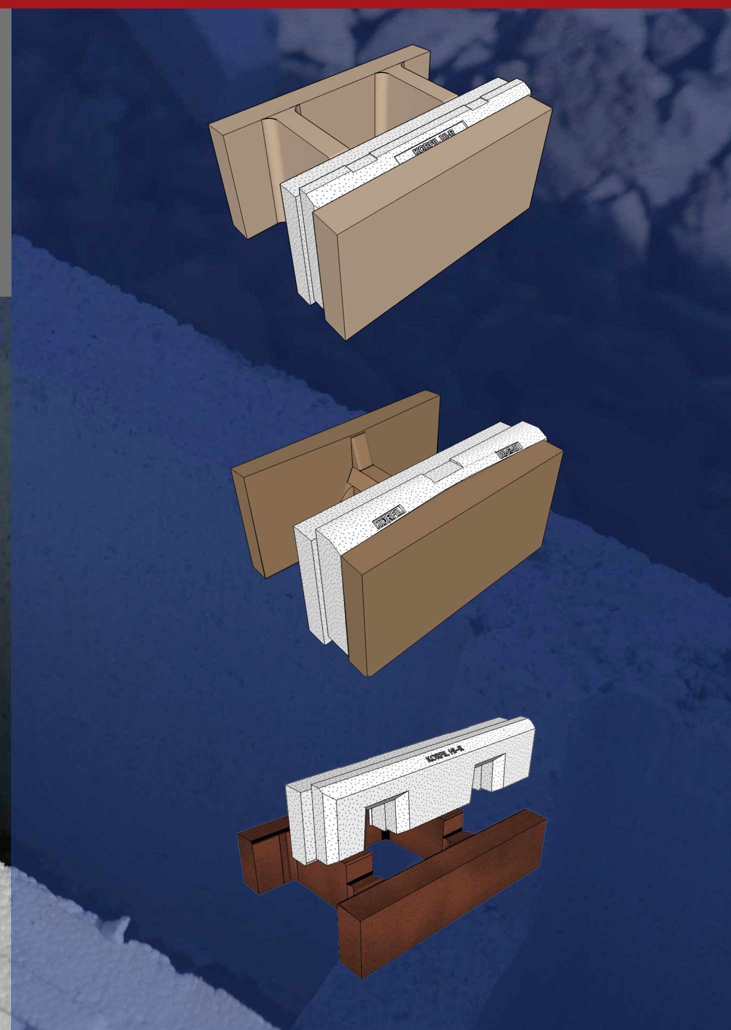
STRUCTURAL DESIGN MANUAL

Korfil® Hi-R and Hi-R H Pre-Insulated Concrete Masonry Products



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CONCRETE PRODUCTS GROUP
INNOVATIVE CONCRETE MASONRY SYSTEMS

STRUCTURAL DESIGN MANUAL

Hi-R and Hi-R H Concrete Masonry Products

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Executive Summary

CBIS/Korfil first introduced insulation inserts to the Concrete Masonry Industry in 1971. Since that time, Korfil, Icon and Korfil Hi-R Inserts have become the most widely used pre-installed insulation inserts in the United States and Canada. These inserts have led to the development of Hi-R and Hi-R H concrete masonry wall systems which provide superior thermal performance in single-wythe applications. The unique units do this by providing reduced thermal bridging which results in wall systems capable of achieving higher thermal R-values than conventional masonry. Those high R characteristics have made Hi-R and Hi-R H units favorites among architects and owners that desire superior thermal performance.

This STRUCTURAL DESIGN MANUAL was prepared by Concrete Products Group (CPG) for use by structural engineers, architects and manufacturers to highlight the benefits and applications of Hi-R and Hi-R H pre-insulated concrete masonry wall systems.

The user will learn that these pre-insulated CMU systems which have been in the marketplace for decades appeal to designers, constructors and owners in that they:

For designers:

- reduce design time,
- are designed using conventional masonry design methodologies,
- serve as the building structure and its exterior envelope,
- comply with air barrier code requirements when fully grouted, and
- are barrier-type walls that don't require flashing and weeps when fully grouted.

For constructors and owners:

- shorten construction time and construction cost,
- eliminate the need for a veneer and all its associated cavity wall construction, and
- reduce the size of the building's foundation.

To assist the designers, the manual includes:

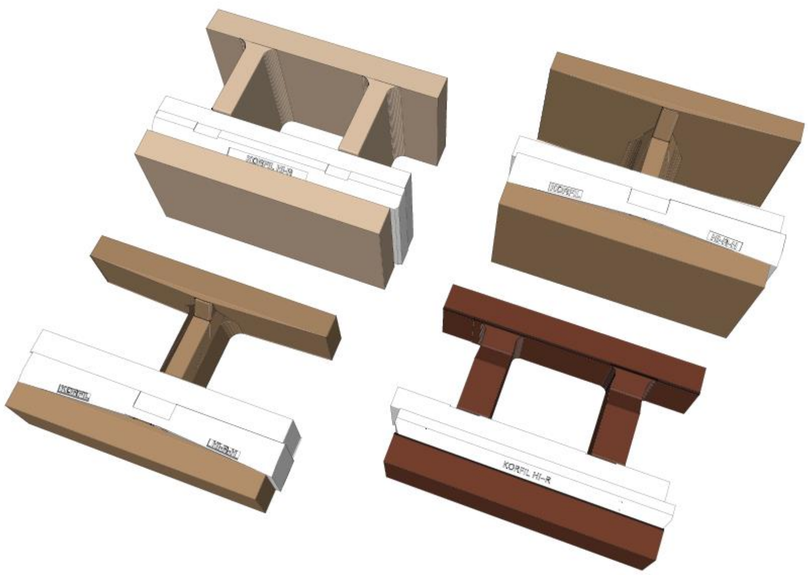
- examples to illustrate design techniques.
- design aids in the form of tabulated values as well as listed CPG resources.

Notice and Disclaimer:

This document is intended to be a voluntary manual for the designer, contractor and owner. It is not intended to relieve the professional engineer or designer of any legal responsibility for the design of structural masonry in accordance with state building codes. While we believe the information contained herein contains proper design techniques, CPG must disclaim responsibility to those who may choose to rely on all or any part of this manual.

Introduction

Hi-R and Hi-R H pre-insulated concrete masonry wall systems were developed to provide superior thermal performance in single-wythe applications. The unique units do this by providing reduced thermal bridging which results in wall systems capable of achieving higher thermal R-values than conventional masonry. Those high R characteristics have made Hi-R and Hi-R H units favorites among architects and owners that desire superior thermal performance.



While much attention has been received regarding the thermal characteristics, there is more. Not only do these systems provide cost savings for creating the thermal envelope, they:

- reduce design and construction cost,
- shorten construction time,
- serve as the building structure and envelope,
- comply with air barrier code requirements when fully grouted,
- are barrier-type walls that don't require flashing and weeps when fully grouted,
- eliminate the need for a veneer and all its associated cavity wall construction, and
- reduce the size of the building's foundation.

We'll discuss these more later.

The genesis for these systems occurred in 1971 with the development by CBIS/Korfil of Korfil, Icon and Korfil Hi-R Inserts. These inserts have become the most widely used Block Plant Installed Insulation Inserts in the United States and Canada.

Concrete Products Group provides a series of technical brochures and documents for the Hi-R and Hi-R H systems as related to their thermal characteristics. This Design Manual is aimed directly at the engineers who prepare reinforced concrete masonry structural designs using these systems. Based upon market acceptance, the emphasis of this manual is based upon walls constructed using either 10-inch or 12-inch wide units. While 8-inch units are available, they are not as commonly used for reinforced masonry applications.

Overall, the design information presented here is consistent with standard masonry design. Structurally, the key feature to recognize is that HI-R and Hi-R H systems are characteristically unsymmetrical.

Conventional masonry systems utilize a symmetrical design. A symmetrical design has either a single reinforcement bar placed in the center of the cell or two bars are placed equidistant from the wall centerline. These designs provide equal flexural strength in both directions making them a symmetrical design. Figure 1 shows a typical example.

Figure 1- Conventional reinforced masonry with reinforcement in center

For the Hi-R & Hi-R H systems, the reinforcement is placed unsymmetrical due to the insulation inserts yet they utilize standard masonry design techniques. Figure 2 shows the unit and the insulation insert along the exterior face shell. The remaining cell area is grouted with the reinforcement centered in the grouted area. Each unit has a reduced web height to accommodate the insulation inserts and provide superior thermal characteristics. Recent changes to ASTM standards make these units a reality so they can accommodate the insulation inserts.

Figure 2 – Reinforcement centered in grouted area of 12 inch Hi-R H

A common question from structural engineers is how the insulation inserts affect the performance of the units with the grout. Tests have shown that the masonry grout works compositely with the units despite the bond break between the grout and the face shells. The web size and the remaining bond are sufficient so that the units and the grout may be designed as a composite section.

Codes and References

From a structural perspective, Hi-R and Hi-R H systems can be designed using the building codes and masonry standards that govern all masonry construction. The justification for this is based on the structural testing that indicates the grouted cells act compositely with the units despite the presence of the insulating inserts.

For the purpose of this Design Manual, the following versions of the code, masonry standards and references will be used.

2015 International Building Code

*Building Code Requirements for Masonry Struc*tures (TMS 402-13/ACI 530-13/ASCE 5-13). When referring to this document in this Design Manual, TMS Section XXX will be used unless noted otherwise.

Specification for Masonry Structures (TMS 602-13/ACI 530.1-13/ASCE 6-13)

ASTM C90 – 14, *Standard Specification for Loadbearing Concrete Masonry Units*

Masonry Designer's Guide 2013 from The Masonry Society

Structural Masonry Design System 7.0, Concrete and clay masonry design software from National Concrete Masonry Association, Western States Clay Products Association, Brick Industry Association, and International Code Council with support from the NCMA Foundation.

Materials

<u>Masonry Units</u>

As previously noted, the material standard governing concrete masonry units is ASTM C90-14. In the past decade, state building codes and standards have been changing at a fast pace. The masonry industry has responded through the development of new products and systems, including HI-R and Hi-R H. In addition, masonry representatives have been instrumental in the evolution of the ASTM C90 standard to stay current with modern needs. In particular, there are new criteria related to concrete masonry units that permit the use of reduced web heights so more insulation can be used within the cells.

The technical information for changes to ASTM C90 includes:

a. ASTM has generally required two criteria for determining an adequate amount of web material between face shells. One criterion is based upon minimum web thickness (t_w). For this, there has been a reduction of the minimum thickness from 1 1/8 inches in the 2002 standard to ¾ inches in the 2014 standard provided a second criterion is met.

b. The second criterion for webs is that there is a minimum required amount of web section area for the units.

   Prior to 2011, ASTM required a minimum Equivalent Web Thickness determined by the summation of the individual web thicknesses of the unit. The presumption then was that the webs were typically the full height of the units. Reduced height web units such as bond beams as well as open-ended units were considered to be units of unusual size and shape despite their commonplace presence in the masonry industry.

   In 2011, a change was made to ASTM C90 that effectively acknowledged the presence of units in the marketplace of unusual size and shape including alternatively shaped masonry units, such as Hi-R and Hi-R-H pre-insulated wall systems. To accommodate additional unit changes such as for insulation inserts, the masonry webs no longer have to be consistently full height of the units. Thus, the minimum equivalent web thickness criterion was replaced by a minimum Normalized Web Area (A_{nw}) of 6.5 in^2/ft^2. This translates to 5.8 in^2 of web area required for each unit with a nominal 8" x 16" face shell and 2.9 in^2 of web area required for each unit with a nominal 4" x 16" face shell.

   ASTM C90 also allows for webs with less than the minimum A_{nw} provided they meet structural requirements.

   Hi-R and Hi-R-H units all meet the Normalized Web Area criteria yet should be checked for grouted applications due to the presence of insulation inserts.

CPG manufacturers can supply concrete masonry units to meet the structural demands of the project. Prior to 2014, the ASTM C90 minimum net area compressive strength for units was 1,900 psi. That minimum strength was increased to 2,000 psi in the 2014 standard. Designers may specify even higher values as needed. Many manufacturers routinely provide units with higher strengths as part of their regular manufacturing operation. Consult with your local CPG supplier for available unit strengths.

Insulation inserts

Another main aspect to these systems is the insulation inserts. These inserts meet ASTM C 578, Type X, replacing Federal Specifications HH-I-524C, *Specification for Rigid Cellular Polystyrene Thermal Insulation*.

Grout

The grout used in the wall systems must meet ASTM C476-10, *Standard Specification for Grout for Masonry*. The grout may be fine or coarse as determined based upon lift height and grout clearances. The strength of the grout (f'_g) must equal or exceed the strength of the masonry units. CPG recommends using a grout additive to increase flowability.

Mortar

ASTM C270 is the standard for all masonry mortars. ASTM C270, *Standard Specification for Mortar for Unit Masonry* provides TABLE X1.1 *Guide for the Selection of Masonry Mortars* which provides recommendations for mortar selection. Typically, Type S mortar is specified for structural masonry and Type M is used for high stress, extreme exposures, as well as at or below grade. Designers must determine the appropriate mortar type for their application.

Reinforcement

Bar reinforcement is governed by ASTM A615/A615M, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*. Grade 60 (F_y = 60 ksi) reinforcement is primarily used in accordance with *Building Code Requirements for Masonry Structures* (TMS 402-13/ACI 530-13/ASCE 5-13).

Horizontal joint reinforcement is governed by ASTM A951/A951M-11 *Standard Specification for Steel Wire for Masonry Joint Reinforcement*. For reinforced masonry, ladder-type joint reinforcement is preferred rather than truss-type to minimize the possible chance for interference with the vertical bar reinforcement.

The use of horizontal reinforcement is an optional design choice available to the engineer and architect for either crack control or structural purposes or both.

Compressive strength (f'_m)

CPG members will provide available values of f'_m (compressive strength of concrete masonry) using their Hi-R and Hi-R H units. The compressive strength f'_m can be determined using either the Unit Strength Method per TMS 602, Table 2, or the Prism Strength Method. CPG recommends the use of the Unit Strength Method to minimize the need for prism testing.

Typically, Hi-R and Hi-R H units vary from 2,000 psi to 3,250 psi. Using TMS 602, Table 2 with either Type S or M mortar, the compressive strengths can vary from f'_m = 2,000 psi to 2,500 psi.

Contact your CPG member for further information and the availability of higher strength units.

Section Properties

As previously stated, the HI-R and HI-R H units are unsymmetrical due to the presence of the insulation inserts along the exterior face shells of the units. Table 1 lists the properties of the fully-grouted masonry wall properties. The standard unit values are without insulation inserts; the Hi-R and Hi-RH unit values are based upon there being insulation inserts. The area of the grout for fully-grouted units and the insulation inserts are shown in Figures 3 through 6. For partially-grouted walls, only the cells containing reinforcement are grouted. Webs adjacent to grouted cells are mortared also. These tabulated values will be useful for various calculations throughout this Manual.

The variation in the Section Modulus values highlights the unsymmetrical nature of the units.

Table 1 – Properties of Fully-Grouted Units						
Unit	Area (in^2/ft.)	y_c = Centroid (in.)		I_n = Gross Moment of inertia (in^4/ft.)	Interior Section Modulus (in^3/ft.)	Exterior Section Modulus (in^3/ft.)
		from interior face	from exterior face			
10" Standard	116	4.8	4.8	892	179	179
10" Hi-R	86	4.2	5.5	742	178	136
10" Hi-RH	79	4.1	5.5	737	180	133
12" Standard	140	5.8	5.8	1571	270	270
12" Hi-R	109	5.0	6.6	1251	249	190
12" Hi-RH	103	4.9	6.7	1225	249	183

Note: These values are the same for full-height and half-high units.

For out-of-plane reinforced masonry design, the design values vary dependent upon the direction of the loads. For design purposes, the *d* distance for flexural capacity is given to the center of the grouted portion of the cell. The engineer should use the appropriate values to meet their design requirements.

Table 2 was developed based upon 10-inch and 12-inch Hi-R and Hi-R H units with insulation inserts. These apply to both fully-grouted and partially-grouted walls, unless noted otherwise. The values for the "standard" C90 units are without inserts. The *d* values are rounded from those shown in Figures 3-6. The "*a* maximum dimension" will be explained later. The values apply to both fully-grouted and partially-grouted walls.

Table 3 provides section properties for partially-grouted and fully-grouted Hi-R walls. We will explain later that HI-R H walls must be fully grouted and Table 1 applies.

Table 2 – Design Dimensions for Walls (out-of-plane)				
Unit	From Interior face of unit		From Exterior face of unit	
	d dimension (in.)	*"a"* maximum dimension (in.)	*d* dimension (in.)	*"a"* maximum dimension (in.)
10" Standard	4.8	2.11	4.8	2.11
10" Hi-R	3.6	1.58	6.0	1.75*
10" Hi-R H	3.3	1.45	6.3	1.75*
12" Standard	5.8	2.55	5.8	2.55
12" Hi-R	4.6	2.02 [1.75]	7.1	1.75*
12" Hi-R H	4.3	1.89 [1.75]	7.3	1.75*

<u>Notes</u>: 1. "*a*" is typically 0.44 x d except * denotes value when the value is governed by the face shell thickness.
2. These values are the same for full-height and half-high units.
3. [XX] values apply to partially-grouted walls.

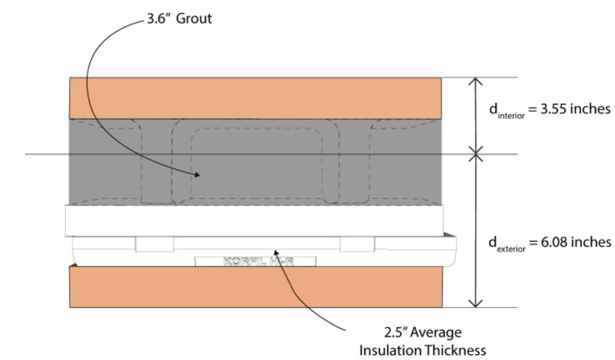


Figure 3 – 10-inch Hi-R Unit

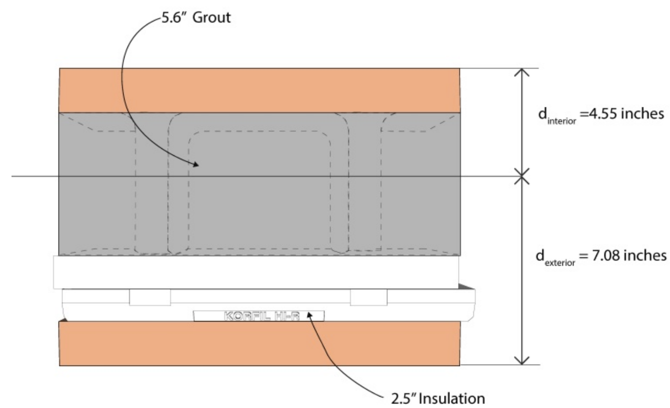


Figure 4 – 12-inch HI-R Unit

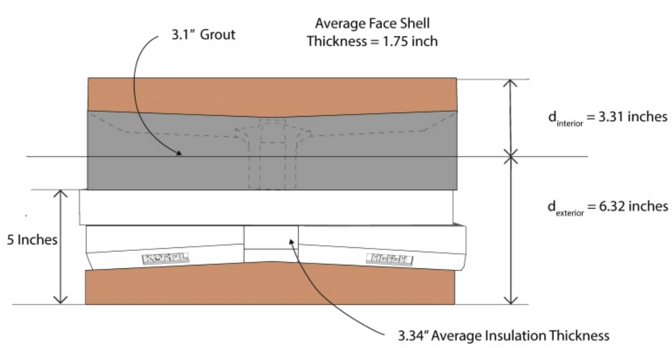


Figure 5 - 10-inch Hi-R H Unit (full and half-height units)

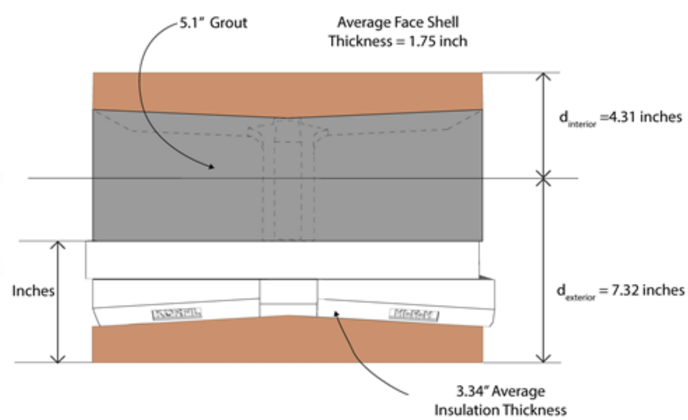


Figure 6 – 12-inch Hi-R H Unit (full and half-height units)

Table 3 – Properties of Grouted HI-R Units						
Unit	Area (in^2/ft.)	y_c = Centroid (in.)		I_n = Gross Moment of inertia (in^4/ft.)	Interior Section Modulus (in^3/ft.)	Exterior Section Modulus (in^3/ft.)
		from interior face	from exterior face			
10" Hi-R						
Fully grouted	86	4.2	5.5	742	178	136
Grout 16" oc	67	4.3	5.3	703	162	133
Grout 24" oc	59	4.5	5.2	688	155	133
Grout 32" oc	54	4.5	5.1	680	150	133
Grout 40" oc	52	4.6	5.1	675	148	133
Grout 48" oc	50	4.6	5.0	671	146	134
12" Hi-R						
Fully grouted	109	5.0	6.6	1251	249	190
Grout 16" oc	93	5.1	6.5	1156	226	178
Grout 24" oc	76	5.3	6.4	1132	216	178
Grout 32" oc	67	5.3	6.3	1110	208	177
Grout 40" oc	62	5.4	6.2	1097	203	176
Grout 48" oc	59	5.5	6.2	1088	199	176

Note: These values are the same for full-height and half-high units.

Wall Weights

Hi-R and Hi-R H units can be supplied in a variety of densities for the thermal characteristics of the overall wall. For structural design purposes, Tables 4 and 5 provide estimated wall weights for the units alone as well as for grouted sections based upon a grout density of 125 pcf. Check with your local CPG Manufacturer on which densities they offer.

Since grout densities can be variable, tables for correction values are provided for the overall wall weights when different grout densities are used.

Note: Partial grouting applies only to Hi-R walls; Hi-R H walls must be fully grouted.

Table 4 - Design Weight for 10-inch Walls							
Density of CMU (pcf)	80	95	100	110	120	125	135
Wall weight fully grouted (psf)*	60	63	67	70	73	75	79
Wall weight grouted 16 in. on center (psf)*	46	49	53	56	59	61	63
Wall weight grouted 24 in. on center (psf)*	41	44	48	51	54	56	58
Wall weight grouted 32 in. on center (psf)*	38	41	45	48	51	53	55
Wall weight grouted 40 in. on center (psf)*	36	39	43	46	49	51	53
Wall weight grouted 48 in. on center (psf)*	35	38	42	45	48	50	52
Wall weight ungrouted (psf)	30	33	37	40	43	45	47

* Use the following modifications for different grout densities in Table 4.

Grout density (pcf)	Correction values - change tabulated values in Table 4 for Design Weight based upon grout density (psf)	
	Fully grouted	Partially grouted
120	-1	-1
125	0	0
130	+1	+1
135	+2	+1
140	+3	+2
145	+5	+2

Table 5 - Design Weight for 12-inch Walls							
Density of CMU (pcf)	80	95	100	110	120	125	135
Wall weight fully grouted with units (psf)**	79	83	86	90	94	95	99
Wall weight grouted 16 in. on center (psf)*	56	60	63	67	71	72	76
Wall weight grouted 24 in. on center (psf)*	47	51	54	58	62	63	68
Wall weight grouted 32 in. on center (psf)*	43	47	50	54	58	59	63
Wall weight grouted 40 in. on center (psf)*	41	45	48	52	56	57	61
Wall weight grouted 48 in. on center (psf)*	39	43	46	50	54	55	59
Wall weight units only (psf)	31	35	38	42	46	47	51

** Use the following modifications for different grout densities in Table 5.

Grout density (pcf)	Correction values - change tabulated values in Table 5 for Design Weight based upon grout density (psf)	
	Fully-grouted walls	Partially-grouted walls
120	-2	-1
125	0	0
130	+2	+1
135	+4	+1
140	+6	+2
145	+8	+3

Design Methodology

The structural design of HI-R and HI-R H systems is similar to designing conventional masonry using the *Building Code Requirements for Masonry Structures (TMS 402-13/ACI 530-13/ASCE 5-13)*. Engineers can use either the Allowable Stress Design (ASD) procedures or the Strength Design (SD) procedures. This Manual is intended to simplify that design process.

Hi-R and Hi-R H walls can be constructed with either running bond or "other than running bond" as defined in TMS 402. TMS 402 provides criteria for load distribution for each and the need for horizontal reinforcement. Running bond is preferred.

As previously stated in the Introduction, Hi-R and Hi-R H systems have numerous benefits including that they will speed up construction and reduce costs while providing superior thermal characteristics. While recent changes in codes and standards have affected the design of CMU walls, CPG has recommendations to address these changes that make designing Hi-R and Hi-R H systems economical and efficient.

The following issues will be addressed to indicate the unique characteristics and flexibility of designing with HI-R and Hi-R H systems. All of these should be addressed in a building design and include:

a. Grouting
b. Reinforcement (bar size and lap splice lengths)
c. Movement Joints
d. Horizontal joint reinforcement - optional
e. Connectors
f. Beams and lintels
g. Out-of-plane loadings (Flexure)
h. Out-of-plane loadings (Web shear)
i. In-plane loadings (Shear)
j. Axial loadings
k. Using Computer Software

a. <u>Grouting</u>

In conventional reinforced masonry design, the engineer must make the decision early in the design process as to whether the design will require partial grouting or full grouting.

When using the Hi-R system, the engineer can also choose either partial grouting or full grouting. If the Hi-R walls are partially grouted, all masonry unit webs at grouted cells must be mortared to restrict grout flow into the cells that are intended to remain hollow and the hollow cells must be flashed and weeped. Some manufacturers offer a version of the Hi-R unit with full height webs behind the insert to avoid the need for mortar application to prevent lateral grout flow. Check with your local manufacturer regarding the availability of these units. If the Hi-R walls are fully grouted, the webs need not be mortared and there are no hollow cells to be flashed or weeped. If the Hi-R walls are fully grouted, the webs need not be mortared and there are no hollow cells to be flashed or weeped.

When using the Hi-R H system there is no choice in grouting techniques; partial grouting is not an option. The wall must be fully grouted because the short height of the web does not allow for cross web mortaring; the grout flows freely from one unit to the next.

CPG experience with numerous projects throughout the United States and Canada is that, for economic reasons, many contractors prefer to fully grout all walls. Some of the stated benefits are:

- fully-grouted walls provide the maximum vertical load carrying capacity,
- there is less engineering time spent designing a fully-grouted wall than a partially-grouted wall,
- fully-grouted walls are barrier-type walls that don't require flashing and weeps as with partially-grouted walls, and
- as a result of the insulation inserts, fully grouted Hi-R and Hi-R H walls use only 60% to 70% of the amount of grout in comparison to a fully-grouted conventional CMU wall.

Note: Partially-grouted walls require internal flashing and weeps. Flashing fully-grouted walls is a design option.

Actual grouting techniques for Hi-R and Hi-R H systems are the same as those for conventional masonry units. Contractors have no special requirements specific to Hi-R and Hi-R H systems.

b. Reinforcement (bar size and lap splice lengths)

The insulation inserts of the Hi-R and Hi-R H units reduce the grouted area within the cells of the units. This reduction has an effect on the size of the reinforcement that can be used in a cell and the design lap length of a bar splice.

Table 6 shows the area of the grouted cells and indicates the limits on available reinforcement.

- The "Grout space in cell" should be used with TMS 602, Article 3.5, and TMS 602, Table 7 to determine maximum pour heights for fine and coarse grout.
- The "6% of grouted area" represents the maximum area of reinforcement that can be bundled within a cell with no bar exceeding #9. Note: TMS 402 allows a bar as large as #11 when designing with ASD and a bar as large as #9 when designing with SD. CPG recommends limiting the bar size to #9 for either method.

Table 6 – Grouted Cells				
Unit	Grout space in cell (in. x in.)	Grouted Area (in^2)	6% of grouted area (in^2)	Maximum bar size
10” Hi-R	6.38 x 3.6	23.0	1.39	#9
12” Hi-R	6.38 x 5.6	35.7	2.14	#9
10” Hi-RH	13.5 x 2.9	39.2	2.35	#9
12” Hi-RH	13.5 x 4.9	66.1	3.97	#9

Lap splice lengths are determined from TMS Section 9.3.3.4 using the larger of 12 inches or the development lengths obtained from TMS Equation 9-16.

$$l_d = \frac{0.13\, d_b{}^2 f_y\, \gamma}{K\sqrt{f'_m}} \qquad \text{TMS Eq. 9-16}$$

Table 7 provides the K values and γ values for use in TMS Eq. 9-16 that are obtained from TMS Section 9.3.3.3

Table 7 – K and γ values for Lap Splices												
Unit	#4 bar		#5 bar		#6 bar		#7 bar		#8 bar		#9 bar	
	K	γ	K	γ	K	γ	K	γ	K	γ	K	γ
10” Hi-R	1.55	1.0	1.49	1.0	1.43	1.3	1.36	1.3	1.3	1.5	1.24	1.5
12” Hi-R	2.55	1.0	2.49	1.0	2.42	1.3	2.36	1.3	2.3	1.5	2.24	1.5
10” Hi-RH	1.20	1.0	1.14	1.0	1.08	1.3	1.01	1.3	0.95	1.5	0.56	1.5
12” Hi-RH	2.20	1.0	2.14	1.0	2.08	1.3	2.01	1.3	1.95	1.5	1.56	1.5

For example, if f'_m = 2,000 psi and the wall is constructed with 10” Hi-RH units, the lap splice length required for a #6 bar is 118 inches. The K and γ values are taken from Table 7.

$$l_d = \frac{0.13\, d_b{}^2 f_y\, \gamma}{K\sqrt{f'_m}} = \frac{0.13\,(0.75)^2(60{,}000)(1.3)}{1.08\,\sqrt{2{,}000}} = \text{118 inches}$$

If 10” standard units without insulation inserts were used, the lap splice would be 29 inches.

Repeating the same calculations for 12” Hi-R units and 12” standard units, the lap splice lengths would be 53 inches and 25 inches, respectively.

These values assume perfect placement of the reinforcement bars in the center of the grouted area. If the code allowable tolerance for bar placement (+/-1/2 inch) occurs, the effects would be significant in increasing the required lap splice lengths,

All of this indicates that the lap splice lengths are significantly impacted by the reduced grout area utilized by HI-R and Hi-R H units. In the examples provided above, lap splice lengths of 118 inches and 53 inches are generally impractical.

Therefore, several possible options are proposed for splicing reinforcement in Hi-R and Hi-R H systems:
Consider using 12-inch Hi-R or Hi-R H units to maximize the grout area available, and

- Use small bars that are more closely spaced, and
- Increase the f'_m, and
- Use high lift grouting techniques and avoid most lap splices, or
- Use mechanical splices to avoid lap splices altogether.

For a discussion on mechanical splices, refer to "Splicing Options for Masonry Construction" by Healy and Biggs in the January 2007 edition of Masonry Construction magazine reprinted at www.resources.concreteproductsgroup.com. Log on and go to the article under Design Notes

No matter what design method is used, specify special inspections to verify the accuracy of the bar placement.

Note: The engineer should be aware that IBC has allowed alternate lap splice lengths that differ from what is used in TMS 402. The alternate method may yield shorter splice lengths compared to TMS 402. However, the IBC method is based upon historical data where bars are placed in the center of walls without any insulation inserts. For that reason, CPG recommends using the TMS 402 methodology with Hi-R and Hi-R H systems rather than the IBC alternate.

c. Movement Joints

The National Concrete Masonry Association offers several TEK Notes that address control joints (movement joints) in concrete masonry walls. These joints control the cracking due to shrinkage and thermal effects.

Under certain conditions, architects and engineers can use either reinforcement in bond beams or horizontal joint reinforcement to control cracking and extend the spacing between the movement joints in comparison the not using any horizontal reinforcement. The TEK notes that apply to the design layout of movement joints include:

TEK 10-1a, Crack Control In Concrete Masonry Walls

TEK 10-2c, Control Joints For Concrete Masonry Walls – Empirical Method

TEK 10-3, Control Joints For Concrete Masonry Walls - Alternative Engineered Method

Engineers are encouraged to study these documents for developing a layout for movement joints using Hi-R and Hi-R H systems.

d. Horizontal Joint Reinforcement

As previously stated, horizontal joint reinforcement is a design option for the architect and engineer. Horizontal joint reinforcement is an option to using mild reinforcement in bond beams.

If the designers choose to utilize horizontal joint reinforcement in the CMU wall design, the horizontal joint reinforcement can be used for crack control and/or for structural reinforcement.

Figure 7a shows a wall prior to installing the joint reinforcement. The insulation inserts set down into the unit to allow the joint reinforcement to be placed without interference. Figure 7b shows a wall with the joint reinforcement and mortar in place. As with standard masonry units, ladder-type reinforcement is preferred for use with Hi-R and Hi-R H walls to minimize interference with vertical reinforcement in the cells.

Figure 7a – prior to joint reinforcement

Figure 7b – with joint reinforcement

As mentioned in the Movement Joints section, using joint reinforcement in a design helps determine the spacing for the movement joints in accordance with the NCMA TEK notes. Dependent upon the crack control design, joint reinforcement can be used alone or in combination with reinforced bond beams.

While horizontal joint reinforcement is primarily use for crack control in CMU walls, it can also be used as structural reinforcement. As one example, walls spanning horizontally can use the longitudinal wires for flexural tension resistance. Only the longitudinal wire on the tension side at any section is effective.

A second example of horizontal joint reinforcement as structural reinforcement is for shear reinforcement in wind and low seismic zones. The longitudinal wires serve as the A_v for shear capacity. TMS 402, Section 9.3.3.7 allows joint reinforcement as shear reinforcement in high seismic zones with several limitations.

e. <u>Connectors</u>

When designing connectors for HI-R and Hi-R H walls, the engineer must take into account the presence of the insulation inserts. Figures 8-10 show several options to consider. TMS 402 provides design criterion for the embedded bolts.

Figure 8 requires either of two options for the bolt support at the exterior of the wall:

1. The bolt is supported by the exterior face shell. The hole in the shell should be fully-mortared tight and the bolt spans the insulation space. TMS does not cover

this design condition. The engineer should use first principles with the code criteria.

2. The bolt is not supported by the exterior face shell. The hole in the face shell should be loose around the bolt so that the bolt cantilevers from the grout and through the insulation space and face shell.

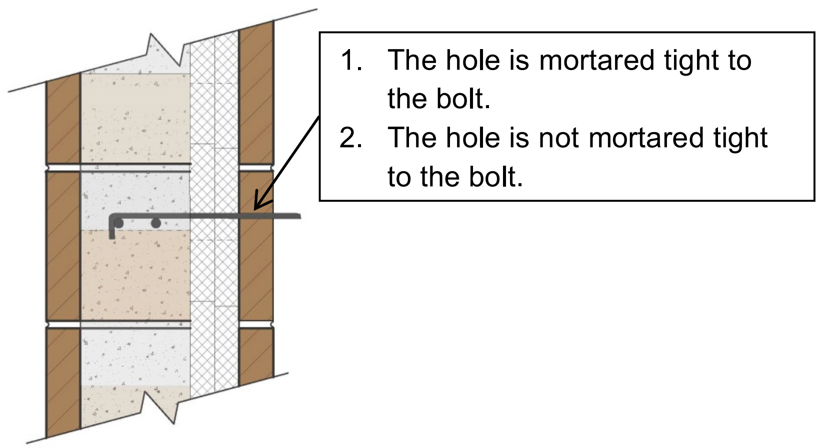


Figure 8 - bolting to exterior

Figure 9 requires the insulation inserts to be removed at the bolt. The hole in the face shell should be fully-mortared tight. In partially-grouted walls, the engineer must decide if grouting one cell is adequate to support the loads on the bolt. Otherwise, multiple cells may have to be grouted.

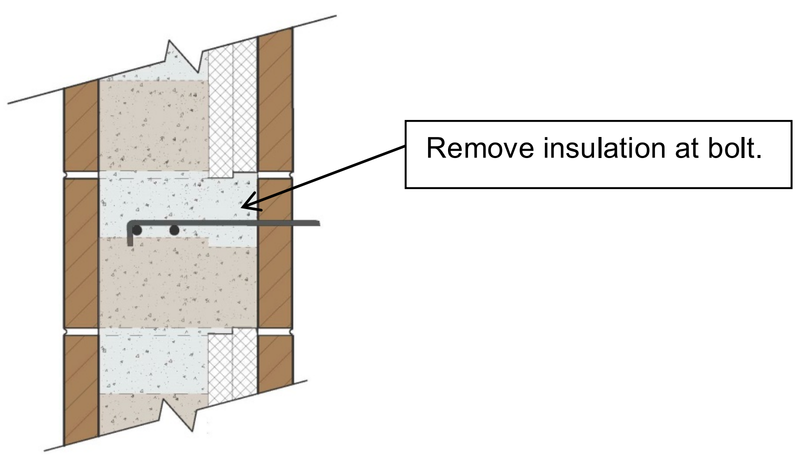


Figure 9 – modification for bolting to exterior

Figure 10 shows the bolt supported by the interior face shell. The hole in the shell should be fully-mortared tight.

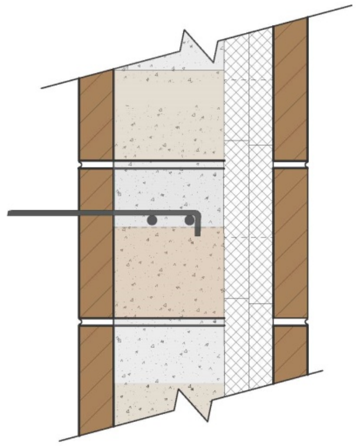


Figure 10 – bolting to interior

f. Beams and Lintels

1. Testing in 1986 on Hi-R specimens indicated that the stresses determined by ASD were consistent with theory.

2. Comparing the test results to Strength Design procedures, the shear stresses are consistent with test results when the design width, *b,* for the member is equal to the grouted area plus one face shell.

However, the flexural capacity was determined to be 70% of M_n as calculated by TMS 402. Therefore, CPG recommends that engineers use:

M_n = 0.7 ($A_s f_y$ (d-a/2) for beams and lintels.

Using the ϕ = 0.9 from the code,

M_u (capacity) = ϕ [0.7 ($A_s f_y$ (d-a/2)] = 0.63($A_s f_y$ (d-a/2)

3. For the deflection of Hi-R and Hi-R H beams and lintels with simple supports:

$$\delta_s = \frac{5\, M_a l^2}{48 E_m I_{eff}}$$

$$I_{eff} = I_n \left(\frac{M_{cr}}{M_a}\right)^3 + \quad I_{cr}\left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] \leq I_n$$ TMS Eq. 5-1

$I_n = \frac{bh^3}{12}$ $\quad M_{cr} = f_r\, S$ where f_r is obtained from Table 9.9.1.2. TMS 5.2.1.4.3

where the *b* for beams and lintels is conservatively the grouted area and the interior face shell as shown in Figures 11a and 11b. Figure 11a shows the bond beam unit in its typical orientation. Figure 11b shows the bond beam inverted so that the reinforcement is lower and there is an increased "*d*" distance.

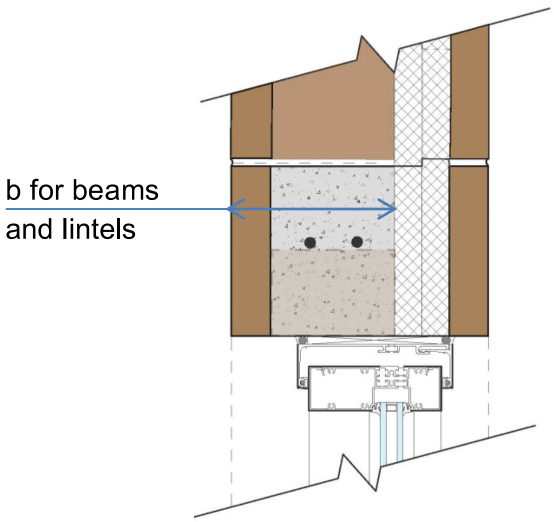


Figure 11a – Beams and lintels

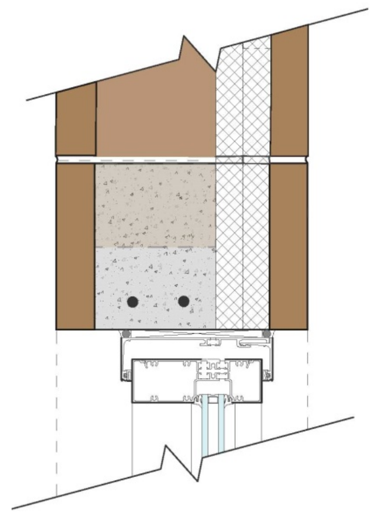


Figure 11b – Inverted unit

g. Out-of-Plane Loadings (Flexure)

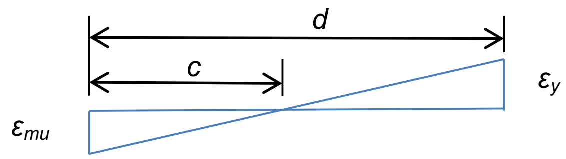
1. Most structural engineers are familiar with conventional reinforced masonry for buildings where the reinforcement is either placed in one layer centered in the wall or is placed in two layers that are equidistant from the unit center line. Structurally, both of these options provide a symmetrical section with equal strength in both directions for out-of-plane loadings. While not commonly done, the vertical reinforcement can be placed unsymmetrically if loadings dictate a need. An example where this would be more common is for earth retaining walls where the dominant loading is primarily from one side only.

   As previously discussed, Hi-R and Hi-R H units have insulation inserts that result in unsymmetrical grouted areas as noted previously in the Section Properties. For out-of-plane loads that act both inward and outward, the wall has two section moduli with the weaker of the two usually controlling. However, both directions should be evaluated since the eccentricity of the axial loads could affect the design. Tables 1 through 3 provide structural properties that will be useful.

2. M_u (capacity):
   The equation in TMS Commentary 9.3.5.2 for moment capacity is intended for unsymmetrical or symmetrical reinforcement for sections where the centroid is at the center of the wall.

M_u (capacity)= $\phi\left[\left(A_S f_y + \frac{P_u}{\phi}\right)\left(\frac{t_{sp}-a}{2}\right) + (A_S f_y)(d - \frac{t_{sp}}{2})\right]$

The compression block is calculated as $a = \frac{A_S f_y + \frac{P_u}{\phi}}{0.8 f'_m b}$. The assumption inherent in this equation is that the steel yields at f_y. For that to happen, the *c* value must be limited which then limits the *a* value. The following figure shows the strain relationship between the masonry and the steel reinforcement.



From this, $\varepsilon_{mu}/c = \varepsilon_y / (d-c)$ and from flexural theory, *a = 0.8c.*

Based upon ε_{mu} = 0.0025 for CMU and ε_y = f_y/E_s = 0.0021 for Grade 60 reinforcement, the steel will yield for *a* < 0.44*d.* This "a maximum dimension" value to check for yielding is listed in Table 2. Due to the insulation inserts, the calculated "*a*" value for the exterior face shell should also be less than the thickness of the face shell when the exterior face is in compression.

<u>For full-grouted walls</u>

Since there is grout against the interior face shell, the "*a*" value can exceed the face shell thickness. This is also taken into consideration in Table 2.

In the unlikely condition that the actual calculated "*a*" exceeds the tabulated "*a* maximum dimension" for the interior face, the steel does not yield and the engineer must re-evaluate the actual steel stress and moment capacity.

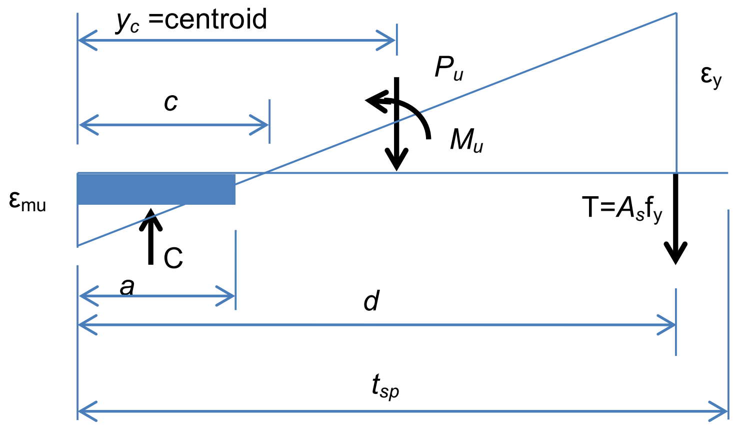
If the actual calculated "*a*" exceeds the tabulated "*a* maximum dimension" for the exterior face, a complicated T-beam analysis is required. Therefore, it is simpler to keep $a \leq t_{face\ shell}$ for the exterior face shell.

<u>For partially-grouted walls</u>

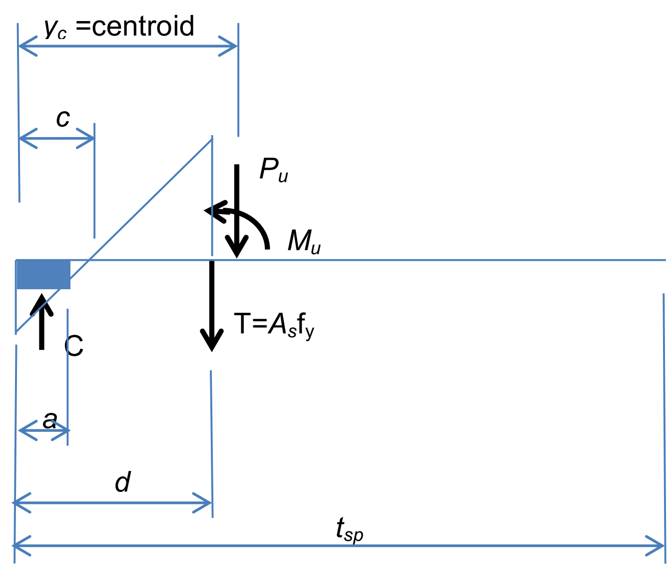
The "a" values for both the interior and exterior faces should not exceed the face shell thickness to avoid the T-beam analyses.

Returning to *M_u,* the following figure which is a combination of the stress and strain diagrams applies to the Hi-R and Hi-R H systems when *d* ≥ y_c *.* While the TMS equation is based upon *d* ≥ $t_{sp}/2$ where $t_{sp}/2$ represents the centroid of the center of the wall, the following generalized form of the equation applies to walls with Hi-R and Hi-R H units:

M_u *(capacity)*= $\phi\ [(A_S f_y)\ \left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c\ - \frac{a}{2})]$



The equation further applies when $d \leq y_c$ and $a < 0.44d$. See the next figure.



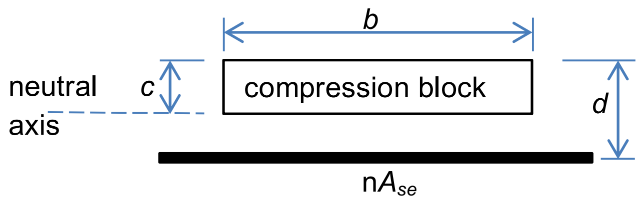
Therefore, the modified equation for Hi-R and Hi-R H units is:
M_u *(capacity)*= $\phi\ [(A_S f_y)\ \left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c\ - \frac{a}{2})]$ when $a < 0.44d$.

3. Wall Deflections

For service load deflections, $\delta_s = \frac{5\,M_{cr}h^2}{48E_mI_n} + \frac{5\,(M_{ser} - M_{cr})\,h^2}{48E_mI_{cr}}$.

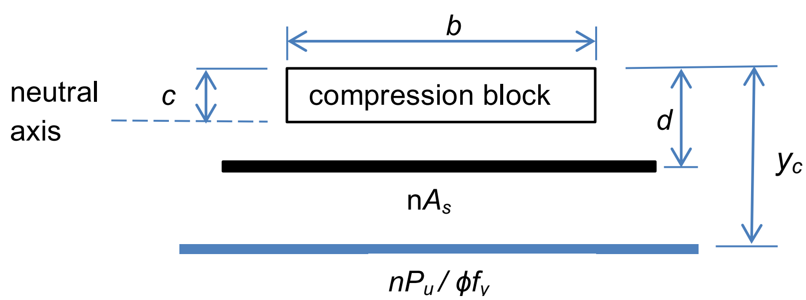
The I_n value is provided in Table 1 for full-grouted walls and Table 3 for partially-grouted walls..

I_{cr} is based upon the following section where the reinforcement and axial load are transformed into an equivalent area of masonry.



A_{se} is the effective area of steel that includes the actual area of steel plus the effect of the axial load based upon elastic stresses. It is derived from the more generalized diagram that includes the area of steel and the axial effect separately. Both are transformed into equivalent masonry areas.

c is the compression block = $\frac{A_sf_y + \frac{P_u}{\phi}}{0.64f'_mb}$ TMS Eq. 9-35



Equating these two diagrams for Hi-R and Hi-R H walls, the A_{se} becomes:

$$A_{se} = A_s + \left(\frac{P_u}{\Phi f_y}\right)\frac{(y_c - \frac{c}{2})^2}{(d - \frac{c}{2})^2}$$

Note: This calculation of A_{se} is only for use with I_{cr} because it was developed based upon elastic stresses.

Now turning to the general form of the cracked moment of inertia,

$I_{cr} = I_0 + Ad^2 = n\,(A_{se})(d-c)^2 + \frac{bc^3}{3}$. First solve for A_{se} and then I_{cr}.

Note: In TMS 402, this equation is approximated by the following for the specific condition of $y_c = t_{sp}/2$.

$$I_{cr} = n\,(A_s + \frac{P_u}{\Phi f_y}\frac{t_{sp}}{2d})(d-c)^2 + \frac{bc^3}{3}\,. \qquad \text{TMS Eq. 9-34}$$

h. Out-of-Plane Loadings (Web Shear)

1. For out-of-plane loadings, one special design aspect for HI-R and Hi-R H systems that should be addressed is shear. It is very uncommon to evaluate shear for out-of-plane loading in a masonry wall, especially when the wall is fully grouted. However since the insulation inserts produce a grout bond discontinuity with the exterior face shells, the shear transfer from the interior face shell and the grout to the exterior face shell is achieved totally by the webs.

   There are criterion in TMS 402 (Section 8.2.6.3 for ASD and Section 9.2.6.2 for SD) criteria for evaluating the web(s) of the units. The criterion requires that the unit either satisfy the minimum requirement for normalized web area of 6.5 in^2/ft^2 or that shear stresses/forces be checked. Since the Hi-R and Hi-R H units do not meet the normalized area criteria, engineers must make a design check of the shear transfer between the webs and the face shells. To assist engineers with performing this shear check, CPG has created Tables 8, 9 and 10 for both ASD and SD procedures that apply to fully-grouted and partially-grouted walls.

2. If the engineer is designing with Allowable Stress Design procedures, the web shear check uses:

   $$f_v = \frac{VQ}{I_n b} \qquad \text{TMS Section 8.2.6.1, Eqn 8-20}$$

   This equation was developed for unreinforced masonry using service loads. The allowable stress is based upon Section 8.2.6.3 which refers to:

   $$F_v = 1.5\sqrt{f'_m}\,. \qquad \text{TMS Section 8.2.6.2 (a)}$$

   Using a minimum f'_m = 2,000 psi, F_v = 65 psi. For higher strength units of 3,250 psi with f'_m = 2,500, the F_v = 75 psi.

   If the engineer is designing with Strength Design procedures, the web shear check uses:

   $$V_n = 3.8\sqrt{f'_m}\,I_n\,b/Q \qquad \text{TMS Section 9.2.6.2}$$

Note: This differs from the code version that has an error that has been corrected in the 2016 TMS 402 edition.

The effective ultimate stress here is $3.8\sqrt{f'_m}$.

Using a minimum f'_m = 2,000 psi, the effective ultimate stress = 170 psi. If you require high strength units of 3,250 psi with f'_m = 2,500 psi, the effective ultimate stress = 190 psi.

These values are extremely low when you consider that testing on Hi-R walls (*Research Investigation of the Structural Properties of Korfil Hi-R Concrete Masonry* by National Concrete Masonry Association and Englekirk-Hart, Consulting Engineers, March 1986) achieved ultimate shear stresses close to 1,000 psi before shear failures occurred that produced face shell delamination. These shear values were achieved on units with a net area compressive strength = 2,810 psi and assumed f'_m = 1,700 psi.

To aid in the web shear check analysis, Table 8 provides values of web shear coefficients that were developed specifically for the Hi-R and Hi-R H units.

For ASD, $f_v = \frac{VQ}{I_n b} = K_Q\,(V)$ where $K_Q = \frac{Q}{I_n b}$. This is a constant for each unit type of masonry unit.

For SD, $V_n = 3.8\sqrt{f'_m}\,I_n\,b/Q = \frac{3.8\sqrt{f'_m}}{K_Q} = K_S(\sqrt{f'_m})$

$K_S = 3.8/K_Q$

Table 8 – Web Shear Coefficients		
Units	Allowable Stress Design	Strength Design
	K_Q	K_S
10” Hi-R	0.070	64.3
12” Hi-R	0.048	79.2
10” Hi-R H	0.143	30.2
12” Hi-R H	0.111	39.6

Note: These apply to fully-grouted and partially-grouted walls.

From these values, Tables 9 and 10 were developed to give engineers the allowable shear capacity ($V_{capacity} = \frac{F_v}{K_Q}$) for ASD procedures as well as the ultimate shear capacity $V_n = K_S\,(\sqrt{f'_m})$ for SD procedures.

Table 9 – Allowable Web Shear (lb/ft) of wall using ASD with unfactored loads					
f'_m	$F_v = 1.5\sqrt{f'_m}$	$V_{capacity}\ (\frac{lb}{ft}\ of\ wall)$			
		10” Hi-R	12” Hi-R	10” Hi-R H	12” Hi-R H
1,900	65.4	934	1,363	457	589
2,000	67.1	959	1,398	469	605
2,500	75.0	1,071	1,563	524	676
3,000	82.2	1,174	1,713	575	741
3,500	88.7	1,267	1,848	620	799
4,000	94.9	1,356	1,977	664	855
4,500	100.6	1,437	2,096	703	906

Note: These apply to fully-grouted and partially-grouted walls.

Table 10 – Allowable Web Shear (lb./ft.) of wall using SD with factored loads					
f'_m	$\sqrt{f'_m}$	$V_n\ (\frac{lb}{ft}\ of\ wall)$			
		10” Hi-R	12” Hi-R	10” Hi-R H	12” Hi-R H
1,900	43.6	2,673	3,606	1,317	1,727
2,000	44.7	2,740	3,697	1,350	1,770
2,500	50.0	3,065	4,135	1,510	1,980
3,000	54.8	3,359	4,532	1,655	2,170
3,500	59.2	3,629	4,896	1,788	2,344
4,000	63.2	3,874	5,227	1,909	2,503
4,500	67.1	4,502	5,549	2,026	2,657

Note: These apply to fully-grouted and partially-grouted walls.

i. In-Plane Loadings (Shear)

1. Based upon their aspect ratio (length/height) shear walls are either dominated by flexure or shear. Flexure-dominated shear walls are generally taller than they are long while shear-dominated walls are generally longer than they are tall.

   For SD procedures, flexure-dominated shear walls rely on the vertical reinforcement to yield and provide inelastic response. Shear-dominated walls don’t impose the same demand on the vertical reinforcement; the walls perform as shear elements. TMS 402 provides criteria for identifying and designing both types of shear walls.

2. One special concern is in regards to lap splice lengths of vertical reinforcement as previously noted. Due to the reduced grout area as result of

the insulation inserts, highly stressed large bars require significant lap lengths. Here, the engineer should avoid lap splices whenever possible.

3. A second concern is the grouted area of the cells. Since the insulation inserts occupy a portion of the grouted cells in-plane, shear transfer within the grouted cells should be considered. Although the walls are fully grouted, the presence of the insulation inserts suggests using γ_g = 0.75 that was developed for partially-grouted walls. CPG recommends that engineers use TMS Section 9.3.4.1.2 with γ_g = 0.75 for shear wall design to conservatively reflect the shear capacity of the system.

   The A_{nv} used in the shear calculations includes both face shells and the grouted cell area.

   TMS Section 9.3.4.1.2 is as follows:

   "9.3.4.1.2 Nominal shear strength — Nominal shear strength, V_n, shall be calculated using Equation 9-21, and shall not be taken greater than the limits given by 9.3.4.1.2 (a) through (c).

   $V_n = (V_{nm} + V_{ns})\gamma_g$ *(Equation 9-21)*

   $V_{nm} = [4.0 - 1.75(\frac{M_u}{V_u d_v})]\, A_{nv}\sqrt{f'_m} + 0.25\, P_u$ (Equation 9-24)

   $V_{ns} = 0.5\left(\frac{A_v}{s}\right) f_y d_v$ (Equation 9-25)

   (a) Where $M_u/V_u d_v \leq 0.25$

   $V_n \leq 6A_{nv}\sqrt{f'_m}\gamma_g$ *(Equation 9-22)*

   (b) Where $M_u/V_u d_v \geq 1.0$

   $V_n \leq 4A_{nv}\sqrt{f'_m}\,\gamma_g$ *(Equation 9-23)*

   γ_g *= 0.75 for partially grouted shear walls and 1.0 otherwise.*

   (c) The maximum value of V_n *for* $M_u/V_u d_v$ *between*

   0.25 and 1.0 shall be permitted to be linearly interpolated."

j. <u>Axial Loadings</u>

1. Given that Hi-R and Hi-R H systems are unsymmetrically reinforced, axial loads on walls need to be assessed based upon their location relative to the center of gravity (y_c) of the section. For fully-grouted standard units, the center of gravity is in the center of the wall. Table 1 provides the center of

gravity location for fully-grouted walls constructed from standard units as well as fully-grouted HI-R and Hi-R H units. Table 3 provides the center of gravity location for partially-grouted walls constructed from HI-R units.

2. Concentrated vertical loads on masonry walls can be handled in several ways. The first is to analyze the wall directly below the concentration. TMS 402 provides guidance on load distribution. Figure 12 was taken from a portion of TMS 402 Commentary figure to illustrate load distribution for a wall constructed in running bond. See TMS 402 for additional information.

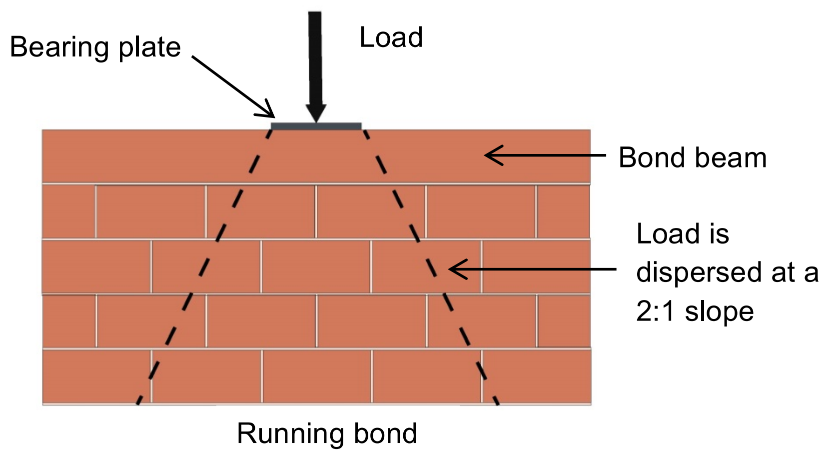


Figure 12 - taken from Figure CC-5.1-5. in TMS 402

3. A second option for supporting concentrated loadings is to provide pilasters. These pilasters can be interior or exterior (Figure 13). for either type, it is important to maintain the continuity of the insulated portion of the wall for thermal reasons.

   The interior pilaster of Figure 13a is easier to construct. The exterior pilaster of Figure 13b maintains the flush wall on the interior that is desirable to many owners.

   The shaded areas in Figures 13a and 13b represent pilasters constructed of standard masonry units that are reinforced and integrated into the wall. TMS, *Figure CC-5.4-1 — Typical pilasters* shows several variations on CMU pilasters that could be used.

   The shaded portions of Figure 13 can be designed as the pilasters alone or the wall can be designed to act compositely with them. For economy of construction, the pilaster portions should use full and half units to avoid significant cutting. For further economy, loads should be applied concentrically to the shaded areas of the pilasters.

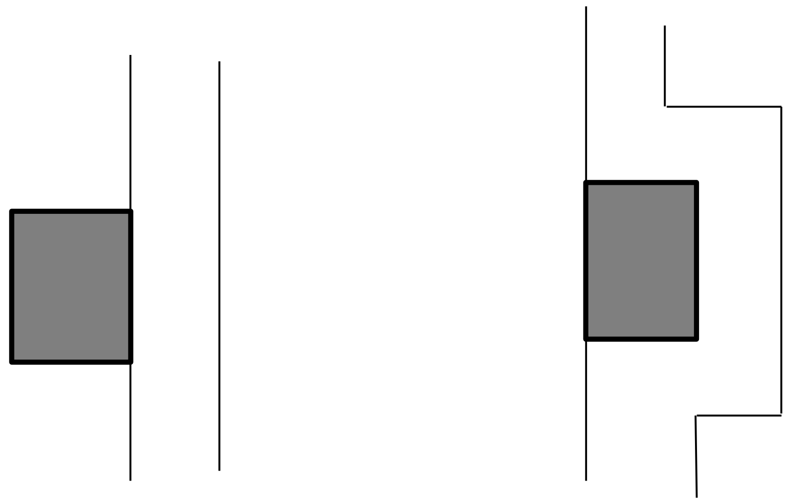


Figure 13 – Pilasters a) Interior b) Exterior

k. <u>Computer Software</u>

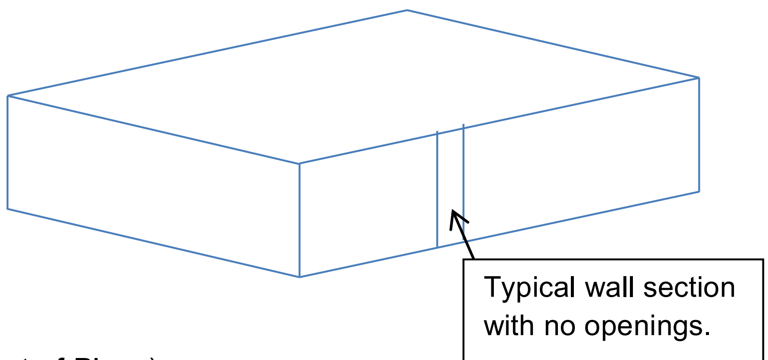
There are numerous computer programs available commercially for the analysis of structures. Many are Finite Element Analysis (FEA) programs and are not masonry specific. Within these programs, the user can create material and section properties specific to their project. Tables 1 and 3 offer section properties such as areas and moments of inertia and Tables 4 and 5 provide wall weights that can be used directly in these programs. The output obtained from the FEA models (moments and shears) must then be used with the section moduli and other section properties to provide the stresses and reinforcement.

For masonry specific software, such as the *Structural Masonry Design System* which allows the user to design concrete and clay masonry, the programs are generally based upon symmetrically reinforced sections. Therefore, these programs do not currently accept Hi-R and Hi-R H units directly because the units have two different d distances making them unsymmetrically reinforced. Therefore, the user must perform two analyses (one with the lateral load inward and the other with the lateral load outward) for each Hi-R or Hi-R H design. This will be demonstrated in an example.

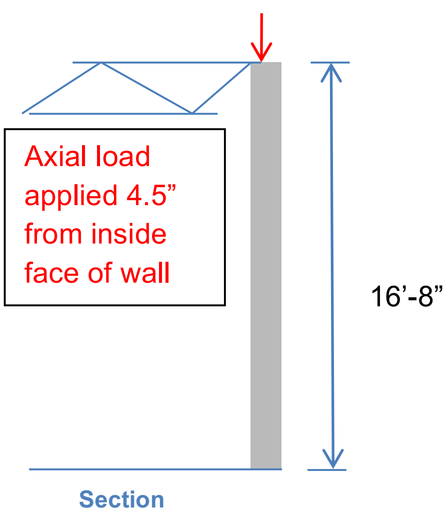
Design Examples

The following examples are intended to illustrate the design methodology previously noted using SD procedures. The engineer can determine whether to use ASD or SD design procedures for their project. For each example, the loads will be provided as though they were developed from IBC and/or ASCE 7. In addition, the material properties will be assumed. The design is based upon TMS 402 and appropriate references will be provided.

1. ONE-STORY BUILDING (FULLY-GROUTED WALLS)



Flexural Design (Out-of-Plane)



Materials:

12” Hi-R, fully grouted, running bond
Type S mortar (P-C-L)
Grout 3,000 psi (f'_g); density = 125 pcf.
CMU Density = 120pcf;
wall weight = 94 psf (Table 5 for grout density =125 pcf)
f'_m = 2,500 psi
$E_m = 900\ f'_m = 2.25 \times 10^6$ psi TMS 4.2.2.2.1
f_y = 60,000 psi; $E_s = 29 \times 10^6$ psi
$n = E_s/E_m = 12.9$
Assume simple supported vertical wall.

Loads (Unfactored):

Roof dead load: 400 plf
Roof snow load: 500 plf
Load applied on bearing whose centerline is 4.5 inches from inside face of wall.

Loads (Factored):

Site lateral loads that would be determined from ASCE 7-10, assuming an SDC C site.
S_{DS} =0.4
ρ = 1.0
Wind: 35 psf
Seismic: F_p = 0.22w = 20.7psf

For this example, we'll check one load combination. For an actual project, check all pertinent load combinations. Note: the wind loads are higher than the seismic for this example but we are only checking the seismic.

We will have the seismic loadings acting outward on the wall. We selected the outward direction because this places the interior side of the wall into compression thereby requiring the smaller *d* distance for the section. In addition, the seismic loads will be additive to roof eccentricity effects.

Let's try U = (1.2 + 0.2S_{DS})D + ρQ_E + L + 0.2S (ASCE 7-10, Load Combination 5 with vertical component of earthquake per 12.4.2.3)

Axial load at bearing: P_{uf} = (1.2 + 0.2(0.4))200 + 0.2(500) = 356 lb/ft
Eccentricity at top: ecc.= (y_c - 4.5 in.) = (5.0 from Table 1 - 4.5) = 0.5 inches.
Moment from axial: P_{uf} ecc. = 178 in-lb = 15 ft-lb/ft (tension on exterior)
Moment due to lateral loads: $M_{u,0} = F_p\,(16.67)^2/8$ = 719 ft-lb/ft (use E outward so its additive to P_{uf} ecc.). This produces flexural tension on exterior.

At mid-height, $M_{u,0}$ = 719 + 15/2 = 727 ft-lb/ft
$P_u = P_{uf} + P_{uwall}$ = 356 + (1.28)(16.67/2)94 = 1359 lb/ft
$V_u = F_p$ (h/2) = 173 lb << ϕ(4,135)lb where 4,135 is taken from Table 10 for web shear check using only one foot of the wall which is extremely conservative.
V_u (capacity)> 0.8(4,135) = 3,308 lb **OK**

h/t = 16.67(12)/11.625 = 17.2 < 30
P_u/A_g = 1359/(11.625 x 12) = 10 psi << 0.2 f'_m **OK** TMS Section 9.3.5.4.

Assume an "*a*" value close to the face shell thickness (*a* = 1.6") to get a first estimate for A_s.
We'll have to check this assumption later.
Use *d* = 4.6 inches from Table 2.
Note that y_c = 5.0 inches from Table 1.
Capacity reduction factor, Φ = 0.9 TMS 9.1.4.4 (Flexure)

Using M_u based upon these assumptions, solve for A_s:

$$M_u = \phi\ [(A_S f_y)\left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c - \frac{a}{2})]$$

$$A_s = [M_u - (P_u)(y_c - \frac{a}{2})]\ /[\phi f_y \left(d - \frac{a}{2}\right)]$$
$$A_s = [727x12 - (1359)(5.0 - \frac{1.6}{2})]\ /[0.9(60{,}000)\left(4.6 - \frac{1.6}{2}\right)] = 0.02\ \text{in}^2/\text{ft}$$

To account for second order moments that will be added, use a higher value.
For this example, we'll try #4 @ 48 in. oc (A_s = 0.05 in^2/ft)

Perform a second-order analysis per TMS 402 using the P-Delta Method of TMS Section 9.3.5.4.2 or the Moment Magnifier method of TMS Section 9.3.5.4.3. For this example, we'll use the P-Delta method.

M_u at midheight = 727 + $P_u\delta_u$
[$M_{u.0}$ = 727 was previously calculated from $w_u h^2/8 + P_{uf} e_u/2$ for TMS Eq. 9-27]

Values used or calculated include:

$S_{interior}$ = 249 in^3/ft from Table 1.

f_r = 84 psi from TMS 402 Table 9.1.9.2. Due to the presence of the insulation inserts, the ungrouted value is used to be conservative.

I_n = 1,394 in^4/ft from Table 1

Calculate:

$M_{cr} = S_{interior} \times f_r$ = 1,743 ft-lb/ft

Note: $M_{cr} > M_u$, so theoretically the wall does not crack. Therefore slender wall theory is hardly necessary for this design. So, we will skip the iterations and moment magnifier.

The code also requires a check of the service deflection.

M_{ser} = [20.7/1.6](16.67)^2/8 = 449 ft-lb.

$\delta_s = \frac{5 M_{ser} h^2}{48 E_m I_n} = \frac{5\,(449\,x12)(200)^2}{48(2.25x10^6(1{,}394)} =$ 0.01 in. << the allowable of 0.007h (2.0 inches) **OK** .
Justifies not using any second order effects.

Checking the M_u(capacity):

$$a = \frac{A_s f_y + \frac{P_u}{\phi}}{0.8 f'_m b} = \frac{(0.05)(60{,}000) + \frac{1{,}359}{0.9}}{0.8(2{,}500)12} = 0.19 \text{ in.}$$

Note: "*a*" maximum from Table 2 = 2.02 in. **OK**

M_u (capacity) = $\phi\ [(A_S f_y)\left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c - \frac{a}{2})]$

M_u (capacity) = $0.9\ [(0.05(60{,}000)\left(4.6 - \frac{0.19}{2}\right) + \frac{1{,}359}{0.9}(5.0 - \frac{0.19}{2})]/12$ = 1,278 ft-lb.

Therefore, the design works since M_u (capacity) exceeds M_u (applied) = 729 ft-lb/ft and #4@48 in. oc vertical reinforcement is adequate for this load combination.

Note: TMS requires the engineer also check P_n for the h/r using TMS Equations 9-19 and 9-20. These calculations are not shown here but the engineer should do them.

Please note that this design was for one load combination. Check all the necessary combinations using the same procedures. One method for doing this would be to create an interaction diagram for the wall design and plot the load combinations onto the diagram. The *2013 Masonry Designer's Guide* provides directions as to creating the diagram.

Remember, the previous example was developed for the seismic loading outward from the wall. Now, let's consider what would be different had the seismic loading been in the opposite direction (inward on the wall).

a. The moment at midheight would have been reduced due to the roof eccentricity.
b. The *d* distance for the reinforcement would have been greater (Table 2).

c. The section modulus (Table 1) is reduced and the cracking moment would also be reduced, but still greater than M_u.
Thus, the reinforcement would likely be adequate for this load case also. The engineer should perform the calculations to validate this conclusion.

Using Table 6, we can check that #4 bars are less than the maximum allowed in the cell.

Using K and γ values from Table 7, calculate the splice length as:

$$l_d = \frac{0.13\ d_b{}^2 f_y\, \gamma}{K\ \sqrt{f'_m}} = \frac{0.13\ (0.5)^2 60{,}000\ (1.0)}{2.55\ \sqrt{2{,}500}} = 16 \text{ inches.}$$

In this calculation, $\gamma = 1.0$ for the #4 bar (TMS Section 9.3.3.3). In the previous discussion on lap splices, the #6 bar had a $= 1.3$. So, by using a smaller bar diameter (#4 versus #6) and a higher f'_m , the required lap length dropped from 53 inches to 16 inches.

Check the maximum reinforcement percentage (TMS Section 9.3.3.5)

$$\rho_{max} = \frac{0.64 f'_m \left[\dfrac{\varepsilon_{mu}}{1.5\varepsilon_y + \varepsilon_{mu}}\right] - \dfrac{P_u}{bd\varphi}}{f_y}$$

$$\rho_{max} = \frac{0.64(2{,}500)\left[\dfrac{0.0025}{1.5(0.00207) + 0.0025}\right] - \dfrac{1359}{12(4.6)0.9}}{60{,}000} = 0.0115$$

$$\rho_{actual} = \frac{0.20}{48\ x\ 4.5} = \ 0.001 \ll \ \rho_{max} \quad \textbf{OK}$$

Now let's look at designing the same out-of-plane example using the ***Structural Masonry Design System 7.0*** software. The current version of the software uses *2011 TMS 402* while this Manual is based upon *2013 TMS 402*. Since both versions use ASCE 7-10 load combinations, the software should be acceptable.

As a reminder, the example used 12" Hi-R units. From Table 2, we find these units have a *d* = 7.1 inches from the exterior and *d* = 4.6 inches from the interior.

Pertinent project data to be maintained as close as possible: Wall weight = 94 psf. Because the software uses standard units and the Hi-R units have less grout, we might not be able to get the weight exact.

The calculations are run twice. The first is with the seismic loads acting outward from the wall to maximize the moment for the interior *d*. The second is with the loads acting inward.

Figure 14 is taken from the first run with *d* = 4.6 in. The software assumes $y_c = t_{sp}/2$ or 5.81 in. whereas the actual is 5.0 in. from Table 1. Thus, the computer analysis should be conservative.

We reduced the unit weight as best was possible to account for the actual weight of the Hi-R wall fully grouted. The most significant variation is in the f_r and the over-estimation of I_n. We assume f_r is based upon a partially-grouted value whereas the computer uses the fully-grouted value. Both values will affect the deflection calculations. It is not significant to this problem, but the engineer should evaluate on a case-by-case basis.

From the computations, we get a comparable *a* value. Also, the M_u (1,489 ft-lb) is approximately the calculated value of 1,278 ft-lb.

Design of a Reinforced Masonry Wall with Out-of-Plane Loads
Using the 2011 MSJC Strength Design Code

Material and Construction Data
12 in. CMU, Full grout, running bond
Wall Weight = 120.2 psf
Actual = 94 psf
Type S Masonry cement / Air-entrained PCL Mortar, Coarse Grout
CMU Density = 105.0 pcf
f'_m = 2,500 psi (Specified)
$E_m = 900f'_m$ = 2,250,000 psi

Wall Design Details
Thickness = 11.63 in.
Height = 200.0 in. (Simply Supported Wall, Effective height = H)
x = 4.600 in.
#4 Bars, Fy = 60,000
Reinforcement Spacing = 48.00 in. On-Center
Effective Width = 48.00 in.

Wall Flexural Capacity with Specified Steel
a = 0.170 in.
d = 4.600 in.
M_n = 19,850 lb-in/ft
M_u available = 0.900(19,850) = 17,870 lb-in/ft (1,489 lb-ft/ft)

f_r = 153.0 psi
I_{gr} = 1,571 in^4/ft
Computer software does not allow changing these.

Figure 14 – *d* = 4.6 inches

Figure 15 repeats the calculations for *d* = 7.1. The software assumes $y_c = t_{sp}/2$ or 5.81 in. whereas the actual is 6.6 in. from Table 1. Thus, the computer analysis should be slightly unconservative. To compensate, we can increase the d by (6.6 - 5.8) = 0.8". Again, the weight is over estimated compared to our example.

Our calculated M_u would be 2,740 ft-lb compared to the computer solution 2,231 ft-lb.

Design of a Reinforced Masonry Wall with Out-of-Plane Loads
Using the 2011 MSJC Strength Design Code

Material and Construction Data
12 in. CMU, Full grout, running bond
Wall Weight = 120.2 psf
Type S Masonry cement / Air-entrained PCL Mortar, Coarse Grout
CMU Density = 105.0 pcf
f'_m = 2,500 psi (Specified)
$E_m = 900f'_m$ = 2,250,000 psi

Wall Design Details
Thickness = 11.63 in.
Height = 200.0 in. (Simply Supported Wall, Effective height = H)
x = 7.900 in.
#4 Bars, Fy = 60,000
Reinforcement Spacing = 48.00 in. On-Center
Effective Width = 48.00 in.

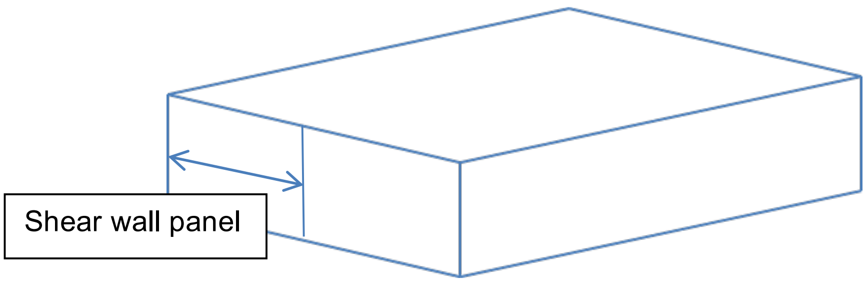
Wall Flexural Capacity with Specified Steel
a = 0.170 in.
d = 7.900 in.
M_n = 29,750 lb-in/ft
M_u available = 0.900(29,750) = 26,780 lb-in/ft (2,231 lb-ft/ft)

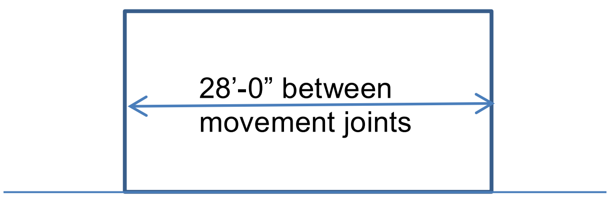
f_r = 153.0 psi
I_{gr} = 1,571 in^4/ft

Figure 15 – *d* = 7.1 inches compensated to 7.9 in.

Note that in both cases, the computer solutions did not give the exact values for M_u. Therefore, software solutions should only be used for preliminary design. The engineer should perform the hand calculations or complete their own spreadsheet to capture the accurate deflection calculations and wall weight.

Shear Wall Design (In-Plane)





Elevation

Materials:

Same as previous example.

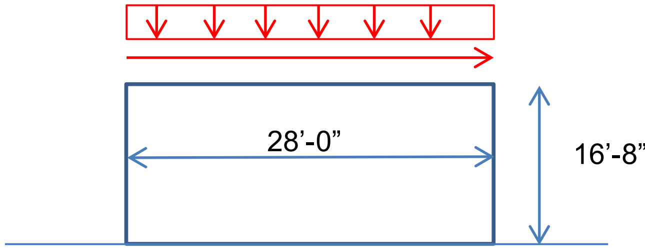
Loads:

Roof dead load: 50 plf
Roof snow load: 63 plf
Load applied on an edge angle bolted to face of wall. Load is 3.0 inches from face of wall.

Site lateral loads that would be determined from ASCE 7-10, assuming an SDC C site.

S_{DS} = 0.4 (from Out-of-plane example)
R = 5 (wall is fully grouted and meets Special Reinforced Masonry Shear Wall) from ASCE 7-10, Table 12.2-1.
I_e is given as 1.25

Wind shear at top from diaphragm:14,000lb
Seismic: E = 21,000 lb



For this example we will start with the reinforcement determined by the out-of-plane example, #4@48 in. on center. We will also use ASCE 7-10, Load Combination 7, U = 0.9D + 1.0E. Remember to check all the other combinations as well.

V = C_sW (ASCE 7-10, 12.8 Equivalent Lateral Force Procedure)

C_s = $S_{DS}/(R/I_e)$ = 0.091 and it is less than maximum required and more than minimum required.

P_{uDL} = 0.9(50 roof + 94 x 16.67 wall) = 0.9(50 + 1,567) = 1,455 lb/ft at base of wall
V_u = (21,000 roof + C_sW of wall) = (21,000 + 0.091(1,567 x28)) = (21,000 + 3,993)
V_u = 24,993 lb at base of wall
M_u = 21,000 (16.67) = 350,070 ft-lb at base of wall from the roof
3,993(16.67/2) = 33,282 ft-lb at base of wall from the wall
M_u = 383,352 ft-lb at base of wall

For the wall, d_v = 336 inches

$$\frac{M_u}{V_u d_v} = \frac{383{,}352(12)}{24{,}993(336)} = 0.55$$

Check shear: $V_u < \varphi\ V_n = \varphi\ (V_{nm} + V_{ns})$

$V_n\ maximum\ is\ between\ \left(4A_{nv}\sqrt{f'_m}\right)\gamma_g$ and $\left(6A_{nv}\sqrt{f'_m}\right)\gamma_g$ based upon $\frac{M_u}{V_u d_v}$.

Interpolating $\frac{0.55-0.25}{1.0-0.25}$ = 0.4, therefore, $V_n\ maximum = \left(4.8A_{nv}\sqrt{f'_m}\right)\gamma_g$.

$$V_n\ maximum = \left(4.8\left(110\frac{in2}{ft}from\ Table\ 1\ x\ 28ft\right)\sqrt{2{,}500}\right)(0.75) = 554{,}400\ lb\ \gg \frac{V_u}{\varphi}$$

V_{nm} $= [4.0 - 1.75(\frac{M_u}{V_u d_v})]\ A_{nv}\sqrt{f'_m}$ = $[4.0 - 1.75(0.55)]\ (110\ x28)\sqrt{2{,}500}$ = [3.04](154,000)

V_{nm} = 468,160 lb > $\frac{V_u}{\varphi}$ = $\frac{24{,}993}{0.8}$ = 31,241lb **OK**, no shear reinforcement required by loads.

For Special Reinforced Masonry Shear Wall (TMS 7.3.2.6)

Maximum spacing of vertical reinforcement = 48 in. or h/3 or l/3. <u>48 in. controls</u>
Maximum spacing of horizontal reinforcement = 48 in. or h/3 or l/3 <u>48 in. controls</u>
Minimum area of reinforcement = 0.002 x gross area = 0.002(11.625 x 28 x12)
Minimum area of reinforcement = 7.81 in^2

This results in A_v = 7@ 0.2in^2 = 1.4in^2

$A_{h\ \text{required}}$ = 7.81 - A_v = 6.41in^2 or 15 - #6 distributed over 16'-8". This requires too much horizontal reinforcement.

2nd Try: Increase A_v to #6 bars @ 4' gives A_v = 3.1in^2; Therefore require A_h = 11-#6

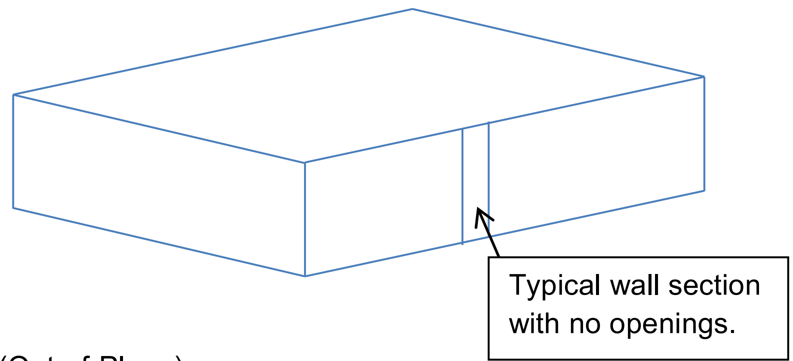
3rd Try: Increase A_v to #6 bars @ 32" gives 11-#6 or A_v = 4.8in^2; Therefore require A_h = 3.0in^2 or 7-#6 @ 24"

<u>Use</u> #6@32" vertical bars; #6@24" horizontal bars in bond beams

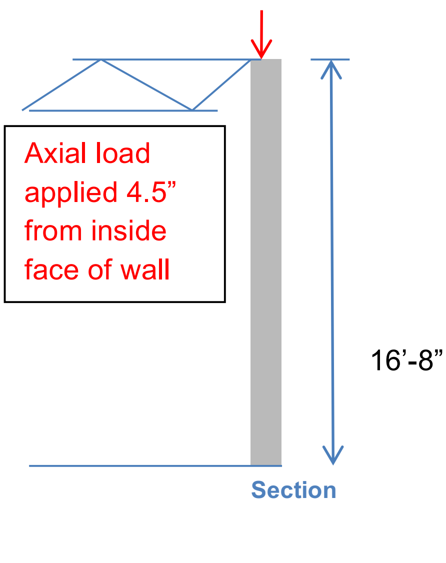
A second alternative for the overall shear wall design would be to treat the wall as Intermediate Reinforced masonry Shear Wall and reduce the minimum reinforcement. That includes redesigning the wall for an R=4. Based upon the excess capacity from the calculations performed, this will likely work. The reader is encouraged to do the example.

2. *ONE-STORY BUILDING (PARTIALLY-GROUTED WALLS)*

This is the same overall example as the previous one except that the walls are partially grouted.



Flexural Design (Out-of-Plane)



Materials:

12” Hi-R, partially grouted, running bond
Type S mortar (P-C-L)
Grout 3,000 psi (f'_g); density = 140 pcf
CMU Density = 120pcf;

assume grouted reinforcement at 40 in. on center to select an estimated wall weight = 56 psf due to partial grouting and +2 psf due to the grout density = 58 psf (Table 3).

f'_m = 2,500 psi
$E_m = 900\ f'_m = 2.25 \times 10^6$ psi TMS 4.2.2.2.1
f_y = 60,000 psi; $E_s = 29 \times 10^6$ psi
$n = E_s/E_m = 12.9$
Assume simple supported vertical wall.

Loads (Unfactored):

Roof dead load: 400 plf
Roof snow load: 500 plf
Load applied on bearing whose centerline is 4.5 inches from inside face of wall.

Loads (Factored):

Site lateral loads that would be determined from ASCE 7-10, assuming an SDC C site.

S_{DS} =0.4
ρ = 1.0
Wind: 35 psf
Seismic: F_p = 0.22w = 11.9 psf

Partially-grouted section properties: Compared to the first problem that used full grouting, this one uses partial grouting. The result is the wall weight is less (58 psf on previous page versus 94 psf for full grouting).
In addition, the centroid for the partially-grouted section is 5.4 in. (Table 3 for 12" Hi-R grouted 40 in on center) versus 5.0 in. for the fully-grouted wall. This increases the moment from the applied load.

For this example, we'll check one load combination using wind loads. For an actual project, check all pertinent load combinations.

We will have the wind loadings acting outward on the wall. We selected the outward direction because this places the interior side of the wall into compression thereby requiring the smaller *d* distance for the section. In addition, the wind loads will be additive to roof eccentricity effects.

Let's try U = 0.9D + 1.0W (ASCE 7-10, Load Combination 6)

Axial load at bearing: P_{uf} = (0.9)(400) = 360 lb/ft
Eccentricity at top: ecc.= (y_c - 4.5 in.) = (5.4 from Table 1 -4.5) = 0.9 inches.
Moment from axial: P_{uf} ecc. = 324 in-lb = 27 ft-lb/ft
Moment due to lateral loads: $M_{u,0}$ = 35(16.67)^2/8 = 1,216 ft-lb/ft (use W outward so its additive to P_{uf} ecc.). This produces flexural tension on exterior.

At mid-height, $M_{u,0}$ = 1,216 + 27/2 = 1,230 ft-lb/ft
$P_u = P_{uf} + P_{uwall}$ = 360 + (0.9)(16.67/2)58 = 795 lb/ft
V_u = *35*(h/2) = 292 lb << ϕ(4,135)lb where 4,135 is taken from Table 10 for web shear check using only one foot of the wall which is extremely conservative.
V_u (capacity)> 0.8(4,135) = 3,308 lb **OK**

h/t = 16.67(12)/11.625 = 17.2 < 30
P_u/A_g = 795/(11.625 x 12) = 6 psi << 0.2 f'_m **OK** TMS Section 9.3.5.4.

Assume an "*a"* value close to the face shell thickness (*a* = 1.6") to get a first estimate for A_s.
We'll have to check this assumption later.
Use *d* = 4.6 inches from Table 2.
Note that y_c = 5.4 inches from Table 1 (12" H-R grouted 40in on center).
Capacity reduction factor, Φ = 0.9 TMS 9.1.4.4 (Flexure)

Using M_u based upon these assumptions, solve for A_s:

$$M_u = \phi\ [(A_S f_y)\ \left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c\ - \frac{a}{2})]$$

$$A_s = [M_u - (P_u)(y_c\ - \frac{a}{2})]\ /[\phi f_y \left(d - \frac{a}{2}\right)]$$
$$A_s\ = [1{,}230x12\ - (795)(5.4\ - \frac{1.6}{2})]\ /[0.9(60{,}000)\left(4.6 - \frac{1.6}{2}\right)] = 0.06\ \text{in}^2/\text{ft}$$

To account for second order moments that will be added, use a higher value.
For this example, we'll try #4 @ 40 in. oc (A_s = 0.06 in^2/ft)

Perform a second-order analysis per TMS 402 using the P-Delta Method of TMS Section 9.3.5.4.2 or the Moment Magnifier method of TMS Section 9.3.5.4.3. For this example, we'll use the P-Delta method.

M_u at midheight = 1,230 + $P_u\delta_u$
[$M_{u.0}$ = 1,230 was previously calculated.]

Values used or calculated include:

$S_{interior}$ = 203 in^3/ft from Table 3.

f_r = 84 psi from TMS 402 Table 9.1.9.2. Due to the presence of the insulation inserts, the ungrouted value is used to be conservative.

I_n = 1,097 in^4/ft from Table 1

Calculate:

M_{cr} = $S_{interior}$ x f_r = 1,421 ft-lb/ft

<u>Note</u>: $M_{cr} > M_u$, so theoretically the wall does not crack. Therefore slender wall theory is hardly necessary for this design. So, we will skip the iterations and moment magnifier.

The code also requires a check of the service deflection.

M_{ser} = [35/1.6](16.67)^2/8 = 760 ft-lb.

$\delta_s = \frac{5\, M_{ser} h^2}{48 E_m I_n} = \frac{5\,(760\, x 12)(200)^2}{48(2.25x10^6(1{,}097)} =$ 0.02 in. << the allowable of 0.007*h* (2.0 inches) **OK** .
Justifies not using any second order effects.

Checking the M_u(capacity):

$$a = \frac{A_s f_y + \frac{P_u}{\phi}}{0.8 f'_m b} = \frac{(0.06)(60{,}000) + \frac{1{,}097}{0.9}}{0.8(2{,}500)12} = 0.20 \text{ in.}$$

<u>Note</u>: "*a* maximum" from Table 2 = 1.75 for partially grouted. **OK**

M_u (capacity) = $\phi\ [\left(A_s f_y\right)\left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c - \frac{a}{2})]$

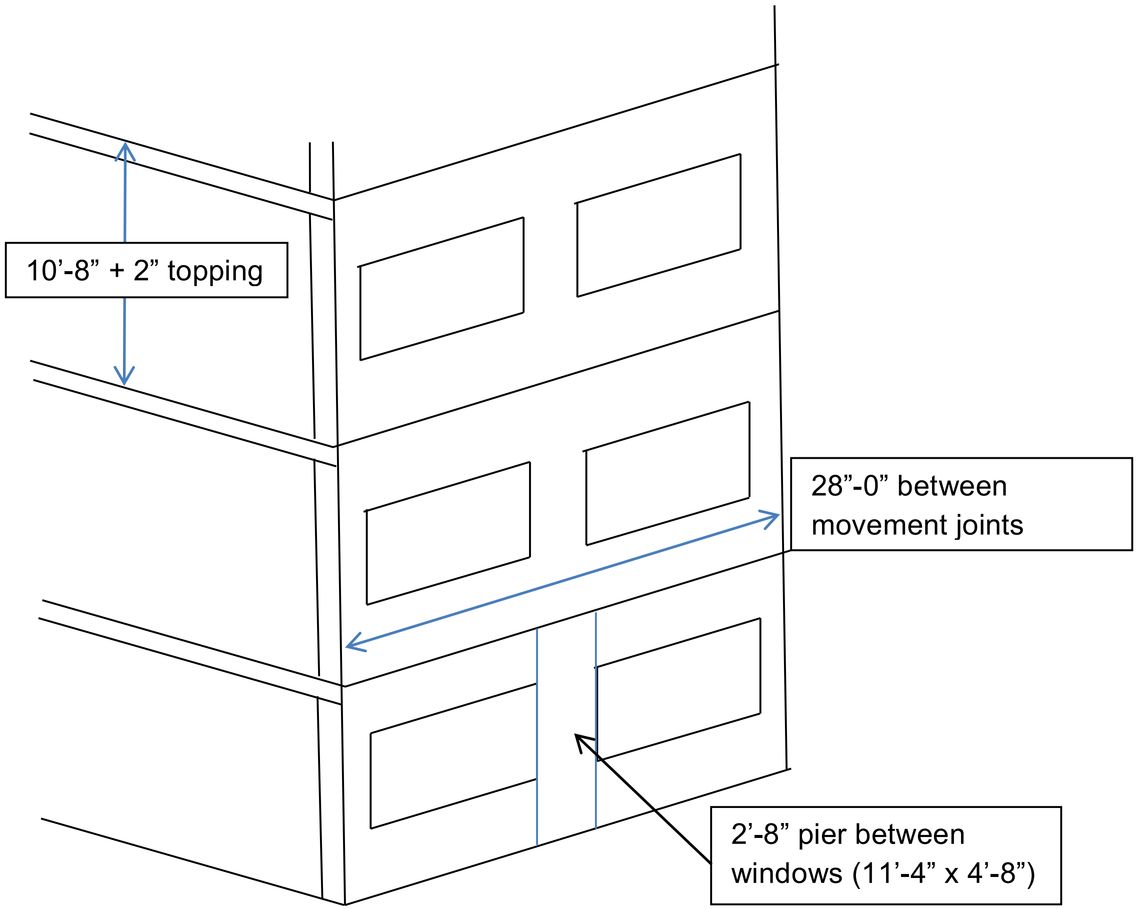
M_u (capacity) = $0.9\ [(0.06(60{,}000)\left(4.6 - \frac{0.20}{2}\right) + \frac{1{,}097}{0.9}(5.4 - \frac{0.20}{2})]/12$ = 1,700 ft-lb.

Therefore, the design works since M_u (capacity) exceeds M_u (applied) = 1,230 ft-lb/ft and #4@40 in. oc vertical reinforcement is adequate for this load combination.

<u>Note</u>: TMS requires the engineer also check P_n for the *h/r* using TMS Equations 9-19 and 9-20. These calculations are not shown here.

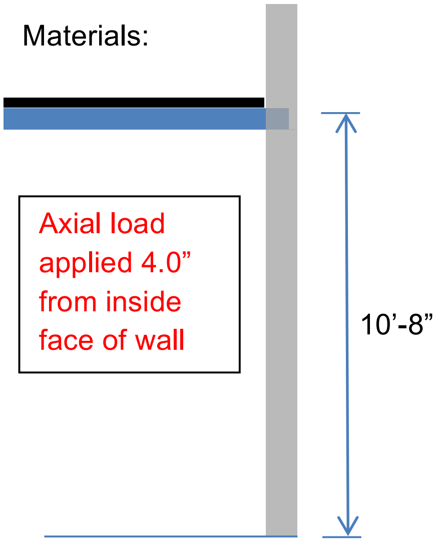
Please note that this design was for one load combination. Check all the necessary combinations using the same procedures. One method for doing this would be to create an interaction diagram for the wall design and plot the load combinations onto the diagram. The *2013 Masonry Designer's Guide* provides directions as to creating the diagram.

3. MULTI-STORY BUILDING



Construction:
5 Stories.
Roof and floors: 8 in. precast plank + 2” concrete topping spanning 28 ft.
Windows: 4’-8” high X 11’-4” wide
12” Hi-R H walls, fully grouted.

Materials:



12" Hi-R H, fully grouted, running bond
Type S mortar (P-C-L)
Grout 3,500 psi (f'_g); grout density= 125 pcf.
CMU Density = 120pcf;
wall weight = 94 psf (Table 5)
f'_m = 2,500 psi
E_m = 900 f'_m = 2.25 x10^6 psi TMS 4.2.2.2.1
f_y = 60,000 psi; E_s = 29 x 10^6 psi
n = E_s/E_m = 12.9
Assume simple supported vertical wall.

Loads on wall:

Roof dead load: 1,330 plf
Roof snow load: 560 plf
Floor dead load: 1,568 plf
Floor live load: 560 plf
Site lateral loads that would be determined from ASCE 7-10, assuming an SDC D site.

S_{DS} =0.65
ρ = 1.0
Wind: 35 psf
Seismic: F_p = 0.38w = 35.7psf

Loads on first floor pier (tributary width = 11'4" +2'-8" = 14'-0"):

Roof:	DL	1,330	SL	560
Floors 2-5:	DL	4 x 1,568 = 6,272	LL	4 x 560 = 2,240

Pier DL + interior finish: 99 psf x 4.5 levels x 10'-8" x 2.67' = 12,692 lb
Spandrels + interior finish: 11'-4" (6'-0 x 4 + 2'-8")(99) = 29,915 lb
Windows: 11'-4"(5'-4")(4)(6.5) = 1,570 lb

Axial load at mid-height of pier:

$Roof_{DL}$ = 1,330(14) = 18,620		$Roof_{SL}$ = 560(14) = 7,840 lb
$Floor_{DL}$ = 6272 (14) = 87,808		
Pier =	12,692	$Floor_{LL}$ = 2,240(14) = 31,360 lb (no LL reduction)
Spandrels	29,915	
Glass	1,570	
Total	150,605 lb	

Lateral load: E = 35.7 (2.67')(10.67)^2 /8 = 1,357 ft-lb per pier acting outward

Load Combination 2:

P_u = 1.2 D + 1.6L + 0.5 S = 1.2(150,605) + 1.6(31,360) + 0.5(7,840) = 234,822 lb
M_u = 1.2(1,568 x 14)(0.3 in./12)/2 = 329 ft-lb due to Floor 2 DL; e = y_c -bearing = 0.3 in.
1.6(560 x 14)(0.3 in./12)/2 = 157 ft-lb due to Floor 2 LL
486 ft-lb (compression on interior face)

Check: h/t = 128/11.625 = 11.0 < 30; allows P_u/A_g = 0.2 f'$_m$

P_u/A_g = 234,822/(32 x 11.625) = 631.2 psi requires f'_m = 631.2/0.2 = 3,156 psi < 3,500 **OK**

Load Combination 5:

P_u = 1.2 D + 1.0L + 1.0E = 1.2(150,605) + 1.0(31,360) + 1.0(0) = 212,086 lb

M_u = 1.2(1,568 x 14)(0.3 in./12)/2 = 329 ft-lb due to Floor 2 DL

1.0(560 x 14)(0.3 in./12)/2 = 98 ft-lb due to Floor 2 LL

1.0(1,357) = 1,357 ft-lb due to seismic at First Floor

1,784 ft-lb

Load Combination will be checked since P_u is less and M_u is higher than Load Combination 2.

Determine reinforcement: The high axial compression will not require much flexural reinforcement (if any) for out-of-plane loads. Try minimum reinforcement of #4 bars at jambs and add another at center of pier.

A_s = 3(0.2 in^2) = 0.6 in^2. Use the full b =32 in. for the pier.

Check h/r. $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1{,}226}{104}}$ = 3.43 Note: Values are per ft from Table 1. h/r = 128/3.43 = 37.3

$$P_n = 0.80\left[0.80f'_m\,(A_n - A_{st}) + f_yA_{st}\right][1-(\frac{h}{140r})^2$$

$$P_n = 0.80[0.80\,(2{,}500)\,(104(2.67) - 0.6) + 60{,}000(0.6)][1-(\tfrac{37.3}{140})^2 = 438{,}614 \text{ lb}$$

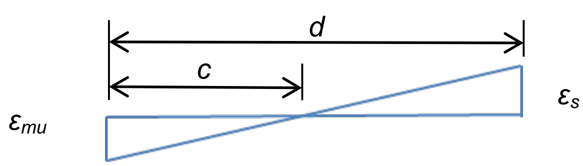
P_u (capacity) = ϕ P_n = 0.9 P_n = 394,753 lb >> P_u = 234,822 lb Load Combination 2 **OK**

Note: y_c = 4.9 in. > d = 4.3 in. from interior face,

$$a = \frac{A_sf_y+\frac{P_u}{\phi}}{0.8f'_mb} = \frac{(0.6)(60{,}000)+\frac{212{,}086}{0.9}}{0.8(2{,}500)32} = 4.24 \text{ in.}$$

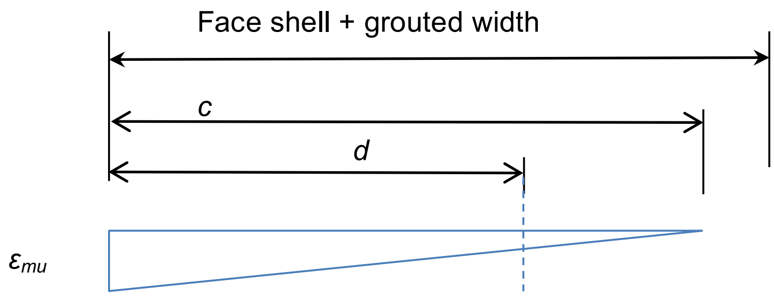
Note: this value exceeds the face shell thickness of the Hi-R, but it is still within the grouted area. However, the calculated "*a*" exceeds the "*a* maximum" value of 2.02 in. in Table 2 for fully-grouted walls. This means the steel does not yield and we have to determine the actual steel stress to get the moment capacity.

This changes to $a = \frac{A_sf_s+\frac{P_u}{\phi}}{0.8f'_mb}$ and $\varepsilon_s = \varepsilon_{mu}(d-c)/c$. Additionally, $f_s = E_s\,\varepsilon_s$ and a=0.8c.



Knowing A_s, P_u, ϕ, f'_m, b, d, ε_{mu} and E_s, solve the equations for c , a and f_s. However, the solution produces c ≥ d so that fs = 0 psi because the steel is in the compression zone.

The strain diagram changes to:



Note: c must not exceed the face shell plus grouted width = 7.35 in.

$a = \frac{\frac{P_u}{\phi}}{0.8f'_m b}$ and a=0.8c gives c = 4.60 in. (< 7.35 in.) and a = 3.68 in. Again, f_s =0

Using this information, determine the M_u (capacity) = $\phi\ [(A_S f_s)\left(d - \frac{a}{2}\right) + \frac{P_u}{\phi}(y_c - \frac{a}{2})]$

M_u (capacity)= $0.9\ [(0.6)(0)\left(4.3 - \frac{3.68}{2}\right) + (\frac{212{,}086}{0.9})(4.9 - \frac{3.68}{2})]/12$

M_u (capacity)= 60,091 ft-lb >> M_u = 1,784 ft-lb from Load Combination 5 **OK**

By observation, second order effects are not significant.
Therefore, pier reinforcement has 3 - #4 bars.

Two load combinations were evaluated, the engineer should check every loading combination.

4. STAIRWELLS

Schematic Plan – Building with exterior stairwells

On this project, there are elevator stairwells at each end of the building. During the schematic phase, the engineer must decide whether the stairwells are to be part of the lateral-force resisting system or not.

If the walls of the stairwells are included as part of the lateral-force resisting system, the building diaphragms must be connected to walls of each stairwell. In this condition, the connected wall of the stairwell is integral with the exterior wall of the building (Figure 16a) and can be used with the exterior building wall as a shear wall. That stairwell walls perpendicular to building are connected to the diaphragm also and can be sued as shear walls in that direction.

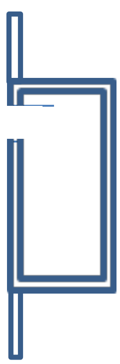


Figure 16a - Stairwell integral with building

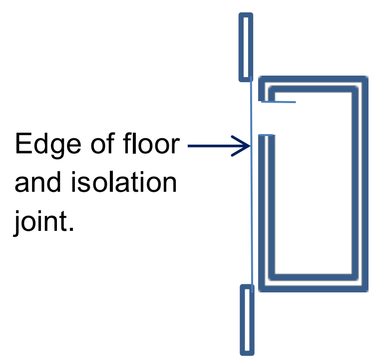
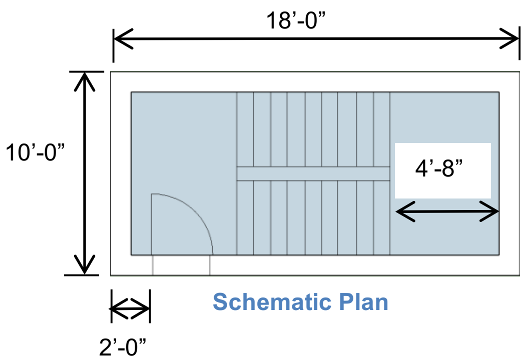


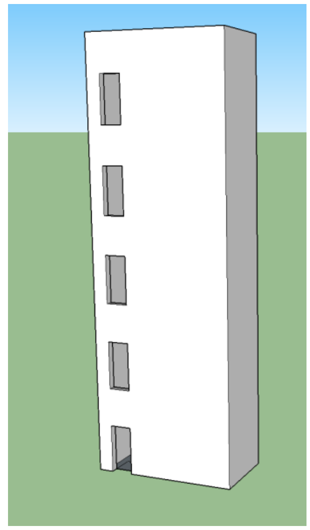
Figure 18b - Stairwell isolated from building

If the stairwells are isolated from the building through floor and wall expansion joints or seismic isolation joints (Figure 16b), the stairwell walls function independently of the building. The four walls function more as a tube-like structure to resist the lateral loads applied to stairwell alone.

For this example, the stairwell will be isolated from the building. The integral stairwell solution is similar but the loads will vary. To provide maximum load capacity, the stairwell will be fully grouted.



Schematic Plan



Elevation - Isolated from Building

Stairwell: 5 stories @ 12’-0” Floor-to-floor (total height 60’-0”)
3’-0” x 6’-8” doors at each level

Materials:

12” Hi-R, fully grouted, running bond
Type S mortar (P-C-L)
Grout 3,000 psi (f'_g); grout density = 120 pcf
CMU Density = 125 pcf;
wall weight = 95 psf (Table 4)
f'_m = 2,500 psi
E_m = 900 f'_m = 2.25 x10^6 psi TMS 4.2.2.2.1
f_y = 60,000 psi; E_s = 29 ksi
n = E_s/E_m = 12.9

Loads:

Roof dead load: 90 psf
Roof snow load: 45 psf
Stair dead load: 40 psf
Stair live load: 100 psf

Site lateral loads that would be determined from ASCE 7-10, assuming an SDC C site.

Wind: 35 psf
Components and cladding load = 53 psf
Seismic: F_p = 0.18w = 17.1psf

For this example, use the wind loading. For an actual project, check all pertinent load combinations.

U = 0.9D + 1.0W ((ASCE 7-10, Strength Load Combination 6)

Since the stairwell does not have a continuous diaphragm at each level, span the wall horizontally using bond beams at intervals.

Try bond beams at 4 ft on center vertically and assume a simple span between corners. Effective width of 12" wall bond beam = 48 inches < 6*t* per TMS 402, Section 5.1.2
Check the wind on the wall in suction since that section has the minimum d for reinforcement and the minimum section modulus for cracking. Use the Component and Cladding pressures.

w_u = (53 psf x 4 ft) = 212 plf using Component and Cladding loads.
M_u (required) = 1.0W = w_u $L^2/8$ = (212)(18^2)/8 = 8,586 ft-lb per bond beam
R_u = w_u L/2 = 1,908 lb

Flexural Reinforcement:
Estimate M_u (capacity) = $A_s f_y$ (d-a/2) try a = 2"; d = 4.6 inch (Table 2); f_y = 60 ksi

A_s = [M_u/ϕ]/ f_y (d-a/2) = [8,586/0.9]/60,000(4.6 – 2/2) = 0.44 in^2
<u>Try</u> #6 bar (A_s = 0.44in^2)

Calculate $a = \frac{A_s f_y}{0.8 f'_m b} = \frac{0.44(60{,}000)}{0.8(2{,}500)48}$) = 0.28 inches; <u>Note</u>: a < "a maximum" in Table 2 and also < $t_{face\ shell}$. **OK**

Calculate M_u (capacity) = ϕ $A_s f_y$ (d-a/2) = (0.9)0.44(60,000)(4.6 - 0.28/2)/12
M_u (capacity) = 8,831 ft-lb > M_u (required) = 8,586 ft-lb **OK**

Check M_n > 1.3M_{cr} TMS 9.3.4.2.2.2

M_n = M_u/ϕ = 8,831/0.9 = 9,812 ft-lb

M_{cr} = S x f_r S = 190 in^3 (Table 1) ; f_r = 167 psi (TMS Table 9.1.9.2 using parallel to bed joints and ungrouted value)
M_{cr} = 190(167)/12 = 2,644 ft –lb
M_n = 9,812 ft-lb > 1.3 M_{cr} = 3,437 ft-lb **OK**

Check shear, V_u < ϕ V_n ϕ = 0.8 TMS 9.1.4.5 (Shear)

V_n = (V_{nm} + V_{ns}) γ TMS Eq. 9-21

V_{ns} = 0 ; γ = 0.75 assumes the wall is partially grouted due to the insulation inserts.

$V_{nm} = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25\,P_u$ TMS Eq. 9-24

$\frac{M_u}{V_u d_v} = 0;\ P_u = 0$

This gives $V_{nm} = 4.0 A_{nv}\sqrt{f'_m}$ = 4.0 (4ft x 110 in^2/ft from Table 1)($\sqrt{f'_m}$)

V_{nm} = 4(440)($\sqrt{2{,}500}$) = 88,000 lb which is less than the maximum allowed per TMS Eq. 9-22. **OK**

V_u (capacity) = ϕ (88,000) γ = 52,800 lb >> R_u = 1,908 lb. **OK**, no need to go to the effort of determining the reduced V_u at d/2 from the support.

Therefore, Use #6 in bond beams at 4 ft oc over the height of the stairwell. The reinforcement is in the center of the grouted area of the unit (Figure 17). This bond beam reinforcement may be used in accordance with NCMA TEK 10-3 to accommodate crack control so as to avoid any movement joints in the stairwell. No horizontal joint reinforcement is needed either.

<u>Note</u>: Another design option would have been to try horizontal joint reinforcement as the structural reinforcement. Redoing the calculations with d = 10.5 in. (from compression face to the centerline of the tension face shell) and horizontal joint reinforcement f_y = 70,000 psi would require a wire approximately 1/4 in. diameter in every bed joint. This is impractical so that bond beams are the preferred option for the exterior walls.

Now design the vertical reinforcement for the overall structure. This reinforcement will be offset from centerline of the grouted area because the horizontally spanning reinforcement is more important. However, this decision will affect the required lap splice lengths of the vertical reinforcement. See the Section below.

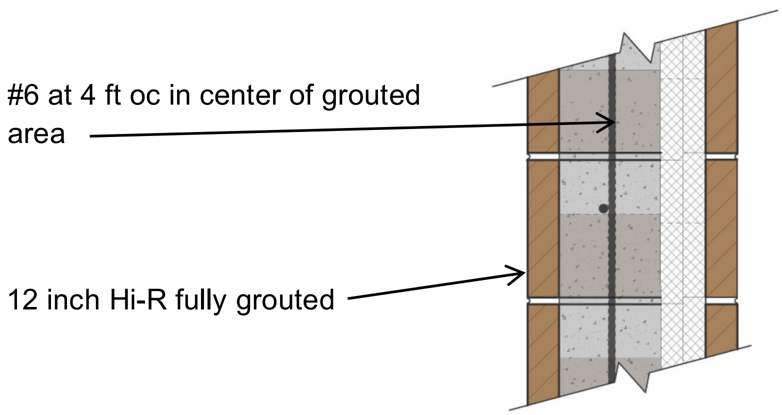


Figure 17 Bond beam section

The exterior sidewalls have a shorter span (10'-0) and could be designed with less steel reinforcement or even with a greater spacing. However for consistency, use the same design on all four sides of the stairwell.

There is one additional condition that affects this design. The door openings interrupt the bond beams on the interior elevation. However, there is no external wind pressure on that wall. Check the load on the bond beam above the door such that the wall spans vertically 8 ft. The tributary area for the load is approximately 6 ft (Figure 18).

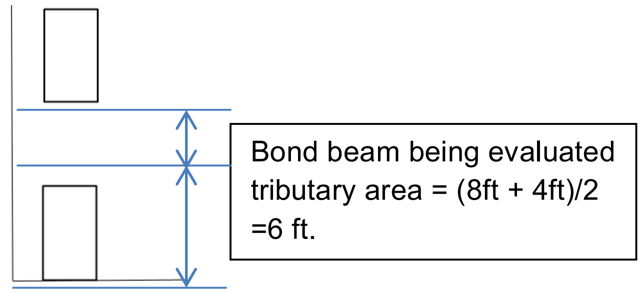


Figure 18 – Interior bond beam

Because this wall in interior, the design of the interior bond beams load will be governed primarily by the seismic loading rather than wind.
Therefore, M_u = 1.0E where E = F_p $L^2/8$ = (20.7 psf)($\frac{6\,ft\,trib}{2}$)(18^2)/8 = 5,030 plf .
Note: This M_u < M_u for exterior (8,663 ft-lb). So, the same design works.

Next, evaluate the flexural and shear effects on the entire stairwell using the winds loads, not the Components and Cladding loads.

For the entire stairwell, apply the wind to the 18 ft side (Figure 19). Use the overall wind pressure.

The stairwell functions as a cantilever:

$M_{Wu\ \mathrm{base}} = w_u\,(h^2)/2$ = [35psf](18 ft) ($60^2/2$) = 630 plf (1,800)/1,000 = 1,134 ft-kips

$V_{Wu\ \mathrm{base}} = w_u\,h$ = 630 (60)/1,000 = 37.8 kips

P_u = 0.9D

Walls: 95 psf (18+18+8+8)(60)/1000 = 296.4 kips (ignore door openings)

Roof: 90 (18)10 /1,000 = 16.2 kips
Landings floors: 40 psf (4.67 x 8)(10)/1000 = 14.9 kips
Stairs: 40(6.67 x 3.67)(2)(10)/1,000 = 19.6 kips

$P_u = (0.9)P_{DL}$ = (0.9)[296.4 + 50.7] = 312.4 kips

$M_{overturning} = M_{Wu}$ = 1,134 ft-kips
$M_{restraining} = P_u$ (10/2) = 1,562 ft-kips > $M_{overturning}$ **OK**

Check whether full stairwell can function as a tube by evaluating the effective flange width attributed to the intersecting walls. The wind direction was selected to check the minimum wall area adjacent to the doors for compression.

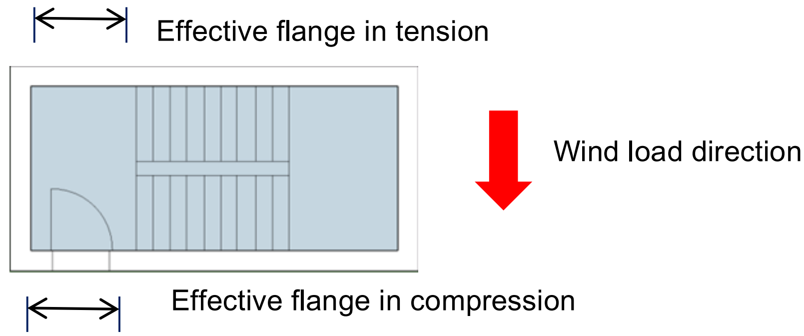


Figure 19 – Effective width

Flange width in compression: $6t$ = 6'-0" TMS 5.1.1.2.3 (a)
Width a 3'-0" door opening in the corner, the effective flange = 3'-0". Provide a lintel over the door opening to transfer the high compressive stresses.

Flange width in tension: $0.75h_{floor}$ TMS 5.1.1.2.3 (c)
Since there is a landing floor at 12'-0, flange = 9'-0". A very conservative approach would be to use the bond beam spacing = 4'-0". Use 9.0 ft for this example.

These results indicate that the entire length stairwell is not effective as the reinforced section for compression but that the entire length is effective for tension (Figure 20).

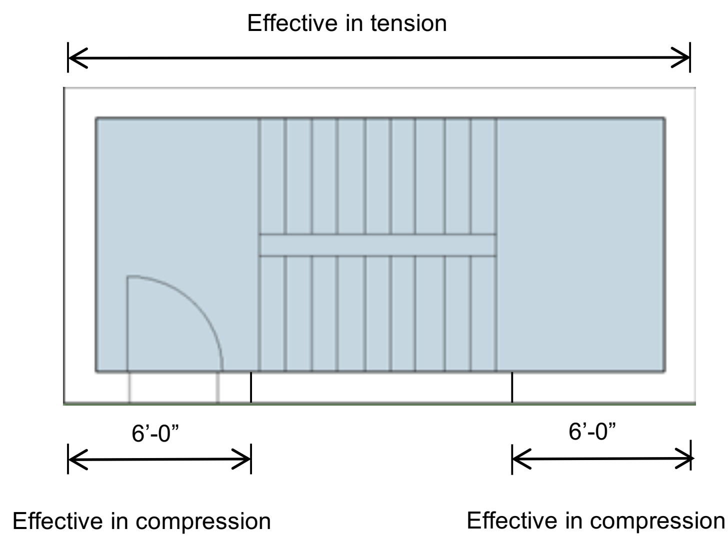


Figure 20 – Effective area for reinforcement

Determine the center of gravity and section modulus for the effective section (use ½ the section near the door) shown in Figure 21. The areas, centroids and moments of inertia of the individual sections are obtained from Table 1. For Section 1, ignore the area above the door.

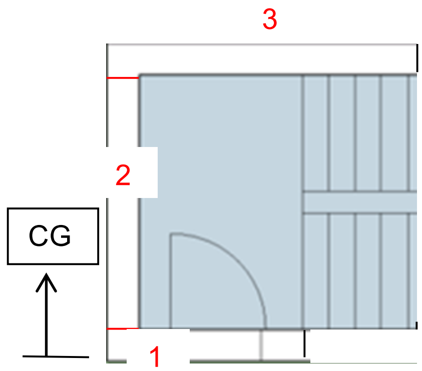


Figure 21 – Section Properties

Section	Area (in^2)	Distance to centroid of each section (y_c) (in.)	Ay_c (in^3)	Moment of inertia I_o (in^4)	$\lvert(y_c\text{-CG})\rvert$ (in.)	$A\lvert(y_c\text{-CG})\rvert^2$ (in^4)
1	3'(110)=330	(11.625-5.0)	2,186	3'(1394)= 4,182	69.2	1,580,251
2	8'(110)=880	60	52,800	8(1394)=11,152	15.8	219,683
3	9'(110)=990	(108+5)	111,870	9'(1394)=12,546	37.2	1,370,002
	Σ = 2,200		Σ =166,856	Σ =27,880		Σ =3,169,936
$CG = \Sigma\, Ay_c / \Sigma A$ = 166,856/2200 = 75.8 in.				$I = \Sigma\, I_0 + \Sigma\, A\,\lvert(y_c\text{-}CG)\rvert^2$ = 3,197,816 in^4		
				$S_{compression} = I/CG$ = 42,188in^3 $S_{tension} = I/(120\text{-}CG)$ = 72,349in^3		

Check stresses using ASD and ASCE 7, Load Combination 7: 0.6D + 0.6W

Use one-half the loads since we are checking half of the section.

$P = 0.6\,DL$ = (0.6)(296.4 + 50.7)(1/2) = (0.6)(347.1)(0.2) = 104.2 kips

$M = 0.6W$ = (0.6)(1,134)((0.5) = 340.2 ft-kips

Combining stresses: $f = P/A$ +/- M/S

Compression f = 1000[104.2/2200 + 340.2(12)/42,188] = 47.4 + 96.8 = 144.2 psi

Tension f = 47.4 – 1000[340.2(12)/72,349 = 47.4 – 56.4 = 9.3 psi

Conservatively using TMS Table 8.2.4.2, Type S mortar allows a tension stress of 33 psi for ungrouted, hollow units. This is not being used for design but only to indicate that the wall is very lowly stressed. By these calculations, the wall will not crack. However, we will design reinforcement using SD as though the wall did crack.

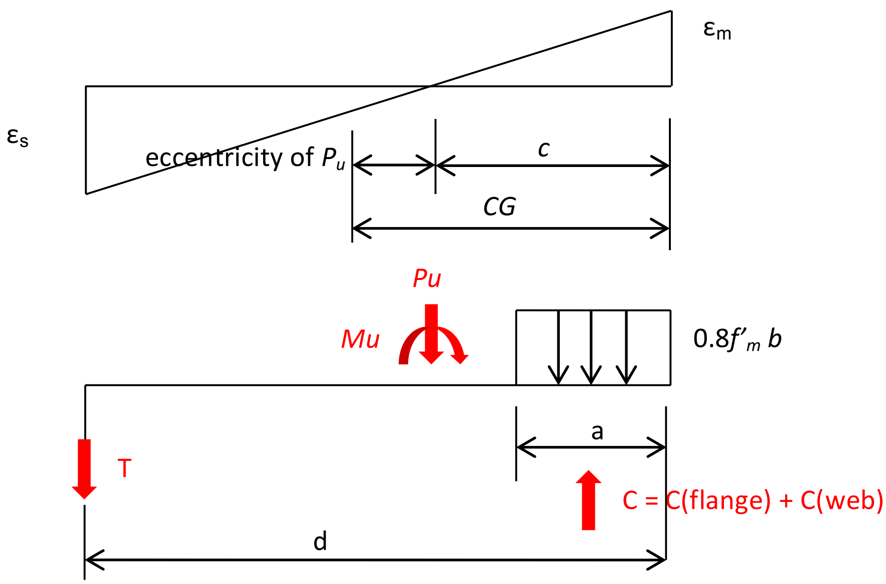
Now design the reinforcement using Strength Design (Strength Load Combination 7):

P_u = 312.4 kips/2 = 156.2 kips (dead load)

M_u = M_{Wu} + P_u (eccentricity). The eccentricity is a result of the unsymmetrical section (Figure 22). The eccentricity that will be used here is based upon a cracked section. Figure 22 shows a partial plan on the stairwell that is rotated from Figure 19. Reinforcement will be placed in the flanges of the walls.

Figure 22 - Partial Plan

Looking at the strain and stress diagrams assuming the masonry reaches yield:



ε_m = 0.0025 TMS 9.3.2 (c)

$\varepsilon_m/c = \varepsilon_s/(d-c)$ TMS 9.3.2 (d)

$\varepsilon_s < f_y/E_s$ TMS 9.3.2 (e)

$a = 0.80c$ TMS 9.3.2

$T = A_s f_y$

$C = (0.8f'_m\, b)\, a$ TMS 9.3.2 Note: You need to adjust for flange and partial web in compression.

Due to the flange, need to distribute between flange and web when $a > t_{flange}$.

$M_u = M_{wu} + P_u$ (eccentricity) = 1,134/2 + 156.2 (eccentricity) = 567 +156.2(ecc)

d = 120in. – approximately 5 in. to the vertical reinforcement

From statics: P_u + T = C

M_u = C(flange)(c-centroid of flange) + C(web)[c-t_{flange} – (a-t_{flange})/2] + T (d-c)

<u>Note</u>: [c-t_{flange} – (a-t_{flange})/2] = c -t_{flange} – (0.80c - t_{flange})/2 = 0.6c - t_{flange}/2

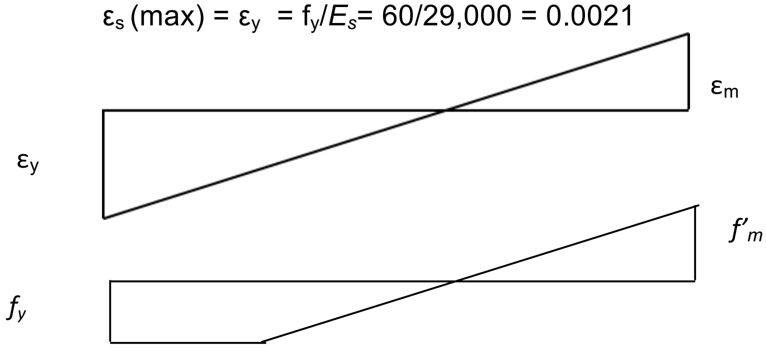
See Figure 23.

Using trial and error and iteration to solve for A_s and a:

1st Try c = 40 in.

a = 32 in. (a/d = 32/115 = 0.28 < 0.44. Therefore reinforcement yielding is expected).

ε_s = 0.0025(115-40)/40 = 0.0047
ε_s (max) = ε_y = f_y/E_s= 60/29,000 = 0.0021



C = C(flange) + C(web)
See Figure 23.
C (flange) = [(0.8(2,500)] [(3ft)(110in^2/ft)/1000 = 660.0 kips
C (web) = [(0.8(2,500)](110)(32-11.625)/12]/1000 = 373.6 kips
C = C(flange) + C(web) = 660.0 + 373.6 = 1,034 kips

M_u = M_{Wu} + P_u (eccentricity) = 567 ft-kips + 156.2 kips (CG-c)
M_u = 566 + (156.2)(75.8-40)/12 = 567 + 466 = 1,033 ft-kips

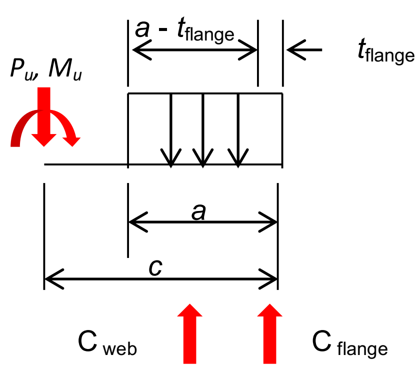


Figure 23 - Compression zone that includes flange and part of web

M_u = C (flange)(c – 6.625in…centroid of grouted area)/12
+ C(web)(0.6c-t_{flange}/2)12
+ T (115)/12

M_u = 660.0(33.4)/12 + 373.6(18.19)/12 +T (9.58)
1,033 = 1,837 + 566 + 9.58T
T = [1,033-1,837-566]/9.58 = -143 kips **NG** (can't be negative)

2nd Try c = 20

a = 16
ε_s = 0.0025(115-20)/20 = 0.0119
ε_s (max) = f_y/E_s= 60/29,000 = 0.0021 < ε_s Use maximum

C = C(flange) + C(web)
C (flange) = 660.0 kips
C (web) = [(0.8(2,500)](110)(16-11.625)/12]/1000 = 80.2 kips
C = C(flange) + C(web) = 660.0 + 80.2 = 740.2 kips

M_u = M_{Wu} + P_u (eccentricity) = 567 ft-kips + 156.2 kips (CG-c)
M_u = 566 + (156.2)(75.8-20)/12 = 567 + 726 = 1,293 ft-kips

M_u = C (flange)(c – 6.625in.)/12
+ C(web)(0.6c-t_{flange}/2)12
+ T (115)/12

M_u = 660.0(13.4)/12 + 80.2(6.12)/12 +T (9.58)
1,293 = 737 + 41+ 9.58T
T = [1,293-737-41]/9.58 = 54 kips **OK**

Since $\varepsilon_s > \varepsilon_s$ (max), use $f_s = f_y$
Therefore, T = $A_s f_y$ = 54 kips or A_s = T/ f_y = 0.90 in^2

Try: 4-#5 @ approximately 32in. such that for 9 ft. flange A_s = 1.24 in^2 **OK**

The engineer can design the reinforcement with the wind reversed as well as in the perpendicular direction.

Next, check the shear wall. For this example, use the same reinforcement (#5 @ 32 in. on center and check for shear capacity in the plane of the 10 ft walls).

V_u = 37.8 kips/2 for each side wall = 18.9 kips

Using only the wall as the web for shear:

$V_u < \phi\ V_n = 0.8\ V_n$ ϕ=0.8 for shear

$$V_n = (V_{nm} + V_{ns})\gamma_g$$ TMS Eq, 9-21

Where $M_u/V_u d_v \le 0.25$, $V_n \le 6A_{nv}\sqrt{f'_m}\,\gamma_g$ TMS Eq.9-22

Where $M_u/V_u d_v \ge 1.0$, $V_n \le 4A_{nv}\sqrt{f'_m}\,\gamma_g$ TMS Eq.9-23

γ_g = 0.75 for partially grouted shear walls

From TMS 402, the maximum value of V_n for $M_u/V_u d_v$ between 0.25 and 1.0 shall be permitted to be linearly interpolated.

For this example, consider one shear wall using the same loading condition

U = .9D + 1.0W

$M_u/V_u d_v$ = 566 ft-kips/[(37.8/2)(110 in./12)] = 3.3 <u>use 1.0</u> TMS 9.3.4.1.2.1

Therefore, V_n (max) = $4A_{nv}\sqrt{f'_m}\,\gamma_g$

V_n (max) = 4 (10ft x 110in2/ft)($\sqrt{2500}$(0.75)/1000 = 165.0 kips >> V_u **OK**

$$V_{nm} = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_{nv}\sqrt{f'_m} + 0.25\,P_u$$ TMS Eq. 9-24

$V_{nm} = [4.0 - 1.75(1.0)](10\ x\ 110)\sqrt{2{,}500} + 0.25(0)$ $V_{nm} = [4.0 - 1.75](10\ x\ 110)\sqrt{2{,}500}$ /1000 = 124.1 kips

If V_{ns} = 0, V_u (capacity) = 0.8 V_{nm} γ_g = 74.4 kips > V_u (required) = 18.9 kips **OK**

For this example, no shear reinforcement is required. However, the horizontal reinforcement in the bond beams could have been used to increase the shear capacity with no additional expense.

Using TMS Eq. 9-25, $V_{ns} = 0.5(\frac{A_v}{s})\,f_y d_v$ we can determine the increased shear capacity produced by of the bond beams.

$V_{ns} = 0.5(\frac{0.44}{48})\,60(110)$ = 30.3 kips

This gives an increased shear capacity = ϕ V_{ns} γ_g = 18.2 kips

Therefore, the total shear capacity of the wall = 78.6 + 18.2 = 96.8 kips

Finally, determine the loading on the reinforced lintel over the doorways. We will only look at the loadings here (Figure 24). The design of lintels is addressed in the next example.

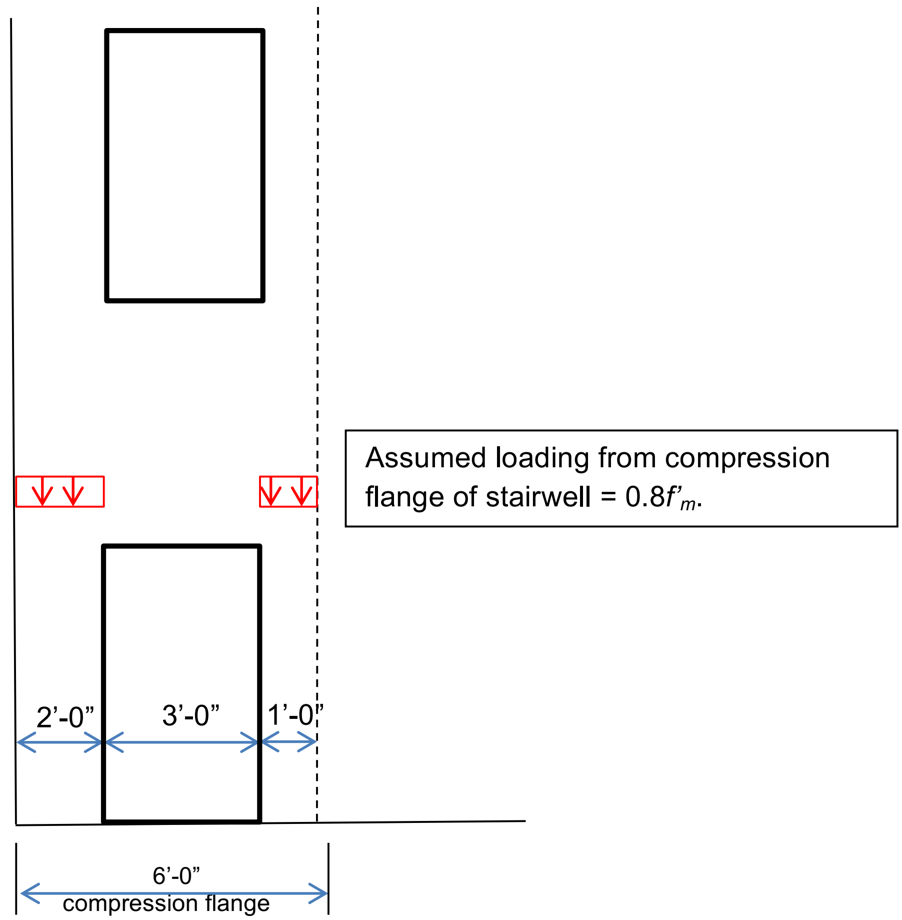


Figure 24 – Loadings at doors

The stress on right hand side of the opening has to traverse the opening.

Conservatively, an engineer could provide the lintel capacity of the lowest opening to resist $0.8f'_m$ (110in^2/ft x 1 ft) = 220,000 lb. Note: the load reduces on the upper lintels.

Distributed over 3 ft + (4"brg x2) = 3.67 ft; w_u = 59.9 kip/ft. See Figure 25.

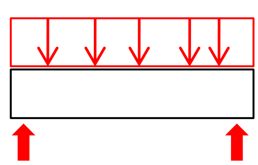


Figure 25 – Possible lintel load

This is an extremely high load for a masonry lintel. If the calculations are performed, this fully-grouted wall has the flexural capacity, but not the shear capacity. Under this loading condition, a steel lintel would be required.

By acknowledging the arching action of the wall over the opening (Figure 26), the loading can be reduced to the triangular weight of the wall over the opening. This loading can be easily accomplished by the masonry lintel (see later examples for lintel design).

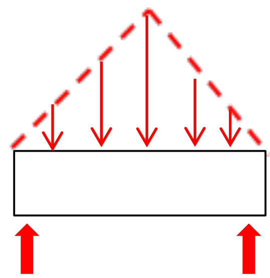


Figure 26 –Arching action over lintel

The results of the design are shown schematically in Figure 27 with the reinforcement placement. Lap splices are not shown.

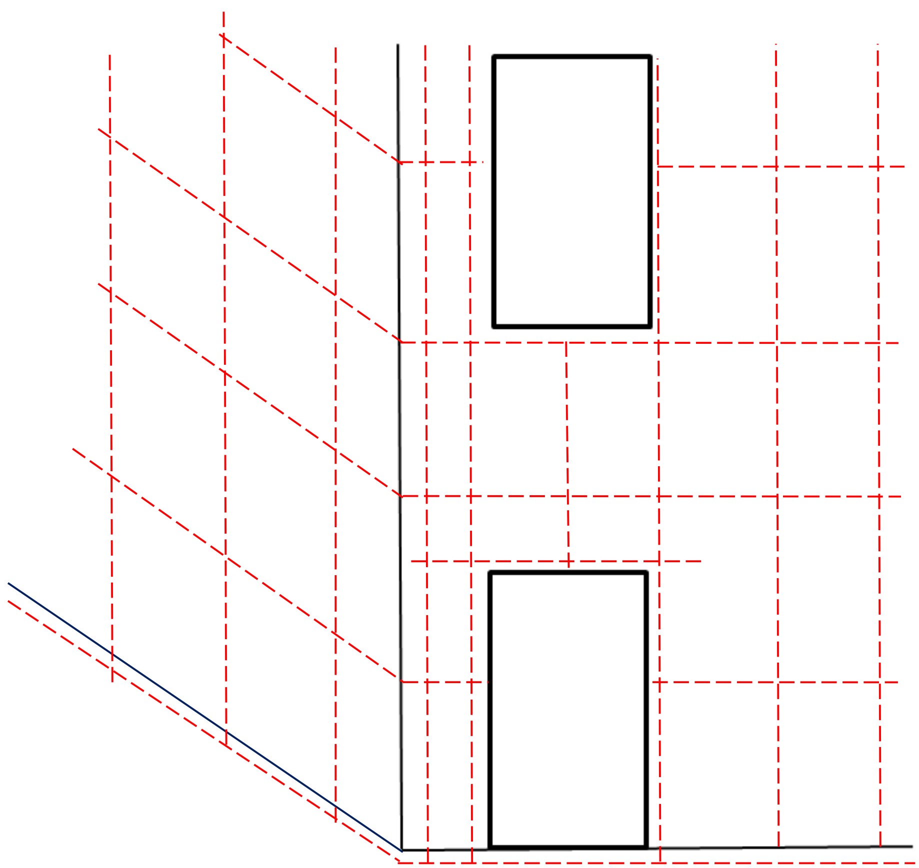


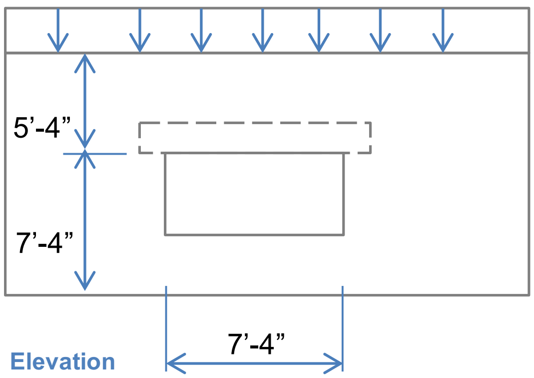
Figure 27 – Partial Elevation with Reinforcement layout

The methods in this example illustrate the use of the design methods at the base of the stairwell. The engineer should re-evaluate the design at various levels and reduce the reinforcement, if desired. The same bond beams were used on all sides. However, the interior wall could be redesigned for the lower loads and the sidewalls could be redesigned for the shorter spans. This would reduce the size of the reinforcement but the spacing of the bond beam should be maintained.

Please note that this stairwell design was primarily for one load combination. Check all the necessary combinations using the same procedures. In addition, the design should evaluate the foundation for overturning and bearing stresses.

5. BEAMS AND LINTELS

Using the standard CPG CAD Details from the Design Resource Center, design a masonry beam using 12" Hi-RH.



Elevation

Materials:

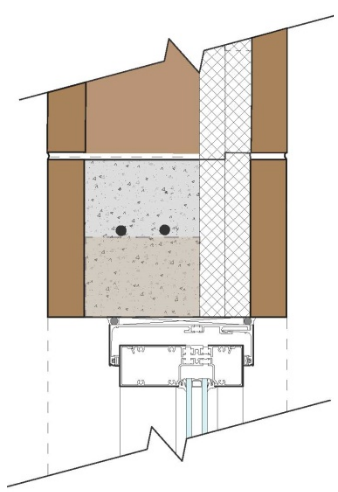
12" Hi-R, fully grouted over the beam; running bond
Type S mortar (P-C-L)
Grout 3,000 psi (f'_g); density=125 pcf
CMU Density = 135pcf;
wall weight = 99 psf (Table 5)
f'_m = 2,500 psi
$E_m = 900\ f'_m = 2.25 \times 10^6$ psi
f_y = 60,000 psi; E_s = 29 ksi
$n = E_s/E_m = 12.9$

Applied Loads at top of wall:

Dead load: 800 plf
Live load: 400 plf

Site lateral loads that would be determined from ASCE 7-10, assuming an SDC B site.

Wind: 35 psf
Seismic: $F_p = 0.12w = 11.9$ psf



Typical Section from CPG Details

For this example, we'll check one load combination. For an actual project, check all pertinent load combinations.

$U = 1.2D + 1.6L$ (ASCE 7-10, Load Combination 2)

$\Phi = 0.6$ for shear; $\Phi = 0.9$ for flexure

$w_u = 1.2\,[800 + 5.33(99)] + 1.6\,[400] = 1{,}593 + 640 = 2{,}233$ lb/ft

Span = opening + 4" bearing at each end = 8'-0"

$R_u = w_u L/2 = 2{,}233 \times 8/2 = 8{,}932$ lb = reaction

$M_u = w_u L^2 / 8 = 17{,}864$ ft-lb

$V_n = 56\,A_{nv}$ TMS 9.2.6 (c). This assumes Strength Design but unreinforced for shear. The "not fully grouted" value is used to be conservative due to the presence of insulation inserts.

Most structural designs are usually developed for flexural capacity and then checked to determine whether shear reinforcement is required. For masonry beams and lintels, it is recommended to first determine whether the beam or lintel can be sized without shear reinforcement. If the depth to accommodate the full shear required is reasonable, the flexural reinforcement capacity can then be designed for that depth.

If the shear depth required is not possible or otherwise unreasonable, shear reinforcement can be added.

The use of fully grouted walls makes this a reasonable approach. Using this methodology, the following example will be evaluated for necessary shear depth.

For first approximation, try $V_u = R_u$

$V_u < \phi\, V_n$ gives $8{,}908 = (0.6)\,56\,A_{nv}$ or $A_{nv} = 266$ in^2

For the given section, $b = 1.75 + 4.9 = 6.65$ inches (interior face shell + grout)

Height of beam required = $A_{nv}/6.65 = 40$ inches **OK** 5'-4" available

Use $d = 40$ inches + 1.5 inches in lowest course = 41.5 inches

1st trial: assume $a = 4$ inches

$A_s = (M_u /0.63)/\,[f_y\,(d\text{-}a/2)] = (17{,}864/0.63) \times 12/\,(60{,}000(41.5 - 4/2) = 0.14$ in^2

<u>Note</u>: $M_u = 0.63\,A_s\,f_y\,(d\text{-}a/2)$ as noted in the Design Methodology section.

Try 1- #5 $A_s = 0.31$ in^2

Calculate $a = A_s\,f_y/\,0.8f'_m b = 0.31(60\text{ ksi})/(0.8\,(2.5\text{ksi})(6.65\text{ in.}) = 1.40$ inches

Recheck M_u (capacity) = $0.63M_n = 0.63\ [A_s\ f_y\ (d\text{-}a/2)] = 0.63\ [0.31(60{,}000)(41.5 – 1.40/2)]\ /12$

M_u (capacity) = 39,841 ft-lb >> M_u (required) = 17,864 ft-lb

Because the wall is fully grouted, the section depth available to function as a beam is very large. The reinforcement is minimal for a very large capacity.

The TMS code also requires $M_n > 1.3M_{cr}$ TMS 9.3.4.2.2.2

$S = 6.65(41.5)^2/6 = 1{,}909\ in^3$

f_r = 167 psi TMS Table 9.1.9.2 for Type S P-C-L mortar, partially grouted

$M_{cr} = S \times f_r = 1909 \times 167/12 = 26{,}567$ ft-lb.

$M_n = M_u/0.63 = 39{,}864/0.63 = 63{,}276$ ft-lb > $1.3M_{cr} = 1.3(26{,}567) = 34{,}537$ ft-lb **OK**

Note: It would be possible to use a smaller depth by checking V_u at d/2 away from bearing and also adding shear reinforcement. For this example, that was not necessary since the wall is fully grouted. There is no increase in cost for using the deeper beam.

While the design only requires a #5, consider using a #6 to match the bond beams in the walls (Figure 28).

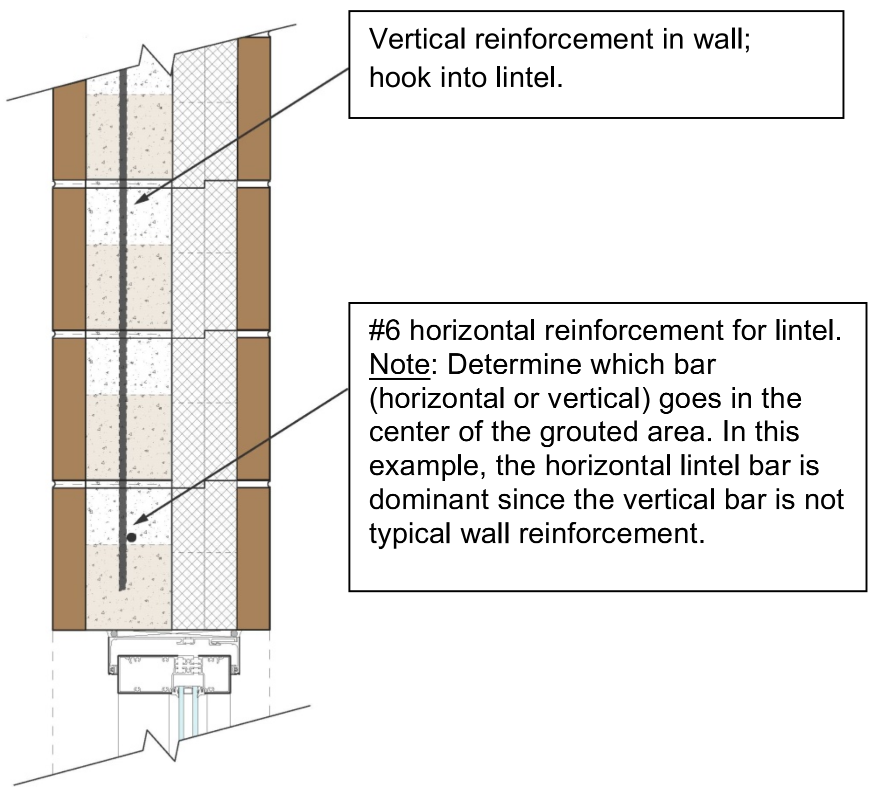
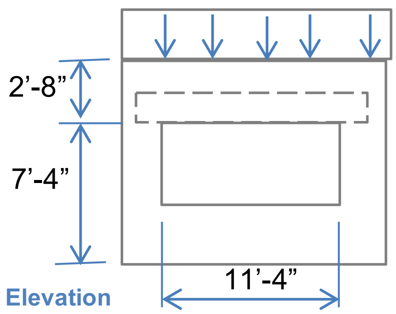


Figure 28 - Final lintel section

For our next example, design the masonry beam from the Multi-Story Example.



Materials:

12" Hi-R H, fully grouted, running bond
Type S mortar (P-C-L)
Grout 3,500 psi (f'_g); density = 125 pcf
CMU Density = 120pcf;
wall weight = 94 psf (Table 5)
f'_m = 2,500 psi
$E_m = 900\, f'_m = 2.25 \times 10^6$ psi
f_y = 60,000 psi; E_s = 29 ksi
$n = E_s/E_m = 12.9$

Loads:

Dead load: 2,132 plf (Floor DL and wall above)
Live load: 560 plf (Floor LL)

1. Loading

w_u = 1.2DL + 1.6LL = 1.2(2,132) + 1.6(560) = 3,454 plf

$R_u = w_u$ L/2 where L = 11.33 + 2(0.67)/2 brg = 12.0

R_u = 3,454 (6.0) = 20,724 lb.

If d = 32" – 6.5" = 25.5 in. (2.13 ft), determine V_u @ $d/2$ from face of opening.

$M_u = w_u L^2/8 = 3{,}454(12)^2/8$ = 62,172 ft-lb at mid span.

2. Shear

$V_u = R_u - w_u(0.33 + 25.5/(2\text{x}12))$ = 20,724 – 3,454(1.39) = 15,923 lb.

$M_u = R_u\,(1.39\text{ft}) - w_u\,(1.39)^2/2$ = 22,133-3,337 = 18,796 ft-lb @ $d/2$ from support

$M_u/V_u d_v$ = 18,796/(15,923)(2.13) = 0.55

Determine:

$$Maximum\ V_n \le 4.8 A_{nv} \sqrt{f'_m}\ \gamma_g = 4.8\left(\frac{25.5}{12}\right)\left(\frac{104 in^2}{ft}\right)\sqrt{2{,}500}(0.75) = 39{,}780\text{ lb.}$$

The 4.8 term is derived by interpolating for $\frac{M_u}{V_u d_v}$ = 0.55 TMS 9.3.4.1.2 (c)

$V_{nm} = [4.0 - 1.75(\frac{M_u}{V_u d_v})]\, A_{nv}\sqrt{f'_m}$

$V_{nm} = [4.0 - 1.75(0.55)]\,(104\, x2.13)\sqrt{2{,}500} = 33{,}560$ lb. **OK,** less than *Maximum* V_n

Therefore, V_u (capacity) = ϕ V_{nm} = 0.8 (33,560) = 26,848 lb >> Vu = 15,923 lb **OK**, only minimum transverse reinforcement required.

3. Determine flexural reinforcement:

$A_s = (M_u /0.63)/ [f_y (d-a/2)]$ try a = 3 in.

A_s = (62,172 x 12 /0.63)/ [60,000 (25.5 - 4/2)] = 0.84 in^2 Try 2-#6 (A_s = 0.88 in^2)

Calculate $a = A_s f_y / 0.8f'_m b$ = (0.88)(60,000)/(0.8)(2,500)(6.88) = 3.84 in.

Note: a/d = 0.15 < 0.44. Therefore, steel reinforcement yielding is expected.

M_u (capacity) = 0.63 [$A_s f_y$ (d-a/2)] = 0.63[(0.88)(60,000)(25.5 – 3.84/2)/12]

M_u (capacity) = 65,364 ft-lb > M_u (required)= 62,172 ft-lb **OK**

4. Check $M_n > 1.3M_{cr}$:

S = 6.88(25.5)^2/6 = 746 in^3

f_r = 167 psi TMS Table 9.1.9.2 for Type S P-C-L mortar, partially grouted

M_{cr} = S x f_r = 746 x 167/12 = 10,381 ft-lb

$M_n = M_u/0.63$ = 65,364/0.63 = 103,752 ft-lb >> 1.3M_{cr} = 1.3(10,381) = 13,495 ft-lb **OK**

5. Service load deflections:

TMS 5.2.1.4.3 does not require that deflections be checked if the span < 8d.

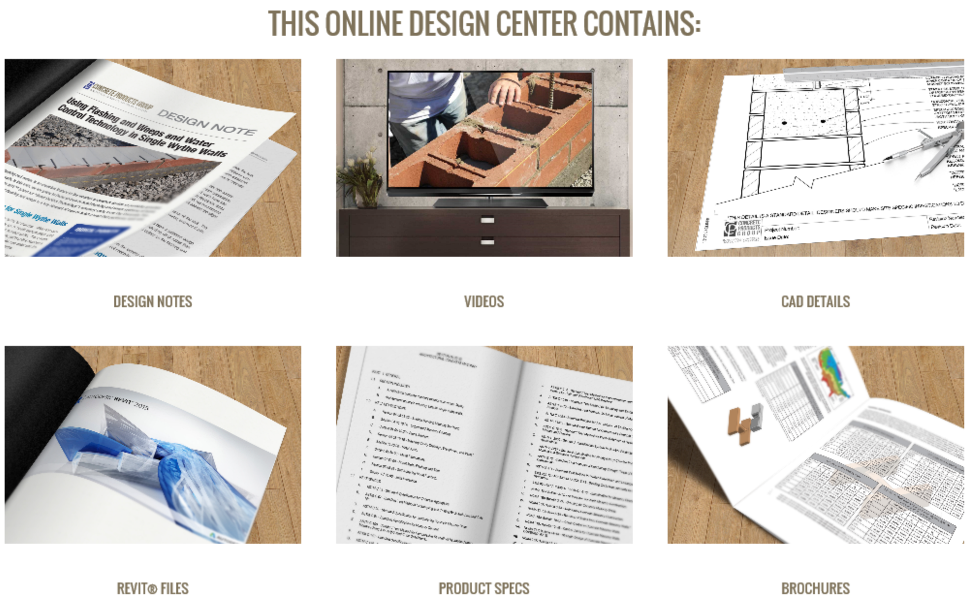
For this beam, L=12’-0” and 8d = 8(2.13) = 17’-0”. Therefore, no check is required.

Therefore, 2-#6 with no shear reinforcement is acceptable for the beam. The top of the beam is the bond beam under the plank bearing. Provide 2-#6 there as well unless the chord design for the floor diaphragm requires more.

Design Resources

Concrete Products Group provides a series of resources for the engineer designer to assist with the development and assembly of design documents. The resources include:

1. Design Resource Center
   This center is available on-line at the CPG web site (http://concreteproductsgroup.com/) after the user creates a free account and creates a login. The site contains technical information for designing and detailing Hi-R and HI-R H systems.



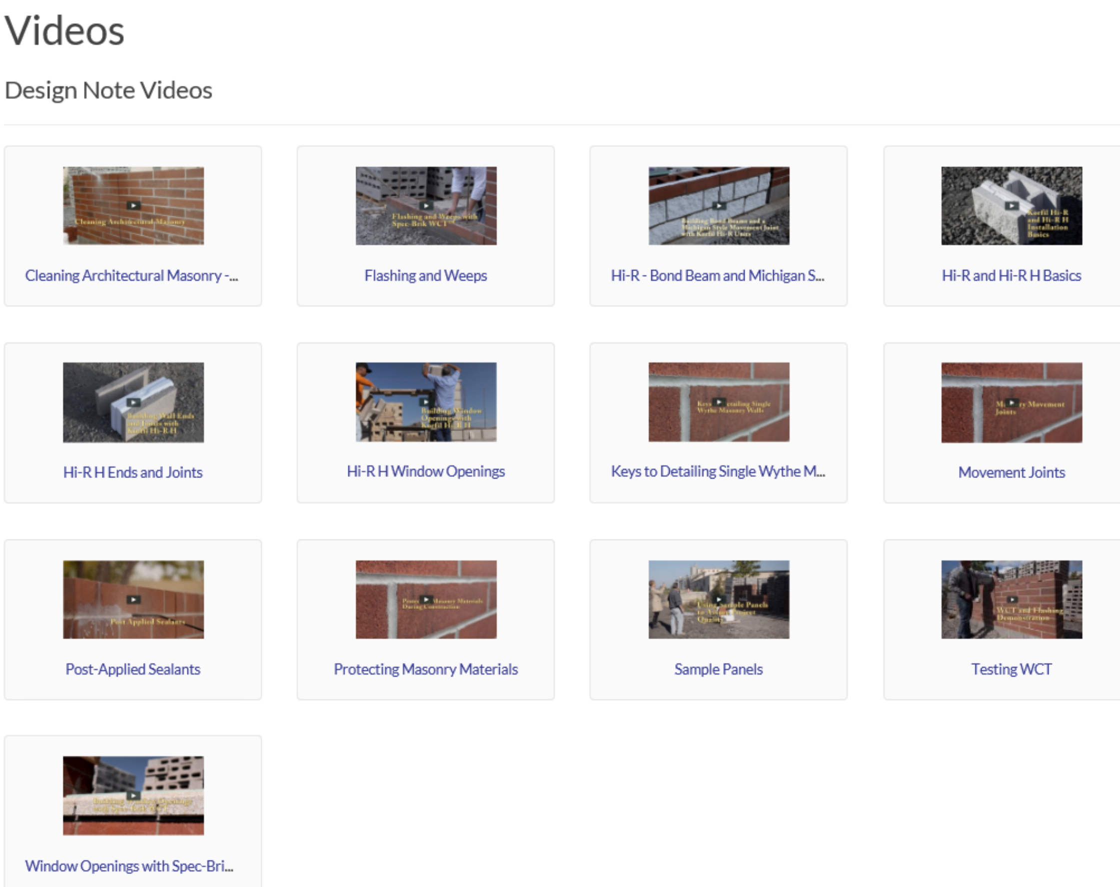
2. Masonry Designer:
   CPG Masonry Designer Software is an on-line design tool (http://www.concreteproductsgroup.com/index.php/innovation/masonry-designer) that is versatile and user-friendly to render wall sections with the perfect combination of block color, texture and mortar color for your project. It features CPG's 12 standard colors for both Spec-Brik units and Spec-Split architectural split face units along with our grey precision Spec-Block CMU.

   The tool allows the designer to render wall sections using each color. You can go to the limits of your creativity by combining block heights, textures and colors in running bond, stack bond or quarter bond patterns with the mortar color of your choice. CPG also provides an instructional video for Masonry Designer.

3. Revit PlugIns:
   Working along with the Masonry Designer, CPG also provides an Autodesk Revit plugin (http://www.concreteproductsgroup.com/index.php/resources/revit-plug-ins). A video tutorial is included to assist the designer upload and use the free download.

4. Detailing Manual
   CPG offers numerous standard details that are project ready. These include a free 70-page Single Wythe Detailing Manual (http://www.concreteproductsgroup.com/index.php/resources/detailing-manual) based upon the popular Spec Brik. Several graphics in this manual are taken from the Detailing Manual.

5. Videos
   The CPG web site also provides a number of instructional videos of use to designers and contractors.



6. Besides CPG technical data, users have access to the CBIS (Concrete Block Insulating System) library (www.cbisinc.com/library.html). Included in that library is the technical report cited regarding 1986 structural testing if Hi-R masonry elements (www.cbisinc.com/PDF/HI-R/310_HI-R_NCMA_Structural_Engineering_Report.pdf).

Contract Documents

Specifications

CPG offers designers a sample specification titled "*SECTION 04 22 23 ARCHITECTURAL CONCRETE MASONRY".* The sample is written based upon Spec-Brik but can be modified for any of the Hi-R and Hi-R H products. It is available on-line under Spec-Brik Specifications in the Product Specifications tab. It is a free download at: http://www.concreteproductsgroup.com/index.php/resources/product-specifications.

This specification could be modified for use in conjunction with the engineer's Reinforced Masonry specification. TMS also requires that "Contract documents shall specify the minimum level of quality assurance as defined in Section 3.1, or shall include an itemized quality assurance program that equals or exceeds the requirements of Section 3.1."

Drawing Information

TMS Section 1.2.1 requires that the design professional show the following on the drawings:

(a) Name and date of issue of Code and supplement to which the design conforms.

(b) Loads used for the design of masonry structures.

(c) Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, for each part of the structure, except for masonry designed in accordance with Part 4 or Appendix A.

(d) Size and location of structural elements.

(e) Details of anchorage of masonry to structural members, frames, and other construction, including the type, size, and location of connectors.

(f) Details of reinforcement, including the size, grade, type, lap splice length, and location of reinforcement.

(g) Reinforcing bars to be welded and welding requirements.

(h) Provision for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature, and moisture.

(i) Size and permitted location of conduits, pipes, and sleeves.

These items are required for all reinforced masonry projects. Specific to Hi-R and Hi-R H systems, engineers should be note that (f) requires lap splice lengths to be provided unless mechanical splices/couplers are included in the specifications. In addition, the locations of movement joints are effectively required by (h). These are often not included on many design drawings but are required by code.

Acknowledgements

The author acknowledges Mr. William Dawson, CPG Executive Director, and Mr. Canan D’Avela, CPG Technical Director, for their assistance in developing this Manual. Special thanks go to Professor Arturo Schultz, PhD for his technical review and wise counsel.

The Masonry Society is acknowledged for the many references.

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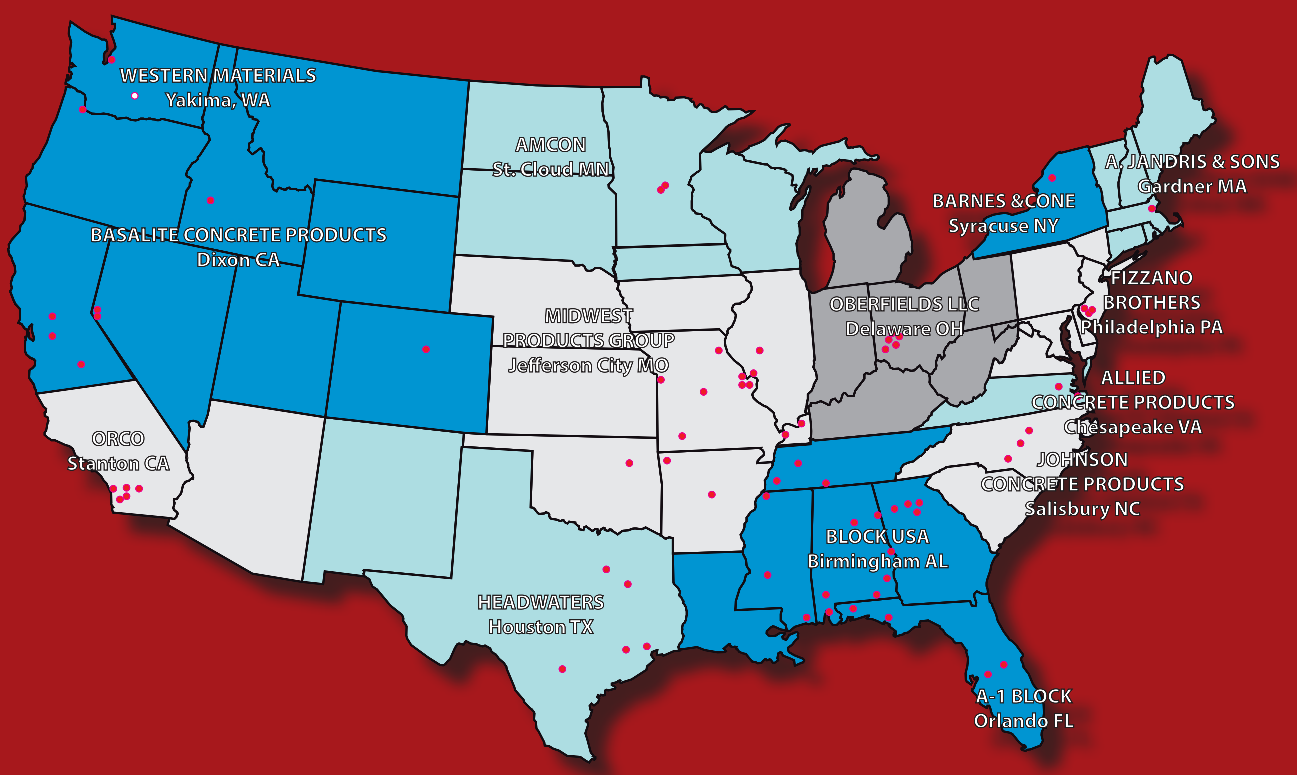

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