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Sustainable Concrete Products for Structures and Hardscapes

DESIGN OF CONCRETE MASONRY INFILL

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Structural (2012)

INTRODUCTION

Masonry infill refers to masonry used to fill the opening in a structural frame, known as the bounding frame. The bounding frame of steel or reinforced concrete is comprised of the columns and upper and lower beams or slabs that surround the masonry infill and provide structural support. When properly designed, masonry infills provide an additional strong, ductile system for resisting lateral loads, in-plane and out-of-plane.

Concrete masonry infills can be designed and detailed to be part of the lateral force-resisting system (participating infills) or they can be designed and detailed to be structurally isolated from the lateral force-resisting system and resist only out-of-plane loads (non-participating infills).

Participating infills form a composite structural system with the bounding frame, increasing the strength and stiffness of the wall system and its resistance to earthquake and wind loads.

Non-participating infills are detailed with structural gaps between the infill and the bounding frame to prevent the unintended transfer of in-plane loads from the frame into the infill. Such gaps are later sealed for other code requirements such as weather protection, air infiltration, energy conservations, etc.

Construction of concrete masonry infilled frames is relatively simple. First, the bounding frame is constructed of either reinforced concrete or structural steel, then the masonry infill is constructed in the portal space. This construction sequence allows the roof or floor to be constructed prior to the masonry being laid, allowing for rapid construction of subsequent stories or application of roofing material.

The 2011 edition of *Building Code Requirements for Masonry Structures* (MSJC Code, ref. 1) includes a new mandatory language Appendix B for the design of masonry infills that can be either unreinforced or reinforced. Appendix B provides a straightforward method for the design and analysis of both participating and non-participating infills. Requirements were developed based on experimental research as well as field performance.

MASONRY INFILL LOAD RESPONSE

Several stages of in-plane loading response occur with a participating masonry infill system. Initially, the system acts as a monolithic cantilever wall whereby slight stress concentrations occur at the four corners, while the middle of the panel develops an approximately pure shear stress state. As loading continues, separation occurs at the interface of the masonry and the frame members at the off-diagonal corners. Once a gap is formed, the stresses at the tensile corners are relieved while those near the compressive corners are increased.

As loading continues, further separation between the masonry panel and the frame occurs, resulting in contact only near the loaded corners of the frame. This results in the composite system behaving as a braced frame, which leads to the concept of replacing the masonry infill with an equivalent diagonal strut, as shown in Figure 1. These conditions are addressed in the masonry standard.

Participating masonry infills resist out-of-plane loads by an arching mechanism. As out-of-plane loads increase beyond the elastic limit, flexural cracking occurs in the masonry panel. This cracking (similar to that which occurs in reinforced masonry) allows for arching action to resist the applied loads, provided the infill is constructed tight to the bounding frame and the infill is not too slender.

Keywords: building codes, connectors, masonry infill, structural design

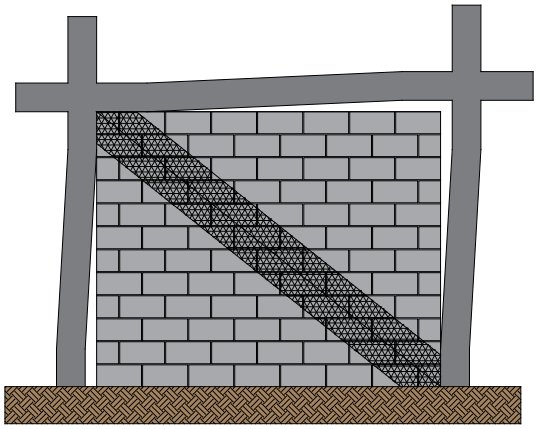
IN-PLANE SHEAR FOR PARTICIPATING INFILLS

For participating infills, the masonry is either mortared tight to the bounding frame so that the infill receives lateral loads immediately as the frame displaces, or the masonry is built with a gap such that the bounding frame deflects slightly before it bears upon the infill. If a gap exists between the infill and the frame, the infill is considered participating if the gap is less than $^3/_8$ in. (9.5 mm) and the calculated displacements, according to MSJC Code Section B3.1.2.1. However, the infill can still be designed as a participating infill, provided the calculated strength and stiffness are reduced by half.

The maximum height-to-thickness ratio (h/t) of the participating infill is limited to 30 in order to maintain stability. The maximum thickness allowed is one-eighth of the infill height.

The MSJC Code requires participating infills to fully infill the bounding frame and have no openings—partial infills or infills with openings may not be considered as part of the lateral force resisting system because structures with partial infills have typically not performed well during seismic events. The partial infill attracts additional load to the column due to its increased stiffness; typically, this results in shear failure of the column.

The in-plane design is based on a braced frame model, with the masonry infill serving as an equivalent strut. The width of the strut is determined from Equation 1 (see Figure 1).



Note that gaps are exaggerated for demonstration purposes.

Figure 1—Concrete Masonry Infill as a Diagonal Strut

$$w_{\inf} = \frac{0.3}{\lambda_{strut} \cos\theta_{strut}} \qquad \text{Eqn. 1}$$

where:

$$\lambda_{strut} = \sqrt[4]{\frac{E_m t_{net\inf} \sin 2\theta_{strut}}{4 E_{bc} I_{bc} h_{\inf}}} \qquad \text{Eqn. 2}$$

The term λ_{strut}, developed by Stafford Smith and Carter (ref. 2) in the late 60s, is the characteristic stiffness parameter for the infill and provides a measure of the relative stiffness of the frame and the infill. Design forces in the equivalent strut are then calculated based on elastic shortening of the compression-only strut within the braced frame. The area of the strut used for that analysis is determined by multiplying the strut width from Equation 1 by the specified thickness of the infill.

The infill capacity can be limited by shear cracking, compression failure, and flexural cracking. Shear cracking can be characterized by cracking along the mortar joints (which includes stepped and horizontal cracks) and by diagonal tensile cracking. The compression failure mode consists of either crushing of the masonry in the loaded diagonal corners or failure of the equivalent diagonal strut. The diagonal strut is developed within the panel as a result of diagonal tensile cracking. Flexural cracking failure is rare because separation at the masonry-frame interface usually occurs first; then, the lateral force is resisted by the diagonal strut.

As discussed above, the nominal shear capacity is determined as the least of: the capacity infill corner crushing; the horizontal component of the force in the equivalent strut at a racking displacement of 1 in. (25 mm); or, the smallest nominal shear strength from MSJC Code Section 3.2.4, calculated along a bed joint. The displacement limit was found to be a better predictor of infill performance than a drift limit.

Generally, the infill strength is reached at lower displacements for stiff bounding columns, while more flexible columns result in the strength being controlled at the 1-in. (25-mm) displacement limit. While MSJC Code Section 3.2 is for unreinforced masonry, use of equations from that section does not necessarily imply that the infill material must be unreinforced. The equations used in MSJC Code Section 3.2 are more clearly related to failure along a bed joint and are therefore more appropriate than equations from MSJC Code Section 3.3 for reinforced masonry.

The equations used in the code are the result of comparing numerous analytical methods to experimental results. They are strength based. The experimental results used for comparison were a mixture of steel and reinforced concrete bounding frames with clay and concrete masonry.

While some methods presented by various researchers are quite complex, the code equations are relatively simple.

OUT-OF-PLANE FLEXURE FOR PARTICIPATING INFILLS

The out-of-plane design of participating infills is based on arching of the infill within the frame. As out-of-plane forces are applied to the surface of the infill, a two-way arch develops, provided that the infill is constructed tight to the bounding frame. The code equation models this two-way arching action.

As previously mentioned, the maximum thickness allowed for calculation for the out-of-plane capacity is one-eighth of the infill height. Gaps between the bounding frame on either the sides or top of the infill reduce the arching mechanism to a one-way arch and are considered by the code equations. Bounding frame members that have different cross sectional properties are accounted for by averaging their properties for use in the code equations.

NON-PARTICIPATING INFILLS

Because non-participating infills support only out-of-plane loads, they must be detailed to prevent in-plane load transfer into the infill. For this reason, MSJC Code Section B.2.1 requires these infills to have isolation joints at the sides and the top of the infill. These isolation joints must be at least $^3/_8$ in. (9.5 mm) and sized to accommodate the expected design displacements of the bounding frame, including inelastic deformation due to a seismic event, to prevent the infill from receiving in-plane loadings. The isolation joints may contain filler material as long as the compressibility of the material is taken into consideration when sizing the joint.

Mechanical connectors and the design of the infill itself ensure that non-participating infills support out-of-plane loads. Connectors are not allowed to transmit in-plane loads. The masonry infill may be designed to span vertically, horizontally, or both. The masonry design of the non-participating infill is carried out based on the applicable MSJC Code sections for reinforced or unreinforced masonry (Section 3.2 for unreinforced infill and Section 3.3 for reinforced infill using strength design methods). Note that there are seismic conditions which may require the use of reinforced masonry.

Because they support only out-of-plane loads, non-participating infills can be constructed with full panels, partial height panels, or panels with openings. The corresponding effects on the bounding frame must be included in the design.

BOUNDING FRAME FOR PARTICIPATING INFILLS

The MSJC Code provides guidance on the design loads applied to the bounding frame members; however, the actual member design is governed by the appropriate material code and is beyond the scope of the MSJC Code.

The presence of infill within the bounding frame places localized forces at the intersection of the frame members. MSJC Code Section B.3.5 helps the designer determine the appropriate augmented loads for designing the bounding frame members. Frame members in bays adjacent to an infill, but not in contact with the infill, should be designed for no less than the forces (shear, moment, and axial) from the equivalent strut frame analysis. In the event of infill failure, the loading requirement on adjacent frame members ensures adequacy in the frame design, thus preventing progressive collapse.

The shear and moment applied to the bounding column must be at least the results from the equivalent strut frame analysis multiplied by a factor of 1.1. The axial loads are not to be less than the results of that analysis. Additionally, the horizontal component of the force in the equivalent strut is added to the design shear for the bounding column.

Similarly, the shear and moment applied to the bounding beam or slab must be at least the results from the equivalent strut frame analysis multiplied by a factor of 1.1, and the axial loads are not to be less than the results of that analysis. The vertical component of the force in the equivalent strut is added to the design shear for the bounding beam or slab.

The bounding frame design should also take into consideration the volumetric changes in the masonry infill material that may occur over time due to normal temperature and moisture variations. Shrinkage of concrete masonry infill material may open gaps between the infill and the bounding frame that need to be addressed. Guidance for these volumetric changes is provided in MSJC Code Section 1.7.5.

CONNECTORS

Mechanical connectors between the bounding frame and the infill provide out-of-plane support of the masonry, for both participating and non-participating infills. Connectors are required only for the direction of span (i.e., at the top and bottom of the infill for infill spanning vertically, for example). The connectors must be designed to support the expected out-of-plane loads and may not be spaced more than 4 ft (1.2 m) apart along the perimeter of the infill. Figure 2 shows an example of a mechanical connector composed of clip angles welded to the bottom flange of the steel beam.

Connectors for both participating and non-participating infills are not permitted to transfer in-plane loads from the bounding frame to the infill. For participating infills, in-plane loads are assumed to be resisted by a diagonal compression strut (see Figure 1), which does not rely upon mechanical connectors to transfer in-plane load. Research (ref. 3) has shown that when connectors transmit in-plane loads they create regions of localized stress and can cause premature damage to the infill. This damage then reduces the infill's out-of-plane capacity because arching action is inhibited.

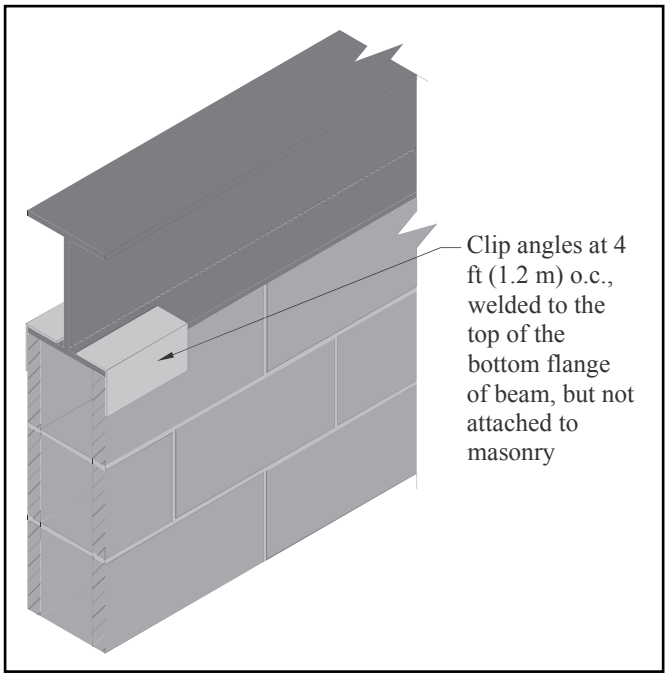


Figure 2—Example of Mechanical Infill Connector

EXAMPLE 1: DESIGN OF PARTICIPATING MASONRY INFILL WALL FOR IN-PLANE LOADS

Consider the simple structure of Figure 3. The east and west side walls are concrete masonry infills laid in running bond, while the north and south walls are store-fronts typical of convenience stores. Steel frames support all gravity loads and the lateral load in the east-west direction. The bounding columns are W10x45s oriented with the strong axis in the east-west direction. The bounding beams above the masonry infill are W10x39s. The masonry infill resists the lateral load in the north-south direction.

Use nominal 8-in. (203-mm) concrete masonry units, f'_m = 1,500 psi (10.34 MPa), and Type S PCL mortar. Assume hollow units with face-shell bedding only. The total wall height measures 16 ft-10 in. (5.1 m) to the roof with the infill being 16 ft (4.9 m). The building is loaded with a wind load of 24 lb/ft^2 calculated per ASCE 7-10 (ref. 6) in the north-south direction. The roof acts as a one-way system, transmitting gravity loads to the north and south roof beams. Infill and bounding beam properties are summarized in Tables 1 and 2.

MSJC Code Section B.3.4.3 requires $V_{n\ inf}$ to be the smallest of the following:

- (6.0 in.)$t_{net\ inf}$ f'_m

Table 1—Infill Properties

Property:	Value:	Notes
Vertical dimension of infill, h_{inf}	192 in. (4,877 mm)	
Plan length of infill, l_{inf}	360 in. (9,144 mm)	
Specified thickness of infill, t_{inf}	7.625 in. (194 mm)	
Net thickness of infill, $t_{net\ inf}$	2.5 in. (64 mm)	Face shell thickness x 2
Specified compressive strength of masonry, f'_m	1,500 psi (10.34 MPa)	
Modulus of elasticity of masonry in compression, E_m	1,350,000 psi (7,300 MPa)	

Table 2—Bounding Frame Properties for In-Plane Loads

Property:	Value:	Notes
Modulus of elasticity of bounding beams, E_{bb}	29,000,000 psi (200,000 MPa)	
Modulus of elasticity of bounding columns, E_{bc}	29,000,000 psi (200,000 MPa)	
Moment of inertia of bounding beams for bending in the plane of the infill, I_{bb}	209 in.^{4} (8.7 x 10^{-5} m^4)	Bounding beam strong axis
Moment of inertia of bounding columns for bending in the plane of the infill, I_{bc}	53.4 in.^{4} (2.2 x 10^{-5} m^4)	Bounding column weak axis

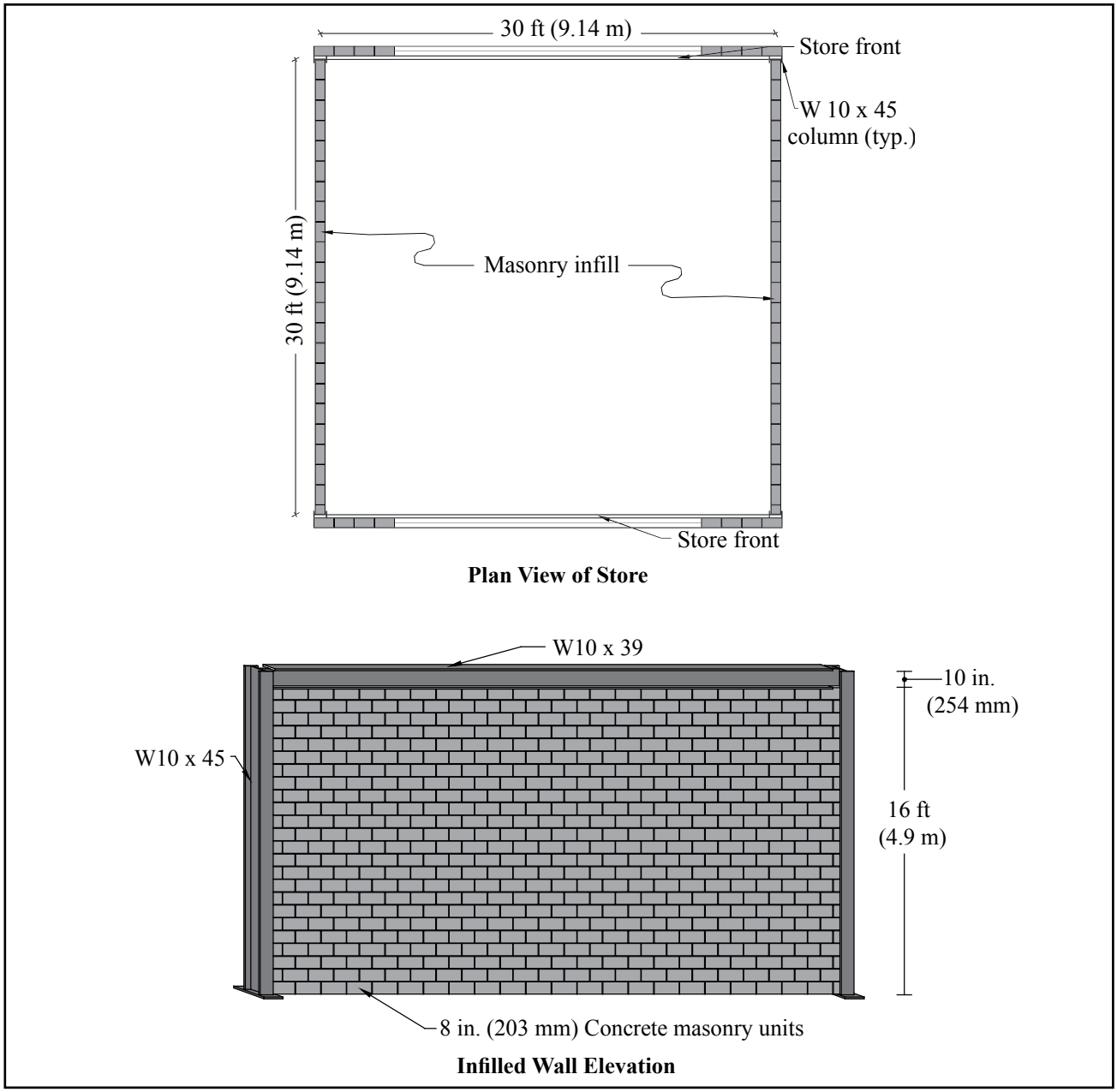


Figure 3—Convenience Store Layout for Design Examples 1 & 2

- the calculated horizontal component of the force in the equivalent strut at a horizontal racking displacement of 1.0 in. (25 mm)
- $V_n/1.5$, where V_n is the smallest nominal shear strength from MSJC Code Section 3.2.4, calculated along a bed joint.

MSJC Code Section 3.2.4 requires the nominal shear strength not exceed the least of the following:

- $3.8\,A_n\sqrt{f'_m}$
- $300A_n$
- $56A_n + 0.45N_v$ for running bond masonry not fully grouted and for masonry not laid in running bond, constructed of open end units, and fully grouted
- $90A_n + 0.45N_v$ for running bond masonry fully grouted
- $23A_n$ for masonry not laid in running bond, constructed of other than open end units, and fully grouted

As a result of the wind loading, the reaction transmitted to the roof diaphragm is:

Reaction = $^1/_2$ (24 lb/ft^2)(16.83 ft)
= 202 lb/ft (2.95 kN/m)

Total roof reaction acting on one side of the roof is
Reaction = (202 lb/ft)(30 ft)
= 6,060 lb (27.0 kN)

This reaction is divided evenly between the two masonry infills, so the shear per infill is 3,030 lb (13.5 kN).

Using the conservative loading case of $0.9D + 1.0W$,
$V_u = 1.0\ V_{unfactored} = 1.0\ (3{,}030\ \text{lb}) = 3{,}030\ \text{lb}\ (13.5\ \text{kN})$

To be conservative, the axial load to the masonry infill is taken as zero.

To ensure practical conditions for stability, the ratio of the nominal vertical dimension to the nominal thickness is limited to 30 for participating infills. The ratio for this infill is:
$h/t = 192\ \text{in.}/8\ \text{in.} = 24 < 30$
The ratio is less than 30 and the infill is therefore acceptable as a participating infill.

The width of the equivalent strut is calculated by Equation 1 (MSJC Code Equation B-1):

$$w_{\inf} = \frac{0.3}{\lambda_{strut} \cos\theta_{strut}}$$

where λ_{strut} is given by Equation 2 (Code Equation B-2).

The angle of the equivalent diagonal strut, θ_{strut}, is the angle of the infill diagonal with respect to the horizontal.
$\theta_{strut} = \tan^{-1}(h_{inf}/l_{inf}) = \tan^{-1}(192\ \text{in.}/360\ \text{in.}) = 28.1^{\circ}$

Using Equation 2, the characteristic stiffness parameter, λ_{strut}, for this infill is then:

$$\lambda_{strut} = \sqrt[4]{\frac{1{,}350{,}000\ \text{psi} \times 2.5\ \text{in.} \times \sin(2(28.1^{\circ}))}{4 \times 29{,}000{,}000\ \text{psi} \times 53.4\ \text{in.}^4 \times 192\ \text{in.}}}$$

$= 0.0392\ \text{in.}^{-1}$

The resulting strut width is then:

$$w_{\inf} = \frac{0.3}{0.0392\ \text{in.}^{-1} \times \cos(28.1^{\circ})} = 8.7\ \text{in.}$$

The stiffness of the equivalent braced frame is determined by a simple braced frame analysis where the stiffness is based on the elastic shortening of the diagonal strut. The strut area is taken as the width of the strut multiplied by the net thickness of the infill.

The stiffness is:

$$\text{stiffness} = \frac{AE_m \cos^2\theta}{d} = \frac{w_{\inf} t_{net\inf} E_m l_{\inf}^2}{d^3}$$

where d is the diagonal length of the infill, 34 ft (10.3 m) in this case.

$$\text{stiffness} = \frac{8.7\ \text{in.} \times 2.5\ \text{in.} \times 1{,}350{,}000\ \text{psi} \times \left(30\ \text{ft} \times 12\ \frac{\text{in.}}{\text{ft}}\right)^2}{\left(34\ \text{ft} \times 12\ \frac{\text{in.}}{\text{ft}}\right)^3}$$

= 56,030 lb/in. (818 kN/m)

The nominal shear capacity, V_n, is then the least of:
- $(6.0\ \text{in.})t_{net\ inf} f'_m = (6.0\ \text{in.})(2.5\ \text{in.})(1{,}500\ \text{psi}) = 22{,}500\ \text{lb}$
- $(56{,}030\ \text{lb/in.})(1\ \text{in.}) = 56{,}030\ \text{lb}$
- $(3.8\sqrt{f'_m}\,A_n)/1.5 = [3.8\sqrt{1{,}500\ \text{psi}}\ (30\ \text{ft} \times 30\ \text{in.}^2/\text{ft})]/1.5 = 88{,}304\ \text{lb}$
- $(300A_n)/1.5 = [300(30\ \text{ft} \times 30\ \text{in.}^2/\text{ft})]/1.5 = 180{,}000\ \text{lb}$
- $(56A_n + 0.45N_v)/1.5 = [56(30\ \text{ft} \times 30\ \text{in.}^2/\text{ft}) + 0.45 \times 0]/1.5 = 33{,}600\ \text{lb}$

$V_n = 22{,}500\ \text{lb}\ (100\ \text{kN})$

The design shear capacity is:
$\phi V_n = 0.6 \times 22{,}500\ \text{lb} = 13{,}500\ \text{lb}\ (60\ \text{kN})$

The design shear capacity far exceeds the factored design shear of 3,030 lb (13.5 kN), so the infill is satisfactory for shear.

Additionally, the provisions of MSJC Code Section B.3.5 require that the designer consider the effects of the infill on the bounding frame. To ensure adequacy of the frame members and connections, the shear and moment results of the equivalent strut frame analysis are multiplied by a factor of 1.1. The column designs must include the horizontal component of the equivalent strut force, while the beam designs must include the vertical component of the equivalent strut force. The axial forces from the equivalent strut frame analysis must also be considered in both the column and beam designs.

Table 3—Bounding Frame Properties for Out-of Plane Loads

Property:	Value:	Notes
Moment of inertia of bounding beams for bending in the plane of the infill, I_{bb}	45 in.^{4} (1.9 x 10^{-5} m^4)	Beam weak axis
Moment of inertia of bounding columns for bending in the plane of the infill, I_{bc}	248 in.^{4} (1.0 x 10^{-4} m)	Column strong axis
All other design parameters are the same as used in Example 1.		

EXAMPLE 2: DESIGN OF PARTICIPATING MASONRY INFILL WALL FOR OUT-OF-PLANE LOADS

Design the infill from the previous example for an out-of-plane wind load W of 24 lb/ft^2 (1.2 kPa) per ASCE 7-10 acting on the east wall, using Type S PCL mortar, and units with a nominal thickness of 8 in. (203 mm). Assume hollow units with face-shell bedding only and that the infill is constructed tight to the bounding frame such that there are no gaps at the top or sides of the infill. See Table 3 for frame properties.

MSJC Code Section B.3.6 provides the equations for the nominal out-of-plane flexural capacity. MSJC Code Equation B-5 requires that the flexural capacity of the infill be:

$$q_{n\inf} = 105\left(f'_m\right)^{0.75} t_{\inf}^2 \left(\frac{\alpha_{arch}}{l_{\inf}^{2.5}} + \frac{\beta_{arch}}{h_{\inf}^{2.5}}\right)$$

where:

$$\alpha_{arch} = \frac{1}{h_{\inf}}\left(E_{bc} I_{bc} h_{\inf}^2\right)^{0.25} < 35$$

$$\beta_{arch} = \frac{1}{l_{\inf}}\left(E_{bb} I_{bb} l_{\inf}^2\right)^{0.25} < 35$$

$\alpha_{arch} = 0$ if a side gap is present,
$\beta_{arch} = 0$ if a top gap is present, and
$t_{inf} \leq {}^1/_8\, h_{inf}$.

Using the conservative loading case of $0.9D + 1.0W$, the design wind load pressure is:

q = $1.0W$ = 1.0 x 24 psf = 24 psf (1.15 kPa)
t_{inf} = 7.625 in. < ($^1/_8$)(192 in.), OK

$$\alpha_{arch} = \frac{1}{h_{\inf}}\left(E_{bc} I_{bc} h_{\inf}^2\right)^{0.25}$$

$$= \frac{1}{192\,\text{in.}}\left(29{,}000{,}000\,\text{psi} \times 248\,\text{in.}^4 \times (192\,\text{in.})^2\right)^{0.25}$$

= 21 lb$^{0.25}$ (31 N$^{0.25}$) < 35

$$\beta_{arch} = \frac{1}{l_{\inf}}\left(E_{bb} I_{bb} l_{\inf}^2\right)^{0.25}$$

$$= \frac{1}{360\,\text{in.}}\left(29{,}000{,}000\,\text{psi} \times 45\,\text{in.}^4 \times (360\,\text{in.})^2\right)^{0.25}$$

= 10 lb$^{0.25}$ (15 N$^{0.25}$) < 35

$$q_{n\inf} = 105\left(f'_m\right)^{0.75} t_{\inf}^2 \left(\frac{\alpha_{arch}}{l_{\inf}^{2.5}} + \frac{\beta_{arch}}{h_{\inf}^{2.5}}\right)$$

$$q_{n\inf} = 105(1{,}500\,\text{psi})^{0.75}(7.63\,\text{in.})^2\left(\frac{21\,\text{lb}^{0.25}}{(360\,\text{in.})^{2.5}} + \frac{10\,\text{lb}^{0.25}}{(192\,\text{in.})^{2.5}}\right)$$

= 41.4 psf (2.0 kPa)

The design flexural capacity is
ϕV_n = 0.6 x 41.4 psf = 24.8 psf (1.2 kPa)

The design flexural capacity exceeds the factored design wind load pressure of 24 lb/ft^2 (1.2 kPa), so the infill is satisfactory for out-of-plane loading

NOTATIONS

A_n	=	net cross-sectional area of a member, in.^{2} (mm^2)
D	=	dead load, psf (Pa)
d	=	diagonal length of the infill, in. (mm)
E_{bb}	=	modulus of elasticity of bounding beams, psi (MPa)
E_{bc}	=	modulus of elasticity of bounding columns, psi (MPa)
E_m	=	modulus of elasticity of masonry in compression, psi (MPa)
f'_m	=	specified compressive strength of masonry, psi (MPa)
h	=	effective height of the infill, in. (mm)
h_{inf}	=	vertical dimension of infill, in. (mm)
I_{bb}	=	moment of inertia of bounding beam for bending in the plane of the infill, in.^{4} (mm^4)
I_{bc}	=	moment of inertia of bounding column for bending in the plane of the infill, in.^{4} (mm^4)
l_{inf}	=	plan length of infill, in. (mm)
N_v	=	compressive force acting normal to shear surface, lb (N)
$q_{n\,inf}$	=	nominal out-of-plane flexural capacity of infill per unit area, psf (Pa)
t	=	nominal thickness of infill, in. (mm)
t_{inf}	=	specified thickness of infill, in. (mm)
$t_{net\,inf}$	=	net thickness of infill, in. (mm)
V_n	=	nominal shear strength, lb (N)
$V_{n\,inf}$	=	nominal horizontal in-plane shear strength of infill, lb (N)
V_u	=	factored shear force, lb (N)
$V_{unfactored}$	=	unfactored shear force, lb (N)
W	=	out of plane wind load, psf (Pa)
w_{inf}	=	width of equivalent strut, in. (mm)
α_{arch}	=	horizontal arching parameter for infill, lb$^{0.25}$ (N$^{0.25}$)
β_{arch}	=	vertical arching parameter for infill, lb$^{0.25}$ (N$^{0.25}$)
λ_{strut}	=	characteristic stiffness parameter for infill, in.$^{-1}$ (mm^{-1})
θ_{strut}	=	angle of infill diagonal with respect to the horizontal, degrees
ϕ	=	strength reduction factor

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