General Shale
Building The American Dream®

Out-of-Plane Wall Design

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. General Shale assumes no responsibility for the use or application of this program.

Codes: This program is based on TMS 402-11/ACI 530-11/ASCE 5-11 Building Code Requirements for Masonry Structures. Loads should be determined using either ASCE 7-05 or ASCE 7-10, Minimum Design Loads for Buildings and Other Structures. Wind loads from ASCE 7-05 are service level wind loads. Wind loads from ASCE 7-10 are strength level wind loads.

Loads: Loads required to be input are the concentric dead load; the eccentric dead, roof live load, live load, and snow load; wind out-of-plane load and uplift; and seismic out-of-plane load and the value of S_{DS}, which is used for determining axial force. A concentric dead load would come, for example, from a parapet. The eccentric loads would typically be from a floor or roof.

The dead load of the wall is taken as 38 psf for ungrouted construction and 56 psf for fully grouted construction. For preliminary design, the ungrouted weight is used for load combinations in which the brick is contributing to the resistance (overturning) and the fully grouted weight is used for the load combinations in which the brick is contributing to the load.

Material Properties: The program is only applicable for Grade 60 reinforcement. Based on Table 1 in TMS 602-11/ACI 530.1-11/ASCE 6-11, a value of f'_m for Type N mortar would be 2440 psi and a value of f'_m for Type S mortar would be 2930 psi based on a unit compressive strength of 8000 psi.

Load Combinations: The following load combinations are checked.
ASCE 7-10 wind loads
(1) 1.4D
(2) 1.2D + 1.6L +0.5(L_r or S)
(3) 1.2D +1.6(L_r or S) + (L or 0.5W)
(4) 1.2D + 1.0W + L + 0.5(L_r or S)
(5) 1.2D + 1.0E +L +0.2S
(6) 0.9D + 1.0W
(7) 0.9D + 1.0E

ASCE 7-05 wind loads
(1) 1.4D

(2) $1.2D + 1.6L + 0.5(L_r \text{ or } S)$
(3) $1.2D + 1.6(L_r \text{ or } S) + (L \text{ or } 0.8W)$
(4) $1.2D + 1.6W + L + 0.5(L_r \text{ or } S)$
(5) $1.2D + E + L + 0.2S$
(6) $0.9D + 1.6W$
(7) $0.9D + E$

The load factor for L in combinations (3), (4), and (5) is conservatively taken as 1.0, although it is permitted to equal 0.5 for all occupancies in which L_o is less than or equal to 100 psf, with the exception of garages or areas occupied as places of public assembly.

The earthquake load, E, is a combination of the vertical and horizontal earthquake load. The vertical earthquake load is $0.2S_{DS}D$.

Wind load: The user has two options for specifying the wind load. One is the user can input the wind load. Three values are required: the out-of-plane wind load (input a negative value for suction; suction generally controls as the moment adds to the moment from eccentric vertical loads), the wind load uplift at the top of the wall from the roof, and the wind pressure at the top of the wall from the roof.

For many common cases, a wind load calculator is provided. A wind speed, exposure category (B or C), roof type (flat, gable, or monoslope), roof pitch (either an angle or pitch), and the roof length parallel to the wind is input. The following assumptions are made:

- The building is enclosed.
- The building is rectangular.
- The roof spans between walls; there is no interior columns. The span is perpendicular to the ridge and the uplift reactions are transferred to the walls parallel to the ridge.
- The uplift is determined as the pressure over half the building times half the length, L. This is an approximation; the reactions should be determined based on windward and leeward pressure, but this gives a good approximation.
- For use with ASCE 7-05, the importance factor, I, is 1.0.
- The topographic factor, K_{zt}, is 1.0.
- Wind pressures are determined from wall zone 4. Although the wind pressures would be higher in corner zone 5, corners of brick buildings generally have support from perpendicular walls giving additional strength to the corners.
- Based on the definition of effective wind area, A, the effective width need not be less than one-third the span length, where the span length is the height of the wall.
- The out-of-plane wind pressure is the negative pressure, or suction, which usually controls since it has a larger magnitude, and also adds to the moment from the eccentric floor/roof load (assuming the eccentricity is towards the interior of the building).

The wind pressure and uplift at the top of the wall are obtained as follows. The direction procedure for the main wind force resisting system is used. Section numbers refer to ASCE 7-10.

A. The mean roof height, *h*, is calculated (26.2).

   a. Flat roofs: $h = h_{wall}$ where h_{wall} = wall height

   b. Monoslope roofs: $h = h_{wall} + \frac{L}{2}\tan(\theta)$

   c. Gable roofs: $h = h_{wall} + \frac{L}{4}\tan(\theta)$

B. The velocity pressure, q_h, is determined based on the following equations(Table 27.3-1 and Table 26.9-1).

   a. Determine K_z

      i. Exposure Category B: $K_z = 2.01\left(\frac{\max(h,15)}{1200}\right)^{\frac{2}{7}}$

      ii. Exposure Category C: $K_z = 2.01\left(\frac{\max(h,15)}{900}\right)^{\frac{2}{9.5}}$

   b. $q_h = 0.00256K_z(0.85)V^2$ where V is the wind speed in mph

C. The roof uplift is determined based on wind parallel to the ridge and using the highest pressure, which is over a horizontal distance from 0 to *h*/2 from the windward edge (Figure 27.4-1). C_p is calculated as:

   a. If $\frac{h}{L} \leq 0.5$ $C_p = -0.9$

   b. If $0.5 < \frac{h}{L} < 1.0$ $C_p = -0.8\frac{h}{L} - 0.5$

   c. If $\frac{h}{L} \geq 1.0$ $C_p = -1.3$

D. The pressure is conservatively determined for $h/L \leq 0.25$ (Figure 27.4-1). This will be the case for most buildings. The pressure differences are small for other values of h/L. The positive values of C_p are determined as follows. Note that the "positive" value is a suction for small angles.

   a. If $\theta \leq 10°$ $C_p = -0.18$

   b. If $10° < \theta \leq 15°$ $C_p = 0.036\theta - 0.54$

   c. If $15° < \theta \leq 20°$ $C_p = 0.04\theta - 0.6$

   d. If $20° < \theta \leq 25°$ $C_p = 0.02\theta - 0.2$

   e. If $25° < \theta \leq 30°$ $C_p = 0.3$

   f. If $30° < \theta \leq 35°$ $C_p = 0.02\theta - 0.3$

   g. If $35° < \theta \leq 45°$ $C_p = 0.4$

   h. If $45° < \theta \leq 60°$ $C_p = \frac{0.04}{3}\theta - 0.2$

   i. If $60° < \theta \leq 80°$ $C_p = 0.01\theta$

   j. If $80° < \theta \leq 90°$ $C_p = 0.8$

E. The uplift forces are calculated as (27.4.2, 26.9.1, Table 26.11-1):

   a. Uplift: $p = q_hG_fC_p - q_hGC_{pi} = q_h(0.85C_p - 0.18)$

   b. Pressure: $p = q_hG_fC_p - q_hGC_{pi} = q_h(0.85C_p + 0.18)$

   c. The force is calculated as $F = p\left(\frac{L}{2}\right)$

   d. The uplift force is used with the load combination 0.9D+W

   e. The pressure force is used with the load combination 1.2D+W

The out-of-plane wind suction is determined as follows. The low rise components and cladding procedures is used (Section 30.4).

A. The effective wind area, *A*, is determined as the wall height multiplied by the effective width (26.2). The effective width need not be less than one-third the wall height. Thus $A = \frac{h_{wall}^2}{3}$

B. The pressure coefficient, GC_p, is determined from ASCE 7-10 Figure 30.4-1. Wind pressures are determined for wall zone 4. Although the wind pressures would be higher in corner zone 5, corners of brick buildings generally have support from perpendicular walls giving additional strength to the corners.

   a. If $A \leq 10,\ GC_p = -1.1$
   b. If $10 < A < 500,\ GC_p = 0.1766 \log_{10}(A) - 1.276$
   c. If $A \geq 500,\ GC_p = -0.8$
   d. If $\theta \leq 10°$ reduce GC_p by 10%. Only take reduction if θ is known.

C. The pressure is obtained as $p = q_h[(GC_p) - (GC_{pi})]$ where the value of GC_{pi}=0.18 (30.4.2).

The wind speed at a given site can be determined from either the maps in ASCE 7, or using the web site at http://www.atcouncil.org/windspeed/. The web site allows users to pick a location on a map of the United States, and it will then return the design wind speed for both ASCE 7-05 and ASCE 7-10.

Earthquake load: The user has the option of selecting the seismic design category if the structure is located in Seismic Design Category A, B, or C. If the Seismic Design Category is chosen, the maximum value of S_{DS} for that particular seismic design category is used per Table 11.6-1 of ASCE 7-05 and ASCE 7-10, and an Occupancy III category (ASCE 7-05) or a Risk Category III (ASCE 7-10). The maximum values are S_{DS} = 0.167 for Seismic Design Category A, S_{DS} = 0.33 for Seismic Design Category B, and S_{DS} = 0.50 for Seismic Design Category C.

The out-of-plane seismic load is determined as $0.4S_{DS}$ multiplied by the weight of the wall, with a minimum force of 10% of the wall weight (ASCE 7-05 and ASCE 7-10, Section 12.11.1). For any value of S_{DS} less than 0.25, the 10 % minimum will control. The wall is conservatively assumed to be fully grouted for determination of the wall weight.

There is no upper bound for S_{DS} for Seismic Design Category D. The user will need to determine the value of S_{DS} based on the location of the structure. An application can be downloaded from http://earthquake.usgs.gov/hazards/designmaps/javacalc.php which allows the user to determine the value of S_{DS} based on latitude/longitude location or the zip code of the location.

Once the value of S_{DS} is determined, the seismic out-of-plane load can be determined from $0.4S_{DS}$ multiplied by the wall weight. For a conservative estimate of the out-of-plane seismic force, the wall can be assumed to be fully grouted, or the weight of the wall is 56 psf.

Example: Consider a structure near the corner of North Main and E Poplar in Collierville, TN (suburb of Memphis). Using Google Earth, the latitude is 35°02’44” N and the longitude is 89°39’46” W. These are converted to decimal values: 35+02/60+44/3600 = 35.05 and 89+39/60+46/3600 = 89.66. These values are input into the Java program from the USGS (see link above), with the longitude being entered as a negative value, -89.66. A soil site class of D is chosen. The Java program gives S_{DS} = 0.685. Using a conservative wall weight of 56 psf, the out-of-plane load is 0.4(0.685)(56psf) = 15.3 psf.

Determination of maximum moment: The wall is assumed to be simply supported. The maximum moment in the wall is determined as follows:

$$M_u = \begin{cases} \dfrac{P_{uf}e}{2} + \dfrac{w_u h^2}{8} + \dfrac{(P_{uf}e)^2}{2w_u h^2} & \text{if x} > 0 \\ \\ P_{uf}e & \text{if x} \le 0 \end{cases}$$

where M_u is the maximum moment, P_{uf} is the load acting at the top of the wall, e is the eccentricity, w_u is the out-of-plane load, and h is the height of the wall. The location of the maximum moment is

$$x = \max\left(\frac{h}{2} - \frac{P_{uf}e}{w_u h}, 0\right)$$

where x is measured down from the top of the wall. The location of the maximum moment is used to find the axial load at this location, including the wall weight.

Design Basis: The required area of reinforcement is determined using the strength design provisions of Chapter 3 of the TMS 402 Code. The moment is increased by 10% to account for second-order (P-δ) effects. The Code also requires a deflection check, which is not made for preliminary design. Often deflections do not control, although they would need to be checked for the final design.

The reinforcement is assumed to be centered in the wall, so d = 2.812 inches. The required area of reinforcement is determined as:

$$a = d - \sqrt{d^2 - \frac{2[P_u(d - t/2) + M_u]}{\phi(0.8 f'_m b)}}$$

$$A_s = \frac{0.8 f'_m ba - P_u / \phi}{f_y}$$

Two axial load limits need to be checked. The first is Code Equation (3-25):

$$\left(\frac{P_u}{A_g}\right) \le 0.20 f'_m$$

If this is exceeded, the wall is not permitted by the TMS 402 Code. The second limit is that when the ratio of effective height to nominal thickness, h/t, exceeds 30, the factored axial stress shall not exceed $0.05f'_m$. For purposes of this calculation, only face shell bedding is assumed for preliminary design.

<u>Verification Problem:</u> Design a wall for the following loads and parameters:
Wall height = 12 ft
Concentric Dead Load: 200 lb/ft
Eccentric Dead Load: 300 lb/ft
Eccentricity = 1.5 inches
Live Load: 0 lb/ft (one-story building)
Roof Live Load: 250 lb/ft
Snow Load: 150 lb/ft
Wind Uplift: 100 lb/ft
Wind Out-of-Plane: 30 psf (determined using ASCE 7-10)
Earthquake Out-of-Plane: 10 psf
S_{DS} = 0.4
Assume that Type S mortar is used; f'_m is taken as 2930 psi.

Determine the factored axial load and factored moment for each load combination, as shown in the following table.

Load Combination	**x (inches)**	**P_u (lb/ft)**	**M_u (lb-in/ft)**
(1) 1.4D	0	1.4(200+300) = 700	1.4(300)(1.5) = 630
(2) 1.2D + 1.6L +0.5(L_r or S)	0	1.2(500) + 1.6(0) + 0.5max(250, 150) = 725	[1.2(300) + 0.5max(250, 150)]1.5 = 728
(3a) 1.2D +1.6(L_r or S) + L	0	1.2(500) + 1.6max(250, 150) + 0 = 1000	[1.2(300) + 1.6max(250,150) + 0]1.5 = 1140
(3b) 1.2D +1.6(L_r or S) + 0.5W	144/2 – 1140/(0.5(30)*144)*12 = 65.7	P_{uf} = 1.2(500) + 1.6max(250, 150) + 0= 1000 P_u = 1000 + 1.2(56)(65.7)/12) = 1368	$P_{uf}e$ = [1.2(300) + 1.6max(250, 150) + 0]1.5 = 1140 M_u = 1140/2 + 0.5(30)(144)^2/8*(1/12) + (1140)^2/[2(0.5)(30)(144)^2]*(12) = 3835
(4) 1.2D + 1.0W + L + 0.5(L_r or S)	144/2 – 728/(30*144)*12 = 70.0	P_{uf} = 1.2(500) + 0 + 0 + 0.5max(250, 150) = 725 P_u = 725 + 1.2(56)(70.0)/12) = 1117	$P_{uf}e$ = [1.2(300)+ 0 + 0 + 0.5max(250,150)]1.5 = 728 M_u = 728/2 + 30(144)^2/8*(1/12) + (728)^2/[2(30)(144)^2]*(12) = 6849
(5) 1.2D + E + L + 0.2S	144/2 – 645/(10*144)*12 = 66.6	P_{uf} = [1.2 +0.2(0.4)]500 + 0 + 0.2(150) = 670 P_u = 670 + 1.28(56)(66.6)/12) = 1068	$P_{uf}e$ = [1.2(300) + 0.2(0.4)(500) + 0.2(150)]1.5 = 645 M_u = 645/2 + 10(144)^2/8*(1/12) + (645)^2/[2(10)(144)^2]*(12) = 2494

(6) 0.9D + 1.0W	144/2 – 255/(30*144)*12 = 71.3	P_{uf} = 0.9(500) - 100= 350 P_u = 350 + 0.9(38)(71.3)/12) = 553	$P_{uf}e$ = [0.9(300)-100]1.5 = 255 M_u = 255/2 + 30(144)^2/8*(1/12) + (255)^2/[2(30)(144)^2]*(12) = 6608
(7) 0.9D + E	144/2 – 345/(10*144)*12 = 69.1	P_{uf} = [0.9 - 0.2(0.4)]500 = 410 P_u = 410 + 0.82(38)(69.1/12) = 589	$P_{uf}e$ = [0.9(300)-0.2(0.4)(500)]1.5 = 345 M_u = 345/2 + 10(144)^2/8*(1/12) + (345)^2/[2(10)(144)^2]*(12) = 2336

For load combinations 3b and 4, no axial wind load was considered. This load combination is for an additive wind load, or a wind pressure on the roof, which is taken as 0.

Detailed calculations are shown for load combinations (3a), (5), and (6) in the following.

Load Combination (3a): With no out-of-plane distributed load, the maximum moment is at the top of the wall, or *x*=0.

$$\begin{aligned}P_u &= 1.2D + 1.6(L_r \ or \ S) + L \\ &= 1.2(200lb/ft + 300lb/ft) + 1.6\max(250lb/ft, 150lb/ft) + 1.0(0lb/ft) \\ &= 1000lb/ft\end{aligned}$$

$$\begin{aligned}\boldsymbol{M_u} &= \boldsymbol{(1.2D + 1.6(L_r \ or \ S) + L)e} \\ &= \boldsymbol{\left[1.2(300lb/ft) + 1.6\max(250lb/ft, 150lb/ft) + 1.0(0lb/ft)\right]1.5in} \\ &= \boldsymbol{728lb - in/ft}\end{aligned}$$

Load Combination (5): This load combination illustrates the inclusion of earthquake loads.

$$\begin{aligned}P_{uf} &= 1.2D + E + L + 0.2S = (1.2 + 0.2S_{DS})D + L + 0.2S \\ &= (1.2 + 0.2(0.4))(200lb/ft + 300lb/ft) + 1.0(0lb/ft) + 0.2(150lb/ft) \\ &= 670lb/ft\end{aligned}$$

$$\begin{aligned}\boldsymbol{P_{uf}e} &= \boldsymbol{(1.2D + E + L + 0.2S)e = \left[1.2D_{ecc} + 0.2S_{DS}(D_{conc} + D_{ecc}) + L + 0.2S\right]e} \\ &= \boldsymbol{\left[1.2(300lb/ft) + 0.2(0.4)(200lb/ft + 300lb/ft) + 1.0(0lb/ft) + 0.2(150lb/ft)\right]1.5in} \\ &= \boldsymbol{645in - lb/ft}\end{aligned}$$

$$x = \max\left(\frac{h}{2} - \frac{P_{uf}e}{w_u h}, 0\right) = \max\left(\frac{144in}{2} - \frac{645in - lb/ft}{10lb/ft^2(144in)}\frac{12in}{ft}, 0\right) = 66.6in$$

$$P_u = P_{uf} + (1.2 + 0.2S_{DS})D_{wall}$$
$$= 670lb/ft + (1.2 + 0.2(0.4))(56psf)(66.6in)\frac{1ft}{12in}$$
$$= 1068lb/ft$$

$$M_u = \frac{P_{uf}e}{2} + \frac{w_u h^2}{8} + \frac{(P_{uf}e)^2}{2w_u h^2}$$
$$= \frac{645in-lb/ft}{2} + \frac{10lb/ft^2(144in)^2}{8}\frac{1ft}{12in} + \frac{(645lb-in/ft)^2}{2(10lb/ft^2)(144in)^2}\frac{12in}{ft}$$
$$= 2494lb-in/ft$$

Load Combination (6): This load combination illustrates counteracting loads, in which the dead load acts as a resistance.

$$P_{uf} = 0.9D + W = 0.9(200lb/ft + 300lb/ft) - 100lb/ft = 350lb/ft$$

$$P_{uf}e = 0.9D + W = [0.9D + W]e = [0.9(300lb/ft) - 100lb/ft]1.5in = 225in-lb/ft$$

$$x = \max\left(\frac{h}{2} - \frac{P_{uf}e}{w_u h}, 0\right) = \max\left(\frac{144in}{2} - \frac{225in-lb/ft}{30lb/ft^2(144in)}\frac{12in}{ft}, 0\right) = 71.3in$$

$$P_u = P_{uf} + 0.9D_{wall} = 670lb/ft + +(1.2 + 0.2(0.4))(56psf)(66.6in)\frac{1ft}{12in} = 553lb/ft$$

$$M_u = \frac{P_{uf}e}{2} + \frac{w_u h^2}{8} + \frac{(P_{uf}e)^2}{2w_u h^2}$$
$$= \frac{225in-lb/ft}{2} + \frac{30lb/ft^2(144in)^2}{8}\frac{1ft}{12in} + \frac{(225lb-in/ft)^2}{2(30lb/ft^2)(144in)^2}\frac{12in}{ft}$$
$$= 6608lb-in/ft$$

Before the required area of reinforcement is determined, the axial load limits are checked. The maximum factored axial load is 1368 lb/ft. The stress based on the gross area is:

$$\left(\frac{P_u}{A_g}\right) = \left(\frac{1368lb/ft}{5.625in(12in/ft)}\right) = 20.3psi \le 0.20f'_m = 0.20(2930psi) = 586psi$$

OK

Check the stress based on net area, conservatively only considering the face shells.

$$\left(\frac{P_u}{A_n}\right)=\left(\frac{1368lb/ft}{2(1.25in)(12in/ft)}\right)=45.6psi \le 0.05f'_m=0.05(2930psi)=146.5psi$$

OK

If the net stress had been greater than 0.05f'$_m$ the wall would have been limited to a height to nominal thickness ratio, *h*/*t*, of 30. The height limit would be 30(6in.) = 180 in. = 15 ft. The wall would have been acceptable even the stress were greater than 0.05f'$_m$ since the wall is only 12 ft tall, provided the gross stress was not greater than 0.20f'$_m$.

The depth of the equivalent rectangular stress block and the required reinforcement is determined for each load combination. This is summarized in the following table, with a sample calculation given for load combination (6). The factored moments, M_u, are increased by 10% over those calculated in the previous table to account for second-order effects.

Load Combination	P_u (lb/ft)	M_u (lb-in/ft)	a (in)	A_s (in^2/ft)
(1) 1.4D	700	693	0.0098	-0.0084
(2) 1.2D + 1.6L +0.5(L_r or S)	725	801	0.0112	-0.0081
(3a) 1.2D +1.6(L_r or S) + L	1000	1254	0.0177	-0.0102
(3b) 1.2D +1.6(L_r or S) + 0.5W	1368	4218	0.0599	0.0027
(4) 1.2D + 1.0W + L + 0.5(L_r or S)	1117	7534	0.1079	0.0299
(5) 1.2D + E + L +0.2S	1068	2743	0.0388	-0.0016
(6) 0.9D + 1.0W	553	7269	0.1040	0.0385
(7) 0.9D + E	589	2570	0.0363	0.0061

For load combination (6):

$$a=d-\sqrt{d^2-\frac{2[P_u(d-t/2)+M_u]}{\phi(0.8f'_m b)}}$$

$$=2.812in-\sqrt{(2.812in)^2-\frac{2[553lb/ft(2.812in-5.625in/2)+7269lb-in/ft]}{0.9(0.8)(2930lb/in^2)12in/ft}}=0.1040in$$

$$A_s=\frac{0.8f'_m ba-P_u/\phi}{f_y}=\frac{0.8(2930lb/in^2)(12in/ft)(0.1040in)-(553lb/ft)/0.9}{60{,}000lb/in^2}=0.0385in^2/ft$$

where *d* is based on the bars being the center of the wall, or 5.625/2 = 2.812 inches.

Check that the maximum value of *a* is in the face shell. If not, the load is too great for the preliminary design method. In this case, the maximum value of *a* is 0.1079 inches, which is less than the face shell thickness of 1.25 inches, so OK.

The maximum required area of steel is 0.0385in^2/ft. Chapter 3 of the TMS 402 limits the reinforcement to 4% of the core, which is 0.04(3.125)^2=0.39in^2. Therefore, the maximum bar size is a #5. Spacings are determined for #3, #4, and #5 bars.

#3: $s = 6in * floor\left(\frac{0.11in^2}{0.0385in^2/ft}\frac{12in}{ft}\frac{1}{6in}\right) = 30in.$

#4: $s = 6in * floor\left(\frac{0.20in^2}{0.0944in^2/ft}\frac{12in}{ft}\frac{1}{6in}\right) = 60in.$

#5: $s = 6in * floor\left(\frac{0.31in^2}{0.0944in^2/ft}\frac{12in}{ft}\frac{1}{6in}\right) = 96in.$

Use either #3@30in., #4@60in., or #5@96in.