are compared to a moment diagram.

CASE A: maximum moment occurs below H_L.

h = 12 ft
H_L = 40 lb/ft
h_L = 10 ft
P_L = 100lb/ft
e = 6 in
w_E = 14.4 psf

$$R_b = 0.75\left(\frac{0.7 w_E h}{2} + \frac{H_L(h-h_L)}{h} + \frac{P_L e}{h}\right)$$

$$= 0.75\left(\frac{0.7(14.4 psf)12 ft}{2} + \frac{40 lb/ft(12 ft - 10 ft)}{12 ft} + \frac{100 lb/ft(0.5 ft)}{12 ft}\right) = 53.5 lb/ft$$

Is $R_b < 0.75 \cdot 0.7 w_E h_L$? $0.75(0.7)w_E h_L = 0.75(0.7)(14.4 psf)(10 ft) = 75.6 lb/ft$

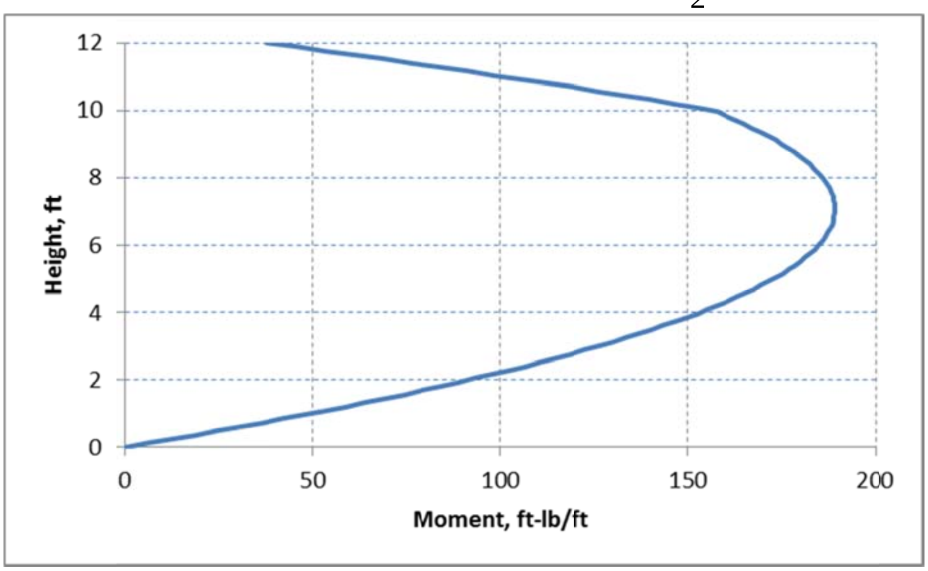
Yes, 53.5lb/ft < 75.6 lb/ft. Case A controls.

$$x = h - \frac{R_b}{0.75(0.7 w_E)} = 12 ft - \frac{53.5 lb/ft}{0.75(0.7)(14.4 psf)} = 4.92 ft$$

Maximum moment occurs at 4.92 ft below top of wall, or 7.08 ft up from the bottom of the wall.

$$M_{max} = R_b(h-x) - 0.75\frac{0.7 w_E (h-x)^2}{2}$$

$$= 53.5 lb/ft(12 ft - 4.92 ft) - 0.75\frac{0.7(14.4 psf)(12 ft - 4.92 ft)^2}{2} = 189 ft-lb/ft$$



CASE B: maximum moment occurs at H_L.

h = 12 ft

H_L = 40 lb/ft

h_L = 5 ft

P_L = 100lb/ft

e = 6 in

w_E = 14.4 psf

$$R_b = 0.75\left(\frac{0.7w_E h}{2} + \frac{H_L(h-h_L)}{h} + \frac{P_L e}{h}\right)$$

$$= 0.75\left(\frac{0.7(14.4\,psf)12\,ft}{2} + \frac{40lb/ft(12\,ft-5\,ft)}{12\,ft} + \frac{100lb/ft(0.5\,ft)}{12\,ft}\right) = 66.0lb/ft$$

Is $0.75*0.7w_Eh_L \le R_b < 0.75(0.7w_Eh_L+H_L)$?

$$0.75(0.7)w_E h_L = 0.75(0.7)(14.4\,psf)(5\,ft) = 37.8lb/ft$$

$$0.75(0.7w_E h_L + H_L) = 0.75(0.7(14.4\,psf)(5\,ft) + 40lb/ft) = 67.8lb/ft$$

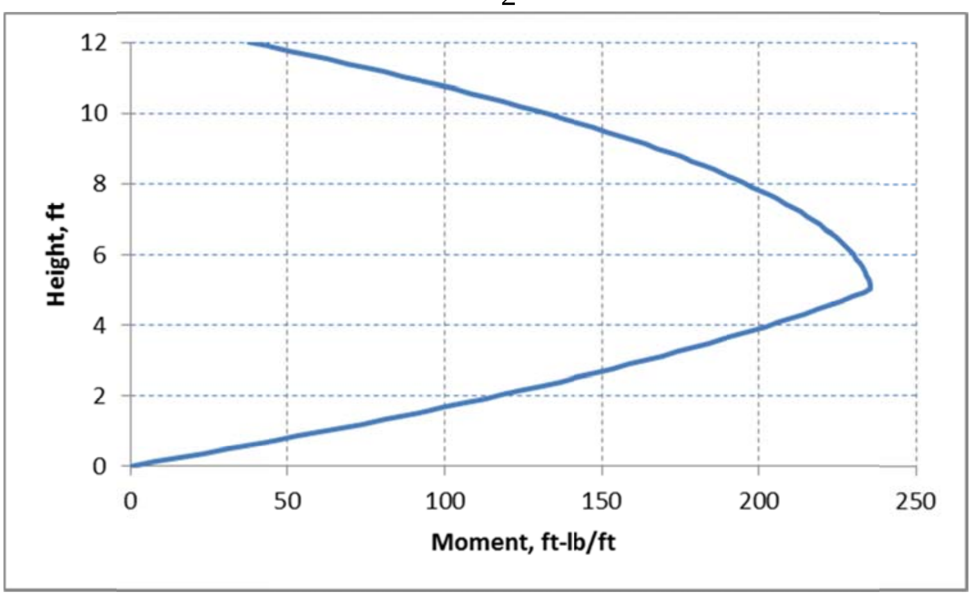
Yes, 37.8lb/ft ≤ 66.0lb/ft < 67.8 lb/ft. Case B controls.

$$x = h - h_L = 12\,ft - 5\,ft = 7\,ft$$

Maximum moment occurs at 7 ft below top of wall, or 5 ft up from the bottom of the wall.

$$M_{\max} = R_b h_L - 0.75\frac{0.7w_E h_L^2}{2}$$

$$= 66.0lb/ft(5\,ft) - 0.75\frac{0.7(14.4\,psf)(5\,ft)^2}{2} = 235\,ft-lb/ft$$



CASE C: maximum moment occurs above H_L, but below top.

h = 12 ft

H_L = 40 lb/ft

h_L = 3 ft

P_L = 100lb/ft

e = 6 in

w_E = 14.4 psf

$$R_b = 0.75\left(\frac{0.7w_E h}{2} + \frac{H_L(h-h_L)}{h} + \frac{P_L e}{h}\right)$$

$$= 0.75\left(\frac{0.7(14.4psf)12ft}{2} + \frac{40lb/ft(12ft-3ft)}{12ft} + \frac{100lb/ft(0.5ft)}{12ft}\right) = 71.0lb/ft$$

Is $0.75(0.7w_E h_L+H_L) < R_b \leq 0.75(0.7w_E h+H_L)$?

$$0.75(0.7w_E h_L + H_L) = 0.75(0.7(14.4psf)(3ft) + 40lb/ft) = 52.7lb/ft$$

$$0.75(0.7w_E h + H_L) = 0.75(0.7(14.4psf)(12ft) + 40lb/ft) = 120.7lb/ft$$

Yes, 52.7lb/ft ≤ 71.0lb/ft < 120.7lb/ft. Case C controls.

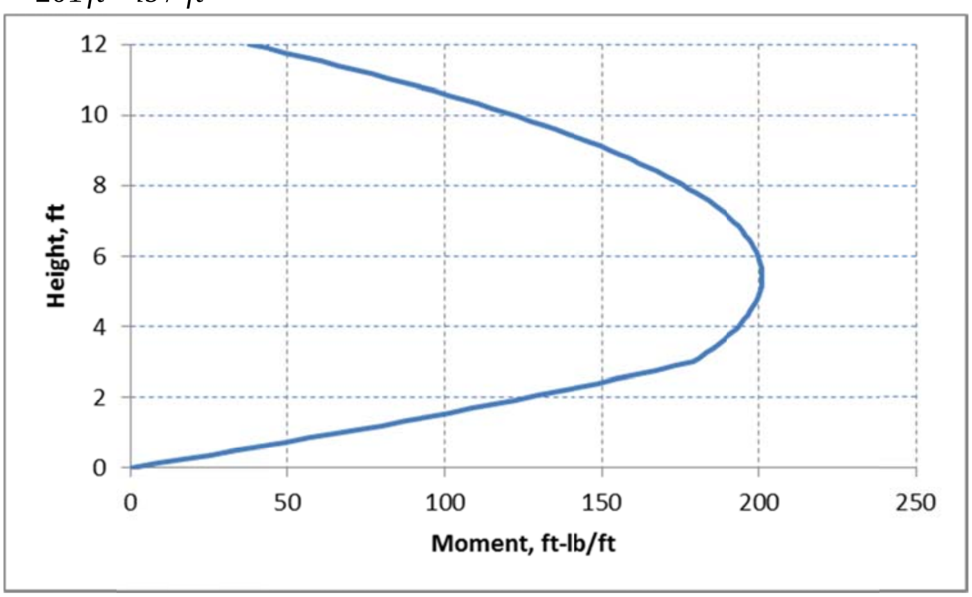
$$x = h - \frac{R_b - 0.75H_L}{0.75(0.7w_E)} = 12ft - \frac{71.0lb/ft - 0.75(40lb/ft)}{0.75(0.7)(14.4psf)} = 6.58ft$$

Maximum moment occurs at 6.58 ft below top of wall, or 5.42 ft up from the bottom of the wall.

$$M_{max} = R_b(h-x) - 0.75\left(\frac{0.7w_E(h-x)^2}{2} + H_L(h-x-h_L)\right)$$

$$= 71.0lb/ft(12ft-6.58ft) - 0.75\left(\frac{0.7(14.4psf)(12ft-6.58ft)^2}{2} + 40lb/ft(12ft-6.58ft-3ft)\right)$$

$$= 201 ft-lb/ft$$



CASE D: maximum moment occurs at top.

h = 12 ft

H_L = 20 lb/ft

h_L = 3 ft

P_L = 200lb/ft

e = 48 in

w_E = 14.4 psf

$$R_b = 0.75\left(\frac{0.7w_E h}{2} + \frac{H_L(h-h_L)}{h} + \frac{P_L e}{h}\right)$$

$$= 0.75\left(\frac{0.7(14.4psf)12ft}{2} + \frac{20lb/ft(12ft-3ft)}{12ft} + \frac{200lb/ft(4ft)}{12ft}\right) = 106.6lb/ft$$

Is $R_b > 0.75(0.7w_E h + H_L)$?

$$0.75(0.7w_E h + H_L) = 0.75(0.7(14.4psf)(12ft) + 20lb/ft) = 105.7lb/ft$$

Yes, 106.6lb/ft > 105.7lb/ft.   Case D controls.   x = 0

Maximum moment occurs at top of wall.

$$M_{\max} = 0.75Pe = 0.75(200lb/ft)(4ft) = 600ft-lb/ft$$