SE INSIGHTS

FOR MASONRY DESIGN

Masonry Exterior Non-Bearing Wall Design Guide

When a building has a structural frame that supports both the gravity and lateral loads, the perimeter walls are then isolated from the structural frame and only need to provide a barrier between the interior space and the exterior elements. This means these exterior walls only need to resist out-of-plane loads from component and cladding wind and self-weight seismic loads. Masonry makes an excellent option for the exterior wall material as it has advantages over other building materials. To start with, masonry offers durability and security as well as fire and sound control. Additionally, masonry can offer energy savings due to its thermal mass, ungrouted cores can be filled with insulation, and the assembly requires less maintenance than other building materials. The exterior face of masonry can be painted, burnished, rock-faced, or stacked with various bond patterns allowing many aesthetic options while removing the need for other trades/materials to cover up the structure. The prevalence of masonry in many building types clearly demonstrates these architectural and structural advantages are frequently chosen.

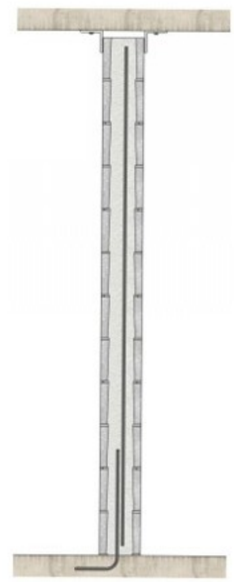


Figure 1: Wall Section IMI Detailing Series

When it comes to the design requirements for non-load-bearing masonry walls in the TMS 402 masonry code, there are two options to consider. First is design per the main code body and second is Appendix B for masonry infill. There are differences in these two approaches which can have a significant effect on the design of the wall reinforcement layout and is the focus of the first section of this article followed by detailing of connection to the main structure and finishing with a reinforcement example. For a comparison, there is a companion masonry insight article titled "Appendix B; Non-Participating Masonry Infill" that provides an alternate reinforcement example.

Masonry Wall Definitions

To assist with the main discussion of this article, a few select definitions from the TMS 402 code are included here to help ensure everyone is starting at the same point.

Load-Bearing Wall: "Wall supporting vertical loads greater than 200 pounds per linear foot in addition to its own weight."

Shear Wall: "A wall, load-bearing or non-load bearing, designed to resist lateral forces acting in the plane of the wall."

Infill Wall: "Masonry constructed within the plane of, and bounded by, a structural frame".

Standard Non-Load-Bearing Masonry vs. Non-Participating Infill

Masonry walls are typically designed per the main body of the TMS 402 code per chapters 8 or 9 which are included in Part 3: Engineered Design Methods. The engineering design works in conjunction with the general requirements of Parts 1 and 2 for analysis and design. Standard non-load bearing, non-shear walls fit into this design methodology including exterior walls that span between floors of the building without supporting any gravity or in-plane lateral loads. There is also another category of non-load bearing, non-shear walls called "Infill Walls" that are used at the building exterior to fill in the space between the beam and columns of a structural frame. These structural frames often times support all of the gravity and lateral loads for the building and transfer these forces directly into the soil through the building foundations. That means the masonry infill will be considered "non-participating infill" which is designed per Appendix B sections B.1 & B.2 with reference to Parts 1 & 2 of the main code.

When directly comparing the parameters of the non-load bearing, non-shear standard masonry wall with the non-participating infill, these two design cases appear to be essentially the same from a structural standpoint. Each case resists out-of-plane loads with connections that do transfer these loads into the structure while simultaneously allowing in-plane movement of the structure so as not to transfer in-plane load from the structure into the wall. The two main differences between the two wall types comes down to 1) whether the masonry is just between floors or is filling in a frame and 2) the code section triggered by the terminology. See figure 2 for a direct comparison summary.

Looking solely at non-participating infill in appendix B, section B.1 references parts 1 and 2 of the main code so that all infill walls will be designed with the same requirements as typical masonry including the seismic detailing requirements of chapter 7. Note that appendix B does deviate from the main code in two significant ways. First, the appendix only references the strength design method and secondly specifies a reduction factor $\varphi = 0.60$ for shear, flexure, and axial loading in place of the main code values of $\varphi = 0.90$ for flexure/axial and $\varphi = 0.80$ for shear. Be aware that the strength reduction factors for anchorage and bearing remain unchanged and shall be determined per TMS 402-16 section 9.1.4. The difference in φ creates a significant difference in strength and efficiency for non-participating infill walls. Per the B.1 commentary, the design for all infill walls is noted as being based on a combination of experimental research and anecdotal performance which is the likely reason for the lower reduction factor. This makes perfect sense for participating infill walls as there is a complex interaction between the frame and the infill causing struts to form in the masonry. However, when masonry infill is detailed per Appendix B.2 (non-participating), the infill will be isolated from the main structural frame, and thus there will not be any interaction between the two elements. As described earlier, the structural behavior for non-participating infill will be the same as a non-load-bearing standard masonry wall with the same top of wall detailing.

The requirement for non-participating infill walls to use a lower strength reduction factor ($\varphi = 0.60$), despite their isolation from the main structural frame, originates from conservative design assumptions based on experimental and historical performance data rather than direct structural behavior. Unlike load-bearing masonry walls, which are explicitly designed to carry gravity or in-plane lateral loads, non-participating infill walls are designed to be isolated from the structural frame.

However, in reality, achieving perfect isolation can be difficult, and unintended load transfer may occur through improper detailing, construction tolerances, or unexpected frame deformations. The lower φ-factor accounts for potential uncertainties in load path behavior and possible unintentional interactions with the surrounding frame. Since the infill panel's true behavior under lateral loads remains somewhat unpredictable, the code committee opted for a conservative strength reduction factor to ensure safety.

Non-load-bearing, Non-shear Wall	**Non-Participating Infill**
• TMS 402 Chapter 8 (ASD) or 9 (LRFD)	• TMS Appendix B.1 & B.2 LRFD Only
• Located between floors with gap at top of wall (no structural framing at ends of wall)	• Located in line with structural frame with gap at top and both ends of wall
• Out-of-plane wind/seismic loading only	• Out-of-plane wind/seismic loading only
• Flexure/Axial φ = 0.90, Shear φ = 0.80, Bearing & Anchorage φ per TMS 9.1.4	• Flexure/Shear/Axial φ = 0.60, Bearing & Anchorage φ per TMS 9.1.4
• Design in conjunction with TMS 402 Parts 1 & 2	• Design in conjunction with TMS 402 Parts 1 & 2
• Seismic design & detailing per TMS Chapter 7 for non-participating elements	• Seismic design & detailing per TMS Chapter 7 for non-participating elements
• No maximum connector spacing (except for high seismic areas)	• Maximum 48” connector spacing along perimeter on all sides

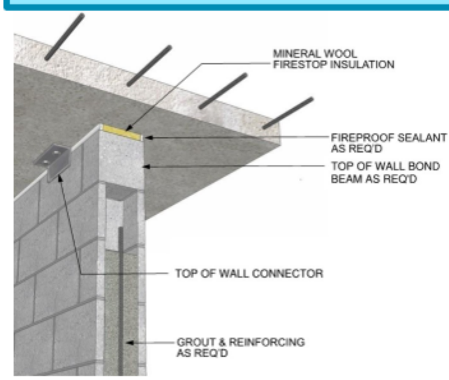


Figure 2a: Non-Load-Bearing, Non-Shear Wa

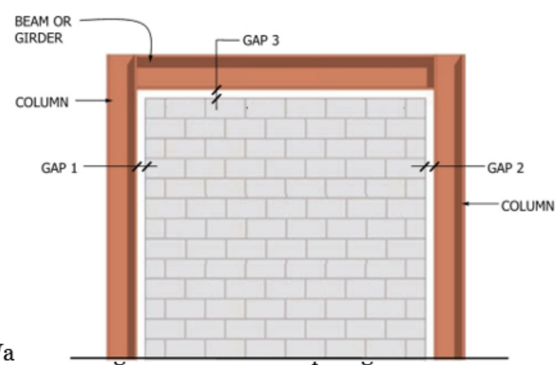


Figure 2: Masonry Wall Comparison

Out-of-Plane Wind Load Reinforcement

All exterior infill walls must resist out-of-plane wind and seismic loading. In low seismic areas, the wind load will most likely govern. Per the TMS code, there are no minimum reinforcement requirements for wind loading. Similarly, per TMS 402-16 Section 7.4.1, non-participating seismic elements located in Seismic Design Category (SDC) A or B also do not have minimum reinforcement area or maximum spacing requirements. However, be aware that buildings with SDC C or higher do minimum seismic reinforcement requirements which must be checked against the wind design to determine the final masonry wall reinforcement.

When choosing reinforcement for masonry walls, the general rule of thumb for economy is to space the rebar as far apart as possible to minimize the number of grouted cells thus using less labor and materials. However, for non-bearing exterior walls, other items affect cost such as reinforcement lap splice length, bond beam locations, and connector capacity which may lead to closer rebar spacing once all factors are considered. This section will focus on reinforcement while the next section on detailing will discuss the last two items listed. Many factors influence the lap splice length including masonry assembly strength (f ’m), rebar size, and cover distance. Using f ’m values higher than the code minimum is recommended as masonry units off the shelf can easily develop higher strength than many engineers expect. F’m = 2500psi is a good starting point as this can be produced in virtually all locations across the United States. Values of 3000psi or 3500psi (or even higher) can also be achieved fairly easily, but it is best to first verify availability with local suppliers. For exterior walls discussed in this article, it is recommended to use a single bar centered in the masonry cores which also helps to minimize lap lengths by maximizing cover distance. Finally, bar size is the biggest driver of lap lengths. Shorter height walls generally need less reinforcement and may not even need rebar splices at all. Rebar sizes of #4 and #5 generally have low lap lengths. Bar sizes of #6 and larger begin to have much longer lap lengths that can make rebar installation unwieldy. As can be seen in table 1, the recommendation is to use #4 or #5 bars as much as possible saving #6 bars for when it is necessary.

To have an idea of the reinforcement required for non-bearing standard masonry walls, see the following example. Consider a hypothetical 60ft tall building located in Chicago, Illinois with a wind speed of 107 mph per ASCE 7-16. This location was chosen to represent an example applicable to a majority of the country. The reinforcement is based on component & cladding wind loads in wind zone 5 and out-of-plane load combination wind coefficient = 0.42W for evaluating deflection. Type S mortar with medium weight masonry (115 pcf) is assumed at f ’m = 2,500psi. All reinforcement is 60ksi with a single bar centered in the masonry core. Table 1 shows the required reinforcement for standard masonry walls designed per the main TMS 402-16 code provisions.

	10 ft	12 ft	14 ft	16 ft	18 ft	20 ft	24 ft	28 ft	30 ft

4” Brick	#4 @ 40	#4 @ 24	#4 @ 16	#4 @ 16	#4 @ 8	-	-	-	-
6” Brick	#5 @ 104	#5 @ 72	#5 @ 56	#5 @ 40	#5 @ 32	#5 @ 24	#5 @ 16	#6 @ 16	#6 @ 16
6” Block	#5 @ 104	#5 @ 80	#5 @ 56	#5 @ 40	#5 @ 32	#5 @ 24	-	-	-
8” Block	#5 @ 120	#5 @ 112	#5 @ 80	#5 @ 64	#5 @ 48	#5 @ 40	#5 @ 24	#5 @ 16	#5 @ 16
10” Block	#4 @ 120	#5 @ 120	#5 @ 112	#5 @ 80	#5 @ 64	#5 @ 48	#5 @ 32	#5 @ 24	#5 @ 24
12” Block	#4 @ 120	#4 @ 120	#5 @ 120	#5 @ 104	#5 @ 80	#5 @ 72	#5 @ 48	#5 @ 32	#5 @ 32

Table 1: Standard Exterior Wall (φ = 0.9)
Design for Out-of-Plane C&C Wind

Detailing Requirements

Next is a discussion on the requirements in section B.2 specific to non-participating infill walls. The most important detail item is that the infill wall must be isolated from the surrounding structural frame so that no vertical or lateral load is imparted to the masonry in the plane of the wall. To ensure this, the code requires a minimum 3/8” joint on the top of the infill wall and at both ends. The joint may need to be bigger than 3/8” depending on the expected deflection of the frame members including inelastic deformation during seismic events. The joints must be made with a resilient compressible material and must not have any mortar, debris, or any other rigid material to ensure that no in-plane lateral load is transferred into the infill wall. The proper selection of compressible joint material is essential in seismic-prone areas to allow masonry walls to accommodate frame deformations without engaging in load transfer. The selected joint material should meet the following performance criteria: high compressibility, energy absorption, non-rigid composition such as polyurethane foam, neoprene, or closed-cell foam. Similar detailing should be used for non-load-bearing standard masonry walls at the upper slab. Failure to do so may transfer unintended transfer unintended loads into the wall which could unintentionally make it function like a shear wall or participating infill wall. If lateral loads were to transfer into the wall, compression struts would form sending resultant loads into the nearby structural elements which could lead to failure if this load path has not designed for (See TMS Appendix B.3 commentary for more information). Thus, it is very important to properly size these isolation joints with proper.

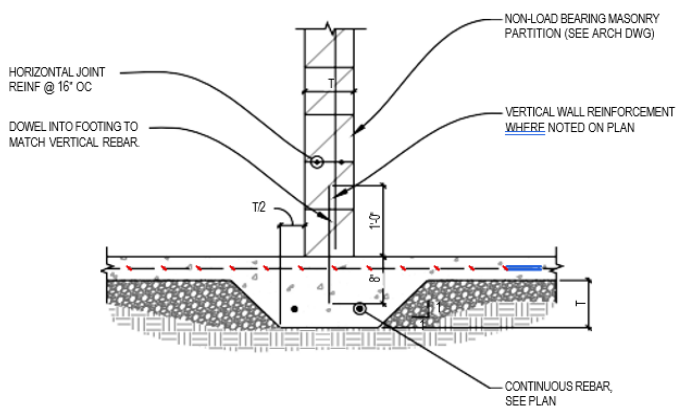


Figure 3: Simple Base of Masonry Wall Connection

In contrast, both the non-load-bearing standard masonry wall and the non-participating infill wall do need to resist loads out-of-the-plane of the wall. The difficult part of the design are the connectors to the surrounding structural frame or structural elements. The connectors must be able to transfer the out-of-plane loads, but, as noted earlier, not transfer any in-plane loads while allowing both vertical and horizontal deflections of the structural frame. For the out-of-plane loads, the masonry can either span vertically, horizontally, or both, but the most common is vertical. Additionally, the wall must be designed to span between connectors. The connectors must be spaced as required based on the capacity of the connection and the loads present but the maximum spacing for non-participating infill per TMS Appendix B is 48" along the perimeter. Depending on the spacing of the connectors, the spacing of the internal wall reinforcement, and the magnitude of the out-of-plane load, a top of wall bond beam may or may not be needed to transfer the loads to the connectors. Our recommendation is, when possible, to locate the connectors at reinforced cores and avoid a top of wall bond beam to minimize cost and maximize efficiency of the masonry.

Connectors meeting all of these requirements can be difficult to design, particularly the simultaneous vertical and horizontal slip requirement. Following is a discussion of various connection options.

Base of Wall Connectors

The base connection of a non-load-bearing masonry wall plays a crucial role in its overall stability, load distribution, and deformation behavior. The two primary types of base connections are fixed (moment-resisting) and pinned (simple support). The choice between these depends on the wall's function, structural behavior, and detailing requirements. A pinned base typically achieved by grouting the base course of masonry into a shallow recess in the foundation or using dowels to transfer shear but not moment. The wall spans vertically, resisting out-of-plane wind and seismic loads but relying on top connections to accommodate movement. A fixed base is achieved by fully

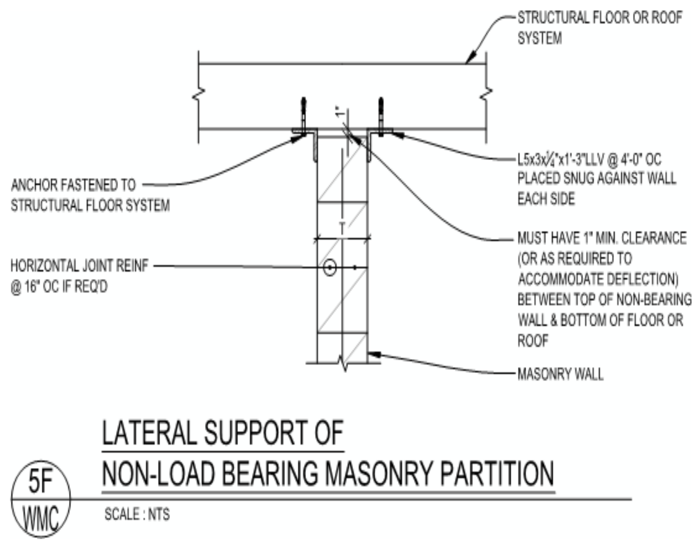


Figure 4: Top of Masonry Wall Detail

grouting the vertical reinforcement into the foundation or embedding rebar into the footing. This type of connection forces the wall to develop bending resistance, similar to a cantilever. Since the walls discussed are usually designed as a simple span, a dowel from the floor or foundation below will typically extend into the bottom of the wall as shown in Figures 3 & 5. However, this dowel is not required by code since masonry walls can be designed as unreinforced. When the dowel is used to transfer shear from the wall to the support, it does not need to be lapped or even located in the same cell as the wall reinforcement (if wall reinforcement is required). This allows the contractor flexibility, and the result is a more affordable design.

Top of Wall Connectors

The top-of-wall connection for non-load-bearing masonry walls is critical for accommodating structural deflections, preventing unintended load transfer, and resisting out-of-plane loads from wind or seismic forces. The connection must allow vertical movement of the structure while providing adequate lateral stability to the masonry wall. Non-load-bearing exterior masonry top of wall connection details includes a gap to allow for vertical deflection of the structure above. The size of gap must be large enough to accommodate deflections of the structure. Note that the wall fire rating still needs to be maintained at the gap with fire stop materials, so the gap size must be coordinated to meet all applicable design requirements. See figures 2, 4, 5, 6, and 7. The first structural aspect of the connection is whether there will be a bond beam at the top of the wall. Since the structure above will already be in place, placing grout in the top course of the masonry wall will be very difficult for the mason to install resulting in increased labor costs. When possible, masons prefer the wall connection be a direct connection located at the grouted cells. This can be achieved in many ways but depends on the location of the wall relative to the structure above and the architectural design. For example, if a concrete slab extends past the exterior side of the masonry wall and there is a soffit, then angles on both sides of the wall can be an option as shown in Figures 4 and 5.

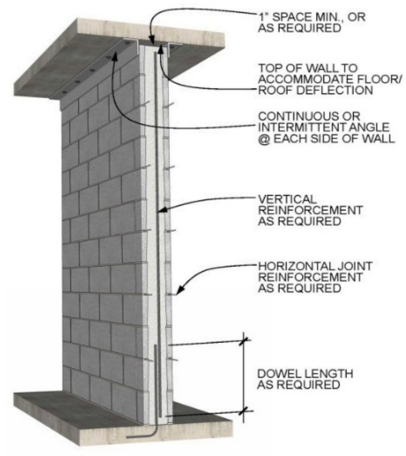


Figure 5: Full Masonry Wall Detail

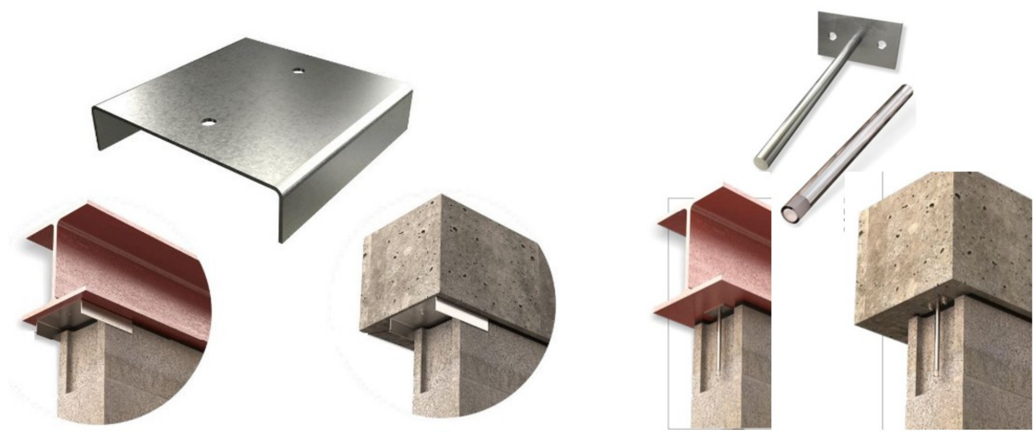


Figure 6: Partition Top Anchors (PTA)
source: www.wirebond.com

For high-seismic zones, PTAs or compressible joints should be used. For high-wind zones, steel angles or PTAs spaced closely together are recommended. The final choice depends on structural demands, cost, and aesthetic considerations. The angles can either be continuous or intermittently located at the grouted reinforcement cores to ensure a direct load path for out-of-plane forces. The advantage to this detail is that the top of wall bond beam is not required and is likely a cost-efficient option. The disadvantages include the requirement of ample connection space on the outside of the wall, sufficient distance from concrete edge to develop the anchor capacity, and coordination of the intermittent angles with the grouted core locations. One could detail a heavier intermittent angle that is not coordinated with the wall reinforcing — for instance, the angle could be specified at 6'-0 on center. The disadvantage is that now the continuous top of wall bond beam is required for the wall to span horizontally between connector angles.

Figure 7: Partition Top Anchors (PTA)
source: www.h-b.com

initially. However, many of these anchors are only designed to resist the much smaller 8psf interior partition load so they either need to be spaced very close together or modified to increase the capacity for exterior wind loads. The PTAs can be attached to concrete or steel and work by using a rod or rebar inserted in a tube with compressible material in the bottom that allows the connector to slide vertically while still providing bearing against the masonry for load transfer. The rod and tube are typically grouted into the head joint of a stretcher block at interior partition walls. To reach the capacities needed at exterior walls, the rod and tube should either be located in the grouted rebar cell by using an open-ended masonry unit or in a continuous bond beam. Generally, the breakout and bearing strength of the masonry has sufficient capacity so the controlling strength factor is the flexural strength of the steel rod or top plate. Below is an example demonstrating how to calculate the capacity of one top of wall anchor type.

Top of Wall Anchor Design Example

Design the top of wall anchorage for a 6” CMU wall spanning 12’-0” vertically with a 3/4” gap between the top of wall and structure above.

GEOMETRY & LOADING

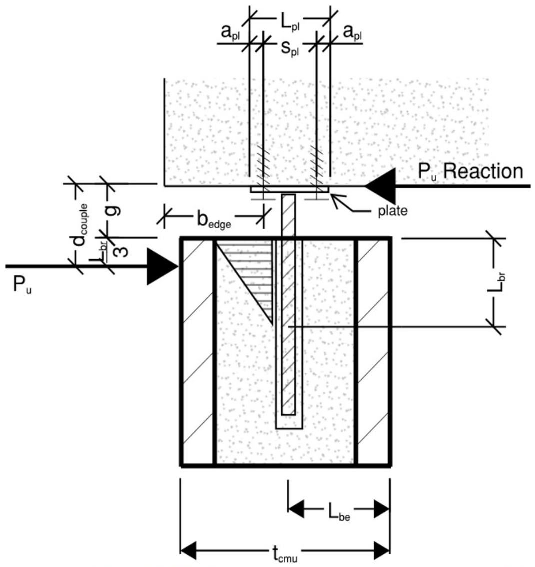
w_u = 33.7 *psf zone 5 wind load based on effective area 1.0W factor. 6”*

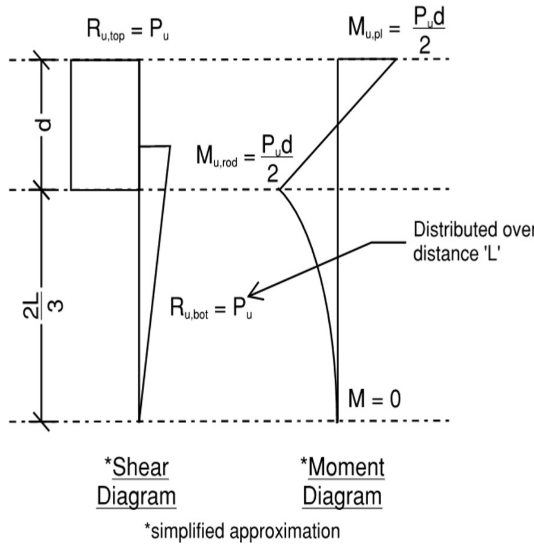
CMU Wall Reinforcing = #5@72” on center

Locate top of wall connector centered in each vertically grouted core to avoid the need for a bond beam in the top course of the wall.

Top of wall connector reaction = P_u = 33.7psf * (12ft /2) * 72in /(12in /ft) = 1,213#

t_{cmu}





$f'_m = 2{,}500\ psi$ gap $g = 0.75\ in$ $d_{couple} = g + \frac{L_{br}}{3}$

$t_{cmu} = 5.625\ in$ $L_{be} = \frac{t_{cmu}}{2} = 2.8125\ in$

ROD DESIGN

Rod = #4 Rebar x 7" Long A706 Weldable Rebar $F_{y,rod} = 60\ ksi$

$\phi P_{n,br} = \phi * 0.8 * f'_m = 0.6 * 0.8 * f'_m = 1{,}200\ psi$

$A_{br} = 2P_u/\phi P_{n,br} = 2.02\ in^2$ *(based on a triangular bearing distribution)*

$d_{rod} = \frac{1}{2}$ *in (diameter of rod)* $d_{sleeve} = d_{rod} + \frac{3}{16} in = \frac{11}{16}$ *in (diameter of rod sleeve)*

$L_{br} = A_{br}/d_{sleeve} = 2.94$ *in (minimum bearing length of rod)*

$d_{couple} = g + \frac{L_{br}}{3} = 1.73\ in$

$Z_{rod} = \frac{d_{rod}^3}{6} = 0.021\ in^3$ $S_{rod} = \frac{\pi d_{rod}^3}{32} = 0.012\ in^3$

$M_{u,rod} = \frac{P_u d_{couple}}{2} = 1{,}049$ *in-#*

$\phi M_{n,rod} = 0.9F_{y,rod} * min(Z_{rod}, 1.6S_{rod}) = 1{,}060$ *in-# > 1,049 in-# OK*

[AISC 360-16 Eq. F11-1]

ROD SHEAR ANCHORAGE IN CMU

$A_{pv} = \frac{\pi L_{be}^2}{2}$ = 12.4 in² (area fits within single vertical grout core) [TMS402-13 Eq. 6-2]

$A_{pt} = \pi l_b^2$ [TMS402-13 Eq. 6-1]

(use bearing depth since rebar is not headed and can slip)

Since $L_{br} = 2.941 in > L_{be} = 2.8125 in$ there is a slight reduction in projected tensile area. To simplify calculation, use L_{be} as a conservative estimate.

Reduced $A_{pt} \approx \pi L_{be}^2 = 24.9$ in²

$B_{vnb} = 4A_{pv}\sqrt{f'_m} = 2{,}485\#$ [TMS402-13 Eq. 9-6]

$B_{vnc} = 1050\sqrt[4]{f'_m A_b} = 4{,}942\#$ [TMS402-13 Eq. 9-7]

$B_{vnpry} = 8A_{pt}\sqrt{f'_m} = 9{,}940\#$ [TMS402-13 Eq. 9-8]

$B_{vns} = 0.6A_b f_y = 7{,}069\#$ [TMS402-13 Eq. 9-9]

$\phi B_{vn} = 0.8 * MIN(B_{vnb}, B_{vnc}, B_{vnpry}, B_{vns}) = 1{,}988\# > P_u = 1{,}213\#$ OK

PLATE FLEXURAL DESIGN

$F_{y,pl} = 50$ ksi (A572 Gr. 50) PL1/4 x 2 x 3

$b_{pl} = 1\frac{1}{2}$ in $t_{pl} = \frac{1}{4}$ in $s_{pl} = 1\frac{1}{2}$ in $a_{pl} = \frac{3}{4}$ in

$Z_{pl} = \frac{b_{pl}t_{pl}^2}{4} = 0.023$ in³ $S_{pl} = \frac{b_{pl}t_{pl}^2}{6} = 0.016$ in³

$M_{u,pl} = \frac{P_u d_{couple}}{2} = 1{,}049$ in-#

$\phi M_{n,pl} = 0.9F_{y,pl} * min(Z_{pl}, 1.6S_{pl}) = 1{,}055$ in-# > 1,049 in-# OK

[AISC 360-16 Eq. F11-1]

CONCRETE ANCHORAGE

Out of the scope of this article but these are the anchor design loads for attachment to the structure above:

$V_{u,anchor} = \frac{R_{top}}{2} = \frac{P_u}{2} = 607$ #/anchor $T_{u,anchor} = \frac{M_{u,pl}}{(s+a)} = 466$ #/anchor

Summary

Masonry is a great choice for exterior walls due to the durability and aesthetic options. The difficult part is keeping track of all the different terminology for masonry walls in the TMS code and the associated requirements for each. The most important part of the design is the detailing especially when isolating from the rest of the structural system for gravity and lateral loads. Any time a wall is isolated on the top and ends, engineering judgement should be used when determining which provisions of the masonry code should apply based on the expected behavior. Whether installing an exterior wall or infilling a frame, the structural behavior of the masonry will be the same. Remember to call out appropriate connectors that consider architectural aesthetic requirements plus deflections of the structure to ensure full compatibility with the design while not transferring unintended loads into the wall. Finally, for economy, locate the connectors at the vertical reinforcement grout locations, when possible, to avoid the installation difficulties of bond beams at the top of the wall. A good, coordinated design makes masonry an effective, cost-competitive option for exterior non-load-bearing walls.

