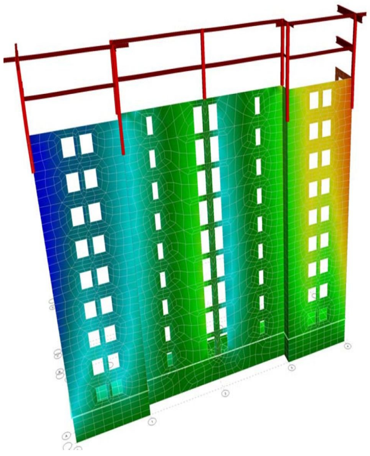
SE INSIGHTS
FOR MASONRY DESIGN

What Non-Engineers Need to Know About Structural Masonry

The use of masonry on a project brings several benefits in its durability, its high fire resistance, sustainability through its use of local raw materials, and it does not require a long lead time during construction. Masonry also has structural benefits in that it has high stiffness and masonry resists compression and shear loads well, making masonry walls a sound structural option. Therefore, if masonry is chosen for its durability and fire resistance, it is helpful to understand all the advantages this material also has structurally, as well as good detailing practices to be aware of, in order to deliver a cost-effective, high-performing masonry solution for projects.

Evolution of Masonry Design Approaches

Masonry, one of the oldest building materials, has seen significant advancements in engineering methodologies. Early design approaches relied on empirical methods, using simplified height-to-thickness ratios and prescriptive tables to determine wall dimensions. Modern practice has shifted towards designing based on actual load requirements and structural geometry, utilizing both allowable stress and strength design principles. The adoption of Finite Element Analysis (FEA) has further enhanced masonry design, enabling more accurate modeling of complex wall configurations, including those with multiple openings or irregular shapes. This transition allows for a more optimized structural design.



Optimizing Compressive Strength, f’m

One significant way to achieve a more efficient masonry structure is to utilize a compressive masonry strength, f’m, that is based not on the minimum Code values, but rather by selecting a higher f’m based on the demand needed for a project.

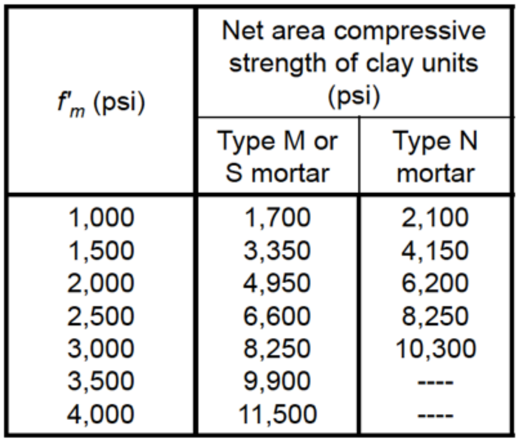
An analogy to consider is concrete design. Consider a concrete-framed building assumes a compressive strength, f’c, of 4 ksi for its columns. The design team selected 24” x 24” columns to meet constraints within the floor space. However, after structural analysis, it is found that the columns need to be 30” x

30” at 4ksi to meet load demand. A natural next step would be to increase the f’c to 5ksi to determine if the columns can remain at 24” x 24”.

A similar approach can be implemented with masonry. The design masonry compressive strength, f’m, can be increased to meet a desired masonry wall thickness. To better understand this approach, we will explore how f’m is determined masonry.

Masonry is an assembly of several components - the individual clay or concrete block units, mortar, grout, and reinforcement. The total compressive strength, f’m, for a masonry assembly is based on the strength and interface of the clay/concrete unit + mortar working together to resist loads. The main component that contributes to f’m is the compressive strength of the clay or concrete block unit. However, it is important to recognize that the strength of these individual units are dependent on raw materials found locally and mix proportions. Therefore, the strength of the individual units can vary across different regions, based on factors such as local aggregate, sand, lime, etc.

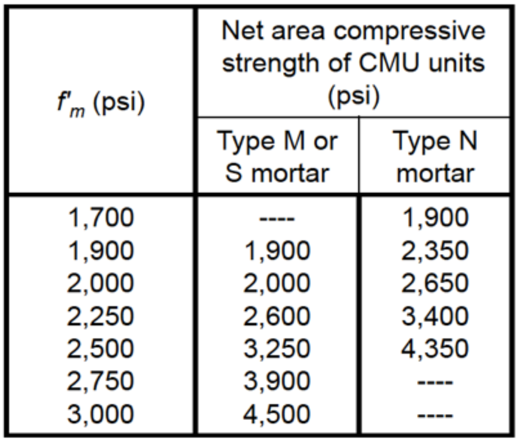
Much testing and research has been done to better understand compressive strength, primarily tests on prisms of block unit and mortar assemblies across different regions. From this research, tables were developed to allow designers to assume a compressive strength of the assembly, f’m, for masonry design. These tables are the basis for the Unit Strength Method, which is one method to determine assembly compressive strength, f’m. Refer to Tables 1 and 2 in the TMS 602-13 Specification. This Method takes the block unit’s strength + mortar type to result in an assembly compressive strength, f’m.



f'_m (psi)	Net area compressive strength of clay units (psi)	
	Type M or S mortar	Type N mortar
1,000	1,700	2,100
1,500	3,350	4,150
2,000	4,950	6,200
2,500	6,600	8,250
3,000	8,250	10,300
3,500	9,900	----
4,000	11,500	----

Table 1: Compressive strength of masonry based on the compressive strength of clay units and type of mortar

TMS 602-13 Table 1



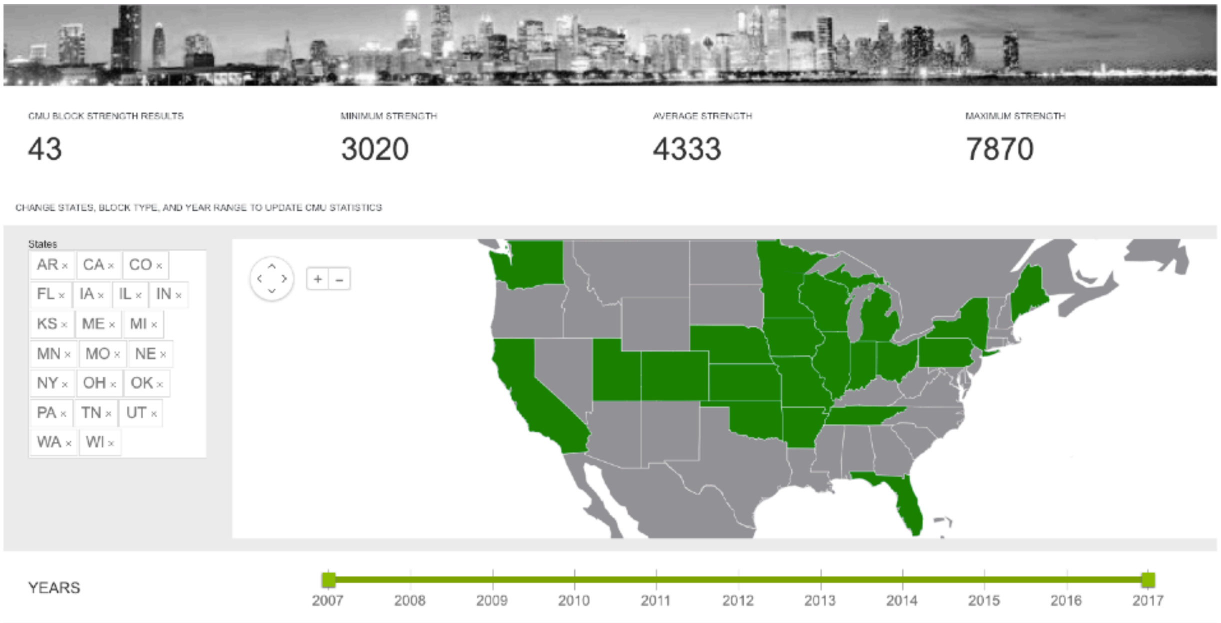
f'_m (psi)	Net area compressive strength of CMU units (psi)	
	Type M or S mortar	Type N mortar
1,700	----	1,900
1,900	1,900	2,350
2,000	2,000	2,650
2,250	2,600	3,400
2,500	3,250	4,350
2,750	3,900	----
3,000	4,500	----

Table 2: Compressive strength of masonry based on the compressive strength of concrete units and type of mortar

TMS 602-13 Table 2

Historically, designers have specified f’m = 1500psi for concrete masonry compressive strength. This is based on the minimum value of 1900 psi compressive block unit strength per Tables 1 and 2 in TMS 602-2011 Code and prior versions. However, designers do not need to extract the minimum value from this table for design. In fact, it has been found that the minimum value of 1900 psi concrete block unit strength is not commonly produced by block producers. It is more common to find concrete block of much higher strength being produced and transported to job sites,. Although project drawings may specify f’m = 1500psi (based on 1900 psi strength of block units), producers typically produce concrete block in the 3000 to 4500 psi range for block unit strength. This would yield f’m = 2400psi to 3000psi.

This can be evidenced in Figure 1. FORSE has developed a database compiling concrete block unit compressive strength test results from a number of states, highlighted in green.



CMU BLOCK STRENGTH RESULTS 43

MINIMUM STRENGTH 3020

AVERAGE STRENGTH 4333

MAXIMUM STRENGTH 7870

CHANGE STATES, BLOCK TYPE, AND YEAR RANGE TO UPDATE CMU STATISTICS

States: AR × CA × CO × FL × IA × IL × IN × KS × ME × MI × MN × MO × NE × NY × OH × OK × PA × TN × UT × WA × WI ×

YEARS 2007 – 2017

Figure 1: Concrete Masonry Block Compressive Strength Test Results Database
http://forsecmustats.azurewebsites.net/

The database reports minimum and average compressive strengths of individual concrete block units. Designers can enter this map and select states in or surrounding project locations to find block unit compressive strengths. Based on this output, one can better understand minimum and average compressive strengths for block local to projects, and arrive at a desired unit compressive strength with a better understanding of block availability. You can then enter TMS 602-13 Table 2, incorporate mortar type, and determine a design f’m of the assembly for your project. For example, Table 2 for concrete masonry strength shows a minimum strength for concrete units to be 1900 psi. However, Figure 1 shows minimum concrete unit strength of 3020 psi based on compiled test results in the database, which results in f’m = 2400psi using Type S mortar.

In addition, TMS 602-2016 and ASTM C90-2014 has updated the minimum value for concrete masonry unit strength to be 2000 psi and f’m = 2000 psi minimum. This is higher than the 1900psi unit strength and f’m = 1500psi per previous Codes. Therefore, as a minimum, we should consider f’m = 2000 psi, but based on local block strengths, we can utilize even higher f’m strengths.

It is recommended to look at local strengths, specify what is needed for projects rather than specifying a minimum f’m value, and consider a starting point for concrete masonry f’m to be 2250psi or 2500psi. The database above shows that block is commonly produced at a higher compressive strength than the minimum strength per the Code.

F’m is prevalent in all aspects of masonry design. Increasing the f’m from 1500 to 2250psi or 2500psi can greatly increase the structural efficiency of masonry design, and yield a more cost effective solution at potentially no additional cost. Some benefits of designing for an increased f’m include less reinforcement in walls, shallower masonry lintels, and reduced lap lengths.

Movement Joints

Unreinforced veneer walls will respond differently to thermal movement than reinforced concrete masonry structural walls. For this reason, control joints can be placed at different locations between veneer walls and structural backup walls. In walls without reinforcement, control joints should be placed adjacent to wall openings to alleviate high stress concentrations at the wall corners, thereby minimizing the potential for crack propagation.

In walls with reinforcement, assuming there is a masonry lintel with horizontal reinforcement and jamb reinforcement, control joints should be placed far from wall openings. In the latter condition, the intersection of vertical and horizontal reinforcement within the wall will help to prevent cracks from propagating from high stress corners of the wall opening. Control joints are not needed for crack control at openings within reinforced walls. Placing the joint far from the opening allows for distribution of gravity and lateral loads from the wall above the opening to the wall segments on both sides of the opening. The loads have a path to the foundation or wall below. Placing a control joint adjacent to a reinforced wall opening prevents loads from transferring above the opening to the wall on either side of the opening, creating a much less efficient structural resistance system. NCMA Tek 10-2C provides guidance on control joint location (including maximum spacing criteria) for both reinforced and unreinforced walls. In addition, TMS 602 Mandatory Requirements Checklist requires that movement joints shall be located on the project drawings. Therefore, it is recommended that structural engineers specify control joint locations for reinforced walls designed to resist loads, and architects specify expansion joint locations for non-load bearing, unreinforced walls.

A few optimal areas to consider placing control joints in reinforced walls include the following locations, but are not limited to:

- Away from wall openings; Recommended minimum 2’-0” from edge of openings
- Where steel columns are located within a masonry wall, to allow for differential movement
- At steps in the top of wall
- At changes in thickness of walls

- Where movement joints are placed in floor framing and/or slabs

Lintels for Masonry Walls

It is common to see steel lintels used to support the wall above a masonry opening. Although common, there are adverse effects that a steel lintel can have within a masonry wall, that may result in a less efficient masonry design.

Steel moves at different rate than masonry, and this movement must be accommodated in the steel lintel-to-masonry wall connection. If the steel lintel is restrained from movement, stresses will occur in the masonry, and cracks will develop to relieve these stresses (Figure 2). Therefore, in order to allow the steel to move within the wall, it is recommended to use a sliding bearing joint at the end of the steel lintel (Figure 3). This sliding joint should have long slotted holes for its anchor bolts. To avoid cracks at the anchor bolts, U-bars should be provided around the anchors, extending into the masonry wall. In addition, the steel lintel conflicts with jambs in the cells directly adjacent to the wall opening. Therefore, jamb reinforcement should shift over one cell beyond lintel bearing to allow continuity of vertical reinforcement. Lastly, steel bearing plates can invite future corrosion and rust if not detailed correctly. Bearing plates should not project beyond the face of the masonry wall, and it is optimal for the plate thickness not to exceed the mortar joint thickness.

Figure 2: Differential movement in steel lintel within masonry wall

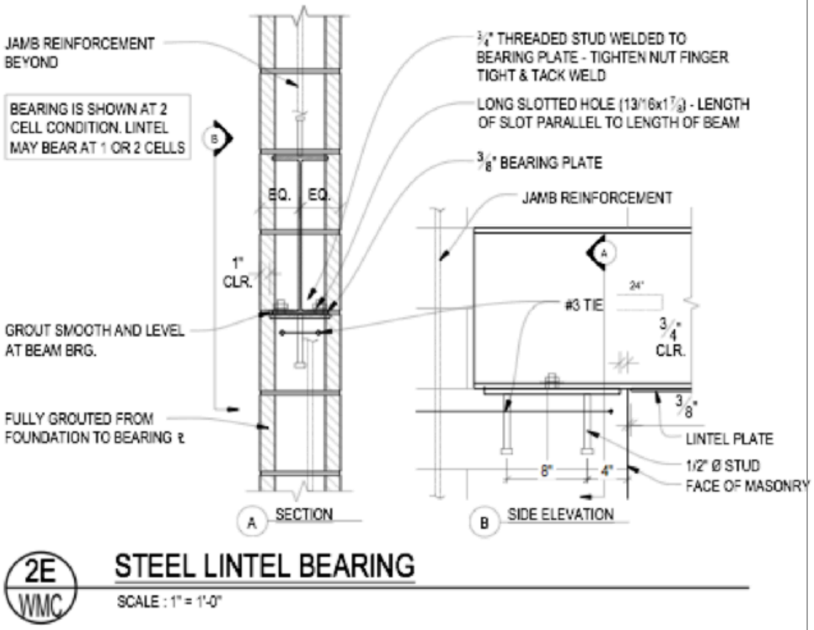


Figure 3: Steel Lintel Bearing Detail

Due to the complexity in detailing steel lintels to accommodate differential movement and the potential for crack propagation, masonry lintels are an optimal solution. Masonry lintel reinforcement intersects jamb reinforcement, resulting in a simple yet interconnected joint without the potential for differential movement. More importantly, masonry lintels allow for arching action within a masonry wall, which results in reduced loads onto the lintel, and an overall more efficient lintel design. Lintels are essentially arches supported by the masonry walls or jambs on either side of a wall opening. At the base of the lintel arch are vertical and horizontal (thrust) reactions that transfer to the jambs. If there are sufficient jamb widths on either side of the opening, then the loads can appropriately transfer from the wall above the opening to the walls on either sides of the opening via arching action. Arching action and the transfer of vertical and horizontal reactions can be achieved with a masonry lintel, but difficult to achieve with a steel lintel that is sliding to accommodate thermal movement.

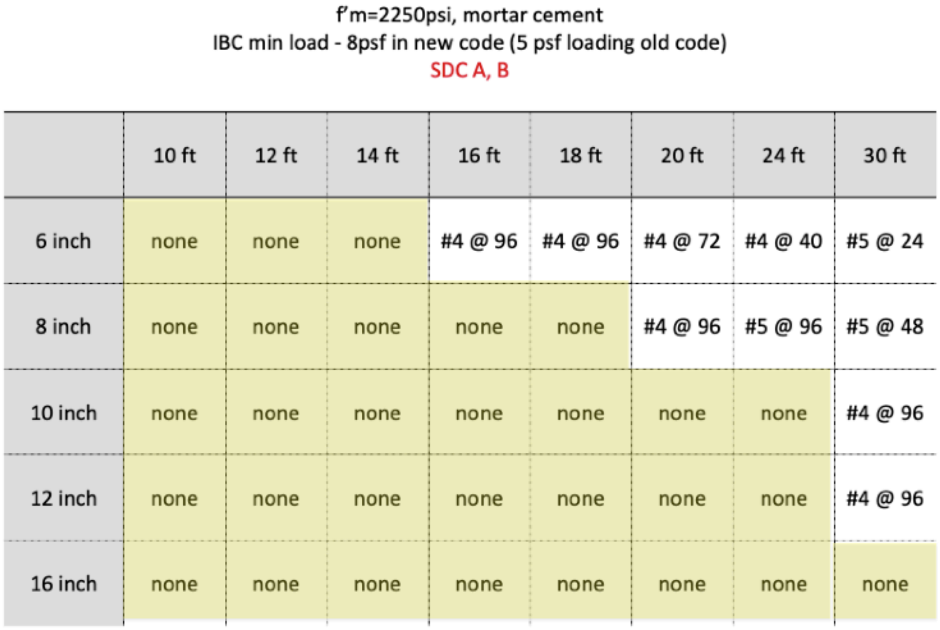
However, at times, there is not sufficient jamb width on either side of the opening. This can occur when a control joint is placed too closely to the wall opening, or if the wall opening is placed too close to a wall end or corner. In this later condition, the lintel arch does not have adequate support to resist both vertical and horizontal reactions, resulting in an over-reinforced lintel and/or jamb, or the lintel simply cannot be found to work. It is critical to allow sufficient jamb width on both sides of a wall opening, to allow loads to transfer from above the opening to its supporting walls. 2'-0" minimum distance is recommended between the edge of an opening and any control joints, corners, or ends of walls.

Interior Partition Walls

Masonry is a sound material choice for use in non-load bearing partition walls due to its high fire resistance and durability. If these walls are intended for non-load bearing applications, and if they are detailed as such, then they can be unreinforced or lightly reinforced, and cost efficient. The key is to detail the walls so as not to allow gravity, shear, or out-of-plane loads from entering the walls. To prevent gravity loads, it is common to provide a gap between the top of the wall and the bottom of any floor framing beams or slabs. This gap should be greater than the anticipated deflection due to the floor beam or slab above. It is also important to provide lateral bracing for the top of the wall, if the partition wall was considered pinned-pinned in its design. Lateral bracing often takes the form of an intermittent or continuous angle on each side of the wall, anchored to the beam/slab above the wall. This connection should only serve to laterally brace the top of wall, but should not transfer unintended shear loads to the non-load bearing partition walls.

If partition walls are properly detailed not to transfer structural loads and if located in a low seismic region, Table 3 below is an example of reinforcement that can potentially be used. This is based on 8 psf interior pressure per ASCE 7-10 Code, for seismic design categories A and B. This reinforcement must be confirmed and designed by the engineer for specific project site, geometry, loading, and seismic region.

General Notes may state to provide reinforcement in all masonry walls at 48" o.c. maximum, including at non-load bearing partition walls. Table 3 shows that partition walls in low seismic regions can be unreinforced or lightly reinforced for fairly tall story heights, thereby reducing the cost of reinforcement, grout, and labor. Oftentimes, the total length in lineal feet of partition walls on a project exceed the total length of structural reinforced masonry walls. Detailing partition walls correctly and providing reinforcement for interior pressures only, in lieu of a blanket minimum amount of reinforcement in all masonry walls, can greatly bring cost savings to a project.



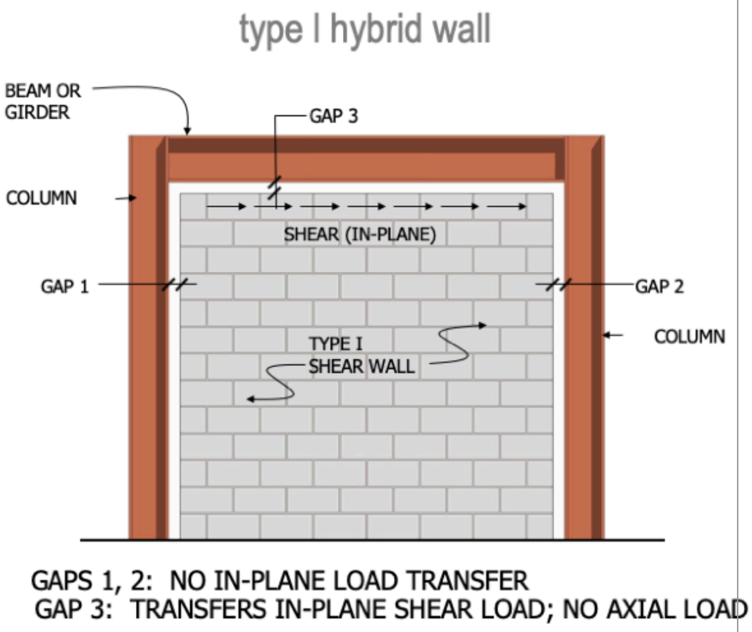
f'm=2250psi, mortar cement
IBC min load - 8psf in new code (5 psf loading old code)
SDC A, B

	10 ft	12 ft	14 ft	16 ft	18 ft	20 ft	24 ft	30 ft
6 inch	none	none	none	#4 @ 96	#4 @ 96	#4 @ 72	#4 @ 40	#5 @ 24
8 inch	none	none	none	none	none	#4 @ 96	#5 @ 96	#5 @ 48
10 inch	none	none	none	none	none	none	none	#4 @ 96
12 inch	none	none	none	none	none	none	none	#4 @ 96
16 inch	none	none	none	none	none	none	none	none

Table 3: Possible Reinforcement for Non-Load Bearing Interior Walls

Hybrid Systems

In structures that utilize a combination of materials, such as steel beams and columns with masonry backup walls, one solution is to detail the steel frame to resist gravity loads, and the masonry backup walls to resist shear loads. The masonry walls can be detailed in the same plane as the perimeter steel beam and columns. One option is for the gravity load path to remain within the steel beams and steel columns through to the foundation below. Steel beams carrying lateral forces axially would transfer this lateral shear down to the masonry walls below its bottom flange via angles and anchors to the masonry walls.The masonry walls then resist lateral loads as shear walls, precluding the need for steel braced or moment frames. This solution is termed a Type 1 Hybrid System and is shown in the Figure 4.



GAPS 1, 2: NO IN-PLANE LOAD TRANSFER
GAP 3: TRANSFERS IN-PLANE SHEAR LOAD; NO AXIAL LOAD

Figure 4: Steel and Masonry Type 1 Hybrid Wall

Enhancing masonry design efficiency requires thoughtful consideration of compressive strength values (f'm), strategic placement of movement joints, and proper detailing of both lintels and partition walls. By specifying higher f'm values, optimizing joint locations, and selecting masonry lintels over steel, engineers can achieve more cost-effective and high-performance masonry structures. Implementing these advanced design strategies leads to structures that are not only durable but also structurally optimized.