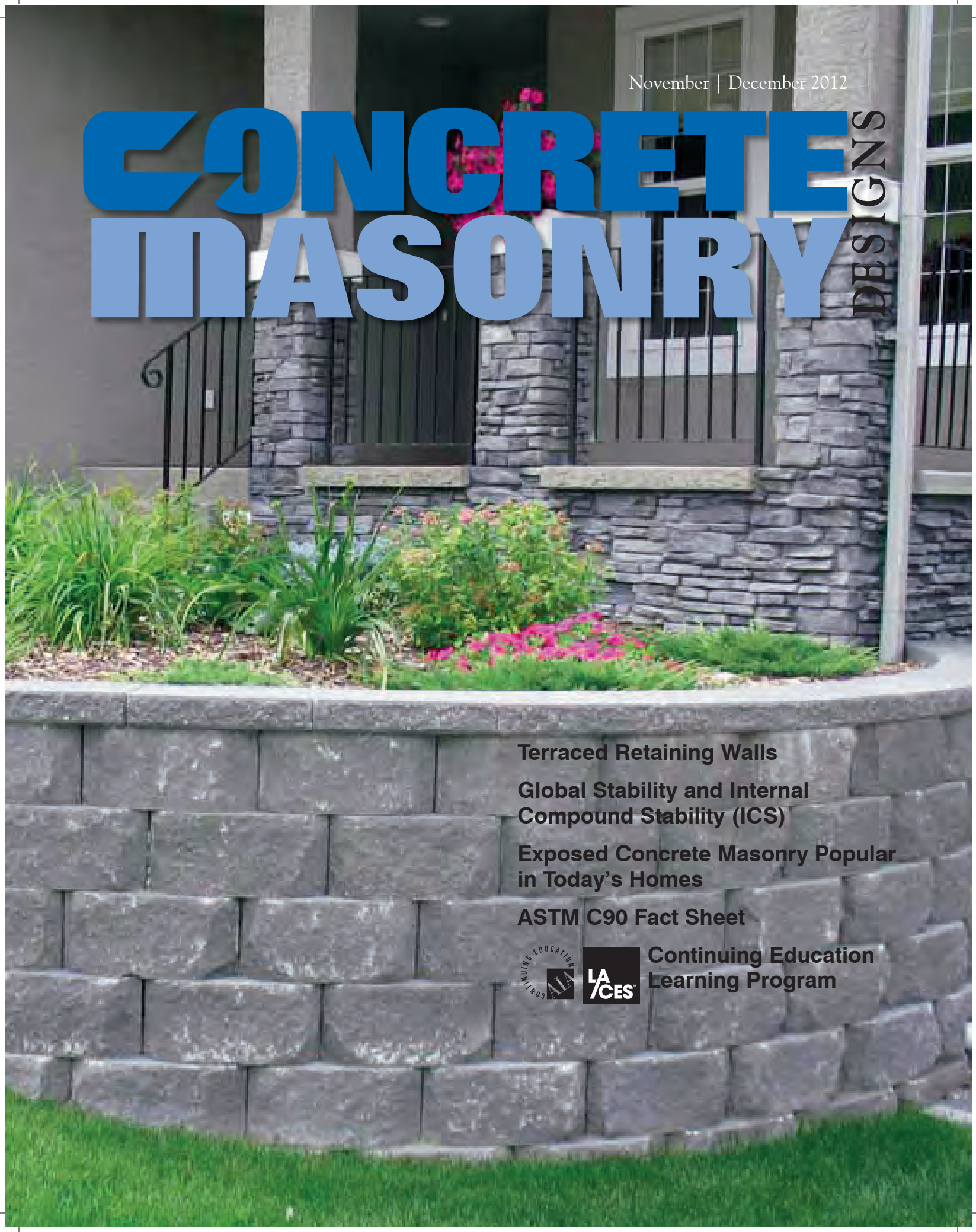
November | December 2012

CONCRETE MASONRY DESIGNS



Terraced Retaining Walls

Global Stability and Internal Compound Stability (ICS)

Exposed Concrete Masonry Popular in Today's Homes

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CONCRETE MASONRY DESIGNS

November | December 2012

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Concrete Masonry Designs magazine showcases the qualities and aesthetics of design and construction using concrete masonry.

Concrete Masonry Designs is devoted to design techniques using standard and architectural concrete masonry units, concrete brick, unit concrete pavers, segmental retaining walls, and other concrete masonry products around the world. We welcome your editorial comments, ideas, and submissions.

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TERRACED

PROJECT NAME & LOCATION
Tocher Residence
West Kelowana, BC

ENGINEER
Cascade Geotech

SRW CONTRACTOR
Sunnyslope Landscape & Custom Design

SRW PRODUCER
Expocrete

SRW LICENSOR
Allan Block

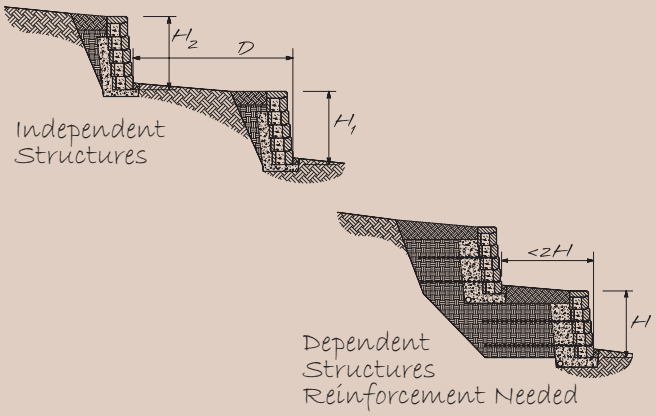
GEOGRID MANUFACTURER
Strata Systems

RETAINING WALLS

Transforming a steeply sloped property and creating useable land is a primary function of segmental retaining walls (SRWs). This particular project is a good illustration of just how dramatic a property transformation can be. Using SRW units, this project was able to reclaim unusable land on the slopes below a residence and create a level, useable space for the homeowners to enjoy the outdoors.

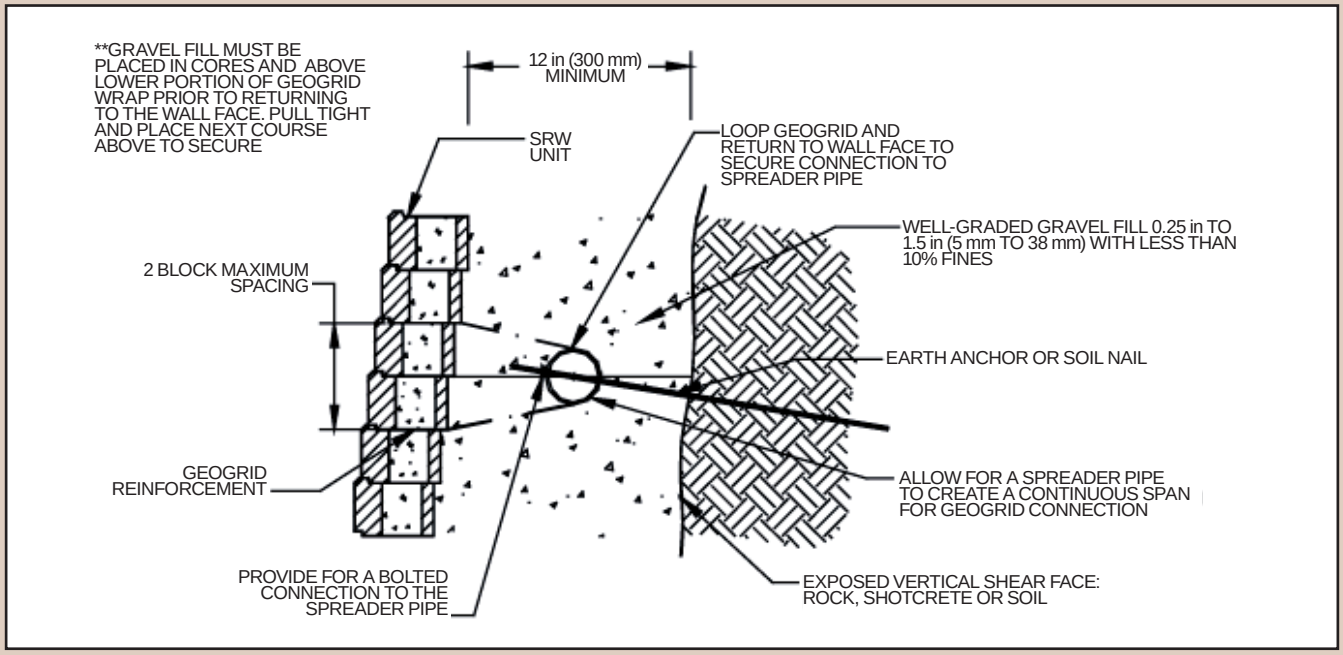
PLAN

The Tocher residence is perched on a small plot of land overlooking Okanagan Lake in West Kelowna, British Columbia. Although the location provided beautiful vistas, the steep slopes and small lot size limited use of the tiny backyard. For the Tocher's, the plan was to replace the smaller retaining wall located close to the house with a substantially larger wall near the property line to create enough useable space to install a swimming pool.



Use this simple equation to determine if walls are independent:

$H_2 \leq H_1$ and $D > 2H_1$



DESIGN

The walls had to handle more than 40 ft (12.2 m) of grade change from the back property line to achieve the desired height of the backyard. Local ordinance allowed for exposed wall heights up to 8 ft (2.4 m) with at least 6 ft (1.8 m) between tiered walls. To achieve their goals and adhere to the local ordinance, it required the installation of five retaining walls - four of the walls were 10 ft (3 m) tall and one was 4 ft (1.2 m) tall with a slight slope between walls to meet the 8 ft (2.4 m) exposed height limit.

Maximizing useable space required creating a somewhat rectangular area that closely mimics the property boundaries. The landscape contractor created a unique design that adhered to the need for a rectangular space but incorporated a unique series of curves, between the 90° corners, to soften the lines and give a more natural appearance to the wall. The color and geometry of the walls also mimic the orientation and color of the rock outcroppings below which makes them fit in with the natural surroundings better than many large wall projects.

Due to the close proximity of the rock outcroppings, the lowest wall had to be anchored to the rock face with 1 in. (2.5 cm) stainless threaded rods (see detail above). The rod was inserted into a drilled hole in the rock face and epoxy grouted into the hole. The rod was attached to a 3 in. (7.6 cm) diameter schedule 40 galvanized pipe with the geogrid extending from the wall, wrapping around the pipe and extending back toward the wall to the next course above.

The second wall would utilize geogrid to reinforce the wall in lengths of 32 ft (9.8 m), where space allowed, and pinning to the rock face where insufficient clearance occurred. Subsequent walls above would also be reinforced with geogrid.

BUILD

A development below the property had blasted rock and created a 30 ft (9.1 m) cliff face near the back property line. The contractor was left with a sloped rock surface where the wall was to be located. Sunnyslope Landscape had to blast, jackhammer and excavate rock to create a solid, level ledge to build the wall.

Two blasts were required on this job. The first blast was done to create an adequate ledge for the lowest wall to be built and the second was to gain sufficient clearance between the second wall and the steeply sloped rock face. Although blasting required additional work, the blasted rock left behind provided good granular material to be used as infill behind the wall. Very little fill needed to be brought on-site to complete the job.

Difficult site access created challenges for this project as well. A road had to be constructed through the back of the steeply sloped neighbor's property to reach the construction area. Blasted and excavated rock had to be hauled out five yards (3.8 sq m) at a time and stored temporarily on a vacant lot next to the neighbor's home.

Upon completion of the walls, the customer was pleasantly surprised at the amount of useable space created. Original plans called for a fiberglass in-ground pool, but once the space was created, the Tochers asked their engineer if a concrete pool with an infinity edge could be installed above the new walls. The engineer's calculations indicated the wall could sufficiently handle the load.

The Tocher's small backyard, with a few plantings, a small sitting area and strip of grass was transformed into a magnificent outdoor living space, complete with a pool, pool house and an artificial putting green. The new wraparound pool deck and patio now provides the Tocher's family and friends a great place to enjoy the view of the valley below. CMD

Global Stability and Internal Compound Stability (ICS)

The general mass movement of a segmental retaining wall (SRW) structure and the adjacent soil is called global stability failure. Global stability analysis is an important component of SRW design, particularly under the following conditions:

- groundwater table is above or within the wall height of the SRW,
- a 3H:1V or steeper slope at the toe or top of the SRW,
- for tiered SRWs,
- for excessive surcharges above the wall top,
- for seismic design, and
- when the geotechnical subsurface exploration finds soft soils, organic soils, peat, high plasticity clay, swelling or shrinking soils or fill soil.

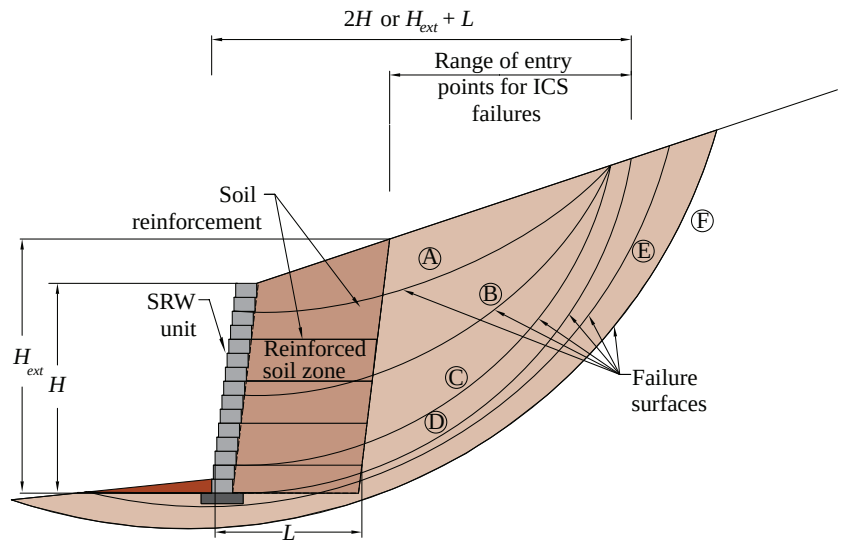
The designer should also review local code requirements applicable to designing soil retention structures.

There are two primary modes of global stability failure: deep-seated and compound. A deep-seated failure is characterized by a failure surface that starts in front of an SRW, passes below the base of the wall and extends beyond the tail of the geosynthetic reinforcement (see Figure 1, surface F).

Compound failures are typically described by a failure surface that passes either through the SRW face or in front of the wall, through the reinforced soil zone and continues into the unreinforced/retained soil (Fig. 1, surfaces A through E). A special case of the compound failure is the Internal Compound Stability (ICS) failure surface that exits at the SRW face above the foundation soil (Fig. 1, surfaces A through D).

Internal Compound Stability (ICS) affects the internal components of the retaining wall system, including the facing elements and reinforced zone. Because ICS is influenced by loading conditions outside the reinforced fill area, it is a special case of a larger compound analysis.

The *NCMA Design Manual for Segmental Retaining Walls* provides specific guidelines for ICS analysis.



Ⓐ Ⓑ Ⓒ Internal Compound Stability: For a given pair of entry points and exit points (block/block interface only), a series of circular failure surfaces are generated and analyzed. Included in the analysis is the contribution of (when applicable);

- Block/Block Shear Capacity
- Block/Reinforcement/Block Shear Capacity
- Connection Capacity
- Reinforcement Pullout Capacity
- Leveing Pad sliding resistance

Ⓓ Internal Compound Stability (Bottom Block): Only circular failure surfaces tangent to the base of the bottom block will be analyzed.

Ⓔ Outside entry point limits: This mode of failure will not be considered in the ICS analysis.

Ⓕ Global Stability: This mode of failure will not be considered in the ICS analysis.

Figure 1 - Global Stabiliy Failures

Factors Affecting the Global Stability and Internal Compound Stability (ICS) of SRWs

SOIL CHARACTERISTICS—Weak foundation soils increase the potential for deep-seated stability problems. Low strength reinforced soil will contribute to compound stability problems and low strength retained soils may contribute to either deep-seated or compound failure modes.

GROUNDWATER TABLE—If the groundwater table is shallow (i.e., close to the toe of the wall) the long-term shear strength (i.e., effective shear strength) of the foundation soil will be reduced. This reduction in strength is directly related to the buoyant effect of the groundwater. The effective weight of the soil is reduced by approximately 50%, which reduces the shear strength along the failure surface.

GEOMETRY—A sloping toe at the bottom of an SRW reduces the resisting forces when analyzing failure surfaces exiting in front of the SRW (deep-seated or compound). As the resisting force decreases, the global factor of safety also decreases. The ICS does not evaluate the influence of front slopes on the stability of SRWs.

Figure 2 illustrates the design case for a parametric analysis with top and toe slopes condition for a 10-ft (3.05-m) high wall with a horizontal crest slope founded on a foundation soil with a friction angle of 30°.

Figure 3 shows the change in factor of safety for deep-seated failure as a function of the toe slope angle. However, ICS analysis is not influenced by these

changes and remains constant for the different toe variations.

An increase of the slope above the wall decreases the SRW global stability factor of safety. Figure 4 shows the change in factor of safety for the design case used earlier (with the exception that the toe is level and the crest slope varies). In this case, evaluation of the wall with this geometry shows a larger reduction in safety factor for ICS than for global stability.

TIERED WALLS—The *NCMA Design Manual for Segmental Retaining Walls* provides specific guidelines for tiered SRWs with respect to the spacing between tiers and the effect of the upper wall on the internal and external stability of the lower wall (see Figure 5). When the setback of the upper wall, *J*, is greater than the height of the lower wall, H_1, the internal design of the lower wall is not affected by the upper wall. However, this is not true for global stability. Global stability must be checked for all tiered walls.

Figure 6 shows the variation in the global factor of safety for two 10-ft (3.05-m) high tiered walls with horizontal crest slopes as a function of the setback *J*. In this example, the reinforcement length for both walls is 12 ft (3.66 m), which is 0.6 times the combined height of both walls. For this particular example, constructing a tiered wall versus a single wall 20 ft (6.10 m) high (i.e., $J = 0$) reduces the global factor of safety from 1.3 to 1.2. From the ICS analysis, a tiered wall has better safety factors and the stability is increased when the distance between tiers is increased.

SOIL REINFORCEMENT—Generally speaking, increasing the spacing between reinforcement layers increases the potential for compound failures. Shortening the length of the reinforcement will also increase the potential for both compound and deep-seated failure. Changes in the design strength of the reinforcement often have the smallest impact on the global stability.

The global stability analysis (deep-seated and compound) of an SRW is an important consideration during the SRW design stage in order to assess the overall wall performance and the coherence of the system. Whenever the structure is influenced by weak soils, ground water tables, slopes at the top or toe of the structure or seismic conditions, an experienced professional should verify that all possible failure conditions have been evaluated.

A full discussion of this topic can be found in the NCMA publication, *TEK 15-04B Segmental Retaining Wall Global Stability.* The full TEK library can be accessed for free on the NCMA website (www.ncma.org). CMD

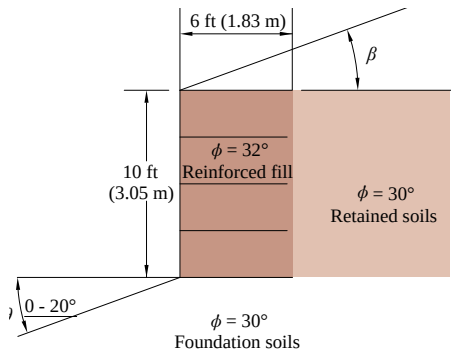


Figure 2—Typical Section for Figures 3 and 4

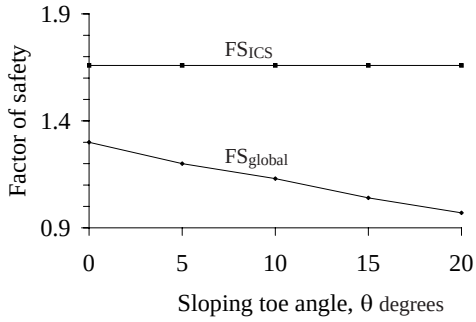


Figure 3—Effect of Sloping Toe Angle

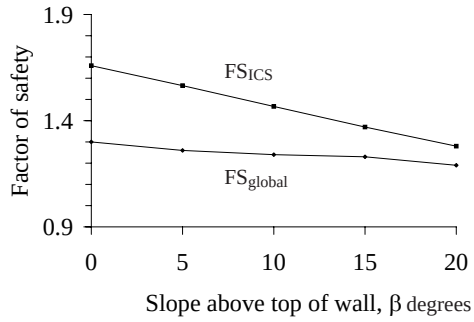


Figure 4—Effect of Slope Above Top of Wall

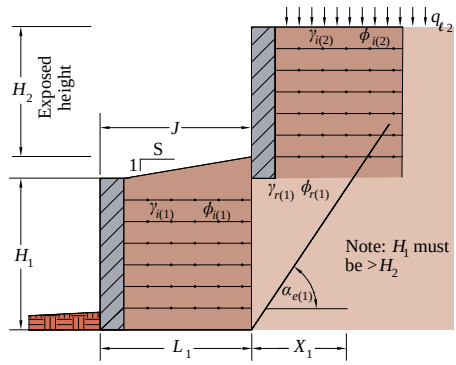


Figure 5—Tiered SRW

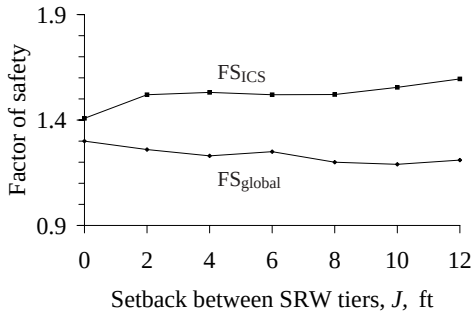


Figure 6—Effect of Tiered SRW Setback

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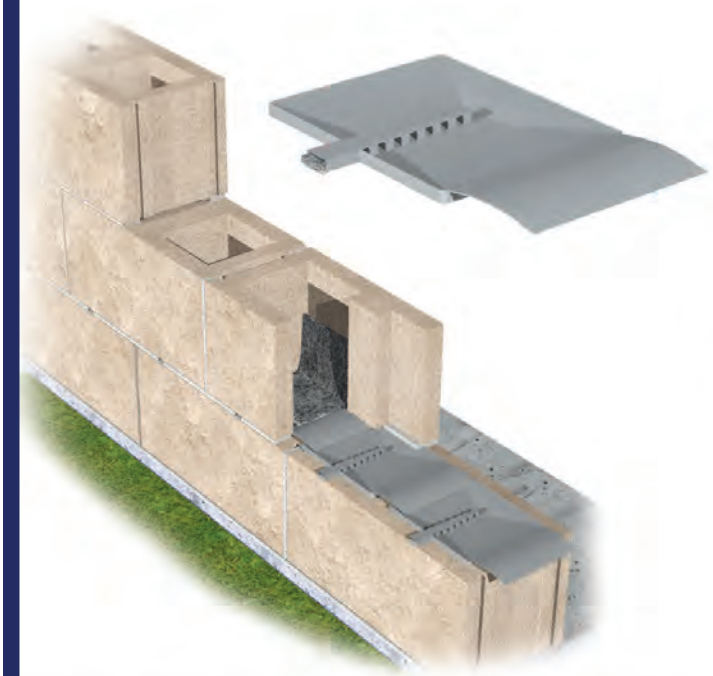

Patent No. 6,912,820 & No. 7,448,175

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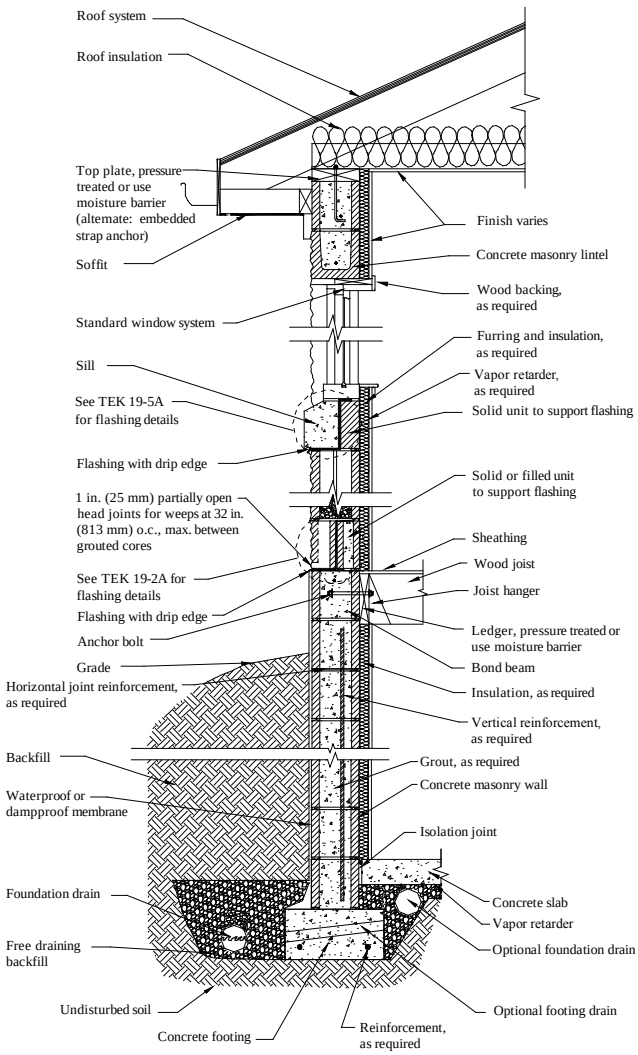


Concrete masonry homes reflect the beauty and durability of concrete masonry materials. Masonry housing provides a high standard of structural strength, design versatility, energy efficiency, termite resistance, economy and aesthetic appeal.

A wide range of architectural styles can be created using both architectural concrete masonry units and conventional units. Architectural units are available with many finishes, ranging from the rough-hewn look of split-face to the polished appearance of groundface units, and can be produced in many colors and a variety of sizes. Concrete masonry can also be finished with brick, stucco or any number of other finish systems if desired. But many designers and consumers are choosing to keep the blocks exposed.

Concrete masonry's mass provides many consumer benefits. It has a high sound dampening ability, is energy efficient, fire and insect proof, durable, and can easily be designed to resist hurricaneforce winds and earthquakes.

Single wythe walls offer the economy of providing structure and an architectural facade in a single building element. They supply all of the attributes of concrete masonry construction with the thinnest possible wall section. To enhance the performance of this wall system, two areas in particular need careful consideration during design and construction—water penetration resistance and energy efficiency.

The figure to the left shows a residential wall section with exposed concrete masonry on the exterior and a furred-out and insulated interior. Concrete masonry can be exposed on the interior as well. In this case, integral insulation (placed in the masonry cores) can be used as required.

Many publications to assist with the design of concrete masonry walls are available for free at www.ncma.org.

Design for water resistance is discussed in detail in TEK 19-1, *Water Repellents for Concrete Masonry Walls*, TEK 19-2A *Design for Dry Single-Wythe Concrete Masonry Walls*, and TEK 19-5A *Flashing Details for Concrete Masonry Walls*.

A full discussion of options for energy efficient concrete masonry walls is contained in NCMA's latest publication, *Thermal Catalog of Concrete Masonry Assemblies, Second Edition*. CMD

NATIONAL CONCRETE MASONRY ASSOCIATION
Sustainable Concrete Products for Structures and Hardscapes

SOUND TRANSMISSION CLASS RATINGS FOR CONCRETE MASONRY WALLS

TEK 13-1C
Sound (2012)

INTRODUCTION

Unwanted noise can be a major distraction, whether at school, work or home. Concrete masonry walls are often used for their ability to isolate and dissipate noise. Concrete masonry offers excellent noise control in two ways. First, it effectively blocks airborne sound transmission over a wide range of frequencies. Second, concrete masonry effectively absorbs noise, thereby diminishing noise intensity. Because of these abilities, concrete masonry has been used successfully in applications ranging from party walls to hotel separation walls, and even highway sound barriers.

Sound is caused by vibrations transmitted through air or other mediums, and is characterized by its frequency and intensity. Frequency (the number of vibrations or cycles per second) is measured in hertz (Hz). Intensity is measured in decibels (dB), a relative logarithmic intensity scale. For each 20 dB increase in sound there is a corresponding tenfold increase in pressure.

This logarithmic scale is particularly appropriate for sound because the perception of sound by the human ear is also logarithmic. For example, a 10 dB sound level increase is perceived by the ear as a doubling of the loudness.

The speed of sound through a particular medium, such as a party wall, depends on both the density and stiffness of the medium. All solid materials have a natural frequency of vibration. If the natural frequency of a solid is at or near the frequency of the sound which strikes it, the solid will vibrate in sympathy with the sound, which will be regenerated on the opposite side. The effect is especially noticeable in walls or partitions that are light, thin or flexible. Conversely, the vibration is effectively stopped if the partition is heavy and rigid, as is the case with concrete masonry walls. In this case, the natural frequency of vibration is relatively low, so only sounds of low frequency will cause sympathetic vibration. Because of its mass (and resulting inertia) and rigidity, concrete masonry is especially effective at reducing sound transmission.

DETERMINING SOUND TRANSMISSION CLASS (*STC*) FOR CONCRETE MASONRY

Sound transmission class (*STC*) provides an estimate of the acoustic performance of a wall in certain common airborne sound insulation applications.

The *STC* of a wall is determined by comparing sound transmission loss (*STL*) values at various frequencies to a standard contour. *STL* is the decrease or attenuation in sound energy, in dB, of airborne sound as it passes through a wall. In general, the *STL* of a concrete masonry wall increases with increasing frequency of the sound.

Many sound transmission loss tests have been performed on various concrete masonry walls. These tests have indicated a direct relationship between wall weight and the resulting *STC*—heavier concrete masonry walls have higher *STC* ratings. A wide variety of *STC* ratings is available with concrete masonry construction, depending on wall weight, wall construction and finishes.

In the absence of test data, standard calculation methods exist, which tend to be conservative. *Standard Method for Determining Sound Transmission Ratings for Masonry Walls*, TMS 0302 (ref. 1), contains procedures for determining *STC* values of concrete masonry walls. According to the standard, *STC* can be determined by field or laboratory testing in accordance with standard test methods or by calculation. The calculation in TMS 0302 is based on a best-fit relationship between concrete masonry wall weight and *STC* based on a wide range of test results:

$$STC = 20.5W^{0.234} \qquad \text{Eqn. 1}$$

[SI: $STC = 14.1W^{0.234}$]

Equation 1 is applicable to uncoated fine- or medium-textured concrete masonry and to coated coarse-textured concrete masonry. Because coarse-textured units may allow airborne sound to enter the wall, they require a surface treatment to seal at least one side of the wall. At least one coat of acrylic latex, alkyd or cement-based paint, or plaster are specifically called out in TMS 0302, although other coatings that effectively seal the surface are

Related TEK:
13-2A

Keywords: : acoustics, noise control, sound insulation, sound transmission class, sound transmission loss, *STC*, *STL*, testing

also acceptable. One example is a layer of drywall with sealed penetrations, as shown in Figure 2. Architectural concrete masonry units are considered sealed without surface treatment for the purposes of using Equation 1.

Equation 1 also assumes the following:

1. walls have a thickness of 3 in. (76 mm) or greater,
2. hollow units are laid with face shell mortar bedding, with mortar joints the full thickness of the face shell,
3. solid units are fully mortar bedded, and
4. all holes, cracks and voids in the masonry that are intended to be filled with mortar are solidly filled.

Calculated values of *STC* are listed in Table 1.

Because the best-fit equation is based solely on wall weight, the calculation tends to underestimate the STC of masonry walls that incorporate dead air spaces, which contribute to sound attenuation. See the following section for the effect of drywall with furring spaces on STC.

For multi-wythe walls where both wythes are concrete masonry, the weight of both wythes is used in Equation 1 to determine *STC*. For multi-wythe walls having both concrete masonry and clay brick wythes, however, a different procedure must be used, because concrete and clay masonry have different acoustical properties. In this case, Equation 2, representing a best-fit relationship for clay masonry, must also be used. To determine a single *STC* for the wall system, first calculate the *STC* using both Equations 1 and 2, based on the combined weight of both wythes, then linearly interpolate between the two resulting *STC* ratings based on the relative weights of the wythes. Equation 2 is the *STC* equation for clay masonry (ref. 1):

$$STC = 19.6W^{0.230} \qquad \text{Eqn. 2}$$

[SI: $STC = 13.6W^{0.230}$]

For example, consider a masonry cavity wall with an 8-in. (203-mm) concrete masonry backup wythe (W = 33 psf, 161 kg/m^2) and a 4-in. (102-mm) clay brick veneer (W = 38 psf, 186 kg/m^2).

STC (Eqn. 1) = $20.5(33 + 38)^{0.234} = 55$

STC (Eqn. 2) = $19.6(33 + 38)^{0.230} = 52$

Interpolating:

$STC = 55(33/71) + 52(38/71) = 53$

When *STC* tests are performed, the TMS 0302 requires the testing to be in accordance with ASTM E90, *Standard Test Method for Laboratory Measurement of Airborne Sound Transmission Loss of Building Partitions and Elements* (ref. 2) for laboratory testing or ASTM E413, *Standard Classification for Rating Sound Insulation* (ref. 3) for field testing.

CONTRIBUTION OF DRYWALL

Drywall attached directly to the surface of a concrete masonry wall has very little effect on sound attenuation other than the same benefit as sealing the surface. Adding $^1/_2$ or $^5/_8$ in. (13 or 16 mm) gypsum wall board to one side of the wall with an unfilled furring space will generally result in a slight increase in *STC*. However, when placed on both sides of the wall with a furring space of less than 0.8 in. (19 mm) a reduction in *STC* is realized due to mass-air-mass resonance similar to the action of drum. Better results are realized when the furring space is filled with sound insulation. Sound insulation consists of fibrous materials, such as cellulose fiber, glass fiber or rock wool insulation, are good materials for absorbing sound; closed-cell materials, such as expanded polystyrene, are not, as they do not

Table 1—Calculated *STC* Ratings for Concrete Masonry Walls (ref. 1)

Nominal unit thickness, in. (mm)	Density, pcf (kg/m^3)	*STC*[A]			
		Hollow unit	Grout-filled unit	Sand-filled unit	Solid unit
4 (102)	85 (1,362)	40	45[B]	44	44
	95 (1,522)	41	46[B]	44	45
	105 (1,682)	42	46[B]	45	46
	115 (1,842)	43	47[B]	46	46
	125 (2,002)	44	48[B]	46	47
	135 (2,162)	45	48[B]	47	48
6 (152)	85 (1,362)	42	51	48	48
	95 (1,522)	43	51	49	49
	105 (1,682)	44	52	50	50
	115 (1,842)	45	52	50	51
	125 (2,002)	45	53	51	52
	135 (2,162)	46	53	51	53
8 (203)	85 (1,362)	44	55	52	52
	95 (1,522)	45	55	52	53
	105 (1,682)	46	56	53	54
	115 (1,842)	47	56	54	55
	125 (2,002)	48	57	54	56
	135 (2,162)	49	57	55	57
10 (254)	85 (1,362)	46	58	55	55
	95 (1,522)	48	58	55	56
	105 (1,682)	49	59	56	57
	115 (1,842)	50	59	57	58
	125 (2,002)	50	60	57	59
	135 (2,162)	51	60	58	60
12 (305)	85 (1,362)	48	61	57	57
	95 (1,522)	49	61	58	58
	105 (1,682)	50	62	58	60
	115 (1,842)	51	62	59	61
	125 (2,002)	52	63	60	62
	135 (2,162)	53	63	60	63

[A] Based on: grout density of 140 lb/ft^3 (2,243 kg/m^3); mortar density of 130 lb/ft^3 (2,082 kg/m^3) sand density of 90 lb/ft^3 (1,442 kg/m^3); unit percentage solid from mold manufacturer's literature for typical units (4-in. (100-mm) 73.8% solid, 6-in. (150-mm) 55.0% solid, 8-in. (200-mm) 53.0% solid, 10-in. (250-mm) 51.7% solid, 12-in. (300-mm) 48.7% solid). Other unit configurations may have different *STC* values. *STC* values for grout-filled and sand-filled units assume the fill materials completely occupy all voids in and around the units. *STC* values for solid units are based on all mortar joints solidly filled with mortar.

[B] Because of small core size and the resulting difficulty consolidating grout, these units are rarely grouted.

NATIONAL CONCRETE MASONRY ASSOCIATION
Sustainable Concrete Products for Structures and Hardscapes

DESIGN OF CONCRETE MASONRY INFILL

TEK 14-23
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

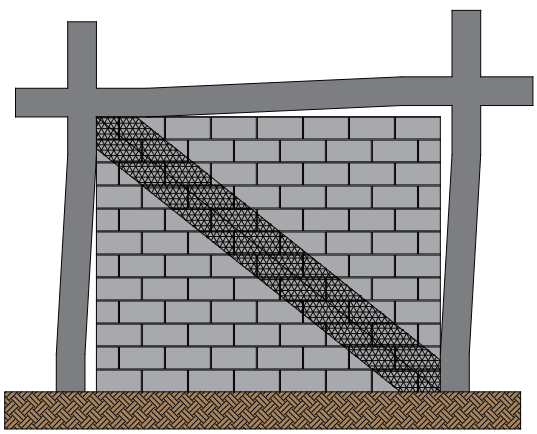
IN-PLANE SHEAR FOR PARTICIPATING INFILLS

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:

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

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.

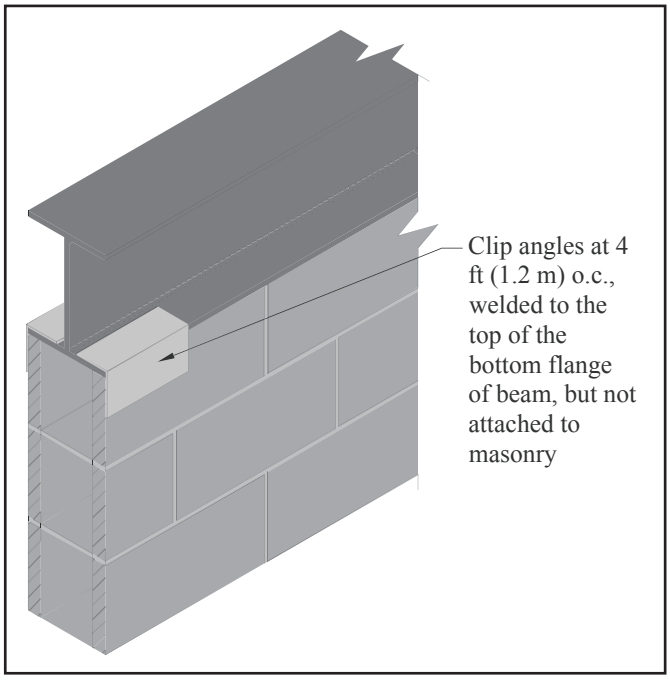


Figure 2—Example of Mechanical Infill Connector

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

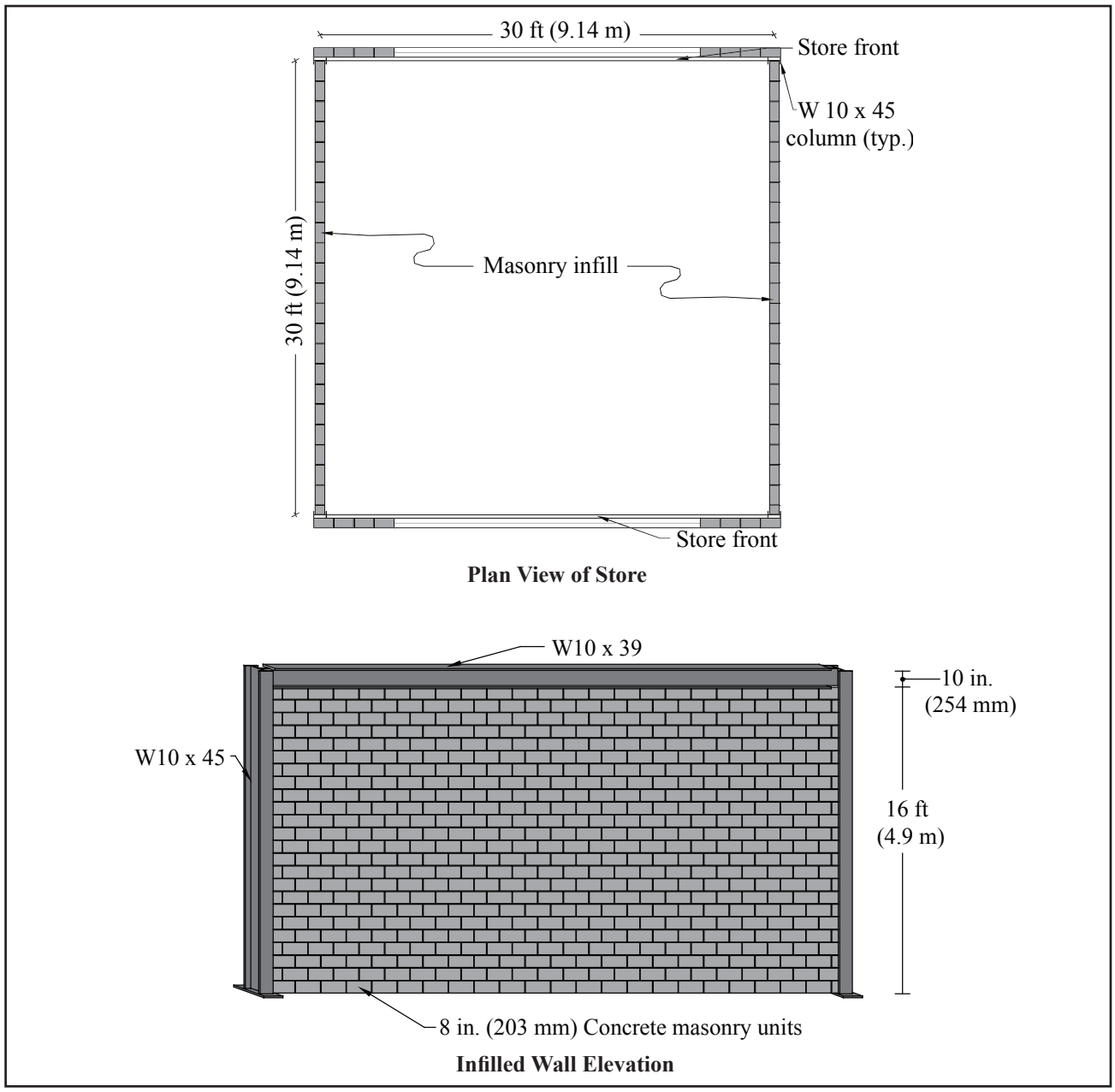


Figure 3—Convenience Store Layout for Design Examples 1 & 2

- 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 lb) = 3,030 lb (13.5 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 in./8 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 in./360 in.) = 28.1°

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 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 in.)$t_{net\ inf}\ f'_m$ = (6.0 in.)(2.5 in.)(1,500 psi) = 22,500 lb
- (56,030 lb/in.)(1 in.) = 56,030 lb
- $(3.8\sqrt{f'_m}\ A_n)/1.5 = [3.8\sqrt{1{,}500\ \text{psi}}$ (30 ft x 30 in.^{2}/ft)]/1.5 = 88,304 lb
- $(300A_n)/1.5$ = [300(30 ft x 30 in.^{2}/ft)]/1.5 = 180,000 lb
- $(56A_n + 0.45N_v)/1.5$ = [56(30 ft x 30 in.^{2}/ft) + 0.45 x 0]/1.5 = 33,600 lb

V_n = 22,500 lb (100 kN)

The design shear capacity is:
ϕV_n = 0.6 x 22,500 lb = 13,500 lb (60 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

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

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

= 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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significantly absorb sound (refs. 1, 7). Note that most of these materials are susceptible to moisture so care must be taken when applying these types of insulation to exterior walls.

Equations to determine the change in *STC* when adding drywall are as follows (Table 2 lists calculated values of Δ*STC* based on Equations 3 through 6):

- For drywall on one side of the wall with no sound absorbing material in the furring space:
  $\Delta STC = 2.8d - 1.22$ Eqn. 3
  [SI: $\Delta STC = 0.11d - 1.22$]
- For drywall on both sides of the wall and no sound absorbing material in the furring spaces:
  $\Delta STC = 3.6d - 2.78$ Eqn. 4
  [SI: $\Delta STC = 0.14d - 2.78$]
- For drywall on one side of the wall with sound absorbing material in the furring space:
  $\Delta STC = 3.0d + 1.87$ Eqn. 5
  [SI: $\Delta STC = 0.12d + 1.87$]
- For drywall on both sides of the wall and sound absorbing material in the furring spaces:
  $\Delta STC = 11.2d - 7.37$ Eqn. 6
  [SI: $\Delta STC = 0.44d - 7.37$]

BUILDING CODE REQUIREMENTS

The *International Building Code* (ref. 4) contains requirements to regulate sound transmission through interior partitions separating adjacent dwelling units and separating dwelling units from adjacent public areas, such as hallways, corridors, stairs or service areas. Partitions serving the above purposes must have a sound transmission class of at least 50 dB for airborne noise when tested in accordance with ASTM E90. If field tested, an *STC* of 45 must be achieved. In addition, penetrations and openings in these partitions must be sealed, lined or otherwise treated to maintain the *STC*. Guidance on achieving this for masonry walls is contained below in Design and Construction.

The *International Residential Code* (ref. 5) contains similar requirements, but with a minimum *STC* rating of 45 dB when tested in accordance with ASTM E90 for walls and floor/ceiling assemblies separating dwelling units.

DESIGN AND CONSTRUCTION

In addition to *STC* values for walls, other factors also affect the acoustical environment of a building. For example, a higher *STC* may be warranted between a noisy room and a quiet one than between two noisy rooms. This is because there is less background noise in the quiet room to mask the noise transmitted through the common wall.

Seemingly minor construction details can also impact the acoustic performance of a wall. For example, screws used to attach gypsum wallboard to steel furring or resilient channels should not be so long that they contact the face of the concrete masonry substrate, as this contact area becomes an effective path for sound vibration transmission.

TMS 0302 includes requirements for sealing openings and joints to ensure these gaps do not undermine the sound transmission characteristics of the wall. These requirements are described below and illustrated in Figures 1 and 2.

Through-wall openings should be completely sealed, After first filling gaps with foam, cellulose fiber, glass fiber, ceramic fiber or mineral wool. Similarly, partial wall penetration openings and inserts, such as electrical boxes, should be completely sealed with joint sealant.

Control joints should also be sealed with joint sealants to minimize sound transmission. The joint space behind the sealant backing can be filled with mortar, grout, foam, cellulose fiber, glass fiber or mineral wool (see Figure 2).

To maintain the sound barrier effectiveness, partitions should be carried to the underside of the structural slab, and the joint between the two should be sealed against sound transmission in a way that allows for slab deflection. If the roof or floor is metal deck rather than concrete, joint sealants alone will not be effective due to the shape of the deck flutes. In this case, specially shaped foam filler strips should be used. For fire and smoke containment walls, safing insulation should be used instead of foam filler strips.

Additional nonmandatory design and building layout considerations will also help minimize sound transmission. These are covered in detail in TEK 13-2A (ref. 6).

NOTATIONS

Δ*STC* = the change in *STC* rating compared to a bare concrete masonry wall

d = the thickness of the furring space (when drywall

Table 2—Increase in *STC* Ratings Due to Furring Space and Drywall (ref. 1)

Furring space condition:	Drywall on:	Δ*STC* for furring space thickness[A] (in., (mm)) of:							
		0.5 (13)	0.8 (19)	1 (25)	1.5 (38)	2 (51)	2.5 (64)	3 (76)	3.5 (89)
No sound-absorbing material in the furring space	one side	0.2	0.9	1.6	3.0	4.4	5.8	7.2	8.6
	both sides[A]	-1.0	-0.1	0.8	2.6	4.4	6.2	8.0	9.8
Furring space filled with sound-absorbing material[B]	one side	3.4	4.1	4.9	6.4	7.9	9.4	10.9	12.4
	both sides[A]	-1.8	1.0	3.8	9.4	15.0	20.6	26.2	31.8

[A] When drywall is used on both sides of the masonry, use the thickness of the furring space on one side of the wall to determine Δ*STC*. The furring space and insulation condition must be the same on both sides to use this provision.

[B] Fibrous materials, such as cellulose fiber, glass fiber or rock wool insulation, are good materials for absorbing sound; closed-cell materials, such as expanded polystyrene, are not, as they do not significantly absorb sound.

is used on both sides of the masonry, *d* is the thickness of the furring space on one side of the wall only), in. (mm)

STC = Sound Transmission Class

STL = Sound Transmission Loss

W = the average wall weight based on the weight of the masonry units; the weight of mortar, grout and loose fill material in voids within the wall; and the weight of surface treatments (excluding drywall) and other components of the wall, psf (kg/m^2)

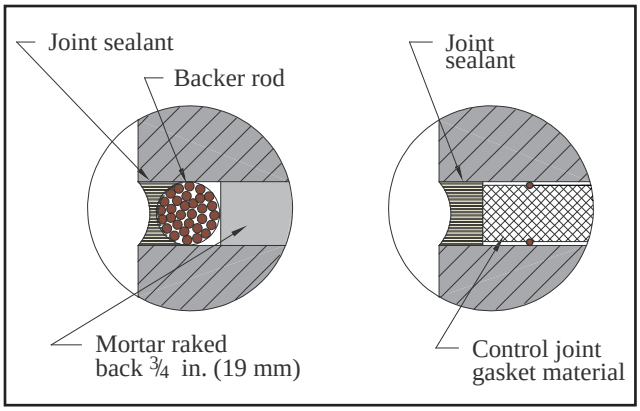

Figure 1—Sealing Wall Intersections & Control Joints

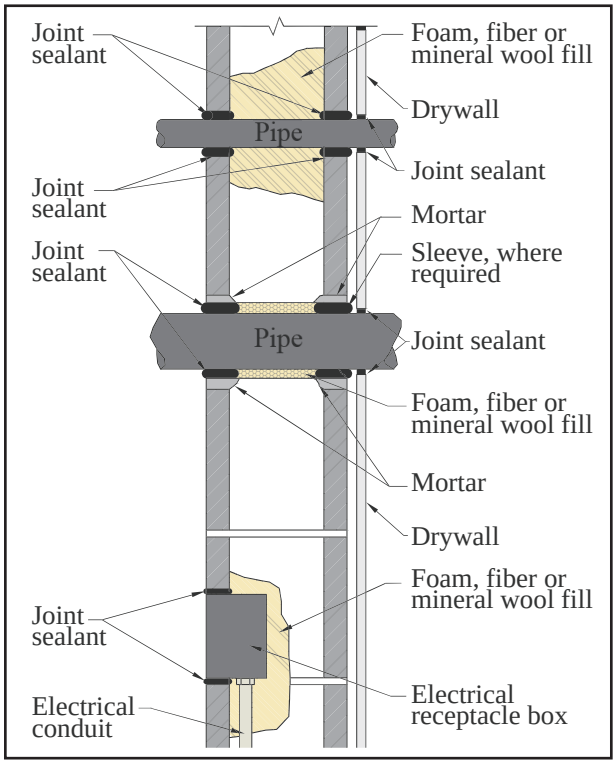

Figure 2—Sealing Around Penetrations & Fixtures

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Architecture/Engineering/Construction Industry Update

ASTM C90-11b - Hollow Loadbearing Concrete Masonry Units

In late 2011 ASTM C90, the material specification for hollow loadbearing CMU, was modified to permit the cross-webs of units to be configured in different ways to meet specific project needs and performance requirements. As this allowance works its way into local construction, here is a short review of how this new flexibility on unit configurations can benefit your next project.

What's changed?

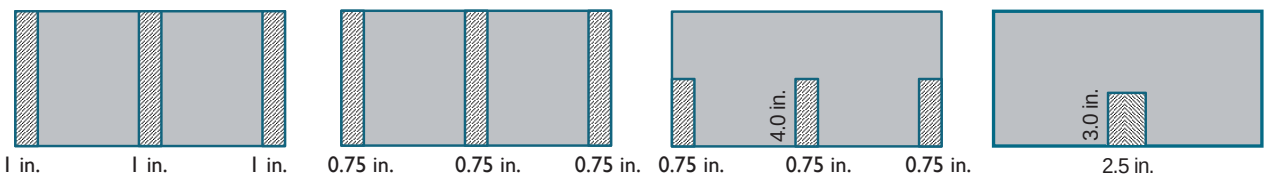
Under new C90 requirements, web configurations are no longer regulated by their thickness, but rather the cross-sectional area of the web connecting the face shells of a unit.

Literally, this new requirement means that for every square foot of unit surface, no less than 6.5 in.^{2} (4193 mm^2) of web must connect the front and back face shells, with no web measuring less than 0.75 in. (19 mm) in thickness.

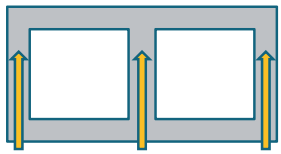
Examples of acceptable web configurations

Allowing the option to reduce the cross-web thickness and/or area, allows for a greater array of unit configurations. Not every unit configuration is available in every market. Contact your local CMU manufacturer to explore the range of possibilities available.

Cross-Section Views



Potential impact of ASTM C90 change



Smaller webs = Larger R-values

Increased R-Values

Reducing the cross-webs has the potential to substantially increase the energy efficiency of concrete masonry assemblies by reducing the thermal bridges that result from the cross-webs. R-values can increase 2 to 3 times above baseline; while retaining all the intrinsic benefits of thermal mass.

Fire Ratings

Because the most common method of determining fire ratings is based on the Code-approved method of equivalent thickness (the amount of concrete that remains if the unit was recast without voids), reducing the size and thickness of cross-webs reduces the equivalent thickness of the unit, which for a given mix design will reduce the calculated fire resistance rating of the assembly. Nevertheless, units can be produced to meet any fire resistance rating required by code and can always use the equivalent thickness method to calculate the fire resistance rating.

To assure wall performance, specific fire ratings for walls should be identified on drawings/wall sections and listed in project specifications.

Sustainable Attributes

Using less material in production reduces: the demand on resources; the energy necessary to manufacture products; and the fuel required to transport units to job sites—while maintaining the high durability, low impact solution inherent in concrete masonry.

Structural

This change does not reduce the structural performance for load bearing masonry. Other unit properties such as face shell thickness and unit compressive strength remain unchanged.

In addition, larger cell areas reduce the reinforcement congestion and facilitates grout placement.

Impact on Wall Cost

Lighter weight units can increase construction productivity and reduce worker fatigue and injuries. This can lead to lower costs for concrete masonry wall systems.

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What are the exact changes to ASTM C90?

Here is the old table showing revisions to be incorporated (before some editorial tweaking from the Committee).

TABLE 1 Minimum ~~Thickness of~~ Face Shells and Webs Requirements[A]

Nominal Width (W) of Units, in. (mm)	Face Shell Thickness (t_{fs}), min, in. (mm)[B,C]	Webs ~~Thickness (t_w)~~	
		Web Thickness[C] (t_w), ~~Webs[B,D,C]~~ min, in. (mm)	~~Equivalent Web Thickness, min, in./linear ft[E] (mm/linear m)~~ Web Area (A_w), min, in.^{2}/ft^2 (mm^2/m^2)[D]
~~3 (76.2) and 4 (102)~~	~~¾ (19)~~	~~¾ (19)~~	~~1 5/8 (136)~~
~~6 (152)~~	~~1 (25)~~	~~1 (25)~~	~~2 ¼ (188)~~
~~8 (203)~~	~~1 ¼ (32)~~	~~1 (25)~~	~~2 ¼ (188)~~
~~10 (254) and greater~~	~~1 ¼ (32)~~	~~1 1/8 (29)~~	~~2 ½ (209)~~
3 (76.2) and 4 (102)	¾ (19)	¾ (19)	6.5 (45,140)
6 (152)	1 (25)	¾ (19)	6.5 (45,140)
8 (203) and greater	1 ¼ (32)	¾ (19)	6.5 (45,140)

[A] Average of measurements on a minimum of 3 units when measured as described in Test Methods C140.
[B] When this standard is used for units having split surfaces, a maximum of 10 % of the split surface is permitted to have thickness less than those shown, but not less than 3/4 in. (19.1 mm). When the units are to be solid grouted, the 10 % limit does not apply and Footnote C establishes a thickness requirement for the entire face shell.
[C] When the units are to be solid grouted, minimum face shell and web thickness shall be not less than 5/8 in. (16 mm).
~~[D] The minimum web thickness for units with webs closer than 1 in. (25.4 mm) apart shall be 3/4 in. (19.1 mm).~~
[D~~E~~] Minimum web cross-sectional area ~~Equivalent web thickness~~ does not apply to the portion of the unit to be filled with grout. The length of that portion shall be deducted from the overall length of the unit for the calculation of the

And here is the final approved table reflected in ASTM C90-11b.

TABLE 1 Minimum Face Shells and Web Requirements[A]

Nominal Width (W) of Units, in. (mm)	Face Shell Thickness (t_{fs}), min, in. (mm)[B,C]	Webs	
		Web Thickness[C] (t_w), min, in. (mm)	Normalized Web Area (A_{nw}), min, in.^{2}/ft^2 (mm^2/m^2)[D]
3 (76.2) and 4 (102)	¾ (19)	¾ (19)	6.5 (45,140)
6 (152)	1 (25)	¾ (19)	6.5 (45,140)
8 (203) and greater	1¼ (32)	¾ (19)	6.5 (45,140)

[A] Average of measurements on a minimum of 3 units when measured as described in Test Methods C140.
[B] When this standard is used for units having split surfaces, a maximum of 10 % of the split surface is permitted to have thickness less than those shown, but not less than ¾ in. (19.1 mm). When the units are to be solid grouted, the 10 % limit does not apply and Footnote C establishes a thickness requirement for the entire faceshell.
[C] When the units are to be solid grouted, minimum face shell and web thickness shall be not less than ⅝ in. (16 mm).
[D] Minimum normalized web area does not apply to the portion of the unit to be filled with grout. The length of that portion shall be deducted from the overall length of the unit for the calculation of the minimum web cross-sectional area.

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ASSOCIATION NEWS

NCMA BOARD OF DIRECTORS ELECTIONS

Congratulations are extended to the following newly elected board members:

- Region I = Brendan Quinn, Ernest Maier, Inc. Bladensburg, MD., and Gregory McElwee, Cinder & Concrete Block Corp, Cockeysville, MD.
- Region II = Kevin Vogler, Block USA, Birmingham, AL.
- Region III = Bruce Loris, Oberfields LLC, Deleware, OH.
- Region IV = Peter Browning, Salina Concrete Products, Salina, KS., and Bob Whisnant, Headwaters Construction Materials, Alleyton, TX.
- Region V = Kendall Anderegg, Mutual Materials Company, Bellevue, WA.
- Region VI = Tony Neves, Brampton Brick Limited, Milton, Ontario, CAN.

New board members will begin their term of service at the conclusion of the annual meeting in Indianapolis on Jan. 13, 2013. These dedicated professionals serve voluntarily at the expense of their respective organizations and their service is sincerely appreciated.

ENTER YOUR PLANT FOR THE 2012 NCMA SAFETY AWARDS

Reduced entry fees, no stuffy banquet to attend – it has never been easier to obtain that coveted safety award plaque to show OSHA when they come by to inspect your plant. Because of the earlier than normal 2013 Annual Meeting (early January), the 2012 Safety Awards will be held after the convention to allow members adequate time to submit their applications and for staff to prepare the award plaques. The safety awards will be announced in the March/April issue of *Concrete Masonry Designs Magazine,* and award plaques will be shipped to the winning companies. Also, because there is no awards banquet this year, entry fees have been substantially reduced from the 2011 amount. Small producers are highly encouraged to submit applications as well – show OSHA your commitment to safety. Visit www.ncma.org for more information and the entry form. If you have any questions, contact NCMA Director of Technical Publications Dennis Graber.

2ND EDITION THERMAL CATALOG NOW AVAILABLE

The NCMA Thermal Catalog of Concrete Masonry Assemblies — 2nd Edition is now available on the NCMA website for download. The updated version of the catalog includes thermal calculations using units complying with the new ASTM C90 requirements for web area. The catalog includes three sections with differing CMU configurations, to provide more options for designers to take advantage of the energy efficiencies of concrete masonry construction. To access this catalog go to www.ncma.org/resources.

NCMA MEMBERS CELEBRATE COLUMBIA MACHINE'S 75TH ANNIVERSARY

Members of the National Concrete Masonry Association (NCMA) celebrated Columbia Machine's 75th Anniversary during its Executive and Long Range Planning meetings held at The Nines Hotel in Portland, Oregon, November 15-16, 2012.

As part of the celebration Columbia Machine hosted the NCMA members at an open house at their Vancouver, Washington facility on Thursday, November 15th. The celebration included a tour of the operation and dinner at the 320,000-square-foot (30,000 square-meter) facility. During the evening, NCMA President Robert Thomas presented company President and CEO Rick Goode with a celebratory plaque and a United States Flag which was flown over the U.S. Capitol in honor of Columbia Machine's 75 years of service to the construction industry. Columbia also hosted a brewery tour for the group on Friday, November 16th at several local breweries. The evening culminated with a dinner at the Bridgeport Brewery which houses Columbia Machine product handling and packaging equipment for their brewing operation.

MEMBERS IN THE NEWS

CarbonCure — BuildingGreen.com recently announced that CarbonCure's CO^2-absorbed Concrete Masonry Unit won its widely cited "Top 10 Building Products" 2013 award. "CarbonCure and its partners have proven that absorbed CO^2 into concrete, in a mass production environment, is both cutting-edge and practical. The whole direction of LEED is moving toward lower embodied inputs and carbon lifecycle assessment," said Alex Wilson, veteran green building leader and editor of Buildinggreen.com.

Columbia Machine was founded in 1937 by Fred Neth Sr., who opened Columbia Forge and Machine Works when he was 23 years old. In 1945, the company pioneered a hydraulically operated concrete block-making machine. Many years later, the company modified that technology, creating a "palletizer" that could stack consumer products. "We made our first palletizer for Lucky brewery in Vancouver," Goode said.

Nowadays, thousands of the company's palletizers move and stack everything from beer and paper towels to fruit juice and bottled water or, as Goode put it, "pretty much any consumer product on a pallet in Costco." And while it celebrates 75 years in business in 2012, Goode said, the company has every intention of sticking around for at least 75 more.

NCMA THANKS GENEROUS SPONSORS FOR THE 2013 CONVENTION

TheNational Concrete Masonry Association wishes to thank several companies for their generous support of the 2013 NCMA Annual Convention in Indianapolis.

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GOVERNMENT AFFAIRS NEWS

FEDERAL RULEMAKING EXPECTED TO HEAT UP IN 2013

Rulemaking by the federal regulatory agencies is poised to heat up again now that the elections are over. Last week, Deputy Assistant Secretary of Labor Jordan Barab told the National Advisory Council for Occupational Safety and Health that OSHA regulatory enforcement and rulemaking will "exponentially increase" next year. He identified the Injury and Illness Prevention Program and Crystalline Silica as OSHA's top rulemaking priorities. Likewise, if EPA gets its way, the agency is expected to publish its long delayed final rule on coal combustion residuals rule in 2013. Last-minute procedural developments by the government could delay the three rules a little while farther into 2013, but informed sources say that all are likely to be finalized before the end of next year. On a more positive note, NCMA has learned that adding a Musculoskeletal Disorder Column to the OSHA 300 reporting system has been withdrawn by legislative action.

INDUSTRY NEWS

COHRS ELECTED CHAIRMAN OF PORTLAND CEMENT ASSOCIATION

The Portland Cement Association board of directors elected Cary O. Cohrs as chairman during the association's fall board meeting in Washington, D.C. Cohrs succeeds Aris Papadopoulos of Titan America. John Stull, president and CEO of Lafarge North America Inc., was elected vice chairman. Cohrs is the current president of American Cement Company LLC, a joint venture of CRH's Oldcastle Materials Inc., and Trap Rock Industries Inc., based in Sumterville, Fla. President of American Cement since 2005, he has decades of experience in the cement industry. In 2000, he was appointed vice president of operations for Florida Rock Industries, where Cohrs also served as plant manager and construction manager.

PUBLIC COMMENT PERIOD OPEN FOR TMS 402 MASONRY DESIGN STANDARD

The Masonry Standards Joint Committee has proposed revisions to the 2011 Edition of its Building Code Requirements for Masonry Structures (TMS 402-11/ACI 530-11/ASCE-5-11), Specification for Masonry Structures (TMS 602-11/ACI 530.1-11/ASCE6-11) and their companion commentaries for a planned 2013 edition of the Standards. The MSJC is sponsored by the The Masonry Society, American Concrete Institute, and the Structural Engineering Institute of the American Society of Civil Engineers. In accordance with the rules of The Masonry Society, and consistent with the rules of the other sponsoring organizations, proposed changes to standardized documents must be open for public comment for a period of not less than 45 days. Details on the public comment period, including a summary of major proposed changes to these documents and a complete Working Draft of the proposed revisions to MSJC Code, Specification, and Commentaries can be accessed at the link below. If you wish to submit a comment on the provisions, please use the table provided at www.masonrysociety.org before 11:59 p.m. ET on Jan. 14. If you have questions about the Public Comment Period or the MSJC, contact TMS at 303-939-9700.

FOUNDATION NEWS

DEMAND SURGES AT CLEMSON UNIVERSITY FOR MASONRY DESIGN CLASS

As reported in a previous NCMA e-News Brief article, the NCMA Foundation awarded a grant for an exciting new engineering/architectural student design competition to Clemson University where students will not only design a full-scale single-wythe concrete masonry wall with prescribed openings, but they will then construct their designed walls and test them for structural capacity. This competition is a mandatory term project for the Masonry Structural Design Class. Professor Sez Atamturktur indicates that they were planning on a class capacity of around 20 students based on previous enrollments - but were wrong. They twice moved to larger classrooms, with the last having a capacity of 45. However, they still have students on the waiting list and are looking for an even larger classroom. The competition is endorsed by the Carolinas Concrete Masonry Association who will provide educational block plant tours, materials, facilities for testing, guidance for the judging criteria and skilled masons for hands-on construction training for the students. With this type of success, this project is being envisioned as a prototype for a large scale national block design competition. For more information on NCMA Foundation sponsored student design competitions contact NCMA Director of Technical Publications, Dennis Graber.

NATIONAL BUILDING INFORMATION MODELING FOR MASONRY INITIATIVE (BIM-M) HOLDS ROUNDTABLE MEETINGS IN ATLANTA

The National Building Information Modeling for Masonry Initiative (BIM-M) was organized in 2012 and is working to identify barriers to and strategies for the full implementation of masonry materials and systems into building information modeling (BIM) software for the design and construction industries. BIM and digital technology are poised to revolutionize the design and construction industries and this initiative is the opening effort to include masonry construction.

On September 4-6, 2012, Georgia Institute of Technology's Digital Building Laboratory hosted architects, engineers, contractors, construction managers, material suppliers, software developers, masonry experts, and corporate executives from across the United States in roundtable discussions on implementing BIM into masonry design and construction practices. Over 45 attendees from five working groups assembled to develop criteria in the areas of architectural design with masonry, engineering analysis, construction procurement and scheduling, and construction coordination. The goal of the initiative is for Georgia Tech under the direction of BIM pioneer, Professor Chuck Eastman, to develop a BIM roadmap for the masonry industry early in 2013. The roadmap will establish detailed requirements for the use of BIM and pave the way for future developments by software developers.

The BIM-M initiative is sponsored by the International Union of Bricklayers and Allied Craftworkers (IUBAC), the Mason Contractors Association of America (MCAA), the International Masonry Institute (IMI), the National Concrete Masonry Association (NCMA), the Western States Clay Products Association (WSCPA) and The Masonry Society (TMS) and is being overseen by the Digital Building Laboratory at the Georgia Institute of Technology. For more information on the BIM-M initiative, contact Russell Gentry: russell.gentry@coa.gatech.edu or David Biggs: biggsconsulting@att.net.

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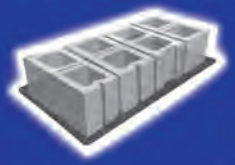

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Continuing Education Learning Program

AIA, LA CES, and Professional Development Hours (PDH) Learning Units Quiz

NOV/DEC 2012

To receive one learning unit, read "Terraced Retaining Walls", "Global Stability and Internal Compound Stability (ICS)" and complete the questions on this page. Return forms before December 2013 to receive learning unit credits.

Questions (Circle the correct answer)

1. Terraced Segmental Retaining Walls (SRWs) are considered independent when:
a. The distance between the walls is more than twice the lower wall height
b. The distance between the walls is more than the height of lower wall
c. Terraced walls are never independent

2. In the SRW project featured, which design modification was used to solve the lack of space to accommodate geosynthetic reinforcement?
a. Rock was blasted to allow for 60% of geosynthetic
b. The SRW units were attached to specially designed earth anchors
c. No special design considerations were included

3. Local ordinances can override industry and/or manufacturer recommendations?
a. True
b. False

4. When would global stability analysis be particularly necessary?
a. When ground water is close to the reinforced soil, steep toe or top slopes, and/or high surcharges are present
b. When the project involves tiered walls, seismic design, and when weak or problematic soils are found around the structure
c. All of the above
d. None of the above

5. There are two primary modes of global stability failure: deep-seated and compound.
a. True
b. False

6. Factor like soil characteristics, ground water table, geometry of the walls or tiered walls do not affect the global and compound stability.
a. True
b. False

7. Ground water reduces the strength of the soils.
a. True
b. False

8. A toe slope reduces the global stability safety factor of a wall more than a top slope
a. True
b. False

9. Increasing the spacing of the soil reinforcement on the SRW will not affect the compound stability
a. True
b. False

10. The space between tiered walls does not affect the global and compound stability of the system
a. True
b. False

CMD712

Information

Name _____

Company _____

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(list all that apply, if applicable - AIA, ASLA, CLARB, or CSLA)

☐ I certify that I have complied with Continuing Education Guidelines and that the above information is true and accurate to the best of my knowledge.

Request Certificate of Completion:

☐ **AIA**

☐ **LA CES**

☐ **PDH**

Send completed quiz to:

National Concrete Masonry Association,
13750 Sunrise Valley Drive
Herndon, VA 20171-4662
or fax to NCMA at 703-713-1910.

If you have questions,
please contact NCMA at 703-713-1900.

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