Engineering Notes For Design With Concrete Block Masonry

MASONRY

CHRONICLES

Spring 2010

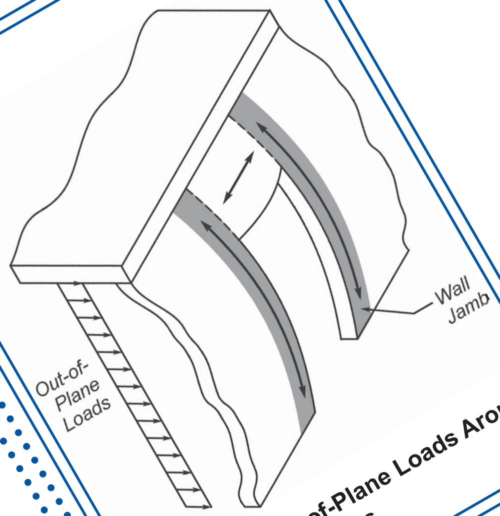

Figure 1 - Out-of-Plane Loads Around Openings

Openings in Concrete Masonry Walls (Part III): Out-of-Plane Forces

Introduction

Previous editions of *"Masonry Chronicles"* discussed the design of concrete masonry around openings to resist in-plane gravity and lateral loads. This edition will discuss the design of masonry around openings to resist out-of-plane lateral loads.

As shown in Figure 1, there are basically two aspects of the design of masonry to resist out-of-plane loads around openings. The first aspect involves the design of the masonry above the opening (and below the opening for windows). The next step is the design of the jamb reinforcement on either side of the opening.

Theoretically, the direction of span of the masonry above an opening is determined by the geometry of the wall, with the primary span being in the shorter direction. However, for reinforced masonry, the engineer can control the behavior of the masonry and decide which direction the masonry spans. This is because if sufficient reinforcement is not provided in one direction, the masonry will crack and resist load in the direction with adequate strength. The engineer can therefore select which direction he or she wants the masonry above the opening to span, and then ensure that the masonry has sufficient strength in that direction. It is generally more convenient for the masonry to span horizontally, because all the reinforcement above the opening can then be used to resist the out-of-plane forces.

The jamb reinforcement on either side of the opening must also be designed to support the reaction from the masonry above and below the opening. A simplifying approach that is often used when designing the jambs is to ignore any reduction in out-of-plane loading due to the opening. Then, the jamb reinforcing is determined using additional load from a tributary width equal to half the width of the opening. This approach is reasonably accurate for wind loading and typically conservative for earthquake loading, since the weight of the door or window in the opening is usually much less that of the masonry assumed in the calculations. A more rigorous procedure, in which the actual load imposed on the jamb by the masonry above the opening is calculated, may be used to reduce the conservatism of the design.

When using strength design to determine the jamb reinforcement, the design must be based on an analysis that includes the influence of cracking on member stiffness, and the effect of deflections on moments and forces (P-Δ effects).The equations included in TMS-402 [1] for incorporating P-Δ effects in the design of wall are valid for the walls with simple supports and uniformly distributed out-of-plane loads. They would therefore be appropriate if the simplified approach described above is used. However, when a more rigorous approach is used, a different approach is required to account for P-Δ effects when designing the jamb reinforcement. This is because the out-of-plane loads on the jamb are not uniformly distributed over the jamb height.

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One approach is to use equations similar to those for the moment magnifier procedure described in ACI 318 [2]. The maximum moment calculated without P-Δ effects, M_u, is magnified to account for P-Δ effects as follows:

$$M_c = \frac{M_u}{1 - \frac{P_u}{0.75P_e}} \tag{1}$$

where P_u is the factored axial load, and P_e is the critical buckling load, which is given by:

$$P_e = \frac{\pi^2 E_m I_{eff}}{H^2} \tag{2}$$

E_m is modulus of elasticity of the concrete masonry and H is the effective height of the wall. The effective moment of inertia, I_{eff}, depends on whether the jamb is cracked or uncracked:

$$\begin{aligned} I_{eff} &= I_g \quad ; \quad M_c \le M_{cr} \\ I_{eff} &= I_{cr} \quad ; \quad M_c > M_{cr} \end{aligned} \tag{3}$$

where M_{cr} is the cracking moment, I_g is the gross moment of inertia and I_{cr} is the cracked moment of inertia, which is given by:

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$

The design of concrete masonry to resist out of plane loads around openings is best illustrated in the following example.

Example

Design the concrete masonry to resist the out-of-plane earthquake loads around the opening shown in Figure 2. The wall is constructed with 12-inch thick (nominal), fully-grouted masonry with a self-weight of 124 psf and a specified compressive strength of 1500 psi. Type S mortar is used and reinforcing steel is Grade 60. The spectral acceleration for short periods, S_{DS}, is equal to 1.46g. The reaction from roof loads is located is located 9.3 inches from the center of the wall.

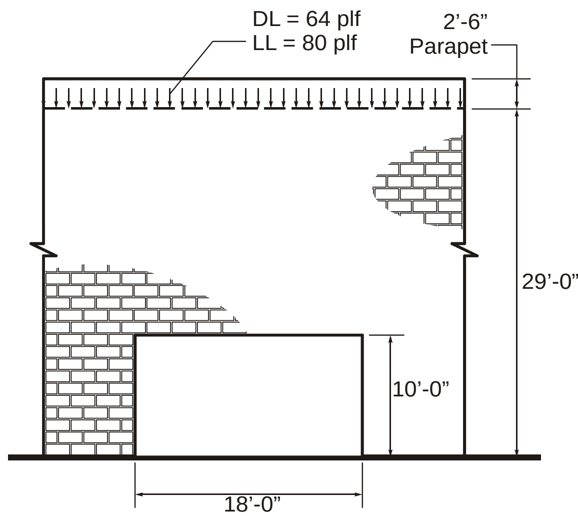

Figure 2 - Example Out of Plane Loads Around Openings

Simplified Solution

In accordance with Section 12.11.1 of ASCE 7 [3], the out of-plane force on the wall is given by:

$$\begin{aligned} F_p &= 0.4S_{DS}IW \\ &= 0.4(1.46)(1.0)(124) = 72.4 \text{ psf} \end{aligned}$$

Conservatively assuming simple supports at the jambs, the moment in the masonry above the opening is equal to:

$$M_u = \frac{72.4(18)^2}{8(1000)} = 2.93 \text{ kip-ft /ft}$$

With two layers of reinforcement we can assume that the effective depth of the reinforcement in the out-of-plane direction is equal to 8.5 inches (the horizontal reinforcement is the inner layer). Then, the required area of steel can be estimated:

$$A_{sreq} \approx \frac{M_u}{\phi f_y 0.9d} = \frac{2.93(12)}{0.9(60)(8.5)} = 0.08 \text{ in}^2/\text{ft}$$

Try two layers of #5 bars at 24 inches (A_s = 0.15 in^2 /ft). Noting that the compression steel is ignored, the nominal moment capacity is equal to:

$$\begin{aligned} M_n &= f_y A_s \left(d - \frac{f_y A_s}{1.6 f'_m b} \right) \\ &= (60)(0.15)\left(8.5 - \frac{60(0.15)}{1.6(1.5)12} \right)\frac{1}{12} \end{aligned}$$

$= 6.14$ kip-ft /ft

$\phi M_n = 0.9(6.14) = 5.52$ kip-ft /ft ...OK

The moment capacity must be greater than 1.3 times the cracking moment. From Table 3.1.8.2.1 of TMS 402-08 (fully-grouted wall with type S mortar, tensile stresses parallel to bed joints) the modulus of rupture is equal to 200 psi. Therefore:

$$1.3M_{cr} = 1.3S_n f_r = \frac{1.3(12)(11.63)^2}{6}\left(\frac{200}{12000}\right)$$

$= 5.86$ kip-ft /ft $< M_n$ ...OK

Note that the reinforcement above the opening must be checked to ensure that it can transfer the in-plane drag/collector forces between the walls on each side of the opening. Additional reinforcing steel must be placed above the opening for the lintel as described in the Winter 2008-09 edition of "*Masonry Chronicles*". Since the distributed roof loads are above the apex of the assumed triangle arch, they do not contribute to the loads on the lintel. In addition, the weight of masonry outside the arch is assumed to be distributed to the sides of the opening by arch action. Therefore, the weight of wall supported by the lintel is:

$$W = 124\left(\frac{1}{2}L \times \frac{L}{2}\right)$$

$$= 124\left(\frac{18^2}{4}\right) = 10044 \text{ lbs}$$

And the maximum moment and shear on the beam are given by:

$$M = \frac{WL}{6} = \frac{10044(18)}{6(1000)} = 30.1 \text{ kip-ft}$$

$$V = \frac{W}{2} = \frac{10044}{2(1000)} = 5.0 \text{ kips}$$

And the factored loads are equal to:

$$M_u = 1.2(30.1) = 36.1 \text{ kip-ft}$$

$$V_u = 1.2(5.0) = 6.0 \text{ kips}$$

Assuming the depth of the lintel to the reinforcement is equal to half the span (108 in, d~104 in) we can try 2-#7 bars ($A_s = 1.2$ in^2). Then:

$$\phi M_n = \phi f_y A_s\left(d - \frac{f_y A_s}{1.6f'_m b}\right)$$

$$= 0.9(60)(1.2)\left(104 - \frac{60(1.2)}{1.6(1.5)11.63}\right)\frac{1}{12}$$

$= 548$ kip-ft $> M_u$ ...OK

The moment strength of the cross-section must be compared with the cracking moment. Cracking is determined by the modulus of rupture of the masonry, which is equal to 200 psi. Then:

$$M_{cr} = S_n f_r$$

$$= \frac{(11.63)(108)^2}{6} 200 \frac{1}{12000}$$

$= 377$ kip-ft

$$1.3M_{cr} = 1.3(377) = 490 \text{ kip-in} < M_n$$ ...OK

From Section 3.3.3.5 of the MSJC code and commentary, the maximum reinforcement ratio is equal to:

$$\rho_{\max} = 0.64\left(\frac{f'_m}{f_y}\right)\left(\frac{\varepsilon_{mu}}{1.5\frac{f_y}{E_s} + \varepsilon_{mu}}\right)$$

$$= 0.64\left(\frac{1.5}{60}\right)\left(\frac{0.0025}{1.5(60)/29000 + 0.0025}\right)$$

$$= 0.0071$$

$$\rho = \frac{A_s}{bd} = \frac{1.2}{11.63(104)} = 0.00099 < \rho_{\max}$$ ...OK

Check the maximum shear strength:

$$V_{n,\max} = 4A_n\sqrt{f'_m} = 4(11.63)(108)\sqrt{1500}$$

$= 195$ kips $> V_u$ ...OK

The shear strength provided by the masonry is given by:

$$\phi V_m = 0.8(2.25)A_n \cdot \sqrt{f'_m}$$

$$= \frac{0.8(2.25)(11.63 \times 108)\sqrt{1500}}{1000}$$

$$= 88 \text{ kips} > V_u$$

The shear strength provided by the masonry is sufficient to resist the demand and no shear reinforcement is required.

When designing the jamb reinforcement, the width of the jamb that resists the load around the opening must be selected. Using an approach similar to the determination of the effective flange width in shear walls, it is reasonable to assume that the effective jamb width should not exceed 6 times the nominal wall thickness. Thus, an effective jamb width of 48 inches can conservatively be used and all the steel that resists the out-of-plane forces around the opening must be placed within this width. With the simplified solution, the effect of the opening is ignored. Therefore, the out-of-plane loading due to the tributary width of the opening and the jamb self-weight is given by:

$$72.4\left(\frac{18}{2}\right) + 72.4(4) = 941.2 \text{ lbs /ft}$$

And since the jamb is assumed to be 48 inches wide, the load per foot is equal to:

$$\frac{941.2}{4} = 235.3 \text{ psi}$$

For brevity, only one load combination will be considered (0.9*D* + *E*). For a complete solution, the procedure described here should be repeated for all applicable load combinations. The vertical component of earthquake load is given by $0.2S_{DS}$. Therefore the factored axial load from the roof dead load mid-height, which is where maximum moment occurs, is equal to:

$$P_{uf} = [0.9 - 0.2(1.46)](64)\left(\frac{18}{2} + 4\right)$$

$$= 0.61(832) = 507.5 \text{ lbs}$$

$$= 126.9 \text{ lbs /ft}$$

The total factored axial load, including the self-weight of the wall is given by;

$$P_u = P_{uf} + P_{uw}$$

$$= 507.5 + 0.61\left[124\left(\frac{29}{2} + 2.5\right)\right]\left(\frac{18}{2} + 4\right)$$

$$= 17224 \text{ lbs} \quad = \quad 4306 \text{ lbs /ft}$$

$$\frac{P_u}{A_g} = \frac{4306}{12(11.63)} = 30.9 \text{ psi} \le 0.05 f'_m$$

...<u>OK</u>

From Table 3.1.8.2.1 of TMS 402-08 fully-grouted masonry wall with type S mortar and tensile stresses parallel to bed joints has a modulus of rupture of 163 psi. Therefore, the cracking moment is given by:

$$M_{cr} = S\left(f_r + \frac{P_u}{A_n}\right)$$

$$= \frac{12(11.63)^2}{6}\left(163 + \frac{4306}{12(11.63)}\right)$$

$$= 52{,}440 \text{ lbs-in/ft} = 4370 \text{ lbs-ft /ft}$$

If we try 6-#7 bars (*d* = 9.2 inches) in the 48-inch wide jamb:

$$A_s = \frac{6(0.60)}{4} = 0.9 \text{ in}^2 \text{ /ft}$$

The depth of the compression block, *a*, neutral axis, *c* and effective area of steel A_{se} are calculated as follows:

$$a = \frac{P_u + A_s f_y}{0.8 f'_m b}$$

$$= \frac{4306 + 0.9(60000)}{0.8(1500)12} = 4.05 \text{ in}$$

$$c = \frac{a}{0.8} = \frac{4.05}{0.8} = 5.06 \text{ in}$$

$$A_{se} = \frac{P_u + A_s f_y}{f_y}$$

$$= \frac{4306 + 0.9(60000)}{60000} = 0.97 \text{ in}^2 \text{ /ft}$$

Then the cracked of moment of inertia, I_{cr}, can be calculated:

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{900(1500)} = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$$

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$

$$= 21.5(0.97)(9.2-5.06)^2 + \frac{12(5.06)^3}{3}$$

$$= 875.7 \text{ in}^4 \text{ /ft}$$

While the gross moment of inertia is equal to:

$$I_g = \frac{bt^3}{12} = \frac{12(11.63)^3}{12} = 1573 \text{ in}^4 \text{ /ft}$$

The wall deflection, including P-Δ effects, can be calculated with a closed-form equation, assuming that the wall is cracked:

$$\delta_u = \frac{\left(\frac{w_u h^2}{8} + \frac{P_{uf} e}{2}\right) - M_{cr}\left(1 - \frac{I_{cr}}{I_g}\right)}{\frac{48 E_m I_{cr}}{5H^2} - \left(P_{uw} + P_{uf}\right)}$$

$$= \frac{\left(\frac{235.3\left(29\times12\right)^2}{8(12)} + \frac{126.9(9.3)}{2}\right) - 4370(12)\left(1 - \frac{875.7}{1573}\right)}{\frac{48(1350000)(875.7)}{5\left(29\times12\right)^2} - 4306}$$

$$= 3.07 \text{ in}$$

Thus the moment demand at the mid-height of the wall is given by:

$$M_u = \frac{w_u H^2}{8} + \frac{P_{uf} e}{2} + \left(P_{uf} + P_{uw}\right)\delta_u$$

$$= \frac{235.3(29)^2}{8} + \frac{126.9}{2}\left(\frac{9.3}{12}\right) + 4306\left(\frac{3.07}{12}\right)$$

$$= 25887 \text{ lbs-ft /ft}$$

The moment demand is larger than the cracking moment, so the assumption of a cracked wall is correct. The flexural strength with the reinforcement closer to one face of the wall is given by:

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

$$= \left(0.9\times60000 + 4306\right)\left(9.2 - \frac{4.05}{2}\right)\frac{1}{12}$$

$$-4306\left(9.2 - \frac{11.63}{2}\right)\frac{1}{12}$$

$$= 33647 \text{ lbs-ft /ft}$$

$\phi M_n = 0.9 M_n = 30283$ lbs-ft /ft > M_u …OK

Figure 3 shows the jamb reinforcement for the simplified design.

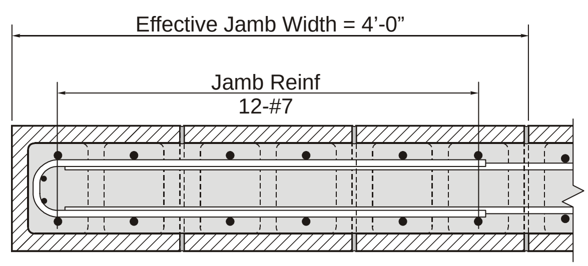

Figure 3 – Jamb Reinforcement for Simplified Design

Alternative Solution

If we assume a 36-inch effective width of the jamb, the out-of-plane load from the masonry above the opening is given by:

$$72.4\left(\frac{18}{2}\right) = 651.6 \text{ lbs /ft}$$

And the out-of-plane load due to the self-weight of the jamb is equal to:

$$72.4(3) = 217.2 \text{ lbs /ft}$$

Figure 4 shows the loading and moment diagram on the jamb, taking into account the effect of the opening.

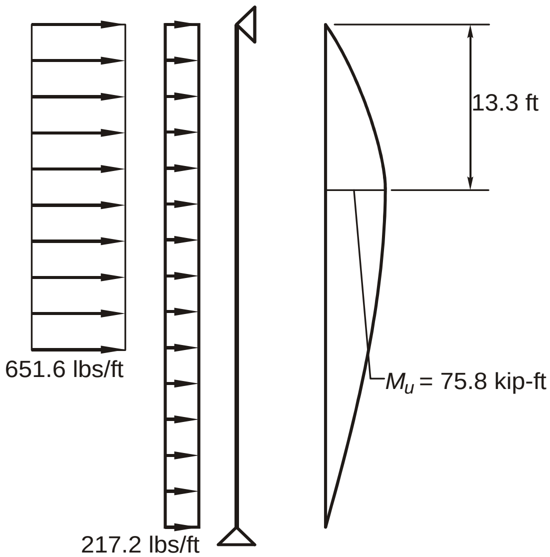

Figure 4 – Loads and Moments on Jamb

For the load combination 0.9*D*+*E*, The factored axial loads at that location of maximum moment, which occurs 13.3 feet from the roof level, are given by:

$$P_{uf} = [0.9 - 0.2(1.46)](64)\left(\frac{18}{2} + 3\right)$$

$$= 0.61(768) = 468.5 \text{ lbs}$$

$$P_u = P_{uf} + P_{uw}$$
$$= 468.5 + 0.61[124(13.3 + 2.5)]\left(\frac{18}{2} + 3\right)$$

$$= 14810 \text{ lbs}$$

If we try 4-#7 (A_s = 2.4 in^2) in the jamb, the depth of the neutral axis, *c*, and effective area of steel A_{se} are given by:

$$a = \frac{P_u + A_s f_y}{0.8 f'_m b}$$

$$= \frac{14810 + 2.4(60000)}{0.8(1500)36} = 3.68 \text{ in}$$

$$c = \frac{3.68}{0.8} = 4.6 \text{ in}$$

$$A_{se} = \frac{P_u + A_s f_y}{f_y}$$

$$= \frac{14810 + 2.4(60000)}{60000} = 2.65 \text{ in}^2$$

By inspection the wall is cracked and the cracked moment of inertia, I_{cr}, is equal to:

$$I_{cr} = nA_{se}(d - c)^2 + \frac{bc^3}{3}$$

$$= 21.5(2.65)(9.2 - 4.6)^2 + \frac{36(4.6)^3}{3}$$

$$= 2374 \text{ in}^4 \text{ /ft}$$

The critical buckling load is obtained from Equation (2):

$$P_e = \frac{\pi^2 E_m I_{eff}}{H^2}$$

$$= \frac{\pi^2(1350)(2374)}{(29 \times 12)^2} = 261 \text{ kips}$$

And the amplified moment is calculated from Equation (1):

$$M_c = \frac{M_u}{1 - \dfrac{P_u}{0.75 P_e}} = \frac{75.8}{1 - \dfrac{14.8}{0.75(261)}} = 82 \text{ kip-ft}$$

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

$$= \left(2.4 \times 60000 + 14810\right)\left(9.2 - \frac{3.68}{2}\right)\frac{1}{12}$$
$$-14810\left(9.2 - \frac{11.63}{2}\right)\frac{1}{12}$$

$$= 93226 \text{ lbs-ft /ft}$$

$\phi M_n = 0.9 M_n = 83.9$ kip-ft $> M_c$ …OK

Figure 5 shows the jamb reinforcement for the alternative design and Figure 6 shows an elevation of the wall with the reinforcement around the opening.

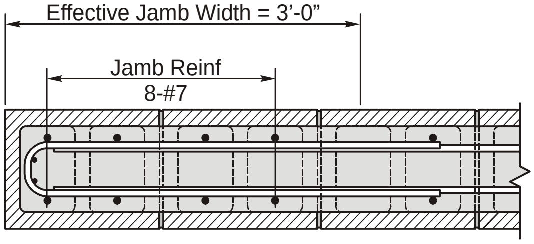

Figure 5 – Jamb Reinforcement for Alternative Design

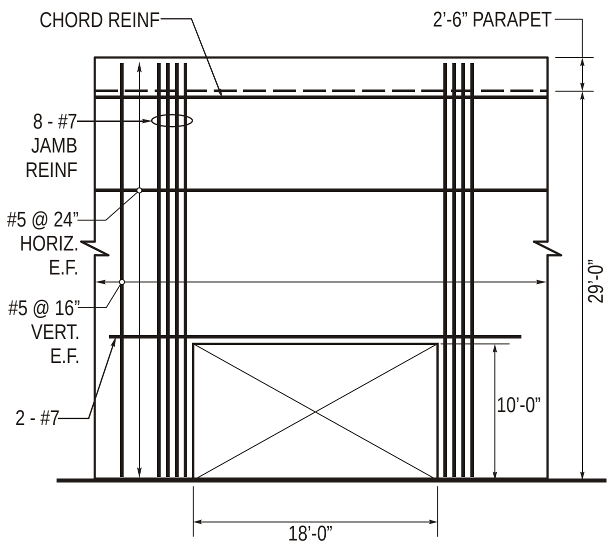

Figure 6 – Wall Reinforcement Around Opening

Conclusions

As expected, a design using a more rigorous design approach results in significantly lower reinforcement requirements than an approximate approach that ignores the reduction in out-of-plane earthquake loads due to the presence of openings in the wall.

It should be noted that the moment magnification procedure used with the alternative design approach may be applied to any load distribution or boundary conditions on a member. It is also generally results in a larger increase of moments due P-Δ effects than the code equations that are only valid for uniformly distributed loads on simply supported members.

This edition of Masonry Chronicles was written by Chukwuma Ekwueme, PhD., S.E. of Weidlinger Associates, Inc., Marina Del Rey, California.

Nomenclature

A_s = area of reinforcing steel

A_{se} = effective area of reinforcing steel including the effect of axial load

a = depth of equivalent compression stress block

b = width of cross-section

c = distance from fiber with maximum compressive strain to the neutral axis

d = distance from fiber of maximum compressive strain to centroid of tension reinforcement

E_m = modulus of elasticity of masonry in compression

E_m = modulus of elasticity of steel

H = effective height of wall

I_{cr} = cracked moment of inertia

I_{eff} = effective moment of inertia

I_g = gross moment of inertia

M_c = factored moment demand magnified to account for P-Δ effects

M_u = factored moment demand

M_n = nominal moment strength

n = modular ratio

P_e = critical buckling load

P_u = factored axial load

P_{uf} = factored axial load from floor or roof

P_{uw} = factored axial load from wall self weight

t = thickness of wall

ϕ = strength reduction factor

References

1. Masonry Standards Joint Committee (MSJC), *Building Code Requirements for Masonry Structures (TMS 402-08/ACI 530-08/ASCE 5-08),* The Masonry Society, Boulder, Colorado, 2008.

2. ACI Committee 318, Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, American Concrete Institute, Farmington Hills, MI, 2008.

3. ASCE, Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-05), Including Supplement 1, American Society of Civil Engineers (ASCE), Reston Virginia, 2006.

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