

STRUCTURAL BRICK INFORMATION

Physical Analysis of Concrete Masonry Units	Pg 2
Hollow Brick Specs Sheet	Pg 6
Compressive Strength Sheet	Pg 7
Design Guide for Structural Brick Veneer	Pg 8
Fire Endurance Ratings of Clay Brick Masonry	Pg 70
Allied Associates and Web Addresses	Pg 83
American Testing Services reports 2022, 2024, 2025	Pg 84
Reinforced Hollow Clay Masonry	Pg 87
Reinforced Structural Clay Brick	Pg 146
Interstate Brick Types and Safety Data	Pg 152



EXPERIENCED | INNOVATIVE | AUTHENTIC

Physical Analysis of Concrete Masonry Units

for

Northfield Block

8-in Regular

**Northfield Block Company
One Hunt Court
Mundelein, IL 60060**

March 7, 2022

March 7, 2022

Northfield Block Company
One Hunt Court
Mundelein, IL 60060**REPORT OF TESTS**

SUBJECT: Physical Analysis of Concrete Masonry Units

PROJECT: Northfield Block – 8 x 8 x 16-in Regular Concrete Masonry Units

SPECIFICATION: ASTM C90-21, "Specification for Loadbearing Concrete Masonry Units"

TEST METHODS: ASTM C140-21, "Test Methods for Sampling and Testing Concrete Masonry Units and Related Units."
ASTM C426-16, "Standard Test Method for Linear Drying Shrinkage of Concrete Masonry Units"
ACI 216.1, "Standard Method for Determining Fire Resistance of Concrete and Masonry Construction Assemblies."

MATERIALS: Delivered to NTL on February 21, 2022

NTL PROJECT #: 22-1046

PAGE: 1 of 3

TEST DATA

Dates of Testing: February and March 2022

Specimen ID	<u>1</u>	<u>2</u>	<u>3</u>	<u>Average</u>
Dimensions (in)				
Height	7.7	7.6	7.7	7.7
Width	7.6	7.6	7.6	7.6
Length	15.6	15.6	15.6	15.6
Min. Face Shell Thickness (in)	1.3	1.3	1.3	1.3
Min. Web Thickness (in)	1.1	1.1	1.1	1.1
Normalized Web Area (in ² /ft ²)	29.2	29.3	29.2	29.2
Gross Area (in ²)	118.6	118.6	118.6	118.6
Net Area (in ²)	60.8	60.1	60.4	60.4

March 7, 2022
Northfield Block – 8-in Regular Concrete Masonry Units
NTL Project #22-1046
Page 2 of 3

TEST RESULTS

Specimen ID	<u>1</u>	<u>2</u>	<u>3</u>	<u>Average</u>
<u>Density</u> (lbs/ft ³)	134.0	135.0	133.7	134.3
<u>*Absorption</u> (%)	6.3	7.0	7.2	6.9
<u>*Absorption</u> (lbs/ft ³)	8.5	9.5	9.7	9.2
<u>Solids</u> (%)	51.3	50.7	51.0	51.0
<u>Equivalent Thickness</u> (in)	3.9	3.9	3.9	3.9
<u>**Fire Rating</u> (hours)	2.0	2.0	2.0	2.0

Specimen ID	<u>4</u>	<u>5</u>	<u>6</u>	<u>Average</u>
<u>*Compressive Load</u> (lbs)	249,900	238,700	243,380	243,990
<u>Net Compressive Strength</u> (psi)	4,140	3,950	4,030	4,040

Specimen ID	<u>7</u>	<u>8</u>	<u>9</u>	<u>Average</u>
<u>Shrinkage</u> (%)	0.038	0.042	0.042	0.041

**Absorption and compressive strength testing on full units.*

***Calculations based on aggregate types.*

SUMMARY

The test results listed above complied with their corresponding requirements as set forth in ASTM C90-21, "Specification for Loadbearing Concrete Masonry Units".

March 7, 2022
Northfield Block – 8-in Regular Concrete Masonry Units
NTL Project #22-1046
Page 3 of 3

Respectfully submitted,

NELSON TESTING LABORATORIES



Mark R. Nelson
President

Notes: The results listed within this report relate only to the materials submitted for testing. This report shall not be reproduced, except in full, without written approval of this laboratory. The test materials not consumed in this testing will be discarded 14 days from the date of this report unless we receive written notification requesting otherwise.

Nelson Testing Laboratories is accredited/inspected by AASHTO and CCRL under ASTM C1093.

Specification for Atlas® Hollow Brick

Part 1 Quality Assurance

Brick Tests

1. All tests shall be performed by an independent certified testing laboratory.
2. All tests shall be in accordance with ASTM C-67.

Submittals

1. Submit test report and certificate of conformance document for each type and color of brick specified on contract documents for architect's approval.
2. Test reports shall include:
 - a. Compressive strength
 - b. 24 hour cold water absorption
 - c. 5 hour boil absorption (if required)
 - d. Saturation Coefficient (if required)
 - e. Initial Rate of Absorption (I.R.A.)
 - f. Efflorescence
 - g. Weather classification
3. Certificate of conformance shall state that brick meets or exceeds applicable ASTM specifications.

Sample Panels

1. Sample panel size shall be 4'x4' showing the proposed color range, texture, bond, mortar, workmanship, cleaning, and water repellents where applicable.
2. Final brick selection shall be made only following architect's review of sample panel.
3. Brick from manufactured material for project shall be shipped to site and sample panel erected.
4. No brick shall be shipped from manufacturer to site until architect's acceptance of job panel constructed from actual material for project. This panel shall replace all other sample panels and shall remain on site throughout construction, and become the project standard for bond, mortar, workmanship, and appearance.

Part II Products

Hollow Brick

1. Manufacturer: **Interstate Brick Company** or **H.C. Muddox**
2. ASTM C-652, Grade SW, Type HBX or better.
3. Color and texture
4. Dimensions (*width*) x (*height*) x (*length*).
5. Minimum compressive strength 9,000 psi.
6. Maximum Saturation Coefficient 0.78 (if required).
7. Minimum IRA 6 g/ min/30 in².
8. Maximum IRA 30 g/min/30 in². Where IRA exceeds 30 g/min/30 in², pre-wetting brick is recommended.
9. Shapes; where special shapes are shown on architectural drawings, manufacturer shall provide shop drawings for

architect's approval prior to manufacturing shapes.

Mortar

Mortar shall conform to ASTM C 270 Type S

Proportion Specification:

1 part Portland cement (ASTM C 150 Types I, IA, II, IIA, III, IIIA or V) less than 6 months old
 ½ part hydrated lime (ASTM C-207)
 3-½ to 4-½ parts sand (ASTM C-144)
 OR

Property Specification:

Portland cement - lime mortar
 Compressive strength 1800 psi
 Water retention 75% minimum
 Air content 12% maximum
 Sand: 2 ¼ -3 ½ the sum of the separate volumes of cementitious materials sand (ASTM C-144)

Grout

Grout shall conform to ASTM C 476
 Grout strength shall be greater than or equal to f'_m

Part III Execution

Bond

Bond shall be running bond unless otherwise shown on contract documents.

Jointing

Mortar joints shall be concave unless otherwise shown on contract documents.

Construction

All construction strictly adheres to International Building Code (IBC) or TMS 402/ACI530/ASCE5.

Cleaning

Cleaning shall conform to Interstate Brick Technical Bulletin 4 Brick Cleaning Recommendations, Section 040120 and BIA Technical Note #20 Cleaning Brickwork. Do not use muriatic acid or sand blasting.
 Contact manufacturer for recommendations.

Water Repellent & Coatings

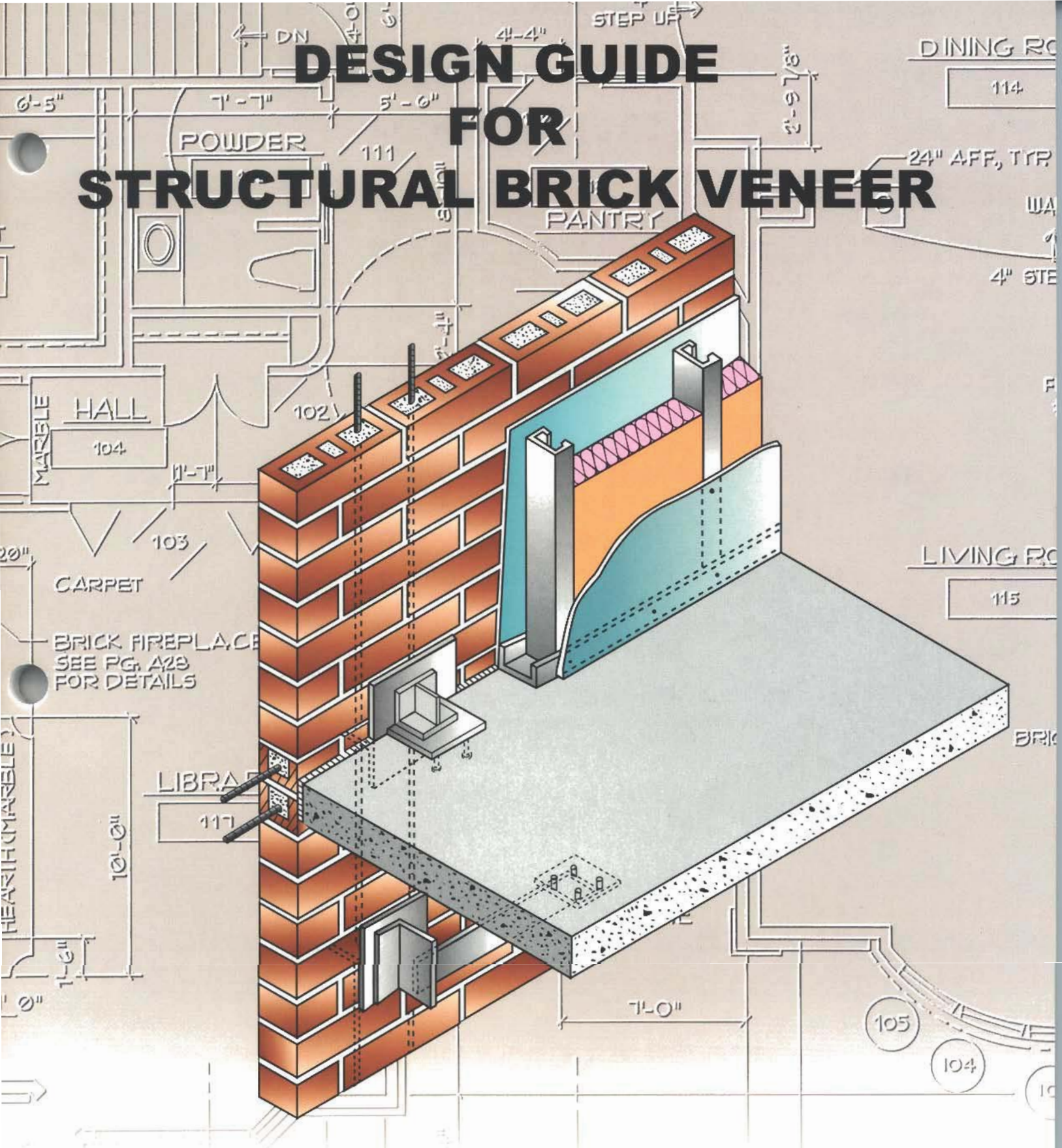
Where water repellents are required, consult Interstate Brick Technical Bulletin 1 Water Repellent Coatings and Section 071900.

Structural Brick Compressive Strength Data Table

Selected Brick Colors	Average Net Area Unit Compressive Strength		Assumed Wall Assembly Design Strength (f'_m)	
	psi	(MPa)	psi	(MPa)
Arctic White	11,000	(75.84)	3,844	(26.50)
Autumn Red	12,000	(82.74)	4,000	(27.58)
Bronzestone	11,900	(82.05)	4,000	(27.58)
Canyon Rose	13,600	(93.77)	4,000	(27.58)
Cedar	13,200	(91.01)	4,000	(27.58)
Copperstone	14,000	(96.53)	4,000	(27.58)
Desert Sand	14,000	(96.53)	4,000	(27.58)
Golden Buff	15,400	(106.18)	4,000	(27.58)
Ironstone	14,600	(100.66)	4,000	(27.58)
Midnight Black	15,000	(103.42)	4,000	(27.58)
Monterey	14,400	(99.28)	4,000	(27.58)
Mountain Red	15,600	(107.56)	4,000	(27.58)
Ochre Buff	14,700	(101.35)	4,000	(27.58)
Park Rose	13,600	(93.77)	4,000	(27.58)
Platinum	11,000	(75.84)	3,844	(26.50)
Smokey Mountain	15,500	(106.87)	4,000	(27.58)
Tumbleweed	15,400	(106.18)	4,000	(27.58)
Walnut	12,800	(88.25)	4,000	(27.58)

1. Clay and shales are naturally occurring materials that vary, to some degree, throughout the deposits and strata layers. Consequently, various physical properties of the brick units, including the unit compressive strength values noted above are variable. The data above is based on historic data, not future projections. The data for the noted colors, and other colors, is kept on file, available upon request.
2. Assumed Wall Assembly Design Strengths f'_m , are based on TMS 602-22, Section 1.4-B-2(a), Table 1, and represent the design strength of wall assemblies comprised of masonry units, mortar, and grout. Mortar is assumed to be ASTM C270 compliant Type M, or Type S mortar. Grout is assumed to be ASTM C476 compliant grout with 28-day minimum compressive strength (f'_c) equal to or greater than f'_m .
3. Wall Assembly Design Strengths (f'_m) in excess of 4,000 psi can be achieved based on job specific prism testing.

DESIGN GUIDE FOR STRUCTURAL BRICK VENEER



prepared for

Western States Clay Products Association

by

KPFF Consulting Engineers

DESIGN GUIDE

FOR

STRUCTURAL BRICK VENEER

Prepared for:

Western States Clay Products Association
www.wscpa.us

Submitted by:

KPFF Consulting Engineers
1601 5th Ave. Suite 1600
Seattle, WA 98101

Second Edition, July 2004

Acknowledgments

The Design Guide for Structural Brick Veneer was sponsored by Western States Clay Products Association and written by KPFF Consulting Engineers. Principal-in-Charge for KPFF was John G. Tawresey. The preparation of the report was directed by the Technical Committee of the Western States Clay Products Association. The authors are appreciative of the help provided by Jim Anderegg, Mutual Materials Company, James Amrhein, Gregg Borchelt, Brick Institute of America, Jeff Elder, Interstate Brick Company and Don Wakefield.

John G. Tawresey, KPFF Consulting Engineers prepared the 2004 Second Edition.

The material presented in this publication, including technical and engineering data, figures, drawings and tables, is for general information only. It should not under any circumstances be relied upon for specific applications of the Structural Brick Veneer without independent evaluation by a licensed design professional familiar with its specific use and application. Anyone making use of this material does so at their own risk and assumes any and all liability resulting from such use.

Copyright 2004 by Western States Clay Products Association,

All rights reserved. No part of this publication may be reproduced, stored in a computer or retrieval system or transmitted in any form or by any means, electronic, mechanical, photocopy, fax, recording or otherwise, without the prior written consent of the author or Western States Clay Products Association.

Summary

Structural Brick Veneer is a unique approach to the design and construction of brick exterior walls. Strengthening the brick with steel reinforcement provides new opportunities for reducing the cost of the wall, increasing design flexibility and improving wall performance. In use for more than thirty years, the approach has been used extensively in the Pacific Northwest.

The Structural Brick Veneer system is similar to conventional brick veneer except that the brick is reinforced to allow it to span further between ties and supports. The system allows the architect a variety of opportunities to create traditional walls or dramatic brick forms. Sloping windowsills, brick soffits, lintels without exposed ledger angles and precast concrete bands and inserts are only a few examples of the design opportunities available.

In areas of high seismic exposure, the Structural Brick Veneer system can be easily isolated from the primary structure making “immediate occupancy” performance more cost effective.

The following document will provide the structural engineer and the architect with an introduction to the design and specification of the Structural Brick Veneer system. It includes some design examples and dialogue on our experiences with the system over the past 20 years.



List of Figures

FIGURE 1 TYPICAL HOLLOW BRICK	8
FIGURE 2 TYPICAL STRUCTURAL BRICK VENEER CONNECTOR	8
FIGURE 3 BRICK SUPPORTED ON A LEDGER	8
FIGURE 4 TYPICAL STRUCTURAL BRICK VENEER DEADLOAD CONNECTOR	8
FIGURE 5 DEAD LOAD ON THE FOUNDATION	9
FIGURE 6 FLEXIBLE CONNECTORS	9
FIGURE 7 SLOPING SILL BRICK SOFFITS	10
FIGURE 8 BRICK SOFFIT	10
FIGURE 9 PRECAST IN STRUCTURAL BRICK VENEER	11
FIGURE 10 BRICK EXPANDING RESISTED BY THE REINFORCEMENT	12
FIGURE 11 CRACKED BRICK	17
FIGURE 12 DIFFERENTIAL VERTICAL DEFLECTION	18
FIGURE 13 HORIZONTAL DRIFT JOINT	18
FIGURE 14 CORNER CONNECTED TO THE STRUCTURE	19
FIGURE 15 CORNER NOT CONNECTED TO THE STRUCTURE	19
FIGURE 16 HOSPITAL - PROJECT 1	21
FIGURE 17 PROJECT 1 – PIER CONCEPT BETWEEN WINDOWS	22
FIGURE 18 LATERAL CONNECTION	22
FIGURE 19 DRIFT JOINT BEFORE LEDGER INSTALLATION	23
FIGURE 20 LEDGER INSTALLATION	23
FIGURE 21 PROJECT 2	23
FIGURE 22 HOSPITAL WITH MANY CORNERS	24
FIGURE 23 TEMPORARY RIGID FOAM BOARD SUPPORT	24
FIGURE 24 CONSTRUCTION SEQUENCE	25
FIGURE 25 BUILDING WITH ANTICIPATED DIFFERENTIAL SETTLEMENT	25
FIGURE 26 CONCEPT FOR SETTLEMENT	26
FIGURE 27 STIFFNESS OF BRICK MASONRY	27
FIGURE 28 STRIP SYSTEM EXAMPLE	29
FIGURE 29 SUPPORT FOR THE WALL	29
FIGURE 30 BRICK USED IN THE EXAMPLE	30
FIGURE 31 PLAN VIEW OF CONNECTOR	32
FIGURE 32 SIMPLE LATERAL CONNECTOR	34
FIGURE 33 FINAL DESIGN	36
FIGURE 34 LOCATION OF CONNECTORS	36
FIGURE 35 PLAN VIEW OF CONNECTOR	37
FIGURE 36 DEAD LOAD CONNECTOR	38
FIGURE 37 DEAD LOAD MOMENT	38
FIGURE 38 RESISTING MOMENT	39
FIGURE 39 WARPING CORNER	41
FIGURE 40 FLUID GROUT	43
FIGURE 41 PROTRUDING FLASHING DETAIL	45
FIGURE 42 FLUSH FLASHING DETAIL	45
FIGURE 43 VARIATION OF FIELD MORTAR TESTS	54
FIGURE 44 MORTAR 7 DAY AND 28 DAY TESTS	54
FIGURE 45 VARIATION OF FIELD GROUT TESTS	54
FIGURE 46 SEVEN DAY AND 28 DAY GROUT STRENGTH	55

List of Figures	4
1.0 Introduction	6
1.1 Purpose & Scope.....	6
1.2 History of the System.....	6
2.0 System Description	7
2.1 Structural Concept.....	7
2.2 Concept Configurations	9
2.3 Weather Protection	11
3.0 Design.....	12
3.1 Who Designs the Wall?	12
3.2 Design Criteria	13
3.3 Designing the Wall	21
3.4 Design Examples	29
4.0 Specification	42
4.1 Quality Control and Assurance.....	42
4.2 Masonry	43
4.3 Steel for Connectors	44
4.4 Flashing/Weeps	44
4.5 Sealants	45
4.6 Water Repellents	45
4.7 Backup Wall	47
4.8 Cavity	47
4.9 Expansion Joints	47
4.10 Window Anchorage.....	48
5.0 Construction	48
5.1 General.....	48
5.2 Construction Sequence	49
5.3 Pre-Construction	49
5.4 Submittal Review.....	51
5.5 Site Visits.....	52
5.6 Non-Conforming Quality Control Tests.....	53
5.7 Troubleshooting During Construction	56
6.0 Testing	60
6.1 Air.....	60
6.2 Water	60
6.3 Structural	60

1.0 Introduction

Structural Brick Veneer is the name that we have chosen to describe hollow reinforced clay brick curtainwall systems. These systems commonly replace brick veneer walls.

The Structural Brick Veneer system is similar to conventional brick veneer because it supports no gravity loads other than its own weight, the weight of windows, and possibly other miscellaneous loads. The difference is that in the Structural Brick Veneer system the masonry is reinforced to allow the brick to span further between ties and provide structural capacity to create more intricately shaped walls.

Structural Brick Veneer can be laid in place similar to conventional brick veneer, or they can be prefabricated at another location and lifted and installed to their final position.

The system has many advantages over conventional brick veneer. Some of these advantages are:

1. Greater design flexibility.
2. Reduced backup requirements.
3. Enhanced design life through heavier connections.
4. Reduced tie connections, which provides more continuous moisture barrier.
5. Greater seismic resistance and more ductility.
6. Less restrictive deflection requirements of the backup structure.
7. Reduced cost of the backup system.
8. Often lower construction cost.

9. Greater resistance to cracking.
10. Greater water resistance.

The system has been used on more than 100 projects over the last 20 years.

1.1 Purpose & Scope

The purpose of this guide is to provide the architect, structural engineer and owner with information about the design and construction of Structural Brick Veneers. It is intended to be easily understood by someone experienced with reinforced brick masonry design and construction. For those not familiar with the design of reinforced brick masonry, the Western States Clay Products Publication, *“Notes on the Selection, Design and Construction of Reinforced Hollow Clay Masonry”* is recommended.

As the design and construction of reinforced brick masonry varies from location to location, so does the design and construction of Structural Brick Veneer vary from location to location. The information and recommendations in this guide are based on the design and construction of projects in the Pacific Northwest and are not intended to replace local experience and engineering judgment.

1.2 History of the System

The origin of Structural Brick Veneer dates back to the early sixties. In 1962, a mechanical equipment penthouse was built on top of the nine-story United Fund office building in Denver, Colorado. The 15-foot high, load

bearing, 4-inch thick clay brick prefabricated panels supported long span, prestressed, twin-tee concrete slabs that were the roof structure of the penthouse. This construction was made possible by the use of a new "tensile strength intensive" exotic mortar and some backup steel reinforcing.

This 4-inch brick and exotic mortar system was used for several years thereafter in the Colorado area in prefabricated and in situ, hand-laid brick panel and curtainwall applications on many commercial buildings designed by George Hanson, P.E. of the firm of Sallada & Hanson, Engineers. This strong thin-wall system intrigued the designers who used this system on horizontal soffits, cantilevered balcony railings, post-tensioned panels, load-bearing and non-load-bearing walls on schools, office buildings, hospitals, the walls in vehicular tunnels, highway rest area toilet modules and picnic shelters. Even though this brick curtainwall system was very successful, it was relatively expensive due to the use of the high tensile strength mortar.

In the middle 1960's, Donald A. Wakefield, P.E. of the Structural Clay Products Institute developed, in Colorado, a new clay unit and method of construction that reduced the cost and allowed for the use of regular reinforcing and standard mortar and grout. This unit was a 3-1/2" x 3-1/2" x 11-1/2" hollow clay brick using ASTM C-212 recommendations. This system accommodated both horizontal and vertical reinforcing and permitted high-lift grouting.

This thinner reinforceable, hollow clay unit was more economical, ductile, flexible and more predictable; thus expanding its use in commercial curtainwall systems as well as load-bearing residences, four-story load-bearing apartment buildings, and prefabricated panels.

During the 1970's, a similar system was developed and perfected by Barkshire Panel Systems, Seattle, WA. Barkshire's system used a 3 1/2-inch thick hollow clay brick similar to the one mentioned earlier and verified by testing conducted by Western States Clay Products Association under the direction of John G. Tawresey.

The secret to Barkshire's system was advancements in the connections to the frame, and the technical knowledge of the overall building's physical needs.

As a consequence, the system is found on multi-story high-rise office buildings, schools, apartment buildings, residences and many other applications throughout the Northwest.

2.0 System Description

The system relies on a simple structural concept that will be described, followed by the presentation of some typical configurations and a brief discussion of weather protection.

2.1 Structural Concept

Structural Brick Veneer is essentially the same as conventional brick veneers except that the brick is reinforced. Hollow bricks that can be reinforced are a necessary part of the system.

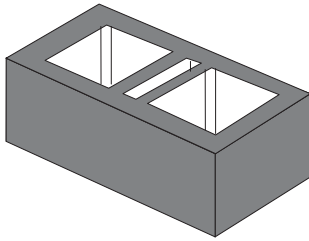


Figure 1 Typical Hollow Brick

The reinforcement increases the structural capacity of the brick wall. The spacing of the ties, typical in conventional veneers, can be increased and in most cases, the spacing can be increased to the distance between building floors or columns. The conventional veneer ties are eliminated and are replaced by more substantial connectors. The connectors are usually attached to the primary structural system of the building instead of a separate backup wall.

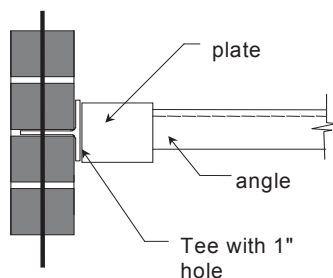


Figure 2 Typical Structural Brick Veneer Connector

Ledgers very similar to the ledgers of more conventional veneers can support

the dead load of the Structural Brick Veneer.



Figure 3 Brick Supported on a Ledger

Or, separate discontinuous connectors can be used to support the dead load. These connectors are similar in design to those used to support precast concrete panels.

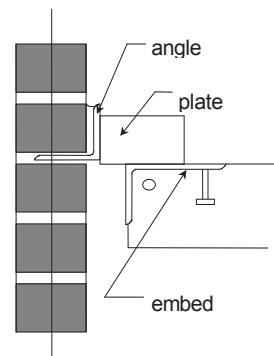


Figure 4 Typical Structural Brick Veneer Deadload Connector

Another available option is to support the dead load of the Structural Brick Veneer on the building foundation. Because the Structural Brick Veneer is designed in accordance with the

structural chapters of the code, height limitations are generally more liberal.

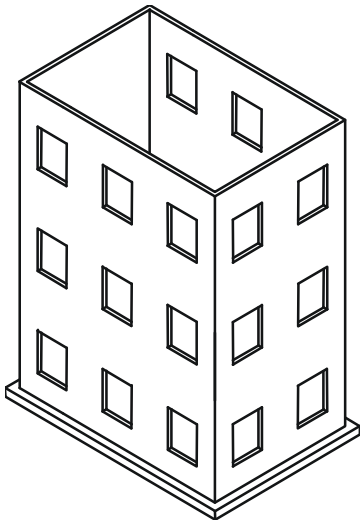


Figure 5 Dead Load on the Foundation

The connectors of the Structural Brick Veneer are designed and constructed to be flexible in one or more directions and rigid in other directions. Thus, the Structural Brick Veneer can be isolated from the movements of the building.

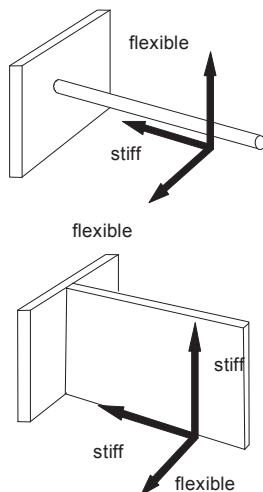


Figure 6 Flexible Connectors

The design of the Structural Brick Veneer is based on the following principles:

1. The Structural Brick Veneer is designed to be isolated from the primary building structure. For non-essential facilities, the isolation should be adequate to insure that the brick will not be damaged by a moderate earthquake. For essential facilities, the isolation should be adequate to insure that the brick will not be damaged by a maximum considered earthquake. The Structural Brick Veneer must not support the building or provide any assistance to the stability of the building as a whole.
2. Structural Brick Veneer is commonly designed to have **mortar joint** cracks at service wind and seismic loading. However, **brick cracking** should not occur. Cracking should be limited to the cracking of the horizontal bed joints at the brick to mortar interface. This is an aesthetic design criteria rather than a structural performance criteria. Experience has shown that cracking of the brick unit is considered by most owners to be a failure of the system.
3. The Structural Brick Veneer is designed to transfer the loading to the connectors and the connectors are designed to transfer the loading to the primary structure.

2.2 Concept Configurations

Structural Brick Veneer buildings can be configured in limitless ways. Because the brick wall has more capacity to resist

loading, the designer has more choices to configure and attach the wall. The number of different forms is controlled only by the designer's imagination.

The Structural Brick Veneer system offers the architect a variety of opportunities to create dramatic brick forms. Sloping windowsills, brick soffits, lintels without exposed ledger angles and precast concrete bands are only a few examples.

Sloping Sills

Sloped sills are a common accent in brick construction. Small slopes created from special brick shapes can be readily incorporated into the Structural Brick Veneer wall. Larger, more dramatically sloped sills may require shoring to construct.

Where the depth of the slope does not allow the weight of the brick veneer to cantilever from the vertical wall, then adding connections or spanning horizontally to column supports can provide alternative support.

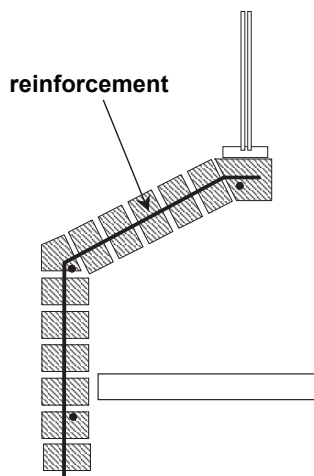


Figure 7 Sloping Sill Brick Soffits

Brick soffits can be designed and constructed similar to the conditions at a sloping sill.

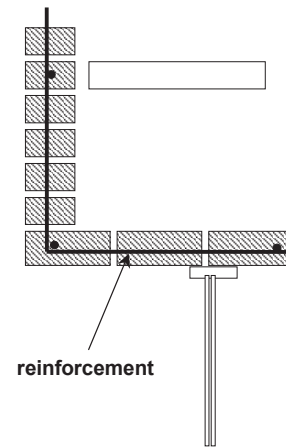


Figure 8 Brick Soffit

Brick with Concrete Masonry

Another architectural variation is to combine concrete masonry with Structural Brick Veneer. This commonly takes the form of banding, either horizontal or vertical, or as an accent pattern. In some circumstances, concrete masonry may be used instead of brick where it doesn't show. In either arrangement, concrete masonry can be incorporated into the Structural Brick Veneer with ease. The designer should recognize however that the spacing of reinforcement in brick might not match the cells in the concrete masonry. Some effort should be made to space the reinforcement to match both modules. Also, when using horizontal brick and concrete masonry bands, the designers must recognize the opposing behavior and strengths of the two materials and detail accordingly with movement joints and reinforcement. Brick will expand with exposure to moisture while concrete masonry is more dynamic and

will shrink with drying and expand with moisture. Extra reinforcement may be added to reduce the impact of this effect.

Brick with Precast Concrete

Precast concrete window sills and heads as well as accents can be successfully included in a Structural Brick Veneer System. Where the precast elements are small, they can be added by providing holes through the precast for the reinforcement. Large precast components may require separate connectors to the building frame or to the Structural Brick Veneer.

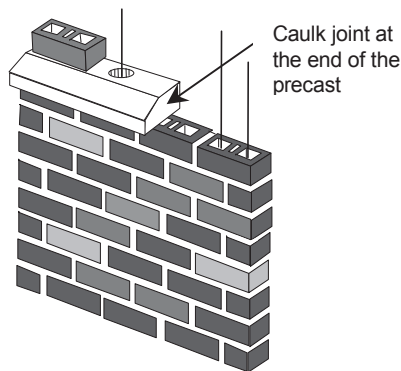


Figure 9 Precast in Structural Brick Veneer

Also, precast concrete elements shrink with time and drying, and expand with moisture. Again, the designer should consider the compatibility of the expanding brick and shrinking concrete.

2.3 Weather Protection

The primary function of the exterior wall is to protect the interior of the building from the weather.

The Structural Brick Veneer System provides two methods of weather protection. The exterior brick veneer, caulking, and windows act as the primary barrier. The interior cavity, flashing, weeps, window channels and other elements act as the secondary barrier.

The system performance depends to a large extent on the prevention of water leakage through the reinforced brick. Reinforced brick is more water-resistant than unreinforced brick. The most obvious reason is that the cracks are smaller and more evenly distributed due to the resistance provided by the reinforcement.

Another reason is that brick expands with age. When the brick is made, it contains no water. With time, the brick absorbs water and reaches equilibrium with its moist environment. The clay expands (just like unfired clay). The moisture can come from the humidity in the air. This expansion can take years.

ACI 530-02/ASCE 5-02/TMS 402-02 *Building Code Requirements for Masonry Structures* recommends a value of .0003 inch/inch for moisture expansion of clay masonry.

When the brick expands, the reinforcement tends to resist the expansion. The reinforcement is stretched in tension and the brick masonry compressed. The consequence is smaller and sometimes there are fewer cracks.

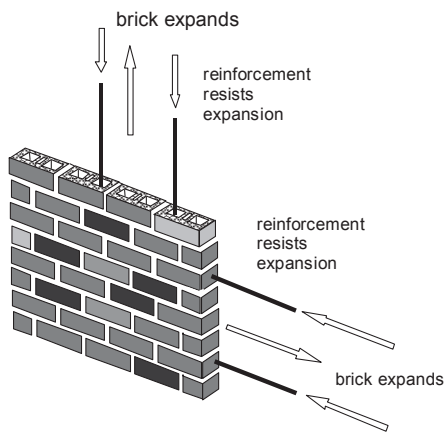


Figure 10 Brick Expanding Resisted by the Reinforcement

Even with smaller cracks, leakage can still occur. Wind-driven water flows in sheets in all directions over the wall and concentrates at discontinuities such as joints. Lateral movement of water is greatest near the windward corners. Movement upward is greatest near the top of the building. Tall buildings have greater accumulation of water flow. Greater distances between irregularities will result in larger flows.

It should be assumed that all masonry might leak and allow water to penetrate. Masonry leaks more through the mortar and brick interface than through the masonry unit itself. If the mortar and brick interface is cracked, leakage may increase.

There are two concepts used to control the water. The first is the drainage wall, and the second is the “rainscreen” principle.

A drainage wall consists of a secondary method of removing the moisture once it

has penetrated the outside wall. Flashing is an example.

The principle of the rainscreen consists of providing a cavity behind the wall that equalizes the outside pressure thus preventing the water from penetrating the outside surface. A properly proportioned rainscreen wall needs no caulk on the outside barrier. Unfortunately, the available technology for designing rainscreen walls is limited and while many walls use some of the principles, most installations rely on a drainage wall system. For more information on the rainscreen walls, see Western States Clay Products publication “*Design Guide for Anchored Brick Veneer Over Steel Studs*”.

3.0 Design

Once the decision to use Structural Brick Veneer is made, the next step is to provide for a design. The Structural Brick Veneer is not part of the primary structural system and, consequently, may, or may not be designed by the building design team. It may not be in their scope of services. Thus the first issue to address is who designs the wall? Whether designed by the building design team or by a designer working for the contractor or sub-contractor, the design criteria and methods of analysis are the same. In the next paragraphs, these methods are described followed by three design examples.

3.1 Who Designs the Wall?

Structural Brick Veneer, like other wall systems, can be designed by either the design team and bid, or it can be specified as a bidder-designed item. It is done both ways.

Some owners complain to their architect that the design is incomplete without drawings and specifications to bid the exterior wall. It is like “buying a car without the wheels.” Yet, contractors design most exterior walls because they have special knowledge necessary to complete the task.

Questions that should be addressed early in the design of any wall system include the following. How much of the design of the wall should be shown on the design drawings? Where does the designer of record’s responsibilities end and the contractor’s responsibilities begin?

Often, the structural engineer of record does not show bracing because he is unaware of the wall system to be used and the method of attachment is left to the contractor. The designer should clearly identify on the drawings the responsibility for the design and construction of each component of the wall system, including bracing.

In some locations, it is common for brick veneer supports to be shown on the design drawings. In Seattle, it is common for the structural engineer of record to design precast concrete panels as part of the “standard services”. In most locations, including Seattle, the contractor prepares the design of Structural Brick Veneer.

These examples are presented to illustrate that there is no consensus about who should design an exterior wall. The “*National Practice Guidelines for the Structural Engineer of Record*” published by the Council of American Structural Engineers clearly states that

the design of curtainwall systems is not considered part of Basic Services. It is considered Additional Services under Special Services. Unfortunately, the document does not provide us with help for the bracing and stiffening problem. Nor does it show where to draw the line of responsibility between the contractor and the structural engineer of record.

No matter who designs the wall, the contractor or the engineer of record, the following information about the design criteria and methods applies.

3.2 Design Criteria

The design criteria for Structural Brick Veneer includes the applicable code sections, the appropriate design life, seismic performance, loads, allowable stresses, code prescriptive requirements and special connection requirements.

3.2.1 The Applicable Code Sections

The term “Structural Brick Veneer” may cause some confusion when applying the building code. The use of the term “Veneer” implies “non-structural”. However, the structural analysis of the Structural Brick Veneer uses the structural portions of the masonry design codes. But, they are used in combination with the performance criteria of the masonry veneer sections of the code.

3.2.2 Design Life

Design life is an important quantitative measure that defines the quality of the project. Buildings will not last forever. The owner and designer should establish a reasonable design life for each project. This requires consideration of

the economic factors such as initial cost and maintenance costs. The design life will have an impact on the selection of materials, maintenance procedures, and the selected factors of safety.

The expected performance is also an important qualitative measure for the design of the project. The minimum performance level is set by the building code, however, there are aspects of a Structural Brick Veneer System performance that are not explicitly covered by the code and require judgment.

It has been useful to define two distinct levels of expected life and performance:

Level 1 (institutional) is intended to signify a high level of quality and long life. Buildings of this type might include public or institutional buildings. Specifically, these are buildings where the additional costs associated with higher quality are judged to be necessary in meeting the overall project requirements.

Level 2 (commercial) is intended to signify a good level of quality and an average design life. Buildings of this type might include general office, industrial, and residential buildings. These are buildings where the additional cost of Level 1 (institutional) quality is not economically justified or necessary.

Increasing the quality of the connectors, improving the weather resistance of the materials and expanding on the amount of inspection and testing are the normal means to increase the design life.

3.2.3 Seismic Performance

Seismic performance of Structural Brick Veneer is a complex subject since under certain levels of seismic shaking, damage can occur to the veneer. There are currently several organizations preparing standards for seismic design. The National Earthquake Hazards Reduction Program (NEHRP-2000) divides the performance of structures into four levels:

1. Operational: "Structures meeting this level when responding to an earthquake are expected to experience only negligible damage to their structural systems and minor damage to nonstructural systems" (the structural brick veneer). "Repairs if necessary can be conducted at the convenience of the owner." "The risk to life is negligible."
2. Immediate occupancy: "Structures meeting this level are expected to sustain more damage to nonstructural systems" (the structural brick veneer). "Exterior nonstructural wall elements and roof elements continue to provide a weather barrier, and are otherwise serviceable" (although they may be damaged).
3. Life safety: "Significant structural and nonstructural damage has occurred." "Nonstructural elements of the structure, while secured and not presenting falling hazards, are severely damaged and can not function" (the structural brick veneer).
4. Collapse prevention: "The structure has sustained nearly complete damage. Nonstructural elements of the structure have experienced substantial damage and may have become dislodged creating falling hazards" (the structural brick veneer).

The NEHRP provisions also distinguish between building uses by assigning each structure a Seismic Use Group.

1. Group III are essential facilities requiring post-earthquake use.
2. Group II are facilities with a large number of occupants.
3. Group I are all other facilities.

Combining the performance classification with the occupancy distinction results in the following chart describing expected seismic performance.

	Operational	Immediate Occupancy	Life Safety	Near Collapse
Frequent Earthquakes (50% in 50 years)				
Design Earthquake (2/3 of MCE)				
Maximum Considered Earthquake (2% in 50 years)				

The building code is not specific about the seismic performance of curtainwall and Structural Brick Veneer. Judgment is required on the part of the engineer to develop appropriate seismic criteria based on the project performance objective. For example, failure of a Structural Brick Veneer over the firehouse door during a major earthquake is not acceptable. Whereas, complete separation of a Structural Brick Veneer from the frame on a suburban office building with surrounding planters may be acceptable.

Because of the nature of the connections, Structural Brick Veneer can be designed to perform to a higher seismic performance level than conventional veneer. For buildings with operational performance or immediate occupancy

criteria, Structural Brick Veneer offers significant advantages. If high seismic performance criteria are combined with a complex geometry, sills, soffits and articulated surfaces, it is likely that Structural Brick Veneer will be the least costly system.

3.2.4 Design Loads

Loads applied to a Structural Brick Veneer include dead load, wind load, and seismic load. The Structural Brick Veneer should support no vertical load other than its own weight. In normal practice, it may also support the weight of window systems, small air handling units and possibly some ornamentation.

Most modern building codes contain two levels of loading, service loads (allowable stress design) and ultimate loads (strength design). Both levels of loading are used in the design of Structural Brick Veneer. Local jurisdictions must be consulted for the correct design load criteria.

3.2.5 Design Assumptions and Allowable Stresses

Working Stress Design

Working stress design methods are recommended. Strength or limit states design methods are still in the development stage and have not been extensively used for the design of structural brick or Structural Brick Veneer. The ACI 530-02/ASCE 5-02/TMS 402-02 *Building Code Requirements for Masonry Structures* contains a new strength design section, Chapter 3. It may offer less conservative designs for Structural Brick Veneers resisting out-of-plane

wind and seismic loading particularly when compared to allowable stress design that does not allow a one-third increase in allowable stress for load combinations, including wind or seismic loading.

Allowable Stresses

The allowable stresses permitted in the Structural Brick Veneer are the same as those allowed for structural reinforced masonry. The one-third increase in allowable stress is typically used for load combinations, including wind and seismic loads.

The design of connectors requires additional allowable stresses not typically included in the building codes. These additional required allowables are as follows:

1. The shear cone capacity of masonry for pull-out is typically taken as the beam allowable shear stress ($1.0 \times (f'_m)^{1/2} \leq 50$ psi).
2. The shear cone angle is conservatively assumed to be 20 degrees instead of the more commonly used 45 degrees.
3. The torsion allowable stresses are assumed equal to the beam shear allowable stress ($1.0 \times (f'_m)^{1/2} \leq 50$ psi). Unpublished testing on several projects has confirmed the validity of this assumption for several projects. The tests consisted of square panels fixed on three corners and lifted on the fourth corner. Failure stresses were typically 2.5 times the allowable values.
4. The concrete shear friction equation is assumed to apply with a friction factor of 0.4, ($A_v = V_u / \phi F_y$). The ultimate shear is taken as 2.0 times the design shear.
5. The tension allowable stress of the brick mortar interface is typically neglected.
6. The tensile capacity of the brick is assumed to be 10% of the compression capacity. The allowable stress is determined by applying an appropriate factor of safety for the load condition being considered. This allowable is for the brick, not the masonry. The code provides allowable tension values for the masonry in tension parallel and perpendicular to the bed joints and for different bonding patterns and mortars.
7. The allowable bond stress to plane steel is assumed to be 60 psi. or the same value as the allowable bond stress to plane bars.

Cracking of Brick

At service loads, the brick units should not crack. Cracking of the interface between the brick and mortar is acceptable. However, a cracked brick could be an aesthetic problem even though the structural capacity may be adequate. The engineer should decide the appropriate service design loading and the factor of safety to be applied.

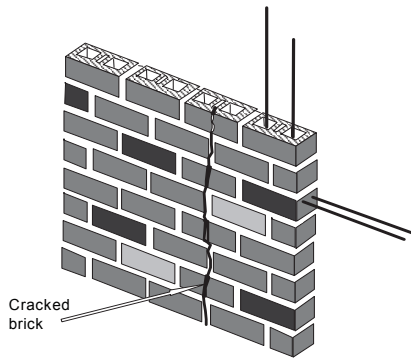


Figure 11 Cracked Brick

Code Minimum Requirements

It could be interpreted that because the Structural Brick Veneer is not classified as structural in the code, the prescriptive minimums of most structural codes do not apply. An example is that the usual minimum reinforcement spacing does not apply. However, the practice is to conform to these minimum requirements as though the Structural Brick Veneer is structural reinforced masonry.

Connector Requirements

Most seismic codes have special minimum criteria for the design of curtainwall connections. These provisions are principally directed towards precast concrete panels used as curtain wall and it is reasonable to assume they apply to Structural Brick Veneer.

One example is IBC 2003 Chapter 16, Section 1621 which references ASCE 7 (2002) Section 9.6.2.4:

1. Connection and panel joints shall allow for the story drift caused by relative seismic displacements (D_p) determined in Section 9.6.1.4, or $\frac{1}{2}$ in. whichever is greatest.

2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, and other connections providing equivalent sliding and ductility capacity.

3. The connecting member itself shall have sufficient ductility and rotation capacity so as to preclude fracture of the concrete (masonry) or brittle failures at or near welds.

4. All fasteners in the connecting system such as bolts, inserts, welds and dowels and the body of the connectors shall be designed for the force (F_p) determined by Eq. 9.6.1.3-2 with values of R_p and a_p taken from Table 9.6.2.2 applied at the center of mass of the panel.

5. Anchorage using flat straps embedded in concrete or masonry shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel or to assure that pullout of anchorage is not the initial failure mechanism.

3.2.5 Isolation from the Building

The most challenging part of the design is to isolate the Structural Brick Veneer from the building frame. There are many ways that this can be accomplished and a few will be discussed.

The amount of isolation is an important factor and the code does not provide precise criteria. There are three directions in space: vertical, horizontal in the plane of the wall, and horizontal perpendicular to the plane of the wall. The building behind

the wall, while seemingly static, is in fact subject to a variety of different movements in all three directions.

Vertical Isolation

The vertical movement can result from several different sources. The following figure shows one source.

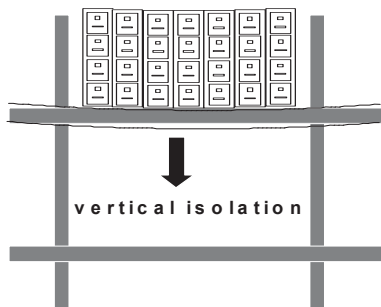


Figure 12 Differential Vertical Deflection

The different amount of vertical loading on the floors will result in a shortening of the distance between the floors. In many buildings, this can be an important factor and can result in differential deflections of greater than three-quarters of an inch. If the system does not provide compliance for this movement, the file cabinet load will be reacted by the Structural Brick Veneer instead of the floor beams and could cause failure.

If the building is constructed of concrete, the concrete shrinkage and creep will contribute to the shortening between floors. In high-rise construction, the elastic and differential elastic shortening may become important and may affect the construction schedule and sequencing.

Horizontal Isolation in the Plane of the Wall

Lateral forces from wind and seismic loading cause horizontal movement of the building frame. When one floor moves horizontally relative to the adjacent (higher or lower) floor, the wall system must accommodate the movement. If the Structural Brick Veneer is attached to both floors, then the veneer would resist the lateral forces and possibly fail. The amount of differential horizontal movement can be large; up to four inches is common in areas of high seismic activity.

A movement joint at the window head is commonly used to accommodate the horizontal movement.

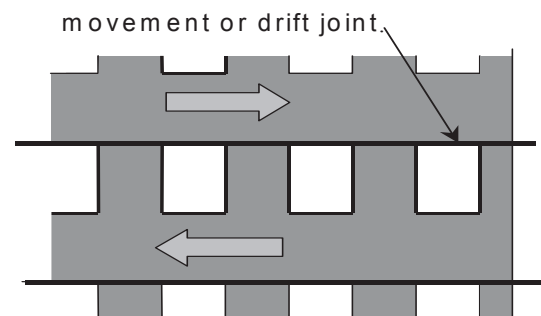


Figure 13 Horizontal Drift Joint

These joints are usually called drift joints and can be located at any horizontal plane of the building. When horizontal joints change elevation at different surfaces of the wall, it is difficult to accommodate horizontal displacement.

This concept is simple until the joint reaches the corner. Corners provide additional natural restraint, and can result in an unwanted attachment to the frame.

The following figure shows an example of a Structural Brick Veneer where the connection to the structure near the corner was designed to be rigid for forces perpendicular to the wall.

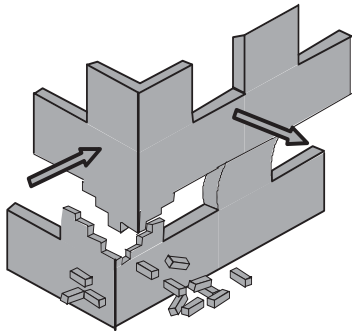


Figure 14 Corner Connected to the Structure

There are several methods available to provide isolation at a corner. One is to eliminate the corner connections and have the Structural Brick Veneer resist the loading.

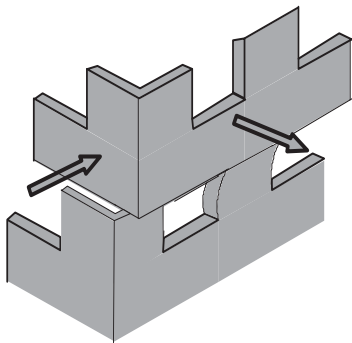


Figure 15 Corner Not Connected to the Structure

Horizontal Isolation Perpendicular to the Plane of the Wall

Another is to accommodate the movement with warping of the corner panel. This will be shown in one of the examples to follow.

For walls away from corners, horizontal movements perpendicular to the plane of the wall are accommodated by out-of-plane bending of the masonry. Typically, reinforced masonry has the capacity to accommodate large deflections in this direction and the isolation in this direction is seldom a design consideration.

Typical Deflection Magnitudes

The amount of isolation required depends on loading and expected performance. There are no known accepted national standards, but as a guide, Table 1 presents typical values and code requirements for isolation.

TYPICAL MOVEMENT DESIGN CRITERIA ⁽¹⁾

Movement Type	Structural System	Source of Movement	Limitations	Recommended Values	Typical Values	Isolation Method
Vertical Floors	Steel or concrete	Differential application of live load	L/600	<0.60 inch	1/4"	Compensation channel at the window head
						Soft joint under the ledger angle
	Concrete	Shrinkage with drying	(2)	(3)	1/16"	Allow the concrete to dry and cure before installing the veneer
		Creep	(2)	(3)	(3)	Provide vertical control joints every 20 to 30 feet
Vertical Columns	Steel	Differential elastic shortening	(2)	(3)	(3)	Only applies to high-rise buildings where the veneer is installed prior to finishing the building frame
	Concrete	Shrinkage	(2)	(3)	1/16"	Allow the concrete to dry and cure before installing the veneer
		Creep	(2)	(3)	1/16"	Compensation channel at the window head
						Soft joint under the ledger angle
Lateral	Frame of steel or concrete	Wind	(2)	.0025H	3/8"	Usually absorbed elastically in the system
		Seismic	Per analysis Minimum 1/2"	Depends on occupancy	2 1/2 TO 3"	Compensation channel at the window head
						Soft joint under the ledger angle
	Shear Wall	Wind	(2)	.0025H	1/8"	Usually absorbed elastically in the system
		Seismic	Per analysis Minimum 1/2"	Depends on occupancy	1/4 TO 1/2 "	Compensation channel at the window head
						Soft joint under the ledger angle

1. This table should not be used for design. Each project has unique requirements.

2. No known values.

3. Depends on the structure.

3.3 Designing the Wall

The discussion describing the design of a Structural Brick Veneer wall will be divided into four parts:

1. The layout or configuration.
2. The design of the wall for code prescribed loading.
3. The design of the wall to prevent cracking of the brick.
4. The design of the connections.

In the normal process of design, all four parts are accomplished at the same time. Following the four parts, several design examples demonstrating the methods will be presented.

3.3.1 Layout or Configuration

Usually the designer begins with an architectural rendering of the wall elevations. The designer must decide on the method of isolation and the locations to react the dead load, wind and seismic loads applied to the wall.

It is difficult to describe the process for deciding the configuration of the veneer. It is part of the art of structural engineering and experience will help. Three example projects are presented to provide some insight into the issues. These represent complex applications of the Structural Brick Veneer configurations and were chosen to demonstrate the flexibility of the system. Simpler applications are more common.

Project 1

A hospital project was in the contract document phase of design. The architect's concept for the wall was brick veneer over metal studs. Unfortunately, the design had assumed only a 9-inch thickness for the wall: Three and one half inches of brick, a 1-1/2 inch cavity and 4" deep steel studs. The floor height was 14'-8"



Figure 16 Hospital - Project 1

When the project structural engineer was asked to size the steel studs, it became apparent that the 4" depth was insufficient to support the conventional veneer. Adding thickness to the exterior wall reduced building space and had a major impact on the already designed interiors and room layout. The design of the project stopped because of the problem.

Structural Brick Veneer provided a solution. However, a 4" reinforced brick wall could not span the 14'-8" as a simple span between floors. One option was to use a 5" or 6" brick to make a simple span between floors, but this did not leave enough depth for the interior studs

to support the wallboard without thickening the wall. The solution was to provide a 4" thick (specified thickness is 3.5") Structural Brick Veneer wall system supported as shown in the following Figure.

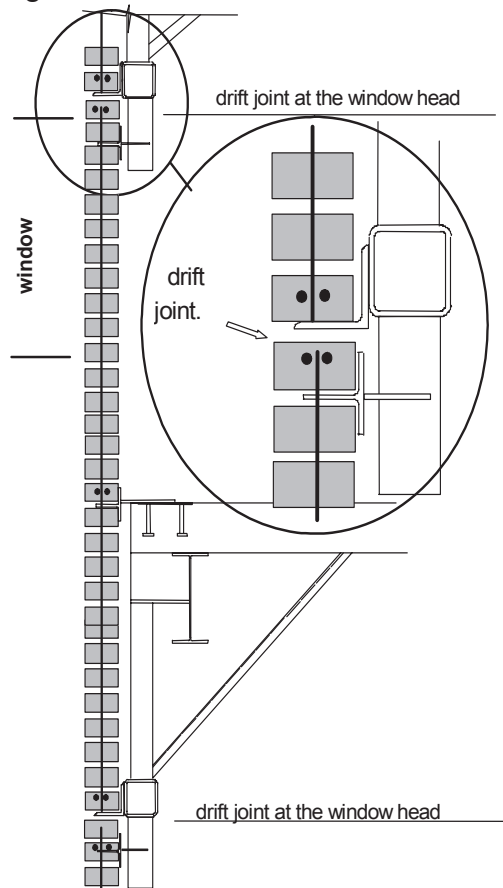


Figure 17 Project 1 – Pier Concept Between Windows

The brick masonry is supported on continuous angles located at the window head. These angles, or ledgers, are supported on a girt system suspended from hangers with kickers framing back to the underside of the slab.

Lateral bracing at the floor consists of galvanized 1/8-inch thick plate with holes for vertical reinforcement. This brace is flexible in the vertical direction and stiff in both horizontal directions.

The seismic or drift joint is located at the window head, below the ledger. The joint directly below the ledger is a caulk joint. Another lateral brace is located below the ledger. This brace is stiff perpendicular to the plane of the wall and flexible in both the horizontal in the plane of the wall and the vertical directions. This is accomplished by using a 1/8-inch thick galvanized plate with holes for the vertical reinforcement supported by two 1/8-inch thick galvanized plates welded to the bottom of the girt and welded to the plate into the brick.

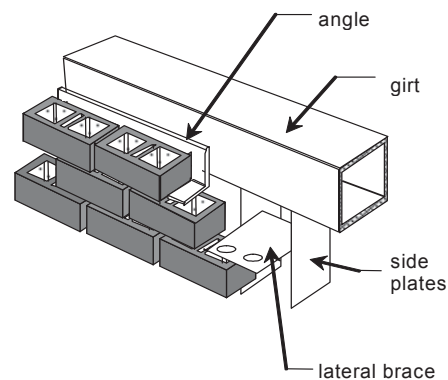


Figure 18 Lateral Connection

This configuration reduced the spans sufficiently to allow the 4" brick to resist the applied loading.

The stud wall was 26-gauge steel studs and was used only for the support of the water barrier and air barrier, and interior wallboard.



Figure 19 Drift Joint Before Ledger Installation

Dowels were welded to the ledgers, and the brick was laid by threading over the dowels. When the construction of masonry reached the head location, bars were dropped into the vertical cells and the wall solidly grouted.



Figure 20 Ledger Installation

The masonry units are nominal 4x4x12 inch with two 1-3/4 x 3-1/2 inch cells. The design f'_m was 4,000 psi .

Project 2

The second project is also a hospital addition located in an area of high seismic risk. The hospital program and site generated a geometric shape consisting of several boxy wings. A brick wall was chosen to match the old building.

The boxy nature of the floor plan made isolation for seismic lateral displacements nearly impossible with conventional veneer construction. Structural Brick Veneer was chosen because of its ability to isolate the brick veneer from the building.



Figure 21 Project 2

This Structural Brick Veneer was designed to accommodate a large lateral

seismic displacement. The building structure was a steel moment frame in one direction and a steel braced frame in the other. Because of the many corners, an unconventional approach to the isolation was used. The concept was to build a Structural Brick Veneer box in the shape of the outside wall. The box was one floor tall extending several courses above the window at the next floor. The box was then attached to the building so that it moved with each floor. The dead load support was at the floor.

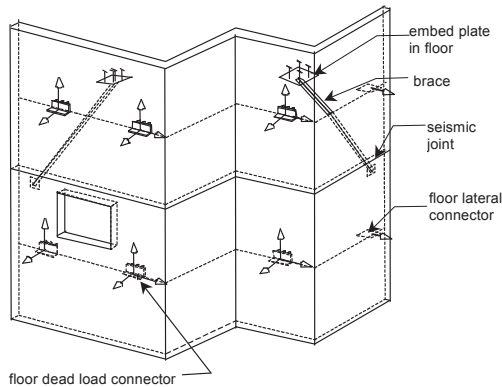


Figure 22 Hospital with Many Corners

The corners of the box resisted the wind and seismic loading perpendicular to the surfaces of the wall. The reaction of the wind and seismic load occurs in the dead load anchors as shear and overturning. Where corners were spaced too far apart to span between them, braces were added to the underside of the slab for the floor above at a sufficient distance from the corners to accommodate the differential horizontal movement of the floor by warping of the masonry.

When the building moves laterally, the rigid corners are unaffected and the

under-floor braces cause warping of the wall. The displacement perpendicular to the wall caused by the brace is at a sufficient distance from the corner to warp the wall without failure.

Continuous ledgers were not used in this situation because they would not provide adequate strength to resist the overturning forces. Consequently, the normal construction sequence needed to be modified.

The construction proceeded as follows: Beginning from the masonry below, rigid foam board was placed on the brick. Brick was then laid on top of the foam board for a height of two feet above the floor. This masonry engaged the dead load connector. At this time, the wall was grouted and allowed to cure for three days. Masonry was then laid to several courses above the window head and the process repeated for each floor. Caulk was placed over the face of the foam board that was left in the wall.

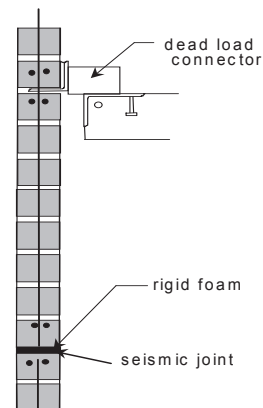


Figure 23 Temporary Rigid Foam Board Support

Project 3

A final example is a combination of laid-in-place Structural Brick Veneer and prefabricated Structural Brick Veneer or brick panels. Brick panels are similar to precast concrete panels except they are constructed of reinforced brick instead of reinforced concrete. The project was built on fill with friction piling used to support the load. The foundation design was such that $\frac{1}{2}$ inch of differential settlement was expected in 50 feet. The building frame was a combination of steel and wood. The corners could be damaged in a moderate earthquake. The typical wall elevation is shown in the following Figure 25.

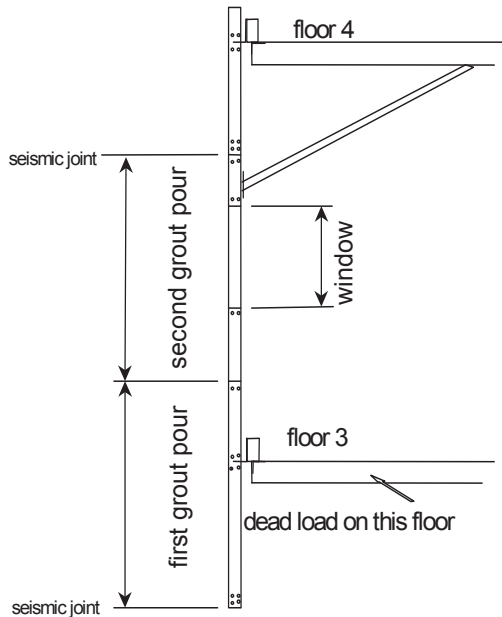


Figure 24 Construction Sequence

The design used isolated connectors instead of a ledger because, under lateral load conditions, the wall produced uplift on the floor and it was felt that the best design was to provide specific points for these forces to be reacted. The corners provide much of the needed lateral bracing, thus the number of connectors to the structure is relatively few.

The brick was a 5-inch unit. Custom shapes were provided. Additionally, the project used prefabricated Structural Brick Veneer (brick panels) where scaffolding was difficult to install or where the support of the brick was not available.

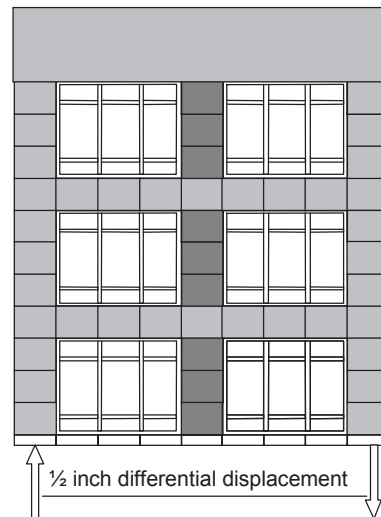


Figure 25 Building with Anticipated Differential Settlement

The solution was to use Structural Brick Veneer column elements with the dead load supported at the foundation. Lateral bracing was provided at each floor. Brick

panels were hung off the Structural Brick Veneer columns with connectors that allowed them to pivot while being stiff for loads perpendicular to the surface.

The columns were constructed of 8" brick and reinforced. They supported the entire dead load of the wall, but the lateral connectors to the building were flexible in the vertical directions so that none of the building dead load was supported on the brick. The panels were constructed of 4" brick. They were shipped 600 miles from the fabrication yard to the building location.

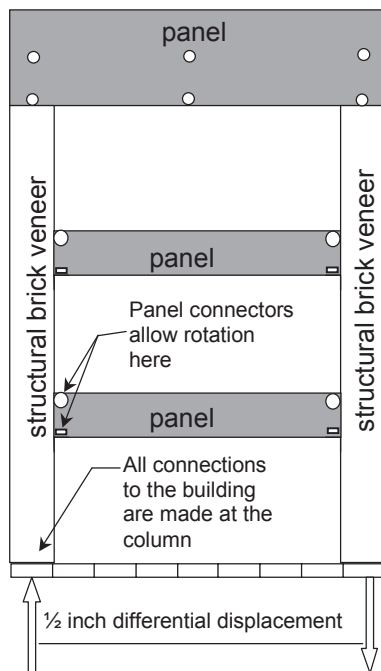


Figure 26 Concept for Settlement

3.3.2 Design for Code Loading

The reinforcement size and spacing are typically determined by conventional allowable stress design. Maximum moments usually occur at mid-span for

simple configurations or at connectors for cantilevered configurations. Both horizontal and vertical reinforcement should be provided because most walls behave as plates with moments in both directions. The design often proceeds, however, by assuming simple spans with full loading in each direction.

Maximum spacing of reinforcement should be less than 4 feet with 3 feet a common value. Typically, bar sizes are No. 3 and 4 bars in 4 inch walls and No. 3, 4 and 5 bars in 5 inch and 6 inch walls.

In areas of high shear, additional reinforcement may be required. Reinforcement should also surround connectors and openings.

3.3.3 Design for Wall Cracking

An important performance criterion is to prevent the cracking of brick. This condition usually is not a structural problem for reinforced brick masonry. But, it is an aesthetic problem. The design for this performance criterion is typically done as follows:

To prevent cracking of the brick, the moments in the horizontal direction (stiff direction) are compared to the resistance of the wall in that direction, see the following figure. Failure would involve cracking of the brick unit and the head joint. Cracking due to vertical moments would result in cracking of the horizontal bed joint which is not an aesthetic problem.

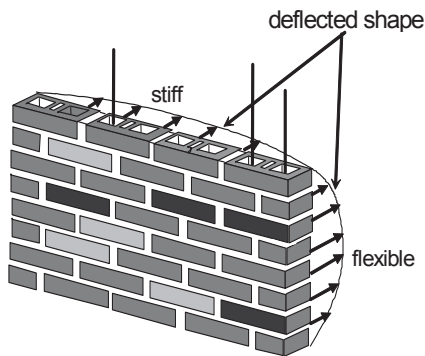


Figure 27 Stiffness of Brick Masonry

Unpublished tests and tests performed by Western States Clay Products in the early 1970's have shown that brick masonry in running bond direction is nearly 10 times as stiff as brick masonry in the vertical direction. For normal brick orientations, loads tend to run horizontally to the supports.

Most code wind loading is based on a mean return period of 50 years and this often represents a reasonable level of wind loading expected once in the life of the average building. However, in some cases, it may be appropriate to increase or decrease these levels to better match the criteria for the project.

Seismic loading would usually correspond to the moderate level earthquake. It is probably unreasonable to design to the major level earthquake to prevent an aesthetic problem.

The resistance of the wall is provided by the masonry without consideration of the reinforcement. The masonry is assumed

not cracked. Without the reinforcement, the resistance is provided by the bricks and mortar joints. It is conservative to assume that there is no capacity for the head joint to resist tension stresses, consequently all of the resistance is provided by the tensile capacity of the brick. Thus, the appropriate structural section available to resist the loading is only the brick which is normally one half the gross cross-section.

There are no national standards available to provide allowable stresses for brick in tension. The brick material supplier may have information or it may be necessary to do testing. In the absence of both, using 10% of the brick's compressive strength has generally proven conservative. Because the minimal consequences of the failure and the loading are expected to occur only once in the building life, it is recommended to use a small factor of safety on the tension capacity of the brick. A value of 1.25 has been successfully applied.

3.3.4 Design of Connections

Each configuration has a unique set of reactions for each code-required load combination. The reactions must be supported by the connectors and the structure. The connectors must be of sufficient strength and ductility to meet the capacity and ductility requirements.

Materials for Connectors

Most connectors are constructed of miscellaneous steel angles, tees and plates. For Level 1 (institutional) performance they are typically galvanized. For Level 2 (commercial) performance

they are shop painted. Stainless steel connectors are probably not warranted and there are only a few known installations.

Design Methods and Assumptions

Conventional design methods apply to the design of the connectors, except for the design of the capacity of the connector to the Structural Brick Veneer. There are several additional design considerations.

First, the connector should engage more than one brick cell. Sometimes bricks have cracks as a result of manufacturing. Concentrating loads at a single cell of the brick could, if cracked, significantly reduce the capacity. Consequently, for the design of connectors in Structural Brick Veneer, the scale of the engagement into the brick should be at least two times that usually associated with connectors engaging reinforced concrete.

Next, because of the seismic requirement for connectors, "Anchorage using flat straps embedded in concrete or masonry shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel or to assure that pullout of anchorage is not the initial failure mechanism", it is often necessary to thread reinforcement through holes placed in the steel. This may restrict the flow of grout at the connection and a recommended mitigation is to use bond beams above and/or below the connector.

Construction Tolerances

An important factor in the design of a connector is construction tolerance. Structural Brick Veneer walls are located in plane and elevation with tolerances more restrictive than the supporting structure. For example, the deviation from plumb (down the height of the building) often is limited to 1/2 inch for Structural Brick Veneer. In a steel frame at the upper floors, the tolerance on the frame is 1-1/4 inches inward and 2-1/4 inches outward. The location of the slab edge is often plus or minus 1 inch.

The elevation of the floor also varies from the planned location. There are several reasons for this. First, there is construction tolerance. Second, the floor is supported on beams that deflect due to loading. Third, there is elastic shortening of the building and for concrete buildings, there is creep and shrinkage. Again, this uncertainty of floor elevation is important to the design of the connector. These tolerances have an important design and construction impact.

The connector must be designed to accommodate these deviations in dimension. The edge of floor connector should be configured to be installable with the slab edge at either extreme of the allowed tolerance. Additionally, the strength of the connector should be adequate to support the loads with the most unfavorable combination of element locations.

The configuration of the connector is often dictated by the tolerance requirements, and the resulting load eccentricity.

ties often (to the untrained eye) appear to oversize the connections.

3.4 Design Examples

Three design examples are presented. The first is a strip spandrel configuration supported on a continuous ledger. The second is the same except the veneer is supported on separate floor dead load connectors. The third example addresses seismic displacement design at a Structural Brick Veneer corner.

3.4.1 Example 1

This example is very simple and unlikely to occur in actual practice. The designer is cautioned not to apply the methods and equations contained in this example to other applications. The purpose of the example is to demonstrate a sequence of assumptions and analysis appropriate for this simple design. Different assumptions and analysis methods should be used when the design changes.

A four-story office building with a “strip window” system serves as a common application. The story height is 12'-6", the window height is 6'-0" and the sill height is 2'-6". The wind load is 30 psf and the building is classified as seismic design category D. The wind load of 30 psf (according to most codes) is typically larger than the seismic inertia load perpendicular to the surface of the veneer and this is assumed for this example. Floor to floor drift isolation is provided by a slip joint in the window head. Warping of the glass accommodates drift at the building corners. In other words, the brick corner remains

rigid and moves with the floor to which it is attached.

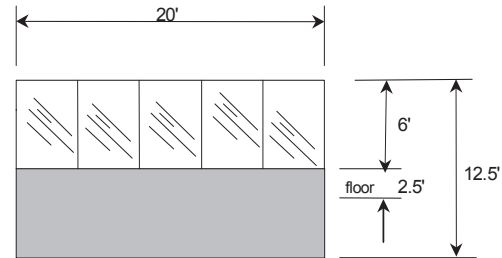


Figure 28 Strip System Example

A continuous ledger system is hung from the bottom of the spandrel beam and braced to the underside of the floor.

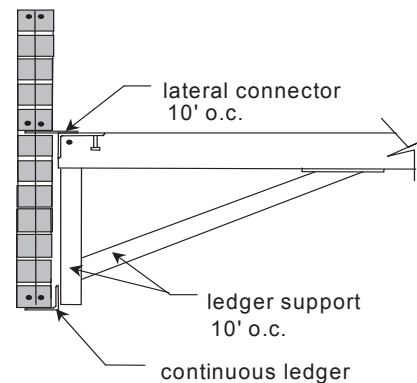


Figure 29 Support for the Wall

The hanger and brace are located at 10 feet on center. This matches the purlin spacing when the floor spans in the direction parallel to the edge of the slab. Assume 4-inch hollow brick is used. The actual thickness is 3.5 inches and the depth of the reinforcement from the surface is 1.75 inches.

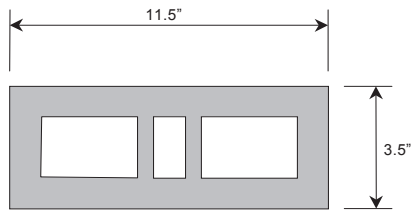


Figure 30 Brick Used in the Example

The design might proceed as follows:

The dead load on the ledger includes the weight of the brick veneer and the windows.

$$\text{Wall} = 40 \text{ psf} \times 6.5 \text{ ft} = 260 \text{ plf}$$

$$\text{Window} = 6 \text{ psf} \times 6 \text{ ft} = 36 \text{ plf}$$

$$\text{Total} = 296 \text{ plf}$$

Since the hangers are spaced at 10 feet, the hanger vertical load is:

$$296 \text{ plf} \times 10 \text{ ft} = 2960 \text{ lbs}$$

The ledger is designed to span between supports. In Structural Brick Veneer, the brick can also be used to span if shoring is used during construction. Assume the brick is used to span between supports, then try (2) #3 in a bond beam at the first course above the ledger.

The bending due to dead load is:

$$M_A = W L^2 / 8$$

$$M_A = 296 \text{ plf} \times 10^2 \text{ ft} / 8 = 3700 \text{ lb-ft}$$

The steel ratio “ ρ ” is used to calculate the capacity to resist the loading.

$$\rho = A_s / bd$$

Where A_s is the area of flexural reinforcement, b is the width or thickness of the masonry and d is the depth of the flexural element from the extreme compression fiber to the centroid of the reinforcement.

$$\rho = .22 \text{ in}^2 / (3.5 \times (78 - 4))$$

$$\rho = .00085$$

Assume the masonry modulus of elasticity is 3,000,000 psi, and the steel modulus of elasticity is 29,000,000 psi then the modular ratio between the reinforcement and the masonry is:

$$n = E_s / E_m = 9.67$$

and then

$$n\rho = .00085 \times 9.67 = .0082$$

Most codes allow a design flexural stress for grade 60 reinforcement as:

$$F_s = 24,000 \text{ psi}$$

Similarly, the allowable masonry stress is 1/3 the specified strength. If 4000 psi is specified, then the allowable flexural masonry compression stress is:

$$F_b = 4000 / 3 = 1333 \text{ psi}$$

By summing forces in the direction along the flexural element, the following

expression for the location of the neutral axis can be obtained:

$$k = [(n\rho)^2 + 2n\rho]^{1/2} - n\rho$$

$$k = .12$$

A convenient term “j” is defined as:

$$j = (1 - k / 3) = .96$$

The moment capacity limited by the allowable reinforcement stress is defined as:

$$M_t = A_s j d F_s$$

Thus:

$$M_t = .22 \times .96 \times 74 \times 24000 / 12$$

$$M_t = 31,200 \text{ lb-ft}$$

The moment capacity limited by the allowable compression stress in the masonry is defined as:

$$M_c = bd^2kj F_b / 2$$

$$M_c = 3.5 \times 74^2 \times .12 \times .96 \times 1333 / (2 \times 12)$$

$$M_c = 122,600 \text{ lb-ft}$$

Since both M_t and M_c are greater than the applied moment, M_a , the two #3 bars provide more than adequate vertical support between ledger hangers.

If shoring is provided, a similar calculation would show that the vertical support could be extended to 20 feet or more and the weight of the wall reacted at the building columns.

However, as will be seen later, the lateral support (for wind and seismic loading) will need to be 10 foot. Thus, the hanger could be spaced at 20', but the braces back to the underside of the slab will need to be at 10'.

The wind load on the wall is 30 psf. This load acts perpendicular to the surface of the brick and the window.

Assume floor connections are spaced at 10 foot at the same location as the lateral support of the ledger.

The floor reaction is found by summing moments about the ledger:

The reaction of the window at the sill (top of the brick) is:

$$R_w = 3 \text{ ft} \times 30 \text{ psf} = 90 \text{ lb/ft}$$

The resulting moment about the ledger is:

$$M_w = 90 \text{ lb/ft} \times 6.5 \text{ ft} = 585 \text{ lb-ft/ft}$$

The moment about the ledger due to the wind loading on the brick is:

$$M_b = 30 \text{ psf} \times 6.5 \text{ ft} \times 6.5 \text{ ft} / 2$$

$$M_b = 634 \text{ lb-ft/ft}$$

The reaction resists the moments and is calculated as:

$$R_F = [M_b + M_w] / 4$$

$$R_F = [634 + 585] / 4$$

$$R_F = 304.7 \text{ lb-ft/ft}$$

Since the spacing of the connectors is at 10 feet on center, the connector reaction is:

$$R_F = 3047 \text{ lbs}$$

By summing forces perpendicular to the surface of the brick, the ledger lateral reaction is:

$$R_L = 12.5 \text{ ft} \times 30 \text{ psf} \times 10 \text{ ft} - R_F$$

$$R_L = 703 \text{ lbs}$$

Drawing the shear and moment diagrams reveals that the maximum moment occurs at the floor and is equal to the moment on the brick due to the window sill load plus the moment on the brick due to the wind on the brick:

$$M_w = 30 \text{ psf} \times 3 \text{ ft} \times 2.5 \text{ ft}$$

$$M_w = 225 \text{ lb-ft/ft}$$

$$M_b = 30 \text{ psf} \times 2.5 \text{ ft} \times 2.5 \text{ ft} / 2$$

$$M_b = 93.75 \text{ lb-ft/ft}$$

$$M_a = M_w + M_b$$

Thus:

$$M_a = 318.75 \text{ lb-ft/ft}$$

and over the 10 foot spacing becomes:

$$M_a = 3187.5 \text{ lb-ft}$$

An estimate of the required reinforcement can be obtained as follows:

$$A_S \cong M_a / [.9 F_s d]$$

$$A_S \cong (3187.5 \times 12) / [.9 \times 24,000 \times 1.33 \times 1.75]$$

$$A_S \cong .75 \text{ in}^2$$

Where the design depth is one-half the wall thickness (1.75 inches), and the stress has been increased by 1/3 for wind short duration loading.

Try (4) #4 bars. The brick module is 6 inches. The code allows an effective width for bending of six times the thickness of the wall. The width for design can be obtained by laying out the bars in the brick cells:

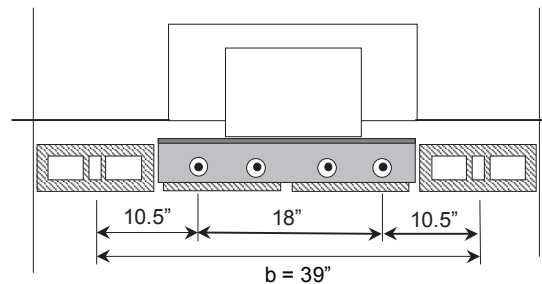


Figure 31 Plan View of Connector

And:

$$\rho = A_S / bd$$

$$\rho = 4 \times .2 \text{ in}^2 / (1.75 \text{ in} \times 39 \text{ in})$$

$$\rho = .0117$$

$$n\rho = .113, k = .37, j = .87$$

The moment limited by the allowable stress in the reinforcement is:

STRUCTURAL BRICK VENEER

$$M_t = A_s j d F_s$$

$$M_t = .8 \times .87 \times 1.75 \times 24,000 \times 1.33 / 12$$

$$M_t = 3250 \text{ lb-ft}$$

The moment limited by the allowable stress in the masonry is:

$$M_c = b d^2 k j F_b / 2$$

$$M_c = 39 \times 1.75^2 \times .37 \times .87 \times 4000 \times 1.33 / (3 \times 2 \times 12)$$

$$M_c = 2840 \text{ lb-ft} \quad \text{n.g.}$$

Compression at the floor in the brick is often the limiting factor.

Add (2) #4 bars for a total of 6 bars. Then

$$b = 6 \times 3.5 + 5 \times 6 = 51 \text{ in.}$$

$$n\rho = (9.67 \times 6 \times .2) / (1.75 \times 51)$$

$$n\rho = .13$$

$$k = .40 \quad j = .87$$

$$M_t = 4870 \text{ lb-ft} \quad \text{ok}$$

$$M_c = 4025 \text{ lb-ft} \quad \text{ok}$$

The wind loading must span horizontally 10 feet to the vertical connector strips. Assume the window head reaction is reacted by the lateral bracing and the ledger, and the window sill reaction is reacted by the brick, then the moment in the brick is the moment caused by the window and the moment caused by the wind load on the brick.

$$M_w = 30 \text{ psf} \times 3 \text{ ft} \times 10^2 / 8$$

$$M_b = 30 \text{ psf} \times 6.5 \text{ ft} \times 10^2 / 8$$

$$M_a = M_w + M_b$$

$$M_a = 3562 \text{ lb-ft}$$

Assume (5) bond beams with (2) #3 in each, then:

$$A_s = 1.10 \text{ in}^2$$

$$\rho = 1.10 / (78 \times 1.75)$$

$$\rho = .00805$$

$$n\rho = .078, \quad k = .32 \quad j = .89$$

$$M_t = 1.10 \times .89 \times 1.75 \times 24000 \times 1.33 / 12$$

$$M_t = 4567 \text{ lb-ft} \quad \text{ok}$$

$$M_c = 78 \times 1.75^2 \times .32 \times .89 \times 1333 \times 1.33 / (2 \times 12)$$

$$M_c = 5038 \text{ lb-ft} \quad \text{ok}$$

The condition of dead load plus wind is satisfied by inspection, but can be checked as follows:

$$\begin{aligned} & [M_a / (M_t \text{ or } M_c)]_{\text{dead}} \\ & + [M_a / (M_t \text{ or } M_c)]_{\text{wind}} \\ & < 1.0 \end{aligned}$$

$$\begin{aligned} & 3700 / 3120 \\ & + 3562 / 4567 \\ & = .89 < 1.0 \end{aligned}$$

The condition of inertia seismic and dead load is also satisfied by inspection since the inertia seismic load was assumed less than the 30 psf wind load. This conclusion by inspection can easily be made when designing the veneer, but should not be so easily made when designing connectors. This is because most seismic codes have special load factors and additional criteria for seismic connector design that are not typical for the wind design of a connector.

The final check is for cracking of the brick.

Assume a service load of 15 psf. The value selected should relate to the design life and the performance criteria selected for the project.

The uniform horizontal loading of the strip between the connectors is a result of the wind load acting on the windows and brick. The moment is estimated as:

$$M_w = 15 \text{ psf} \times 3 \text{ ft} \times 10^2 / 8$$

$$M_w = 562.5 \text{ lb-ft}$$

$$M_b = 15 \text{ psf} \times 6.5 \text{ ft} \times 10^2 / 8$$

$$M_b = 1218.7 \text{ lb-ft}$$

$$M_a = M_w + M_b$$

$$M_a = 1782 \text{ lb-ft}$$

The section available to resist the moment is estimated as half the wall height since half of that height is head joints.

$$I = b t^3 / 12$$

$$I = (78/2) \times 3.5^3 / 12 = 139.3 \text{ in}^4$$

The stress is:

$$\sigma = Mc/I$$

$$\sigma = 1782 \times 12 \times 1.75 / 139.3$$

$$\sigma = 269 \text{ psi}$$

The brick unit should have a tensile strength in excess of 269 psi times the factor of safety. If 1.25 is used, then the tensile strength needs to exceed 335 psi.

According to an unpublished test and experience, to achieve an f'_m of 4000 psi the brick strength probably exceeds 10,000 psi. Using the 10% rule of thumb, the expected tensile strength of the brick is 1000 psi.

A simple lateral connector at the floor is shown in the following figure. It consists of an embedded WT 4 x 5 with a coupler welded to the flange for the attachment of a rod.

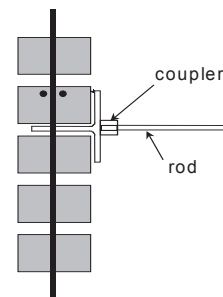


Figure 32 Simple Lateral Connector

The length of the tee will need to be determined based on the loads applied. To start, assume the tee is 22 inches long. The tee contains four 1-inch diameter holes for reinforcement placement. The capacity of the connector can be estimated by using a shear cone.

Assuming the shear cone originates at the bar then the height of the cone is:

$$h = 1.75 + .25 = 2.0 \text{ inches}$$

Assuming a 20 degree slope at the edge of the cone, the approximate surface area of the cone is:

$$A = \text{Length} \times \text{width} \times 2$$

$$A = \{2.0 / \cos 20^\circ\} \times 22 \times 2$$

The ends of the cone are neglected, but can be added if the capacity is insufficient.

$$A = 94 \text{ square inches}$$

Assuming the shear allowable is the same as the maximum shear allowable for beams of 50 psi (the masonry design strength exceeds 2500 psi and the 1/3 stress increase is used) then the allowable for the connector for pull-out is:

$$P = A \times 50 \text{ psi} \times 1.33$$

$$P = 94 \times 50 \times 1.33 = 6250 \text{ Lbs}$$

Another failure mode is the connector breaking the bond between the steel and masonry. The area of contact is:

$$A = \text{Length} \times \text{width} \times 2$$

$$A = 2.75 \times 22 \times 2 = 121 \text{ in}^2$$

The bond strength is assumed to be 60 psi plus the 1/3 stress increase. Thus, the capacity of the connector is estimated as:

$$P = A \times 60 \times 1.33$$

$$P = 121 \times 60 \times 1.33 = 9656 \text{ lbs}$$

The shear friction check results in:

$$A_v = V_u / \phi F_y$$

or

$$V_u = \phi A_v F_y$$

$$V_u = .4 \times 4 \times 2 \times 60,000$$

$$V_u = 19,200 \text{ lbs}$$

$$V = V_u / 2 = 9600 \text{ lbs}$$

The lowest allowable load is 6250 lbs compared to the applied wind load of 3047 lbs. A 10-inch long anchor would likely supply sufficient capacity, but only two of the six bars would be engaged making the design for ductility questionable.

It is assumed that the code level seismic inertial force is .315 times the force of gravity. The working stress design inertia loading is .315/1.4 or .225 times gravity. The brick weight is approximately 40 psf resulting in a surface inertial load of 9 psf and a reaction of $3047 \times 9 / 30$ or 914 lbs. The connector factor of safety exceeds 7 for code level seismic loads. This satisfies the additional seismic criteria for connectors by inspection.

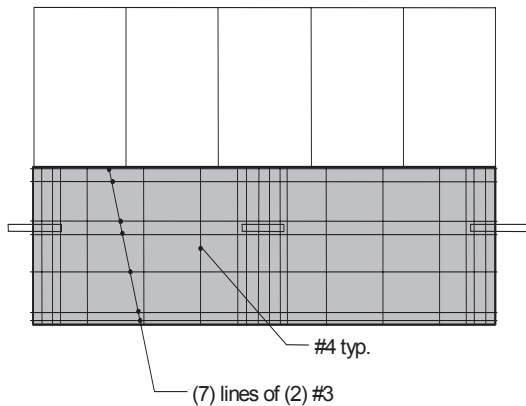


Figure 33 Final Design

Limited testing of connectors has been conducted. One recent test conducted by Western States Clay Products Association validated the design methods for the connection shown above. Other tests specific to special connections on projects have been conducted. They are unpublished, but also verify the above methods.

3.4.2 Example 2

The same strip window system is used for Example 2 except that the dead load support is now on separated connectors located at the floor. Shoring will be used to construct the wall. The design example is for the in-place condition only. In actual practice, the designer of the Structural Brick Veneer would design the system for the shoring conditions as well as the in-place condition.

The following figure presents the new layout of the connections.

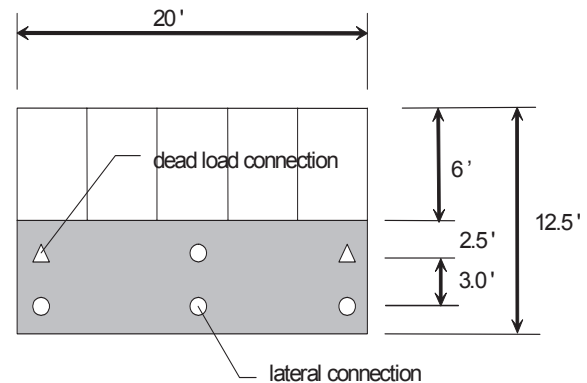


Figure 34 Location of Connectors

The veneer is assumed to have a vertical expansion joint at 20 feet on center. From the previous example, the horizontal span of the brick is limited to less than 10 feet. Thus, a lateral anchor at the floor and a brace will be required at the mid point of the panel.

The design dead load, after the wall is constructed, is the same as the previous example.

Placing the dead load anchors away from the columns will avoid interference with the building structure and make the installation easier. Assume the dead load anchors are placed at 2 feet from the column line. A vertical expansion joint is assumed at the column. The 2 feet dimension was chosen as the center of the head joint so that the connector is centered on the head joint and an even number of bars will pass through the connector. If an odd number of bars passes through the connector, then the anchor would be located at the center of a cell. Two feet three inches or

two feet nine inches from the expansion joint would be examples.

The reaction load on the dead load connector is:

$$R = w \times L / 2$$

$$R = 296 \times 20 \text{ ft} / 2 = 2960 \text{ lbs}$$

The maximum moment occurs at the mid span and is:

$$M_a = R \times \{L/2 - s\} - W \times \{L/2\}^2 / 2$$

$$M_a = 2960 \times (10 - 2) - 296 \times 10^2 / 2$$

$$M_a = 8880 \text{ lb-ft}$$

From the previous example, the wall can support this load.

The bending out of plane on the wall due to wind is assumed reacted by braces one foot above the bottom of the brick and at the floor. The reaction at the floor can be found by summing moments about the location of the brace as follows:

$$M_w = 30 \text{ psf} \times 3 \times [5.5 - 1] \text{ ft}$$

$$M_w = 405 \text{ lb-ft/ft}$$

$$M_b = 30 \text{ psf} \times [5.5^2 / 2 - 1^2 / 2] \text{ ft}^2$$

$$M_b = 438.75 \text{ lb-ft/ft}$$

$$R_{\text{Floor}} = [M_w + M_b] / 3$$

$$R_{\text{Floor}} = 281 \text{ lbs/ft}$$

Summing forces perpendicular to the veneer results in the horizontal brace force:

$$R_{\text{brace}} = 30 \times 12.5 - 281 = 94 \text{ lb/ft}$$

The maximum moment again occurs at the floor line and is:

$$M_a = 30 \times 3 \times 2.5 + [30 \times 2.5^2] / 2$$

$$M_a = 318.8 \text{ ft-lb/ft}$$

The contributing length of veneer for the dead load connector is about 6 feet consisting of the 2 feet of cantilever and half the distance to the center connector. The design moment is:

$$M_a = 318 \times 6 = 1908 \text{ lb-ft}$$

The layout of the connector at the column is shown in the following figure and results in 5 contributing #4 bars.

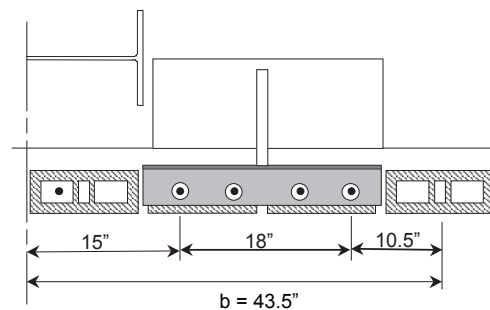


Figure 35 Plan View of Connector

Thus:

$$b = 43.5 \text{ in.}$$

$$A_s = 1.0 \text{ in}^2$$

$$\rho = 1.0 / (43.5 \times 1.75)$$

$$\rho = .013$$

$$n\rho = .127, \quad k = .39 \quad j = .87$$

$$M_t = 1.0 \times .87 \times 1.75 \times 24000 \times 1.33 / 12$$

$$M_t = 4060 \text{ lb-ft} \quad \text{ok}$$

$$M_c = 43.5 \times 1.75^2 \times .39 \times .87 \times 1333 \times 1.33 / (2 \times 12)$$

$$M_c = 3347 \text{ lb-ft} \quad \text{ok}$$

But, this may not be the only out-of-plane moment in the masonry. Depending on the connector design, there could also be a significant moment added into the masonry as a result of dead load eccentricity

The connector will be designed next. Assume a configuration as shown in the following figure:

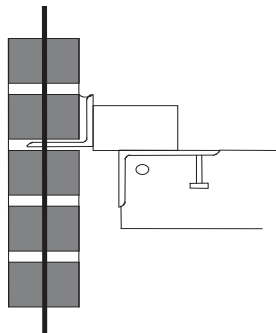


Figure 36 Dead Load Connector

The vertical reaction is 2960 lbs and the wind reaction is 6 feet x 281 lb/ft = 1686 lbs.

The eccentricity of the dead load is important to the design. It is calculated assuming an edge of slab tolerance of 1 inch as shown in the following figure.

$$e = 3.5/2 + 1 \frac{1}{2} + 1 \text{ (tol.)} + 3.0/2$$

$$e = 5.75 \text{ in.}$$

The resulting eccentric moment can be reacted either in the brick veneer or in the floor slab or by both. When the wall is shored, the connector installed and the shoring removed, the eccentric moment is reacted by both the wall and the floor slab. The amount of moment in each is very difficult to determine and will depend on the stiffness of the shoring as well as the manner in which the wall is built.

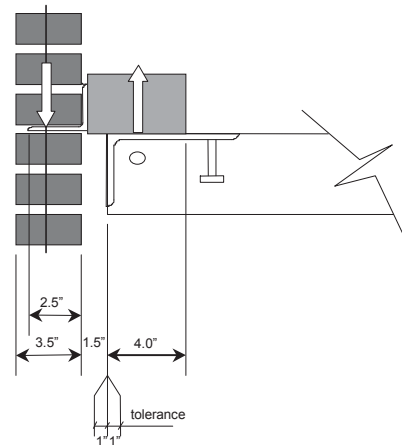


Figure 37 Dead Load Moment

A conservative approach to the design is to design assuming all of the moment is in the brick veneer and then assume all of the moment is in the floor. We will first assume the moment is in the brick veneer.

Assume the connector design is controlled by the combined dead load and wind condition. From the previous example, the dead load on the anchor is 2960 lbs.

The moment is in the brick wall, thus:

$$M_a = 2960 \times 5.75 \text{ in}$$

$$M_a = 17,202 \text{ lb-in}$$

This moment should be combined with the wind moment resulting in:

$$M_a = 17,202 + 1908 \times 12$$

$$M_a = 40,100 \text{ lb-in or } 3340 \text{ lb-ft}$$

The previous design using (5) #4 bars provides adequate resistance.

Now, check the local capacity of the connector to transfer the load into the veneer. Assume the moment is inserted into the veneer wall by forces consisting of shear on the horizontal leg of the angle and bearing on the vertical leg of the angle. If a triangular distribution of bearing on the vertical leg is assumed, the force is:

$$P = 17,202 / (6 \times 2/3)$$

$$P = 4,255 \text{ lbs}$$

Note that the moment due to wind is not included. The wind load moment is already in the veneer. The reaction of the wind at the floor must, however, be added to the horizontal leg of the angle.

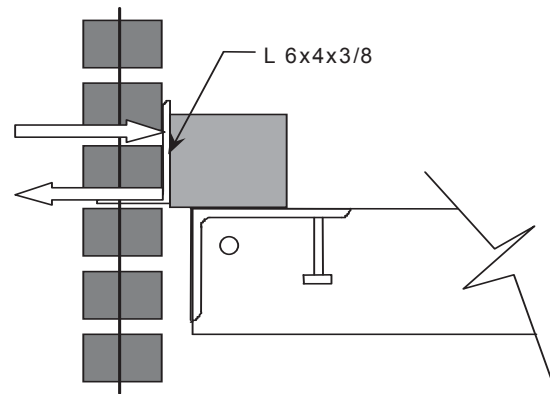


Figure 38 Resisting Moment

$$P = 4255 \text{ lb} + 281 \text{ lb/ft} \times 6 \text{ ft}$$

$$P = 5942 \text{ lbs}$$

Assume the failure is in the bond between the horizontal leg of the angle and the masonry. Using 60 psi shear allowable and 2.5 inch contact length, both top and bottom, the required length of contact is:

$$L = 5942 / (2.5 \times 2 \times 60 \times 1.33)$$

$$L = 14.8 \text{ in}$$

Or, using the cone pull-out check the length becomes:

$$P = 2 \times (d / \cos 20^\circ) \times L \times 50 \times 1.33$$

$$L = P / (141.5 \times d)$$

$$L = 5942 / (141.5 \times 2.0)$$

$$L = 21 \text{ in.}$$

Therefore use a 24 inch long angle.

The shear friction method can also be used:

$$A_v = V_u / \phi F_y$$

or

$$V_u = \phi A_v F_y$$

$$V_u = .4 \times 6 \times 2 \times 60,000$$

$$V_u = 28,800 \text{ lbs}$$

$$P = 28,800 / 2 = 14,400 \text{ lbs ok}$$

The 2003 International Building Code requires the fasteners and body of the connector to be designed to equation 9.6.1.3-2 which is a maximum value. Lower values are likely for many structures using equation 9.6.1.3-1.

$$F_p = 1.6 S_{DS} I_p W_p$$

$$\text{Using } S_{DS} = .75 \text{ (a high value)}$$

$$F_p = 1.65 \times .75 \times 1.0 \times W_p$$

$$F_p = 1.24 \times 40 \text{ psf} = 49.6 \text{ psf}$$

the design level is:

$$F = 49.6 \text{ psf} / 1.4 = 36 \text{ psf.}$$

This is higher than the design wind load of 30 psf and controls.

$$P = 4255 \text{ lb} + 281 \times \{36/30\} \times 6$$

$$P = 6278 \text{ lbs}$$

$$L = 6278 / (141.5 \times 2.0)$$

$$L = 22 \text{ in.}$$

Use a 24 inch angle.

The bearing of the brick wall on the horizontal leg of the angle is seldom limiting in design and is satisfied here by inspection.

Now check assuming the moment is in the floor. By inspection, the brick veneer is adequate. The design of the plate, welds and embedded plate is all that is required. Assume the worst situation of the tolerance for the edge of slab results is 3 inches of connection between the slab embed and the plate.

The analysis is the same as would be performed for a precast concrete panel or other wall system. Examples can be found in many references.

The design results in a ½ inch thick vertical plate. The weld on the vertical leg of the angle should also be a 3/8 inch fillet 4 inches long on both sides. And the weld on the plate to the embed should be a 3/8 inch fillet 3 inches long on both sides.

Notice that the size of the weld exceeds the thickness of the plate. This is because the design factors of safety for the weld loaded by seismic forces was assumed larger than that required for the body of the connection.

The 24 inch angle is a conservative size since both the brick and slab connection are designed for the moment. If a more detailed analysis were made, the length of the angle could probably be reduced. The cost savings, however, would likely not justify the reduction in capacity.

3.4.3 Example 3

The final example will demonstrate the design of a corner for seismic isolation. Assume a 12'-6" story height. The corner is constructed of Structural Brick Veneer with a width of 10 feet from the corner. Given that the elastic seismic drift is .31 inch, the building is a specially reinforced concrete moment frame with a C_d of 5.5, and assuming the building is a hospital with the corner above the emergency entrance, the maximum expected floor drift is:

$$\Delta = 5.5 \times .31 = 1.7 \text{ inches}$$

The corner can support part of the wind loading, but a lateral connector will be needed 10 feet away from the corner.

Since the brick is 10 times stiffer in the running bond direction, most of the resistance to the movement is reacted at the corner.

The analysis of this corner displacement is complex. It can be visualized by considering a flat rectangular panel of veneer 12'-6" by 10 feet laid flat horizontally on the floor with anchors at the corners. The analogous displacement is to lifting up one of the four corners of the flat panel while the other three are held to the floor.

The load deflection calculation for this condition is complex and includes bending in two axis and torsion. And, again the veneer is not isotropic.

Making a simplifying assumption that half the height participates with bending about the stiff direction only, then the

load required to deflect the wall 1.7 inches is calculated as follows:

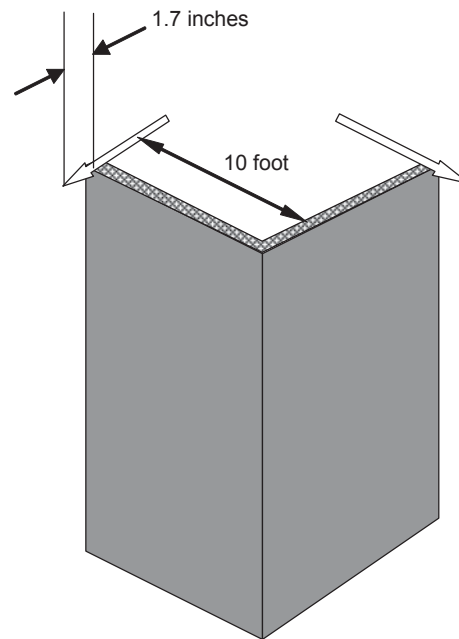


Figure 39 Warping Corner

$$\Delta = PL^3 / (3 \times E_m \times I)$$

The head joints are assumed cracked so the contribution of the wall is half.

$$I = 1/12 \times \{(6' - 3'') / 2\} \times 3.5^3$$

$$I = 134 \text{ in}^4$$

$$\Delta = 1.7 \text{ inches}$$

$$P = 1.7 \times 3 \times 3,000,000 \times 134 / (120^3)$$

$$P = 1186 \text{ lbs}$$

The resulting stress in the brick is:

$$\sigma = M c / I$$
$$\sigma = 1186 \times 120 \times 1.75 / 134$$

$$\sigma = 1860 \text{ psi}$$

If the entire wall height were assumed to resist the displacement, the stress would not change because the stiffness increases in proportion. Because the stress exceeds the likely tensile capacity of the brick, the wall will crack.

This is acceptable according to our criteria provided the panel remains intact. Notice that the wall will likely remain uncracked for half the ultimate displacement resulting in no damage.

Reinforcement will be required to hold the panel together.

Sufficient reinforcement must be provided to assure ductile behavior. The cracking moment is:

$$M = (1186 \times 1000 \text{ psi}) / 1860 \times 10 \times 12$$

$$M = 76,500 \text{ lb-in}$$

Where the tensile capacity of the brick is assumed to be 1000 psi.

The estimate of the reinforcement is:

$$A_s = 76,500 / (60,000 \times 1.75)$$

$$A_s = .72 \text{ in}^2$$

Four bond beams with two #3 bars will be sufficient. Notice that 60,000 psi was used for the steel stress. The assumed

yield strength should be compared to the ultimate cracking strength since this part of the design is for ultimate seismic.

4.0 Specification

The Structural Brick Veneer System consists of several components that must be specified and detailed. The following discussion should help in this process.

4.1 Quality Control and Assurance

According to the International Building Code and ACI 530-02/ASCE 5-02/TMS 402-02 *Building Code Requirements for Masonry Structure*, the designer needs to specify the required inspections and quality control tests. Structural Brick Veneer projects are designed either in accordance with Chapter 2 or Chapter 3 of ACI 530-02/ASCE 5-02/TMS 402-02 or Sections 2106, 2107 or 2108 of the International Code.

The International Code requires inspections at Level 1 (Table 1704.5.1) for non-essential facilities and Level 2 (Table 1704.5.3) for essential facilities. These tables identify activities requiring full time and periodic inspection during construction.

It is recommended that prism tests for each 5000 square foot of wall be performed in accordance with Level 3 quality assurance (Table 1.15.3) in ACI 530-02/ASCE 5-02/TMS 402-02. The verification of f'_m can be either by prism test or the unit strength method.

4.2 Masonry

Generally, brick is selected for its color, texture, and size. The most common brick specified is ASTM C 652 Hollow Brick. More detailed information about the hollow brick can be found at:

http://www.bia.org/html/frmset_thnt.htm,
Technical Note 41 Hollow Brick
Masonry.

Most Structural Brick Veneers are constructed of bricks with nominal thickness of 4, 5 or 6 inch.

Mortar should be Type S, Portland cement, hydrated lime, and sand. Type S mortar exhibits higher flexural bond strength while providing sufficient compressive strength. Type M mortar is generally too stiff at the time of laying to result in good bond, and thus may leak more and is harder to clean.

The durability of the wall is highly influenced by the quality of the mortar joints. Care should be taken to ensure that dense joints are achieved. Joints should be tooled to a concave or "V" finish to densify the mortar surface and improve bond between the mortar and the brick. If raked joints are desired, a "deep V" may achieve the effect. Simple raked joints must be tooled after raking.

Grout is the material placed in the cells of the brick. The proportions are similar to mortar, except that sufficient water has been added to provide a fluid consistency. Grout should be poured in brick masonry with a slump exceeding 10 inches.



Figure 40 Fluid Grout

The excess water in the grout is absorbed into the brick before hydration. Reconsolidation of the grout is required to remove the voids left by the water absorbed into the brick. The specification should require reconsolidation or the additive Grout Aid.

It is recommended to add Sika Grout Aid to all installations. This is a proprietary product, but the only one known to be specifically designed for addition to grout. More information can be obtained at:

http://www.sikaconstruction.com/con/con-admixture_in_con-prod-category-ga.htm

Sika Grout Aid is a balanced blend of expanding, retarding and water-reducing agents for Portland cement grouts. It provides a slow, controlled expansion prior to the grout hardening.

Other manufacturers claim equivalence, but often lack test data to support product performance. In structural brick

masonry, the important property of the grout is to fill all of the space and voids, and to make the connection between the reinforcement and the brick unit. The strength is of secondary importance, since the brick unit compression strengths typically exceed 8000 psi.

If an additive other than the Sika Grout Aid is used, a test should be performed to assure filling of the space and bonding to the units and reinforcement. This likely means the preparing of a grout sample panel and cutting it after curing to visually confirm the performance.

The reinforcement is usually specified as ASTM A 615 Grade 60. Deformed bars are required and sizes are limited by the thickness of the wall. Generally, bars larger than No. 5 are not used. When reinforcement is to be welded, ASTM A706 bars should be specified. It is slightly more expensive but it can be welded without becoming brittle. ASTM A706 bars can be identified by a "W" mark on the bar.

Joint Reinforcement is not recommended for Level 1 projects. Experience with galvanized metal in masonry on the exterior walls indicates a life of 30 years or less.

Reinforcement bars grouted into bond beams will have an expected life greater than 30 years.

4.3 Steel for Connectors

Connectors can be designed and constructed in many forms. Most are made from ASTM A 36 structural steel angles, plate, rods, channels, tees and

other available shapes. Connectors may be shop-painted for Level 2 (commercial) installations. Connectors for Level 1 (institutional) installations should be hot-dip galvanized to meet ASTM A 123 Grade 65 requirements. Connectors that are not protected can possibly corrode, stain the brick, and ultimately fail to resist the loading. Damage to galvanized coatings by welding or other field installation practices should be repaired using cold galvanizing compounds.

The use of 300 series stainless steel is generally not necessary and may be subject to galvanic corrosion if placed in contact with galvanized steel.

4.4 Flashing/Weeps

Continuous flashing is necessary for the removal of water that enters the cavity space. In masonry, water enters the surface of the wall and then gravity pulls it downward. When the masonry is interrupted by openings that provide a horizontal discontinuity, flashing is required to intercept the water and direct it out of the building.

Flashing material for the Structural Brick Veneer System is the same as the materials used in other types of brick wall construction.

Pea gravel, proprietary meshes or screens above the flashing will help to prevent mortar droppings from clogging weep holes. Weep holes should be spaced no more than 32 inches apart. Open head joint weep holes are recommended. Weep tubes and cotton wicks often fail to function when they become clogged or damaged. Screens

in open head joint weeps can be used to deter insect infestations.

It is important that the flashing extend through the thickness of the wall in order to intercept the flow. Whether or not flashing should protrude from the wall is controversial. Since the flow of water on the surface of the wall is in all directions, a flashing that protrudes from the surface intercepts the flow at caulk joint. This is the weakest point for water to penetrate the wall.

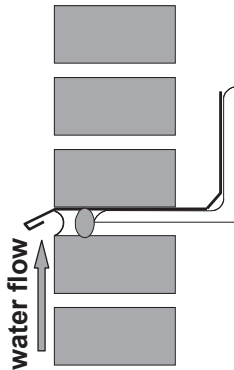


Figure 41 Protruding Flashing Detail

Flashing not protruding will allow the water to pass over the weak point. However, water intercepted by the flashing from the interior and exiting at the weep holes above the flashing may have a better chance of re-entering the building.

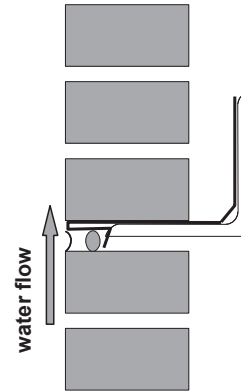


Figure 42 Flush Flashing Detail

4.5 Sealants

Sealants provide the first line of protection against rain intrusion into the system. Placement of sealants should be accomplished in conformance with the manufacturer's recommendations. Joints should be properly prepared, cleaned with solvent, primed for adhesion and backed with a backer rod.

Sealant compatibility tests (peel test) conforming to ASTM C 794 should be conducted for each type of brick unit used and all other materials in direct contact with the sealant, including flashing. The compatibility tests should also be conducted on treated (sealed) brick, unless the treatment is to be done after sealing. The compatibility tests are important to assure bond between the sealant and the brick.

4.6 Water Repellents

Water repellents are desirable for a variety of reasons. The principal reason is to limit water ingress due to wind-driven

rain. By reducing the water that enters the wall, the repellent provides several benefits, both aesthetic and related to engineering performance that would justify their use. Some of these attributes are:

1. Efflorescence control.
2. Reduction of algae growth as a result of the absence of moisture in the pores and capillaries of the masonry.
3. Atmospheric pollutants that are dissolved in precipitation are not absorbed into the masonry.
4. Freeze-thaw damage is reduced when there is no water in the substrate to freeze.
5. Thermal efficiencies of the masonry wall are maintained since water is a good thermal conductor.

Although a number of technologies have been utilized in the formation of masonry water repellents, silane/siloxane monomer and polymer blends have a proven track record. In the Pacific Northwest where there are heavy wind-driven rains, Fabrishield 761 silane/siloxane water repellent has been proven over time to be an excellent clear water repellent. These types of water repellents are formulated to penetrate into the pores and capillaries of the masonry assembly where they react with the moisture and alkalinity and actually chemically fuse onto the mineral material in the linings of the pores and capillaries. During this chemical bonding

process, a conversion occurs in the repellent material that fully develops the hydro phobic (water repellent) properties.

An additional benefit to the manner in which these repellents function is that they tend to bond along the pore linings rather than build films across the pore openings. This ensures that sufficient vapor transmission can occur to maintain “breathability” in the assembly. Repellent manufacturers state a repellent life of up to 10 years, but there are numerous projects in the Pacific Northwest that have never had to have a second application.

One of the most controversial topics nationally is when and if water repellent should be used for brick construction. As outlined above, there are many positive job results obtained by properly applied clear water repellent. While most experts acknowledge the positive attributes, the primary concern is the severe problems that can occur if an improper repellent is applied. Most western states brick manufacturers are knowledgeable about the issues and can recommend which repellents will work with their products. However, the repellent manufacturer must warrantee the performance of the repellent in service. The performance problems arise when other repellent salesmen (and there are many) convince the specifier that their product is equal to the specified repellent and will save money and the “equal” product is then used. Our advice is this: DO NOT SUBSTITUTE! Demand past successful performance of the proposed coating on the specific manufacturer’s brick, for a

number of years, under the same type of exposure. Demand that the repellent manufacturer's application instructions be followed.

4.7 Backup Wall

Structural Brick Veneer installations do not require a separate backup wall. A backup wall is sometimes constructed as a second air and water vapor barrier. Many Level 2 (commercial) buildings do not require the extra water and air protection since a well-designed drainage wall is often sufficient.

If a back-up wall is used, many options are available including metal studs with sheathing. The studs are typically not designed for the wind loading since the brick wythe is structurally designed to resist the wind.

4.8 Cavity

When a backup wall is provided, the cavity or air space behind the brick can be any width. The cavity acts to provide a buffer for wind-driven rain and allows water that penetrates the Structural Brick Veneer to migrate down the backside of the brick without migrating across the cavity space. Connectors should be designed and constructed to direct any of this water toward the backside of the brick instead of into the building.

The cavity should be kept clear of any obstructions that might allow water to bridge across. Mortar droppings should be prevented from falling into the cavity. Construction tolerance on the cavity width should be limited to $\pm 1/2$ inch. At

each floor, fire safing is required to stop smoke and heat from moving between floors. The safing can act as a bridge for water to enter the building. Flashing over the safing should be provided.

4.9 Expansion Joints

Expansion joints need to be provided at various strategic locations in the Structural Brick Veneer wall system. Expansion joint placement is dictated by two considerations. First, expansion joints may be provided in locations where the brick wythe is likely to crack and second, to provide isolation from building movement. In Structural Brick Veneer systems, the spacing of vertical joints can be increased over that of conventional veneers because the reinforcement provides additional resistance to cracking.

The Brick Institute of America Technical Note 18A, Differential Movement - Expansion Joints, contains a valuable discussion on the many considerations involved in expansion joints.

http://www.bia.org/html/frmset_thnt.htm

As a general rule, vertical expansion (movement) joints should be provided at the following locations:

1. At or near wall corners
2. At wall discontinuities
3. At changes in height
4. At changes in thickness
5. At changes in stiffness
6. Adjacent to dissimilar materials
7. At abutments to other building elements

The joint size should be a minimum of twice the calculated amount to meet the limitations of the compressibility of the sealant. As a minimum, it is recommended that the joint not be less than 3/8 inch.

4.10 Window Anchorage

Windows are usually anchored to the Structural Brick Veneer and not to the backup wall. In some installations, connectors are included in the design to make the attachment. In other installations, powder-actuated fasteners shot into the masonry have been used to make the connection. The choice depends on the loading and local practice.

5.0 Construction

5.1 General

The construction phase of the project begins with the award of the Contract for Construction. If the Structural Brick Veneer was specified as bidder designed, then the design of the Structural Brick Veneer may begin with the beginning of the construction. In this event, there is usually limited time for the design and it should begin without delay.

Once designed, a Structural Brick Veneer project construction is similar to any other structural brick project.

The mason contractor is normally a subcontractor to the general contractor. If the Structural Brick Veneer is bidder designed, the mason contractor usually becomes responsible for the design. If the design is part of the contract documents, the mason contractor will still

need to prepare shop drawings detailing the installation.

Most mason contractors do not normally prepare shop drawings for reinforcement. Thus, the responsibility for providing an adequate set of shop drawings can become lost in the process of bidding. The general contractor thinks the mason will prepare the shop drawings and the mason thinks the general contractor will prepare the shop drawings.

Often, the mason contractor does not supply the reinforcement for the wall. The general contractor supplies it. The design team should become aware of who will supply the reinforcement. It will have some influence on how the project proceeds.

Experience has shown that the preferred choice is for the mason contractor to purchase the reinforcement and supply the shop drawings. However, some mason contractors may bid high or even not bid at all as a result of this requirement. The resulting cost pressure may require the design team to be flexible about the requirement for shop drawings and their preparation.

Sometimes the general contractor will supply the shop drawings using his normal reinforcement detailer. Unfortunately, the detailer may have limited experience with masonry (the expertise is concrete) and the drawings submitted are often full of errors or items that cannot be constructed.

5.2 Construction Sequence

Several construction trades construct structural Brick Veneer. The dead load support angle (or connectors) and lateral braces are usually installed by the steel erector or the general contractor. These trades prefer to complete their work with the completion of the building structural frame. But if this is done, the mason is constrained by constructing the brick to match the installed connectors.

For example, for a Structural Brick Veneer supported on a continuous ledger, if the ledgers are all installed prior to the laying of the brick, it is nearly impossible to place the vertical reinforcement and grout the wall. It is usually necessary to delay installation of the ledgers until after the brick below is completed.

If a backup wall is constructed, it is usually installed by a different contractor and at a different time than the Structural Brick Veneer. Normal choices for the inside wall include masonry, concrete or steel studs. For a Structural Brick Veneer System, however, the masonry and concrete choices are unlikely because the stiffness and strength of the backup wall are no longer required.

When metal studs are used, it is common to sheath the outside face of the stud to provide additional water and air infiltration protection. The sheathing must be installed before the Structural Brick Veneer. Thus, the connectors typically penetrate the sheathing. This requires additional coordination between the trades.

5.3 Pre-Construction

Once the general contractor and the mason subcontractor have been selected, the engineer should verify the mason's qualifications with the local masonry institute and the local material suppliers. This information will be helpful for determining the amount of time and effort that will be needed during construction.

At an appropriate time, usually at least two months before the start of masonry construction, arrange for a preconstruction conference to discuss the masonry construction. Attendees should include:

1. The mason contractor and foreman.
2. The general contractor and superintendent.
3. The building official.
4. The architect.
5. The special inspector, when required.
6. The engineer.
7. The owner's representative.
8. The brick supplier.
9. The window supplier.
10. The dry-wall installer.

Subjects for discussion include:

1. Brick:

Determine the availability and delivery schedule of the selected brick. If the unit strength method is used to verify required masonry strength, verify that the brick will meet the required strength.

2. Initial testing:

If the unit strength method was used to establish the design strength f'_m , then mortar, grout and prism testing prior to construction are not required. However, it is recommended that when full allowable stresses were used in the design, prism testing should be conducted prior to construction. As a minimum, unit testing or manufacturer's certification is required.

For Structural Brick Veneer installations, a grouting test panel is often necessary to demonstrate the grouting procedures. The schedule should be defined. The design team, building official and special inspector should be present for the grouting demonstration.

Often the grouting demonstration panel can also be used as a color and quality control panel for the architect.

3. Testing During Construction:

Prism testing is recommended for each 5,000 square feet of wall. During construction, three prisms constitute a test, however, five are recommended. Test the first one at seven days, the next three at 28 days and hold the final sample for testing in case of a problem.

If prism tests are conducted, grout and mortar tests are usually not required.

4. Inspection:

ACI 530-02/ASCE 5-02/TMS 402-02
*Building Code Requirements for
Masonry Structures.*

The inspector should regularly check the preparation of mortar and grout to ensure proper proportions and the laying of units to ensure proper workmanship. The inspector should verify and ensure full compliance with the contract documents for the placement of reinforcement, grouting and the protection of the masonry from rain, dirt, cold and/or hot weather.

5. Observation:

Inform all participants that from time to time representatives of the design team will visit the site to ensure general compliance with the contract documents.

6. Inspection Reports:

Normally, test reports and inspection reports go to the general contractor; then to the architect; and then to the engineer. Deviations from this normal procedure should be discussed, defined and documented.

7. Submittals:

Verify that the required project submittals have been approved or are in the process of being approved.

8. Cleaning and Water Repellents:

The procedures to be used to clean and apply water repellents should be discussed. It is important that the mason contractor verify the cleaning method with the unit manufacturer. If water repellent is to be applied, the method and materials to be used should also be verified with the brick manufacturer.

Proper water repellents do not seal the wall. Masonry must breathe. Painting contractors often are not experienced with applying water repellents to masonry, and assuring understanding of proper methods is important.

9. Construction Sequence and Schedule:

The successful installation of a Structural Brick Veneer project requires coordination of many trades (see previous discussion). Ask the general contractor questions to ensure that the construction sequence and responsibilities have been defined. Discuss the schedule for inspection and testing. Discuss coordination issues. One usually missed item is the coordination with the window and door supplier. The design of the connections should be discussed.

10. Window Attachment:

As previously discussed, the attachment of the window and coordination of the flashing system will be important to the successful performance of the wall. Too often this important coordination is left to the last minute or not done at all. The pre-construction conference is a good opportunity to discuss the issue and assign responsibility.

11. Sealant Installation:

Often the sealant installer is a separate subcontractor from the Structural Brick Veneer and window contractors. The specified sealant may or may not be compatible with the brick, mortar, water repellent and window-supporting member finish. These building elements,

and often others, all come together at the sealant joints. The selected sealant and installation procedures should be checked for compatibility with adjacent materials.

12. Compliance Testing

Most Structural Brick Veneer projects require compliance testing. When the brick supplier is not experienced with the system, testing of prisms in accordance with national standards should be completed before construction begins. Testing during construction is also generally required. During the pre-construction conference, the compliance testing required should be communicated to all involved.

5.4 Submittal Review

Items submitted for review include the following:

1. Mortar proportions and laboratory test:

The mortar submittal is usually only submitted by type. This is satisfactory provided the mortar is specified by proportions, not strength, and provided there is a method to control the proportions of the mortar.

The contractor may submit proportions other than by type. In this case, laboratory tests should be performed to verify the compressive strength of the mortar (ASTM C 270).

2. Grout proportions:

Specify grout in accordance with proportion requirements of ASTM C 476.

Do not specify grout by the minimum strength of 2,000 psi. The proportions will result in strengths well in excess of 2,000 psi.

Lime may be added to grout. This usually improves the grout properties by increasing flow and retention of water, resulting in improved placement and bonding to the unit.

The additive Grout Aid should be added to the grout as recommended by the manufacturer.

For batch-provided grout, the proportions are normally described by weight. The weight proportions should be converted to volume proportions for comparison with ASTM C 476.

3. Unit certifications:

The unit manufacture should provide certificates that the masonry units comply with the requirements specified.

4. Reinforcement shop drawings:

These should be scheduled to provide sufficient time for review and resubmittal prior to construction. Experience has shown that rejection of the first set of drawings is likely.

5. Connector shop drawings:

Shop drawings showing the detailing of reinforcement, connectors and embedded items need to be submitted with sufficient time for review.

6. Quality control program:

When required by the specification, the quality control program should be written by the mason contractor and submitted in time for review and discussion with all involved, including the inspector and general contractor.

5.5 Site Visits

The structural engineer should make site visits to check on the progress and quality of the work. This part of the engineer's scope of services is called construction observation. It is defined in AIA C141 Section 2.6.3 as:

"The consultant shall visit the site at intervals appropriate to the stage of construction for This Part of the Project, or as otherwise agreed with the Architect in writing, to become generally familiar with the progress and quality of the Work completed for This Part of the Project and to determine in general if the Work is being performed in a manner indicating that the Work, when completed, will be in accordance with the Contract Documents. However, the consultant shall not be required to make exhaustive or continuous on-site inspections to check the quality or quantity of the Work for This Part of the Project. On the basis of such on-site observations as a consultant, the consultant shall keep the Architect informed of the progress of the Work for This Part of the Project and shall endeavor to guard the Owner against defects and deficiencies in such work. (More extensive site representation may be agreed to as an Additional Service, as described in Paragraph 3.2)."

5.6 Non-Conforming Quality Control Tests

Experience has shown that the field-testing of masonry is highly variable. The source of the variability can be the materials, the methods used to prepare the samples, the testing of the samples and the interpretation of the results. Before rejecting a wall because of non-conforming field-testing, the engineer should carefully assess the possible causes of the field test non-conformance and possibly remove and test samples from the actual wall.

5.6.1 Unit Compression Strength

If the Unit Strength Method for establishing and verifying the specified compression strength, f_m , then prism tests (or unit compression tests) are required prior to and during construction. The tests prior to construction should provide assurance that the units and masonry will be satisfactory. If the test prior to construction does not comply with the required strength, then another brick may be required, or the project may have to be redesigned for a lower strength.

Sometimes the brick tests prior to construction are not performed and the units are tested during construction and do not conform. In this case, a new analysis may be required to verify strength. One option is to use Strength Design provisions based on UBC Section 2108. These methods are less sensitive to low compression strength. But, the engineer should be careful about excess deflections and other serviceability

issues when designing with Strength Design.

When prism (units) are tested prior to construction and conform to the requirements, but do not conform to the requirements when tested during construction, the problem is probably in the manufacturing of the prisms or there was a change in the testing procedures or materials. The units should be tested (or re-tested). If the units still do not comply, the code allows prism testing to verify strength. Construct prism tests to verify the strength.

If the prism strengths do not comply, a redesign or change in the brick unit may be required. Non-conforming walls will likely require removal.

5.6.2 Mortar Compression

It is recommended that field-testing of mortar not be required. Prism testing every 5000 square feet of wall should be specified. However, the requirement for mortar testing often is not within the control of the structural engineer, and on many projects mortar testing becomes a requirement.

The field sampling and testing for mortar compression strength is highly variable. The following figure is a frequency distribution of field mortar compression tests taken from actual projects in California, Oregon and Washington. There are a total of 205 mortar tests. The coefficient of variation is 36%.

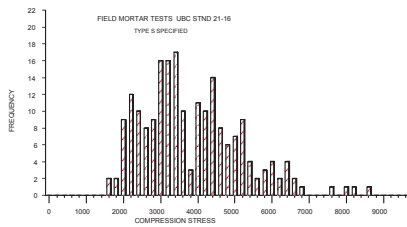


Figure 43 Variation of Field Mortar Tests

With this amount of variability, it should not be surprising to get periodic non-conforming compression mortar tests. If the non-conformance occurs regularly, then the following steps are recommended:

1. Request from the mason the proportions being used.
2. Assess the method being used to control proportions.
3. Verify that the testing lab is using the procedures of ASTM C 780.
4. Visit the site and observe the mortar in the joint. Scratch the mortar with a key. If a white scratch results and the sand does not separate from the mortar, the strength of the mortar is probably acceptable. However, if the masonry is highly stressed (above 1200 psi) it may be necessary to remove a prism from the wall for testing.

The relationship between 7-day mortar strength and 28-day mortar strength is fairly consistent from sample to sample. It is useful to know the 7-day test results since they provide the engineer with an early indication of the 28 day results. The following figure presents the relationship.

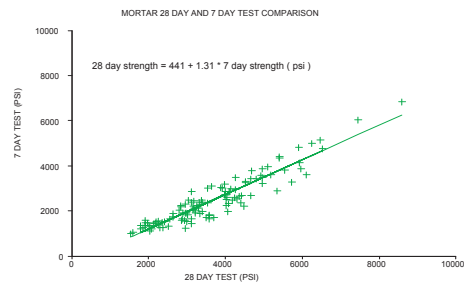


Figure 44 Mortar 7 day and 28 Day Tests

5.6.3 Grout Compression

It is recommended that field-testing of grout not be required. Prism testing every 5000 square feet of wall should be adequate quality control. However, the requirement for grout testing often is not within the control of the structural engineer, and on some projects grout testing becomes a requirement.

The field sampling and testing for grout compression strength is highly variable. The following figure is a frequency distribution of field grout compression tests taken from actual projects in California, Oregon and Washington. There are a total of 323 grout tests. The coefficient of variation is 32%.

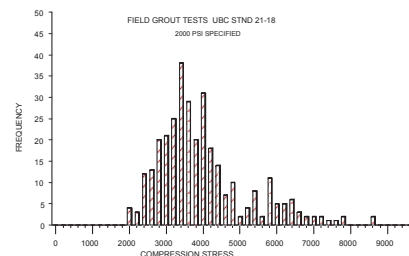


Figure 45 Variation of Field Grout Tests

With this amount of variability, it should not be surprising to get periodic non-conforming compression grout tests. If the non-conformance occurs regularly, then the following steps are recommended:

1. Request from the mason the proportions being used.
2. Assess the method being used to control the proportions.
3. Verify that the testing lab is using the procedures of ASTM C1019.
4. If the cause of the low break is not identified, then taking core samples and testing them may be required.
5. The structural engineer should also consider the reason for requiring the specific grout strength. Often, the purpose of the grout is only to connect the reinforcement to the units. Even low strength grouts (1500 psi) are probably capable of making the connection. Because of the high strength of the brick, the compression contribution of the grout can often be ignored in the analysis.

The relationship between 7-day grout strength and 28-day grout strength is also consistent from sample to sample. It is useful to know the 7-day test results since they provide the engineer with an early indication of the 28-day results. The following figure presents the relationship for the same projects.

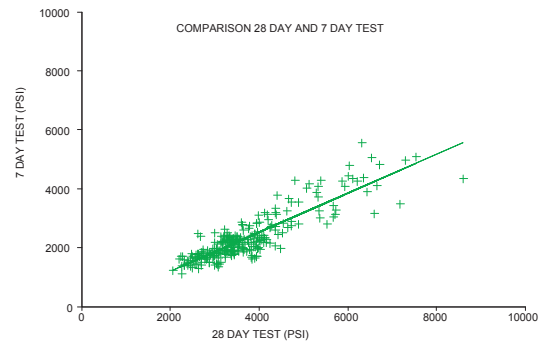


Figure 46 Seven Day and 28 Day Grout Strength

5.6.4 Prism Tests

Prism tests are less variable than either mortar or grout testing and provide the engineer with a higher level of confidence that the masonry system has the desired strength.

When prism tests do not conform, verify that the materials used (units, mortar and grout) conform to the specifications. If they do conform, then either the prism was improperly constructed or the testing procedures were not in compliance with ASTM 1314.

Improper construction of prisms includes not constructing the prism true and plumb. It is very important that the top and bottom planes of the prism are parallel. Another common problem for large cell units (8" units and larger) is that the grout is not properly re-consolidated. Without proper re-consolidation, a dome-shaped void will often form at the mortar joint part way up the height of the prism and render the area of grout ineffective for resisting compression.

Common testing errors include not properly capping the prism so that the top and bottom planes are level and parallel, not using a testing machine with a spherical head, and not providing a thick enough loading platen to distribute the test machine load evenly to the prism. It has also been reported that some testing labs stop loading at the first sound emitted. The first sound may correlate to the failure of a concrete cylinder, but does not typically correlate to the failure of a masonry prism.

5.7 Troubleshooting During Construction

The following table is presented to assist the engineer with problems often occurring during construction. This table was developed over the years based on the experience of the authors. It has been evolving and it often seems that contractors have a special ability to create new situations not previously considered or addressed. Proceed with caution.

Troubleshooting Table for
The Design and Construction of Structural Brick Veneer

PROBLEM	CAUSE	SOLUTION
Prisms fail to reach the design strength.	1. The testing lab has incorrectly tested the prism, usually by not placing the prism correctly in the machine or using a loading platen that is too thin. Or the specimens may have been damaged during transportation. 2. The bricks are below the specified strength. 3. The mortar is under specified strength. 4. Lab reported gross area stress instead of net area stress. 5. Lab stopped testing with first noise.	1. Instruct the lab to retest being careful to follow the ASTM C 1314. 2. Request that the contractor have a lab retest the brick. If still too low, change the brick or redesign. 3. Check mortar proportions. Retest the prisms. 4. Have the lab correct the report. 5. Re-test.
Mortar doesn't reach strength.	1. Incorrect proportions. 2. Incorrect testing.	1. Check mortar quality control procedures. 2. Mortar tests are unreliable. Forget about testing mortar. The code doesn't require it, if prisms are tested.
Colors do not meet expectations.	1. Bricks were not blended. 2. Sample panel has different sealer. 3. The brick production run is different from the sample run.	1. This is a problem for the architect and brick supplier. There are paints available, but results are questionable. 2. Use the sample panel sealer. 3. Approve the production run before beginning construction.
Someone calls and says more expansion joints are required.	Someone was checking on your advice. They should do this, so don't get mad.	Explain how the reinforcement reduces the need for most of the expansion joints.
The mason tells the general who tells everyone that the cells are too small to be grouted with all the congested steel.	1. The mason contractor does not have experience with grouting of reinforced hollow brick. He doesn't understand that he can make the grout with an 8 to 11 inch slump. 2. The cell is too small.	1. Prepare and grout a test panel. Be sure to invite everyone concerned. 2. Use a different cell size.

STRUCTURAL BRICK VENEER

PROBLEM	CAUSE	SOLUTION
Welded bars are breaking off.	ASTM A 706 bars were not used.	Inspect bars. A "W" symbol indicates type A 706. Use the correct bars.
Contractor is not protecting his materials or work.	Sometimes the responsibility for protecting the work is left with the general contractor. He is saving money. Sometimes the responsibility is not well defined.	Write a letter to your client. Explain the consequences. Send a copy to the brick manufacturer.
Cracks in the mortar joints.	1. Shrinkage of the mortar joint. 2. Movement of the supporting structure. 3. Overloading. 4. Too rapid drying.	1. Suggest the contractor decrease the cement content of the mortar and increase the lime. 2. Check supports. 3. Check the loading, the timing of the loading and shoring removal. 4. Pre-wet the units. Wet the wall during curing. Add lime to the mortar.
Shop drawings are not prepared.	The requirement was missed or "value engineered away".	Write a letter to your client explaining the requirement. If the project is underway, require an engineer familiar with the design be on site full time.
The grout strength is specified at a minimum of 2000 psi, how can I get a prism of 4000 psi.	This is normal.	Explain that the prism does not fail in accordance with the weak link theory.
The contractor wants to high-lift grout with lifts larger than 6'	The code restricts the grout lift to 6' even though the grout pour might be higher.	The problem is blow-outs of mortar joints and the ability to reconsolidate. In hollow clay, these problems are unlikely. Have the contractor demonstrate the procedure to you and the inspector.
The contractor doesn't want clean-outs. You want high-lift grouting.	Code requires clean-outs for high-lift, in order to remove the mortar droppings.	In most Structural Brick Veneers the shear stresses are low. It is usually possible to waive the clean-out requirement.

PROBLEM	CAUSE	SOLUTION
The dowels out of the concrete foundation or ledger interfere with the unit cross webs. They miss the cells.	Improper placement of the dowels. However, it is often very difficult to get them in the right place. This situation should not occur in a Structural Brick Veneer.	Cut the unit cross webs to allow the dowel to pass or drill in new dowels. Verify that all the dowels are required to meet strength requirements. Do not, allow the dowels to be bent.
The brick masonry is cracked, with cracks extending through the units.	1. A great deal of force is required for this condition to exist. The cause is likely a problem. 2. The bricks may have been manufactured with the cracks. 3. Foundation cracks extend into brick wall.	1. Find the reason for the cracking. It is likely that something needs to be corrected. Likely candidates include frozen grout, foundation movement, or thermal movement from adjacent structure. 2. Verify the integrity of the units before use. A quick check is to bang the bricks together. If a ringing sound results instead of a thud, then the bricks are sound. 3. Foundation control joints need to be coordinated with the masonry expansion joints.
Contractor doesn't cover the walls at the end of the day.	1. The contractor is attempting to save money. 2. The responsibility for the masonry protection may have been left with the general contractor or worse, left out of all the contracts.	1. Insist on covering the walls. 2. Write a letter stating that the contractor is not in conformance with the likely result being efflorescence and other wall damage.
Corrosion of the joint reinforcement.	1. Too strong an acid cleaning without pre-wetting the wall. 2. Ungalvanized joint reinforcement.	1. Pre-wet the wall and use industry cleaners as recommended by the manufacturer of the units. 2. Use galvanized joint reinforcement.

6.0 Testing

Many different pre-construction mockup tests are available for evaluating the performance of the Structural Brick Veneer System design and construction. Pre-construction mockup testing is not necessary for all projects, and because of the costs involved, is likely feasible only for large projects. Tests are generally conducted to evaluate air, water and structural performance.

Veneer System. A standard test procedure, ASTM E 330, is available for testing exterior windows and curtain walls. Brick panel strength tests can be conducted in accordance with ASTM E 72. There are no standard tests for specifically measuring seismic performance.

6.1 Air

When air infiltration tests are conducted on the building mockup, they should be performed in accordance with ASTM E 783, while those conducted in the laboratory should be performed in accordance with ASTM E 283. Air infiltration tests should normally be done before water penetration tests because water trapped in the brick veneer tends to reduce air leakage.

6.2 Water

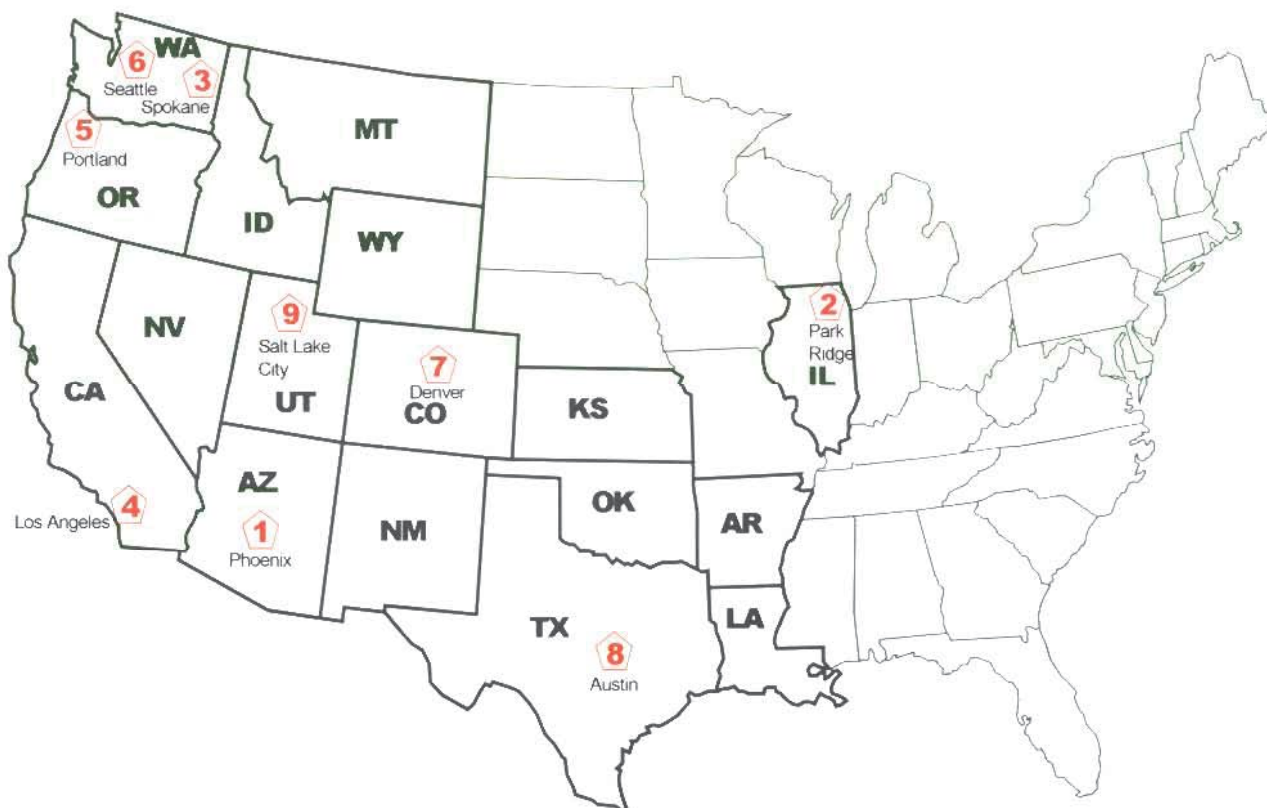
Water penetration tests for the brickwork should be performed in accordance with ASTM E 514, to measure the permeability of the constructed wall. Additional large-scale mockup tests are available using the procedures contained in AAMA 501.3, developed for testing aluminum curtain wall systems.

6.3 Structural

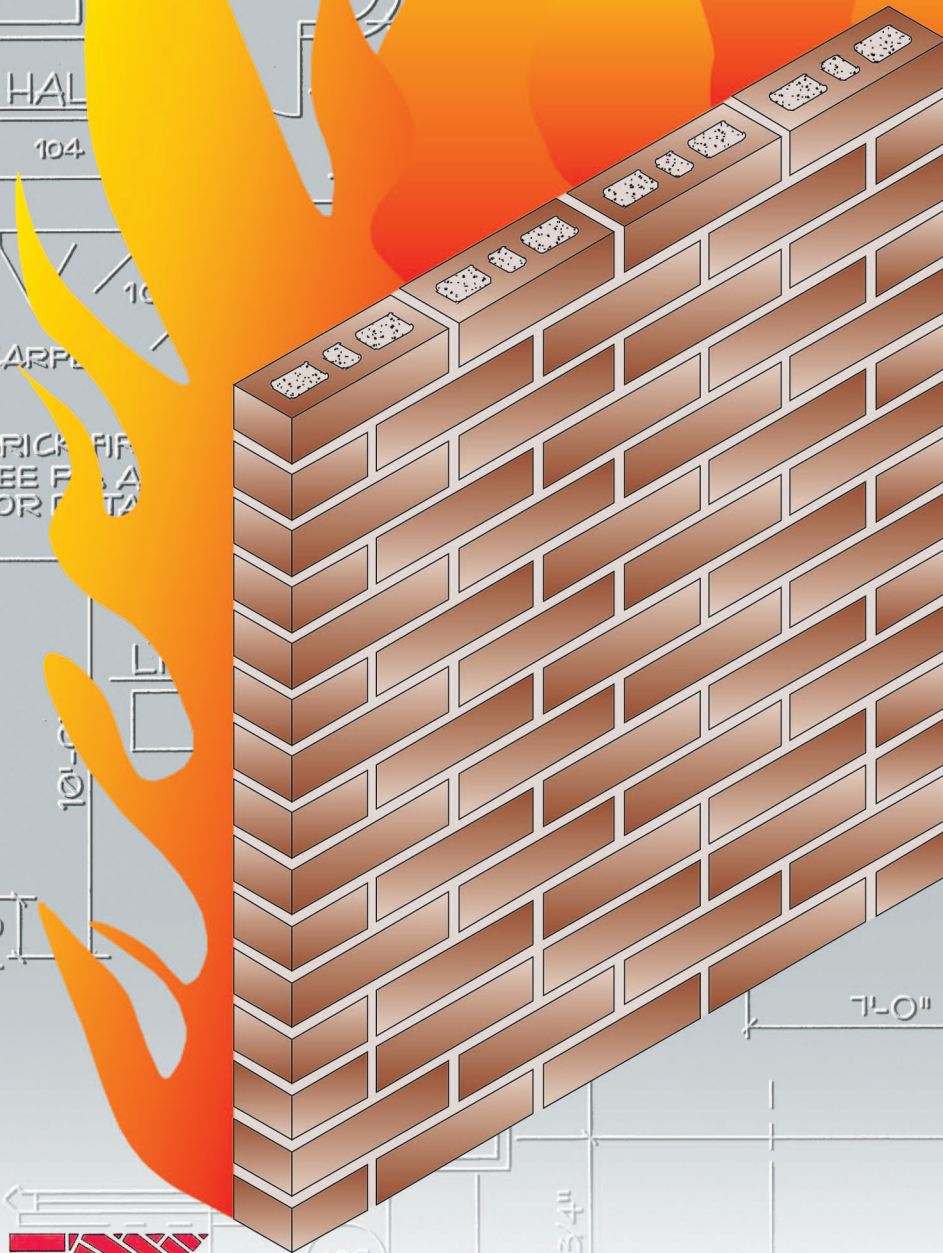
Structural tests measure a system's performance under lateral loading or deflection. This type of testing provides a means for accurately assessing the complex behavior of the Structural Brick

ALLIED ASSOCIATES AND WEB ADDRESSES

- | | |
|---|--|
| 1 Arizona Masonry Guild | (www.masonryforlife.com) |
| 2 Masonry Advisory Council | (www.maconline.org) |
| 3 Masonry Industry Promotion Group | (www.masonrypromotion.com) |
| 4 Masonry Institute of America | (www.masonryinstitute.org) |
| 5 Masonry Institute of Oregon | (www.mioctio.org) |
| 6 Masonry Institute of Washington | (www.masonryinstitute.com) |
| 7 Rocky Mountain Masonry Institute | (www.rmmi.org) |
| 8 Southwestern Brick Institute | (www.swbrick.com) |
| 9 Utah Masonry Council | (www.utahmasonrycouncil.org) |



FIRE ENDURANCE RATINGS OF CLAY BRICK MASONRY



WESTERN STATES CLAY PRODUCTS ASSOCIATION

www.wscpa.us

FIRE ENDURANCE RATINGS OF CLAY BRICK MASONRY

Prepared for:

Western States Clay Products Association

Submitted by:

*Jeffrey L. Elder
Technical Committee Chair
Western States Clay Products Association
22815 Frampton Avenue
Torrance, CA 90501*

December 2008

The material presented in this publication, including technical and engineering data, figures, drawings and tables are for general information only. It should not under any circumstances be relied upon for specific applications of Fire Endurance Ratings of Clay Masonry without independent evaluation by a licensed design professional familiar with its specific use and application. Anyone making use of this material does so at their own risk and assumes any and all liability resulting from such use.

The members of Western States Clay Products Association express appreciation to Walter Dickey (1908-2002) for his tireless efforts in promoting the clay brick industry. Mr. Dickey spearheaded the initial production of this publication providing technical and editorial input that continues in this printing.

1.0 INTRODUCTION

One of the most frequently asked questions of the designer is "How do I achieve a 1, 2, 3, or 4-hour fire rating using various clay brick masonry assemblies?"

Intuitively, we are comfortable with clay bricks as a fire resistant material because of their use as a liner in fireplaces and kilns where temperatures exceed those expected in most construction fires. In addition, we have observed the effect of fires on buildings where the only thing to remain standing is the brick. Most designers do not question that brick is non-combustible, or resistant to fire, what they question is the fire resistance of each of the brick wall configurations in hours as it relates to building code requirements.

For many wall assemblies, the information is available from BIA technical Notes 16, and the International Building Code Table 720.1. For some assemblies, the designer is allowed to use methods of calculating fire resistance. Until recently, designers had to rely on ICBO Evaluation Report #5058 to obtain 1 and 2-hour fire ratings for Adhered and Anchored Veneers. Today this information is found in Chapter 7 of the International Building Code, Table 720.1.

2.0 FIRE RESISTANCE BASICS

Fire resistance refers to the ability of a structure to act as a barrier to the spread of fire and to confine it to the area of origin. Therefore, in addition to withstanding the fire, the intent of the code is to prevent other materials adjacent to the brick from combusting after prolonged increased temperatures from fire, flame, or hot gases. Consequently, the assemblies are given a fire rating which is the time it takes for a prescribed fire on one side of an assembly to reach an average temperature on its other side that would ignite cotton waste (250°F). Refer to ASTM E119.

Fire ratings are required for load bearing and non-load bearing wall assemblies. Load bearing assemblies must be capable of withstanding the same conditions as the non-load bearing assemblies. In addition, they must be capable of supporting their prescribed design load for the duration of the fire.

Fire endurance tests alone cannot supply all of the data needed for intelligent appraisal of fire ratings

of building elements. There are simply too many different types of assemblies and combinations of materials to classify them all through actual tests. Fortunately, the theory of fire endurance ratings is sufficiently advanced to offer guidance in estimating fire endurance ratings.

The ultimate goal of this guide is to bring together in one location all of the approved fire ratings for adhered veneer, anchored veneer, structural brick veneer and load bearing brick. See Table 1.

3.0 MASONRY ASSEMBLIES

Masonry fire resistant assemblies are broken down into adhered brick veneer, anchored brick veneer, structural brick veneer, and load bearing brick.

Fire rated assemblies are often required in two directions. If the fire is on the brick side, the brick assembly must resist flames, heat, and smoke from entering to the other side of the assembly prior to the rated time. If the fire is on the other side, the fire rating may be needed to protect occupants in another room or building from the same effects. In most cases, there is a structure supporting the fire resistant materials and the fire rated assembly is required on both sides to protect the structure. If the material is the structure as is the case in load bearing and structural brick veneer, the material can adequately provide fire resistance in both directions.

In a veneer system, the stud backing is the structure and must be protected in both directions. Therefore, brick most commonly provides fire resistance on the exterior side of the stud, and gypsum or some other material provides the fire resistance on the interior face of the stud.

3.1 ADHERED BRICK VENEER

Adhered brick veneer is defined as brick veneer secured and supported through adhesion to an approved bonding material applied over an approved backing. The most common wall assembly is defined by a facing of thin brick between 1/2" to 1 1/2" in thickness applied over a plaster backing. If this combined masonry layer is 1 3/4", the masonry side of the assembly qualifies for a 1-hr fire rating. If the masonry layer is 2" or more, the masonry side of the assembly qualifies for a 2-hr fire rating. In this assembly, the plaster is applied to metal lath, which in turn is applied to wood or steel framing.

FIRE ENDURANCE RATINGS OF CLAY BRICK MASONRY

The wood or steel framing members must be sized for all design loads. Steel studs are to be a minimum of 2½". Wood framing is to be 2 x 4 minimum. Wider framing members can be used without degrading the fire resistance rating.

3.1.1 TEST PANEL CONSTRUCTION

The test panels were constructed similar to those for the earlier large program of gypsum board fire evaluation tests. Type X gypsum board was applied to one face of standard steel stud construction. Typical backing and adhered brick veneer was installed on the opposite face by masons experienced in such work.

The panel was designed for symmetrical resistance as required by the International Building Codes; however, if one designs for a situation in which fire exposure is limited to 1-face only, the resisting thickness can all be on one side.

3.1.2 FIRE TESTING

The veneered panels were exposed according to the procedures of ASTM E119 – standard furnace temperature rise; thermocouples on the unexposed face to show temperature rise; verification that there were no cracks that would allow passage of flame or hot spots; and resistance to hose stream exposure without penetration. Additional thermocouples were installed during construction to provide information on the thermal gradient through the section during the fire test. This information helped revise design alternates with greater accuracy.

3.1.3 RESULTS

All panels passed the three basic ASTM requirements with considerable margin. Although the results were somewhat conservative, indicating that thinner veneer might be used, no further refinements or reduction in design thickness was attempted. Precise refinements are not justified in variable hand placed materials, especially if no great cost benefits would result.

The maximum temperature rise on the unexposed face is permitted to be a maximum of 325°F for a single thermocouple and 250°F for the average of all thermocouples, at the specified time period. The test panels showed the following maximum and average rise of temperature. All withstood more hose stream exposure than required by code.

	Maximum	Average
Panel 1A:	162°F pt.	158°F
Panel 1B:	145°F pt.	135°F
Panel 2A:	129°F pt.	110°F
Panel 2B:	107°F pt.	104°F

3.1.4 CONCLUSION

Adhered brick veneer of 1¾" combined masonry thickness provides 1-hour of fire resistance. Adhered brick veneer of 2" combined masonry thickness provides 2-hours of fire resistance. These fire rated assemblies are found in Table 720.1(2) Items 15-2.1, and 15-2.2.

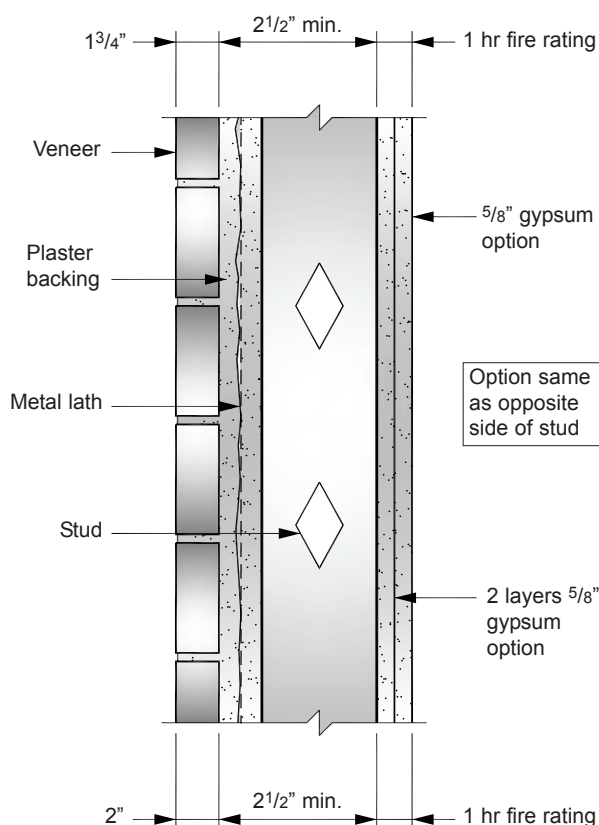


Figure 1 Adhered brick veneer.

3.2 ANCHORED BRICK VENEER

Anchored brick veneer is defined as brick veneer secured to an approved backing by mechanical fasteners. The most common wall assembly is defined by a facing of brick between 2" and 5" in width anchored through mechanical connectors to wood or steel framing.

Excerpts from Table 720.1(2)

TABLE 720.1(2) [Excerpt] RATED FIRE-RESISTANCE PERIODS FOR VARIOUS WALLS AND PARTITIONS						
MATERIAL	ITEM NO.	CONSTRUCTION	4 hour	3 hour	2 hour	1 hour
1. Brick of clay or shale	1-1.1	Solid brick of clay or shale.	6	4.9	3.8	2.7
	1-1.2	Hollow brick, not filled.	5.0	4.3	3.4	2.3
	1-1.3	Hollow brick unit wall, grout or filled with perlite vermiculite or expanded shale aggregate.	6.6	5.5	4.4	3.0
	1-2.1	4" nominal thick units at least 75 percent solid backed with hat-shaped metal furring channel $\frac{3}{4}$ " thick formed from 0.021" sheet metal attached to the brick wall on 24" centers with approved fasteners, and $\frac{1}{2}$ " Type X gypsum wall board attached to the metal furring strips with 1" long Type S screws spaced 8" on center.	—	—	5	—

3.2.1 FIRE RESISTANCE

From the fire endurance test, additional information was provided which reduced the thickness of the veneer to 2" with a $\frac{1}{2}$ " air space for a 1-hour fire rating and a 2" veneer with a 1" air space for a 2-hour fire rating.

Table 720.1(2) of the International Building Code item numbers 1-1.1 through 1-2.1 can also be used to determine the minimum equivalent thickness for brick to achieve a fire rating from 1 to 4 hours. This section of the table is used for veneer brick and structural brick in load bearing and non-load bearing construction.

3.3 STRUCTURAL BRICK VENEER

Structural Brick Veneer is defined as a masonry assembly of brick, mortar, grout, and reinforcing that is designed similar to anchored brick veneer. It carries no gravity loads other than its own weight, the weight of windows, and possibly other miscellaneous loads. In addition, it is not part of the lateral load resisting system. The difference is that the structural brick veneer is reinforced and grouted to allow the brick to span much further between wall ties. Structural brick veneer does not generally require a backup system to carry lateral loads within the panel. In the Structural brick veneer, the lateral loads are transferred through tension and compression stresses in the masonry.

3.3.1 FIRE RESISTANCE

Table 720.1(2) of the International Building Code item numbers 1-1.1 and 1-1.3 can also be used to determine the minimum brick thickness to achieve a fire rating from 1 to 4 hours. This section of the table is used for structural brick veneer. Since the structural brick veneer assembly was tested with vertical loads, the brick is capable of giving the same fire rating in both directions. No additional framing or fire resistant material is required.

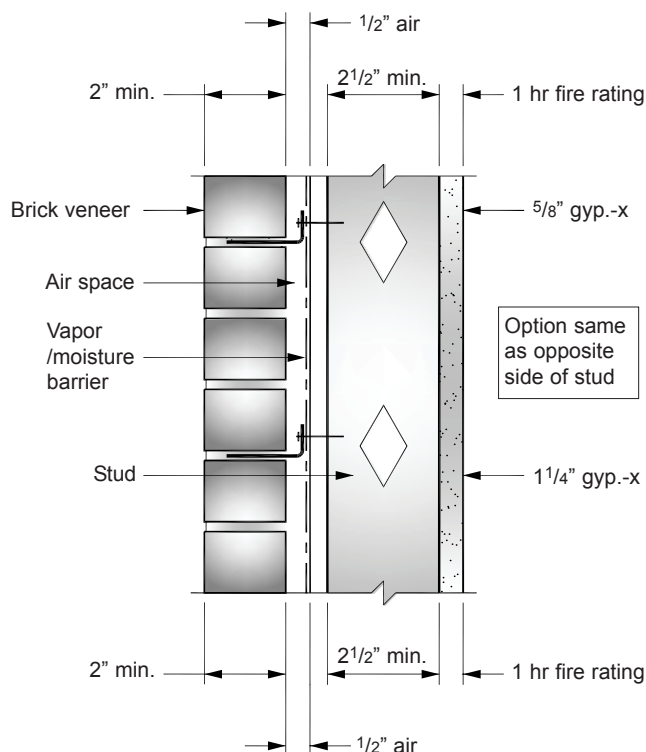


Figure 2 Anchored brick veneer.

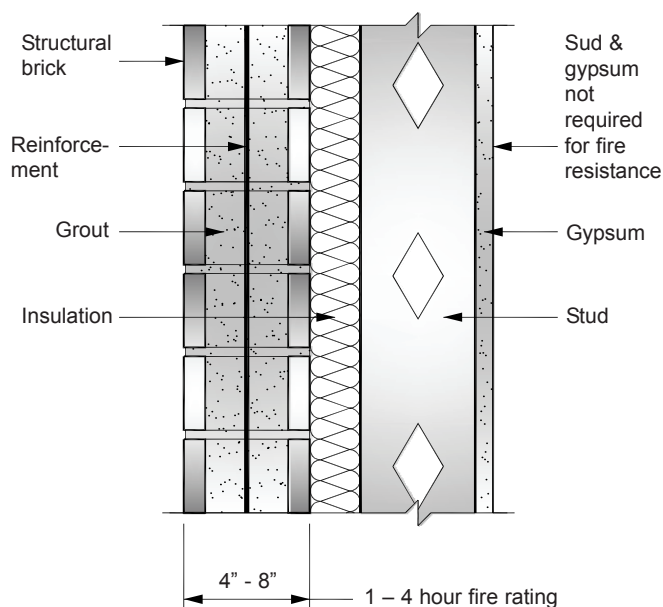


Figure 3 Structural brick veneer.

3.4 STRUCTURAL LOAD BEARING BRICK

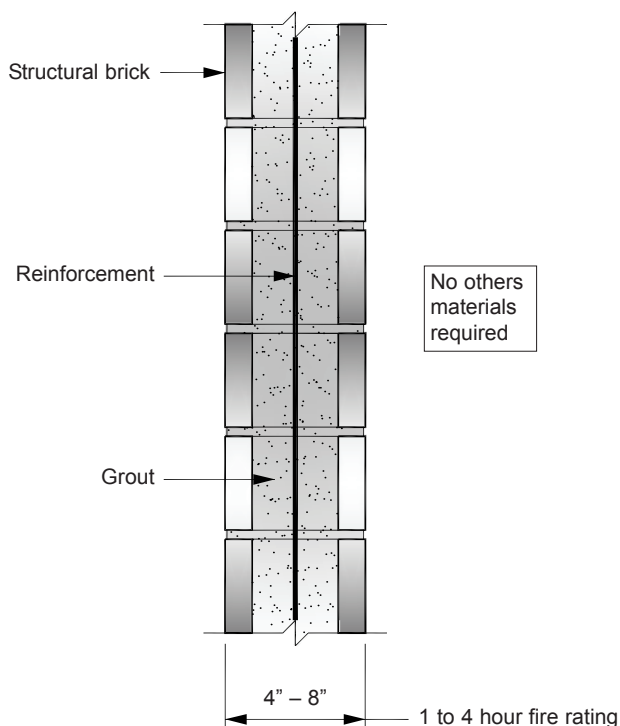


Figure 4 Structural load bearing brick.

Structural load bearing brick is defined as a masonry assembly of brick, mortar, grout and reinforcing that is designed to support all gravity and lateral loads. These masonry elements are generally part of the lateral load resisting system. Greater fire endurance performance and economics can be achieved through the use of reinforced masonry. Fire ratings of 3 and 4 hours can be achieved through the use of up to 8" of hollow clay brick as shown on Figure 4.

3.4.1 FIRE RESISTANCE

Table 720.1(2) of the International Building Code item numbers 1-1.1 through 1-2.1 can also be used to determine the minimum brick thickness to achieve a fire rating from 1 to 4 hours. This section of the table is used for structural brick in load bearing and non-load bearing construction.

4.0 CALCULATED FIRE ENDURANCE RATINGS

As it is impossible to define all of the possible configurations that might be conceived in a wall design, Section 721.4 of the IBC along with IBC Standard 7-7 provides greater assistance in designing with Clay Brick. This section and the standard list more precise rating values for thicknesses of brick, mortar, air space, cement, gypsum plaster, wall board, insulation, studs, siding, etc. They also include more precise calculations for calculating Equivalent Thickness = $T_e = V_n/LH$; R-ratings when adding plaster is defined as $R = (R_n^{0.59} + pl)^{1.7}$ and combinations of clay brick, insulation, air and plaster can be defined using the exponential equation $R = (R_1^{0.59} + R_2^{0.59} + R_n^{0.59} + A_1 + A_2 + \dots PI)^{1.7}$ where R is the fire resistance of each material layer, A is the resistance of each air layer and PI is the resistance of gypsum plaster.

Table 1 identifies a list of R , A , and PI values for use in determining the Fire endurance of miscellaneous clay masonry assemblies.

IBC Standard 7-7 lists 8 rules that are helpful in making a quick assessment of the fire endurance of building elements when fire test data on the elements are not available.

FIRE ENDURANCE RATINGS OF CLAY BRICK MASONRY

Table 1 Fire Resistance (R) in Hours

Table 1 Fire Resistance (R) in Hours			
Masonry Assembly		Actual Wall Thickness (inches)	Fire Resistance (hours)
Adhered Veneer	ASTM C1088	1.75" *	1
		2" *	2
Anchored Veneer	ASTM C216	2" to 5"	3
Structural Veneer and Load Bearing Brick Solid 75%	ASTM C216	3.5"	1
		5.5"	2
		7.5"	3
		8"	3
8" Cavity wall with 2" air space	ASTM C652	10"	4
10" Cavity wall with 2" air space		3.5"	1
Hollow – cells not filled		5.5"	1
		7.5"	2
Hollow – all cells filled ¹	ASTM C652	3.5"	1
		5.5"	3
		7.5"	4
Air (A)		1/2" to 3 1/2"	0.30
Plaster (PI)		1/2"	0.30
		5/8"	0.37
		3/4"	0.45

Notes:

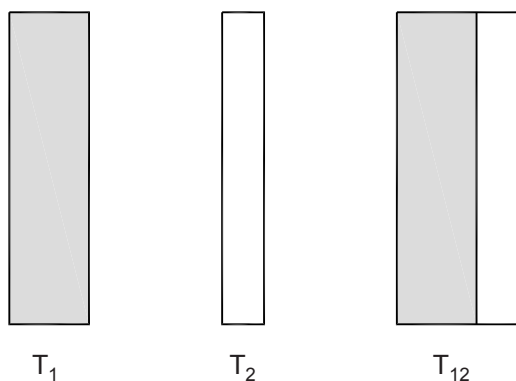
* Thickness includes plaster bedding.

1 Cells may be filled with perlite, vermiculite, expanded shale aggregate or grout.

4.1 RULE 1

The "thermal" fire endurance of a construction assembly consisting of a number of parallel layers is greater than the sum of the "thermal" fire endurance characteristic of the individual layers when exposed separately to fire.

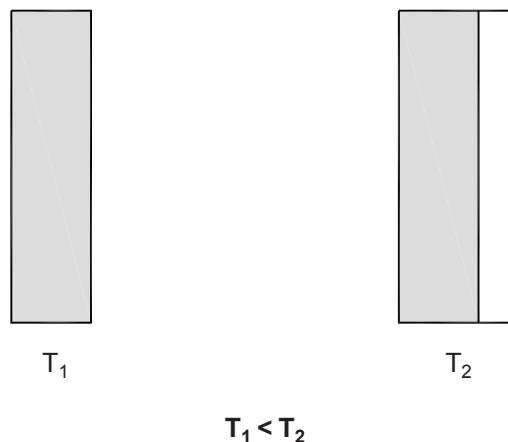
Rule 1



4.2 RULE 2

The fire endurance of a construction assembly does not decrease with the addition of further layers.

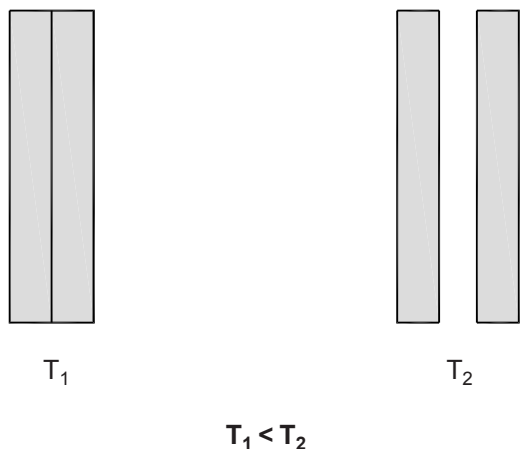
Rule 2



4.3 RULE 3

The fire endurance of a construction assembly containing continuous air gaps or cavities is greater than the fire endurance of similar constructions of the same weight, but containing no air gaps or cavities.

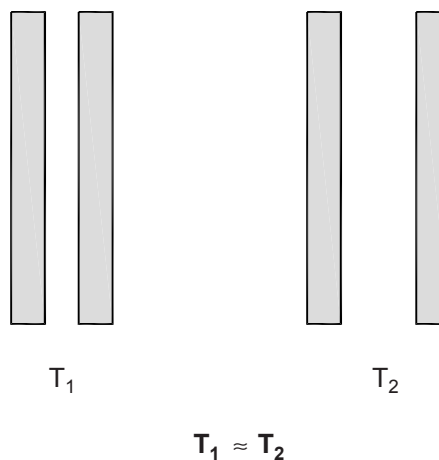
Rule 3



4.5 RULE 5

The fire endurance of a construction assembly can not be increased by increasing the thickness of a completely enclosed air layer.

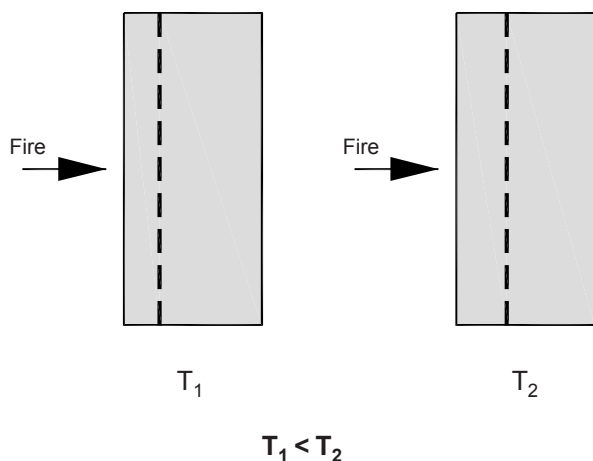
Rule 5



4.4 RULE 4

The farther an air gap or cavity is located from the exposed surface, the more beneficial is its effect on the fire endurance.

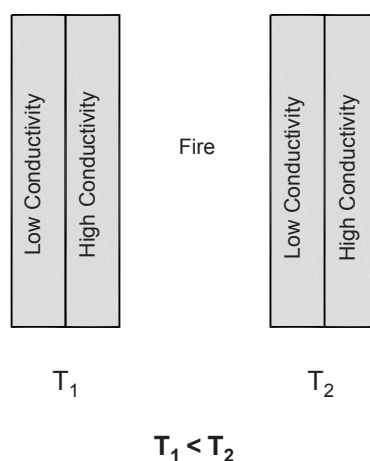
Rule 4



4.6 RULE 6

Layers of materials of low thermal conductivity are better utilized on that side of the construction on which fire is more likely to happen.

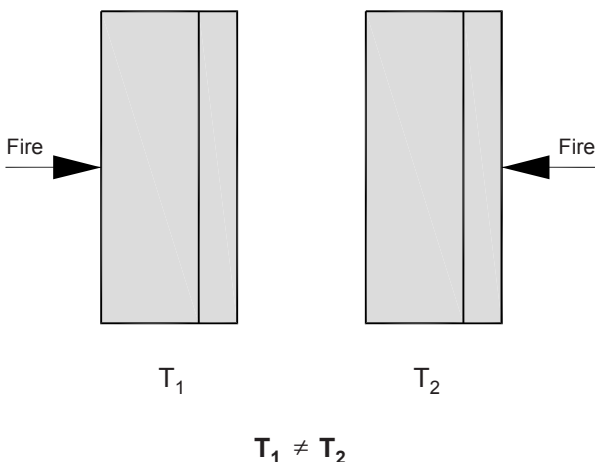
Rule 6



4.7 RULE 7

The fire endurance of asymmetrical construction assemblies depends on the direction of heat flow.

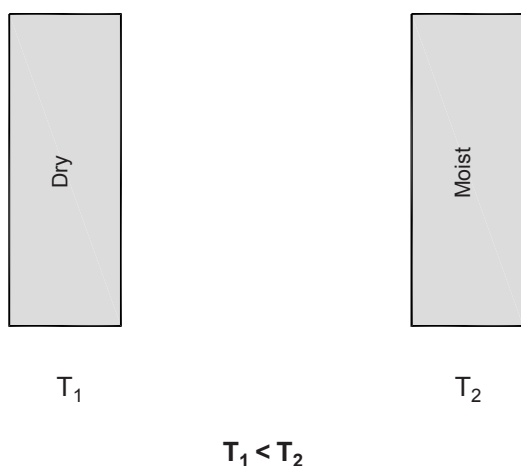
Rule 7



4.8 RULE 8

The presence of moisture, if it does not result in explosive spalling, increases the fire endurance.

Rule 8



5.0 CONSIDERATIONS

When backing materials are used to provide support for the fire resistive materials, the backing material shall be protected with the same fire resistance rating on both sides.

Although life safety is the primary reason for incorporating fire resistance in construction, when considering tradeoffs such as gypsum, or sprinklers, other issues should also be considered such as, fire separation, property protection, water damage, replacement costs and reuse of structure.

APPENDIX – WALL DETAILS

A.1 MATERIALS:

A.1.1 Thin Veneer Brick Units: Units are produced from clay or shale in thicknesses from 1/2 to 1 1/2 inches (12.7 to 38 mm) and comply with ASTM C1088, Grade TBS or better.

A.1.2 Face Brick Units: Units are made from clay or shale in thicknesses exceeding 1 1/2 inches (38 mm) and comply with ASTM C216 for veneer facing units; Type FBS or better and Grade SW in Severe Weather regions.

A.1.3 Hollow Brick Units: Units are made from clay or shale in thicknesses exceeding 4 inches (38 mm) and comply with ASTM C652 for hollow clay brick; Grade HBS or better and Grade SW in Severe Weather regions; Unit strength as specified for the job.

A.1.4 Mortar: Type S as set forth in Table 2103.8(1) or Table 2103.8(2) of the IBC.

A.1.5 Plaster Backing: Portland cement plaster complying with Section 718 and Section 2507 of the IBC.

A.1.6 Lath: Minimum 3.4 pounds per square yard (1.8 kg/m³) metal lath complying with Section 2507 of the IBC.

A.1.7 Steel Framing: Framing with either gypsum plaster or gypsum wallboard must comply with Table 720.1(2) of the IBC.

A.1.8 Wood Framing: Framing with either gypsum wallboard or gypsum plaster must comply with Table 720.1(2) of the IBC.

A.2 WALL CONFIGURATION:

A.2.1 Adhered Veneer: Metal lath is installed in compliance with Section 2506.1 of the UBC. Where lath is attached to steel framing, minimum 1-inch-long (25.4 mm), No. 6 drywall screws are used. For exterior walls, a weather-resistive barrier described in Section 2510.6 of the IBC is required. The Portland cement plaster is applied in compliance with Sections 2510, 2511, and 2512 of the IBC to a minimum $\frac{3}{4}$ -inch (19.1 mm) thickness. The thin veneer units are applied in compliance with Section 1405.9 of the IBC in running bond. For one-hour fire resistance, the total thickness of plaster, mortar and brick veneer shall be at least $1\frac{3}{4}$ inches (45 mm). For two-hour fire resistance, the total thickness of plaster, mortar and thin brick shall be at least 2 inches (51 mm).

A.2.2 Anchored Veneer: Anchored veneer is installed in compliance with Section 1405.5 of the IBC for 2-to-5-inch-thick (51 to 127 mm) units and Section 1405.6 of the IBC for units up to 10 inches (254 mm) thick. Stud spacing is limited to 16 inches (406 mm) on center, and a weather-resistive barrier complying with Section 1403.2 of the IBC is required on the exterior side of exterior walls. Anchored units may be used for one-hour or two-hour fire-resistive assemblies.

A.3 INSTALLATION:

Details of one-hour and two-hour nonbearing walls are noted in Figures 5 and 6. For symmetrical one-hour fire resistance, each face shall have not less than one layer of $\frac{5}{8}$ -inch-thick (15.9 mm) Type X gypsum wallboard, or equivalent gypsum plaster, or $1\frac{3}{4}$ inch (45 mm) thickness of masonry veneer. For two-hour fire resistance, each face shall have not less than two layers of $\frac{5}{8}$ inch-thick (15.9 mm) Type X gypsum wallboard, or equivalent gypsum plaster, or 2 inch (51 mm) thickness of masonry veneer.

A.3.1 Structural Brick Veneer and Structural Load Bearing Brick: Both structural brick systems are installed by combining brick, mortar, reinforcing and grout to provide a structure that combines load resistance to fire resistance. Structural brick fire ratings are generally defined by their equivalent thickness - the more solid the wall, the higher the fire rating. The International Building Code defines the minimum equivalent thickness required to achieve various fire ratings from 1 to 4 hours. Structural brick are defined as hollow without insulation and hollow

with insulation. Hollow units with insulation are to be filled solid with insulation materials and the equivalent thickness is the actual thickness of the wall.

A.4 FIRE RATED WALL SECTIONS

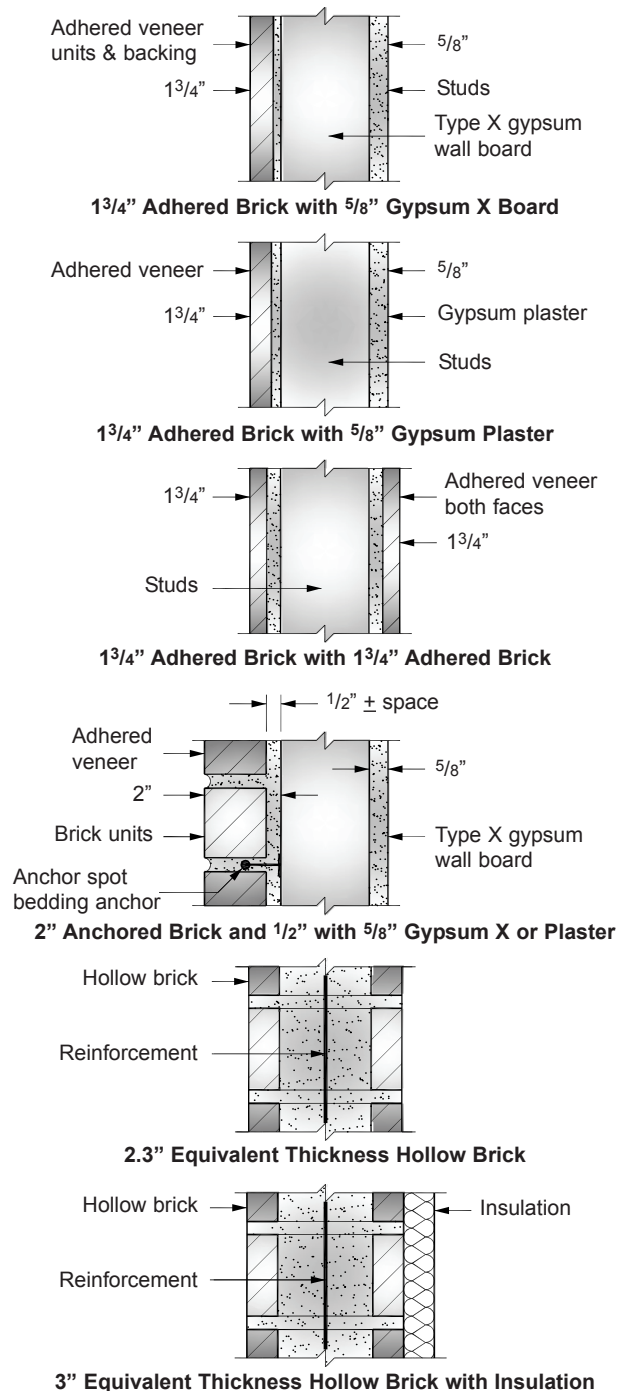


Figure 5 1-hour wall sections.

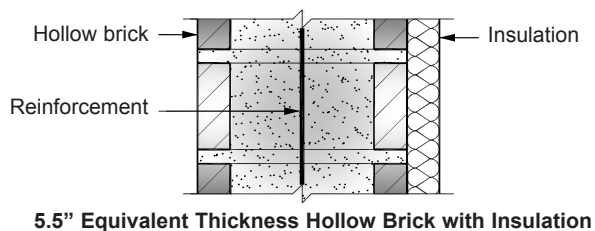
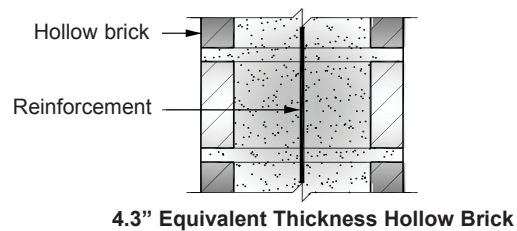
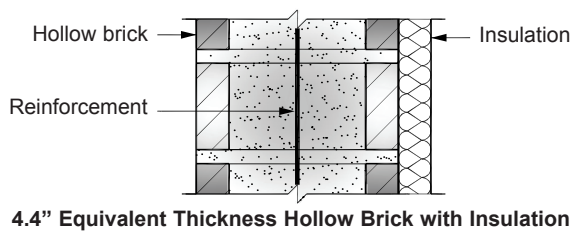
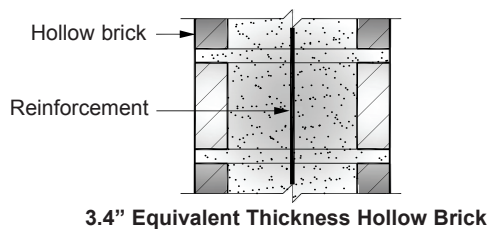
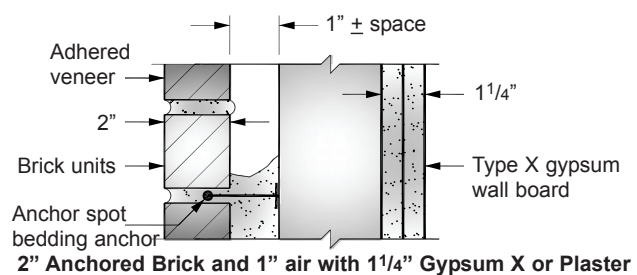
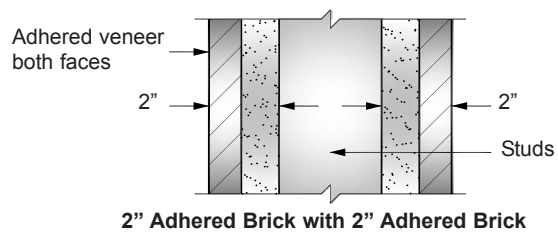
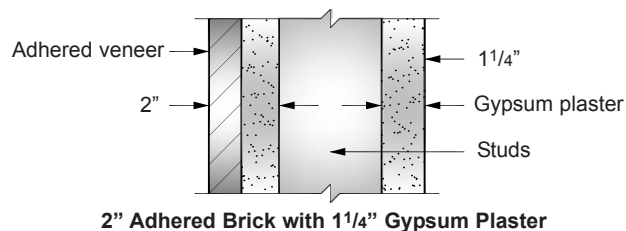
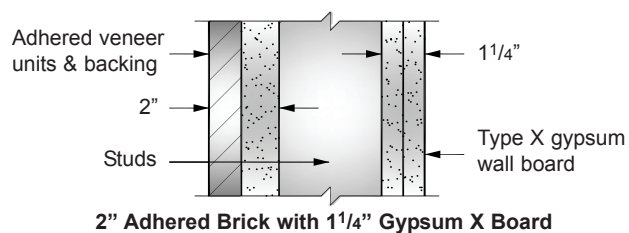


Figure 7 3-hour wall sections.

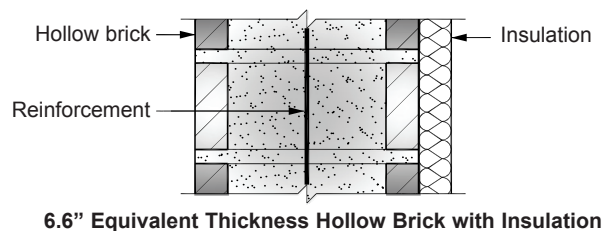
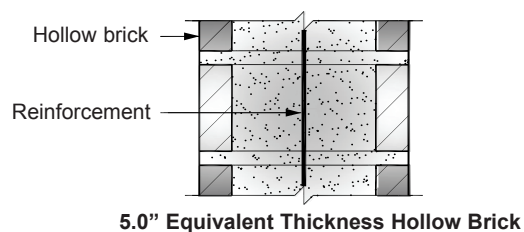
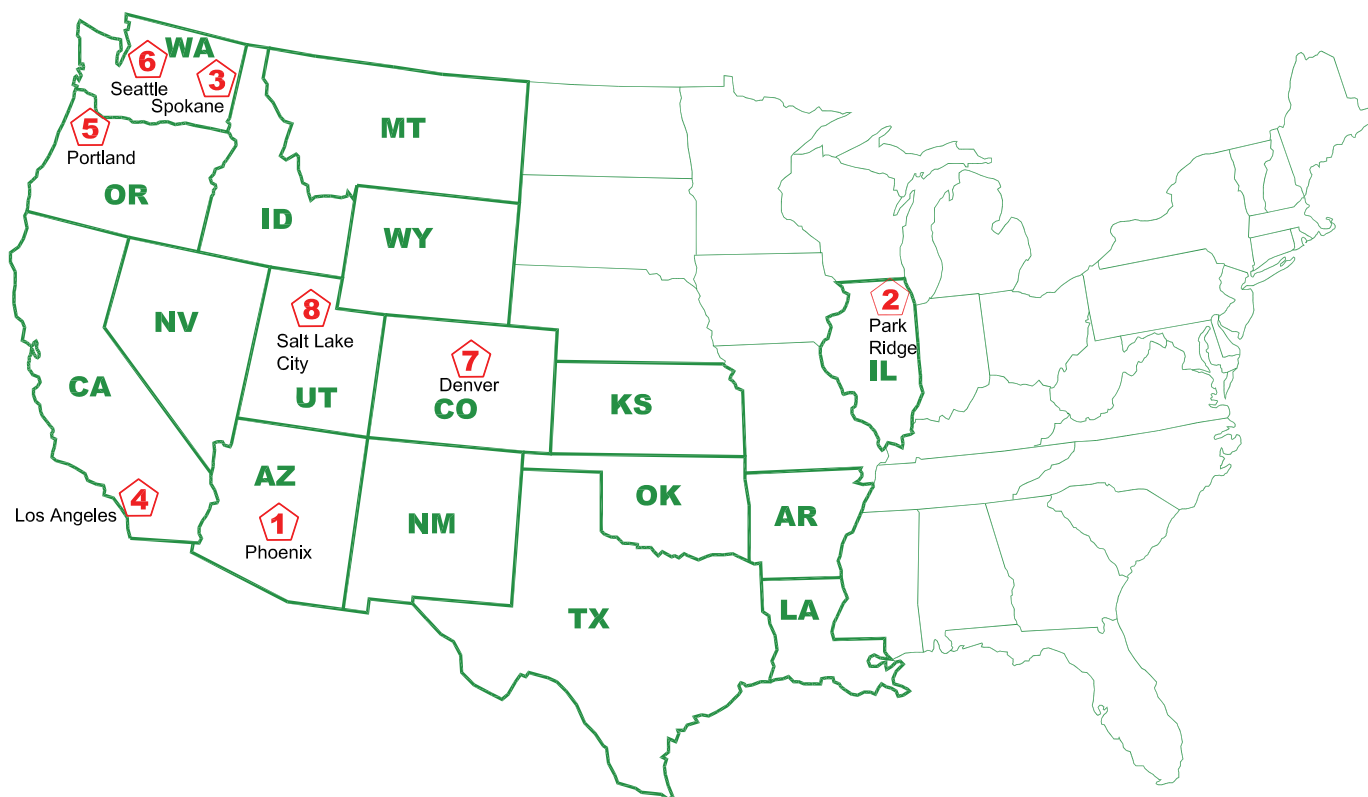


Figure 8 4-hour wall sections.

Figure 6 2-hour wall sections.

ALLIED ASSOCIATES AND WEB ADDRESSES

- | | | |
|----------|---|--|
| 1 | Arizona Masonry Guild | www.masonryforlife.com |
| 2 | Masonry Advisory Council | www.maconline.org |
| 3 | Masonry Industry Promotion Group | www.masonrypromotion.com |
| 4 | Masonry Institute of America | www.masonryinstitute.org |
| 5 | Masonry Institute of Oregon | www.mioctio.org |
| 6 | Masonry Institute of Washington | www.masonryinstitute.com |
| 7 | Rocky Mountain Masonry Institute | www.rmmi.org |
| 8 | Utah Masonry Council | www.utahmasonrycouncil.org |





2580 South West Temple
Salt Lake City, UT 84115
P-801-487-1333--F-801-487-7141

atsrae@qwestoffice.net
www.americantesting.net

Interstate Brick
9780 South 5200 West
West Jordan, Utah 84088-5625

Report #: 2022-3

Date: March 25, 2022

Work Order #: 3596

Dimensions: 8x4x16 - 0440			Test Date: March 21, 2022			
Sample Color: Sawgrass			ASTM Test Method: ASTM C67			
Lab Mix #: 1288			ASTM Classification: ASTM C216 & ASTM C652			
ASTM C 652-19b Test Report for Hollow Brick (Hollow Brick Masonry Units (Made from Clay or Shale))						
	Height	Width	Length	*Dimensions shown are the brick after being cut for testing*		
	3.6250	7.6250	7.6875			
ABSORPTION						
Sample Number	1	2	3	4	5	AVERAGE
Oven Dry Weight (grams)	3653.5	3521.5	3389.6	3664.3	3845.5	
Immersed Weight	2071.3	1944	2035.5	2067.6	2166.5	
Cold Water (SSD)	3913	3768	3848.5	3909.1	4095.6	
5 hour Boil (SSD)	4025	3879.5	3960	4020.6	4217.8	
24 hr. Cold Water (%)	7.1	7.0	13.5	6.7	6.5	
5 hr. Boiling Water (%)	10.17	10.17	16.83	9.72	9.68	
Saturation Coefficient	0.70	0.69	0.80	0.69	0.67	0.71
I.R.A (Suction)						
Sample Number	1	2	3	4	5	AVERAGE
Brick Net Area	30.80	30.80	30.80	30.80	30.80	
Oven Dry Weight (grams)	3568.9	3587.4	3670	3594.4	3719.1	
1 Minute Gain in Weight	3581.1	3600.2	3681.8	3606.9	3733.5	
I.R.A (grams/min/30in. ²)	11.9	12.5	11.5	12.2	14.0	
EFFLORESCENCE						
Sample Number	E1	E2	E3	E4	E5	AVERAGE
Examination and Rating	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced
After drying both control and testing brick, brick was determined to have not effloresced						
CUT BRICK-COMPRESSIVE STRENGTH (Batch #:29198)						
Sample Number	1	2	3	4	5	AVERAGE
Brick Gross Area (in ²)	58.60	58.60	58.60	58.60	58.60	
Brick Net Area (in ²)	30.80	30.80	30.80	30.80	30.80	
Maximum Load (lbf)	387,900	472,010	417,600	436,970	409,740	
Net Compressive Strength (psi)	12,594	15,325	13,558	14,187	13,303	
Gross Compressive Strength (psi)	6,619	8,055	7,126	7,457	6,992	
Tested Specimens Meet the Physical Requirements of ASTM Specification C652 Designation: Grade SW						
Tested Specimens Meet the Physical Requirements of ASTM Specification C216 Designation: Grade SW						

CERTIFIED TEST COPY:

Raelynn Gomez, Lab Manager
AMERICAN TESTING SERVICES, INC.



2580 South West Temple
Salt Lake City, UT 84115
P-801-487-1333--F-801-487-7141

atsrae@qwestoffice.net
www.americantesting.net

Interstate Brick
9780 South 5200 West
West Jordan, Utah 84088-5625

Report #: 2024-10
Date: July 1, 2024
Work Order #: 4543
PO# 45-03690257

Dimensions: 8x4x16			Test Date: July 1, 2024			
Sample Color: Mountain Red Atlas			ASTM Test Method: ASTM C67			
Lab Mix #: 1343			ASTM Classification: ASTM C216 & ASTM C652			
ASTM C 652-19b Test Report for Hollow Brick (Hollow Brick Masonry Units (Made from Clay or Shale))						
	Height	Width	Length	*Dimensions shown are the brick after being cut for testing*		
	3.6875	7.6250	7.1250			
ABSORPTION						
Sample Number	1	2	3	4	5	AVERAGE
Oven Dry Weight (grams)	3590.6	3602.3	3594	3584.5	3526.2	
Immersed Weight	2065.3	2068.8	2066	2060.4	2028.2	
Cold Water (SSD)	3804.5	3804.6	3799	3794.5	3724.6	
5 hour Boil (SSD)	3907	3906.6	3899.8	3896.8	3825.4	
24 hr. Cold Water (%)	5.95	5.62	5.70	5.86	5.63	
5 hr. Boiling Water (%)	8.81	8.45	8.51	8.71	8.49	
Saturation Coefficient	0.68	0.66	0.67	0.67	0.66	0.67
I.R.A (Suction)						
Sample Number	1	2	3	4	5	AVERAGE
Brick Net Area	28.79	28.74	28.69	28.71	28.08	
Oven Dry Weight (grams)	3590.6	3602.3	3594	3584.5	3526.2	
1 Minute Gain in Weight	3599.9	3610.5	3601.9	3593.1	3534.8	
I.R.A (grams/min/30in. ²)	9.80	8.56	8.26	8.98	9.18	
EFFLORESCENCE						
Sample Number	E1	E2	E3	E4	E5	AVERAGE
Examination and Rating	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced
After drying both control and testing brick, brick was determined to have not effloresced						
CUT BRICK-COMPRESSIVE STRENGTH (Batch #:31637)						
Sample Number	6	7	8	9	10	AVERAGE
Brick Gross Area (in ²)	54.32	54.32	54.32	54.32	54.32	
Brick Net Area (in ²)	28.60	28.60	28.60	28.60	28.60	
Maximum Load (lbf)	409,150	403,510	398,190	391,640	388,240	
Net Compressive Strength (psi)	14,316	14,108	13,922	13,693	13,574	
Gross Compressive Strength (psi)	7,532	7,428	7,330	7,209	7,147	
Tested Specimens Meet the Physical Requirements of ASTM Specification C652 Designation: Grade SW						
Tested Specimens Meet the Physical Requirements of ASTM Specification C216 Designation: Grade SW						

CERTIFIED TEST COPY:

Raelynn Gomez, Lab Manager
AMERICAN TESTING SERVICES, INC.



2580 South West Temple
Salt Lake City, UT 84115
P-801-487-1333--F-801-487-7141

atsrae@qwestoffice.net
www.americantesting.net

Interstate Brick
9780 South 5200 West
West Jordan, Utah 84088-5625

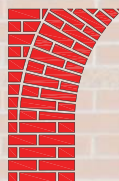
Report #: 2025-18
Drop Off Date: May 28, 2025
Test Date: July 2, 2025
Ticket #: 7325
Po#: 4503868009

Dimensions: 8x4x16				Test Date: July 2, 2025			
Sample Color: Terra Cotta				ASTM Test Method: ASTM C67			
Lab Mix #: 1427				ASTM Classification: ASTM C902 & ASTM C652			
ASTM C 652-19b Test Report for Hollow Brick (Hollow Brick Masonry Units (Made from Clay or Shale))							
		Height	Width	Length	*Dimensions shown are the brick after being cut for testing*		
		3.6875	7.625	7.250			
ABSORPTION							
Sample Number		1	2	3	4	5	AVERAGE
Oven Dry Weight (grams)		3515.7	3481.3	3479.5	3492.8	3531.7	
Immersed Weight		2049.3	2028.8	2041.2	2041.2	2064.5	
Cold Water (SSD)		3775.9	3748.3	3764.2	3767.8	3808.9	
5 hour Boil (SSD)		3861.3	3835.3	3848.2	3850.1	3896.6	
24 hr. Cold Water (%)		7.4	7.6	8.2	7.9	7.8	
5 hr. Boiling Water (%)		9.8	10.2	10.6	10.2	10.3	
Saturation Coefficient		0.75	0.75	0.77	0.77	0.76	0.76
I.R.A (Suction)							
Sample Number		1	2	3	4	5	AVERAGE
Brick Net Area		28.58	28.47	28.52	28.58	28.88	
Oven Dry Weight (grams)		3515.7	3481.3	3479.5	3492.8	3531.7	
1 Minute Gain in Weight		35260.0	3491.4	3490.3	3502.6	3543.1	
I.R.A (grams/min/30in. ²)		10.8	10.6	11.4	10.3	11.8	
EFFLORESCENCE							
Sample Number		E1	E2	E3	E4	E5	AVERAGE
Examination and Rating		Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced	Non Effloresced
After drying both control and testing brick, brick was determined to have not effloresced							
CUT BRICK-COMPRESSIVE STRENGTH (Batch #:31296)							
Sample Number		6	7	8	9	10	AVERAGE
Brick Gross Area (in²)		55.28	55.28	55.28	55.28	55.28	
Brick Net Area (in²)		28.61	28.61	28.61	28.61	28.61	
Maximum Load (lb.)		355,640	359,900	332,300	322,410	317,400	
Net Compressive Strength (psi)		12,430	12,579	11,614	11,269	11,094	
Gross Compressive Strength (psi)		6,433	6,510	6,011	5,832	5,741	
Tested Specimens Meet the Physical Requirements of ASTM Specification C652 Designation: Grade SW							
Tested Specimens Meet the Physical Requirements of ASTM Specification C902 Designation: Class SX Type II							

CERTIFIED TEST COPY:

Raelynn Gomez, Lab Manager
AMERICAN TESTING SERVICES, INC.

NOTES ON THE SELECTION, DESIGN AND CONSTRUCTION OF REINFORCED HOLLOW CLAY MASONRY



WESTERN STATES CLAY
PRODUCTS ASSOCIATION

Acknowledgements

This document was sponsored by the Western States Clay Products Association and written by John G. Tawresey, formerly of KPFF Consulting Engineers, with review and comments by John Hochwalt, KPFF Consulting Engineers, and the Technical Publication and Promotion Committee of the Western States Clay Products Association. The committee members are Steve Judd, Jim Buckley, John Chrysler, Brian Healow, Jeff McNear, Don Sackett, Jim Welte, Joe Welte, and Ann Wolter. Their contribution was very valuable and greatly appreciated.

The material presented in this publication, including technical and engineering data, figures, and tables, is for general information only. It should not be used or relied upon for specific applications without independent evaluation by a licensed design professional familiar with the particular use and application. Anyone using this material does so at their own risk and assumes any liability resulting from such use.

Table of Contents

Introduction	4
What is RHCM?	5
Document Scope	6
Example Projects	7
Basic Applications	7
Reinforced Hollow Clay Masonry Header for Veneer	7
Reinforced Hollow Clay Masonry Retaining Walls	8
RHCM Soffits, Sills, and Cantilevers	8
Reinforced Hollow Clay Masonry Sound Walls	9
Advanced Applications	9
Schools	9
Commercial/Retail	10
Small Office Buildings	10
Single-Family Residential	10
Multi-Story Residential/Hotels	11
Institutional Facilities	11
Other Applications	12
Small Bridges	12
Blast Walls	12
Storm Shelters	12
Curtainwall or Reinforced Veneer, or Structural Brick Veneer	12
Brick Panels	13
Concrete Form	13
Schematic Design	14
Selection of the Structural System	14
Advantages	14
Sustainability and Resilience	14
Expansion Property	14
High Strength	15
Form and Structure	15
Speed of Construction	15
Low Maintenance Cost	15
Aesthetic Flexibility	15
Objections	15
Unfamiliarity with RHCM	15
Higher Cost	15
Water Penetration	15
Another Subcontractor	15
Unreliable Color	15
Inexperienced with Design Methods	16
Inconsistent Information	16
Inconsistent Standard Details	16
Prices are not predictable	16
Complicated Building Code	16
Selection of Materials	16
Initial Design Criteria	17
Dimensions	17
Wall Thicknesses	18

Design Development	19
Bond Pattern	19
Initial Sizing of Members	21
<i>Selection of the Design Strength</i>	21
<i>Selection of Wall Thickness</i>	22
Out-of-Plane	22
Compression	23
Lateral Forces	23
Distribution of Base Shear	24
<i>Selection of Beam or Lintel Size</i>	25
Specifications	26
<i>Specification of Materials</i>	26
Units	26
Mortar	26
Grout	26
Sand	27
Cement	27
Lime	27
Reinforcement	27
Quality Assurance	27
Designer Choices	29
<i>Allowable Stress Design versus Strength Design</i>	29
<i>Fully Grouted Versus Partial Grouted</i>	29
<i>Multiple Lift Versus Single Lift Grouting</i>	29
Clean-outs	30
<i>Horizontal Joint Reinforcement Versus Bond Beams</i>	30
Construction Documents	31
Structural Analysis	31
<i>Design Assumptions</i>	32
General	32
Walls	32
<i>Walls Flexure Plus Compression</i>	33
Shear Walls	33
Columns	34
Pilasters	35
Bar laps and Embedment	35
Deflection	35
Maximum Bar Size	35
Retaining Walls	36
Bearing, Bolts, and Shear Friction	36
Drawing Preparation	37
Layout	37
Structural Notes	37
Details	37
Water and Flashing	38
Connections	38
Movement Joints	38
Specifications	39

Construction	41
Bidding and Award	41
Susmittal Review	42
Pre-Construction	44
Site Visits	45
Non-Conforming Quality Control Tests	45
<i>Prism Compression Strength – Prior to Construction</i>	45
<i>Prism Compression Strength – During Construction</i>	45
<i>Mortar Compression</i>	45
<i>Grout Compression</i>	45
APPENDIX A: WSCPA Members	46
APPENDIX B: Drawing Checklist	47
APPENDIX C: Construction Observation Checklist	48
APPENDIX D: Field Test data for Mortar and Grout	50
APPENDIX E: Troubleshooting Table	52

Introduction

Reinforced Hollow Clay Masonry (RHCM), also known as “Structural Clay Brick” and “Reinforced Structural Clay Units,” offers the building owner many advantages. Unfortunately, the structural engineer often overlooks these advantages. When the structural engineer offers the client alternative structural systems, steel, concrete, concrete masonry, brick veneer, concrete masonry, and wood are usually considered, but reinforced hollow brick masonry is often not.

Most structural engineers are familiar with steel, concrete, wood, and concrete masonry design. RHCM design is similar to concrete masonry design, but there are important differences.

Knowledge of RHCM structural design provides the structural engineer with a meaningful design niche.

Recognizing this lack of knowledge, the Western States Clay Products Association (WSCPA) agreed to produce this RHCM publication specifically for the structural engineer. The first edition was published in 1995, followed by a revision in 1997. This is the third edition.

The document emphasizes the information required to successfully complete a RHCM project, the first being to convince the owner, architect, and sometimes the contractor to consider the RHCM option.

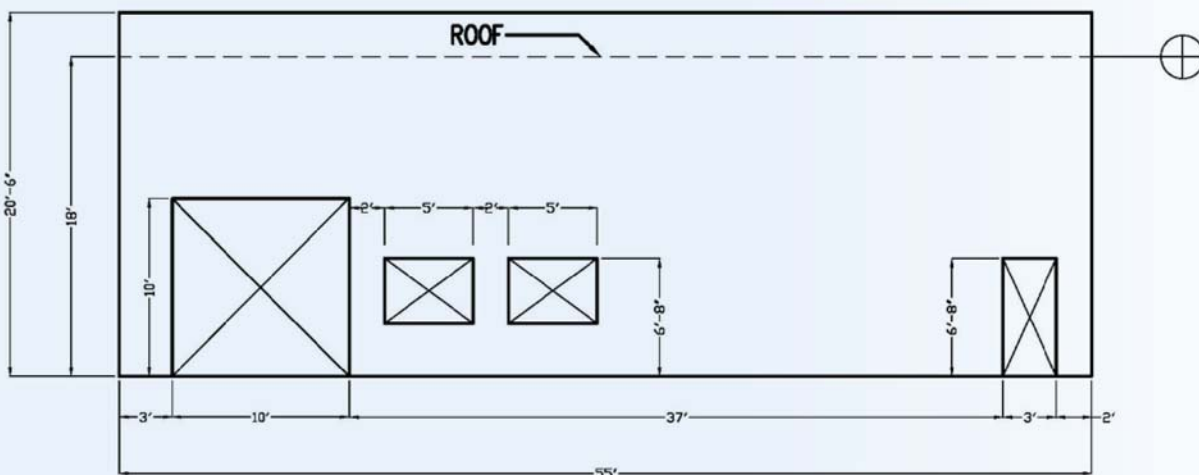
The document begins with example projects followed by helpful information during the traditional project phases: Schematic Design, Design Development, Construction Documents, and Construction.

Cost is always a factor. Suggesting RHCM will usually result in the client saying: “How much does it cost?” Costs are difficult to predict. Here is one example. For the brick wall below* costs was developed by a general contractor and a mason contractor in the Pacific Northwest in August 2022.

Two options were considered; a brick veneer on steel studs and RHCM. The brick veneer required an 8-inch structural stud backing, while the RHCM required only interior partition (furring) studs. The brick veneer option costs \$59.10 per square foot, and the RHCM option costs \$57.33 per square foot.

This cost study only included the wall. Additional savings exist if the RHCM wall supports the roof and provides lateral resistance.

Analogous to unreinforced concrete, unreinforced brick masonry is not cost-effective. Adding reinforcement reduces cost. The purpose of this document is to provide the structural engineer with the knowledge to utilize the RHCM design niche.



*Figure 1 Cost Comparison - Brick Veneer on Metal Studs versus Reinforced Hollow Clay Masonry

What is RHCM?

The most common shapes of hollow clay units (sometimes referred to as structural clay units) are shown below. There are many shapes and sizes available. Custom shapes may be designed, fabricated, and installed on larger, more geometrically complex projects.

Hollow clay units are produced by extruding a mixture of various clays that determine color, strength, water absorption, and saturation coefficient. Different clays are combined to a fine powder, water is added, and the mixture is extruded through a steel die. The extruded column of clay is then sliced perpendicular to the extruded column at the desired unit height. The units are then placed on carts and generally fired to temperatures above 2000 F degrees in a kiln.

Following firing in the kiln, units are placed on pallets for shipping. Units are transported from the manufacturing plant to the project by truck or rail.

Masons place the units in mortar, and reinforcement is placed in the cells and bond beams. The resulting wall is grouted to bind the units, grout, and reinforcement to form the structural wall. In some situations, insulation can be placed in the ungrouted cells to enhance the thermal properties of the wall.

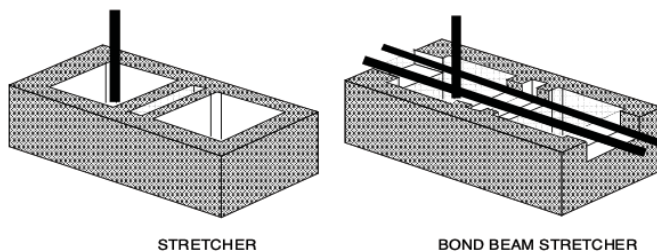


Figure 2 Typical Reinforced Hollow Clay Unit



Figure 3 Hollow Clay Units (Left: 7 3/8" x 7 3/8" x 15 3/8", Right: 3 1/2" x 2 1/2" x 7 1/2", and 3 1/2" x 3 1/2" x 11 1/2")

Document Scope

Detailed information about structural design methods is not included in this document. Instead, this document contains “rules of thumb” to assist in creating the design. More detailed information on structural design methods can be found in numerous textbooks and other references. Two resources available are the Reinforced Masonry Engineering Handbook and the Masonry Designers Guide. These resources contain example designs with example structural analysis. They can be found at The Masonry Society Library.

Design software is also available from many sources.

Mostly, the information is based on Building Code Requirements and Specifications for Masonry Structures, The Masonry Society 402/602. TMS 402/602 is adopted by reference in ASCE 7 and the IBC with minor overriding provisions.

It is intended that most of the information presented is independent of the code edition selected. Specific code references are not included as code provisions change with time and may be revised by the local jurisdiction. In this document, there are statements that a provision is required by code. This statement means that the provision has existed in the code for many years or may be new and is likely to continue being a code provision. But, the provision may or may not apply to your project and may not be current. The structural engineer must check current code provisions applicable to the project based on the jurisdiction having authority.

The traditional design and construction project delivery system (design-bid-build) are assumed. In the traditional project delivery system, the structural engineer contracts with and reports to the project architect as a specialty consultant. The architect typically contracts with and reports to the owner. The contractor is selected after the design is completed, and the

selection is often based on price. The contractor enters into an agreement with the owner and reports to the architect serving as the owner’s representative.

Other project delivery methods exist, such as the Construction Manager/General Contractor (CM/GC) and Design-Build. Because there is usually more interaction with contractors in these alternative methods, converting a project to RHCM may be easier. However, for this publication, the traditional approach is assumed.



Figure 4 Reinforced Hollow Clay Masonry Wall with Shear Keys for the Slab Connection

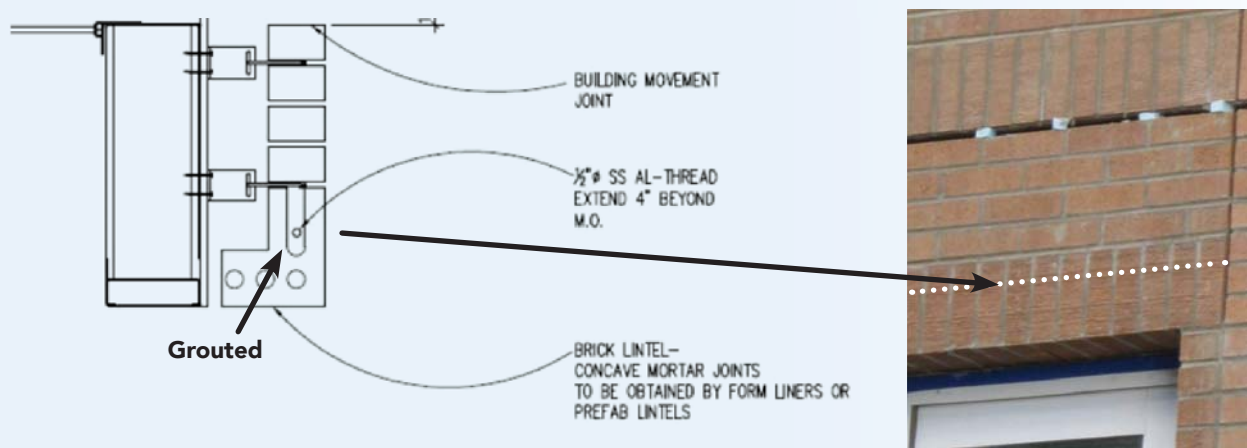


Figure 5 RHCM Lintel

Example Projects

Basic Applications

Reinforced Hollow Clay Masonry Header for Veneer

Brick veneer over various veneer backings is often used for a building's exterior wall. Above openings, the brick veneer is usually supported by steel angles called loose lintels prone to rusting. Observing the rusted angle above an opening makes it easy to determine that a wall is brick veneer. There is a better design.



Figure 6 Custom Lintel Unit for an Inset Window

Replace the steel loose lintel angle with a Reinforce Hollow Clay Masonry lintel. The simplest design is a half-hollow clay soldier. A horizontal reinforcing bar is placed in the cell, and the cell is grouted.

Because the reinforced lintel is typically exposed to water on both sides, it is recommended to use a stainless steel all-thread for the reinforcement. Stainless steel all-thread is much less expensive than stainless steel reinforcement and is readily available. For most applications, a 3/8-inch or 1/2-inch diameter all-thread is sufficient.

Construction is accomplished by shoring the opening, usually with wood framing. Some masons elect to prefabricate the lintel as a RHCM panel, either on-site or off-site, in a controlled environment.

Reinforced Hollow Clay Masonry Retaining Walls

Many retaining walls are constructed using concrete. Concrete forms often use brick pattern forms for the exposed surface. Why not use brick instead?

Typical hollow clay masonry design strengths (ranging between 2600 and 4000 psi) match that of concrete (higher strength than concrete masonry). Reinforcement requirements are comparable to concrete.

A significant cost advantage is the geometric flexibility of RHCM. Hollow clay masonry units are laid one unit at a time. There is flexibility to have articulated shapes for a small increase in cost.



Figure 7 Articulated Retaining Wall

The serpentine retaining wall is a classic example of the relationship between form and structure. The geometry of a RHCM retaining wall can significantly increase the structural capacity and is more economical. Varying the geometry of a concrete retaining wall is expensive. RHCM can be considered similar to reinforced concrete with the forming left in place.

Additionally, RHCM expands, and concrete and concrete masonry shrinks. In RHCM, the expansion puts the reinforcement in tension, reducing cracking and water penetration. Also, a brick retaining wall can be built from one side, often minimizing the expense of the back-side excavation.



Figure 8 RHCM Retaining Wall

RHCM Soffits, Sills, and Cantilevers

Brick veneer walls may include brick soffits, sills, and cantilevers. Instead of complicated veneer backing, RHCM can be used.

Spanning between connectors to the structure, the hollow clay masonry transforms the veneer into a structural element, eliminating the structural veneer backing. Some local building codes restrict the use of veneers on overhead horizontal surfaces. RHCM would be code compliant and solve the problem.

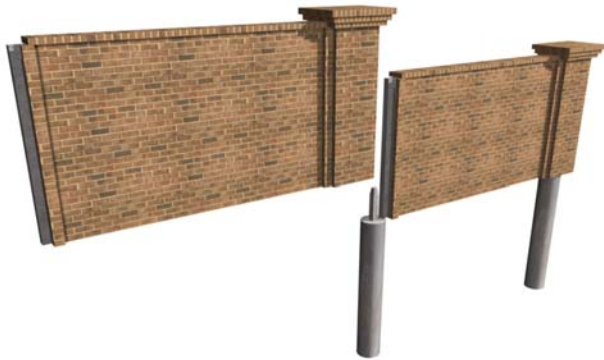


Figure 9 RHCM Cantilevered Exterior Wall

Reinforced Hollow Clay Masonry Sound Walls

RHCM sound barrier walls are cost-effective. In addition, the flexibility of color and form offers cost and aesthetic advantages.

Two schemes of sound walls are recognized, the cantilever wall and the pier and panel wall. The cantilever has a continuous footing, and the wall is designed to cantilever from the footing. The pier and panel wall has a foundation below the pier but does not require a continuous footing under the wall panels, a significant saving. The piers are designed to support the loading from the wall panel between the piers.



The Sound Transmission Class of a reinforced hollow clay wall is comparable to a concrete wall. The sound transmission class rating can be found in TMS Standard 0302-12 and Brick Industry Association Technical Note 45.



Figure 10 Pier and Panel Sound Wall

Advanced Applications

Schools

For aesthetic reasons, brick is often preferred for the exterior of a school. Often architects will opt for a brick veneer which they believe costs less. However, brick veneer has a limited life unless properly designed with items like stainless steel ties.

Additionally, school story heights add significant costs to the veneer backup system. When the veneer is converted to an RHCM structural system, the backup wall can remain inexpensive, and the brick wall can be part of the building's primary structure. There are many examples of the RHCM wall costing less than brick veneer walls.

Interior RHCM hall and classroom walls provide fire resistance and a long-lasting, durable surface that resists abuse and requires little maintenance.



Figure 11 RHCM School

Commercial/Retail

One- and two-story commercial buildings, retail malls, or other commercial developments can frequently benefit from RHCM construction. As an alternative to concrete masonry, there is an increase in initial cost for a significant improvement in aesthetics, strength, and durability.

As an alternative to a steel frame with metal stud infill walls, RHCM masonry can be used as both a bearing wall, shear wall, and enclosure wall providing structure and exterior finish simultaneously.

Long lead times for structural steel detailing and fabrication can increase the construction schedule. The construction of RHCM can usually begin shortly after the design is complete.



Figure 12 RHCM Retail Center

Small Office Buildings

The typical office building requires column-free spaces to provide flexibility for interior design. These buildings often employ moment frames or braced frames to resist lateral loads. Load-bearing walls are typically not used because they limit the flexibility of the interior space. However, RHCM shear walls can be substituted for steel

braced frames or moment frames to resist lateral force. Fire-rated stairs and fire-rated elevator shafts can utilize RHCM, providing up to four-hour fire resistance with an interior brick finish. This is especially desirable when shear walls become part of the exterior aesthetics of the building or permanent partitions.

Single-Family Residential



Figure 13 Brick House

RHCM can be substituted for wood frame construction in single-family residential projects. It offers better acoustical properties, less seismic or wind drift (deflection), better fire resistance, less maintenance, insect resistance, and hail resistance. Also, stiff masonry walls may provide better performance in expansive soil locations than flexible walls.

Multi-Story Residential/Hotels

RHCM is used in medium-rise residential projects, including hotels. RHCM exhibits high compressive strength. For buildings in the four-story to the 20-story range, the full strength can be used with little or no increase in cost.



Figure 13 RHCM Residential

The high strength can mean savings and offer an advantage over other forms of construction. The RHCM system is compatible with wood, precast, cast-in-place, or prestressed floor systems. Besides aesthetics, a primary benefit of RHCM bearing wall construction is the sound separation provided by the wall mass. RHCM dwelling unit dividing walls also provide reliable fire separation and thermal mass.



Figure 15 RHCM Hotel

Institutional Facilities

Brick is often the material of choice for libraries, hospitals, fire stations, city halls, and university buildings. In these situations, using RHCM offers an alternative to brick veneer. It can perform double duty when designed to be load-bearing and may provide savings.

Even if not designed to be load-bearing, RHCM offers better seismic isolation and longer life than traditional veneers. RHCM can be designed to isolate the wall from building movements in areas of high seismic exposure. This is difficult to accomplish with conventional brick veneer, particularly at the building corners.



Figure 16 RHCM University Building

Other Applications

Small Bridges



Figure 17 Small Bridge

Building a bridge can be limited by local conditions. A RHCM brick bridge can be built almost anywhere. The materials can be transported one brick at a time. Additionally, the flexible geometry allows for more efficient structural systems.

Blast Walls

Technical information about RHCM and brick blast walls is limited. At this time, it is not a typical application. ASCE/SEI Standard 59-11 Blast Protection of Buildings addresses material and design issues. Masonry is included and, except for minor differences, is equivalent to reinforced concrete. The referenced document addresses concrete masonry. Experience indicates that because of the significant increase in strength of hollow

clay masonry units over concrete masonry units, brick blast-resistant walls will perform equal to or better than concrete masonry blast-resistant walls.

The Department of Defense and the ASCE/SEI 59-11 prohibit using unreinforced or partially grouted masonry for blast resistance. This is because fragmentation of the units causes airborne debris.

Storm Shelters

Test panels made of fully grouted RHCM, produced by Interstate Brick Company, have been tested for debris impact. The test objective was to ensure compliance with a high-performance standard to protect shelter occupants from wind-borne debris. Performance criteria include preventing the perforation of the shelter by the

test missile and preventing damage that could cause injuries to the occupants.

The panels tested met the debris impact guidelines of FEMA 320/361 and ICC – 500 for a tornado shelter. Again, remember that the flexibility of wall geometry gives RHCM a significant cost advantage over reinforced concrete.

Curtainwall or Reinforced Veneer, or Structural Brick Veneer

Brick veneer on metal studs is a popular curtainwall system. An alternative system is an RHCM curtainwall or reinforced veneer. A reinforced veneer is similar to a conventional veneer, except the wall is constructed with RHCM. Typical closely spaced veneer ties are replaced with intermittent connectors spaced farther apart. A typical brick veneer has ties every two square feet. Reinforced veneers have connectors every 100 square feet. The connectors carry more load, are constructed of galvanized thicker materials, offer simpler methods for isolating the veneer from the primary structure, provide less thermal bridging, and are more corrosion-resistant.

A design guide for reinforced veneer or structural brick veneer is available at the Western States Clay Products

website (www.brick-wscpa.org/technical-publications.php, Design Guide for Structural Brick Veneer, Third Addition).

A reinforced veneer is sometimes called “laid-in-place panels.”



Figure 18 RHCM Reinforced Veneer Hospital Exterior Wall in a High Seismic Zone

Brick Panels

Brick panels are an application for RHCM. Brick panels are constructed of hollow clay units, reinforced, fully grouted, and attached to the building in a manner comparable to a precast concrete panel. The system can speed construction, eliminate the requirement for scaffolding, and provide the designer with new opportunities not available with conventional masonry. The system has been used on many buildings ranging from single-story branch banks to 50-story office buildings.

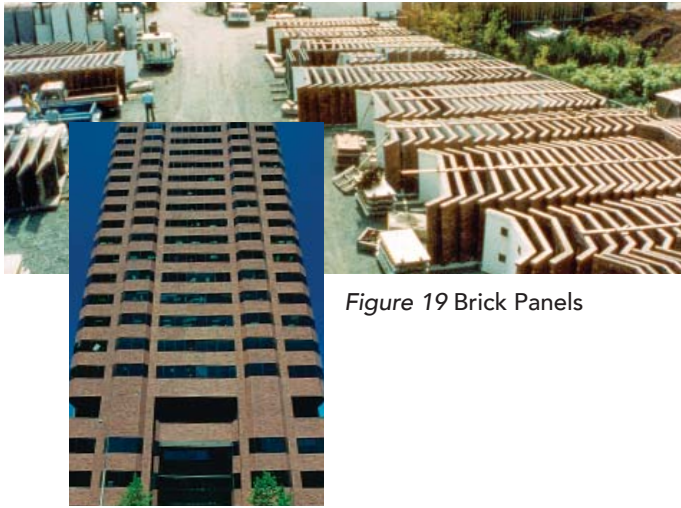
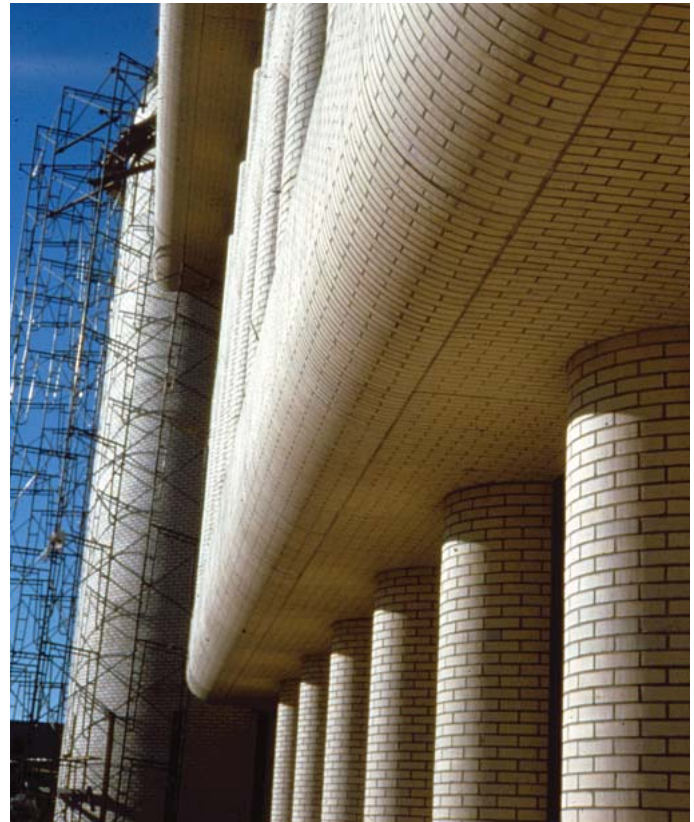


Figure 19 Brick Panels



Concrete Form

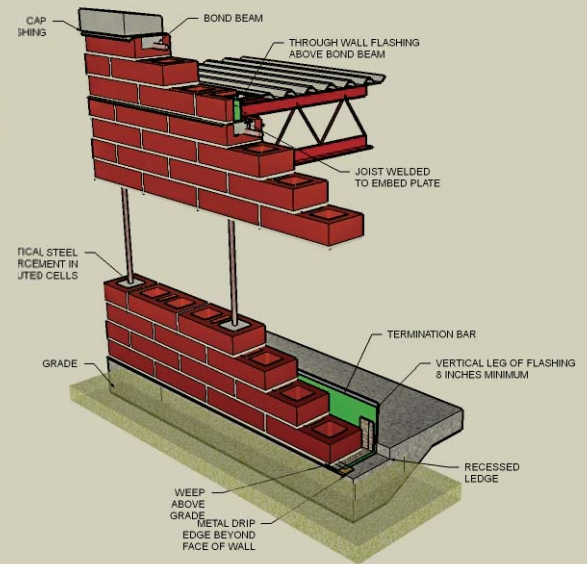
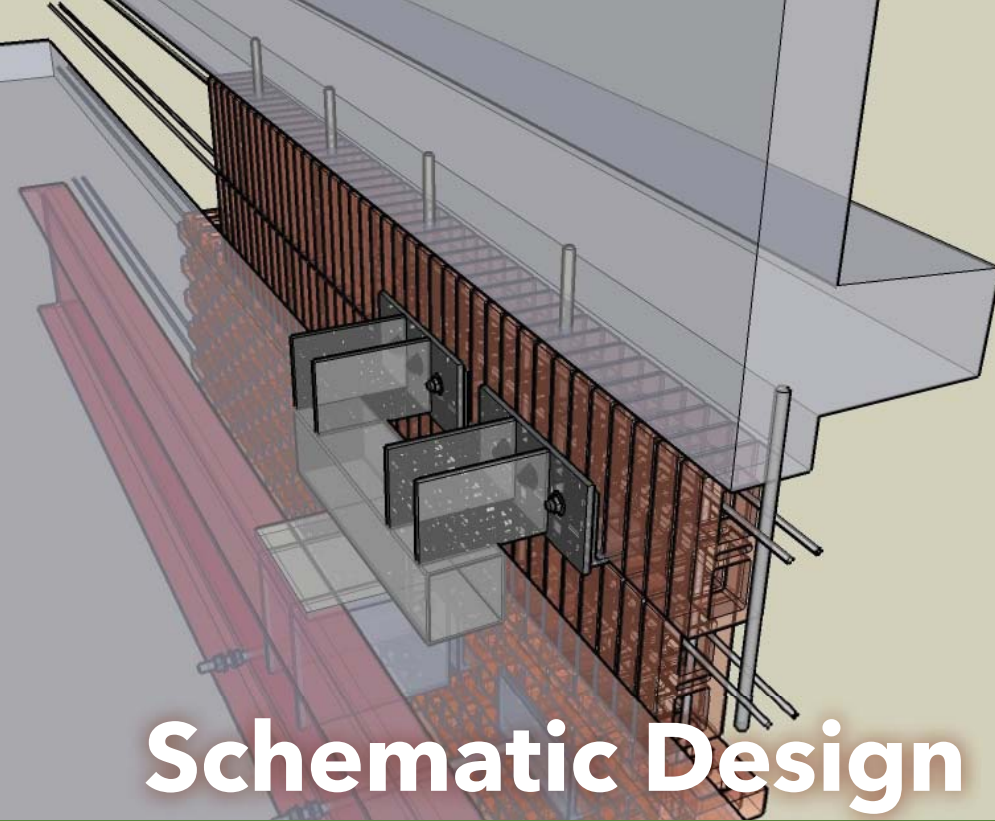
RHCM or, in some cases, unreinforced solid brick can be used as a form for concrete. When a brick exterior is desired, the brick can become the form for concrete structural columns and spandrels. In addition to increasing the element's capacity (the brick concrete form is structural and is not removed), the geometry flexibility offers interesting opportunities for unique designs.



Figure 20 RHCM Columns and Spandrels Used Both as a Form and for Structure



Figure 21 RHCM Used as the Finish Wall and Backing for Concrete Shotcrete (Guniting)



Schematic Design

Selection of the Structural System

During the schematic design phase, the architect evaluates the owner's needs and develops the project concept. Multiple design concepts are explored and developed during this phase. The structural engineer is expected to influence the selection of a structural system with choices that are cost effective for the owner.

The architect and owner will likely have defined a preliminary site plan and layout before the structural engineer's involvement. For example, if the site plan includes retaining walls, the structural engineer should include the retaining walls in the design scope, increasing the fee if possible, and suggest RHCM as a cost-saving option.

Suppose the exterior skin (curtainwall) has not been defined, and the building is of institutional quality or

in a high seismic or hurricane zone. In that case, RHCM curtainwall (reinforced veneer) may be an option for significant cost reduction and enhanced performance. Reinforced veneer usually costs less than conventional brick veneer on steel studs if the story height is over 12 feet.

During the selection dialogue, the following advantages and objections could arise.

Advantages

Sustainability and Resilience

Unlike many modern construction materials, fired clay (bricks) lasts thousands of years in the environment. They are sustainable. Additionally, bricks can be recycled either by reuse or by adding ground-up bricks to clays for making new bricks.

Resilience can be developed through the geometric flexibility of RHCM. Adding intersecting walls and flanges to shear walls will increase resilience at a minimum cost.

Expansion Property

Brick masonry expands with time. This can take as long as five years. It is not reversible. In RHCM, this property places the brick in compression and the steel in tension. The result is reduced cracking and improved water penetration resistance. Expansion joints can be placed further apart or, in some cases, eliminated.

High Strength

It is relatively easy to obtain high-strength hollow clay masonry. Design strengths (f'_m) of 4,000 psi are recognized in TMS 602 Specification based on an 11,500 psi clay unit net area compressive strength. Typical unit strengths are between 10,000 and 15,000 psi. Higher unit strengths are common.

Darker colored bricks generally have higher strengths and lower absorption than lighter colored bricks. There is typically no cost premium for these higher strengths, although there may be local limitations on available colors. In load-bearing applications, the higher strengths offer an advantage over concrete masonry units.

The tensile strength of the brick masonry unit range between 700 and 1500 psi. Brick cracking in structural walls is less common than in concrete masonry (CMU).

Form and Structure

Structural capacity can be significantly increased by modifying geometry. For example, adding a pilaster to the wall. The cost to modify geometry is minimal when bricks are laid one at a time.

Speed of Construction

Most masonry materials, including hollow brick, are readily available on short notice. Therefore, it is not necessary to wait for steel fabrication.

Low Maintenance Cost

RHCM is relatively maintenance-free. Expected life far exceeds the life of metal and other light exterior wall systems. RHCM does not rot. Insects do not eat RHCM.

Aesthetic Flexibility

There are thousands of colors and textures from which to choose. The many brick placement orientations can create unique patterns that challenge the architect's creativity. Fewer expansion (or control) joints (if any) are required. Special shapes can be produced to further broaden the aesthetic possibilities.

Building a bridge can be limited by local conditions. A RHCM brick bridge can be built almost anywhere. The materials can be transported one brick at a time. Additionally, the flexible geometry allows for more efficient structural systems.

Objections

RHCM is viewed as masonry in general and carries many of the perceived negatives often associated with masonry.

Below are a few of the common objections. However, despite these objections, RHCM has been successfully used on many projects for the past 80 years and should continue to receive consideration in many projects.

Unfamiliarity with RHCM

Many designers, both architects, and engineers, are not familiar with RHCM. A simple solution is to consider RHCM as concrete masonry with the CMU units replaced with brick units.

Higher Cost

RHCM costs more than concrete masonry. But it may cost less than brick veneer on metal studs. When floor heights exceed 12 feet, many examples exist where an RHCM with only an interior partition costs less than a brick veneer with a structural steel stud backup system. This is because the backup structural steel stud wall cost exceeds the increased brick structural cost.

Water Penetration

Masonry is often perceived to leak more than other wall systems. This has proven to be inaccurate for RHCM, particularly when grouted solid. The expansion of the brick masonry against the reinforcement tends to reduce cracking and, thus, reduce leakage significantly.

Another Subcontractor

Adding the mason adds another subcontractor to the job. General contractors often resist adding a masonry subcontractor to the job. One reason is the cost of administering the masonry work. Another reason exists if the general contractor performs their own concrete work. In this situation, they will prefer concrete walls instead of masonry walls to keep a larger percentage of the project.

Unreliable Color

Colors are variable and sometimes not what was expected. Many architects have experienced difficulty in obtaining the colors desired. To a large degree, many of these problems don't exist today. More sophisticated manufacturing processes and adequate color mock-ups have nearly eliminated issues in this area.

Inexperienced with Design Methods

Many structural engineers are not familiar with masonry design methods. There are many new resources (literature and software) available. This document is one example.

Inconsistent Information

Sometimes technical answers obtained from suppliers appear inconsistent. There are many different types of masonry. There are many different uses. Communication between the masonry industry and the designer often needs clarification because of misunderstandings of intended use and terminology. Veneer brick masonry is not designed and constructed like load-bearing brick masonry. CMU masonry behaves very differently from fired clay masonry. One typical example: wetting clay units before laying is often recommended, while it is not recommended to wet CMU units before laying.

Inconsistent Standard Details

Many standard details are available, but the problem is similar to the apparent inconsistency of technical answers. Since there are so many different types of masonry, methods, and systems, a single set of standard details is not possible. Further, details vary by region. Most local masonry institutes have recommended details available.

Prices are not predictable.

Price varies from job to job. Unfortunately, it is difficult to predict the price of a masonry job. Since masonry work is labor intensive, the mason's production is a key factor. Many variables come into play. The cost of the bricks remains relatively predictable and can be obtained from the supplier.

Complicated Building Code

There are many different types of masonry and many different applications. Persistence in understanding the applicable provisions of the code is required. This document is intended to help.

Selection of Materials

When RHCM is selected for the structural system, the next step is to choose the specific type of units to be used. The choices of shape, color, texture, and pattern are almost unlimited.

Thousands of colors exist. Almost any shape can be produced. Units can be shipped thousands of miles.

Local suppliers can provide information on the available strengths, shapes, and colors. The manufacturers who

are members of the Western States Clay Products Association are listed in Appendix A.

Initial Design Criteria

The likely applicable code is The Masonry Society (TMS) 402/602 *Building Code Requirements and Specification for Masonry Structures*. It is adopted by reference in the International Building Code. Associated design loads are found in the *International Building Code* and the *American Society of Civil Engineers (ASCE 7) Minimum Design Loads for Buildings and Other Structures*.

TMS 402/602 includes two design methods: Allowable Stress Design and Strength Design. The Allowable Stress Design and Strength Design methods are divided into Reinforced Masonry Design and Unreinforced Masonry Design. Hollow clay masonry can be designed as reinforced masonry or unreinforced masonry using either Allowable Stress Design or Strength Design.

The code provisions for Unreinforced Design, Reinforced Design, Strength Design, and Allowable Stress Design may be used in combination. For many years ASCE 7 has restricted combining Allowable Stress Design and Strength Design. But the IBC has typically not adopted the ASCE 7 restriction. Before combining methods, the structural engineer must verify if the mixing of design methods is allowed within the project jurisdiction.

Dimensions

It is traditional to describe the masonry unit dimensions in the order of thickness, height, and length. For example, a 6 x 4 x 12 unit is a nominal 6-inch-thick, 4-inch high, and 12-inch long unit. Nominal dimensions are one mortar joint thickness larger than the specified brick dimensions. The typical mortar joint thickness for RHCM is either 3/8 inch or 1/2 inch, depending on the manufacturer (the typical mortar joint thickness for concrete masonry units is 3/8 inch). Thus, a nominal 4-inch height unit laid with a 1/2-inch mortar joint has a specified height dimension of 3 1/2 inches.

Standard nominal thicknesses are 4, 5, 6, and 8 inches. The availability of 10 and 12-inch thick units may be limited. The typical nominal height is 4 inches (other heights are available), and the nominal length is 12 inches or 16 inches. Smaller standard-size brick units (for example, 3 1/2 x 2 1/2 x 7 1/2) can be manufactured with cells large enough to be reinforced.

Hollow clay units are produced in accordance with ASTM C 652 H60V, where the sum of the void area is greater than 40 percent but less than 60 percent of the gross cross-sectional area.

Hollow clay masonry units have cells and cores. A cell is different from a core. A cell's cross-section exceeds 1 1/2 square inches. A core's cross-section is less than 1 1/2 square inches. Cores do not need to be grouted for a wall to be classified as fully grouted, but all cells need to be grouted. Reinforcement is not generally placed in cores.

Hollow clay masonry units are easily reinforced, both vertically and horizontally. Vertical reinforcement is placed in the cells that align vertically. Horizontal reinforcement is placed in bond beams that align horizontally. Smaller standard-size units manufactured with cells can also be easily reinforced. Diagonal reinforcement is very difficult to construct and is almost never designed.

Special units can be manufactured for custom patterns. The additional cost for custom units, shapes, and colors on large projects is generally small. A project that offers the opportunity for custom units provides the architect and structural engineer with an opportunity for creative, sustainable, and resilient design. One example is the custom units used in a large hospital for a six-inch thick wall that visually projected a standard four-inch corner.

For analysis, the specified dimensions of the masonry are used, except the nominal dimensions are used for the following:

1. Effective flange width.
2. Reinforcement spacing.
3. Maximum size of reinforcement based on the thickness of the unit.
4. Height-to-thickness ratios for determining design compression strength.
5. Minimum dimensions (Example: Minimum side dimension of a column is eight nominal inches).

It is desirable, but not necessary, that the plan dimensions match the unit module. Cutting units is common but can significantly add to the wall's cost. Moreover, a wall or pier of limited length (say, 14 inches long) could pose a problem with aligning cells for reinforcement.

Generally, the masonry contractor can adjust dimensions by plus or minus 1/8 inch per stretcher. This can be accomplished by unit selection or by adjusting mortar joint widths. It should be recognized that units typically are not exactly the same thickness, height, or length. Variations in specified dimensions are addressed in ASTM C 652. The dimension of a darker unit will generally be smaller than the dimension of a lighter unit. Because the thickness varies, hollow clay walls are typically laid to line on one exposed face. When both sides of the wall are exposed and considered finished, additional costs may result, either because of the increased cost for more uniform units or the potential increased cost to lay the units.

A floor height that does not divide evenly by the brick height is not a big problem. Usually, the bed joints can be adjusted. Sometimes the brick is cut. Gabled walls are typically constructed by cutting the units at an angle.

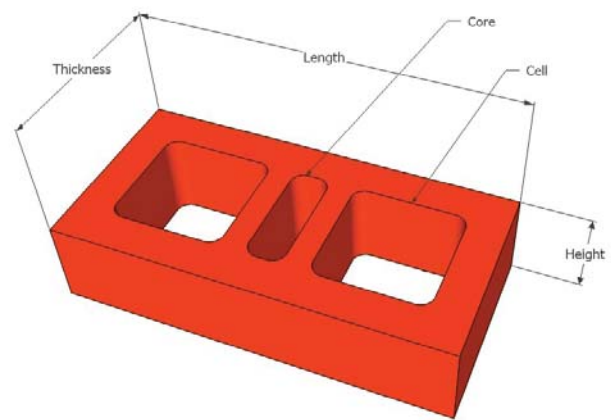


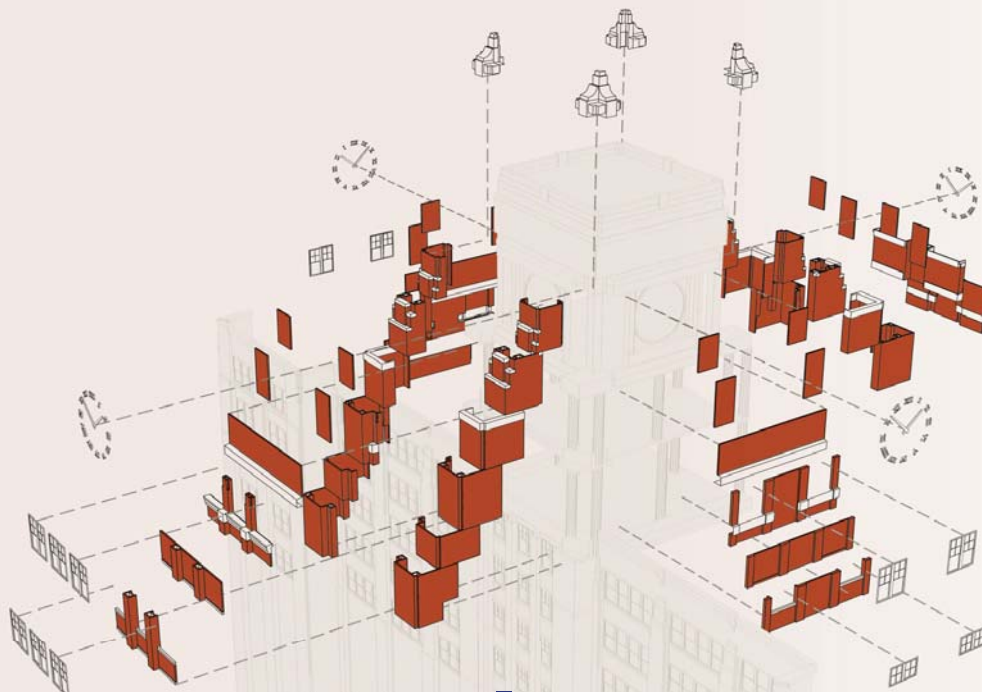
Figure 22 Dimension Order: Thickness, Height and Length



Figure 23 Custom Unit for a Four Inch Corner in a Six Inch Wall

Wall Thicknesses

It is common to use different thicknesses of walls on the same floor or to change the wall thickness from floor to floor.



Design Development

During the design development phase, the design team prepares documents consisting of drawings and specifications to fix and describe the size and character of the entire project.

During this phase, the structural engineer identifies the space required for the structural portions of the project and the elements of the structure that will control the

required strength of the materials, in addition to a beginning draft of the specifications.

Bond Pattern

The building code recognizes only two bonding patterns, running bond and other than running bond (formerly defined as stack bond and that term to be used herein). For running bond, the units must overlap by more than one-quarter of the unit length. If the overlap is less than one-quarter, the pattern is classified as other than running bond for design purposes, or stack bond. The code has lower capacities for stack bond and includes prescriptive requirements for its use.

For RHCM and CMU masonry, the head joint is normally face shell bedded instead of full bedding. The structural engineer should assume that the face shell bedded head joint has no shear or tension capacity. In running bond, the head joints stair step in the wall. The code design capacities take this into account. In stack bond, the head joints line up in the wall, causing a structural discontinuity. The code provision limit capacity accordingly.

There are many bonding patterns other than stack and running bond. The orientation of the units provides the architect with many creative designs.

There are names for the orientation of units in a wall. The un-hatched face is the face exposed to view.

For unreinforced masonry, bed joint code capacities are higher than head joint code capacities. The code defines the bed joint as the horizontal layer of mortar on which the masonry is laid. The capacity is higher because of the compression on the joint during curing supplied by gravity. For example, for a soldier orientation of a unit, the head joint is the end web.

Typically, the architect will select the bonding pattern. Many designs have multiple bond patterns. Some patterns can have a significant impact on the structural design.

When the head joint limits the capacity required, using open-end units (units with the end webs removed) and fully grouting will solve the problem.

The bonding pattern may appear to create a wall that cannot be reinforced and grouted. However, there are usually methods available. Typical methods are:

- Use a single bond beam or back-to-back bond beams for vertical reinforcing in soldier courses.
- Use precast concrete or cast stone with holes (like brick cells) to pass vertical reinforcement. When using precast concrete, it is important to consider that concrete shrinks with age and that brick masonry expands. The precast should not be set until it is at least 28 days old, and the length of the precast should be limited to 10 feet with a soft caulk joint at the end. Placing sand on the surface of the caulk will make the caulk joint appear like the adjacent mortar joints.
- Use various brick thicknesses and special shapes to produce elegant walls. Rustication, quoins, and cornices are examples of a few designs that can be easily produced using different thicknesses of hollow clay.
- Work with the brick manufacturer to design custom extruded shapes.
- Use cut lintel blocks from the end of units for exposed sills and lintels.

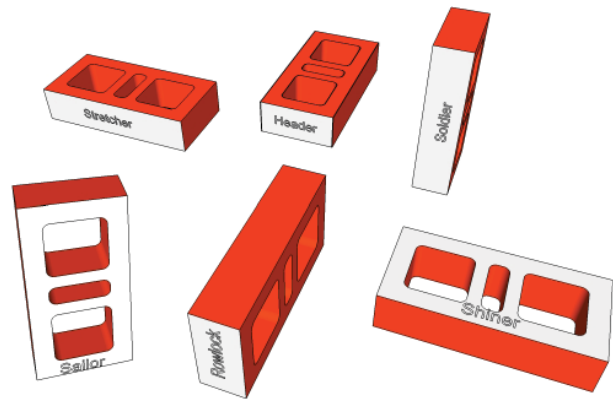


Figure 24 Unit Orientation Definitions

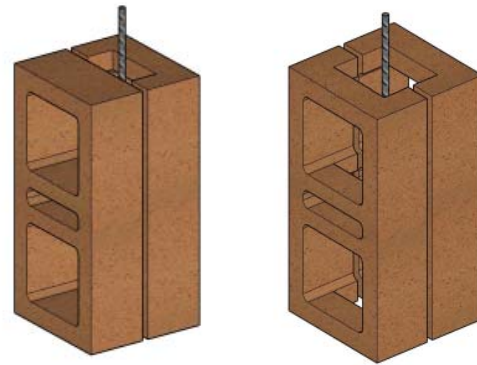


Figure 25 Soldier Reinforcement

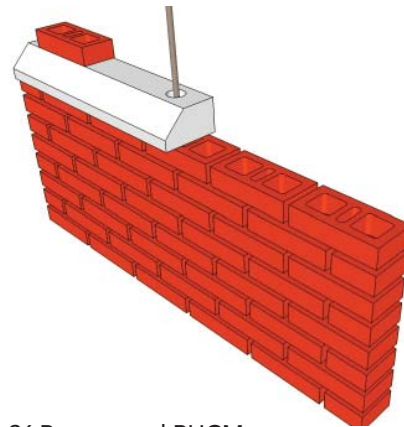


Figure 26 Precast and RHCM

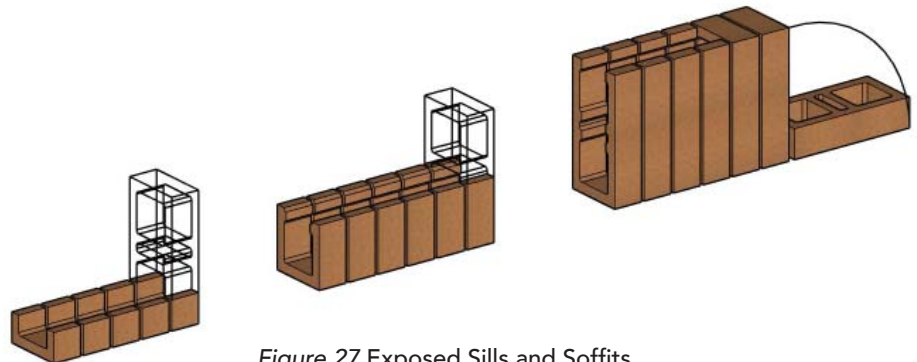


Figure 27 Exposed Sills and Soffits

Initial Sizing of Members

During meetings with the client, questions about member sizes will occur. Structural engineers tend to rely on their computer models, requiring additional time, instead of “back of the envelope” analysis on the spot. The following information will assist the engineer in quickly providing the client with the needed information.

Making money on the structural design directly correlates to the quality of the first guesses.

Designing a structure is an iterative process. It consists of guessing, checking, and resizing to meet code and performance expectations. Fortunately, whole numbers are used to select masonry member sizes, so the engineer doesn't need to run extensive analysis to get started.

Selection of the Design Strength

Masonry is composed of units, mortar, and grout. The f'_m specified is the required strength of the combination of these materials. A common incorrect assumption is that the weakest of the units, mortar or grout, determines the system's strength. This is not the case. It is common to have a combination of materials with a strength of 4000 psi, a grout strength of 2000 psi, and a mortar strength of 1800 psi. This is because laboratory or field tests, not in-the-wall tests, define the grout and mortar strengths. Laboratory and field-prepared grout and mortar test samples are cured in conditions quite different from those in the wall. The strengths in the wall are typically much higher.

Mortar and grout compression tests measure the water-to-cement ratio of the mix. In the wall, the units absorb much of the water from the grout and mortar, resulting in increased strength. Think of the unit as a sponge absorbing water. Accepted standards for the field tests of mortar and grout attempt to mimic this effect. The methods used are unreliable. Moreover, field tests of mortar and grout are sensitive to weather, temperature, sampling method, curing, handling, transportation, and testing method (Appendix D).

For this reason, the structural engineer should rely on the prism test as a measure of strength.

Unfortunately, it is rare to obtain prism test data during the design development phase. The masonry code unit strength method provides an alternative to prism testing and is recommended for the initial design.

In accordance with TMS 602, the specified strength of RHCM can be determined from the unit strength by the following equations.

$$f'_m = 0.3057 * (\text{the net area strength of the unit}) + 480.39$$

(Type M or S mortar)

$$f'_m = 0.2439 * (\text{the net area strength of the unit}) + 487.80$$

(Type N mortar)

For RHCM, the structural engineer can assume a design strength of 2,600 psi in most situations. Lower values for the hollow clay masonry are unlikely. Higher values can be obtained if required.

The modulus of elasticity for hollow clay masonry is 700 f'_m psi (for concrete masonry, it is 900 f'_m psi).

Commonly, one building element controls the required masonry strength. Reducing the strength of other elements not requiring the higher strength may or may not result in cost savings.

It is not unusual to change the wall strength or the unit thickness from floor to floor.

Changing the unit color may mean a change in the masonry strength. Consequently, the architect's choice of color may control the strength of the masonry.

Selection of Wall Thickness

Out-of-plane loads will likely control the exterior wall thickness, except for buildings over six stories, where the compression demand may control wall thickness, or in a high seismic zone, where the shear demand may control the wall thickness.

Out-of-Plane

Whether or not the architect has initially selected the wall thickness, the structural engineer will need to estimate the thickness required to resist the loads. A good initial guess can save a lot of design effort. The following table presents typical wall thicknesses for walls controlled by out-of-plane loading. Wind load is usually the controlling load (For 10-inch and thicker walls, seismic may control in high seismic areas).



Typical Height Limits for Out-of-Plane Loading

Nominal Wall Thickness (in.) ³	Height of Wall (ft.) ^{1,2}		
	Normal Application h/t = 24	Common Application h/t = 30	Extended Application h/t = 36
4"	8	10	12
5"	10	12	15
6"	12	15	18
8"	16	20	24
10"	20	25	30
12"	24	30	36

1. h = effective height, t = nominal thickness

2. Wind loading 45 psf with .0025 net area vertical reinforcement. ($f'_m = 2600$ psi)

3. 10 and 12-inch thick units may not be available - check with the manufacturer.

For walls with h/t ratios less than 24, there is significant capacity for compression (load bearing). Moment magnification resulting from out-of-plane deflection (small deflections) will have little or no effect on the design.

For walls with h/t ratios between 24 and 30, the compression capacity is reduced but still significant in most applications. Generally, the moment magnification factor will need to be considered in the design but will have a small effect.

For walls with h/t ratios between 30 and 36, the compression capacity is reduced, and it is necessary to consider moment magnification in the design. Walls in this category often have roof loads accumulating to a point along the wall (roof trusses). In these circumstances, the structural engineer should consider using pilasters (protruding inside, outside, or both) and span the masonry horizontally between pilasters.

Compression

Compression in the wall results from the dead load, live load, snow load, and overturning due to lateral wind or seismic forces.

The compression from dead, live, and snow loads can be easily determined by the tributary area method. The tributary area method requires engineering judgment. The dead and live load from each floor and the dead load, live load, and snow load on the roof are distributed by dividing the floor and roof into the areas between the walls (except in high snow load locations). Live load reduction should be used, and partition loads should be added if required by the code. Don't forget to include the weight of the wall.

Lateral Forces

Initial sizing for in-plane lateral loading is more complicated. Usually, the in-plane loads are due to wind or lateral seismic forces applied to the entire structure (main lateral force resisting system). For a regular geometry (the center of rigidity approximates the center of mass or the center of pressure), the first step is calculating the base shear. For irregular geometry, the first step is to educate the architect about the performance advantages of regular geometry, then determine the base shear.

Wind Base Shear

The base shear due to wind may be estimated as:

$$V_{base} = WA$$

Where W is the estimated average wind pressure on the building (generally, direct pressure on the windward elevation and plus suction on the leeward elevation) and A is the building area exposed to the wind pressure.

Seismic Base Shear

The layout of shear walls for resistance to seismic loads is important. Maintain the symmetry of shear walls. Getting the architect to add a shear wall to provide symmetry can significantly reduce the required design time. It will also improve the resilience of the building. Methods for seismic design of asymmetric geometry are available but require additional effort. Perhaps you should request an additional design fee if symmetry is not approximated.

For Seismic Design Category C and above, TMS 402 requires that along any line of lateral resistance, not more than 20 percent of the lateral resistance can be provided by columns unless the base shear is determined using a response modification factor (R) of 1.5 (not recognized by ASCE 7 or the IBC but the exception is likely within the normal standard of structural engineering care).

The base shear due to seismic can be estimated. Since Reinforced Hollow Clay Masonry buildings normally have a box system that resists seismic forces with shear walls, the estimating procedure is not very complicated. It can be estimated with hand calculations.

The base shear is:

$$V_{base} = C_s W$$

Where W is the seismic weight of the building (including partitions and sometimes snow. See ASCE 7 for the definition of seismic weight).

The value of C_s is determined by:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

Where S_{DS} is the short period design spectral response acceleration parameter, I_e is the Importance Factor, and R is the Response Modification Coefficient.

R = 5.0 for specially reinforced masonry shear walls (Seismic Design Category A, B, C, D, E, F)

R = 3.5 for intermediate reinforced masonry shear walls (Seismic Design Category A, B, C)

R = 2.0 for ordinary reinforced masonry shear walls (Seismic Design Category A, B, C)

C_s need not exceed:

$$C_s < \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)}$$

Where S_{D1} is the design spectral response acceleration parameter at a period of 1 second.

The building period can be estimated as follows:

$$T_a = .02h_n^{.75}$$

Where h_n is the height of the building, in feet.

And C_s must be greater than:

$$C_s = 0.044S_{DS}I_e \geq 0.01$$

And, if S_1 is equal to or greater than 0.6, then C_s shall be greater than:

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I_e}\right)}$$

Where S_1 is the 5% damped mapped maximum considered earthquake spectral response acceleration parameter at a period of 1 second.

Distribution of Base Shear

The base shear, either wind or seismic, is resisted by the in-plane forces in the building walls. The amount of load in each wall can be estimated as follows.

For flexible diaphragm buildings, the base shear is distributed by the tributary area method. Flexible diaphragm buildings are constructed with wood or metal deck without concrete topping floors and roof. The tributary area method requires engineering judgment. The force in each floor diaphragm is distributed by dividing the floor into areas that split the distance between the walls. The stiffness (rigidity) of the wall is not used.

For rigid diaphragm buildings, the load is distributed to the walls in relation to the relative rigidity of the wall. Rigid diaphragm buildings are constructed with concrete, concrete plank, or concrete over metal deck floors and roof. For these buildings with wall height to length greater than 3, distribute the base shears in proportion to $(H/L)^3$. The stiffness of these walls is dominated by flexure. For walls with (H/L) less than 3, distribute the base shears to the walls in proportion to (H/L) . The stiffness of these walls is dominated by shear. The value of H in the equations is the height from the base to the floor being considered.

Shear walls connected by beam elements at each floor or walls with punched window openings are sometimes designed as "coupled" shear walls. If coupled, for rigid diaphragm buildings, the length of the wall can be approximated as the sum of the walls adjacent to the coupling beam element.

Where the coupling beam stiffness is comparable to or greater than the wall stiffness, assume the wall is coupled; otherwise, assume individual walls. If coupled, the coupling beam elements have a small influence on the base shear distribution to the wall elements. The coupling beams, however, significantly affect the overturning moment. More coupling results in a lower moment because the piers and coupling beams act as a frame. This effect can be important for sizing the amount of vertical reinforcement in the wall and selecting the masonry strength.

In high seismic areas, coupling beams over interior and exterior doorways can be a problem. Distortion of the coupling beams prevents the doors from opening and traps the resident in their unit and the building.

Once the axial and shear loads are estimated, the building overturning moment can be estimated by assuming for wind that the lateral load is applied at the mid-height of the wall and for seismic that the lateral load is applied at two-thirds of the height of the wall.

Once the shear and moment are estimated, the following provides a guide for wall thickness when shear governs. This is usually the case when the H/L is less than 3. The values are conservative.

Wall Thickness for Shear

Nominal Wall Thickness (in.) ⁴	Approximate Shear Capacity (lb/ft) ³	
	Wind ¹	Seismic ²
4"	4,000	2,500
5"	5,000	3,300
6"	6,000	4,000
8"	8,000	5,300
10"	10,000	6,600
12"	12,000	8,000
1. Approximately $2\sqrt{2600}$ 2. Approximately $2\sqrt{2600}/1.5$ (Factor to increase flexural dominance of a wall) 3. Strength level and fully grouted construction 4. 10 and 12-inch thick units may not be available - check with the manufacturer.		

When H/L is high, the overturning moment will often dominate the initial design of the wall. Before developing the finite element model to distribute loads to the wall, an estimate of the wall thickness is needed. One method is to assume the wall consists of two piers, each with 20 percent of the wall length.

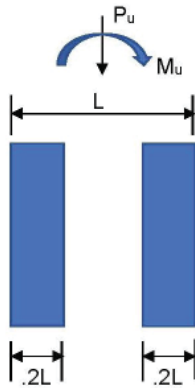


Figure 28 Model for Estimating the Design of a Wall

The axial load is divided between the two piers, and the moment is modeled as a coupled axial load on each pier. Assuming 60,000 psi reinforcement, the tension load is calculated as follows:

$$P_t = \frac{M_u}{.8L} - \frac{P_u}{2}$$

The required reinforcement is estimated as follows:

$$A_s = \frac{P_t}{60,000}$$

High strength reinforcement exceeding 60,000 psi yield is currently not allowed for masonry design. The area of reinforcement should appear reasonable for the wall. If not, the wall may need to be longer or more walls are needed.

The thickness of the wall will depend on the compression zone. An estimate of the compression stress can be obtained using the same model.

$$P_c = \frac{M_u}{.8L} + \frac{P_u}{2}$$

$$f_c = \frac{P_c}{.2t_{sp}L}$$

Where t_{sp} is the specified thickness of the wall, and L is the length of the wall. If the compression stress exceeds $.64 f'_m$, the wall needs to be increased in thickness and/or length.

One wall (or a small percentage of walls) will often not comply using the above initial method. Judgment must be employed to determine if the selected wall thickness is adequate. The above method is usually conservative, and with experience, the degree of conservatism can be readily estimated.

A more refined analysis is required during the construction document phase.

Selection of Beam or Lintel Size

An estimate of a beam or lintel size can also easily be obtained. These elements usually occur over wall openings. To estimate the beam or lintel requirements, first, calculate the applied moment. Normally the maximum moment results from full dead and live to load. If the beam is a shear wall coupling beam, then combined dead load plus seismic load normally controls.

The area of steel can be estimated as follows:

$$A_s = \frac{M_u}{.9F_y d}$$

It is best to use the same size bars in the beam or lintel as those bars used in the wall. Contractors sometimes take bars from the wrong pile. It is good practice to minimize the number of different bar sizes to simplify procurement, job site storage, and reinforcement handling.

Specifications

Specification of Materials

Units

Specify units to conform to ASTM C 652 with the minimum unit net area compression strength determined by the design requirements. ASTM C 652 requires a gross area compressive strength, not a net area compressive strength.

Mortar

Specify mortar as Type S, Portland cement-lime by proportions per ASTM C 270. Do not specify a mortar strength. Do not specify Type M mortar because Type M mortar is too hard and usually causes a crack between the unit and the mortar. For most jobs, masonry cement or mortar cement mortars are allowed by code, but local masonry preferences need to be investigated.

Preblended mortars (proportions of the ingredients are blended by the supplier with water added at the site) are specified per ASTM C 1714/C 1714M. They meet the requirements of ASTM C 270.

Do not use additives in mortar, except colors in accordance with manufacturers' recommendations.

Color additives to mortar generally do not reduce mortar bond strength or compression strength. Most color additives consist of mineral oxides except carbon black, which consists of finely ground carbon. Carbon black (for grey mortars) should be limited to 2% of the weight of the cement. When using colored mortar, follow the recommendations of the color additive manufacturer.

Grout

Specify grout per ASTM C 476 for regular or self-consolidating grout. Specify grout strength (f'_g) equal to f'_m but not less than 2,000 psi (required by code). The ASTM C 476 proportions will result in strengths well in excess of 2,000 psi. Lime may be added to the grout. This usually improves the grout properties by increasing the flow and retention of water, resulting in improved placement and bonding to the unit.

Grout is normally placed at a slump of 8 to 11 inches. The high-water content is permitted because the masonry units absorb the water like a sponge before the grout sets.



Figure 29 Fluid Grout

Consolidation and Re-Consolidation

TMS 402/602 requires mechanical vibration of the grout during placement (consolidation) and, at a later time, mechanical vibration to reconsolidate.

Like concrete, mechanical vibration is used during the grout placement to increase the flow and decrease the chance of grout voids. For fluid fine grout, mechanical vibration during placement (consolidation) is likely unnecessary, and a code modification may be requested if performance is demonstrated with a grout demonstration panel.

Reconsolidation is unique to masonry. After the grout is placed in the unit, the unit absorbs water from the grout. Since the water content of the grout is high (8 to 11-inch slump or more), the volume of water removed results in voids in the grout. These voids occur somewhere between 5 minutes and forty-five minutes after the grout is placed. The amount of time depends on many factors, including the absorption of the units, the grout mix, the weather, and the height of the pour.

Reconsolidation is accomplished by placing a vibrator one foot or less into the top of the grout column for each grouted cell. When successfully done, the top of the grout will settle below its previous position, often settling 2 to 4 inches for a single lift and 4 to 8 inches for multiple lift grouting.

In some situations, mechanical reconsolidation of fine grout in small cells may not be necessary. A code modification may be appropriate if performance is demonstrated with a grout demonstration panel.

Self-consolidating grout mixed to a slump flow of 24 to 30 inches, measured in conformance to ASTM C 1611/C 1611M, may be used. For self-consolidating grout, consolidation, and re-consolidation are not required.

Sand

Specify sand per ASTM C 144. Use mortar sand for fine grout. Mortar sand is finer and produces grout that flows better, particularly in 4-inch and 5-inch hollow units. Some sands are composed of rounded granules instead of sharp granules. The rounded sands can be used in grout with less cement and water to obtain equivalent flow compared to grouts with sharper sands. Use only clean, washed sand.

Coarse aggregate for grout

Specify coarse aggregate per ASTM C404. Use only clean, washed aggregate. Coarse aggregate is not allowed in fine grout; either fine regular grout or fine self-consolidating grout.

Cement

Specify cement to conform to ASTM C 150, Types I, II, or III. Do not use air-entrained cement types IA, IIA or IIIA, etc. Low alkali cement reduces the tendency for the masonry to effloresce. If available, they are recommended.

Lime

Specify lime to conform to ASTM C 207. Do not use air entrained lime, as it will reduce the bond strength between the unit and mortar.

Reinforcement

ASTM A 615 Grade 60 reinforcement is normally used. When welding reinforcement, use ASTM A 706 bars. High strength reinforcing with specified yield strengths in excess of 60,000 psi is not currently allowed for reinforced masonry design.

Quality Assurance

IBC and TMS 402 require a minimum level of quality assurance to be included in the contract documents. It is recommended that the minimum quality assurance requirements be included in both the structural notes and the project specifications. The quality assurance program itemizes the requirements for verifying conformance with the specified materials, material storage and handling, and construction execution. It shall also set forth the procedures for reporting and review and include procedures for the resolution of non-compliance.

TMS 402/602 presents three levels of quality assurance. Level 1 is for prescriptive design methods such as veneers, glass block, and partition walls. Level 2 and Level 3 are for engineered masonry design methods that include RHCM. Level 2 is for Risk Categories I, II, or III. Level 3 is for Risk Category IV. Risk categories are defined by ASCE/SEI 7.

The primary difference between Level 2 and Level 3 quality control is that Level 2 allows more periodic inspections or less continuous inspections. The following table summarizes the quality assurance program code requirements with additional recommendations for RHCM. R means required by code. NR means not required by code but may be required for the project. P means periodic inspection. C means continuous inspection.

Quality Assurance Program

		Level 2	Level 3
Before construction			
	Verify masonry strength by prism test or the unit strength method.	R	R
	Verify compliance with mortar and grout specifications.	R	R
	Verify compliance with submittal requirements.	R	R
	Verify the grade and size of reinforcement, connectors, and anchor bolts.	R	R
	Verify and observe the construction of the sample panel, if required.	R	R
	Require a sample grout panel.	Recommended	Recommended
	Require a pre-construction meeting with the mason contractor and others.	Recommended	Recommended
During construction			
	Require verification of masonry strength every 5000 square feet of the wall.	NR	R
	Require verification that preblended mortar and grout meet specifications and submittals.	NR	R
	Require inspection of site-prepared mortar and grout.	P	P
	If self-consolidating grout, verification of conformance to the required slump flow and visual stability index is required.	R	R
	Require inspection of the grade and size of reinforcement, connectors, and anchor bolts to verify conformance to the design.	P	P
	Require inspection to verify the proper placement of masonry units and mortar joint construction.	P	P
	Require inspection to verify the size and location of structural members.	P	P
	Require inspection of the welding of reinforcement	C	C
	Require inspection to verify the type, size, and location of anchors, including other details of anchorage of masonry to structural members, frames, or other construction.	P	P
	Require inspection for protection of masonry for hot and cold weather construction.	P	P
	Require inspection of the preparation of prisms and, if required, the preparation of mortar and grout specimens.	P	C
Prior to and during grouting			
	Require inspection and verification that the size of the grout space meets the specification.	P	C
	Require inspection of the size and placement of reinforcement, connections, and anchor bolts to verify conformance to the design.	P	C
	Require inspection during the placement of grout.	C	C

In addition, the quality assurance program shall also define the qualifications for testing laboratories and inspection agencies. ASTM C 1093 defines the duties and responsibilities of testing agency personnel and the technical requirements for equipment used to test masonry materials. Testing agencies qualified to test masonry are accredited or inspected for conformance to the requirements of ASTM C 1093 by a recognized evaluation authority.

It is recommended that the following be required before construction to conform to code requirements.:

1. A letter of certification provided by the unit manufacturer stating that the units meet the required strength.
2. When the mortar is not site prepared, a letter of certification provided by the mortar supplier stating that the mortar complies with the requirements.
3. When the grout is not site prepared, a letter of certification provided by the grout supplier stating that the grout complies with the requirements.
4. Prism tests per ASTM C 1314

It is recommended that prism tests be conducted during construction and for every 5000 square feet of masonry placed. Mortar and grout should be verified in the field by checking that the proper proportions are used.

Field mortar and grout tests should not be specified. They are unreliable and too variable due to the difficulty in controlling water content.

Designer Choices

Allowable Stress Design versus Strength Design

Allowable stress or working stress design was the traditional design method for hollow clay masonry. Standards and codes are now available for strength design. For some projects, the use of allowable stress design provides some advantages, or the use of strength design provides other advantages. For example, in the current codes, the maximum shear wall flexural reinforcement in ASD is less restrictive than in SD, while ASD does not have the slender wall provisions contained in SD. Care must be exercised when combining the two design methods on the same project.

Fully Grouted Versus Partial Grouted

A fully grouted wall means that all the cells are filled with grout.

Fully grouted walls perform better than partially grouted walls. Fully grouted walls resist water penetration and cracking. Sound transmission is less in fully grouted walls, a property of the wall's mass. The wall is more able to resist freeze damage because there are fewer voids in the wall.

Partial grouting requires blocking the flow of grout into the areas not intended for grout. This adds cost to the wall. However, there is an offsetting decrease in the cost of the grout material. Most contractors agree that it is less expensive to fully grout when vertical bars are spaced closer than 24 to 30 inches on center.

It is easier, and therefore takes less time, to analyze a fully grouted wall than a partially grouted wall.

Multiple Lift Versus Single Lift Grouting

Grout pours equal to or less than 5 feet 4 inches are called Low-Lift grouting. Grout pours that exceed 5 feet 4 inches are called High-Lift grouting. A grout pour is defined as the total height of masonry to be grouted before the installation of additional masonry. A grout pour consists of one or more grout lifts.

Grouting methods may be considered the contractor's means and methods and not part of the design. However, grouting methods may impact the cost and performance of a masonry wall and may be part of the designer's choice.

In hollow clay masonry, it is advantageous to use High-Lift grouting. When demonstrated with a grout demonstration panel, grout pours with heights exceeding 30 feet have been successfully accomplished. High-Lift grouting will typically have fewer voids. The reinforcement requires fewer or no splices and therefore costs less. Generally, the grout quality improves with the increased placement volume. For these reasons, it may be appropriate for the engineer to specify High-Lift grouting methods. Otherwise, the decision can be left to the contractor.

For highly stressed walls where reinforcement and grout placement are important, it is recommended that the structural engineer specify the grout pour heights and the lap locations in the wall. Where congestion of reinforcing is a concern, staggered laps and/or mechanical splices may be specified by the designer.

To verify the materials and procedures before construction, the structural engineer can recommend to the client that they specify a grouting demonstration panel. This demonstration panel is a sample of the wall built before construction and is often used as the visual mock-up. The architect, engineer, inspector, building official, owner's representative, and the general contractor should witness the grouting demonstration. Several days after the grouting, the panel can be cut open to expose the grouted cells and assess the quality of the materials and the process.

Clean-outs

Clean-outs are access ports at the bottom cell of the grout pour for access to removing mortar droppings. Mortar droppings can prevent the grout from bonding to the surface at the bottom of the grout pour. The mortar droppings cause a weak plane to resist shear forces. For partially grouted walls, clean-outs are installed at each reinforced cell. For fully grouted walls, clean-outs are spaced not less than 32" on center.

The Code requires clean-outs if the grout pour exceeds 5 feet 4 inches.

Where the in-plane or out-of-plane shear stress is less than 15 psi when using only the face shell area, clean-outs may not be necessary. The grouted cell area is not required to transfer the load. This is often the case for long walls.

To eliminate the clean-out requirement, it will be required to ask for a building code variance from the building department.

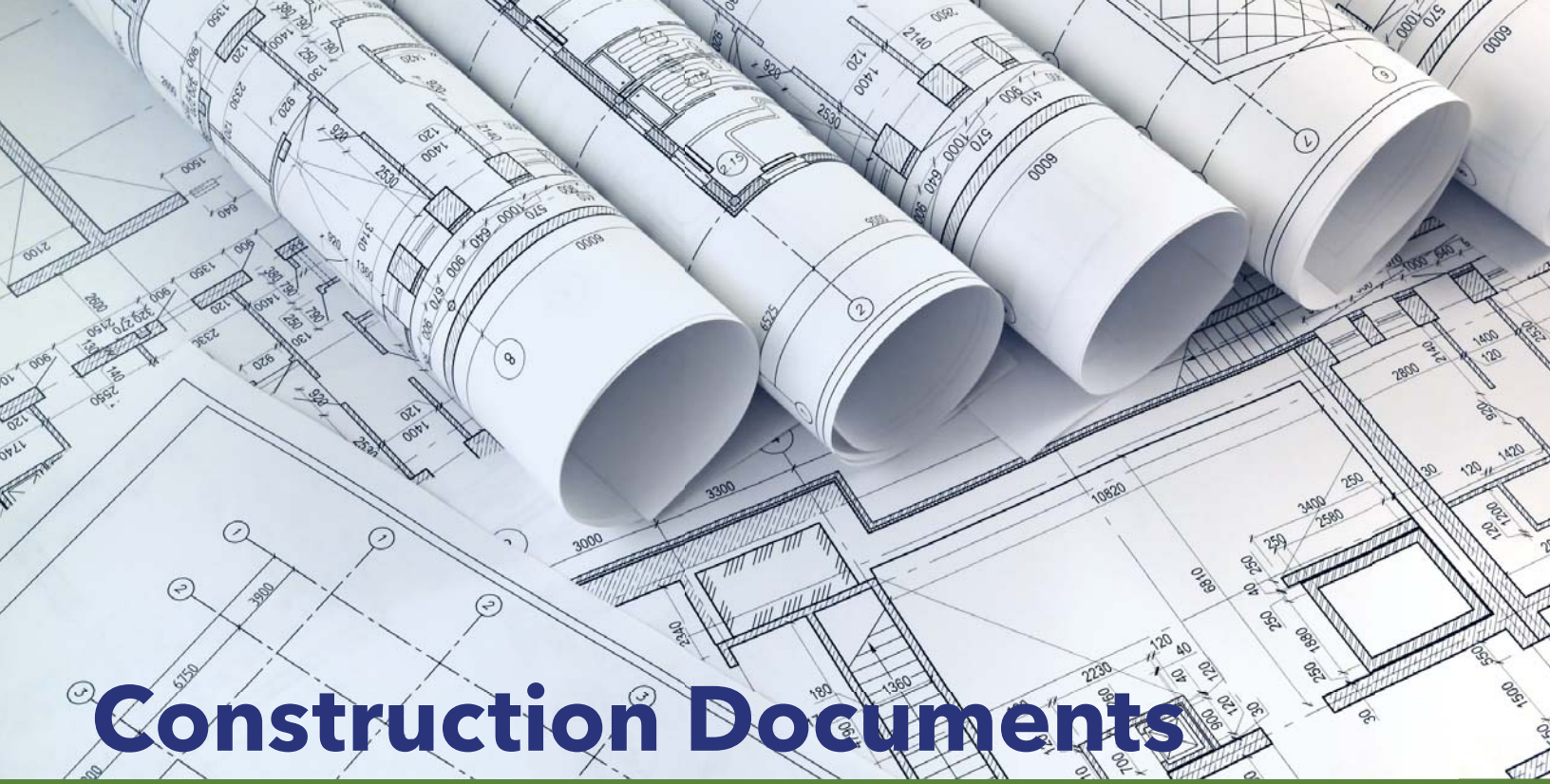
With or without clean-outs, mortar droppings should be minimized through good construction practices. Contractors can reduce droppings at the bottom of the wall by placing sponges in the cells and pulling them up as the wall is constructed. The sponges collect the mortar droppings that would otherwise need to be cleaned out at the bottom of the wall.

Horizontal Joint Reinforcement Versus Bond Beams

When horizontal reinforcement is required to resist shear forces, use bond beams. There is more confidence in the placement of bars in a bond beam than with the placement of horizontal joint reinforcement in the bed joint. This is because it is common for the joint reinforcement to be laid on the unit before the bed joint mortar is placed. The result is a void underneath the wire. It is also not unusual for the wires to be placed without lapping (butted ends). The result is incomplete embedment and development of the steel. The "butting" of horizontal joint reinforcement is a common practice in non-structural masonry and veneer. The mason may not understand the difference or importance. If joint reinforcement is used, it is recommended to require full-time inspection for its placement.

When horizontal reinforcement is not required to resist shear forces, horizontal joint reinforcement may be satisfactory and could save cost. If used, joint reinforcement must be galvanized. It is placed in the mortar joint with limited cover and must be protected from chemical attack and carbonation. Flattened or oval wire used in horizontal joint reinforcement is available in some markets. It is used to help satisfy the code wire size limitations (the wire diameter, height, cannot exceed half the bed joint height).

It is recommended that horizontal reinforcement be placed in bond beams continuously at the window sill, window and door head, and at each floor. Additionally, at least one No. 4 bar should be placed at each window and door jamb. This recommendation is for the generally good performance of the wall even when not required by code.



Construction Documents

During the construction document phase, the architect prepares the contract documents consisting of drawings and specifications set forth the detailed requirements for the construction of the project.

During the construction document phase, the structural engineer prepares the contract documents for the primary structural system consisting of drawings, including structural notes and sometimes the structural portion of the specifications.

Preparing construction documents includes structural design, analysis, drawing preparation, and writing or reviewing specifications. Each part of this process proceeds simultaneously, and they are interrelated.

Structural Analysis

Following the preliminary sizing of the primary structural elements, the global loads applied to the building are distributed to the primary structural elements. The loads are distributed to each element by methods that have become customary in structural engineering. There are several commonly used software programs available for this purpose. Stairs and ramps that are not isolated, while seldom constructed of masonry, resist wind and lateral seismic loads and must be included in the model accordingly.

For RHCM shear wall buildings, if the finite element model is used, wall expansion joints, if any, need to be modeled. If there are intersecting walls, the structural engineer needs to recognize that due to warping effects, the structural elements will have axial loads and moments resulting from torsion on the system. The design may

be simplified by not connecting intersecting walls in the model. Reinforcing details can be simplified, but the overall resilience of the building is reduced. For seismic design, if the mass of the walls is included with the floors or roof, then diaphragm loads will be overestimated.

Once the loads on the walls, columns, and elements are known, the elements can be checked to ensure conformance with acceptable standards. The building code is a minimum acceptable standard. The design may or may not require a higher standard than the building code. The structural engineer should communicate options for increasing the resilience above the code minimum to the architect and owner.

A detailed discussion of the structural analysis procedures is contained in many excellent references. The following paragraphs present information commonly encountered during the structural analysis and design of RHCM structures. Appendix B provides a checklist for the design of RHCM.

Design Assumptions

The analysis for determining the strength of RHCM elements is very similar to that used to check the strength of reinforced concrete elements. Standard assumptions for strength design are:

1. Plane sections before bending remain plane after bending.
2. Masonry components (units, mortar, and grout) combine to form a homogeneous member.
3. Masonry carries no tensile stresses.
4. Reinforcement is completely surrounded by grout and bonded to the masonry materials, so they work compositely as a homogeneous material in the range of working stresses.
5. The building code limits the yield strength of reinforcement to 60,000 psi.
6. The maximum usable strain in clay masonry is 0.0035 (It is 0.0025 for concrete masonry).
7. The compression stress block height is $0.80 f'_m$ for a distance of 0.80 times the distance from the extreme compression fiber to the neutral axis.

Additionally, for ease of design, the following recommendations are made for the area of masonry to be used in resisting the loads.

1. For partially grouted RHCM walls, it is recommended and conservative to use the face shell area of the unit as the shear design area. For partially grouted RHCM, the code allows shear areas to be increased by counting the grouted cells, but the advantage is

usually small. Also, for partially grouted walls, the nominal shear capacity is reduced by a factor of 0.70.

2. For fully grouted (sometimes referred to as solidly grouted) RHCM walls in running bond, the design area is the gross cross-sectional area of the masonry. The area of cores (most hollow clay masonry units have cores) or the void at the head joint when the masonry is laid with only face shell bedding is not subtracted from the gross area.
3. For beams with compression on the head joint and the requirement of full bedding of the head joint is not made, the design area is the face shell area for shear and flexure. This is because the head joint is filled to the face shell's depth, not the unit's full thickness. If bond beams or open-end units are used, then the design area would be the full thickness of the unit.

General

There are provisions of the code limiting the design:

1. Vertical conduits, pipes, or sleeves shall not exceed 2 percent of the net design area.
2. Pipes or conduits containing liquids, gas, or vapors at temperatures above 150 degrees Fahrenheit, pressure greater than 55 psi, or water or other liquids subject to freezing shall not be placed in the masonry.

Walls

For out-of-plane bending, the effective width "b" of the compression block is the lesser of:

1. The distance between bars.
2. In running bond, six times the nominal thickness.
3. In stack bond, the unit width.

If partially grouted, the compression area of the wall (distance to the neutral axis) needs to exclude cells not grouted. This can lead to a more complicated analysis to determine capacity.

For in-plane bending, "b" is the thickness of the wall if fully grouted. If partially grouted, the face shell area is recommended to be used. Using b as the two-face shell thicknesses is conservative.

For in-plane bending, the effective width of intersecting walls differs for compression and tension. For compression, the code allows the flange width to be 6 times the nominal thickness on each side of the intersecting wall; for tension, the maximum is 0.75 times the floor-to-floor height.

Walls Flexure Plus Compression

Flexure plus compression occurs for walls resisting lateral forces from wind or seismic loads. Software is available for checking a wall loaded with combined flexure and compression. Or, create your own. Dead load reduces the required reinforcement area when the tensile moment capacity limits an element. When intersecting walls occur, analysis can be complicated. Don't forget to include the dead load of the wall in your analysis.

For walls with a height to a nominal thickness greater than 24, second order or moment magnification effects should be considered.

Shear Walls

For shear walls resisting seismic forces, there are code-specified reinforcement requirements. For RHCM, the choices of shear wall types are Detailed Plain, Ordinary Reinforced, Intermediate Reinforced, and Special Reinforced.

The minimum reinforcement required for a Detailed Plain shear wall is shown below. Detailed Plain shear walls are only allowed in Seismic Design Categories A and B and are designed as unreinforced masonry.

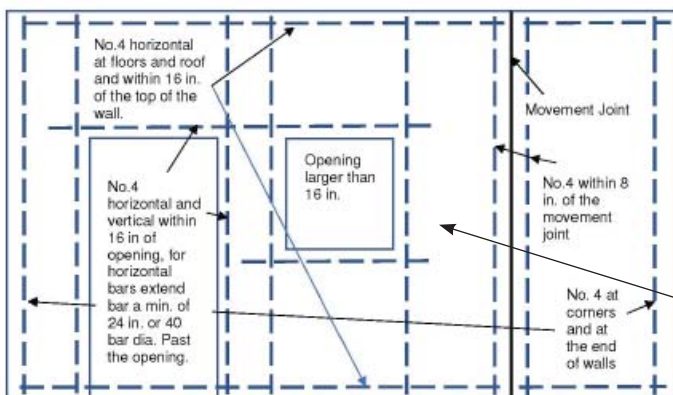


Figure 30 Minimum Reinforcement for RHCM

If your project is a Seismic Design Category C project, then Ordinary Reinforced shear walls, Reinforced Intermediate shear walls, or Special Reinforced shear walls are required. Ordinary Reinforced shear walls have the following additional requirements.

Ordinary Reinforced Shear Wall:

1. Reinforced masonry design is required.
2. Maximum spacing of vertical reinforcement is 120 inches.
3. Maximum spacing of horizontal reinforcement is 120 inches.

Intermediate Reinforced Shear Walls has the following additional requirement:

1. Maximum spacing of vertical reinforcement is 48 inches.

If your project is a Seismic Design Category D, E, or F, then Special Reinforced shear walls are required. Special Reinforced Shear Walls have the following additional requirements:

1. The maximum spacing of vertical reinforcement is smaller than $1/3$ the height of the wall or $1/3$ the length of the wall or 48 inches. For stacked bond masonry (less than $1/4$ the length of the unit overlapping), the maximum spacing is 24 inches.
2. The maximum spacing of horizontal shear reinforcement must not exceed $1/3$ the height or length of the wall. For walls of short length, this can present a problem. For example, a wall consisting of a single stretcher (12-inch length) between windows will require horizontal reinforcement at 4 inches on center, or for 4-inch height units, every unit is a bond beam. In this situation, it may be best to isolate the element from the seismic resisting system. Another possible solution is a displacement compatibility analysis to demonstrate that the element can withstand the required displacements.
3. Maximum spacing of horizontal reinforcement for running bond is 48 inches and 24 inches for stack bond.

Additionally, vertical and horizontal No. 4 bar at 120 inches on center.

4. The sum of vertical and horizontal reinforcement must exceed .002 times the gross cross-section using specified wall dimensions, with a minimum vertical or horizontal reinforcement equal to .0007 times the gross cross-section using specified dimensions. The minimum horizontal reinforcement required for stack bond masonry is .0015 times the gross cross-section using specified dimensions.
5. Stack bond masonry must be fully grouted and use open-end units. Open-end units only need to be open on one head joint for the grout to bond to the next unit.
6. If the shear demand to only the masonry shear capacity ratio (not including the reinforcement capacity) is more than 0.40, 180-degree hooks around the vertical reinforcement at the end of the wall are required. Engineering judgment is required, and a rule of thumb is that if the shear walls' height-to-length ratio is less than 1, hooks will likely not be required.
7. The design shear capacity shall exceed a shear corresponding to 1.25 times the nominal flexural capacity but not exceed 2.0 times the factored shear demand. This provision is intended to enhance flexural ductility by increasing the wall's shear capacity.

Also, in E and F, there is an additional reinforcement requirement for walls that are not shear walls or non-participating walls.

For more detailed information on the design of Special Reinforced Masonry Shear Walls, see *Seismic design of specially reinforced masonry shear walls: A guide for practicing engineers*, NIST GCR 14-917-31, prepared by the Applied Technology Council for the National Institute of Standards and Technology, Gaithersburg, MD.

Columns

There are special code provisions for columns:

1. The distance between lateral supports of a column cannot exceed 99 times the radius of gyration of the column in the direction of support.
2. The minimum side dimension cannot be less than a nominal 8 inches.
3. The minimum number of longitudinal reinforcement bars is 4.
4. Columns must be fully grouted.
5. The area of vertical reinforcement must be greater than 0.0025 times the net area of the column and not more than 0.04 times the net area of the column.
6. The effective height of a column can be taken as the distance between inflection points. But it is recommended to use the unsupported height. For cantilevered columns, use twice the unsupported height.
7. Lateral ties of at least ¼ inch diameter shall enclose the vertical reinforcement. Spacing shall not exceed 16 longitudinal bar diameters, 48 tie diameters, or the least cross-sectional dimension of the column.
8. If allowable stress design is used, the code requires a 10% eccentricity axial load in the column. For strength design, the eccentricity is accounted for in the code equations.
9. In seismic design category C and above, when anchor bolts are used at the top of a column to connect to horizontal elements, adding two No. 4 ties within 5 inches of the top of the column is required.
10. In seismic design category D and above, lateral ties must be spaced at less than 8 inches on center and be 3/8 inch in diameter or greater.
11. In seismic design category C and above, along a line of lateral force resistance, not more than 20 percent of the resistance may be provided by masonry columns. This requirement is overly restrictive and without justification for buildings such as a single-story firehouse where one of four walls has garage door openings separated by columns. There is no reason that the columns properly detailed cannot exhibit adequate ductility to accommodate seismic displacements.

The masonry code offers an exception, but only if designed for an R of 1.5. ASCE/SEI 7 does not recognize this exception. It is not addressed in the IBC. If it is not clear which code applies, check with the authority having jurisdiction.

Placing a bearing pad, allowing lateral displacement while supporting the axial load, could be an option. Another option is to use a concrete or steel column and a brick veneer or a reinforced veneer surround.

Another possible solution is a displacement compatibility analysis demonstrating that the element can withstand the required displacements.

Fortunately, in seismic design categories A, B, and C, the code provides an exception to these restrictive column requirements for lightly loaded columns. Lightly loaded columns are defined as columns with an axial load not exceeding 2000 lbs. The requirements are only an 8-inch minimum dimension, fully grouted, more than 0.2 square inches of reinforcement, and not to exceed a height of 12 feet.

Pilasters

The code definition of a pilaster is a vertical member built integrally with a wall, with a portion of its cross-section typically projecting from one or both faces of the wall. The word typically is inserted in the definition because, in some applications, a pilaster is contained within the wall, not projecting from either side.

For the intersecting wall to be considered part of the pilaster, it needs to be connected to the pilaster element by one of the following.

1. At least 50 percent of the masonry units interlock.
2. Walls need to be anchored by steel connectors.
3. Intersecting reinforced bond beams spaced less than 48 inches on center.

The effective width for compression will depend on the pilaster's geometry and the moment's direction. The effective width is generally the same as that for intersecting walls. For compression, the code allows 6 times the nominal thickness of the intersecting wall; for tension, the maximum is 0.75 times the floor-to-floor height. When the pilaster projects from one side of the wall or unevenly on both sides of the wall, the effective width, if any, will be different depending on the direction of the moment.

When the height divided by the radius of gyration (generally equal to .288 times the thickness) exceeds 99, second-order or moment magnification effects need to be considered. This will need to be considered for both moment directions.

Bar laps and Embedment

The Code provides methods for the design of bar laps.

Alternatively, use 48 bar diameters for grade 60 reinforcement. This is conservative for RHCM units of nominal 8 inches or less for number 6 bars or less and

placed at the center of the wall. For number 7 bars in 8-inch units, it is unconservative by about 10 percent.

Whenever possible, it is recommended that laps do not occur where the stress in the bar approaches the yield stress. If the stress in the bar exceeds 80% of the yield stress (48,000 psi for grade 60 bars), it is recommended to increase the lap length to 72 diameters, although not required by code.

Deflection

Deflection calculations are approximate. This results from the uncertainty about the value of the elastic modulus and the distribution and depth of cracking throughout the section. Fortunately, it is uncommon for a masonry element to be limited or sized based on the expected deflection. When deflection calculations are required, such as when height-to-thickness ratios are high, the methods presented in the building code give some indication of the deflection magnitude. The IBC provides deflection limits for exterior and interior walls with various surface finishes.

Maximum Bar Size

There are building code limitations on the sizes of reinforcement. No. 11 bar was recently allowed, but some previous codes limited the size to a No. 9. The nominal bar diameter cannot exceed one-eighth the nominal unit thickness. The maximum reinforcement in a cell is limited to 4 percent of the cell area or 8 percent with splices. The maximum diameter of the reinforcement shall not exceed one-third of the minimum dimension of the gross grout space. Additionally, there are minimum grout cover requirements for the space between the edge of rebar to the edge of the grout cell of $\frac{1}{2}$ " and $\frac{1}{4}$ ", for coarse grout and fine grout, respectively. The following table presents the typical maximum bar size for the hollow clay units. However, the cell areas vary somewhat from manufacturer to manufacturer, and dimensions should be verified on each job.

Maximum Bar Size by Cell Size

Unit Thickness (Nominal) ²	12" Long Units		16" Long Units	
	Cell Size (Width x Length)	Max. Bar Size #	Cell Size (Width x Length)	Max. Bar Size #
4"	1 3/4 X 3 1/2	4 ¹	1 3/4 x 3	4 ¹
5"	2 1/2 X 3 1/2	5 ¹	2 x 5	5 ¹
6"	3 1/2 X 3 1/2	6 ¹	3 x 5	6 ¹
8"	5 X 3 1/2	8 ¹	5 x 5	8 ¹
10"	6 3/4 X 3 1/2	10 ¹	6.5 x 5	10 ¹
12"	8 1/2 X 3 1/2	11	8 x 5	11
1. Controlled by one eighth of the nominal unit thickness. 2. 10 and 12-inch thick units may not be available – check with the manufacturer.				

Retaining Walls

Failure of retaining walls constructed of concrete, masonry, and other materials occurs frequently. Carefully analyze the soil information (have a soil report, if not, then require one), and use proper analysis methods (it can get complicated). It is recommended to place the reinforcement in the center of the cell, not offset. Many failures are traceable to the contractor placing the bars on the wrong face of the wall. The added reinforcement costs are usually small.

Bearing, Bolts, and Shear Friction

The Code provides a design method.

Drawing Preparation

Layout

The following figure presents a typical layout of the structural drawings for a building project.

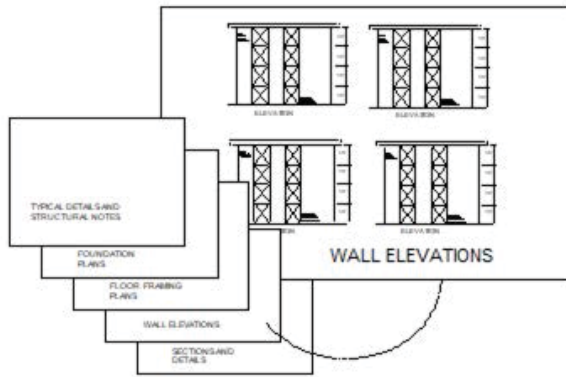


Figure 31 Recommended Drawing Layout

When RHCM is used, it is normal to prepare elevations of each wall. This may appear to be an unnecessary level of drawing detail, but experience indicates that it saves time and effort in the long run. Wall openings, expansion or movement joint locations, and reinforcing should be shown or called out on these elevation drawings. Tabulating reinforcement requirements has not proven to be a very successful method of communicating the design to the mason.

Provide a detail that shows the hollow clay unit dimensions so that the reinforcement detailers have the information to lay out the reinforcement. Additionally, expansion joints should be shown and coordinated with the architect's drawings.

Structural Notes

The structural notes are placed on the drawings to define applicable codes, loads, design assumptions, materials, and inspection requirements. Often this information is redundant to the project specifications. However, the project specifications often become lost with time. The structural notes will stay with the drawings and become useful information for some future users.

In some states, the structural notes take precedence over the specifications. In other states, the reverse is true. The architect's specification and the structural notes should state which takes precedence over the other.

The structural engineer should check the architectural specification to eliminate conflicts with the structural notes.

Details

RHCM can generally be treated like reinforced concrete, except the bond strength between the mortar and a unit is a plane of weak tension capacity. The head joints are particularly susceptible to low tension and shear capacity, and it is appropriate for the designer to assume zero tension and shear capacity for the head joint. This is important for the design of masonry connections. It is recommended that larger scale or more frequent connections be made than are typically used in reinforced concrete.

Unit dimensions restrict reinforcement spacing. Vertical bars need to be spaced to match the cell spacing, and horizontal bars need to be spaced at dimensions matching the unit height. Diagonal bars should not be used. Placing vertical bars that are 16 inches on center for a 12-inch hollow clay brick, with cells at 6 inches on center, will be generally viewed as a design error.

RHCM is normally supported on concrete or steel. The detail of the connection between the concrete or steel and the masonry is affected by the control of water and the transfer of forces. Where the exterior masonry is supported on a concrete floor or slab, it is often desirable to recess the concrete.

Water and Flashing

Water seldom completely penetrates a reinforced hollow clay wall. Instead, it enters the face of the wall and travels downward through open cells or small voids in the grout and mortar. Fully grouted walls are less susceptible to water penetration. The inside face of the wall may become damp; however, it is unusual for water to disengage from the brick or flow on the inside surface.

For this reason, the base or bottom of a wall requires flashing or another method to direct water to the exterior. The water traveling downward through the brick should be intercepted and directed outside the building. A lintel over the top of a window or door is considered a bottom of the masonry and therefore requires flashing or appropriate window head design.

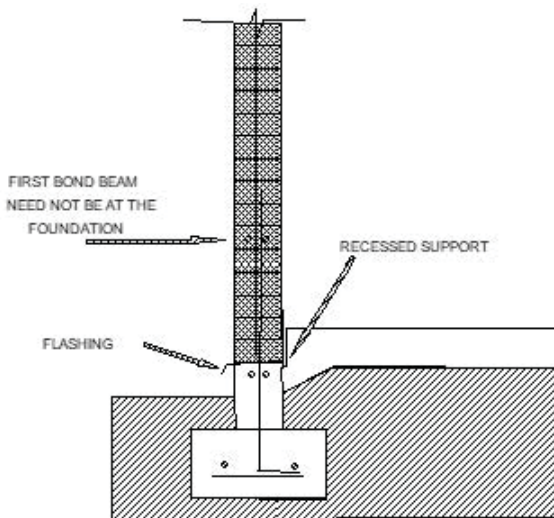


Figure 32 Wall to Foundation Detail

There are many acceptable flashing materials available. The architect should select the flashing. For the structural design, all commonly used flashing materials provide sufficient friction capacity to transfer shear loading. However, dowels will need to penetrate the flashing to transfer the shear load. Shear transfer tests indicate that the organic flashing materials transfer load better than the metal materials. This is probably because the organic materials mold to the roughness of the concrete and mortar more than the rigid material.

The code provides a method to analyze the capacity using shear friction.

Connections

Floor and roof connections to walls can take many forms. They are similar to connections to concrete walls, except the designer has more flexibility because the units are laid one at a time. Consider using embeds to connect ledgers.

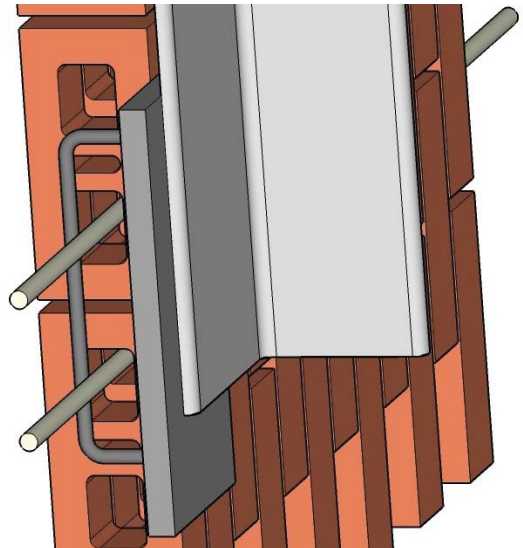


Figure 33 Embed Plate for a Ledger Connection

Movement Joints

Movement joints are a challenging design item. In addition to normal thermal expansion and contraction, brick expands permanently with moisture. This permanent volumetric change is called irreversible moisture expansion and differs from concrete or CMU irreversible shrinkage and cyclic moisture expansion and contraction. Irreversible moisture expansion of brick occurs only once over an extended period of time. It does not reverse.

After the brick is fired, there is no moisture in the brick. When placed in the environment, the brick absorbs moisture through exposure to the atmosphere containing humidity or by direct contact with water. As the brick absorbs moisture, the brick irreversibly expands. The amount of permanent, irreversible expansion depends on the type of clay. The length of time required depends on the clay and exposure. The vast majority of the permanent expansion occurs between one and five years after production. TMS 402 provides a value of 0.0003 inches per inch or 3/8 inches per 100 feet for irreversible moisture expansion. This value is typically used for design. Movement joints

related to the inherent expansion of the clay brick are called expansion joints. Expansion joints are important to the performance of unreinforced clay masonry. For reinforced hollow clay masonry with horizontal reinforcement, expansion joints are less important and often are not required. As the brick expands, the reinforcement resists the expansion. The result is that the reinforcement is placed in tension, and the brick is placed in compression. The consequence is less cracking. Nevertheless, placing expansion joints at 100 feet or less is common practice.

If the wall length exceeds 20 feet, expansion joints should also be placed on one side of a corner, near the corner. Without a corner expansion joint, the expansion of the brick in the wall results in flexural stresses in the brick on the other side of the corner and may cause a crack.

Movement joints other than expansion joints may be needed to accommodate differential movement at interfaces between dissimilar materials or systems, or changes in supports or steps in foundations, or at wall intersections, depending on the design assumptions related to load transfer between walls.

Another requirement for expansion joints occurs when a brick becomes confined by another material. The following figure shows an example of hollow clay confined in a concrete frame.

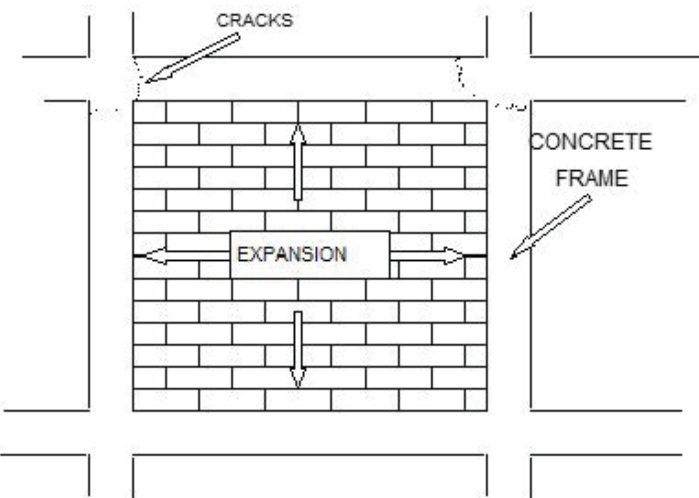


Figure 34 Brick Masonry in a Concrete Frame

Specifications

Specifications are normally prepared by the architect and reviewed by the structural engineer.

Consider including:

1. Require the mason subcontractor to prepare or oversee the preparation of reinforcing shop drawings. The shop drawing preparer should be familiar with reinforced hollow clay masonry design and construction. Concrete reinforcement detailers generally do not have an appreciation for the construction sequencing and dimensional constraints for masonry.
2. Require the water repellent to be the responsibility of the mason subcontractor. This provides a single source of responsibility in the event of problems.
3. On a large project, require a grout placement demonstration test on a sample panel prior to construction. With adequate planning, the visual mock-up panel can be used for this purpose.
4. Limit the choice of mason contractors to those experienced with the construction of RHCM. Reserve the ability to disqualify the contractor based on past performance.

The designer can influence the selection of the mason contractor through the specifications. It is recommended that the following criteria be considered in the section:

- (a) Require the contractor to employ experienced masons.
 - (b) If available, require the mason contractor to be a member of a local masonry institute. Call the institute or the material supplier to check the contractor's qualifications or provide a list of qualified bidders.
 - (c) If possible and where available, require the mason to have appropriate training and be certified by the local masonry institute or council or association.
 - (d) Require masons who have been trained by the International Masonry Training and Education Program, of the International Masonry Institute.
5. Require a preconstruction conference.
 6. Require methods other than shovel count to control proportions for mortar and grout.

7. If site mixed mortar is used, require the sand pile to be protected from rain, snow, and dust. Require the walls to be protected from rain, snow, and freezing for a minimum of 7 days after construction.

The best practice may be to require preblended, pre-bagged mortar to maintain uniformity of material properties and color.

Require the mason contractor to cover the wall at the end of the day. The purpose is to prevent rainwater from entering the masonry and developing water paths through the fresh mortar and grout. If not covered, the likely result is a wall that leaks and effloresces.

8. Require hot and cold weather protection. Code provisions may not be adequate. For example, in the Pacific Northwest, winter rain followed by a sudden freeze may damage the masonry.
9. Require inspection prior to grouting and during grouting.
10. Consider requiring self-consolidating grout.
11. Place a table of tolerances on the plans or include one in the structural notes. The code defines placement tolerances for reinforcement in terms of the design parameter "d". Unfortunately, the mason contractor has no idea what "d" is.
12. Require the mason contractor to submit a written quality control program for the masonry construction.
13. For the installation of foundation dowels, provide an option to use post-installed anchors.



Construction

The project's construction phase begins with the publication of the construction contract for bidding. During the construction phase, the architect is the owner's representative and advises and consults with the owner. Instructions to the contractor are forwarded through the architect.

During the construction phase, the structural engineer advises and consults with the architect as necessary and when requested. The structural engineer responds to bidders' questions about the design. Once the contract is awarded, the structural engineer reviews submittals by the contractor of items designed by the engineer or other items affecting the engineer's design. The

engineer makes periodic site observations to ensure that the contractor is in general conformance with the intent of the structural design.

A reinforced hollow clay masonry project requires about the same effort during the construction phase as other structural systems. However, there are a few unique situations. This section describes some of these situations.

Bidding and Award

The mason contractor is normally a subcontractor to the general contractor. During bidding, the general contractor will typically take bids or cost estimates from more than one mason subcontractor. During this period, the engineer will likely receive requests to clarify the contract documents.

The structural drawings show the design concept for the project. They are not detailed construction drawings. It is likely that during the bidding process, the contractor will find some missing information or contradictory information on the drawings. This is normal, and it is normal to receive questions about the design during the bidding period.

Often the mason contractor does not supply the reinforcement for the project. Instead, the general contractor supplies it. The engineer should become aware of who will supply the reinforcement. It will have some influence on how the project proceeds.

Moreover, most mason contractors do not prepare shop drawings for reinforcement. Thus, the responsibility

for providing an adequate set of shop drawings can become lost in the bidding process. The general contractor thinks the mason will prepare the shop drawings and the mason thinks the general contractor will prepare the shop drawings.

Experience has shown that the best performance occurs if the mason contractor purchases the reinforcement and supplies the shop drawings. However, some mason contractors may bid high or even not bid at all due to this requirement. The resulting cost pressure may require the engineer to be flexible about the requirement for shop drawings and their preparation.

Sometimes the general contractor will supply the shop drawings using his concrete reinforcing detailer. Unfortunately, the detailer may have limited experience with masonry (their expertise is concrete), and the drawings are often full of errors or items that cannot be constructed. An example is when the shop drawing shows bars spaced at 8 inches on center for a RHCM unit with a 6-inch module.

Submittal Review

Items submitted for review include the following:

1. Mortar proportions and laboratory test:

The mortar submittal is usually submitted by Type (Type M, S, or N). This is satisfactory, provided the proportions are used to define the type, not strength.

The contractor may submit by strength. In this case, laboratory tests should be performed to verify the strength of the mortar.

Preblended dry mix mortars require submittal to verify conformance with ASTM C1714.

2. Grout proportions:

The grout submittal is usually submitted by Type (fine or course). This is satisfactory, provided the proportions are used to define the type, not strength.

If the design f'_m exceeds 2,000 psi, laboratory testing is required by the most recent TMS 602 to verify that the grout strength equals or exceeds f'_m . Testing is in accordance with ASTM C 1019.

For batch-provided grout, the proportions are normally described by weight. The weight proportions should be converted to volume proportions for comparison with ASTM C 476. An example calculation of an actual grout submittal on a project follows:

Example Calculation of Volume Proportions from Weights

Item	Batch Weight (Lbs)	Conversion to Volume (Lbs/cu. ft)	Volume (cu. ft)	Proportions (by volume)
Cement	658 or 7 sacks	94 ¹	7.0	1
Lime	50	40 ²	1.25	.18
Sand	2143	80 ³	26.8	3.8
Aggregate	1009	105 ⁴	9.6	1.37
Water	260	62.4	4.17	.6
1. 94 lbs per bag 2. One 50 lbs bag is 1 1/4 cu. ft 3. ASTM C-476 specifies 80 lbs per cu. ft for loose, damp sand (5% moisture). Dry sand weighs approximately 100 lbs per cu. ft. 4. Dry-rodded unit weight obtained from the supplier				

A comparison ASTM C 476 is given in the following table.

Comparison to ASTM C 476

Item	ASTM 476	Mix
Cement	1	1
Lime	.1	.18
Sand	2.5-3.3	3.22 ¹
Aggregate	1.1-2.2	1.16 ¹
1. Proportions of cementitious material include both cement and lime		

The sample mix design does not comply. However, the mix design was accepted on this project because of past experience with the proportions where the additional lime in the mix improved flow while maintaining strength. ASTM C 476 allows proportions of the grout ingredients to be determined by laboratory testing or field experience if a satisfactory history of the grout mix performance is available.

3. Unit certifications:

If the Unit Strength Method was used to establish f'_m , the structural engineer should specify that the brick supplier submits a letter stating that the bricks meet the required compressive strength.

4. Reinforcement shop drawings:

These should be scheduled to provide sufficient time for review and resubmittal prior to construction. However, reviewing the submittal is necessary to verify that the contractor understands the design correctly. If enough time is available, re-submittal is possible without costly delays.

5. Miscellaneous metal shop drawings:

Where miscellaneous steel connections and embedded items are included in the design, shop drawings need to be submitted with sufficient time for review.

6. Quality control program:

TMS 402-22 requires the construction documents to include a written quality control program. A conforming quality control program should be written by the mason contractor and submitted in time for review and discussion with all involved including the inspector and general contractor.

Pre-Construction

Once the general contractor and the mason subcontractor have been selected, the engineer may verify their qualifications with the local masonry institute and material suppliers. This information will help determine the time and effort needed during construction.

At an appropriate time, usually, at least two weeks before the start of masonry construction, arrange for a preconstruction conference to discuss the masonry construction. Attendees should include:

1. The mason contractor and foreman.
2. The general contractor and superintendent.
3. The building official.
4. The architect.
5. The special inspector, when required.
6. The structural engineer.
7. The owner's representative.
8. The brick supplier or designated representative.

Subjects for discussion include:

1. Brick:

Determine the availability and delivery schedule of the selected brick. If the unit strength method is used, verify that the brick will meet the required strength.

2. Initial testing:

If the unit strength method was used to establish the design strength f'_m , mortar, grout, and prism testing prior to construction are not required. However, prism tests prior to construction are recommended.

As a minimum, unit testing or manufacturer's certification is required. If a grouting test panel is to be used, define the schedule. The engineer, building official, and the special inspector should be present for the grouting demonstration. Often the grouting demonstration panel can also be used by the architect as a color and quality control panel.

3. Testing during construction:

For risk category I, II, and III buildings, prism testing is recommended, but not required by code, for every 5,000 square feet of wall. For risk category IV buildings, prism testing is required for every 5,000 square feet of wall. Prior to construction, five prisms constitute a test. During construction, three prisms constitute a test. However, five are recommended

during construction. Test the first one at seven days, the next three at 28 days, and hold the final sample for testing in case of a problem.

4. Inspection:

The inspector should regularly check the batching of mortar and grout to ensure proper proportions or that the supplied preblended dry mortar mix meets the specifications and that the proper amount of water is added. The inspector should regularly check the laying of units to ensure proper workmanship. Continuous inspection during the placement of grout is required. The inspector should verify and ensure full compliance with the contract documents for the placement of reinforcement, grouting, consolidation, reconsolidation, top of the wall shear key, and the protection of the masonry from rain, dirt, and cold and hot weather.

5. Observation:

Inform all participants that, from time to time, the engineer will visit the site to ensure general compliance with the contract documents.

6. Inspection Reports:

Normally, special inspection and test reports and inspection reports go to the general contractor and the architect and then to the engineer. Deviations from this normal procedure should be discussed, defined, and documented.

7. Submittals:

Verify that the contractor is using the approved project submittals.

8. Cleaning and Sealing:

The procedures to clean and seal the masonry with water repellant should be discussed. It is important that the mason contractor verify the cleaning method with the unit manufacturer. If the wall is to be sealed with a water repellant, the method and materials to be used should also be verified with the brick manufacturer.

Proper sealers do not seal the wall. The masonry must breathe. The sealers are actually only water repellents.

9. Construction Sequence and Schedule:

Discuss the schedule for inspection and testing. Discuss coordination issues. One usually missed item is the coordination with the window and door supplier. The design of the connections should be discussed.

Site Visits

The structural engineer should make site visits to check on the progress and quality of the work. This part of the engineer's scope of services is called structural observation. Structural observation is required for most projects by the IBC and is defined as follows:

"The visual observation of the structural system by a registered design professional for general conformance to the approved construction documents."

Structural observation does not include or waive the responsibility for the inspection required by other code sections.

Masonry walls failing during construction, particularly those employing high lift grouting, happen more frequently than they should. While it is not the structural engineer's responsibility to evaluate the contractor's means and methods, if the shoring of the wall does not look adequate, the structural engineer should inform the contractor and document that the information was given.

An example site observation form and checklist are provided in Appendix C.

Non-Conforming Quality Control Tests

Prism Compression Strength - Prior to Construction

If the prism test prior to construction does not comply with the required strength, there are many possible reasons. Often, and particularly for larger units (8 inches and above) with a specified compression strength exceeding 2,600 psi, the problem is often the testing method. It is not uncommon for the platen used to be too thin or the prism surface to be not flat. The brick may not be strong enough, or the mortar and grout may not be the appropriate mix.

Prism Compression Strength - During Construction

When prism tests do not conform, verify that the materials used (units, mortar, and grout) conform to the specifications. If they do conform, either the prism was improperly constructed, or the testing procedures did not comply with ASTM C 1314.

Prism construction errors include not constructing the prism true and plumb. It is very important that the top and bottom planes of the prism are parallel. Another common problem for large cell units (8" units and larger) is that the grout is not properly reconsolidated. Without proper reconsolidation, a dome-shaped void will often form at the mortar joint and render the area of grout ineffective for resisting compression.

Common testing errors include not properly capping the prism so that the top and bottom planes are level and parallel, not providing a thick enough loading platen to distribute the test machine load evenly to the prism, and the prism is not centered in the testing machine when loaded.

When confronted with a non-conforming wall (a low prism test for the wall), the first step is to recalculate the structural design to verify that the strength specified is required. If this does not work, the next step is to cut a prism from the wall and test it. Usually, prisms taken from the wall will test with higher compression than the sample prism. If this fails, the wall will likely need to be removed, materials changed to meet the required strength, and the wall reconstructed.

Mortar Compression

It is recommended that field testing of mortar not be required. However, the requirement for mortar testing often is outside the structural engineer's control, and on some projects, mortar testing becomes a requirement (See Appendix D for more information).

Grout Compression

It is recommended that field testing of grout not be required. However, when f'_m exceeds 2,000 psi, TMS 602-22 Section 2.2 B (Specifications for Masonry Structures) requires verification of grout strength by testing in accordance with ASTM 1019 (See Appendix D for more information). This requirement is interpreted as prior to construction rather than verification during construction.

APPENDIX A: WSCPA Members

Western States Clay Products Association
website: www.wscpa.us

Manufacturing Members

Interstate Brick Company
9780 South 5200 West
West Jordan, UT 84081-5625
Phone: (801) 280-5200
<https://interstatebrick.com>

McNear Brick & Block
1 McNear Brickyard Rd.
San Rafael, CA 94901-8310
Phone: (415) 453-7702
<https://www.mcnear.com>

H. C. Muddox/Gladding McBean
4875 Bradshaw Road
Sacramento, CA 95827
Phone: (801) 280-5200
<https://www.gladdingmcbean.com>

Mutual Materials Company
605 - 119th NE
Bellevue, WA 98005
Phone: (425) 452-2300
<https://www.mutualmaterials.com>

Summit Brick Company
601 E. 13th Street
Pueblo, CO 81001-2942
Phone: (719) 542-8278
<https://www.summitbrick.com>

Allied Associates Contacts

Arizona Masonry Council
3133 West Frye Road Suite 101
Chandler, AZ 85226
Phone: (602) 265-5999
<https://www.azmasonry.org>

Masonry Institute of America
1315 Storm Parkway
Torrance, CA 90501-5041
Phone: (310) 257-9000
<https://www.masonryinstitute.org>

Masonry Institute of Washington
11900 NE 1st Street, Ste 300
Bellevue, WA 98005-3049
Phone: (425) 214-7476
www.masonryinstitute.com

Rocky Mountain Masonry Institute
6145 Broadway Suite 44
Denver, CO 80216
Phone: (303) 893-3838
<https://rmmi.org>

Utah Masonry Council
4001 South 700 East #500
Salt Lake City, UT 84107-2177
Phone: (801) 264-6651
<https://utahmasonrycouncil.org>

APPENDIX B: Drawing Checklist

No.	Item	Checked
1.0	STRUCTURAL NOTES	
	Is the applicable code specified (city and date)?	
	Are the applied loads shown, including wind, seismic, and live loads?	
	Is the masonry strength f'_m specified?	
	Is the method to verify the f'_m specified? (Unit strength method.)	
	Are the units specified in accordance with ASTM C 652 with sufficient compressive strength to meet the f'_m specified?	
	Is cement specified in accordance with ASTM C 150 types I, II, or III? Is low alkaline cement available?	
	Is lime specified in accordance with ASTM C 207?	
	Is sand specified in accordance with ASTM C 144?	
	Is the mortar specified by proportions in accordance with ASTM C 270?	
	Is the grout specified in accordance with ASTM C 476?	
	Is self-consolidating grout specified in accordance with ASTM C 1611/C1611M?	
	Is high or low lift grouting specified?	
	For high lift grouting, are clean-outs to be eliminated with a code variation?	
	Is reinforcement specified in accordance with ASTM C 615 grade 60?	
	Is ASTM A 706 reinforcement specified when welded?	
	Is the quality assurance program included? (Required by code)	
	Is a grouting demonstration panel required?	
	Is there a requirement for a preconstruction meeting?	
	Are reinforcement shop drawings required?	
2.0	DESIGN	
	Is h/t less than 30? If not, verify the design includes analysis for moment magnification.	
	Are the walls at floors and roof laterally supported with straps or other methods capable of resisting at least 400 lb/ft?	
	Do the bars fit in the cells?	
	Are locations and length of laps shown? Are there locations where stresses are more than 80% of the allowable?	
	Are dowel laps sufficient?	
	Is there continuous horizontal reinforcement at the window and door head and jambs?	
	Is there continuous horizontal reinforcement at the floor?	
	Are window and door connections designed and shown on the drawings?	
	Are there expansion joints at the corners, and are they at or near the corners?	
	Are provisions made in connections to accommodate thermal and moisture expansion movement?	
	Is the brick masonry confined between other materials without expansion joints?	

No.	Item	Checked
3.0	SPECIFICATIONS (Additional items not typically in the structural notes)	
	Are the structural notes and specifications consistent?	
	Are control joint size, size, and materials specified?	
	Are control joint sealant compatibility tests required?	
	Are the cleaning methods included?	
	Does the specification allow wetting of the brick?	
	Are the joint finishes specified? If raked joints are used, is this in the analysis?	
	Are weep holes and fill materials specified?	
	Are the sealing, water repellant, procedures, and materials specified?	
	Are cold weather and hot weather construction provisions included?	
	Are requirements for protecting the work included?	
	Is it required to verify dimensions prior to laying the masonry?	
	Is a written quality control procedure required?	
	Is a color, pattern, and workmanship panel required?	

APPENDIX C: Construction Observation Checklist

No.	Item	Checked
1.0	MATERIALS	
	Are the bricks stored above ground and covered?	
	Are the bricks sound? Bang them together to see if they ring. If a thud, the bricks may need to be rejected or prism tests conducted to prove that f'_m is met.	
	Is the cement properly stored?	
	Is the lime properly stored?	
	Is the sand pile covered?	
	Does the sand appear well graded and sound?	
	Is the sand dirty?	
	Is there a method for controlling the sand proportions? Shovel count methods are not sufficiently accurate.	
	Does the person mixing the mortar know the proportions?	
	Are there any additives being added to the mortar?	
	Does the person mixing the mortar know the time limits for mixing? (10 minutes maximum.)	
	Are the grout proportions being controlled?	

No.	Item	Checked
2.0	CONSTRUCTION	
	Is the mortar dropping into the cells?	
	Are mortar fins being controlled? Are the cells clean?	
	Is the mortar being strung out too far on the bed joints?	
	Does the contractor understand cold (hot) weather construction requirements?	
	Are the walls being covered at the end of the day?	
	Is inspection being properly done?	
	Does the shoring of the masonry walls look about right?	
	Are the joints uniform in thickness and full?	
	Is the reinforcement being placed within tolerance? Is reinforcement secured against displacement? Are lap lengths correct?	
	Is the grout of sufficient slump to be placed?	
	Is the grout being vibrated during placement?	
	Is the grout being reconsolidated?	
3.0	DESIGN	
	Is the proper issue of design drawings on site?	
	Are the proper shop drawings on site?	
	Does the contractor understand the structural intent?	
	Is the inspector performing his duties properly?	
4.0	TESTING AND INSPECTION	
	Are prisms being constructed plumb and on a flat surface?	
	Are the materials being used for the prism the same as those used in the wall?	
	Are prisms being properly cured, handled, stored, and transported?	
	Is the prism grout being reconsolidated?	
	Is the frequency and quantity of testing in accordance with the contract documents?	

APPENDIX D: Field Test Data for Mortar and Grout

Field Mortar and Grout Test Data

The field sampling and testing for mortar compression strength are highly variable. Figure 1 is a frequency distribution of Type S mortar compression tests taken in the field during actual projects in California, Oregon, and Washington. There are a total of 205 mortar tests. The coefficient of variation is 36%.

With this amount of variability, it should not be surprising to get periodic non-conforming compression mortar tests. If the non-conformance occurs regularly, then the following steps are recommended:

1. Request that the mason identify the proportions being used. Go to the site and observe the mixing of the mortar.
2. Assess the method being used to control proportions. Check that the correct materials are being used.

3. Verify that the testing lab is using the procedures of ASTM C 270.

4. Visit the site and observe the mortar in the joint. Scratch the mortar with a key. If a white scratch results and the sand does not separate from the mortar, the strength of the mortar is probably acceptable. However, if the masonry is highly stressed (above 1200 psi), it may be necessary to remove a prism from the wall for testing.

The relationship between 7-day mortar strength and 28-day mortar strength is not as variable as the compression strength. It is useful to know the 7-day test results since it provides the engineer with an early indication of the 28-day results. The following figure presents the relationship for the same projects.

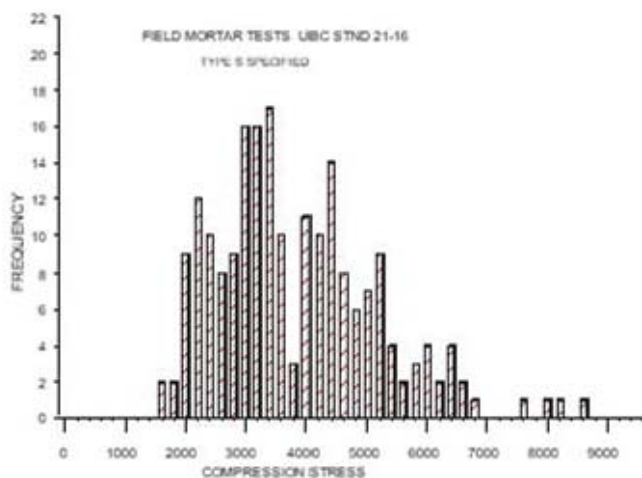


Figure 1 Field Mortar Tests



Figure 2 Comparison of 7 and 28 Day Mortar Tests

Grout Compression

It is recommended that field testing of grout not be required. However, when f'_m exceeds 2,000 psi, TMS 602-22 Section 2.2 B (Specifications for Masonry Structures) requires verification of grout strength by testing in accordance with ASTM 1019. This requirement is interpreted as prior to construction and not verification during construction.

The field sampling and testing for grout compression strength are highly variable. The following figure is a frequency distribution of field grout compression tests taken from actual projects in California, Oregon, and Washington. There are a total of 323 grout tests. The coefficient of variation is 32%.

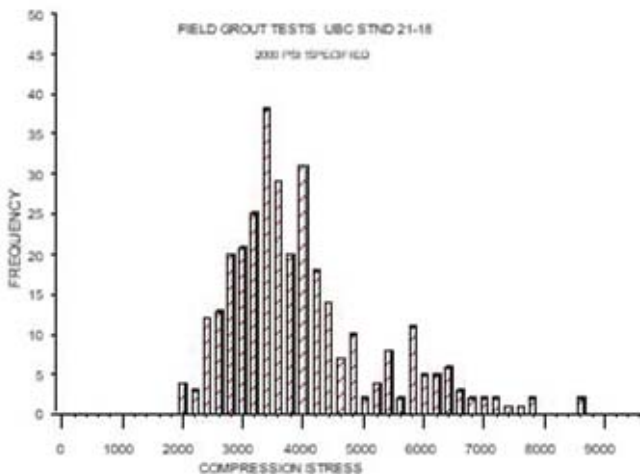


Figure 3 Field Grout Tests

With this amount of variability, it should not be surprising to get periodic non-conforming compression grout tests. If the non-conformance occurs regularly, then the following steps are recommended:

1. Request that the mason identify the proportions being used.
2. Assess the method being used to control proportions.
3. Verify that the testing lab is using the procedures of ASTM 1019.
4. If the cause of the low break is not identified, then taking core samples and testing them may be required.
5. The structural engineer should also consider the reason for requiring a specific grout strength. Often, the purpose of the grout is only to connect the reinforcement to the units. Even low-strength grouts (1500 psi) are probably capable of making the connection. Because of the high strength of the brick, the compression contribution of the grout can often be ignored in the analysis. If grout strength is less than f'_m , then lap lengths should be based on the grout strength and not f'_m .

The relationship between 7-day grout strength and 28-day grout strength is not as variable as the compressive strength. It is useful to know the 7-day test results since it provides the engineer with an early indication of the 28-day results. The following figure presents the relationship for the same projects.

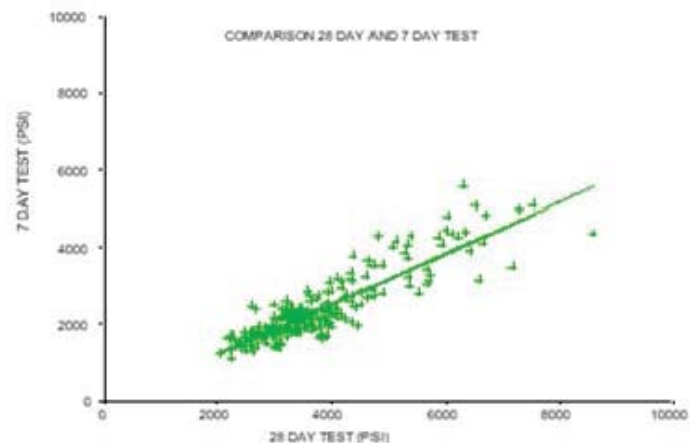


Figure 4 Comparison of 7 and 28 Day Grout Tests

APPENDIX E: Troubleshooting Table

Troubleshooting Table for the Design Structural Engineer

Problem	Cause	Solution
Prisms fail to reach the design strength.	1. The testing lab has incorrectly tested the prism, usually by not placing the prism correctly in the machine or using a loading platen that is too thin. Or the specimens may have been damaged during transportation. 2. The bricks are below the specified strength. 3. The mortar is under specified strength. 4. Lab reported gross area stress instead of net area stress.	1. Instruct the lab to retest, being careful to follow the ASTM C 1314. 2. Request that the contractor have a lab test the brick. 3. Check mortar proportions. Retest the prisms. 4. Have the lab correct the report.
Mortar doesn't reach strength.	1. Incorrect proportions. 2. Incorrect testing.	1. Check mortar quality control procedures. 2. Mortar tests are unreliable. Forget about testing mortar. The code doesn't require it.
The grout mix does not meet ASTM C 476	Wrong mix	Change the mix to conform to ASTM C 476
For f_m exceeding 2,000 psi, the tested grout strength is less than f'_m .	1. Tested during construction 2. Incorrect testing. 3. Wrong design mix	1. Grout tests are unreliable. Forget about testing grout during construction. The code doesn't require it. 2. ASTM C 1019 is not reliable. Repeat the test using different liners. 3. Change the design mix.
The colors do not meet expectations.	1. Bricks were not blended. 2. Sample panel has a different water repellent. 3. The brick production run is different from the sample run.	1. This is a problem for the architect and brick supplier. 2. Use the sample panel sealer. 3. Approve the production run before beginning construction.
The architect calls and says more expansion joints are required.	The architect was checking on your advice. Architects should do this, so don't get mad. The architect's experience may be with concrete masonry units that require closer spaced control joints for shrinkage and is not experienced with structural reinforced hollow clay masonry.	Explain to the architect how the reinforcement reduces the need for most of the expansion joints.
The mason tells the general contractor, who tells everyone that the cells are too small to be grouted with all the congested steel.	1. The mason contractor does not have experience with the grouting of reinforced hollow brick. He doesn't understand that he can make the grout with an 8 to 11-inch slump. 2. The cell is too small.	1. Prepare and grout a test panel. Be sure to invite everyone concerned. 2. Verify that the units conform to ASTM C 652 or re-design the wall with smaller diameter reinforcement, or use mechanical connectors
Welded bars are breaking off.	ASTM A 706 bars were not used. Inspect the bars. A "W" symbol indicates type A 706. Verify that bars were properly cooled and not quenched with water after welding.	Use the correct bars.
The contractor is not protecting his materials or work.	Sometimes the responsibility for protecting the work is left to the general contractor. He is saving money. Sometimes the responsibility is not well defined.	Request your client to set up a meeting with the general contractor and the masonry contractor to address the issue.

Problem	Cause	Solution
Cracks in the mortar joints.	1. Shrinkage of the mortar joint. 2. Movement of the supporting structure. 3. Overloading. 4. Too rapid drying.	1. Suggest the contractor decrease the cement content of the mortar and increase the lime, provided it still is in conformance with ASTM C 270. 2. Check supports. 3. Check loading the timing of the loading and shoring removal. 4. Pre-wet the units. Wet the wall during curing. Add lime to the mortar. Tooling of joints may not be done at the proper time, or joints are not properly tooled.
Shop drawings are not prepared.	The requirement was missed or "value engineered away."	Write a letter to your client explaining the requirement. If the project is underway, require an engineer familiar with the design to be on site full time.
The grout strength is specified at a minimum of 2000 psi; how can I get a prism of 4000 psi?	This is normal.	Explain that the prism does not fail in accordance with the weak link theory. See the Design Development section "Selection of Masonry Strength, f'_m ."
The contractor wants to grout pours to exceed the 5 feet 4-inch lift height and not provide clean-outs.	The code restricts the grout lift to 5 feet 4 inches even though the grout pour might be higher. If the lift is over the 5 feet 4-inch then it is high lift grouting.	The problem is blow-outs of mortar joints and the ability to reconsolidate. In hollow clay, these problems are unlikely. Have the contractor demonstrate the procedure to you and the inspector using a grouting demonstration panel.
The contractor doesn't want clean-outs. You want high-lift grouting.	Code requires clean-outs for high-lift in order to remove the mortar droppings.	If the shear stresses are low, it may be possible to waive the clean-out requirement. The purpose of the clean-out is for shear transfer at the bottom of the cell. If shear stresses are low, this may not be necessary, and a code exception will be necessary.
The dowels out of the concrete foundation interfere with the unit cross webs. They miss the cells.	Improper placement of the dowels. However, it is often very difficult to get them in the right place. This situation is more common than not.	Cut the unit cross webs to allow the dowel to pass or drill in new dowels. Verify that all the dowels are required to meet strength requirements. If not, allow a certain % of them to be removed. It is common to allow the dowels to be bent slightly. A 1:6 slope ratio is a commonly used maximum.
The brick masonry is cracked, with cracks extending through the units.	1. A great deal of force is required for this condition to exist. Frozen grout, foundation movement, or thermal movement from an adjacent structure are a few examples. 2. The bricks may have been manufactured with cracks. 3. Foundation cracks extend into the brick wall.	1. Find the reason for the cracking. It is likely something needs to be corrected. Likely candidates include freezing, foundation settlement, overloading and thermal movement. 2. Verify the integrity of the units before use. A quick check is to bang the bricks together; if a ringing sound results instead of a thud, then the bricks are sound. 3. Foundation control joints are not coordinated with the masonry expansion joints. It may require rework or adding a control joint.

Problem	Cause	Solution
The contractor doesn't cover the walls at the end of the day.	1. The contractor is attempting to save money. 2. The responsibility for the masonry protection may have been left with the general contractor or, worse, left out of all the contracts. 3. The contractor may not be aware of the requirements of TMS 602 to cover walls at the end of the shift.	1. Insist on covering the walls. 2. Write a letter stating that the contractor is not in conformance with the likely result being efflorescence and other wall damage.
Corrosion of the joint reinforcement.	1. Too strong of an acid cleaning without pre-wetting the wall. 2. Ungalvanized joint reinforcement.	1. Pre-wet the wall and use industry cleaners as recommended by the manufacturer of the units. 2. Use galvanized joint reinforcement.
Leaking Walls	1. Improper flashing installation. 2. Improper flashing design. 3. Poor workmanship. 4. Improper grouting. 5. Raked joints or other unprotected horizontal surfaces. 6. Water repellant not applied.	1. Correct flashing. 2. Correct flashing. 3. Repair mortar joints. 4. Pressure epoxy grout. 5. Fill joints. Cover horizontal surfaces. 6. Apply water repellant wall.
Dome shaped voids in the grout.	1. Loss of water to the unit. 2. Improper or no vibration of the grout 3. No reconsolidation	1. Use proper reconsolidation techniques. 2. Use Grout-Aid or equal. 3. Add lime to the maximum allowed. Use self-consolidating grout.
River voids in fine grout.	Loss of water to the unit.	1. This is normal, provided they are less than approximately 1/4 inch in width. 2. Use Grout-Aid or equal. 3. Add lime to the maximum allowed. 4. Reduce water in the mix.
Efflorescence on the wall.	1. Precipitation of salts at the wall surface. 2. Wall was uncovered during construction, and excessive water entered. 3. Flashing not properly detailed, not installed, or not properly installed	1. Clean the wall and then keep water from entering the wall by fixing leaks and sealing it with a water repellant. 2. Cover the wall during construction. 3. Rework the flashing

Notes on the Selection, Design and Construction of

Reinforced Hollow Clay Masonry

Is also available from Western States Clay Products Association at

<http://www.brick-wscpa.org>

Other technical documents are also available

Design Guide for Anchored Brick Veneer Over Steel Studs

and

Design Guide for Structural Brick Veneer

Strengths of RHCM are usually greater than the 2600 psi used in the document

- Increasing the f'_m increases the capacity of the masonry. RHCM design strengths far exceed those available for concrete masonry. The assumed starting value of 2,600 psi is conservative. Some engineers start with a value of 3,500 psi. However, it is recommended to specify the lowest value necessary to meet the project demand strength requirements especially for schematic design.
- Using the higher design strength can reduce or eliminate horizontal shear reinforcing in shear walls, and shear reinforcing (stirrups) in beams, in some cases.
- For specially designed shear walls, the structural engineer may be able to eliminate the transverse reinforcement hooks at the end of the wall if the following condition is satisfied.

$$V_u / (\phi V_{nm}) \leq 0.4$$

- For intermediate and special shear walls, the ductility requirements can be mitigated by increasing f'_m to allow additional tension reinforcement. This will only occur in multi-story load-bearing applications.
- For beams with high moment demand, the limits on tension reinforcement can be increased by increasing f'_m .
- In some situations, increasing f'_m can eliminate the requirement to do a moment magnification analysis.



REINFORCED STRUCTURAL CLAY BRICK

strong, versatile solution

BY JEFF ELDER, PE

Those inexperienced in the use of reinforced structural clay brick might think that a brick is a brick and limit its application to conventional methods associated with veneer. Brick veneers are defined by their ability to support their own weight while limited by mortar to resist tension. When the mortars' resistance to tension is exceeded, brick must be joined or supported by other materials such as brick ties, shelf and ledger angles, wood, steel, concrete and other backup systems.

Researchers have spent years investigating ways to provide tensile

resistance to brick. Patterned after reinforced concrete, reinforceable structural clay brick create the form and become the aggregate to support compression forces. Reinforcing steel resists the tension. Oversized holes are placed in the brick and align vertically to create a place for reinforcing steel. Grout is placed in brick cells to bond reinforcing steel to the brick.

Structural clay brick are specified under ASTM C652 as hollow, fired clay brick with a percentage void volume greater than 25%, generally ranging between 35% and 60%.

net area compressive strength of clay masonry units (psi)

TYPE M OR S MORTAR	TYPE N MORTAR
1700	2100
3350	4150
4950	6200
6600	8250
8250	10,300
9900	
13,200	

net area compressive strength of masonry (psi)

1000
1500
2000
2500
3000
3500
4000

Table 1. Compressive Strength of Clay Masonry. For SI: 1 lb per square inch = 0.00689 Mpa. (IBC 2105.2.2.1.1) 2003 International Building Code. Copyright 2003. Falls Church, Virginia: International Code Council, Inc. Reproduced with permission. All rights reserved.



Learning Objectives

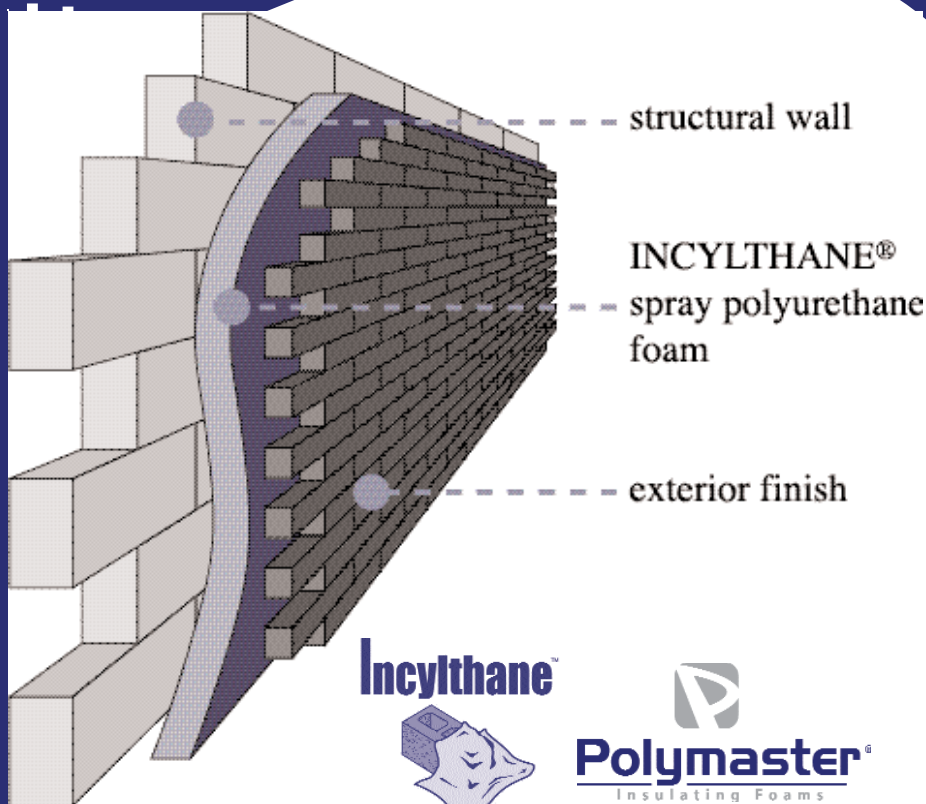
1. Reinforced Structural Clay Brick (RSCB) are designed using most of the same equations and details as Concrete Masonry Units (CMU).
2. RSCB are two to three times stronger than CMU.
3. RSCB have fire ratings of one to four hours.
4. RSCB use expansion joints whereas CMU use control joints
5. Any design detail created using brick veneer can be created using structural brick. Not every structural brick detail can be created using veneer brick.
6. RSCB can act as beams, columns, loadbearing walls, structural panels and curtain walls.
7. RSCB are available in a wide variety of colors, sizes, shapes and textures.

See page 47 for test.

mortar	type	proportions by volume (cementitious materials)							hydrated lime° or lime putty	aggregate measured in a damp, loose condition
		PORTLAND CEMENT ^a OR BLENDED CEMENT ^b	MASONRY CEMENT ^c			MORTAR CEMENT ^d				
			M	S	N	M	S	N		
CEMENT-LIME	M	1	—	—	—	—	—	—	¼	Not less than 2¼ and not more than 3 times the sum of the separate volumes of cementitious materials
	S	1	—	—	—	—	—	—	over ¼ to ½	
	N	1	—	—	—	—	—	—	over ½ to 1 ¼	
	O	1	—	—	—	—	—	—	over 1 ¼ to 2½	
MORTAR CEMENT	M	1	—	—	—	—	—	1	—	
	M	—	—	—	—	1	—	—	—	
	S	½	—	—	—	—	—	1	—	
	S	—	—	—	—	—	1	—	—	
	N	—	—	—	—	—	—	1	—	
	O	—	—	—	—	—	—	1	—	
MASONRY CEMENT	M	1	—	—	1	—	—	—	—	
	M	—	1	—	—	—	—	—	—	
	S	½	—	—	1	—	—	1	—	
	S	—	—	1	—	—	1	—	—	
	N	—	—	—	1	—	—	1	—	
	O	—	—	—	1	—	—	1	—	

Table 2. Mortar Proportions [IBC 2103.7(1)]. a. Portland cement conforming to the requirements of ASTM C150, b. Blended cement conforming to the requirements of ASTM C595, c. Masonry cement conforming to the requirements of ASTM C91, d. Mortar cement conforming to the requirements of ASTM C1329, e. Hydrated lime conforming to the requirements of ASTM C207. 2003 International Building Code. Copyright 2003. Falls Church, Virginia: International Code Council, Inc. Reproduced with permission. All rights reserved.

FREE.



Monolithic Coat seals and insulates the wall system **TIGHT** and **DAMPPROOFS**

APPLIED WITH EXPERIENCE BY

Great Lakes Insulating Systems
877-727-3626 • Grand Rapids

Alpers Insulation
231-946-7450 • Traverse City

Alpha Foam Insulation
989-799-9669 • Saginaw

R14 Wall...and Dampproofing

STRUCTURAL CLAY BRICK



Fox Studio, Livonia, designed by Giffels, Hoyem, Basso, now IDS, Troy, used both 4 x 8 x 16 custom color Brownstone flashed (with center score) as a veneer and 8 x 8 x 16 (with center score) as a structural unit.

The designer must specify the type of brick. Hollow brick are designated as either HBS (hollow brick standard) or tighter spec HBX (hollow brick extra). Use HBX when a higher standard for dimensional and aesthetic tolerances are required and a higher cost can be justified.

When brick are subject to frequent conditions of freezing when saturated, the designer is directed to specify Grade SW (severe weathering) brick.

Structural clay brick are commonly available in nominal widths of 4", 5", 6" and 8". They come in nominal lengths of 8", 12" and 16" with heights that range from 2 1/4" to 8".

Compressive Strength

As strength of brick is often related to color, it is important that an architect specify the color of brick early in the design process. This allows the structural engineer to maximize the strength performance of design elements. For most manufacturers, the lighter the brick color, the lower the compressive strength of the system; conversely, the darker the color, the higher the compressive strength of the system.

The designer is referred to the specified manufacturer to obtain the *compressive strength of unit* for all colors specified. The designer is then directed to the appropriate building code to cross reference *compressive strength of unit* with design value *compressive strength of masonry*.

When using the International Building Code (IBC 2003), refer to Table 2105.2.2.1.1. (See Table 1.) Similar tables are found in the masonry section of other codes. The specified *compressive strength of masonry* is referenced by designers as the f'_m value. Values of historical test data have been adjusted statistically in the table to insure a conservative design value.

Typical allowable design values for hollow clay masonry range between 2600 psi and 5300 psi. At two to three times the standard value for concrete masonry, the use of these higher values can help to reduce deflections, wall thickness, beam depth, bar diameter and bar lap requirements.

To select an f'_m value higher than those listed in the table is acceptable; however, the value must be verified by building prisms of the brick, mortar and grout specified prior to and during construction.

type	parts by volume of portland cement or blended cement	parts by volume of hydrated lime or lime putty	aggregate, measured in a damp, loose condition	
			FINE	COARSE
FINE GROUT	1	0– $\frac{1}{10}$	2 $\frac{1}{4}$ –3 times the sum of the volumes of the cementitious materials	-----
COARSE GROUT	1	0– $\frac{1}{10}$	2 $\frac{1}{4}$ –3 times the sum of the volumes of the cementitious materials	1–2 times the sum of the volumes of the cementitious materials

Table 3. Grout Proportions by Volume for Masonry Construction. (IBC 2103.10) 2003 International Building Code. Copyright 2003. Falls Church, Virginia: International Code Council, Inc. Reproduced with permission. All rights reserved.

material	item number	construction	minimum required equivalent thickness for fire resistance ^{a, b, c} (inches)			
			4 hour	3 hour	2 hour	1 hour
BRICK OF CLAY OR SHALE	1-1.1	solid brick of clay or shale ^d	6	4.9	3.8	2.7
	1-1.2	hollow brick or tile of clay or shale, unfilled	5.0	4.3	3.4	2.3
	1-1.3	hollow brick or tile of clay or shale, grouted or filled with materials specified in Section 721.4.1.1.3	6.6	5.5	4.4	3.0

Table 4. Rated Fire-Resistance Periods of Clay Masonry Walls [IBC 721.4.1(1)]. For SI: 1" = 25.4 mm
a. Equivalent thickness as determined from Section 721.4.1.1, **b.** Calculated fire resistance between the hourly increments listed shall be determined by linear interpolation, **c.** Where combustible members are framed in the wall, the thickness of solid material between the end of each member and the opposite face of the wall, or between members set in from opposite sides, shall not be less than 93% of the thickness shown, **d.** For units in which the net cross-sectional area of cored brick in any plane parallel to the surface containing the cores is at least 75% of the gross cross-sectional area measured in the same plane. 2003 International Building Code. Copyright 2003. Falls Church, Virginia: International Code Council, Inc. Reproduced with permission. All rights reserved.

Masonry

Laid to Last

by Master

Craftsmen



Milan High School

Center for Forensic Study



PERFORMANCE

SKILLED FOR INTRICATE COMPLEXITIES

INNOVATION TO EXCEED ANY CHALLENGE

CREATIVE SCHEDULING AND SEQUENCING

MANPOWER & EQUIPMENT
TO MEET AGGRESSIVE SCHEDULE

ADDED VALUE

Cost saving budget analysis for architects
or construction managers during design

Quality • Safety • Pride in workmanship



Since 1957

Leidal & Hart
Mason Contractors

32225 Schoolcraft Rd • Livonia, MI 48150
(734) 522-2400 • Fax (734) 522-8650



Metro Loft, Royal Oak, designed by Neumann/Smith & Associates. Re-rod and concrete pre-fab flooring attached to 8 x 4 x 12 brick single wythe walls.

Mortar

Mortar used in reinforceable structural clay masonry assemblies is commonly specified by proportions. (See Table 2.) There are three types of mortars: Portland cement-lime, mortar cement and masonry cement. In addition, Type S mortars are prescribed for structural masonry as a minimum.

A common mistake is to over specify the mortar type. Since Type M mortars are the strongest mortars, they are often specified to insure strength of the assembly. Type M mortars have a high cement content which increases shrinkage of mortar and reduces the bond strength. In addition, the extra cement makes cleaning more difficult. Type M mortars are generally limited to below grade applications. When specifying mortars, do not call out for a mortar compressive strength testing.

Grout

Grouts for reinforceable structural clay masonry are prescribed according to Table 3 by proportion. Use coarse aggregate grout (1 part cement, 2 parts pea gravel and 3 parts sand) for applications where there is sufficient room for all of the reinforcing. Fine grouts are commonly used in smaller cells — typically 4" and 5" wide brick.

Fluidity is essential for proper placement of grout around reinforcing. A grout slump of 8" to 11" is recommended. Due to the high moisture

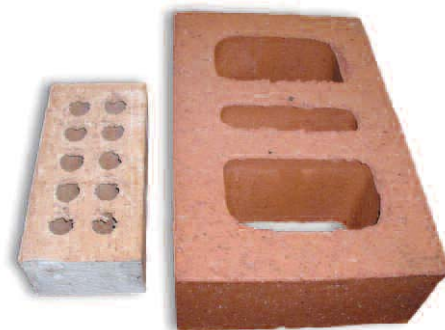
content of grout, a volume expander, fluidifying and water reducing agent are added.

Fire Resistance

Reinforceable structural clay brick also provide fire resistance. Structural clay brick are fired in kilns at maximum temperatures around 2000°F. This allows brick to retain strength during a fire and when subjected to fire hose streams, thus enhancing public safety. Structural brick are non-combustible and do not give off toxic emissions. Prescribed code values are located in Table 4. Fire ratings from 1 to 4 hours can be achieved from this table. To select a fire rating, solid percentages of structural brick are converted to an equivalent brick thickness and compared with minimum required thicknesses for various fire ratings. Structural brick may be made solid for the intent of this table by filling all cells with grout, expanded shale, sand, pea gravel, crushed stone, slag, pumice, scoria, expanded clay, slate, slag, fly ash or cinders.

Insulative R-Values

Structural clay brick can also provide insulative properties. Uninsulated brick have insulation resistance (R)-values of approximately 0.2 R/inch. By insulating the cells that are not grouted, higher insulation values can be achieved. Structural clay brick can be insulated with perlite or foamed-in-place polyurethane to achieve R-values approaching 1-R/inch.



Solid face brick and hollow structural unit.



Metro Loft, Royal Oak, constructed of Mission (yellowish, buffish) and Sunset, flashed to bring out color variation. Larger core holes for re-rod and grouting.

When insulation requirements exceed those provided above, insulate the wall the same as for brick veneer. Add a layer of closed cell polystyrene insulation to the back of the brick and fur out as required.

Reinforcement

From a design standpoint, reinforceable structural clay brick are similar to concrete masonry units (CMU). An architect or engineer familiar with the design of CMU uses many of the same details. Horizontal bar reinforcing is placed in bond beams. A grout stop fabric of paper or mesh is placed directly below bond beams to prevent grout from dropping into intentionally ungrouted cells.

Horizontal joint reinforcing is typically specified at 16" increments and ends are lapped 6". Reinforcing is typically continuous around corners. Horizontal reinforcing helps to control cracking and to transfer loads to vertical reinforcing.

Movement Joints

Concrete masonry uses control joints (CJs) in walls to control shrinkage of the concrete through the dynamic cycle of wetting and drying. CJs are commonly

STRUCTURAL CLAY BRICK

recommended at spacings of 20' to 25' centers and are created by starting and stopping the units to form a vertical joint. Reinforcing is terminated at the joint; sealants protect the joint from the elements. In some cases, a sash block and preformed gasket are used to help control alignment of the two faces. Care should be given to the aspect ratio of the wall to insure that cracking does not occur at more frequent spacings.

With structural clay brick, expansion and contraction will occur with varying temperatures; predominant direction of movement is expansion. When expansion of brick controls the design, the movement joint needs to be free of materials that will not compress. (See Figure 1.)

Expansion joints (EJs) are not required at the same locations and spacing as in brick veneer. This is because brick veneer does not have the ability to resist tension caused from expansion. Common spacing of EJs in horizontally reinforced clay brick are generally spaced at 60' centers. If an EJ is not preferred at the corner, horizontal reinforcing in bond beams must be continuous around the corner, a minimum of 36 bar diameters. Lap continuous reinforcing with corner bars to insure that tension created in the brick at the corner can be transferred to the reinforcing.

Where clay brick and concrete block

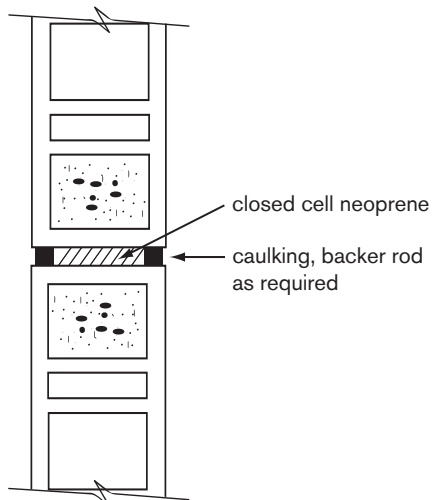


Figure 1.

Hilton Garden Inn, Detroit

Quality SINCE 1973
Masonry



GIANNOLA MASONRY COMPANY

(586) 792-2070

35086 Cordelia St. • Clinton Twp., MI 48035



Introducing... Eldorado Stone Core Products

The Most Believable Architectural Stone Veneer in the World

Core Profile products are the newest stone profiles available from Eldorado Stone and they are distributed locally by Darling Builders Supply Company. They are unsurpassed in appearance and follow Eldorado's strict philosophy of believability. The core line represents years of continuous innovation, research and development, as well as technical milestones in manufacturing, production and coloration.



517-484-5707

1600 Turner Street • Lansing

MASONRY SAWS & SUPPLIES DIAMOND BLADES

STIHL®

**When the cutting
gets tough...
that's where you'll find
STIHL Cutquiks.**



ALSO AVAILABLE

**Equipment Rental
Concrete Work
Concrete Drilling
Concrete Sawing
Parts - Service & Repair
Wall & Wire Sawing
Horizontal Curb Sawing**



**Diamond Chain Sawing
DETROIT
DIAMOND DRILLING,
INC.**

**7021 W. Eight Mile Road
(One Block West of Livernois)
Detroit, MI 48221**

**PH: 313.864.3600
Toll Free: 877.294.7787
Fax: 313. 864.3734**

STRUCTURAL CLAY BRICK

are combined, use CJs as required for concrete block.

Moisture Control

Moisture must be controlled on a regional basis. Experience with concrete masonry unit (CMU) in an area will dictate the preferred method of treating a structure for moisture resistance. For many areas of the United States, singlewythe brick exposing both faces can be treated using a water repellent coating on the exterior face. The expansive characteristic of brick causes reinforcing to go into tension, which then places the brick into compression. This post tensioning reduces cracks and decreases moisture penetration.

When a greater resistance to moisture penetration is required, structural clay masonry must be designed similar to drainage walls with weep holes and flashing running through the wall to the back side of the insulation. Close cell extruded expanded polystyrene, other non-wicking insulations and those not affected by moisture are placed in a continuous layer on the inside face of the brick. All joints in the insulation are taped to create a moisture barrier.

Continuous insulation creates the thermal drop which forces the dew point of the wall to occur in the insulation. This reduces the chance for condensation inside the wall. Any moisture on the surface of the insulation is directed to the flashing where it is directed to the exterior of the building.

Application

Common uses of reinforceable structural clay masonry are loadbearing walls, beams, columns and piers. Other uses are structural brick veneer, curtain walls, brick panels, sound walls and retaining walls.

One and two-story buildings, even those up to 21 stories, are perfect for loadbearing masonry. For a small increase in initial cost as compared with CMU, the owner enjoys major improvements in aesthetics and strength. As an alternative for steel frame and metal stud walls, reinforced structural clay masonry can be



Embassy Suites Hotel, Livonia used both 4" x 8" x 16" Ironstone flashed (with center score) as a veneer and 8" x 8" x 16" (with center score) as a structural unit.

used as both a bearing wall and an enclosure wall providing structure and finish at the same time. Compared with wood framing, structural clay brick offer better acoustical properties, less seismic or wind drift, better fire resistance, less maintenance and no insect or mold damage.

Structural clay masonry is often installed in less time than other framing methods, reducing construction time and finance costs.

Mid-rise hotels and condominiums are a perfect choice for the structural brick curtain wall. High strength of the unit allows taller walls and fewer framing members. Perimeter beam sizes can be reduced and heavy gauge studs are replaced by light gauge ones. On some building sites with limited access, the curtain wall concept has eliminated the need for scaffolding reducing those costs.

Brick veneer on metal studs is laden with complicated construction details and extra labor. Unless properly detailed, wall ties are subject to pullout and corrosion of the stud is possible. Structural brick veneer is a popular alternative to brick veneer as wall ties, heavy structural framing and beams are replaced with reinforced hollow brick. Wall ties are replaced by less frequent structural anchors spaced every 100 sf compared with every 2 sf. Anchors are less susceptible to corrosion. Perimeter

masonry beams can be sized for L/240 deflection criteria and not the L/600 prescribed to control veneer cracking. Brick offer greater design flexibility in a wide array of colors, textures and patterns.

Sound walls and miscellaneous enclosure walls are an efficient use of structural clay masonry. Modular in size, they can accommodate large variations in land form. A wide variety of colors add structural and aesthetic advantages.

Structural clay brick are less frequently used in retaining walls. However, because of the flexibility of brick to accommodate variations in wall geometry, the structural brick retaining wall may offer more efficient design options as compared with reinforced concrete.

It is safe to say that any building designed using brick veneer can be designed using reinforced structural clay brick. The reverse is not true.

Complexity of design can be reduced, performance of materials enhanced, overall cost reduced, performance life increased by using reinforceable structural clay brick. Brick's durability is double the useful life of most building materials.

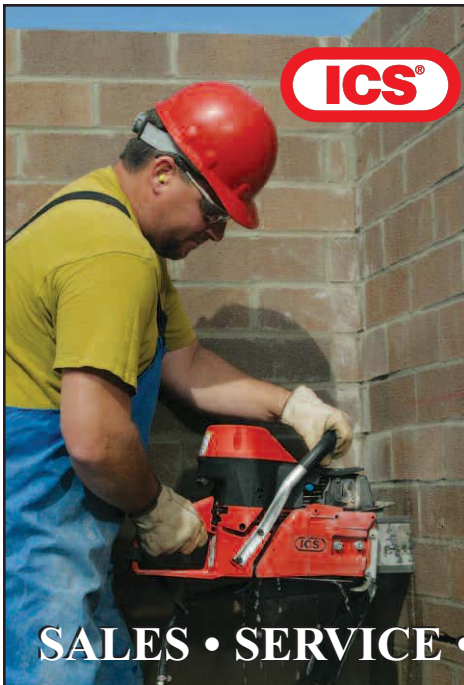
Reinforceable structural clay brick fulfills the intent of sustainable building design.

Take advantage of the structural characteristics of brick while enjoying its long list of other advantages.

For additional information on this subject, go to www.brick-wscpa.org. @

Jeff Elder is sales manager for Interstate Brick Company, nationally recognized structural clay brick manufacturer in West Jordan, UT. He has served as past president and currently chairs the technical committee for Western States Clay Products Association (WSCPA), the structural research arm of the brick industry. He is a member of Masonry Standards Joint Committee, Masonry Alliance for Codes & Standards and The Masonry Society. He may be reached at jeff.elder@paccoast.com, 800-280-5290.







The best way to cut concrete™

- ◆ SAW PACKAGES
- ◆ DIAMOND CHAINS
- ◆ GUIDE BARS
- ◆ ACCESSORIES
- ◆ EQUIPMENT
- ◆ CALL FOR FREE DEMO

SALES • SERVICE • PARTS • RENTAL



Phone: (248) 449-4944 Fax: (248) 449-4946

25806 Novi Rd • Novi, MI 48375

www.acecutting.com

email sales@acecutting.com



Helms Masonry Inc.

IF SATISFIED, TELL OTHERS. IF NOT, TELL US.

PHONE 616-583-0293
FAX 616-583-0296
helmsmasonry@email.msn.com
8625 Byron Commerce Drive
Byron Center, MI 49315

**COMMERCIAL
INDUSTRIAL**

**2002 ABC MASONRY AWARD
OF EXCELLENCE**
 Beechwood Reformed Church, Holland

	EVART HELMS 616-292-3676		DEAN HELMS 616-292-3677		

Serving Western Michigan for over 40 years



Atlas® Brick Dimensions, Weights, Coverage and Packaging

Name	Depth (D)		Height (H)		Length (L)		Weights		Units /Area		Packaging	
	in	mm	in	mm	in	mm	lbs	kg	Units /ft²	Units /m²	Type	QTY
Atlas® Utility 4 x 4 x 12	3 5/8	92	3 5/8	92	11 5/8	295	8.2	3.7	3.00	32.3	Cube	207
Atlas® Emperor® 4 x 4 x 16	3 5/8	92	3 5/8	92	15 5/8	397	11.0	5.0	2.25	24.2	Pallet	160
Atlas® Super Emperor® 4 x 8 x 16	3 5/8	92	7 5/8	194	15 5/8	397	22.5	10.2	1.13	12.1	Pallet	64
Atlas® 6 x 2 1/4 x 12	5 5/8	143	2 1/4	57	11 5/8	295	8.0	3.6	4.5	48.4	Pallet	180
Atlas® 6 x 2 1/4 x 16	5 5/8	143	2 1/4	57	15 5/8	397	10.0	4.5	3.38	36.3	Pallet	150
Atlas® 6 x 4 x 12	5 5/8	143	3 5/8	92	11 5/8	295	10.0	4.5	3	32.3	Pallet	120
Atlas® 6 x 4 x 16	5 5/8	143	3 5/8	92	15 5/8	397	14.0	6.3	2.25	24.2	Pallet	100
Atlas® 8 x 4 x 12	7 5/8	194	3 5/8	92	11 5/8	295	15.3	6.9	3.00	32.3	Pallet	100
Atlas® 8 x 2 1/4 x 16	7 5/8	194	2 1/4	57	15 5/8	397	12.7	5.7	3.38	36.3	Pallet	120
Atlas® 8 x 4 x 16	7 5/8	194	3 5/8	92	15 5/8	397	17.0	7.7	2.25	24.2	Pallet	80
Super Atlas® 8 x 8 x 16	7 5/8	194	7 5/8	194	15 5/8	397	39.0	17.6	1.13	12.1	Pallet	40
Atlas® 10 x 2 1/4 x 16	9 5/8	244	2 1/4	57	15 5/8	397	16.0	7.2	3.38	36.3	Pallet	90
Atlas® 10 x 4 x 16	9 5/8	244	3 5/8	92	15 5/8	397	20.0	9.1	2.25	24.2	Pallet	60

Weights shown are approximate dry weights. Weight will vary with moisture content and with color. Coring configuration varies depending on brick size.

Please refer to individual brick sizes on the website for accurate configurations.

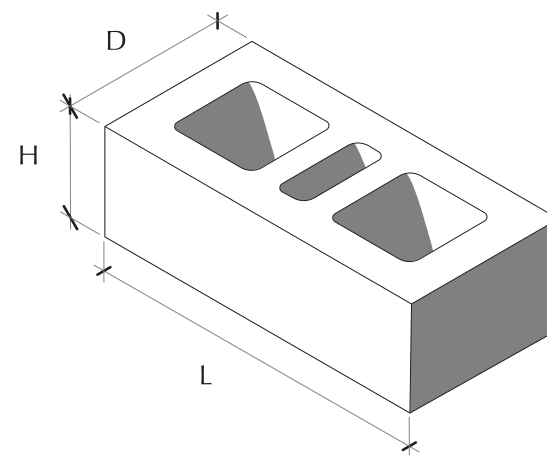
ONLY AVAILABLE AS SPECIAL ORDER

The Above Brick Manufactured by Interstate Brick comply with the following Specifications:

ASTM C652: Grade SW; Type HBA, HBS or HBX as specified in Purchase Order

CSA A82: Grade EG; Type HBA, HBS or HBX as specified in Purchase Order

Revision-0225



SECTION 1: Identification of the substance/mixture and of the company/undertaking

1.1. Product identifier

Product name : Interstate - Standard Brick Products
Product code : Not available

1.2. Relevant identified uses of the substance or mixture and uses advised against

Use of the substance/mixture : Building and construction work

1.3. Details of the supplier of the safety data sheet

Interstate Brick Company
9780 S. 5200 W.
West Jordan, UT 84088 - U.S.A.
T (801) 280-5200

1.4. Emergency telephone number

Emergency number : CHEMTREC 1 (800) 424-9300
CHEMTREC International +1 (703) 527-3887 24 hr

SECTION 2: Hazards identification

2.1. Classification of the substance or mixture

GHS-US classification

Skin irritation 2
Eye irritation 2A
Skin sensitization 1B
Carcinogenicity 1A
Specific target organ toxicity - Single exposure 3
Specific target organ toxicity - Repeated exposure 1

2.2. Label elements

GHS-US labelling

Hazard pictograms (GHS-US) :



GHS07



GHS08

Signal word (GHS-US) :

Danger

Hazard statements (GHS-US) :

Causes skin irritation. Causes serious eye irritation. May cause an allergic skin reaction. May cause cancer. May cause respiratory irritation. Causes damage to organs through prolonged or repeated exposure.

Precautionary statements (GHS-US) :

Contaminated work clothing must not be allowed out of the workplace. Obtain special instructions before use. Do not handle until all safety precautions have been read and understood. Wear protective gloves/protective clothing/eye protection/face protection. Use only outdoors or in a well-ventilated area. Do not breathe dust. Wash hands thoroughly after handling. Do not eat, drink or smoke when using this product. If exposed or concerned: Get medical advice/attention. If on skin: Wash with plenty of water. Take off contaminated clothing and wash it before reuse. If skin irritation or rash occurs: Get medical advice/attention. If in eyes: Rinse cautiously with water for several minutes. Remove contact lenses, if present and easy to do. Continue rinsing. If eye irritation persists: Get medical advice/attention. If inhaled: Remove person to fresh air and keep comfortable for breathing. Call a poison center/doctor if you feel unwell. Store in a well-ventilated place. Dispose of contents and container in accordance with all local, regional, national and international regulations.

2.3. Other hazards

No additional information available

SECTION 3: Composition/information on ingredients

3.1. Substance

Not applicable

Interstate® Brick - Standard Brick Products

Safety Data Sheet

according to the Hazard Communication Standard (CFR29 1910.1200) HazCom 2012

3.2. Mixture

Name	Product identifier	%	GHS-US classification
Clay/Shale Aluminum Silicates	Proprietary	90 - 95	Skin Irrit. 2 Eye Irrit. 2A Skin Sens. 1B Carc. 1A STOT SE 3 STOT RE 1
Quartz	(CAS No) 14808-60-7	40 - 75 ¹	Carc. 1A STOT RE 1
Manganese dioxide	(CAS No) 1313-13-9	< 3	Ox. Sol. 3 Acute Tox. 4 (Oral) Acute Tox. 4 (Inhalation)
Chromium oxide (Cr ₂ O ₃)	(CAS No) 1308-38-9	< 3	Not classified

1: Quartz is present in Clay/Shale Aluminum Silicates at 40 - 75%.

* The specific chemical identity and exact percentage (concentration) of composition has been withheld as a trade secret in accordance with paragraph (i) of §1910.1200.

SECTION 4: First aid measures

4.1. Description of first aid measures

- First-aid measures after inhalation : If inhaled, remove to fresh air. If not breathing, give artificial respiration. If breathing is difficult, give oxygen. Get medical advice/attention if you feel unwell.
- First-aid measures after skin contact : In case of contact, immediately flush skin with plenty of water. Remove contaminated clothing and shoes. Wash clothing before reuse. If skin irritation or rash occurs: Get medical advice/attention.
- First-aid measures after eye contact : In case of contact, immediately flush eyes with plenty of water for at least 15 minutes. If easy to do, remove contact lenses, if worn. If irritation persists, get medical attention.
- First-aid measures after ingestion : If swallowed, do NOT induce vomiting unless directed to do so by medical personnel. Never give anything by mouth to an unconscious person. Get medical advice/attention if you feel unwell.

4.2. Most important symptoms and effects, both acute and delayed

- Symptoms/injuries after inhalation : May cause respiratory irritation. May cause shortness of breath, tightness of the chest, a sore throat and cough. Repeated exposure to very high levels of respirable crystalline silica quartz can cause acute silicosis.
- Symptoms/injuries after skin contact : Causes skin irritation. Symptoms may include redness, edema, drying, defatting and cracking of the skin. May cause an allergic skin reaction.
- Symptoms/injuries after eye contact : Causes serious eye irritation. Symptoms may include discomfort or pain, excess blinking and tear production, with marked redness and swelling of the conjunctiva.
- Symptoms/injuries after ingestion : May be harmful if swallowed. May cause stomach distress, nausea or vomiting.

4.3. Indication of any immediate medical attention and special treatment needed

Symptoms may not appear immediately. In case of accident or if you feel unwell, seek medical advice immediately (show the label or SDS where possible).

SECTION 5: Firefighting measures

5.1. Extinguishing media

- Suitable extinguishing media : Treat for surrounding material.
- Unsuitable extinguishing media : None known.

5.2. Special hazards arising from the substance or mixture

- Fire hazard : Products of combustion may include, and are not limited to: oxides of carbon.

5.3. Advice for firefighters

- Protection during firefighting : Keep upwind of fire. Wear full fire fighting turn-out gear (full Bunker gear) and respiratory protection (SCBA).

SECTION 6: Accidental release measures

6.1. Personal precautions, protective equipment and emergency procedures

- General measures : Use personal protection recommended in Section 8. Isolate the hazard area and deny entry to unnecessary and unprotected personnel. Do not breathe dust.

6.2. Methods and material for containment and cleaning up

- For containment : Contain spill, then place in a suitable container. Minimize dust generation. Do not flush to sewer or allow to enter waterways. Use appropriate Personal Protective Equipment (PPE).
- Methods for cleaning up : Vacuum or sweep material and place in a disposal container. Provide ventilation.

6.3. Reference to other sections

See section 8 for further information on protective clothing and equipment and section 13 for advice on waste disposal.

Interstate® Brick - Standard Brick Products

Safety Data Sheet

according to the Hazard Communication Standard (CFR29 1910.1200) HazCom 2012

SECTION 7: Handling and storage

7.1. Precautions for safe handling

- Precautions for safe handling : Avoid contact with skin and eyes. Do not breathe dust. Avoid generating dust. Good housekeeping is important to prevent accumulation of dust. Do not swallow. Handle in accordance with good industrial hygiene and safety practice. When using do not eat, drink or smoke. Use only outdoors or in a well-ventilated area.
- Hygiene measures : Launder contaminated clothing before reuse. Wash hands before eating, drinking, or smoking.

7.2. Conditions for safe storage, including any incompatibilities

- Storage conditions : Keep out of the reach of children. Store in a well-ventilated place. Avoid any dust buildup by frequent cleaning and suitable construction of the storage area.

7.3. Specific end use(s)

Not available.

SECTION 8: Exposure controls/personal protection

8.1. Control parameters

Clay/Shale Aluminum Silicates (Proprietary)

ACGIH	ACGIH TWA (mg/m ³)	10 mg/m ³
OSHA	OSHA PEL (TWA) (mg/m ³)	15 mg/m ³

Quartz (14808-60-7)

ACGIH	ACGIH TWA (mg/m ³)	0.025 mg/m ³ (respirable fraction)
OSHA	OSHA PEL (TWA) (mg/m ³)	((10 mg/m ³)/(%SiO ₂ +2) TWA (resp)) ((30 mg/m ³)/(%SiO ₂ +2) TWA (total)) ((250)/(%SiO ₂ +5) mppcf TWA (resp))

Manganese dioxide (1313-13-9)

ACGIH	ACGIH TWA (mg/m ³)	0.2 mg /m ³ (Mn)
OSHA	OSHA PEL (TWA) (mg/m ³)	5 mg/m ³ (Mn)

Chromium oxide (Cr₂O₃) (1308-38-9)

ACGIH	ACGIH TWA (mg/m ³)	0.05 mg /m ³
OSHA	OSHA PEL (TWA) (mg/m ³)	0.05 mg /m ³

8.2. Exposure controls

- Appropriate engineering controls : Use ventilation adequate to keep exposures (airborne levels of dust, fume, vapor, etc.) below recommended exposure limits.
- Personal protective equipment : Avoid all unnecessary exposure.
- Hand protection : Wear chemically resistant protective gloves.
- Eye protection : Safety glasses with side shields or goggles are recommended for dust.
- Skin and body protection : Wear suitable protective clothing.
- Respiratory protection : A NIOSH approved dust mask or filtering facepiece is recommended in poorly ventilated areas or when permissible exposure limits may be exceeded. Respirators should be selected by and used under the direction of a trained health and safety professional following requirements found in OSHA's respirator standard (29 CFR 1910.134) and ANSI's standard for respiratory protection (Z88.2).
- Environmental exposure controls : Maintain levels below Community environmental protection thresholds.
- Other information : Do not eat, smoke or drink where material is handled, processed or stored. Wash hands carefully before eating or smoking. Handle according to established industrial hygiene and safety practices.

SECTION 9: Physical and chemical properties

9.1. Information on basic physical and chemical properties

- Physical state : Solid
- Appearance : Vitrified solid
- Color : Various
- Odor : Odorless
- Odor threshold : No data available

Interstate® Brick - Standard Brick Products

Safety Data Sheet

according to the Hazard Communication Standard (CFR29 1910.1200) HazCom 2012

pH	: No data available
Melting point	: No data available
Freezing point	: No data available
Boiling point	: No data available
Flash point	: No data available
Relative evaporation rate (butylacetate=1)	: No data available
Flammability (solid, gas)	: Not flammable
Explosive limits	: No data available
Explosive properties	: No data available
Oxidising properties	: No data available
Vapor pressure	: No data available
Relative density	: 2.6
Relative vapor density at 20 °C	: No data available
Solubility	: Insoluble
Partition coefficient: n-octanol/water	: No data available
Log Kow	: No data available
Auto-ignition temperature	: No data available
Decomposition temperature	: No data available
Viscosity	: No data available
Viscosity, kinematic	: No data available
Viscosity, dynamic	: No data available

9.2. Other information

No additional information available

SECTION 10: Stability and reactivity

10.1. Reactivity

No dangerous reaction known under conditions of normal use.

10.2. Chemical stability

Stable under normal storage conditions.

10.3. Possibility of hazardous reactions

No dangerous reaction known under conditions of normal use.

10.4. Conditions to avoid

None known.

10.5. Incompatible materials

None known.

10.6. Hazardous decomposition products

May include, and are not limited to: oxides of carbon.

SECTION 11: Toxicological information

11.1. Information on toxicological effects

Acute toxicity : Not classified.

Interstate - Standard Brick Products	
LD50 oral rat	> 2000 mg/kg
LD50 dermal rabbit	No data available
LC50 inhalation rat	> 5 mg/l/4h

Manganese dioxide (1313-13-9)	
LD50 oral rat	> 3478 mg/kg

Chromium oxide (Cr ₂ O ₃) (1308-38-9)	
LD50 oral rat	>15000 mg/kg
LC50 inhalation rat	> 5.41 mg/l/4h

Skin corrosion/irritation : Causes skin irritation.

Interstate® Brick - Standard Brick Products

Safety Data Sheet

according to the Hazard Communication Standard (CFR29 1910.1200) HazCom 2012

Serious eye damage/irritation	: Causes serious eye irritation.
Respiratory or skin sensitisation	: May cause an allergic skin reaction.
Germ cell mutagenicity	: Based on available data, the classification criteria are not met.
Carcinogenicity	: May cause cancer.

Quartz (14808-60-7)

IARC group	1 - Carcinogenic to humans
National Toxicology Program (NTP) Status	2 - Known Human Carcinogens

Chromium oxide (Cr₂O₃) (1308-38-9)

IARC group	3 - Not classifiable
Reproductive toxicity	: Based on available data, the classification criteria are not met.
Specific target organ toxicity (single exposure)	: May cause respiratory irritation.
Specific target organ toxicity (repeated exposure)	: Causes damage to organs through prolonged or repeated exposure. Respirable crystalline silica in the form of quartz or cristobalite from occupational sources is listed by the International Agency for Research on Cancer (IARC) and National Toxicology Program (NTP) as a lung carcinogen. Prolonged exposure to respirable crystalline silica has been known to cause silicosis, a lung disease, which may be disabling. While there may be a factor of individual susceptibility to a given exposure to respirable silica dust, the risk of contracting silicosis and the severity of the disease is clearly related to the amount of dust exposure and the length of time (usually years) of exposure.
Aspiration hazard	: Based on available data, the classification criteria are not met.
Symptoms/injuries after inhalation	: May cause respiratory irritation. May cause shortness of breath, tightness of the chest, a sore throat and cough. Repeated exposure to very high levels of respirable crystalline silica quartz can cause acute silicosis.
Symptoms/injuries after skin contact	: Causes skin irritation. Symptoms may include redness, edema, drying, defatting and cracking of the skin. May cause an allergic skin reaction.
Symptoms/injuries after eye contact	: Causes serious eye irritation. Symptoms may include discomfort or pain, excess blinking and tear production, with marked redness and swelling of the conjunctiva.
Symptoms/injuries after ingestion	: May be harmful if swallowed. May cause stomach distress, nausea or vomiting.

SECTION 12: Ecological information

12.1. Toxicity

Ecology - general	: May cause long-term adverse effects in the aquatic environment.
-------------------	---

12.2. Persistence and degradability

Interstate - Standard Brick Products

Persistence and degradability	Not established.
-------------------------------	------------------

12.3. Bioaccumulative potential

Interstate - Standard Brick Products

Bioaccumulative potential	Not established.
---------------------------	------------------

12.4. Mobility in soil

No additional information available

12.5. Other adverse effects

Effect on the global warming	: No known ecological damage caused by this product.
------------------------------	--

SECTION 13: Disposal considerations

13.1. Waste treatment methods

Waste disposal recommendations	: This material must be disposed of in accordance with all local, state, provincial, and federal regulations. The generation of waste should be avoided or minimized wherever possible.
--------------------------------	---

SECTION 14: Transport information

Department of Transportation (DOT)

In accordance with DOT
Not regulated for transport

Additional information

Other information	: No supplementary information available.
-------------------	---

Interstate® Brick - Standard Brick Products

Safety Data Sheet

according to the Hazard Communication Standard (CFR29 1910.1200) HazCom 2012

Special transport precautions : Do not handle until all safety precautions have been read and understood.

SECTION 15: Regulatory information

15.1. US Federal regulations

All components of this product are listed, or excluded from listing, on the United States Environmental Protection Agency Toxic Substances Control Act (TSCA) inventory.

15.2. US State regulations

Interstate - Standard Brick Products

State or local regulations	This product contains Crystalline Silica, Quartz and may also contain trace amounts of other chemicals known to the State of California to cause cancer, birth defects or other reproductive harm.
----------------------------	--

SECTION 16: Other information

Date of issue : 03/26/2015

Other information : None.

Disclaimer: We believe the statements, technical information and recommendations contained herein are reliable, but they are given without warranty or guarantee of any kind. The information contained in this document applies to this specific material as supplied. It may not be valid for this material if it is used in combination with any other materials. It is the user's responsibility to satisfy oneself as to the suitability and completeness of this information for the user's own particular use.

