Concrete Masonry Walls for Metal Building Systems

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DISCLAIMER

This manual is intended as a guide for the designer, contractor, and owner in the preparation of constructing concrete masonry walls with metal building systems. The discussion, design aides, and construction details are intended to assist architects and engineers in the design of masonry walls for metal building systems and to acquaint builders and contractors with recommended construction methods and details.

The material presented does not cover all possible situations but is intended to represent some of the more widely used concrete masonry construction practices and other pertinent information. Care has been taken to ensure that the information included in this manual is as accurate as possible. However, NCMA, MBMA, and ICC do not assume responsibility for errors or omissions resulting from the use of this manual or in preparation of plans or specifications. Additionally, information contained herein may not conform to local building code requirements and should therefore be reviewed carefully to assure compliance.

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Chapter 1

Introduction

1.0 General

Metal buildings are used extensively for warehouses and other structures requiring large, open floor spaces. The ability of metal building manufacturers to meet these needs by standardizing design and construction details has caused these buildings to often be mistakenly referred to as "metal building systems." Metal buildings provide considerable design flexibility, long clear spans and rapid construction. As part of this flexibility, metal buildings can be clad with a variety of materials to provide different appearances or functions to the buildings. Reinforced concrete masonry walls as detailed in this manual are popular enclosure systems for metal buildings because of masonry's impact resistance, strength, and fire resistance. The attractiveness and durability of masonry combined with the steel frame make this building type ideal not only for warehouses but also for commercial buildings, schools, and gymnasiums.

Some of the benefits concrete masonry walls add to metal buildings include:

Aesthetics	Design Flexibility
Low Cost	Rapid Construction
Strength	Durability
Impact Resistance	Security
Energy Efficiency	Fire Resistance
Noise Reduction	Water Resistance

Each of these properties will be discussed in the following sections of this chapter and throughout this publication. More detailed information on these and other topics is available free online through NCMA's website www.ncma.org.

When integrating concrete masonry and steel frames into a single structural system, special considerations need to be addressed to account for the unique properties of each of these building materials. Chapter 1 of this guide provides a general overview of the design and detailing recommendations for serviceability and performance. Structural design considerations are reviewed in detail in Chapter 2. Finally, Chapter 3 provides guidance for the construction of metal buildings integrating concrete masonry. Note that unreinforced (plain masonry) is not addressed in this manual which focuses on design and detailing of reinforced masonry assemblies –with or without a masonry veneer finish. Reinforcing steel provides ductility to concrete masonry walls to make them more compatible with the highly flexible metal building frames.

1.1 Aesthetic Appeal

Historically, most concrete masonry structures have been constructed with standard grey block that can be used as either the finished surface or as a base for other finish treatments, including stucco, paint, and anchored or adhered masonry veneer. More recently, architectural units such as color tinted, split faced, burnished and scored units have been used to add attractive finishes to metal buildings. Architectural units can be used for the entire façade or for banding courses to achieve specific patterns or to highlight certain aspects of the building.

Although construction with staggered vertical mortar joints (running bond) is standard, the appearance of continuous vertical mortar joints (stack bond pattern) can be achieved through the use of scored units or by using reinforced stack bond construction. Note that the design charts used in this manual only address running bond, however.

More information on the aesthetic attributes and options of concrete masonry construction can be found in the following TEK (ref. 13):

- *TEK 2-1A Typical Sizes and Shapes of Concrete Masonry Units*
- *TEK 2-2B Considerations for Using Specialty Concrete Masonry Units*
- *TEK 2-3A Architectural Concrete Masonry Units*
- *TEK 5-2A Clay and Concrete Masonry Banding Details*
- *TEK 5-10A Concrete Masonry Radial Wall Details*
- *TEK 5-15 Details for Half-High Concrete Masonry Units*
- *TEK 5-16 Aesthetic Design with Concrete Masonry*
- *TEK 14-6 Concrete Masonry Bond Patterns*

Design inspiration can also be found in NCMA's bi-monthly publication *Concrete Masonry Designs* (ref. 14).

1.2 Low Cost for Construction and Maintenance, Design Flexibility, and Rapid Construction

Concrete masonry construction costs are relatively low because the materials are inexpensive, the construction is straightforward and the work proceeds rapidly.

Accordingly, concrete masonry construction is competitive with other cladding systems while maintaining high standards of quality and performance. One often overlooked advantage of concrete masonry is its modular nature that can easily accommodate changes in floor plans or wall heights. Corners, tees, and curves can easily be laid out. Forming or framing these floor plan changes with other cladding materials is often difficult, making concrete masonry especially attractive and cost efficient. This flexibility for design and construction also translates to speed of construction. Large areas of the floor slab for casting panels or special casting beds are not required or large equipment to handle panels or extensive curing times before walls can be put in place. For large wall areas, power (crank-up or hydraulic) scaffolds greatly increase construction efficiency and reduce construction time for masonry.

Concrete masonry structures can be constructed using virtually any layout dimension. However, for maximum construction efficiency and economy, concrete masonry elements should be designed and constructed with modular coordination in mind. Modular coordination is the practice of laying out and dimensioning structures to standard lengths and heights to accommodate modular sized building materials. Standard concrete masonry modules are typically 8 in. (203 mm) vertically and horizontally, but may also include 4- in. (102-mm) modules for some applications. These modules provide the best overall design flexibility and coordination with other building products such as windows and doors.

Maintenance costs for concrete masonry walls are low as well because concrete masonry is tough, durable, and colorfast. Masonry resists incidental impact loads from hand carts and forklifts as well or better than other cladding systems, making frequent and costly repairs unnecessary.

More information on construction and maintenance can be found in the following TEK (ref. 13):

- *TEK 3-8A Concrete Masonry Construction*
- *TEK 4-1A Productivity and Modular Coordination in Concrete Masonry Construction*
- *TEK 4-2A Estimating Concrete Masonry Materials*
- *TEK 5-9A Concrete Masonry Corner Details*
- *TEK 5-10A Concrete Masonry Radial Wall Details*
- *TEK 5-12 Modular Layout of Concrete Masonry*
- *TEK 8-1A Maintenance of Concrete Masonry Walls*

1.3 Strength, Durability, and Security

The strength and durability of concrete masonry make it popular for both above grade and below grade construction. Disasters such as hurricanes, earthquakes, and explosions have repeatedly shown the toughness of properly designed and constructed concrete masonry buildings.

This same strength makes concrete masonry ideal for metal building warehouses, manufacturing plants, and commercial buildings. Concrete masonry walls can span large distances, and when reinforced and properly detailed, they can accommodate the deflections and lateral drifts of metal building systems.

Detailing information and design charts are provided in Chapter 2 to assist with the design of typical masonry walls for metal buildings.

More information on the strength, durability and security of concrete masonry can be found on line in the following TEK (ref. 13):

- *TEK 3-2A Grouting Concrete Masonry Walls*
- *TEK 3-8A Concrete Masonry Construction*
- *TEK 5-14 Concrete Masonry Hurricane and Tornado Shelters*
- *TEK 14-5A Loadbearing Concrete Masonry Wall Design*

- *TEK 14-10B Impact Resistance of Concrete Masonry Walls for Correctional Facilities*
- *TEK 14-18B Prescriptive Seismic Reinforcement Requirements for Masonry Structures*
- *TEK 14-21 Blast and Bullet Resistant Concrete Masonry Buildings*

1.4 Energy Efficiency of Concrete Masonry Construction

Concrete masonry walls are easily insulated with a variety of products including rigid insulation, insulation inserts, loose fill insulation, and foamed in-place insulation. Due to the susceptibility of batt insulation to moisture, caution should be used when using this, or any other moisture-sensitive material, to help ensure it remains dry during the structure's service life. Depending on the particular site conditions and owner's preference, insulation may be placed on the outside of block walls, on the interior faces of the walls, in the cores of hollow units, or in the cavity space of multi-wythe walls.

The thermal mass of concrete masonry also contributes to maintaining consistency of the minimum temperatures in buildings required by Model Building Codes (IBC Section 1204), thus providing a more comfortable working and living environment. Similarly, reduced mechanical equipment on-off cycles contributes toward lower heating and cooling bills.

Figure 1.1—Several of Many Insulation Strategies for Concrete Masonry Walls.

Further, contributions of thermal mass as historically associated with mass walls allows the least stringent tabulated R-Values and U-Factors of any of the listed wall assemblies found within the Model International Energy Conservation Code and ASHRAE 90.1 Standard. Multiple insulation strategies are shown in Figure 1.1. As an example, utilizing 3 inches (76 mm) of polyisocyanurate board insulation can result in an opaque masonry wall assembly R-factor in excess of R 26. Therefore taking into account the contribution of thermal mass of concrete masonry results in exemplary energy conservation. It is assumed that proper masonry moisture control such as flashing and integral water repellents along with vapor and air permeance detailing such as interior masonry coatings and/or taped insulation board or otherwise sealed insulation are incorporated. See TEK 6-1B and TEK 6-2B (ref. 13) for more detailed information.

More information on energy efficiency and insulation strategies of concrete masonry walls can be found in the Chapter 6 of the TEK Manual (ref. 13). Select documents include:

- *TEK 6-1B R-Values of Multi-Wythe Concrete Masonry Walls*
- *TEK 6-2B R-Values and U-Factors for Single Wythe Concrete Masonry Walls*
- *TEK 6-3 Shifting Peak Energy Loads with Concrete Masonry Construction*
- *TEK 6-4A Energy Code Compliance Using COMCheck*
- *TEK 6-5A Passive Solar Design Strategies*
- *TEK 6-10A Concrete Masonry Radiant Heating/Cooling Systems*
- *TEK 6-11A Insulating Concrete Masonry Walls*
- *TEK 6-12C International Energy Conservation Code and Concrete Masonry*

The Metal Building Manufacturers Association (MBMA) published the 2010 *Energy Design Guide for Metal Building Systems* (ref. 16) to aid building owners, architects, specifiers, contractors, builders and metal building manufacturers in their efforts for building energy conservation. The *Energy Design Guide for Metal Building Systems* is a synthesis of all of the pertinent information on how to design, construct, and maintain metal buildings to be energy efficient. The guide also includes common ways to insulate a metal building roof and walls to meet today's energy codes and standards.

1.5 Fire Resistance

Concrete masonry is noncombustible and effectively resists the passage of flames, smoke, and heat. Accordingly, building codes assign high fire ratings to concrete masonry walls, making them effective fire walls for warehouses, hotels, apartments, and other structures. These same attributes allow concrete masonry to protect steel columns and beams in metal buildings from fire.

TEK 7-1C (ref. 13) provides code based tables and design information for determining fire resistance ratings for concrete masonry assemblies. For instance, from the tables in the TEK, it can be seen that two hour fire ratings can be achieved by most 8 in. (203 mm) hollow units. Filling the cells with the following materials will result in fire resistance ratings of over 4 hours for 8-in. (203 mm) concrete masonry: grout that complies with ASTM C476 (ref. 8), aggregates in the form of sand, pea gravel, crushed stone, or slag that comply with ASTM C33 (ref. 7), pumice, scoria, expanded shale, expanded clay, expanded slate, expanded slag, expanded fly ash, or cinders that comply with ASTM C331 (ref. 9), perlite meeting the requirements of ASTM C549 (ref. 11), or vermiculite complying with ASTM C516 (ref. 12).

TEK (ref. 13) with more detailed information on concrete masonry fire resistance are:

- *TEK 7-1C Fire Resistance Rating of Concrete Masonry Assemblies*
- *TEK 7-5A Evaluating Fire-Exposed Concrete Masonry Walls*
- *TEK 7-6A Steel Column Fire Protection*

In addition, a Fire, Energy, and Sound Calculator is available free on line (www.ncma.org) to provide easy calculation of fire resistance ratings for concrete masonry assemblies, including concrete masonry fire protection for steel columns.

The Metal Building Manufacturers Association (MBMA) has published the 2010 *Fire Resistance Design Guide for Metal Building Systems* (ref. 17) to address steel fire protection for low-rise metal building systems. The guide includes essential basic background information for practitioners not familiar with the subject, as well as some advanced guidance and insights for more experienced users. The guide also includes twelve MBMA-sponsored assembly listings, as well as non-MBMA listings that can be integrated with metal building systems, such as concrete masonry walls.

1.6 Noise Control

Concrete masonry is an ideal noise control material in two important ways. First, masonry walls act as barriers that block sound transmission over a wide range of frequencies. Outdoor sounds and sounds from other areas of a building are thus reflected away by concrete masonry walls.

Secondly, concrete masonry is also an effective material for absorbing noise generated within a room (see Figure 1.2). This sound absorbing ability of concrete masonry is unique as other barrier surfaces may reflect sound without diminishing it.

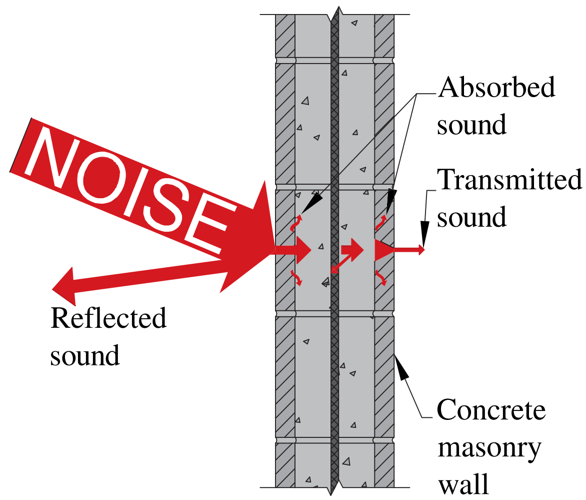


Figure 1.2—Sound Reflection and Absorption Characteristics of Concrete Masonry

The combined ability of concrete masonry to absorb noise and block the passage of sound have made it a material of choice for highway sound walls, manufacturing plants, gymnasiums, apartments, and hotel separation walls.

TEK (ref. 13) that provide more detailed information on noise control are:

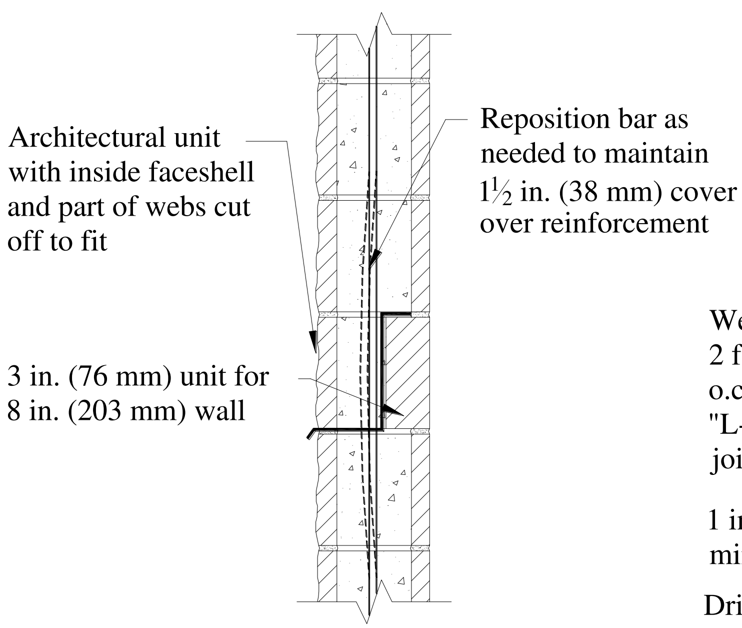
- *TEK 13-1B Sound Transmission Class Ratings for Concrete Masonry Walls*
- *TEK 13-2A Noise Control with Concrete Masonry*
- *TEK 13-3A Concrete Masonry Highway Sound Barriers*
- *TEK 13-4 Outside-Inside Transmission Class of Concrete Masonry Walls*
- *TEK 14-15B Allowable Stress Design of Pier and Panel Highway Sound Barrier Walls*

1.7 Water Penetration Resistance

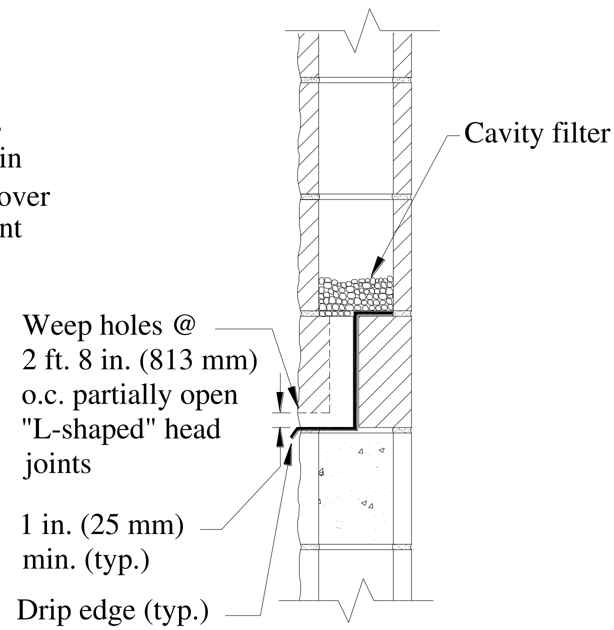
When properly designed, constructed and maintained, concrete masonry walls provide excellent resistance to water penetration. Concrete masonry walls readily accept a wide range of water repellent and waterproof coatings. For additional protection, integral water repellent admixtures can be added into the concrete masonry units and mortar. Cavity wall systems provide an even higher level of moisture resistance, as the outer wythe sheds most of the rain, and the cavity prevents any water that does penetrate the outer wythe from reaching the inner wythe of masonry. Industry recommended construction details to mitigate water penetration are illustrated in Figure 1.3.

Additional information regarding water penetration resistance design and detailing is available on line in Chapter 19 of the TEK Manual (ref. 13). TEK of particular interest are:

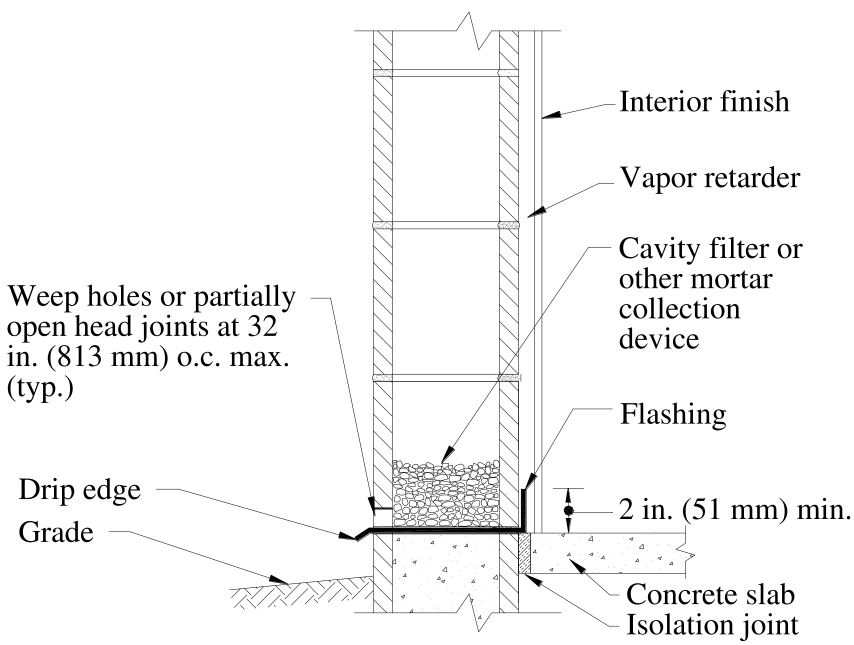
- *TEK 19-1 Water Repellents for Concrete Masonry Walls*
- *TEK 19-2A Design for Dry Single Wythe Concrete Masonry Walls*
- *TEK 19-4A Flashing Strategies for Concrete Masonry Walls*
- *TEK 19-5A Flashing Details for Concrete Masonry Walls*
- *TEK 19-6 Joint Sealants for Concrete Masonry Walls*
- *TEK 19-7 Characteristics of Concrete Masonry Units with Integral Water Repellent*



Single Wythe Reinforced Wall Section

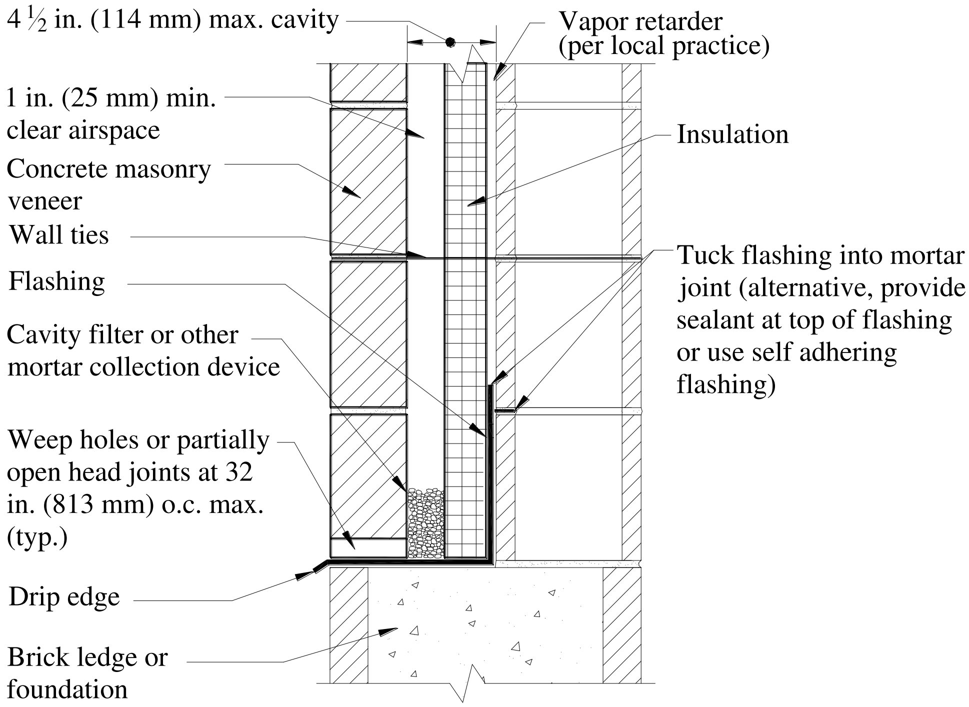


Single Wythe Unreinforced Wall Section



Single Wythe Through-Wall Flashing

Figure 1.3—Typical Flashing Details



Multi-Wythe Wall

Figure 1.3 (cont.)—Typical Flashing Details

Chapter 2

Structural Design Considerations and Detailing

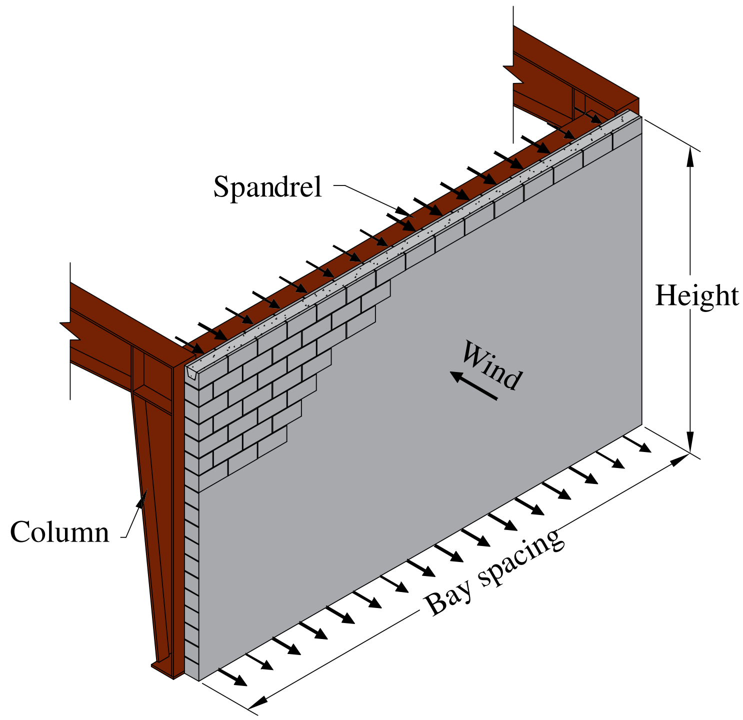
2.1 General

The purpose of this manual is to bridge the gap between the engineer who designs a metal building system and the engineer who designs concrete masonry walls attached to the metal building system. Usually these two design professionals do not ever see, know, or even talk to each other. By observing some design ground rules, a coordinated design can be achieved with these two independent efforts.

This manual is intended to clearly establish spanning directions for the masonry assemblies incorporated into a metal building. Masonry walls resisting out-of-plane loads generally are designed to span in the vertical direction. A vertically spanning wall with its out-of-plane load acting as a one way slab will apply distributed loads to the building floor at the bottom and a horizontal structural member such as a spandrel at the top. Masonry also can act as shear walls to resist in-plane loading transmitting lateral forces from the building to the foundation.

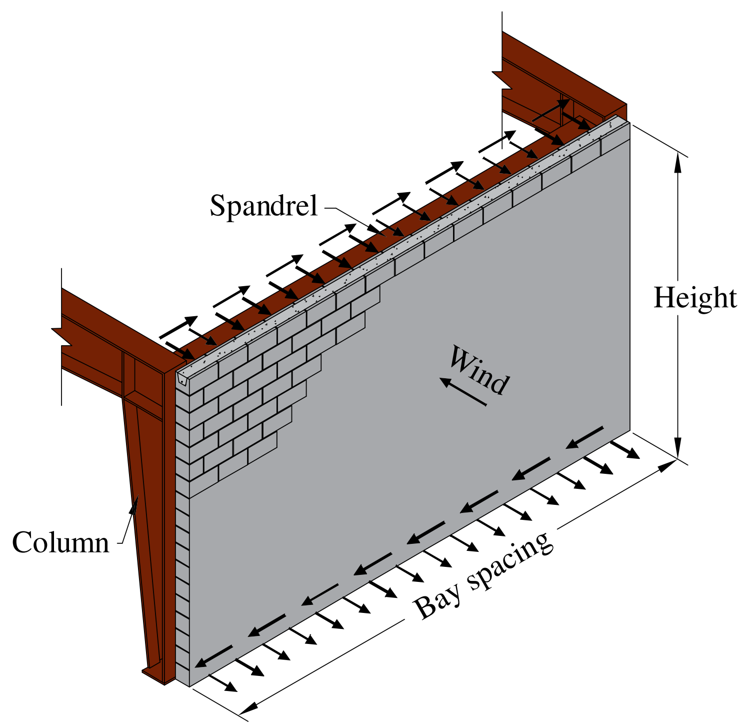
Walls that span horizontally between columns of a metal building are considered nonstandard and require additional treatment. For horizontally spanning walls, the masonry may be designed to distribute forces to the columns by means of horizontal reinforcement in bond beams. In the case of horizontally spanning walls, the supporting columns would need to be designed for the resulting distributed loading. If the masonry engineer decides to use a wall spanning horizontally, which is sometimes considered when the wall height is greater than the column spacing, it should be specified on the bid documents received by the metal building manufacturer in time for bidding.

The cases illustrated in Figures 2.1A through 2.1D define the direction that the masonry spans and its ability to act as a shear wall.



CASE I (Standard)*

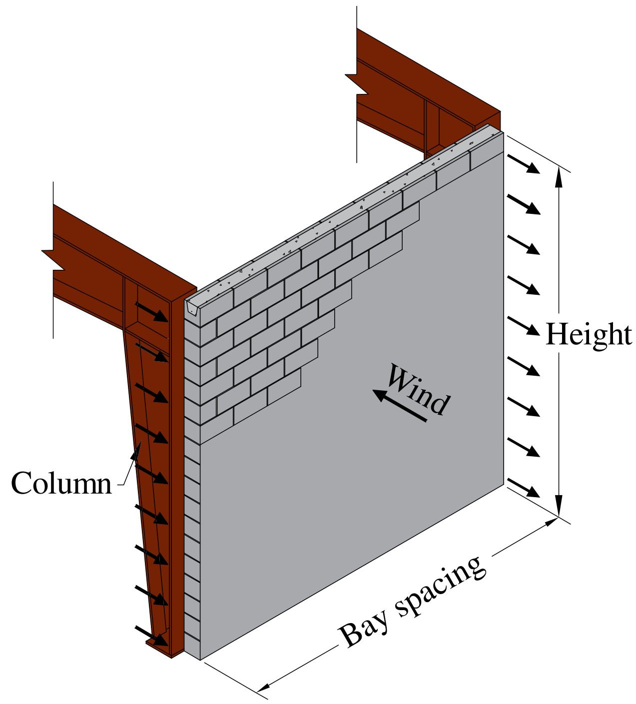
Figure 2.1A—Vertically Spanning Masonry without In-Plane Shear Resistance



CASE IA (Standard)*

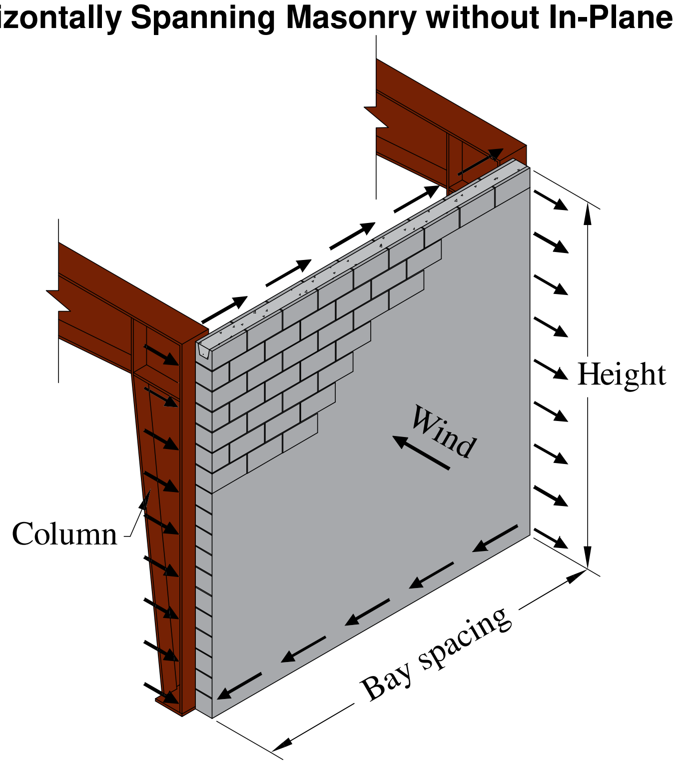
Figure 2.1B—Vertically Spanning Masonry with In-Plane Shear Resistance

*Cases I and IA represent standard design conditions and constitute the default cases. It is essential that both the masonry engineer and the metal building engineer design for this condition in order to have compatible designs for the wall and building.



CASE II (Nonstandard)**

Figure 2.1C—Horizontally Spanning Masonry without In-Plane Shear Resistance



CASE IIA (Nonstandard)**

Figure 2.1D—Horizontally Spanning Masonry with In-Plane Shear Resistance

** Cases II and IIA represent nonstandard design conditions. If the masonry engineer designs for either of these cases, it needs to be specified in the purchase order documents as indicated in Section 2.1.

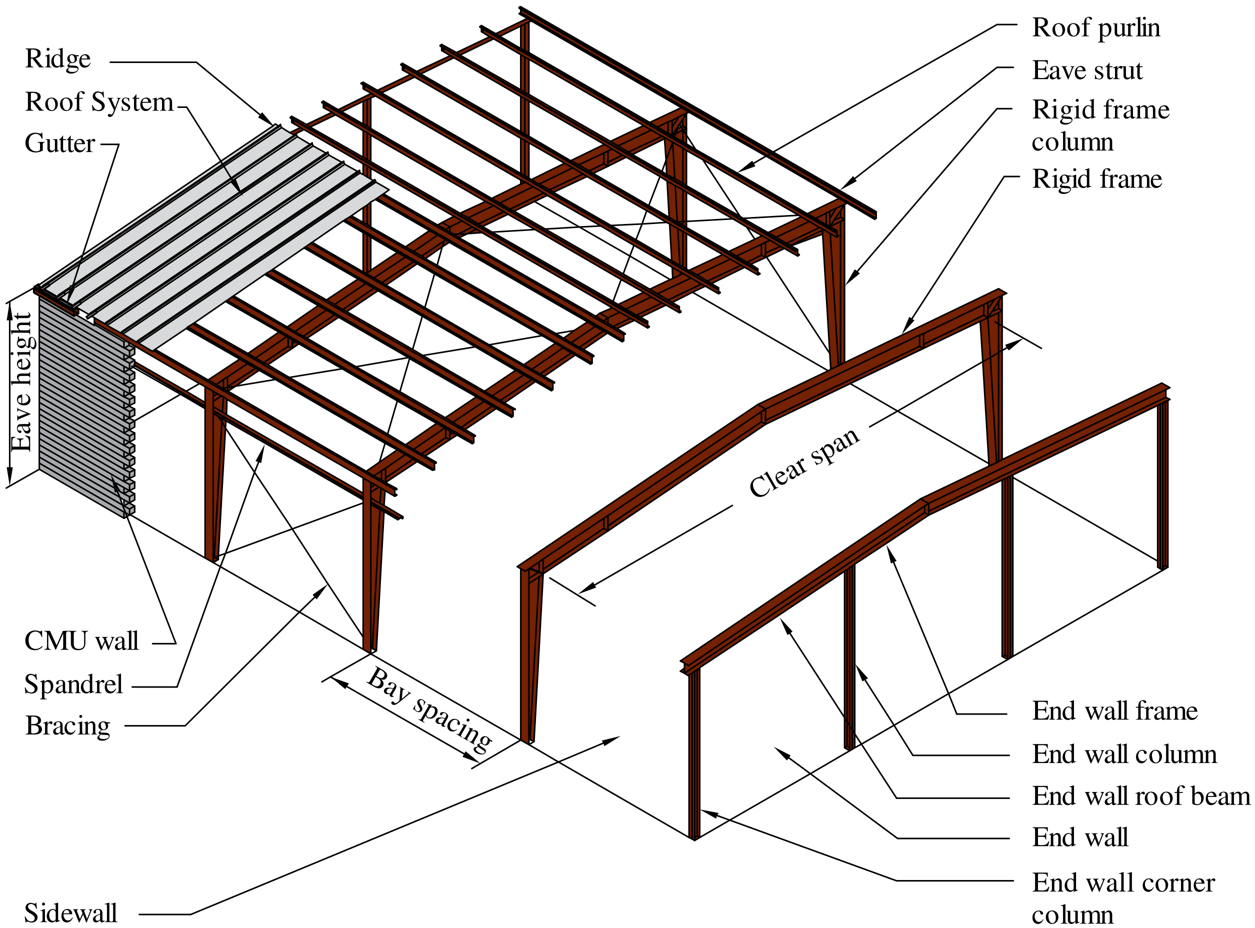
A typical metal building clad with concrete masonry walls is shown in Figure 2.1E. Normally, separate parties design the structural elements of a building - the metal building manufacturer and the masonry engineer. As indicated previously, these two parties do not communicate with each other except through this manual. Therefore the purpose of this manual is to establish guidelines for a standardized design procedure.

The masonry engineer may sometimes serve as the Engineer of Record for the overall project which includes all disciplines of engineering involved in the project. The metal building manufacturer will normally design the structural steel framing members and connections for the rigid frames, end wall frames, roof system, wind bracing (other than masonry walls), metal wall panels (if applicable), flange braces, girts, exterior canopies and fascias. The masonry engineer will design the interior and exterior masonry walls and the anchors that connect the steel frame to the masonry. Other items that are sometimes designed by the metal building manufacturer include overhead crane runways and mezzanine or intermediate floor framing.

Concrete masonry walls typically used for metal buildings include exterior full height walls, either with or without a parapet; exterior partial height walls or wainscots; and interior bearing walls or partitions. Exterior concrete masonry walls are usually designed to transfer their out-of-plane loads to the metal building system through connections at the steel spandrels or columns. In addition, the exterior masonry walls can be designed to act as shear walls to carry in-plane lateral forces. When this is done, the metal building manufacturer will indicate on the metal building drawings the shear reactions that the masonry wall is expected to carry. These loads are transferred from the metal building to the masonry shear wall through the spandrel located at or near the top of the masonry shear wall. The spandrel and connections to the masonry shear wall need to be designed for these in-plane loads in addition to the out-of-plane loads transferred between those two structural elements. If the shear wall reactions are not shown on the metal building drawings, the metal building is designed to carry the lateral forces by means other than masonry shear walls. In either case proper detailing is required to accommodate the different movement characteristics of the two building systems.

Exterior wainscots (partial height walls) are a popular use of concrete masonry with metal buildings. Typically, an eight to ten-foot (2.4 to 3.0-meter) high concrete masonry wall is used starting at grade where durability, security, or aesthetics are of concern. A metal panel wall system designed by the metal building manufacturer is usually used above the masonry wainscot.

It is important to properly integrate a structurally sound steel frame and a structurally sound masonry wall to form a completed building with structural integrity. Figures 2.1F through 2.1U show details of construction that connect the steel and the masonry. Figure 2.1T shows a steel spandrel serving as a window head at the top of a masonry wall. Insulation for the metal building is not shown in these figures for clarity. Consult the MBMA *Energy Design Guide for Metal Building Systems* (ref. 16) for various insulation methods for metal building roofs and walls.



Note: The metal building manufacturer may be required to place holes in the steel structural spandrels, girts, or purlins for purposes of connecting to the concrete masonry walls. It is recommended that a standardized punching of 9/16 in. (14 mm) diameter holes at 17 in. (432 mm) centers for 1/2-in. (13 mm) masonry anchors be utilized. The masonry engineer may choose to place the anchors farther apart than 17 in. (432 mm) centers; however, anchors should not be spaced more than 34 in. (864 mm) as this could affect the lateral stability of the steel member being connected, especially in the case of a spandrel that needs the anchors to prevent torsional flexural buckling. For a hot rolled W-shape or I-shaped built-up spandrel behind a wall as shown in Figure 2.1U, the flange should be punched on two gage lines with 9/16 in. (14 mm) holes for two ½ in. (13 mm) anchors at a 17 in. (432 mm) spacing. Not all holes must be provided with an anchor; however, anchors are not to exceed 34 in. (864 mm) on centers. If this standardized hole size or spacing cannot be accommodated by the masonry engineer, a requirement indicating a change of these dimensions must be inserted into the purchase order documents for the metal building to alert the metal building manufacturer.

Figure 2.1E—Schematic of Metal Building Clad with Concrete Masonry Walls

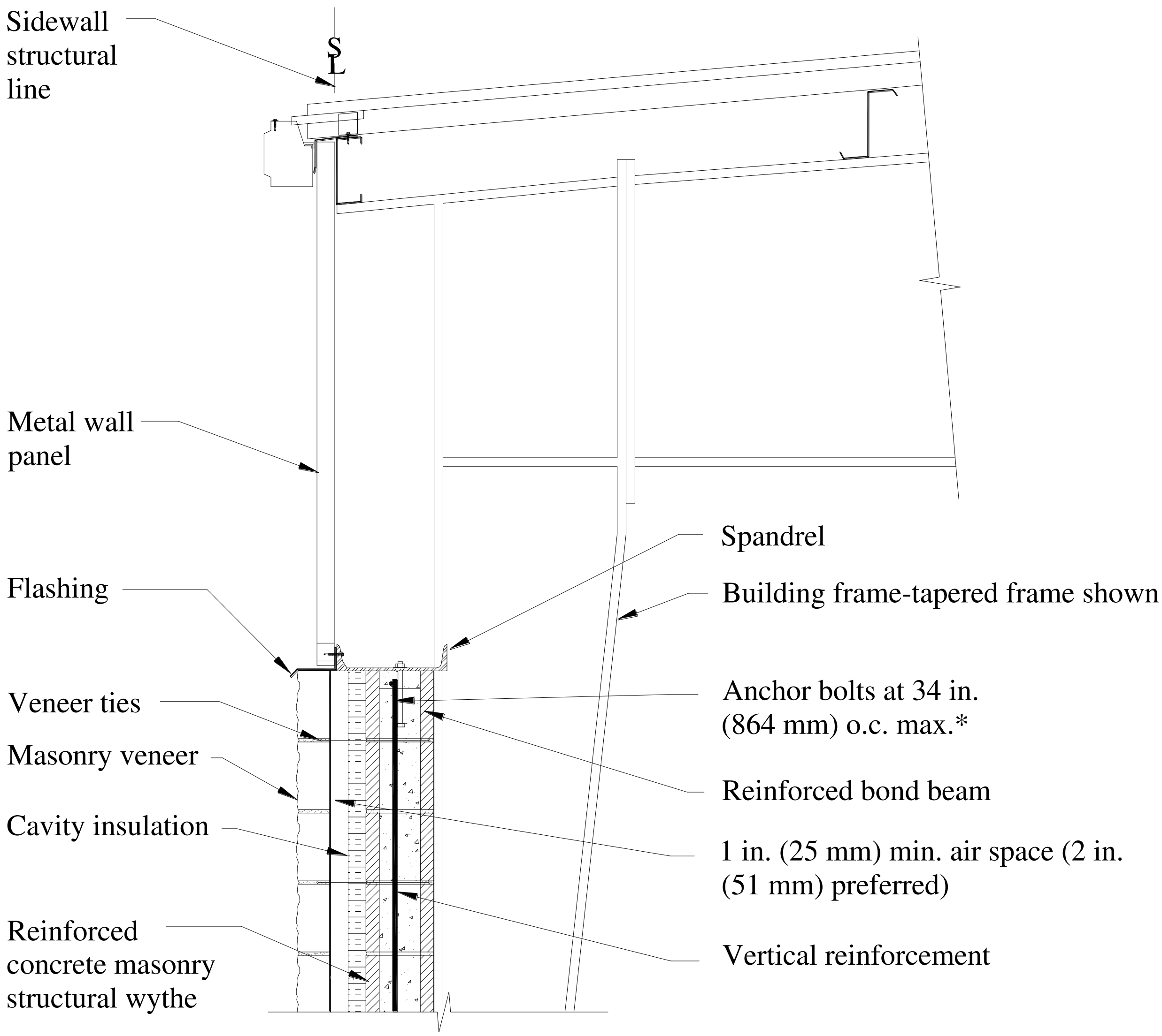
Partial height or wainscot walls are illustrated in Figure 2.1F, 2.1G, 2.1H, 2.1I, 2.1J, and 2.1S. As shown, these walls span vertically between the foundation and the spandrel at or near the top of the wall, which in turn, spans horizontally between metal frame columns. The masonry walls also can be designed as cantilevered assemblies, but significant moments may be induced by large building drifts and large foundations may be required to prevent overturning.

For walls designed to span vertically, a significant structural steel spandrel consisting of a wide flange or built-up section as shown in Figures 2.1N through 2.1S will be required to transfer the out-of-plane load reaction from the wall to the rigid frames. For walls designed to span horizontally, a cold formed channel or zee section can be used to add strength and stiffness to the top of the wall.

Although it is not shown, for horizontally spanning masonry walls, grouting of the cells adjacent at column locations may be needed to accommodate the close spacing of column anchors that may be necessary for shear transfer.

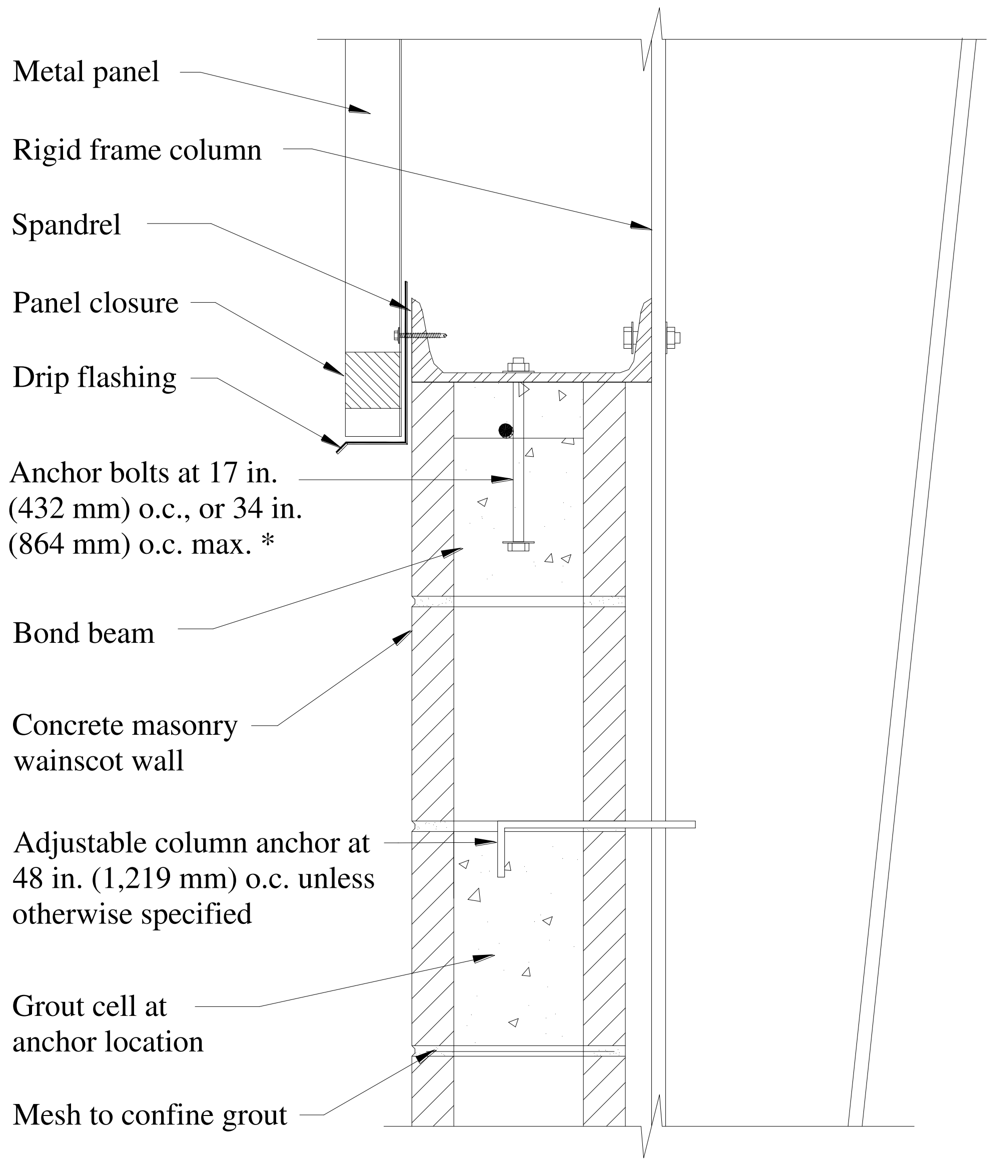
Spandrels should be placed as high as possible to reduce the masonry span above the beam, especially on walls with parapets. Depending on the rigid frame configuration used by the metal building manufacturer, rigid frame connection plates and diagonal stiffeners may restrict the spandrel location. In some cases, it may be possible to place the spandrel at the top flange of the rigid frame to reduce the overall height of the parapet. Placing the spandrel above the rigid frame is difficult and not recommended.

Where it is not possible to place a spandrel high on the wall, additional vertical reinforcement may be required for the design of the cantilevered portion of masonry above the spandrel. Vertical reinforcement added into the cantilever portion should extend below the spandrel elevation by the development length of the bar. The cantilevered height of parapet and wall, measured from spandrel attachment to the top of the wall or parapet, should not exceed the vertical span of the wall below the spandrel divided by 3. For cantilevers meeting these limitations, the resulting loads and stresses induced in the support wall are comparatively small relative to other controlling design loads.



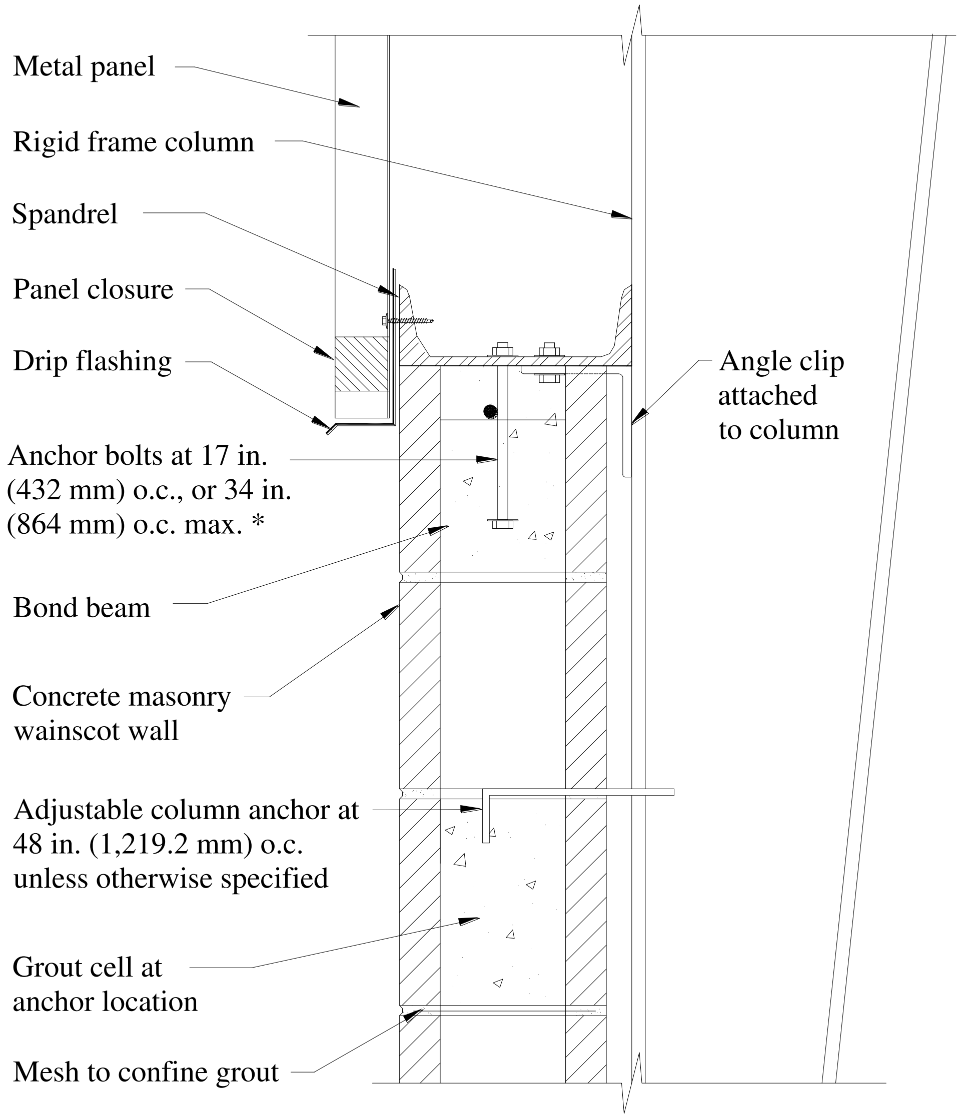
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1F—CMU Multi-Wythe Wainscot at Side Wall



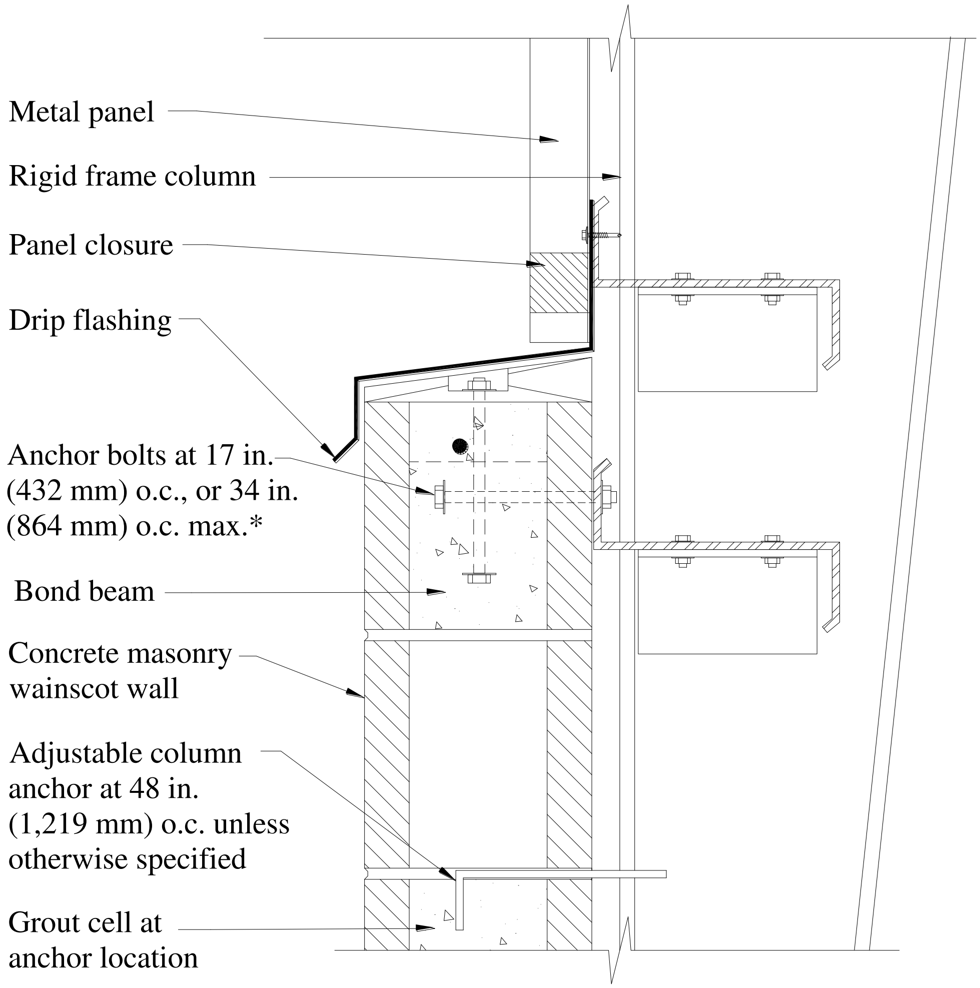
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1G—CMU Single Wythe Wainscot at Side Wall



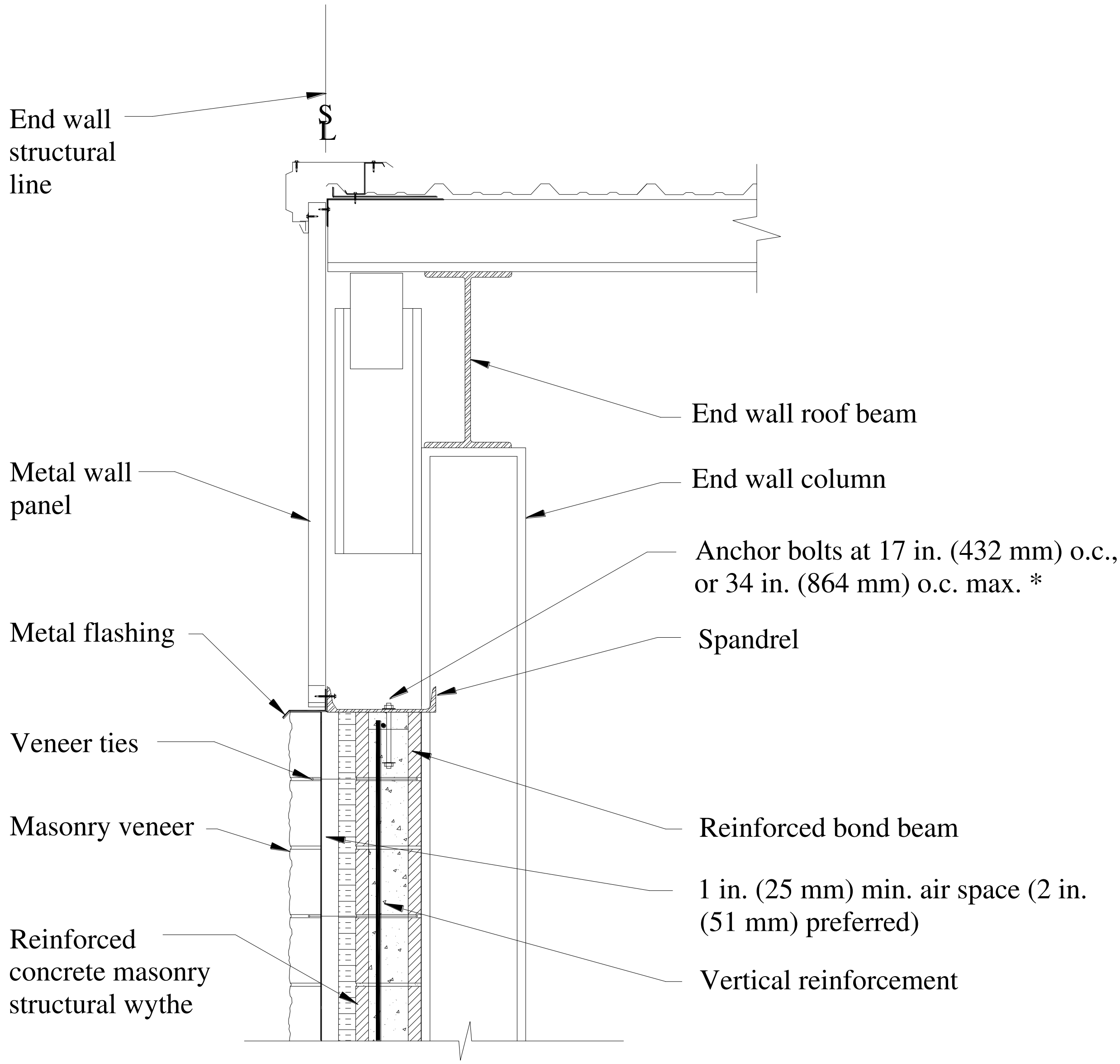
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1H—Alternate CMU Single Wythe Wainscot at Side Wall



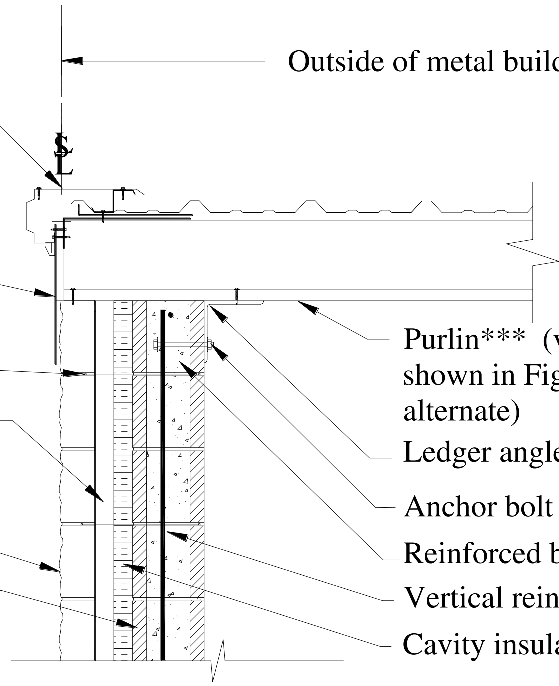
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1I—Alternate CMU Single Wythe Wainscot at Side Wall



* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1J—CMU Multi-Wythe Wainscot at End Wall

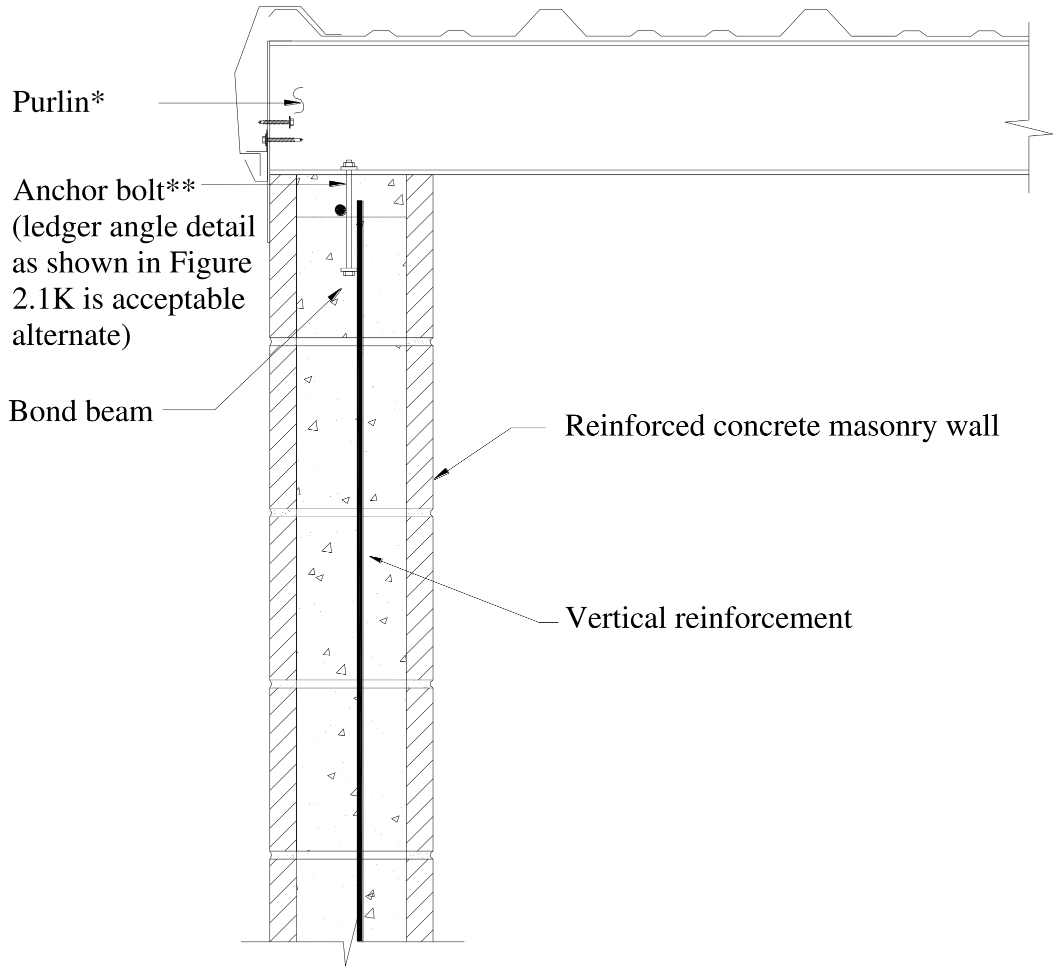


* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

**Note in higher wind or seismic zones, a more substantial member than a steel angle may be required to transfer out-of-plane loads to the building frame.

***Purlins are typically spaced at 5'-0" (1.52 m) o.c. The masonry engineer should design the ledger angle for a 5'-0" (1.52 m) o.c. spacing of the purlins unless notified otherwise.

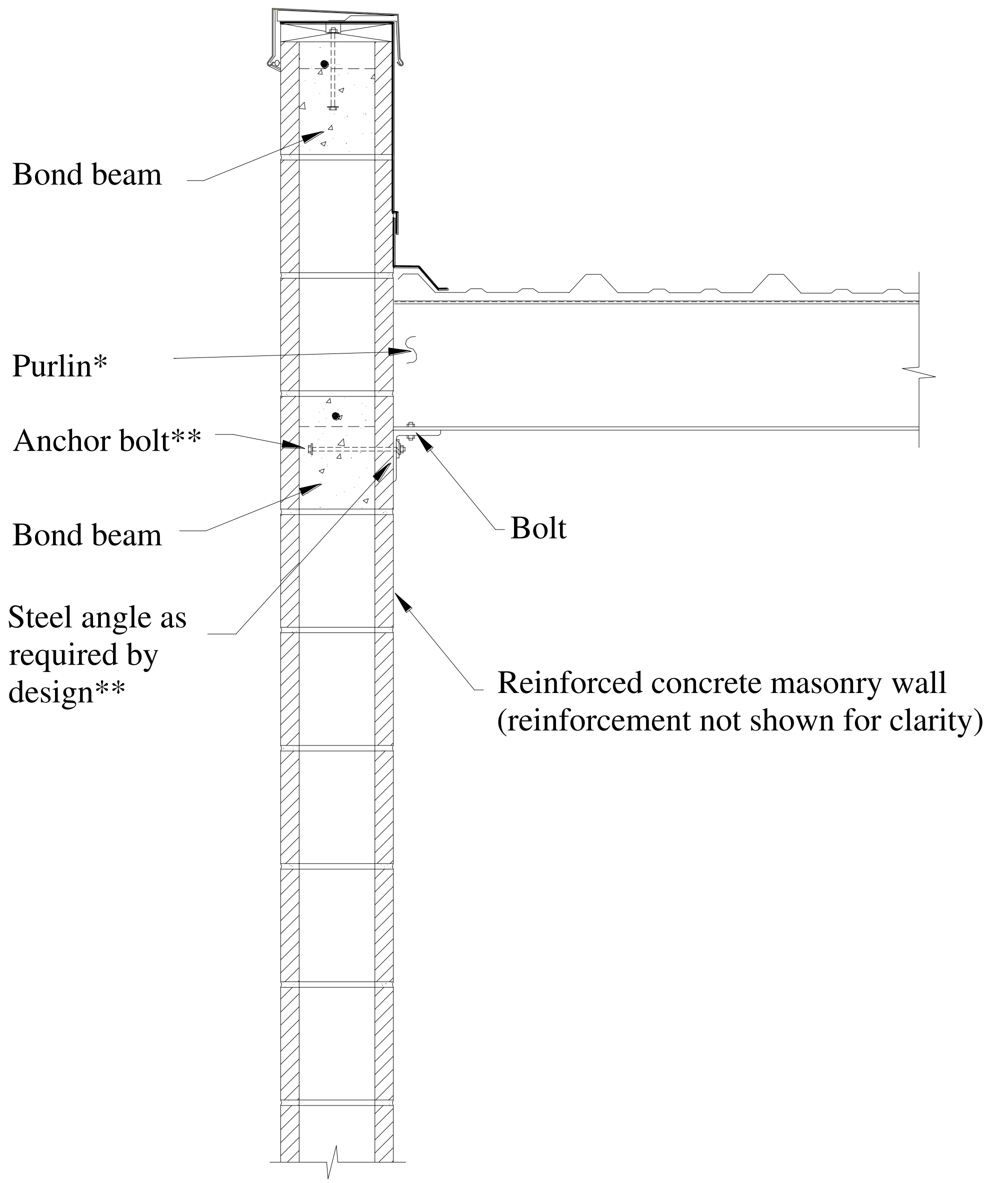
Figure 2.1K—Loadbearing Multi-Wythe Masonry End Wall without Parapet



* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

**Purlins are typically spaced at 5'-0" (1.52 m) o.c. The masonry engineer should design the connection for 5'-0" (1.52 m) o.c. spacing unless notified otherwise.

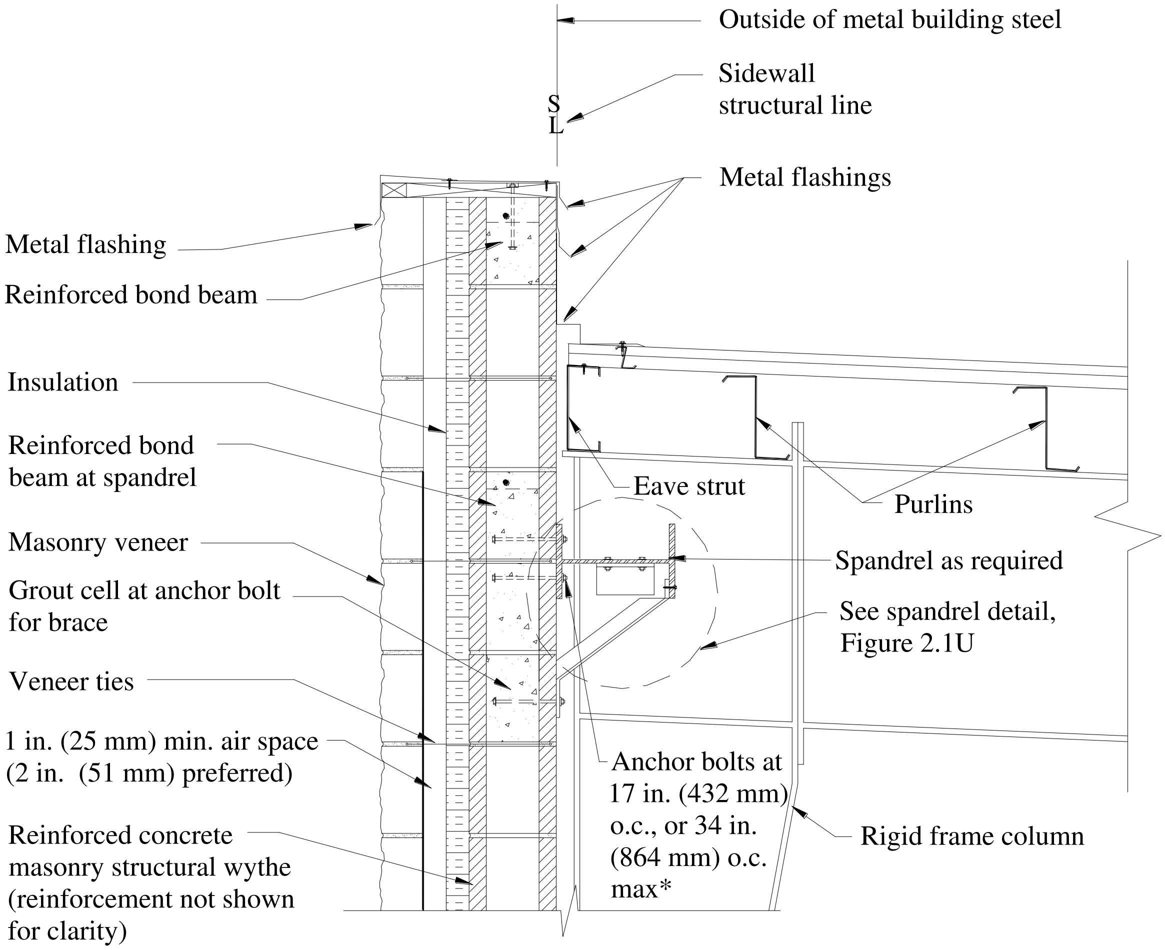
Figure 2.1L—Loadbearing Single Wythe Masonry End Wall without Parapet



* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

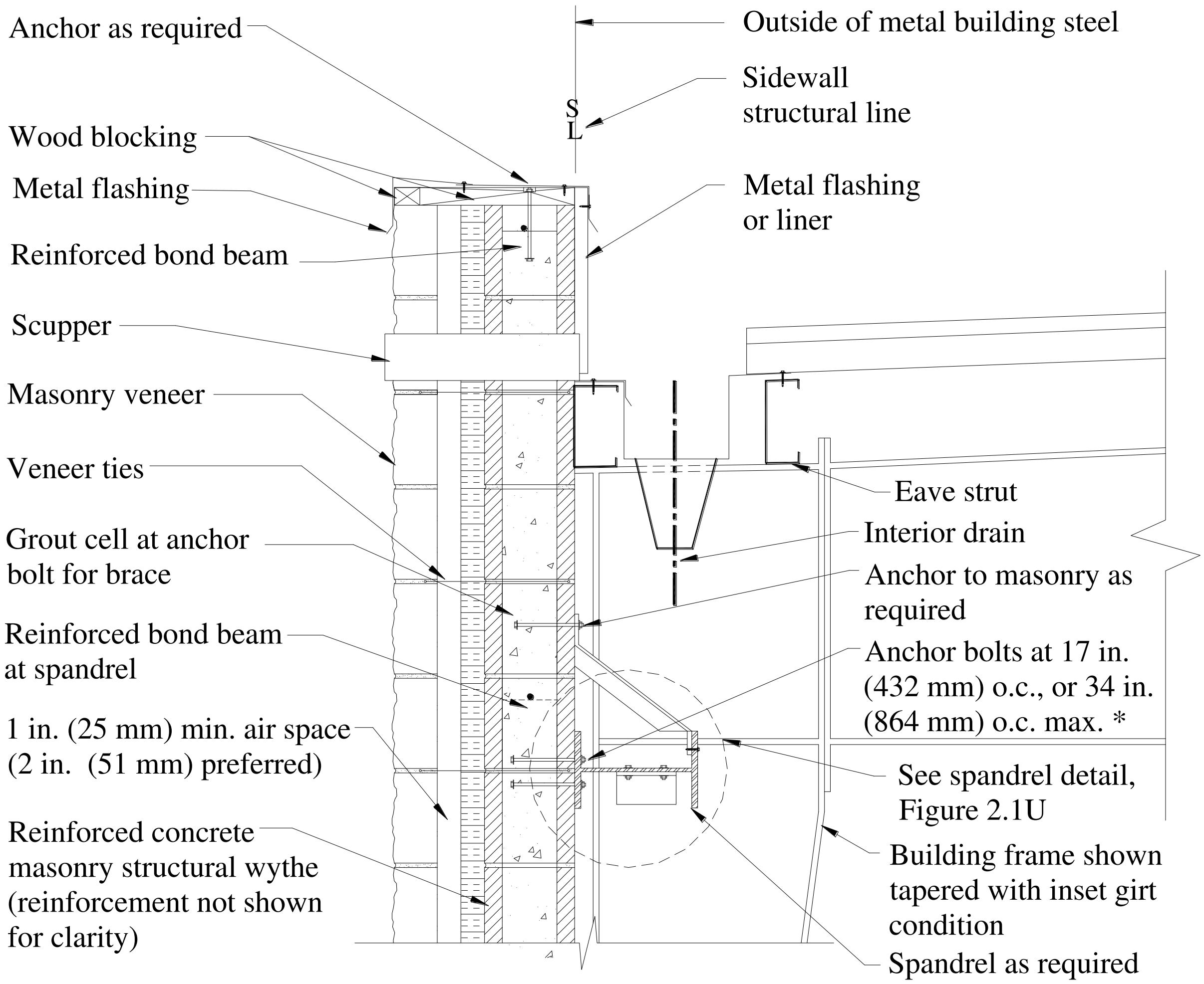
** Purlins are typically spaced at 5’-0” (1.52 m) o.c. The masonry engineer should design the member as spanning 5’-0” (1.52 m) o.c. unless notified otherwise. Note in higher wind or seismic zones, a more substantial member than a steel angle may be required to transfer out-of-plane loads to the building frame.

Figure 2.1M—Loadbearing Single Wythe Masonry End Wall with Parapet



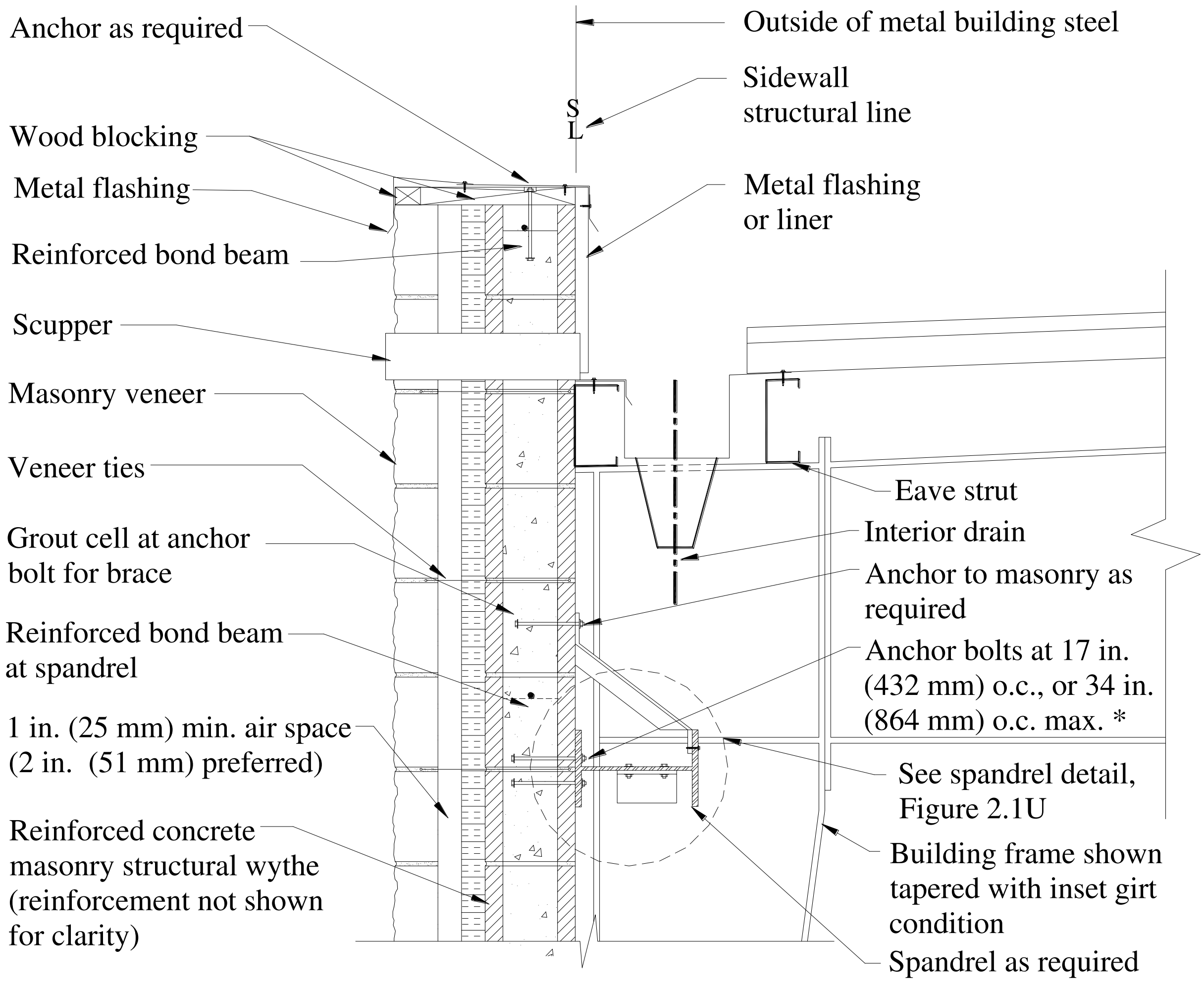
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1N—Multi-Wythe Masonry Wall with Parapet at High Side Wall (Single Wythe Masonry is Similar Except Without Veneer)



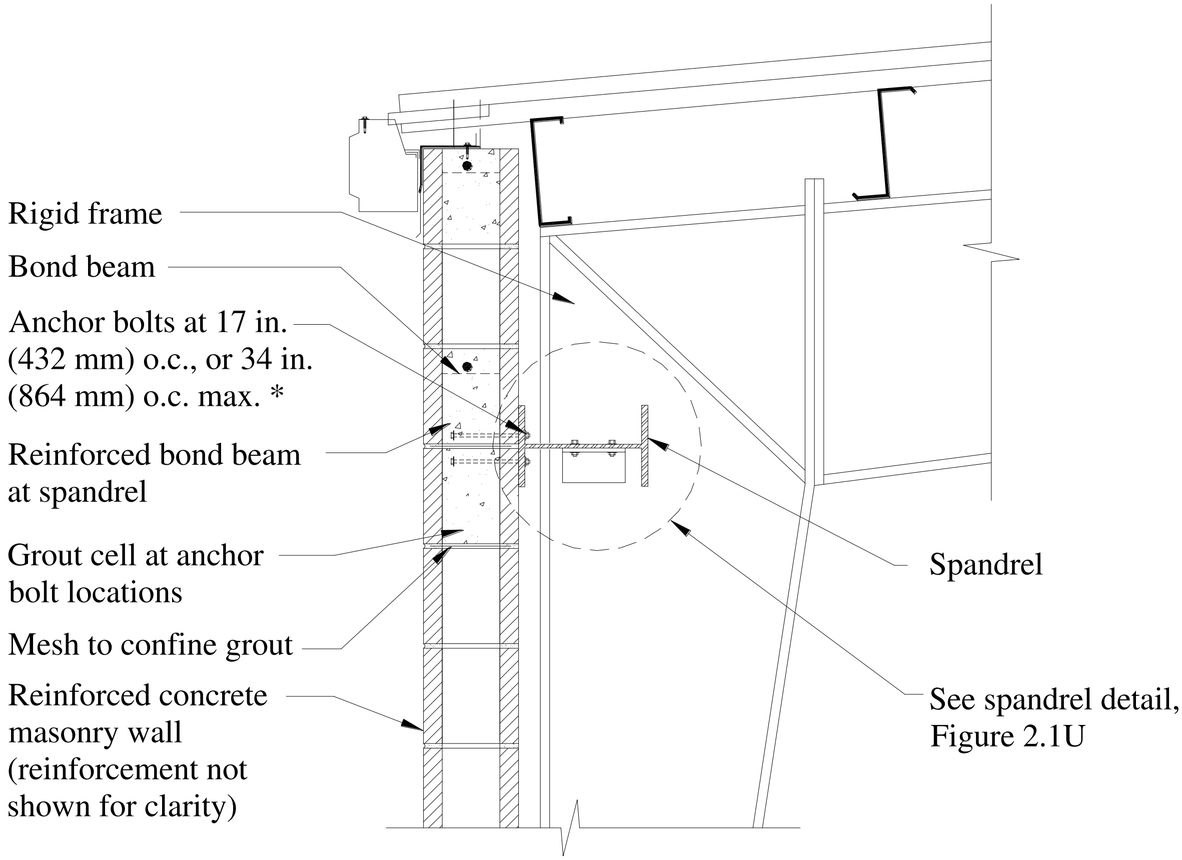
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1O—Multi-Wythe Masonry Wall with Parapet at Low Side Wall (Single Wythe Masonry is Similar Except Without Veneer)



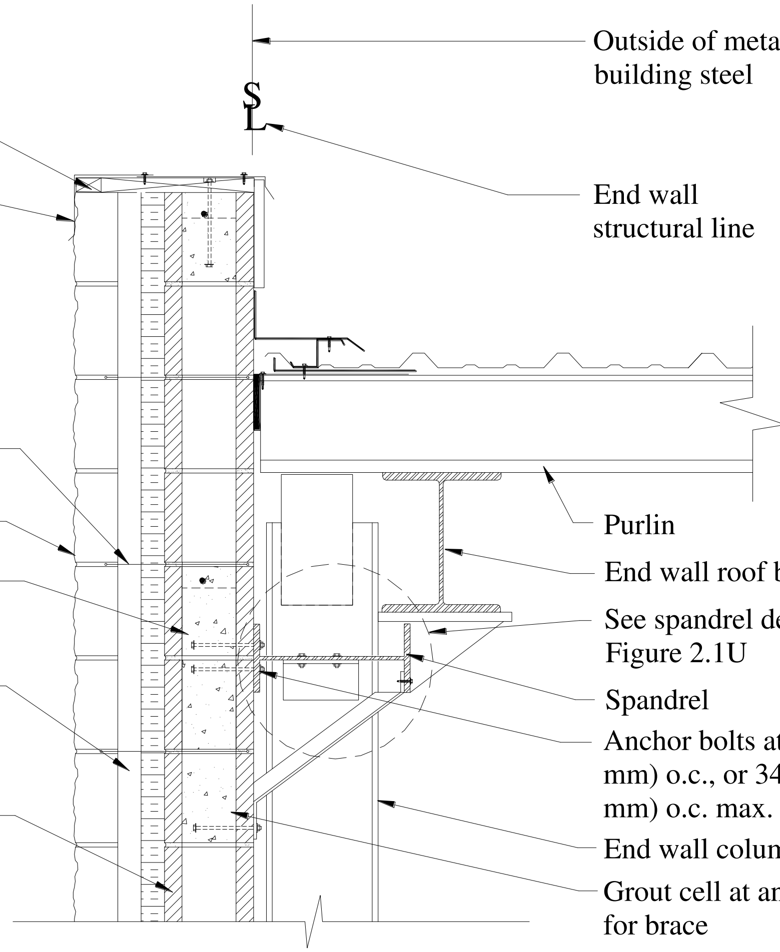
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1P—Single Wythe Masonry Wall without Parapet at High Side Wall (Multi-wythe Similar Except With Veneer)



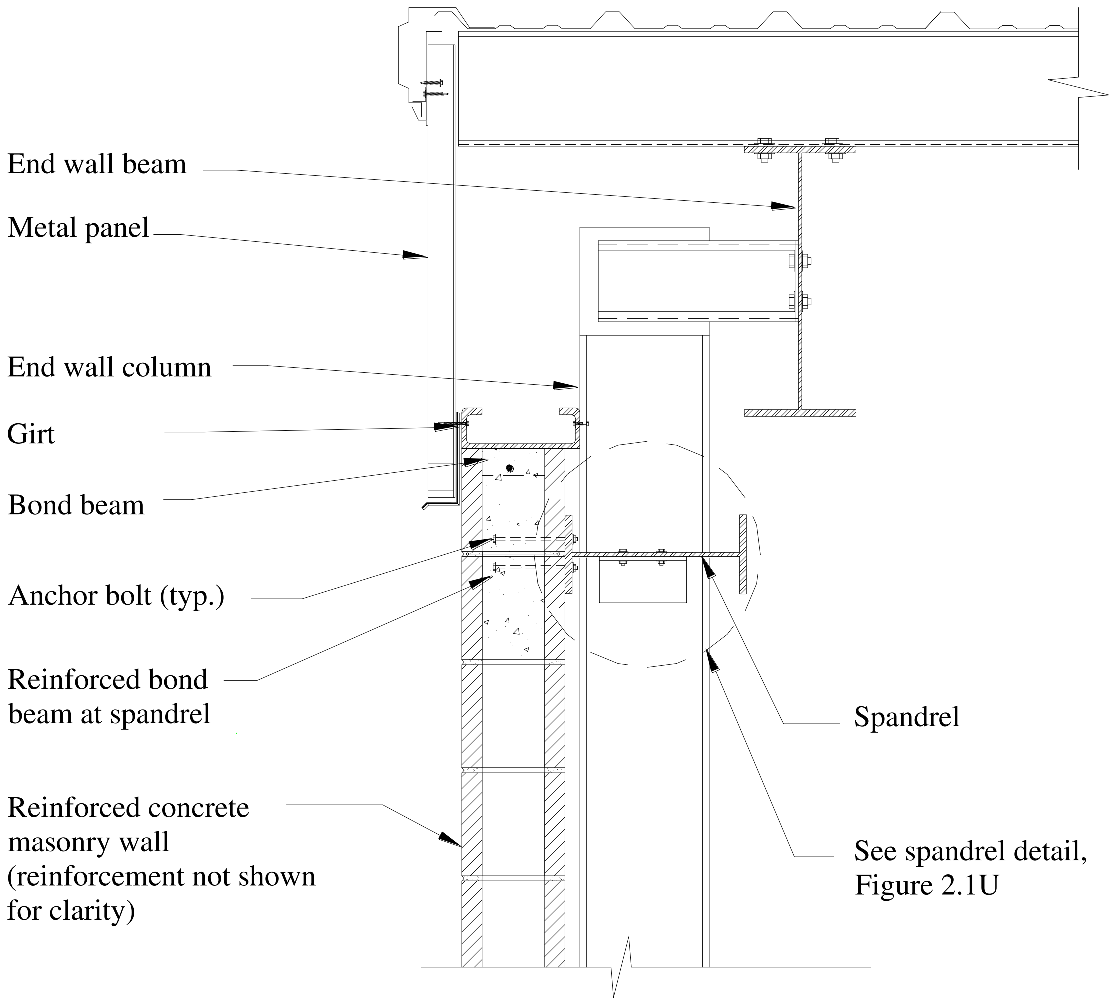
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1Q—Single Wythe Wall without Parapet at Low Side Wall or Eave (Multi-Wythe Similar Except with Veneer)



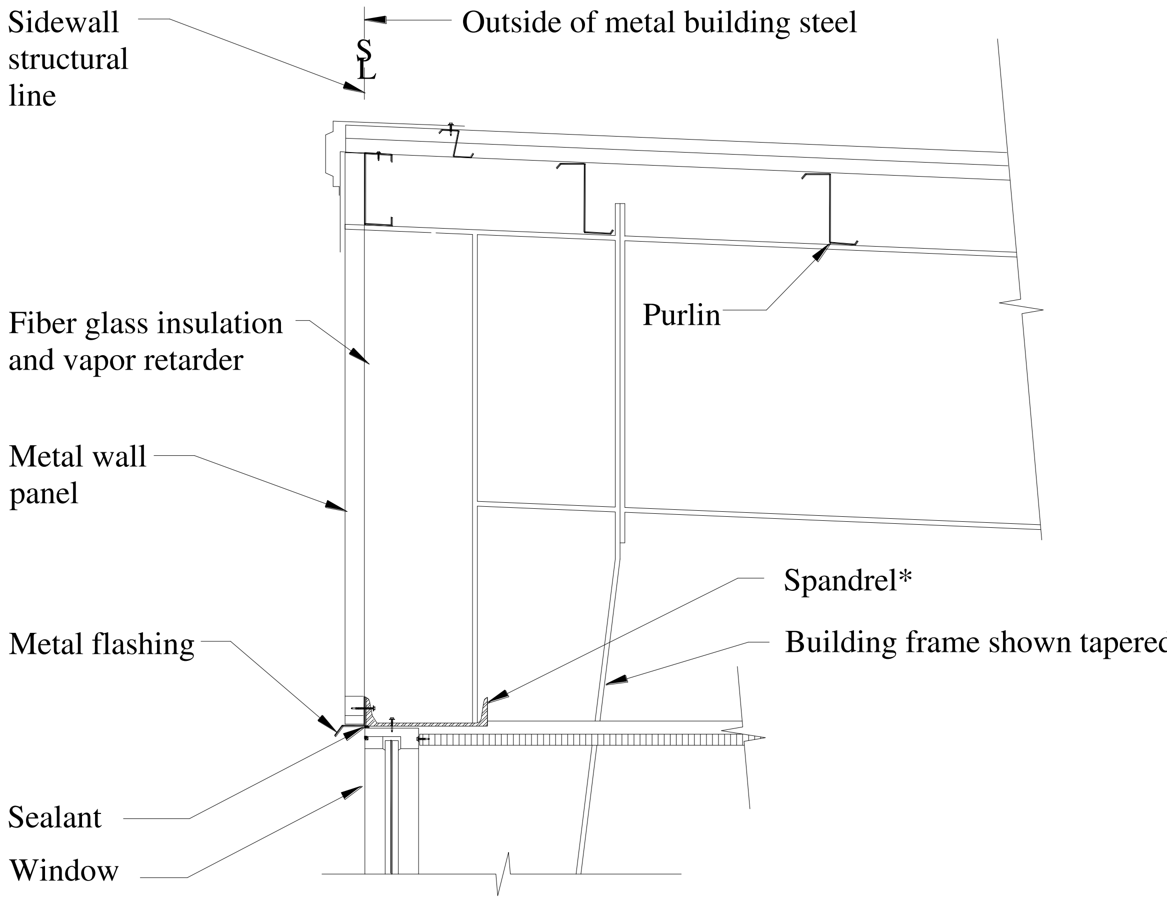
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1R—Multi-Wythe Wall with Parapet at End Wall (Single Wythe Similar Except without Veneer)



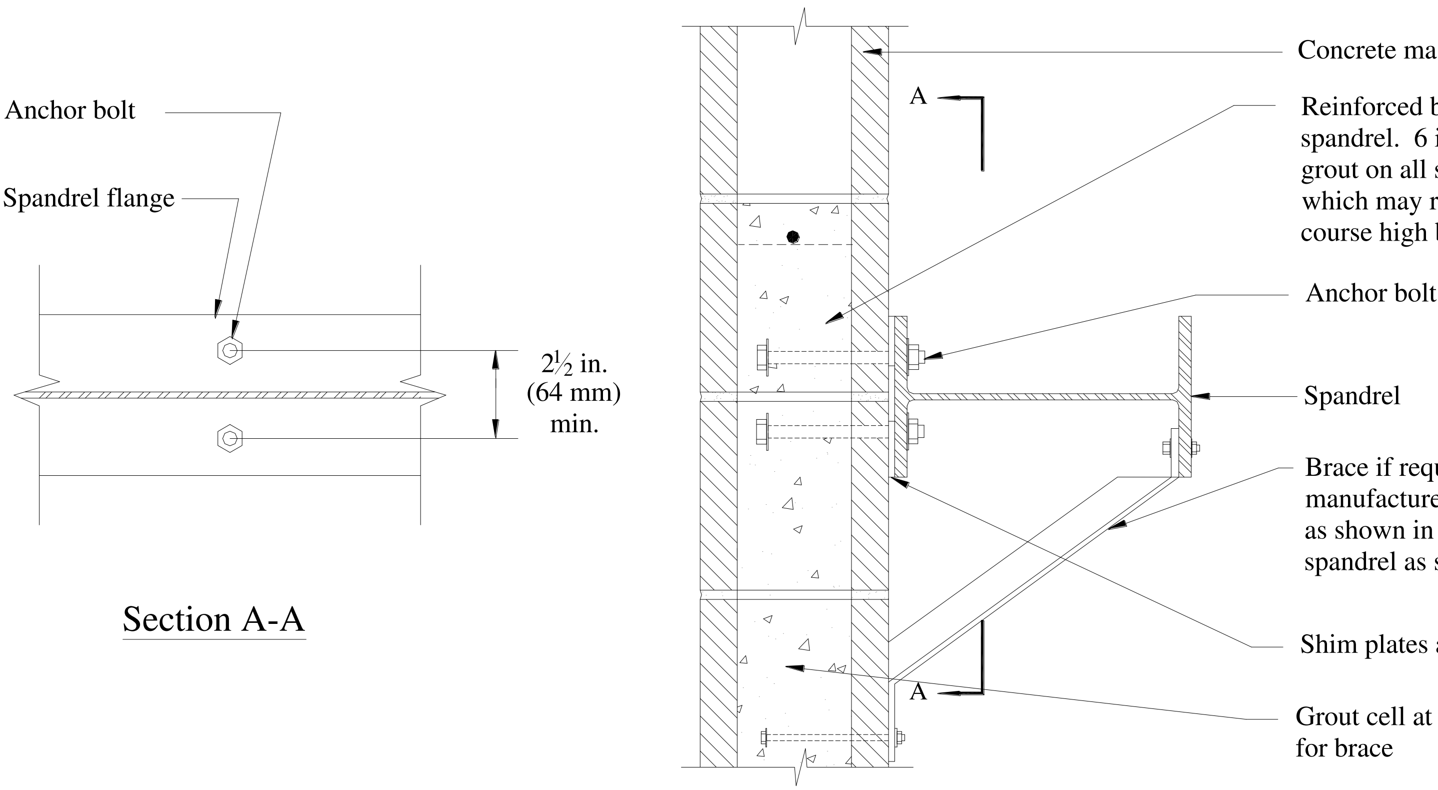
* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to concrete masonry assemblies.

Figure 2.1S—Single Wythe Wall without Parapet at Gable End Wall (Multi-Wythe Similar Except with Veneer)



* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to co
assemblies.

Figure 2.1T—Window Head at the top of a Masonry Wall



* See note to Figure 2.1E for standardized hole size and spacing of structural steel members for connection to co
assemblies.

Figure 2.1U—Structural Spandrel for Lateral Load Detail

2.2 Bracing Flanges of Structural Steel Elements

Flange braces from the masonry to the inner flange of the spandrel may be necessary for the torsional stability of the beam. If required by the metal building manufacturer, the size and spacing of the bracing should be indicated on the metal building drawings similar to Figures 2.1N, 2.1O, 2.1P and 2.1R. If the masonry wall extends above the spandrel, the diagonal brace should be placed above the spandrel as shown in Figures 2.1O and 2.1P for best appearance and to minimize potential interference with a finished ceiling.

Flange braces for the inner flange of exterior columns may also be required by the metal building manufacturer for buckling stability of the column. This may require the approval of the architect or engineer of record before finalizing the drawings. See Figures 2.2A and 2.2B.

Figures 2.2A through 2.2C show details for vertical control joints and the connection of rigid frame columns to concrete masonry side walls. These details show a control joint consisting of sash units and a preformed gasket. Control joints and connection of end wall columns to concrete masonry end walls are similar. Other typical control joint details are available on line in NCMA *TEK 10-2C Control Joints for Concrete Masonry Walls – Empirical Method* (ref. 13).

Figure 2.2A shows the use of adjustable column anchors to connect the wall to the column. For walls designed to span vertically, it is good practice to provide a nominal number of anchors connecting the wall to the column to add stiffness and strength to the edge of the wall. These anchors can usually assist in laterally bracing the outside column flange if they provide adequate rigidity. For walls designed to span horizontally, such anchors may be adequate to transfer the lateral (out-of-plane) load reaction from the wall to the rigid frames. For larger lateral loads, more substantial connections may be required (see Figures 2.2B and 2.2C).

It should be noted that the requirements for the inside column flange braces shown in Figures 2.2A and 2.2B are usually determined by the metal building manufacturer and are typically spaced about 8 ft (2.4 m) on center. The metal building manufacturer needs to clearly show on the drawings where bracing is required, along with the information needed for the masonry engineer to design the braces and their anchorage to the wall. The inside flange braces also provide some resistance to temperature and shrinkage movements that should be considered by the masonry engineer. For some structures it may not be possible to adequately brace the rigid frame column flanges with the masonry wall while allowing required in-plane wall movements, particularly if the masonry walls are not designed to resist in-plane shear loads. If the masonry is designed to resist the in-plane shear, however, the systems will move together and the required support for the column flanges will be effectively provided. The masonry engineer must balance these issues on a case-by-case basis to determine the most economical solutions.

Figure 2.2C shows the use of adjustable anchors to connect the wall to the column without transferring load parallel to the plane of the wall (in-plane). In this case, rigid frame column flanges would not be braced by the concrete masonry wall.

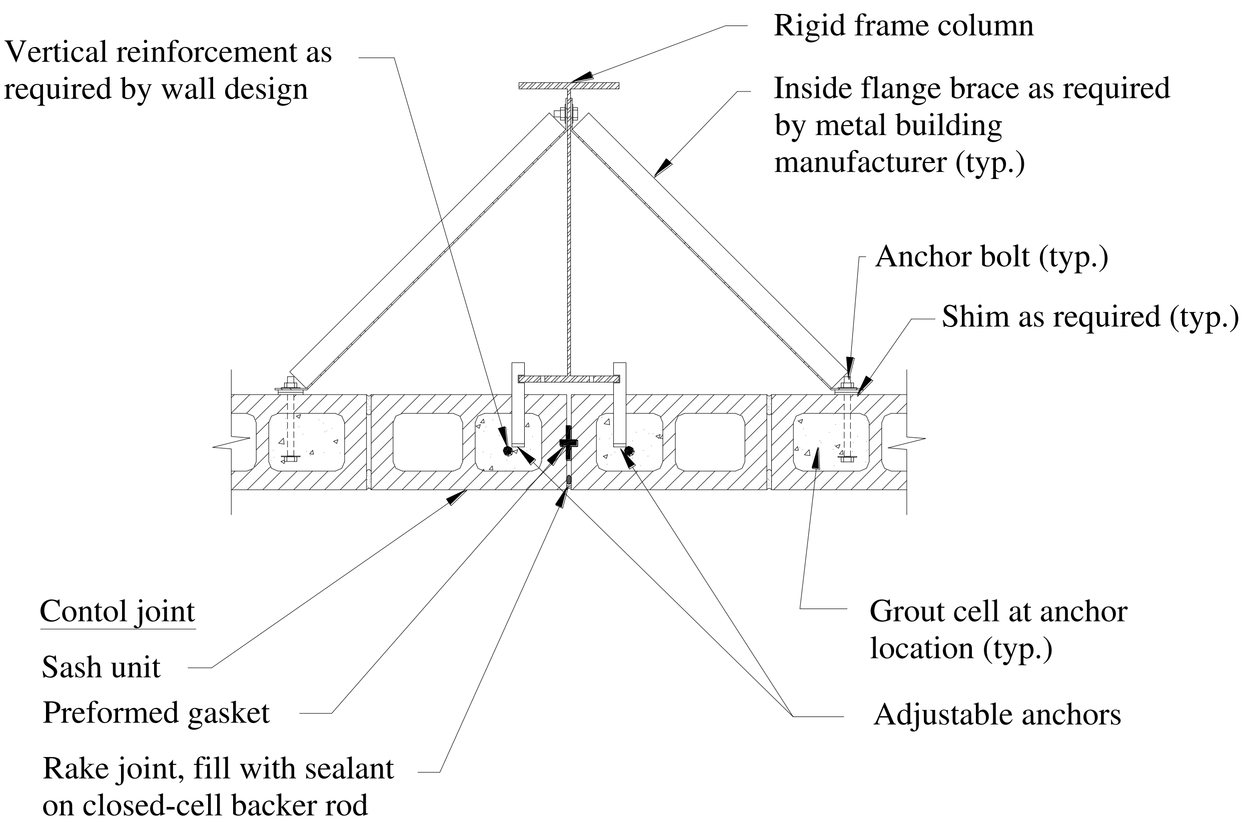


Figure 2.2A—Adjustable Anchor Connection to Rigid Frame Column and Control Joint Detail

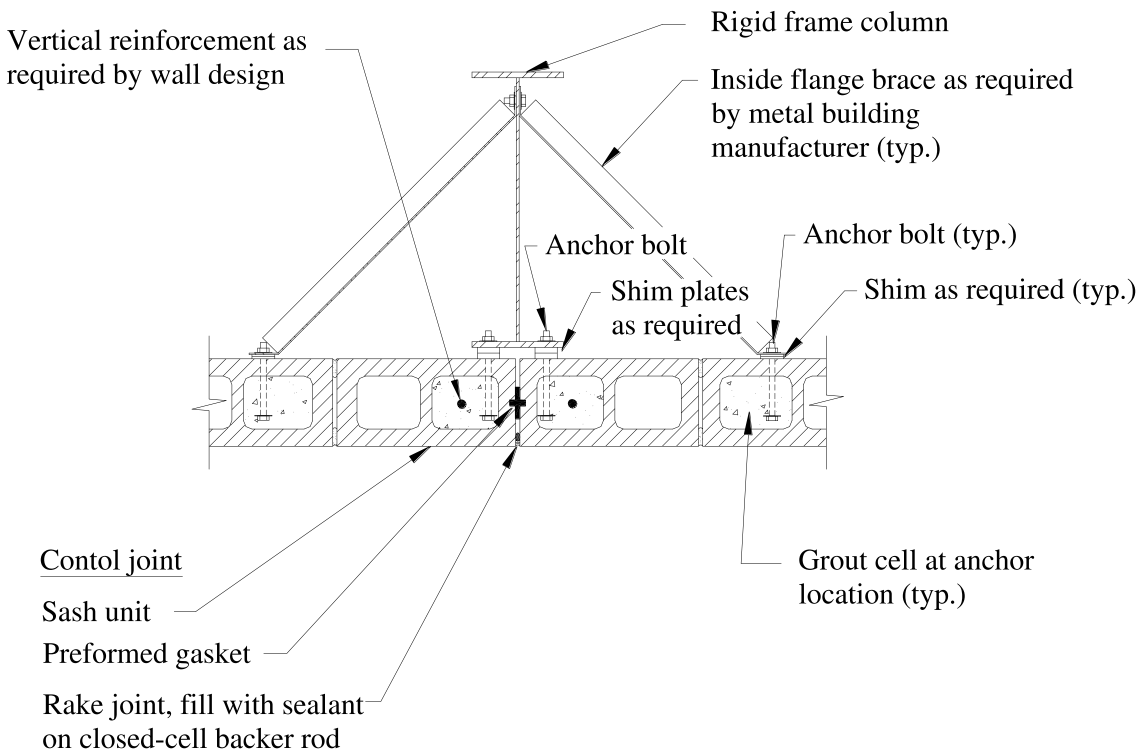


Figure 2.2B—Bolted Connection to Rigid Frame Column and Control Joint Detail

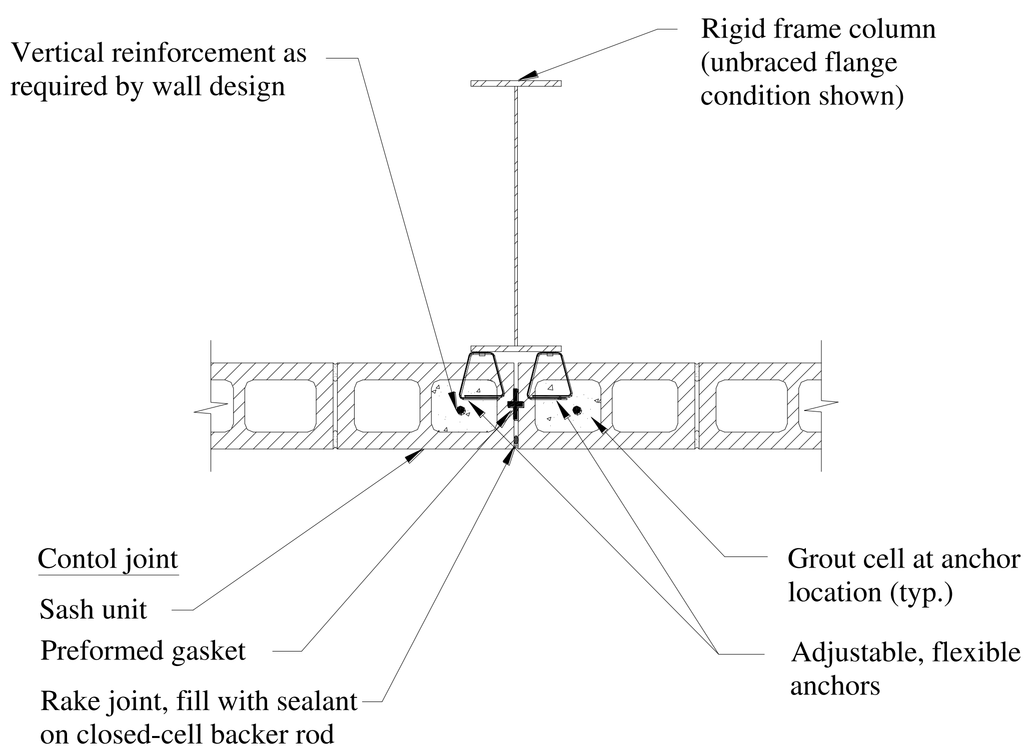


Figure 2.2C—Adjustable Flexible Anchor Connection to Rigid Frame Column and Control Joint Detail

2.3 Movement Design Considerations

2.3.1 General

There are two types of movement design considerations that must be taken into account by the masonry engineer when designing the connections of masonry walls with metal buildings:

1. The normal shrinkage associated with concrete products as well as the difference in thermal movements of the masonry and the metal building components.
2. The much higher deflection limits associated with metal buildings than are normally allowed for masonry buildings.

Connections and details must be designed to accommodate both shrinkage and temperature movements. The use of slotted or flexible connections between the steel and masonry may be employed when support in the direction allowing movement is not required. Control joints are normally placed at column locations as shown in Figures 2.2A through 2.2C. Efficient use of masonry materials would therefore require that the bay spacing of the metal buildings be an even number since even numbers are modular with the 8-in. (203 mm) module of a masonry unit to avoid cutting of masonry units. Control joints do not need to be on the center of the column for vertically spanning masonry, however. If a bay spacing that is not modular with the masonry is selected, the masonry engineer should offset the control joint from the centerline of the column. The offset amount should be at least 8 in. (203 mm) or greater to avoid interference with the anchors at the edge of the columns. When column inside flange braces are used as shown in Figures 2.2A and 2.2B, the control joints should be offset an even greater amount outside the confines of the braces.

More information on control joints types, locations and spacing to accommodate normal shrinkage and thermal movements is available on line in Chapter 10 of the TEK Manual (ref. 13):

- *TEK 10-1 Crack Control in Concrete Masonry Walls*
- *TEK 10-2C Control Joints for Concrete Masonry Walls – Empirical Method*
- *TEK 10-3B Control Joints for Concrete Masonry Walls – Alternative Engineered Method*
- *TEK 10-4 Crack Control for Concrete Brick and Other Concrete Masonry Veneers*

The masonry engineer must also be aware of the practices of the metal building industry in order to maintain the economy of this structural system. Keeping these items in mind, economical connections can be designed using common materials and practices.

When designing the connections between concrete masonry assemblies to structural steel rigid frames or infill framing, the following design assumptions must be known:

1. Were the walls designed assuming that out-of-plane bracing is provided by the structural steel frame?
2. Were the walls designed to span vertically, horizontally or both?

3. Were the walls designed as shear walls to resist lateral loads in the plane of the wall?
4. What is the reinforcing pattern?
5. What is the design lateral drift (or drift limit) of the rigid frame?
6. Were the rigid frame columns designed assuming that one or both of the flanges will be laterally braced?
7. If spandrels are used, were the spandrels designed assuming that one or both of the flanges will be laterally braced?

The connections must be designed considering each of these issues. For concrete masonry, the steel frame most often is used to provide lateral bracing for the concrete masonry walls. Lateral support is typically provided at horizontal spandrels or less commonly at the steel columns. To determine the connection design forces, the span direction of the concrete masonry must be known. (See Section 2.1 for general rules of thumb regarding the direction of lateral support.) If the concrete masonry wall was not designed or detailed to act as a shear wall, care must be taken during the connection design and detailing to avoid the transfer of forces parallel to the plane of the wall. In this manual, the method chosen for isolating these nonparticipating walls from the lateral load resisting system is to detail these walls to be discontinuous at the foundation. Other detailing methods that may be employed include the use of oversized or slotted holes in the steel or the use of flexible anchors. Note that the latter method is difficult to practically accomplish as the amount of movement with metal building frames can be substantial as discussed in Section 2.3.3.

2.3.2 Lateral Drift

In *Serviceability Design Considerations for Low-Rise Buildings* (ref. 5), a lateral drift limit of H/100 for a ten year recurrence wind loading based on main wind force resisting system loads is suggested for low rise buildings with exterior masonry walls reinforced vertically. See Table 12.12.1 of ASCE 7 (ref. 4) for the allowable story drift for seismic loading. To control the location of potential cracks that may result from these large deflections at the top of the wall due to the relative lateral flexibility of metal building systems a “hinge” can be incorporated at the base of the masonry assembly to allow out-of-plane rotation. This isolation joint allowing this type of movement often can be created by the use of through-the-wall flashing at the floor line.

2.3.3 Detailing at the Foundation

The construction details presented in this section provide examples of connections to accommodate movement and deformation between concrete masonry and metal buildings. The examples shown in Figures 2.3A through 2.3E are detailed to span vertically. The masonry engineer should modify these details to span horizontally, if needed. Other details may provide more appropriate connections for certain wall and building configurations. Project details should be developed to conform to local construction practices and metal building manufacturers’ recommendations.

Figures 2.3A, 2.3B, 2.3C and 2.3D show details for concrete masonry foundations that can be applied to either side walls or end walls. Figures 2.3A and 2.3C are to be used for masonry walls that are designed to carry in-plane shear loads. Figures 3.2B and 2.3D conversely are applicable to masonry wall segments that are not designed to resist in-plane loads. A common feature of these details is the use of flashing as a bond breaking "hinge" to control the location of horizontal cracks that will likely occur due to the anticipated drift of the rigid frame systems. For walls that span vertically, this "hinge" is intended to allow the wall to rotate at the same level that the steel frame rotates.

It is recommended that the number of reinforcing bars extending through these base isolation joints be minimized to ensure that the joints will act as assumed. This may only require alternate or fewer foundation dowels be extended through the joints; however, continuous reinforcement (wall vertical reinforcement spliced with foundation dowels) must be provided in shear wall segments to resist overturning forces. In the masonry segments that are not designed to resist in-plane loads, detailing this intersection by extending the foundation dowels only a small distance into the masonry above the flashing and debonding that portion of the dowels can be used to provide additional in-plane and out-of-plane sliding resistance while allowing rotation. See Figures 2.3B and 2.3D.

Weeps should be installed at portions of the wall that provide a drainage path to allow water to escape from the wall. See TEK 19-2A and TEK 19-5A (ref. 13) for weep requirements.

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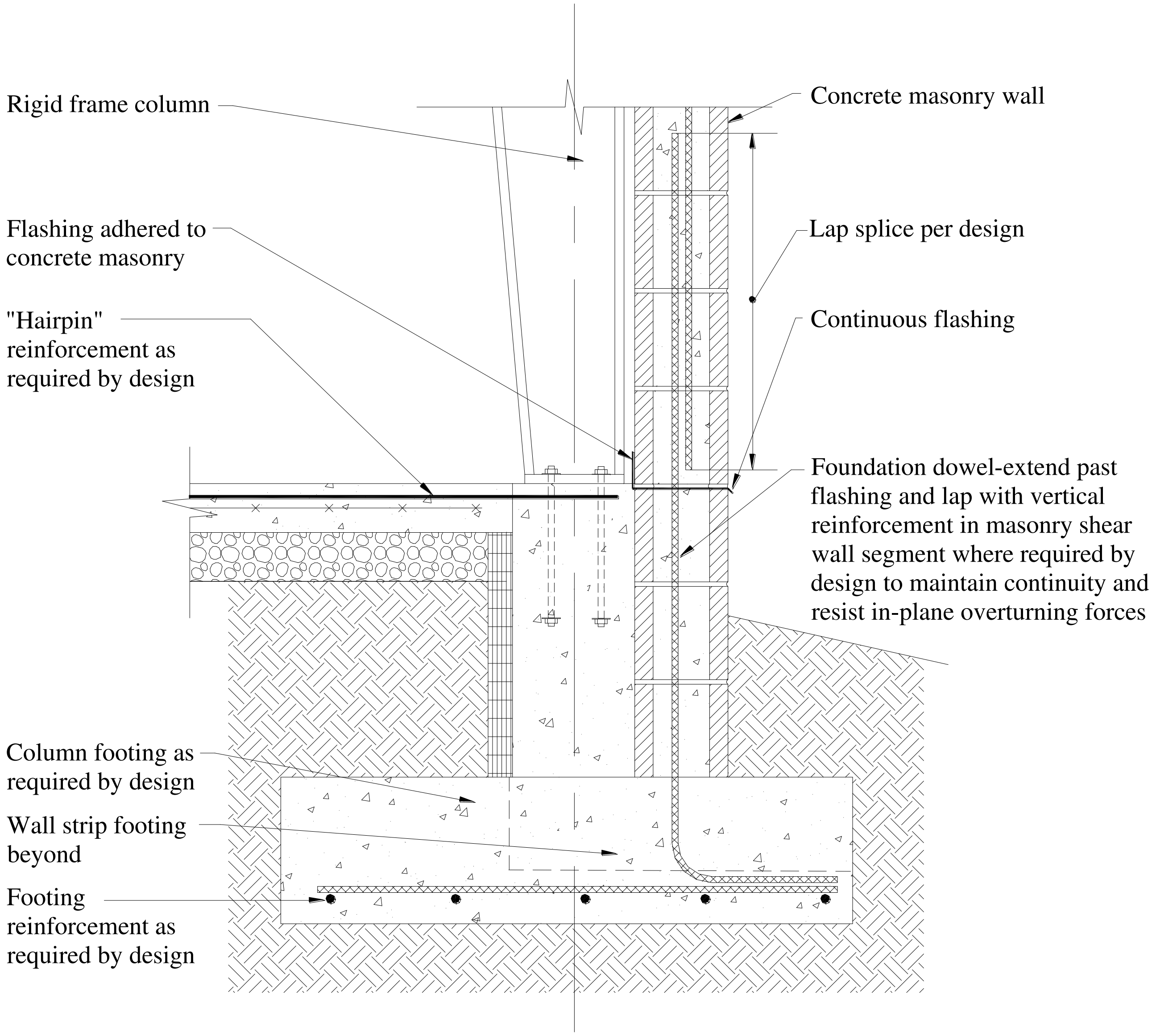

Figure 2.3A—Vertically Spanning Reinforced Concrete Masonry Side Wall Shear Wall Segment Detail at Foundation

Note: Detailing for concrete masonry end walls is similar

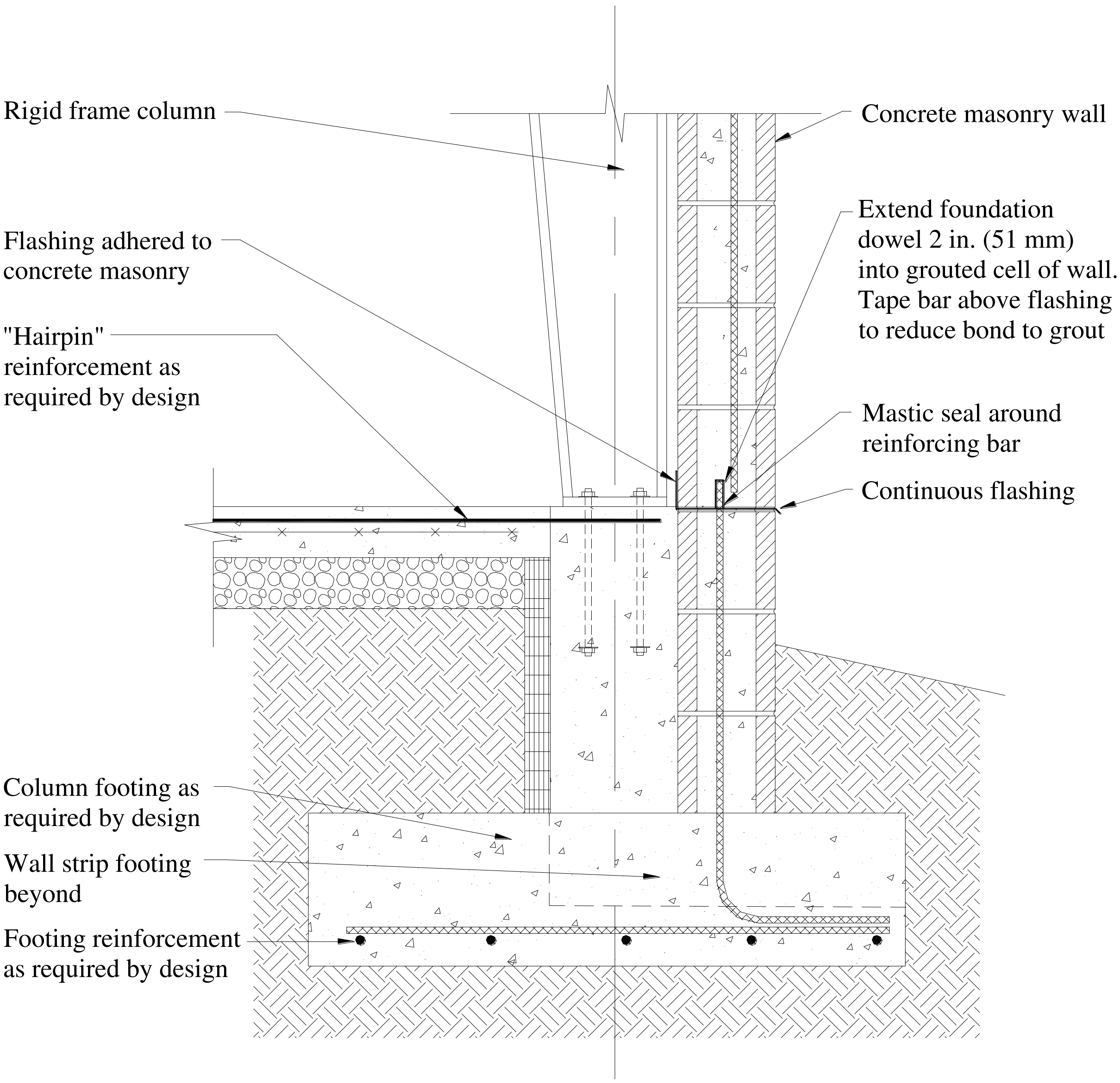

Figure 2.3B—Vertically Spanning Reinforced Concrete Masonry Side Wall at Foundation for Other than Shear Wall Segment

Note: Detailing for concrete masonry end walls is similar

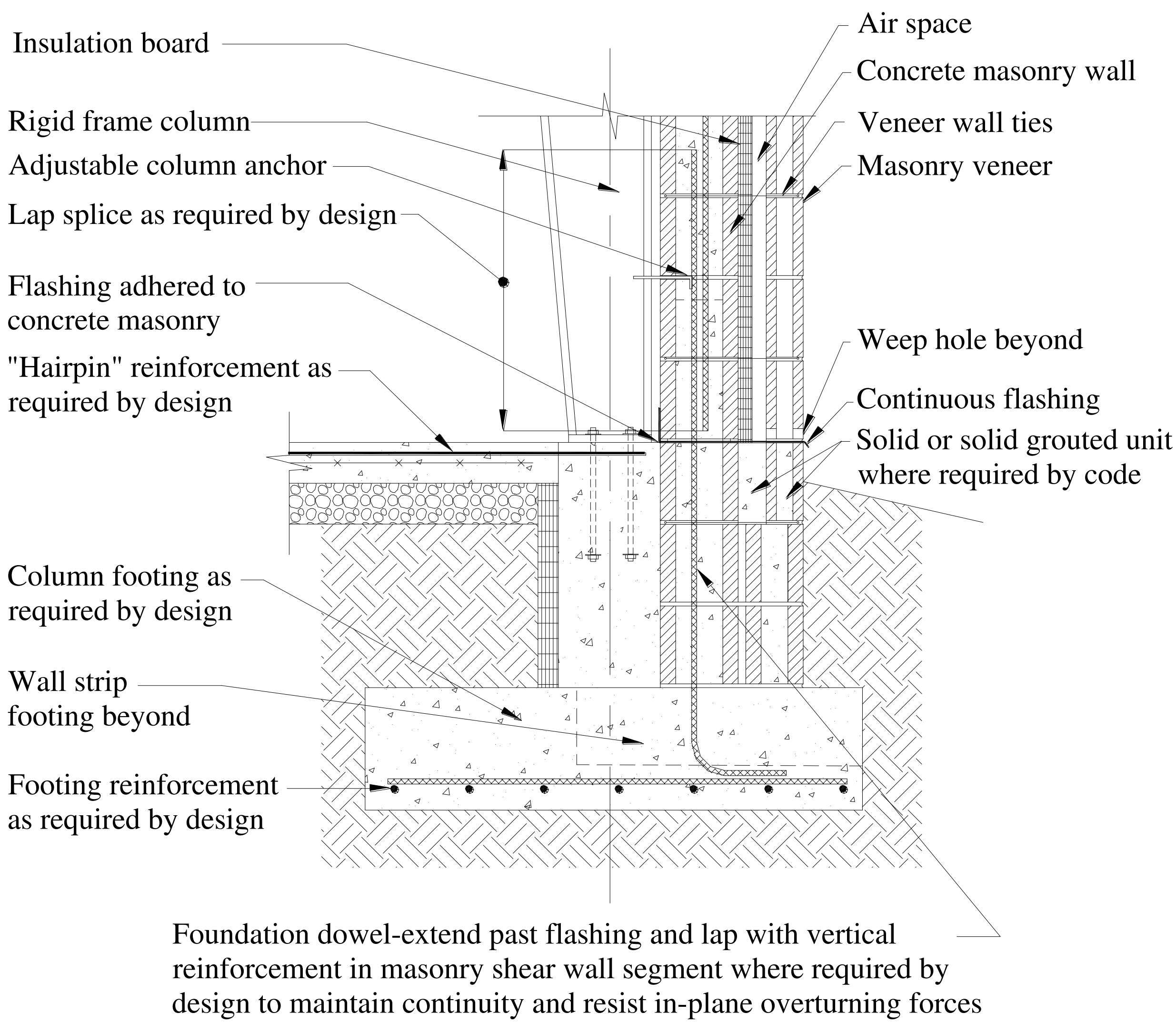

Figure 2.3C—Vertically Spanning Reinforced Concrete Masonry Cavity Side Wall Shear Wall Segment Detail at Foundation

Note: Detailing for concrete masonry end walls is similar

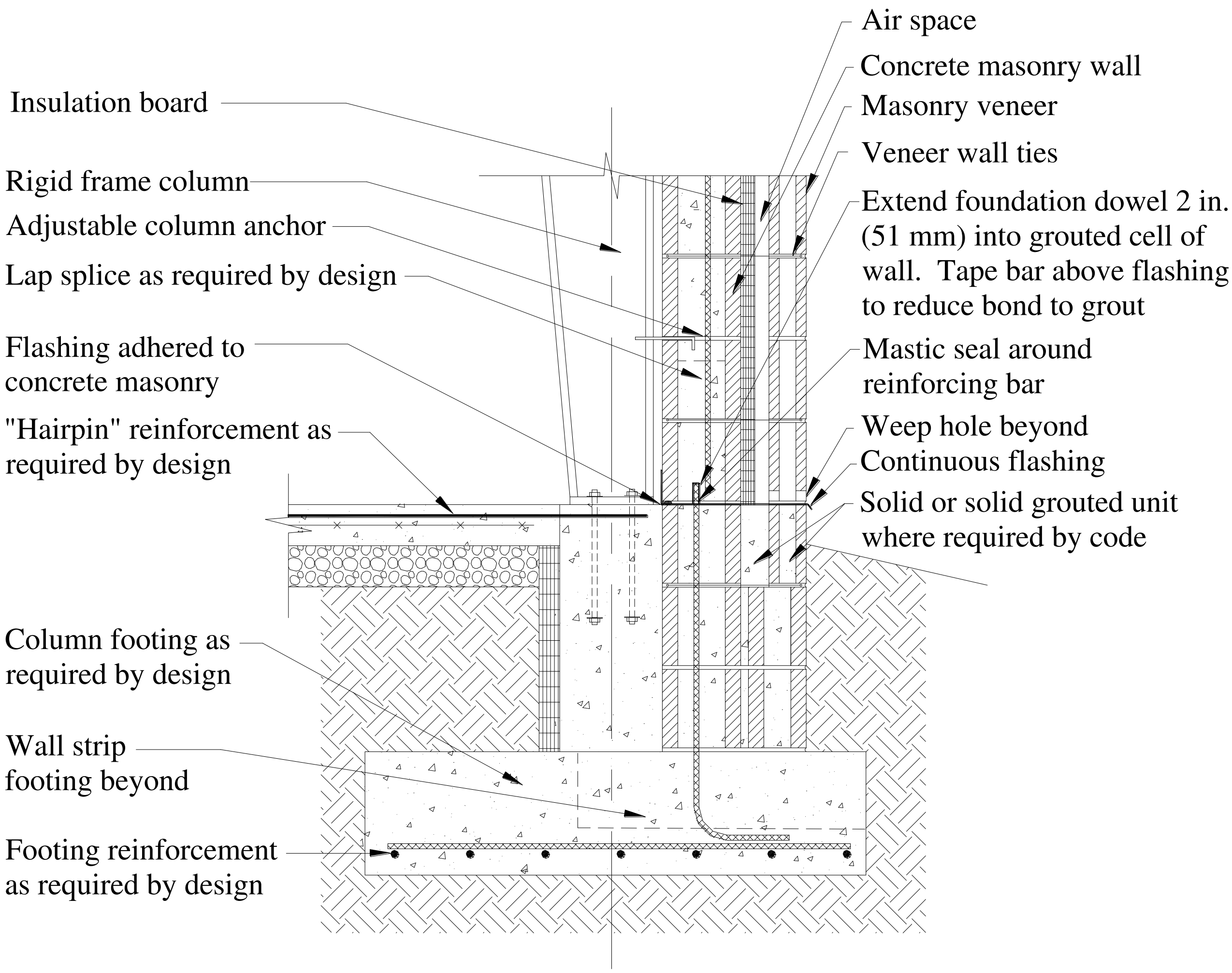

Figure 2.3D—Vertically Spanning Reinforced Concrete Masonry Cavity Side Wall Detail at Foundation for Other than Shear Wall Segment

Note: Detailing for concrete masonry end walls is similar

Figure 2.3E illustrates an expansion/isolation joint that is used to isolate intersecting masonry walls so that the metal building can drift as intended. Control joints are also provided at the column locations. Additional control joints may be required at changes in wall height or thicknesses, at return angles in "L," "T," and "U" shaped structures, near one or both sides of wall openings, and at intermediate locations between columns.

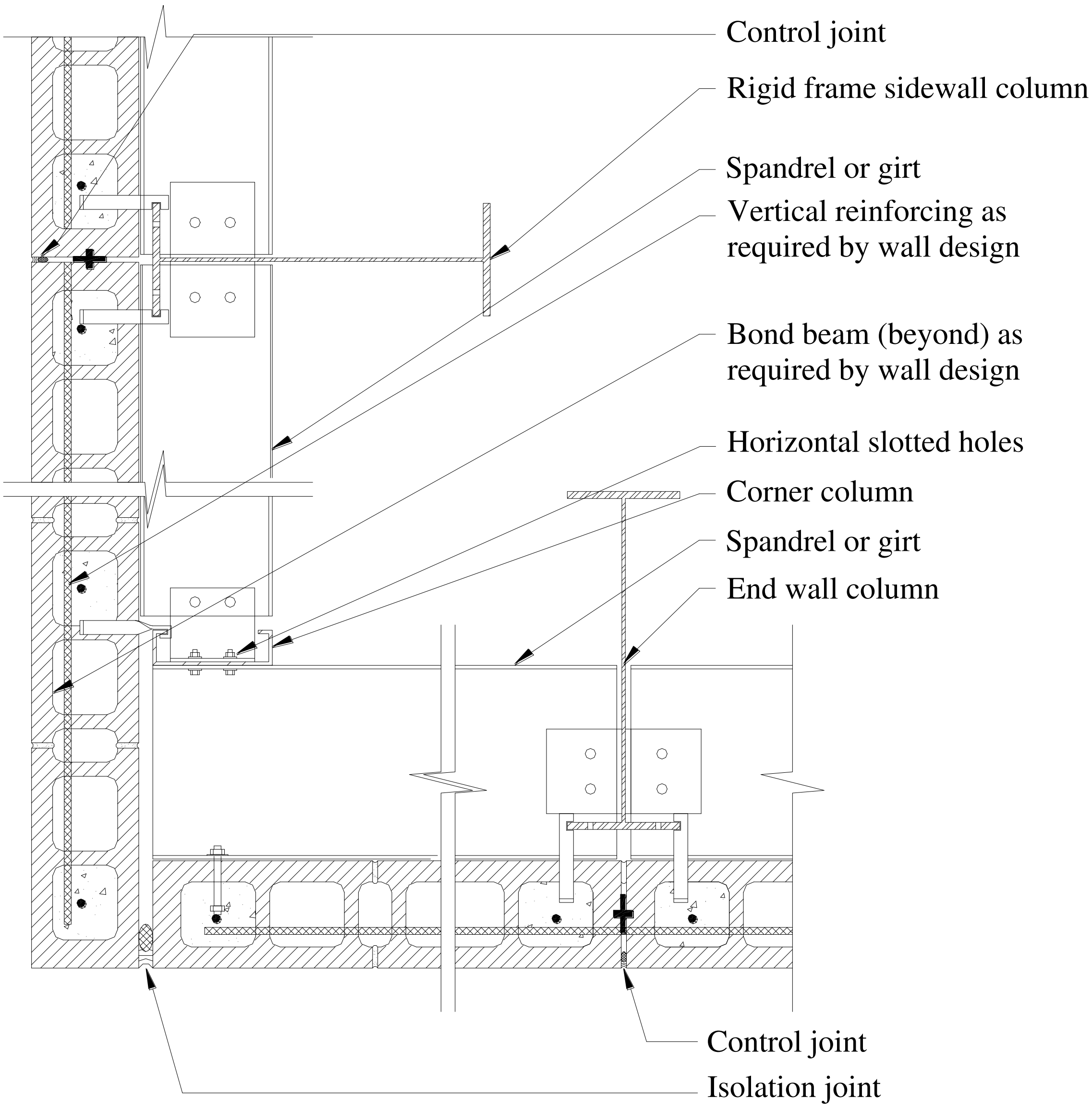

Figure 2.3E—Plan View Isolation Joint at Masonry Intersections

2.4 Reinforced Concrete Masonry Design Requirements

Plain (unreinforced) concrete masonry is not included in the scope of this manual. This manual focuses on reinforced concrete masonry walls, often utilized with metal building systems. Reinforcing steel provides ductility to concrete masonry walls and allows thinner walls to be built to resist wind and seismic loads.

This section provides guidance for the selection of appropriate wall thicknesses and reinforcement without the need to perform an extensive structural analysis. Caution should be used, however, to make sure the conditions specified in the selection tables and text are met at the job site. Additionally, local building codes should be consulted for specific requirements.

Reinforced concrete masonry walls are increasingly becoming the typical masonry wall type for metal buildings because the reinforcing steel provides the needed tensile strength to allow the walls to span significant distances. Reinforced masonry walls are also capable of accommodating substantial deflections and drifts that may be experienced with metal buildings. Appropriate design, detailing and construction of the walls and their connections to the metal buildings are required however in order for the building to function as intended. This section provides guidelines and design aids to assist the designer in sizing the required reinforcing for walls and lintels.

Typically masonry walls are designed as simply supported elements between the floor and roof diaphragms (vertical span) or between the metal building columns (horizontal span). Walls can also be designed as panels supported on four edges to gain the advantage of two-way bending; however, two-way bending of masonry panels is not addressed in this publication.

As discussed in Section 2.3, the deformation compatibility between the more flexible metal building and the stiffer masonry walls must be accommodated in the design. This is typically accomplished by providing a “hinge” at the base of the masonry wall panel so that the wall can rotate at the base (see Figures 2.3A through 2.3D). In this manner, the top of the wall translates with the metal building without causing additional bending stresses at the base of the wall (as would be expected if the wall was fixed at the base and subjected to a deflection at the top of the wall). Additionally, perpendicular walls should generally be isolated at corners so that the metal building frame can deform as designed.

2.4.1 Loads

Masonry walls for metal buildings can be used to resist vertical dead and live loads from the roof, as well as lateral wind and seismic loads. In many building applications, roof loads are resisted by the metal building frame and the masonry walls are designed for out-of-plane wind or seismic forces. Where the masonry walls are designed to be load bearing, the weight of the roof dead load and live load are often relatively light in comparison to the high wind and seismic pressures. It is rare, therefore, that the masonry wall design is governed by vertical loads.

Lateral wind and seismic loads vary greatly depending on the location of the building, the building height and its exposure. Local building codes or *Minimum Design Loads for Buildings and Other Structures* (ASCE 7) (ref. 4) can be consulted for additional information on load determination.

Information on determining seismic design loads is available from both NCMA and MBMA. See:

- *TEK 14-12B, Seismic Design Forces on Concrete Masonry Buildings* (ref. 13)
- *TEK 14-18B Seismic Design and Detailing Requirements for Masonry Structures* (ref. 13)
- *2010 Metal Building Systems Manual Supplement* (ref. 3)
- *Serviceability Design Considerations for Steel Buildings, AISC Steel Design Guide #3* (ref. 5).

2.4.2 Design Procedures

Masonry is relatively strong in compression and shear but is weak in tension. Accordingly, reinforcing steel is incorporated into masonry walls to resist the high tensile stresses that may develop, particularly in areas of high seismicity and/or high wind loads. Reinforced masonry structures have significantly higher flexural strength and ductility than unreinforced structures and provide greater reliability in terms of expected load carrying capacity at failure. Improved ductility of reinforced masonry is also a function of reinforcement, which continues to elongate well beyond the design level, allowing deformation beyond design levels without loss of strength. These deformations allow overloads to be redistributed to other members, thus improving structural performance when actual loads exceed design load levels.

Two methods of designing reinforced masonry structures are commonly used:

- Allowable stress design of masonry (also called the working stress design method) based on service level loads and proportioning members using conservative allowable stresses as provided for in Chapter 2 of the TMS 402/ACI 530/ASCE 5 (ref. 1). More information on allowable stress design of concrete masonry can be found in *TEK 14-7B Allowable Stress Design of Concrete Masonry, TEK 14-5A Loadbearing Concrete Masonry Wall Design,* and *TEK 14-19A Allowable Stress Design Tables for Reinforced Concrete Masonry Walls* (ref. 13).
- Strength design of masonry is based on a realistic evaluation of member strength subjected to factored loads, which have a low probability of being exceeded during the life of the structure. Chapter 3 of the TMS 402/ACI 530/ASCE 5 addresses strength design procedures of concrete masonry. More information on strength design of concrete masonry can be found in *TEK 14-4B Strength Design Provisions for Concrete Masonry* and *TEK 14-11B Strength Design of Tall Concrete Masonry Walls for Axial Load & Flexure* (ref. 13).

This document and the design aides provided are based on allowable stress design (working stress design). While strength design provides for additional economies in wall sections, allowable stress design is still the most common design method used today.

Note that special detailing is required for masonry shear walls for seismic loads. See:

- *TEK 14-18B, Seismic Design and Detailing Requirements for Masonry Structures* (ref. 13)
- *2010 Metal Building Systems Manual Supplement* (ref. 3)
- *Serviceability Design Considerations for Steel Buildings, AISC Steel Design Guide #3* (ref. 5).

2.4.3 Design Aids

Information on properties of concrete masonry walls that are helpful in the design of concrete masonry is found in *TEK 14-1B Section Properties of Concrete Masonry Walls* and *TEK 14-13B Concrete Masonry Wall Weights* (ref. 13).

The design aids in this manual utilize the allowable stress design with the alternative basic load combinations of the *International Building Code* (ref. 2). This option permits allowable stresses to be increased by one-third for load combinations that include wind or seismic loads. This results in a minimal economy for combinations involving wind loads as the wind load must be multiplied by 1.3 (ω) when using the alternative basic load combinations which essentially negates the one-third stress increase. For earthquake loading however, much greater economy is achieved as the amount of earthquake load is virtually the same in the load combinations with and without the one-third stress increase. The factor for earthquake loads for load combinations without the one-third stress increase is 0.7. Earthquake loads in load combinations with a stress increase are divided by 1.4. The net result is essentially the same load applied for earthquake in both cases.

The unfactored reactions for each load type will be provided by the metal building manufacturer for the masonry engineer to design the masonry panels. Note that a 1/3 stress increase is not permitted for structural steel design. Only the W wind loads are to be applied to the metal building steel members, not the factored wind loads ωW (1.3 W) used for designing the masonry members incorporating a one-third stress increase. For earthquake loads, however, the same loads are used for the metal building design as for masonry as pointed out above.

2.4.3.1 Masonry Walls Loaded Out-of-Plane

A wall that is loaded perpendicular to its long plan dimension is considered to be loaded out-of-plane. The same wall may also serve as a shear wall when loads are applied parallel to its long dimension. In Figure 2.4A, Wall A, Wall B, Wall C and Wall D are loaded out-of-plane when wind or other lateral loads are in the north-south direction. Such walls can be designed to span either horizontally or vertically, depending on how they are supported. When spanning vertically, lateral support must be provided at the top and bottom of the wall. In metal buildings, such support is generally provided by a spandrel at the top of the wall that transfers the loads to the rigid frames for the side walls or columns if located in the end wall. Usually in metal buildings, masonry walls designed solely for out-of-plane loads have no external axial load and as such are designed as vertical one-way slabs. If the masonry walls are axially loaded, their moment capacities are usually increased for the normal range of downward acting axial loads. If the walls are subjected to direct tension (uplift, which is seldom the case with metal building systems) their moment capacities are decreased. Shear walls in the end walls normally take just the shear load from the end half of the end bay.

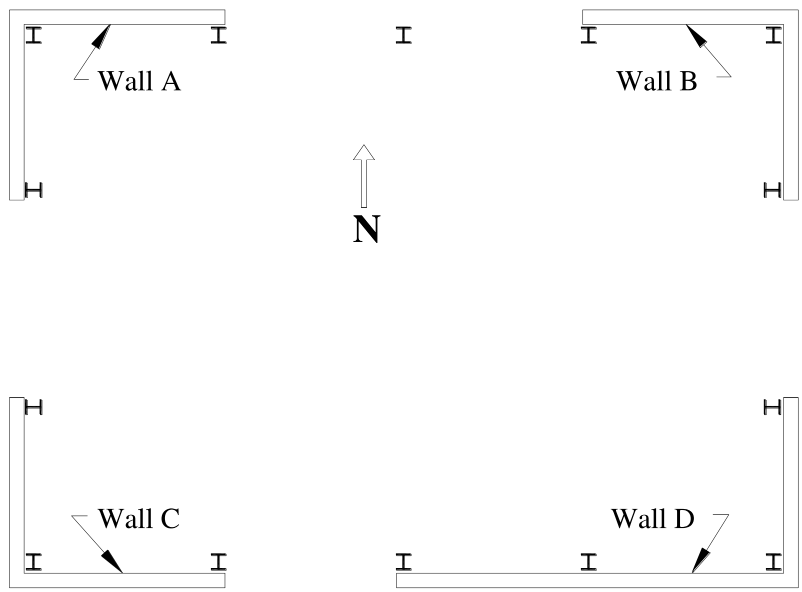

Figure 2.4A—Plan View, Metal Building with Masonry Walls

Figures 2.4B, 2.4C, and 2.4D are design aids for out-of-plane flexural capacity of nominal 8-in. (203 mm) fully grouted or partially grouted concrete masonry walls. The charts are based on allowable stress design with a 1/3 stress increase for IBC alternative basic load combinations, a specified compressive strength of masonry f'_m of 1500 psi (10.3 MPa), and reinforcement centered in the wall with a yield stress f_y of 60 ksi, (414 MPa). The masonry walls are designed as simply supported between the floor and at the spandrel at the top. Cantilevers at the top of the wall do not need additional reinforcement if the height of the wall above the spandrel attachment (including parapets) does not exceed the vertical span of the wall below the spandrel divided by 3. For cantilevers meeting these limitations, the resulting loads and stresses induced in the support wall are comparatively small relative to other controlling design loads. Figures 2.4C and 2.4B also include vertical axial downward acting loads which serve to increase the lateral load carrying capabilities of the wall. These loads include externally applied compressive loads as well as concrete masonry wall self-weight. Figure 2.4B includes no vertical axial load and therefore may be used to design horizontally spanning masonry walls as well. All axial loads are applied at the center of the masonry wall. Where roof uplift is applied to the masonry walls such as in Figures 2.1K, 2.1L, and 2.1M, the design aids included in this manual may not be used to design the masonry walls. The *Structural Masonry Design System* software (ref. 15) does accommodate designs with tension in masonry walls and is acceptable for use in those cases.

Walls spanning horizontally are supported by steel columns and are generally designed as simply supported as control joints at the columns break the continuity. When spanning horizontally, the reinforcement is placed in bond beams usually spaced at 48 in. (1219 mm) or less. In some seismic design categories, joint reinforcement placed in mortar joints can be used to provide horizontal reinforcement.

The shear capacity of walls loaded out-of-plane is almost always sufficient if the wall is adequate in flexure, especially when using allowable stress design. Nevertheless, the shear capacity should be checked.

Connections must be provided at the top and bottom of vertically spanning walls. Anchor bolts are often used in the top of such walls. Because lateral loads can occur in either direction, the bolts must be designed accordingly. At the bottom of vertically spanning walls, lateral support is often provided by the concrete footing. Normally in metal building systems, a slab-on-grade provides lateral support because the foundation wall is generally tied to the slab to resist the horizontal thrust imparted by the rigid frame columns – See Figures 2.3A, 2.3B, 2.3C, and 2.3D. If the foundation wall is not tied to the floor slab however, the point of support at the bottom of the wall should be taken at the footing level. Usually the bottom of a wall loaded out-of-plane is assumed to have no moment transfer to the footing. To be consistent with this assumption, the vertical reinforcing bars do not have to be made continuous into the footing for the out-of-plane loading. Where needed, however, shortened dowels wrapped to prevent bond are often used at the base of walls loaded out-of-plane to provide shear transfer at the interface of the wall and the footing as shown in Figures 2.3B and 2.3D.

For horizontally spanning walls, connections to the supporting columns must be provided, again often using anchor bolts in the masonry. Details for either horizontally or vertically spanning walls are provided in Sections 2.1 and 2.2.

Example 2.4A—Out-of-plane Flexure Design

Wall C in Figure 2.4A is to be designed using the alternative basic load combinations of the 2009 *International Building Code* (ref. 2) for out-of-plane flexure. The wall is modeled as a vertically spanning, simply-supported element designed to resist a lateral wind pressure of 20 psf (0.96 kPa) (before application of the $\omega = 1.3$ factor). No axial load is applied to the wall.

The distance between lateral supports (support at the bottom and the spandrel at the top) is 16 ft-0 in. (4.9 m) and the wall extends 2 ft above the spandrel for a total height of 18 ft (5.5 m). Material properties include:

- Nominal 8 in. (203 mm) concrete masonry units complying with ASTM C90;
- Type S masonry cement mortar;
- Specified masonry compressive strength, $f'_m = 1500$ psi (10.3 MPa); and
- Specified yield strength of reinforcement, $f_y = 60{,}000$ psi (414 MPa).

Increasing the wind pressure by 1.3 as required by the IBC when using the alternative basic load combinations, the design wind pressure for this example is:

$$W = (1.3)(20 \text{ psf}) = 26 \text{ psf } (1.24 \text{ kPa})$$

From Figure 2.4B for a 16 ft-0 in. (4.9 m) span subjected to a 26 psf (1.24 kPa) wind pressure, the minimum required area of reinforcement is approximately 0.10 in.^{2}/ft (0.21 mm^2/m).

From Table 2.4A, the following reinforcement options provide an area of reinforcement equal to or greater than 0.10 in.^{2}/ft (0.21 mm^2/m):

- No.4 bars at 24 in. (M#13 at 610 mm);
- No.5 bars at 32 in. (M#16 at 813 mm);
- No. 6 bars at 48 in. (M#19 at 1,219 mm); or
- No. 7 bars at 72 in. (M#22 at 1,829 mm).

In the above example the self-weight of the masonry wall was conservatively neglected. If, however, the weight of the wall was considered in the design, some design economy may result. Assuming that the installed weight of the wall is estimated at 50 psf (244 kg/m^2), the axial compressive load mid-way between the lines of lateral support (location of critical design stresses) is:

$$P = \left[\frac{16ft}{2} + 2ft\right]50 {}^{lb}\!/_{ft^2} = \text{ 500 plf (7.29 kN/m)}$$

However, the IBC alternative basic load combinations permits the use of only two-thirds of the minimum dead load likely to be in place during a design wind event when it counteracts the effects of the wind loads. Therefore Figure 2.4C (which includes a design axial load of 500 plf (7.29 kN/m)) cannot be used. Note that when the software is used (ref. 15) the dead load is considered because it is designing for the exact condition. See Example C2.4A in Appendix C.

The out-of-plane shear force resulting from the design wind pressure is:

$$V = \frac{\varpi Wl}{2} = \frac{(1.3)(20 {}^{lb}\!/_{ft^2})(16ft)}{2} = \text{208 lb/ft (3,036 N/m)}$$

Given that the vertical reinforcement is centered in the masonry wall (d = 3.81 in. (96.8 mm)), the shear stress resulting from the out-of-plane wind loading is:

$$f_v = \frac{V}{bd} = \frac{208\, lb/ft}{(12\, in.)(3.81\, in.)} = \text{4.55 psi (31.4 kPa)}$$

In accordance with Chapter 2 of *Building Code Requirements for Masonry Structures* (ref. 1), the allowable shear stress, increased by one-third for use with the alternative basic load combinations of the IBC, is:

$$F_v = {}^{4}\!/_{3}\sqrt{f'_m} = \left({}^{4}\!/_{3}\right)\sqrt{1{,}500\, psi} = \text{51.6 psi (356 kPa)}$$

Hence, the allowable shear stress (F_v) is considerably larger than the applied shear stress (f_v) and the shear design is satisfied. Most masonry walls that have been designed adequately for out-of-plane flexure have more than adequate strength for out-of-plane shear.

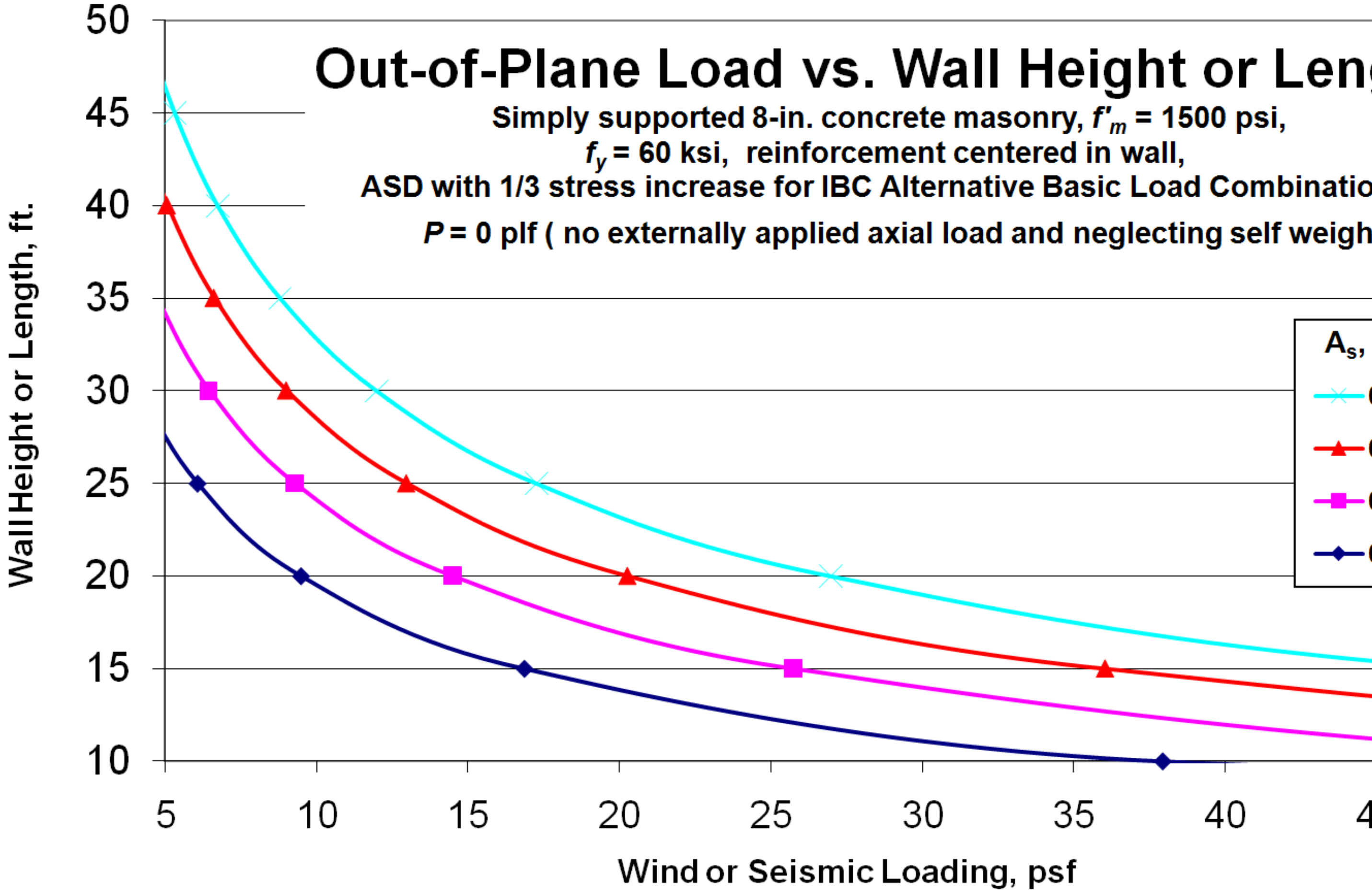

Figure 2.4B—**Out-of-Plane Flexural Capacity of a Simply Supported 8-inch Concrete Ma**
with No Axial Load
(See Appendix D for Metric Conversions)

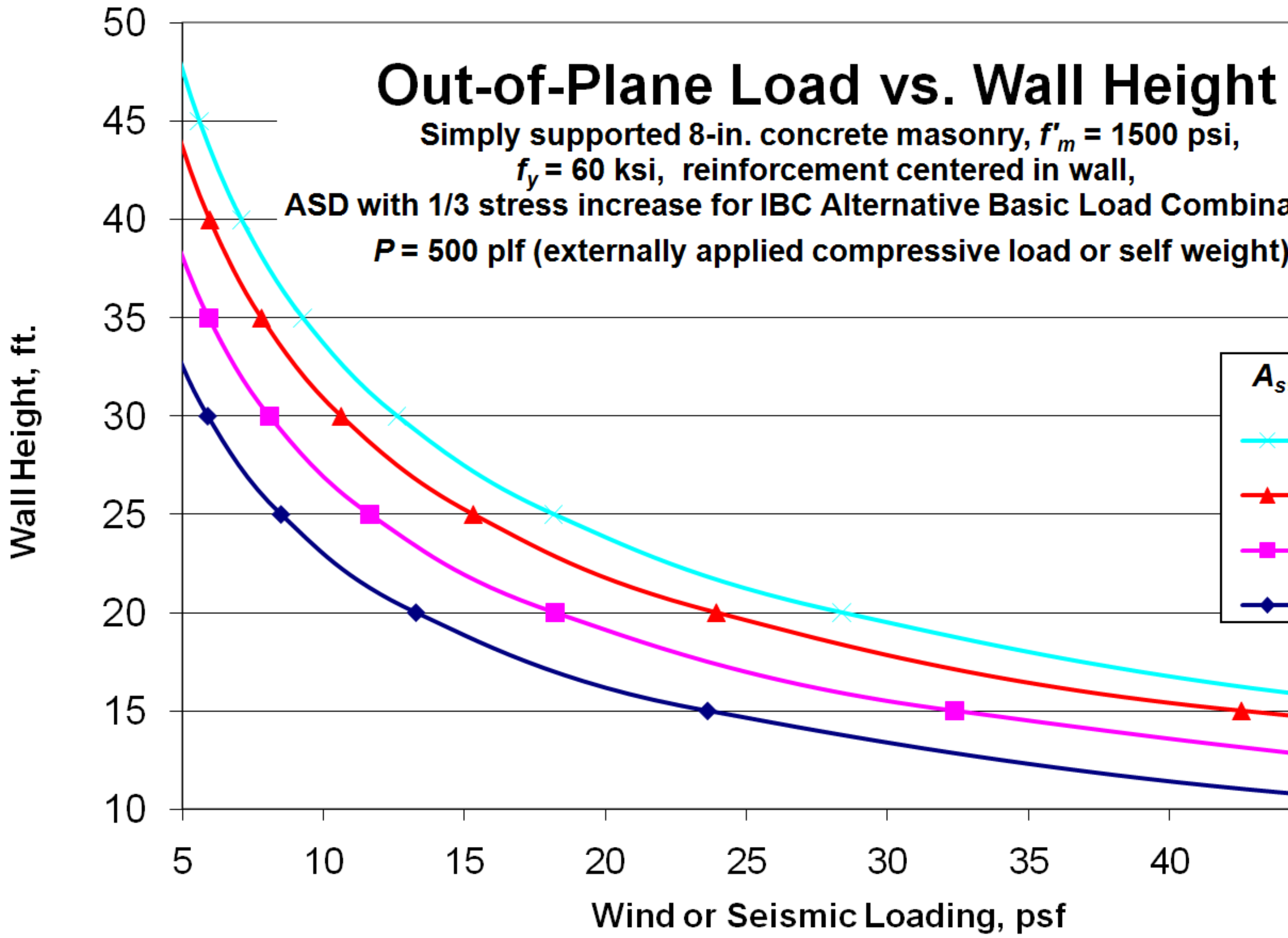

Figure 2.4C—**Out-of-Plane Flexural Capacity, Simply Supported 8-in. (203 mm) Concrete I with 500 plf (7.3 kN/m) Axial Load (See Appendix D for Metric Conversions)**

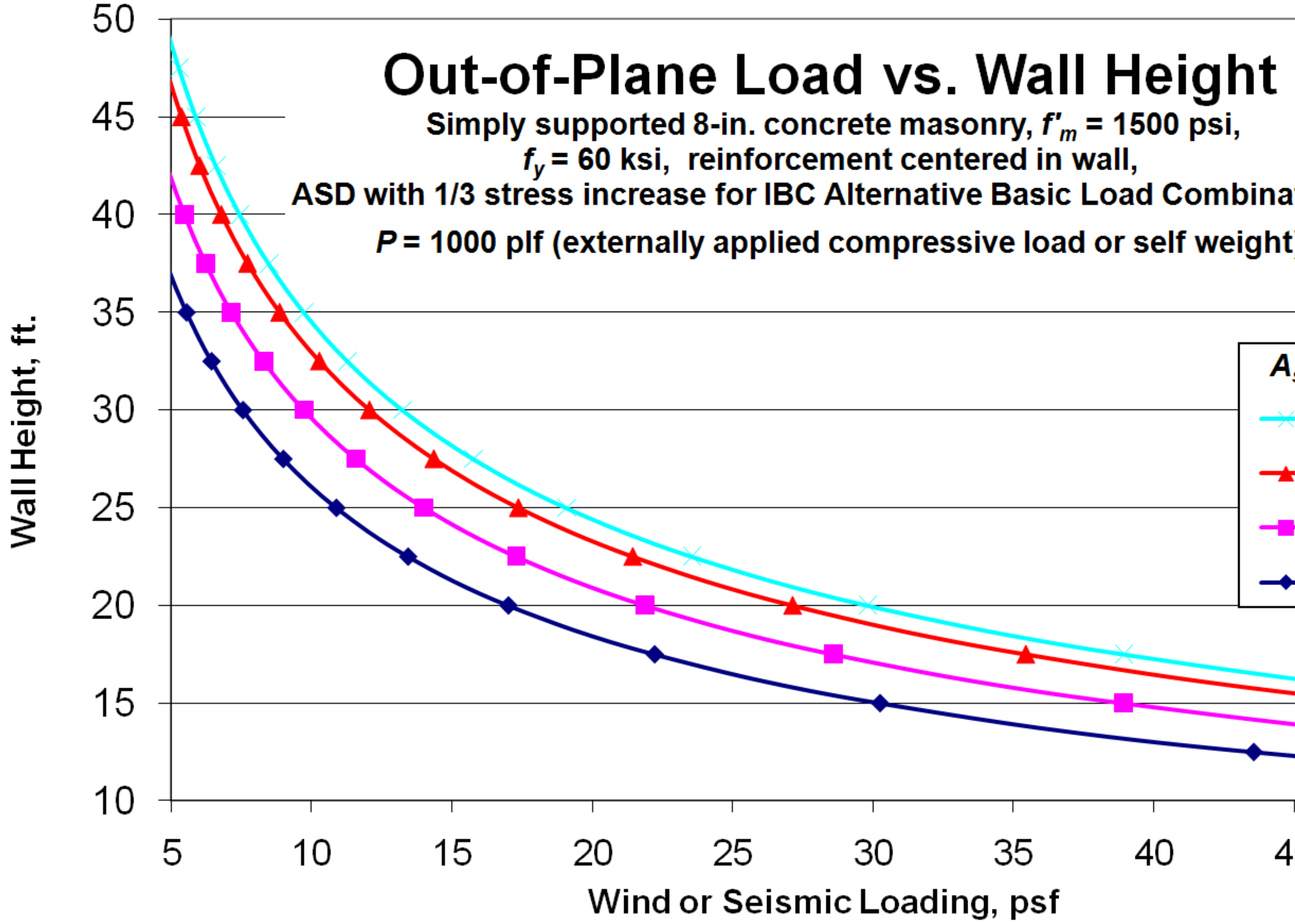

Figure 2.4D—**Out-of-Plane Flexural Capacity, Simply Supported 8-in. (203 mm) Concrete I with 1000 plf (14.6 kN/m) Axial Load (See Appendix D for Metric Conversions)**

Table 2.4A—Distributed Area of Steel Provided, in^2/ft (See Appendix D for Metric Conversions)

Bar spacing, in.	Bar area/size			
	0.20 in^2 No. 4	0.31 in^2 No. 5	0.44 in^2 No. 6	0.60 in^2 No. 7
8	0.300	0.465	0.660	0.900
16	0.150	0.233	0.330	0.450
24	0.100	0.155	0.220	0.300
32	0.075	0.116	0.165	0.225
40	0.060	0.093	0.132	0.180
48	0.050	0.078	0.110	0.150
56	0.043	0.066	0.094	0.129
64	0.038	0.058	0.083	0.113
72	0.033	0.052	0.073	0.100
80	0.030	0.047	0.066	0.090
88	0.027	0.042	0.060	0.082
96	0.025	0.039	0.055	0.075
104	0.023	0.036	0.051	0.069
112	0.021	0.033	0.047	0.064
120	0.020	0.031	0.044	0.060

2.4.3.2 Masonry Walls Loaded In-Plane (Shear Walls)

A shear wall is a wall that is loaded parallel to its long plan dimension. Masonry shear walls are very strong and stiff because of this orientation and are often used to resist lateral loads, especially perpendicular to the rigid frames of the metal building (side walls). In the plan view (Figure 2.4A) of a metal building, Wall A, Wall B, Wall C and Wall D are potential shear walls that can be used to resist lateral loads in the east-west direction. The actual portions of the shear walls that are capable of resisting the shear are the shear wall segments. Shear wall segments are the portions of wall that are uninterrupted from the support at the bottom to the support at the top (spandrel) except for minor penetrations such as utility penetrations. Minor penetrations through masonry shear wall segments designed in accordance with this guide are permitted provided that they do not exceed 6 in. (152 mm) in any dimension at the face of the wall and do not interrupt any reinforcement required by the design aids. In addition, the cumulative area of penetrations are not to exceed 144 in.^{2} (0.093 m^2) in any 10 ft^2 (0.93 m^2) of wall surface area. Shear wall segments must be at least 4 ft (1.2 m) tall when using the design aids in this manual.

Because the lateral load resisting systems of metal building systems are considered flexible, the lateral loads are distributed to the shear wall segments according to tributary area. The spandrel is designed by the metal building engineer to act as the drag strut within a bay (between columns) to distribute the loads to the shear wall segment. In addition, if the in-plane shear loading from one bay is to be resisted by a shear wall segment in another bay, the transfer of load from one bay to the next will be accommodated through the spandrel designed as a drag strut by the metal building engineer. In this case bond beam reinforcement should not be continuous across control joints. If the transfer of loads will not be through the spandrel, the masonry engineer needs to be notified and instructed to design the bond beam in the masonry wall to transfer the accumulated loads. In such cases, the bond beam reinforcement should be continuous through the control joints for the entire tributary length for which the shear wall segment is providing in-plane resistance. The masonry designer should keep in mind the differences in thermal movements between the spandrel and the masonry bond beam and not link together too many consecutive bond beams across control joints. The spandrel also will act as a stiffener providing lateral support for the sections of the bond beam that are in compression, thereby increasing buckling resistance. To minimize torsional irregularities and evenly distribute lateral loads, it is advisable to place shear wall segments at each corner and in each plan direction of a building.

The shear wall segments also must be constructed on a footing that is designed to resist the resulting forces. Usually it is necessary to lap-splice vertical bars in the shear wall segments with hooked bars placed in the footing to provide continuity of force in the bars. The design procedures in this manual are based on all of the vertical reinforcement in a shear wall segment contributing to the shear wall overturning resistance. One bar of the size indicated is located in the end cell at each end of the shear wall segment and the rest are uniformly distributed between them at the determined spacing. Therefore all vertical bars in a shear wall segment must be lap-spliced with foundation dowel bars. The lap splice length can be determined using Table 2.4B. If the masonry engineer cannot demonstrate that the stress in the reinforcement is less than $0.8F_s$, the lap-splice curve used should be the one for stress greater than $0.8F_s$. For vertical reinforcement within the middle 2/3 of the shear wall segment, the stress can reasonably be assumed to be less than $0.8F_s$ and the reduced lap-splice may be used for those bars however. Prescriptive seismic reinforcement should be lap spliced to the foundation at the greater than $0.8F_s$ lap-splice length. Wall sections that are not designed as shear wall segments are not to be continuous with the foundation dowels and must be structurally isolated from the lateral force-resisting system as shown in Figures 2.3B and 2.3D. Wall segments not structurally isolated as shown in those figures must contain the minimum prescriptive seismic detailing.

Example 2.4B—In-Plane Shear Design

Wall C in Figure 2.4A is to be designed using the alternative basic load combinations of the 2009 *International Building Code* (ref. 2) for in-plane shear. The wall is to be designed to resist an in-plane shear force of 6,000 lb (27 kN) in the E-W direction as a result of wind loading. No external axial load is applied to the wall. The wall segment spans 16 ft-0 in. (4.9 m) in the vertical direction between supports (see Figure 2.4E). Material properties include:

- Nominal 8 in. (203 mm) concrete masonry units complying with ASTM C90;
- Type S masonry cement mortar;
- Specified masonry compressive strength, f'_m = 1500 psi (10.3 MPa); and
- Specified yield strength of reinforcement, f_y = 60,000 psi (420 MPa).

The IBC alternative basic load combinations permit a one-third increase in allowable stresses to resist loads that include wind or earthquake. However, loads due to wind must be increased by a factor of 1.3 (ω = 1.3) and for load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event can be used.

Therefore the design shear force due to wind is increased by a factor of 1.3 ($\omega = 1.3$):

$$V = (1.3)(6{,}000 \text{ lb}) = 7{,}800 \text{ lb (35 kN)}$$

Assuming this shear wall segment will be partially grouted resulting in an installed weight of 50 psf (244 kg/m^2), Figure 2.4F can be used to determine the required spacing of the vertical reinforcement based on the following non-dimensionalized values:

$$\frac{P}{f'_m\,t\,l} \quad \text{and} \quad \frac{M}{f'_m\,t\,l^2}$$

Where:
P = (2/3)(50 psf)(18 ft)(8 ft) = 4800 lb (21 kN) – using only 2/3 of the dead load.
M = (7,800 lb)(16 ft) = 124,800 ft-lb (169 kN-m)
f'_m = (4/3)(1,500 psi) = 2,000 psi (13.8 MPa) – increased by one-third
t = 7.625 in. (194 mm)
l = 96 in. (2,438 mm)

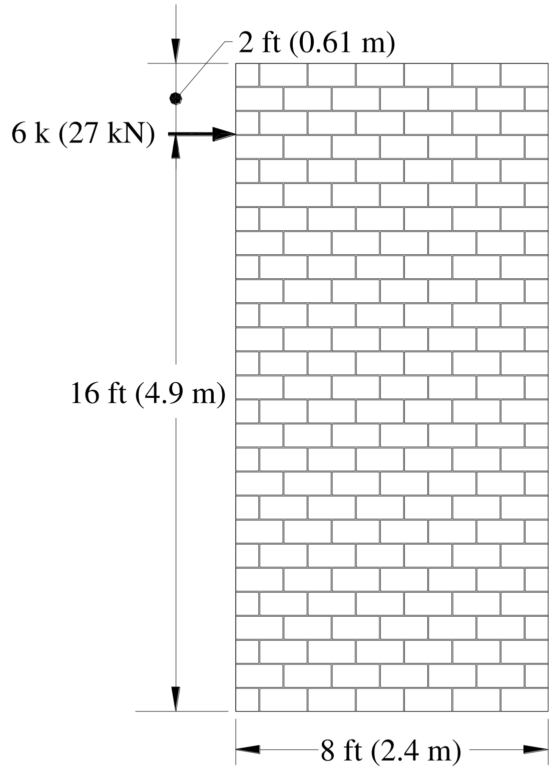

Figure 2.4E—Shear Wall Design Example

Hence:

$$\frac{P}{f'_m\,t\,l} = \frac{4800lb}{(2000psi)(7.625in.)(96in.)} = 0.0033$$

$$\frac{M}{f'_m\,t\,l^2} = \frac{(124800ft-lb)(12in./ft)}{(2000psi)(7.625in.)(96in.)^2} = 0.0107$$

From Figure 2.4F, visually interpolating between the two contours corresponding to a reinforcement ratio (ρ) equal to 0.1% and 0.2%, estimate ρ = 0.0015. The required area of reinforcement is:
A_s = (0.0015)(7.625 in.)(12 in./ft) = 0.137 in^2/ft (0.29 mm^2/mm).
From Table 2.4A, select No. 6 bars at 32 in. (M#19 at 813 mm) on center. Because the end cell at each end of the shear wall segment is required to contain the reinforcement determined by this manual, the remaining vertical reinforcing bars are spaced at a maximum distance of 32 in. (813 mm) for a total of four No. 6 (M#19) vertical reinforcing bars.

Taking d as the total wall length minus the 4 in. (102 mm) cover at the end of the wall, the shear stress on this wall is:

$$f_v = \frac{V}{bd} = \frac{7800lb}{(7.625\,in.)(92\,in.)} = 11.1 \text{ psi (76.5 kPa)}$$

In accordance with Chapter 2 of *Building Code Requirements for Masonry Structures* (ref. 1), the allowable shear stress, increased by one-third for use with the alternative basic load combinations of the IBC, is determined as follows assuming no shear reinforcement is provided:

$$F_v = \tfrac{4}{3}\left[\frac{1}{3}\left[4 - \frac{M}{Vd}\right]\sqrt{f'_m}\right]$$

Where:

$\dfrac{M}{Vd} = \dfrac{(124800 lb - ft)(12 in./ft)}{(7800 lb)(92 in.)} = 2.1$, however, the value of *M*/*Vd* need not be taken greater than 1.0. Hence, the equation for F_v reduces to:

$$F_v = \tfrac{4}{3}\sqrt{f'_m} = \tfrac{4}{3}\sqrt{1500} = 51.6 \text{ psi (356 kPa)}$$

The maximum permitted allowable shear stress is 35 psi (241 kPa) before application of the one-third stress increase. Hence, F_v is taken equal to (35 psi)(4/3) = 46.7 psi (322 kPa), which is significantly greater than the applied shear stress f_v of 11.1 psi (76.5 kPa). If applied shear stress was greater than the allowable, shear reinforcement would need to be provided to increase the strength of the shear wall segment.

Note that for partially grouted construction, the non-dimensionalized interaction diagram (Figure 2.4F) is not theoretically correct due to some simplifying assumptions; although for low axial loads, it is very close. Software distributed by NCMA (ref. 15) can be used to design this wall using partially grouted construction where more accuracy is desired. The solid grouted non-dimensionalized diagram in Figure 2.4G is theoretically correct, however.

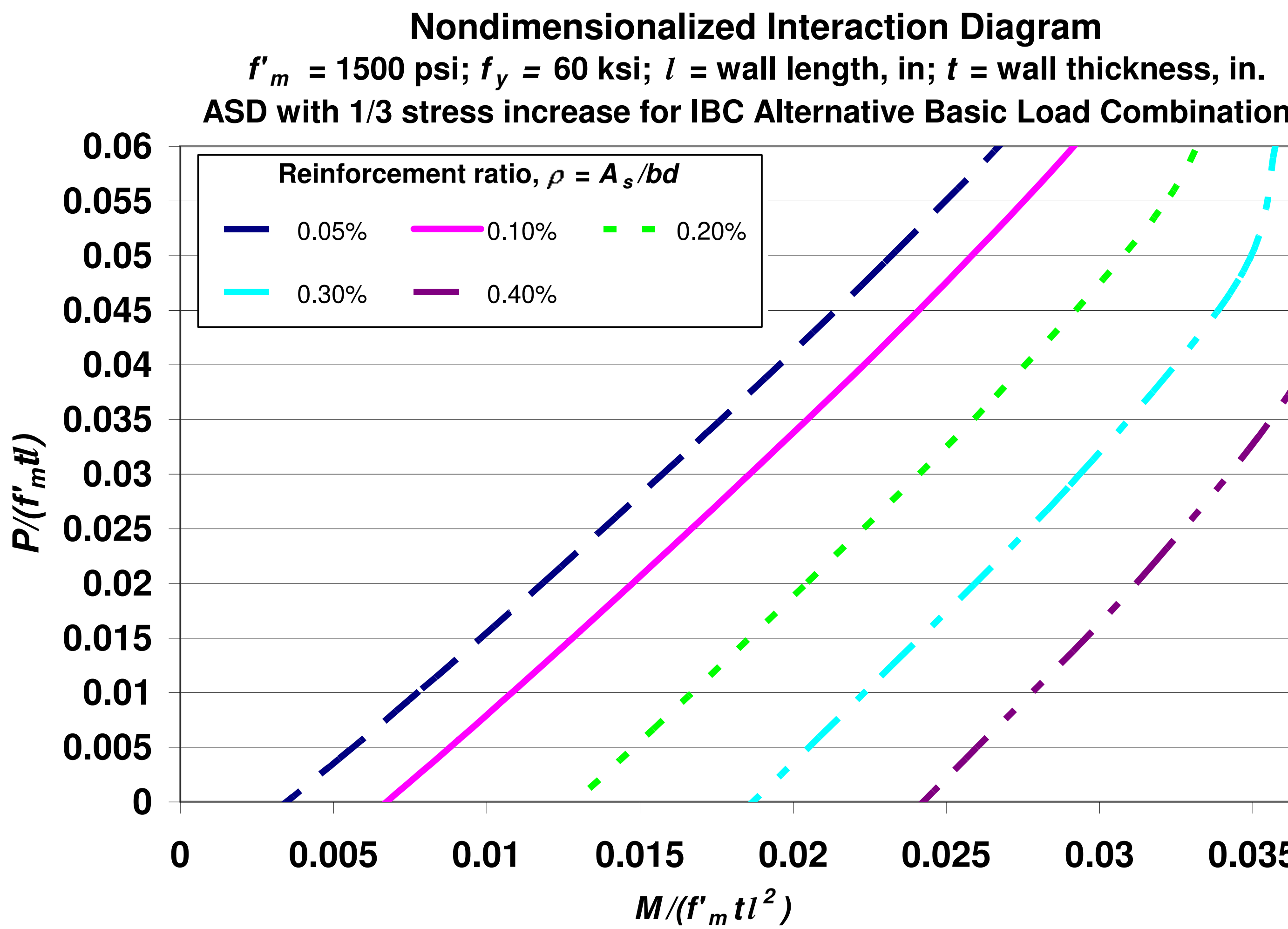

Figure 2.4F—Shear Wall Interaction Diagram for Partially Grouted 8-in. (203 mm) Concre
(See Appendix D for Metric Conversions)

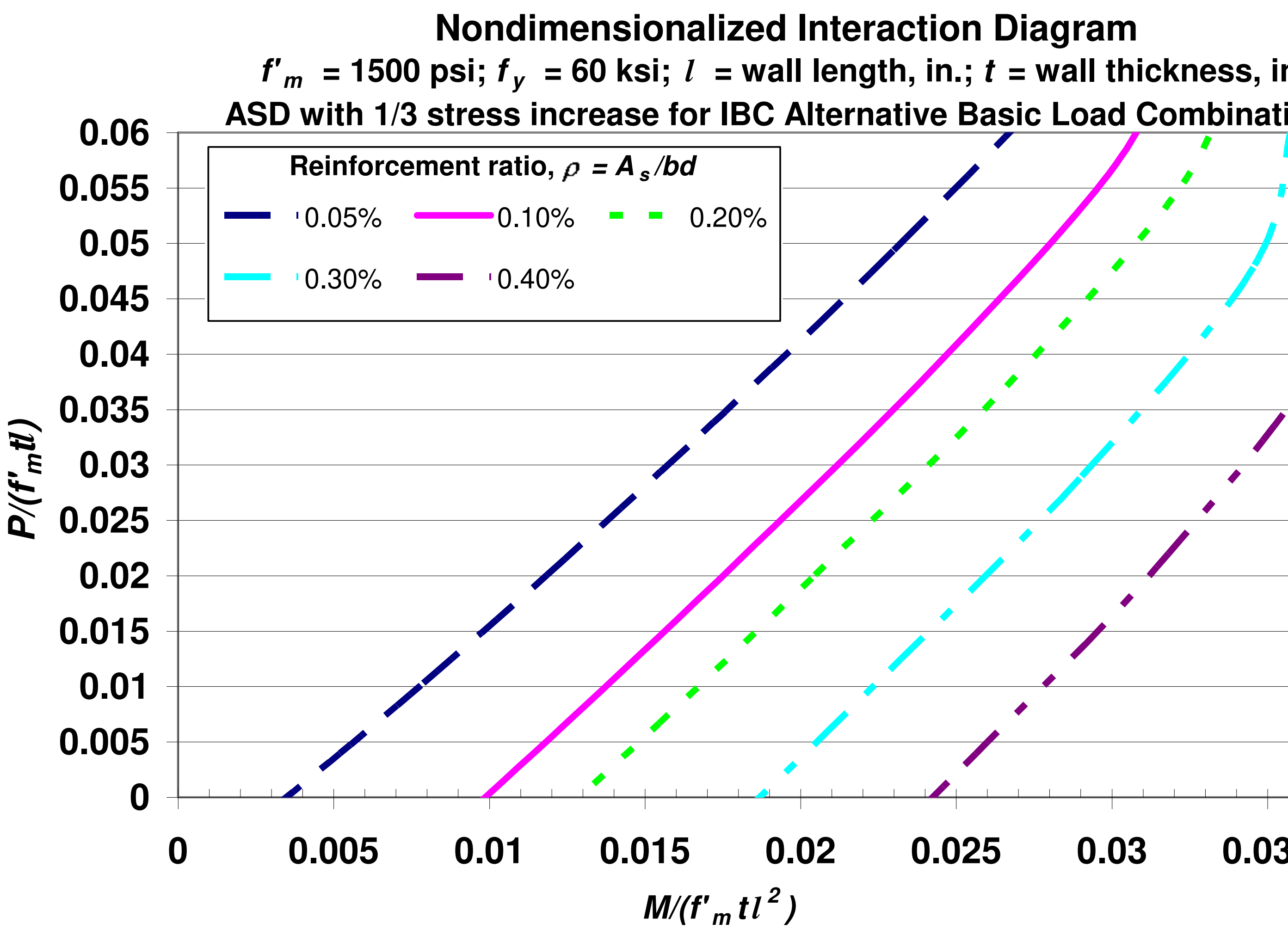

Figure 2.4G—Shear Wall Interaction Diagram for Solid Grouted 8-in. (203 mm) Concret
(See Appendix D for Metric Conversions)

2.4.3.3 Lap-splice Design

To maintain structural continuity and load paths, the vertical reinforcement of shear walls must be spliced to the foundation dowels. The foundation dowels in turn are embedded into the supporting footing, developed or hooked as necessary. See *TEK 12-6 Splices, Development and Standard Hooks for Concrete Masonry* (ref. 13). Minimum lap-splice lengths can be determined using Table 2.4B.

The lap-splice lengths for the No. 6 (M#19) vertical reinforcing bars in the end cells of the shear wall segment from the previous example would need to be a minimum of 54 in. (1,372 mm) because the full allowable design stress in the reinforcement was used. In cases where the tensile stress in the reinforcement does not exceed $0.8F_s$, such as the center two bars which are in the middle 2/3 of the segment, the lap splice length can be reduced to 36 in. (914 mm). Likewise, the lap-splice length could be reduced if smaller diameter vertical reinforcing bars had been used.

Table 2.4B—2009 IBC Allowable Stress Design Lap Splice Lengths[3] (ref. 2)

Bar size	Minimum lap splice length[1], in. (mm)	Minimum lap splice length[2], in. (mm)
No. 4 (M#13)	20 (508)	36 (914)
No. 5 (M#16)	25 (635)	45 (1,143)
No. 6 (M#19)	30 (762)	54 (1,372)

Notes:
1. Based on the stresses in the reinforcing steel f_s being limited to 80% of the allowable reinforcement tensile stress (F_s = 24,000 psi (165 MPa) for Grade 60 reinforcement). Lap splice length not to be less than 12 in. (305 mm) or $40d_b$. Minimum lap lengths may be smaller in cases where $f_s < 0.8F_s$.
2. Based on the stresses in the steel f_s taken equal to 100% of the allowable reinforcement tensile stress of 24,000 psi (165 MPa) for Grade 60 reinforcement. Lap splice length not to be less than 12 in. (305 mm) or $40d_b$.
3. See TEK 12-6 (ref. 13) for more information on splice lengths. (2009 IBC splice length requirements are the same as 2006 IBC).

2.4.3.4 Lintel Design

Openings in masonry walls must be spanned by beams called lintels. Usually the bottom course of masonry above the lintel is built with a "U-shaped" block (referred to as a lintel block or solid bottom bond beam block) that permits inclusion of horizontal reinforcing bars. The most efficient use of an open bottom bond beam block is to turn it upside down to obtain a larger effective depth *d*, particularly when architectural units are used. In this case the reinforcing steel must be tied up to provide the required amount of cover at the bottom. The shoring to support the lintel during construction is used as a removable grout stop to confine the grout.

The masonry must be fully grouted throughout the depth of the lintel. The effective depth, *d*, of the lintel is equal to the distance from the top of the grout to the centroid of the tension steel bars in the bottom course as shown in Figure 2.4H. If the wall is load-bearing, the lintel must carry superimposed loads in addition to its own weight. Otherwise, it must carry only its own weight and that of the masonry above it. The minimum required bearing length by the MSJC Code (ref. 1) is 4 in. (102 mm); however, half the length of one masonry unit (8 in.) (203 mm) is normally used for bearing to maintain continuity. A bearing length of (8 in.) (203 mm) is taken into account in the lintel design charts so that the actual clear span is reflected in Figures 2.4I through 2.4L. Therefore the code requirement for the span to be considered as center of bearing to center of bearing is already incorporated. More information on design of lintels for concrete masonry is found in *TEK 17-1C Allowable Stress Design of Concrete Masonry Lintels* and *TEK 17-2A Precast Lintels for Concrete Masonry Construction* (ref. 13).

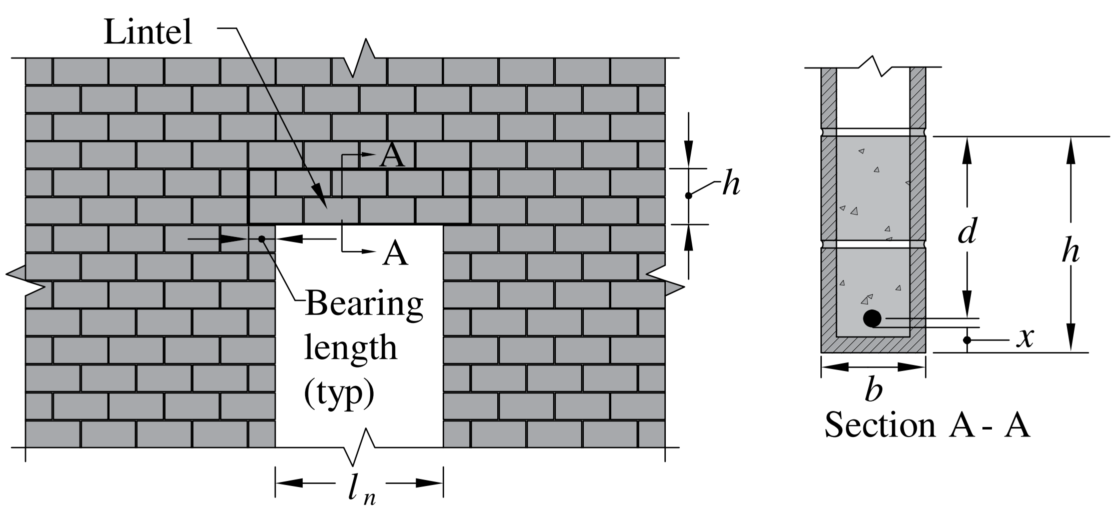

Figure 2.4H—Lintel Nomenclature

Figures 2.4I through 2.4L are design aids that assist in the design of masonry lintels. These figures are limited to wall thicknesses of 8 in. (203 mm) only. The design basis is Allowable Stress Design. The figures show the allowable superimposed load for clear spans (l_n in the illustration above) ranging from 4 ft. to 16 ft (1.2 to 4.9 m). The lintel is assumed to act as a simply supported beam with tension steel near the bottom. The critical section for bending always occurs at midspan. The shear capacity is based on the code provision that allows the maximum shear to be considered at a distance d/2 from the face of the support. For short spans, the shear capacity may control and has been taken into account in the tables so a separate check for shear is not required. The loads plotted in the figures have already been reduced by the lintel weight, therefore the load shown on the graphs is the allowable superimposed load. Additionally, the loads in these designs are gravity loads not wind or seismic. Therefore, these tables do not include a one-third stress increase.

Example 2.4C—Lintel Design

Design a lintel to support the dead load of the wall above it only. The clear span is 15 ft – 4 in. (4.7 m). The height of masonry above the opening is 4 ft – 0 in. (1.2 m) and the bearing length is 8 in. (203 mm) at each end.). Material properties include:

- Nominal 8 in. (203 mm) concrete masonry units complying with ASTM C90;
- Type S masonry cement mortar;
- Specified masonry compressive strength, f'_m = 1500 psi (10.3 MPa); and
- Specified yield strength of reinforcement, f_y = 60,000 psi (420 MPa).

Assume a two course lintel depth, so the height of masonry above the lintel is:

48 in. – 16 in. = 32 in. = 2.67 ft (0.814 m).

The masonry above the lintel is ungrouted, so its weight is 36 psf (176 kN/m^2) (from TEK 14-13B ref. 13).

The superimposed load is therefore (2.67 ft) (36 psf) = 96 plf (1400 N/m).

Because the design charts already include the lintel weight, enter Figure 2.4J on the x axis at 15.33 ft (4.67 m) and on the y-axis at 96 plf (1400 N/m). The result is 1 - No. 5 (M#16) bar with a cover x = 2 in. or 3 in. (51 or 76 mm) over the longitudinal reinforcing steel (see Figure 2.4H). Note that the highest load capacity for any bar selection for this span is around 160 plf (2334 N/m). If the superimposed load had been higher than 160 plf (2334 N/m) more courses would be required.

Also consider that a bond beam is normally placed at the top of the wall and at the location of a spandrel above the lintel. It generally is more economical to grout all the courses between a bond beam and the lintel rather than place grout stops to confine the grout particularly when grout for only three additional courses are involved, such as the case in this example. In so doing, the lintel in this case would be 4 ft (1.2 m) tall; however, the tallest lintel in the design aids in this manual is only 32 in. (813 mm) (Figure 2.4L). This could still be designed as a 32 in (813 mm) lintel supporting 16 in. (406 mm) of solid grouted masonry. From TEK 14-13B (ref. 13) the weight of fully grouted normal weight masonry is 84 lbs/ft^2 (411 kg/m^2).

Total supported weight is:

(84 psf)(16 in / 12 in./ft) = 112 plf (1635 N/m).

From Figure 2.4L, 1 - No. 4 (M#13) bar with x = 2 in. or 3 in. (51 or 76 mm) will work. Alternatively, the NCMA wall software (ref. 15) could be used to design this lintel.

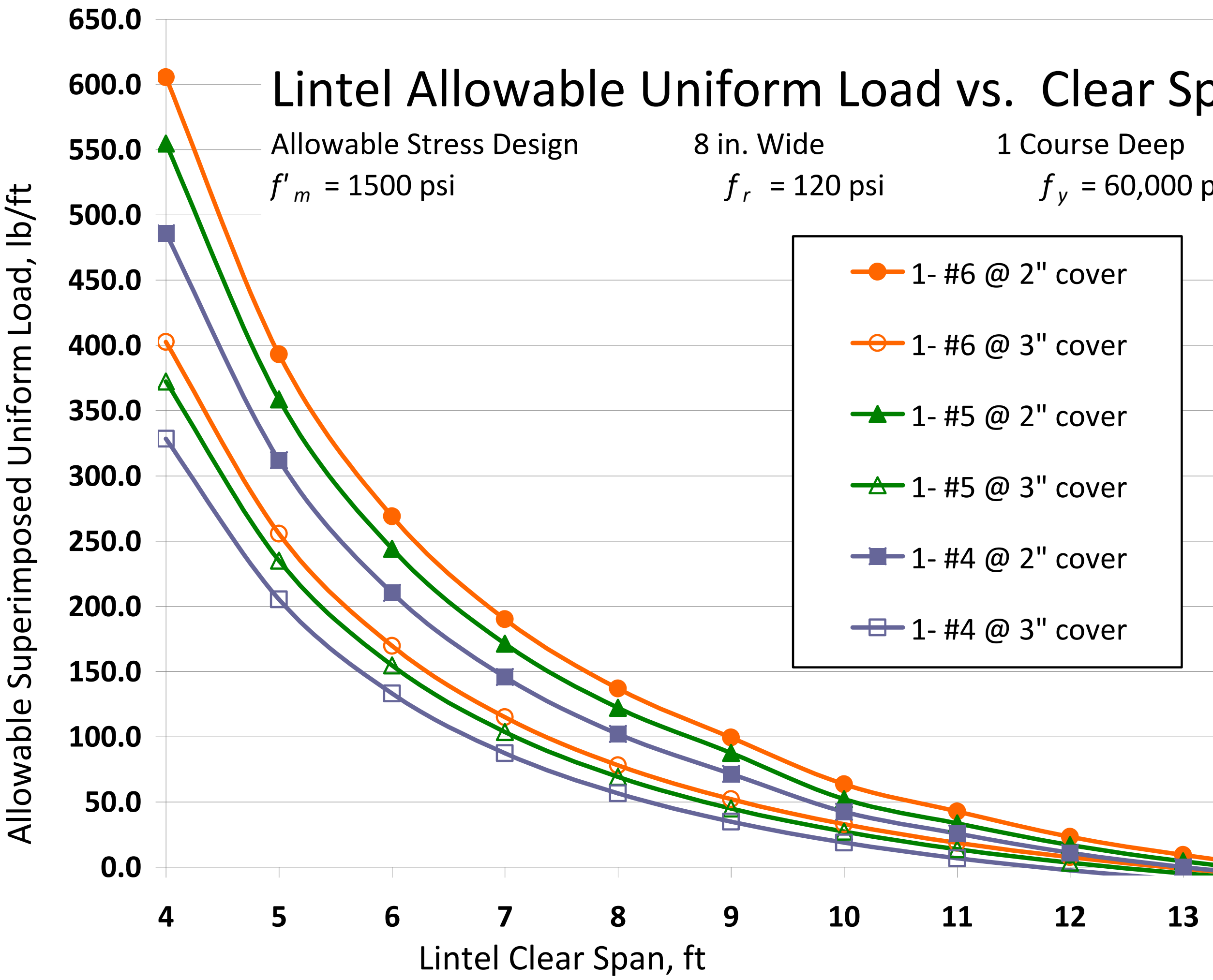

Figure 2.4l—Capacity of 8 x 8 in. (203 x 203 mm) Lintel (See Appendix D for Metric Conversion)

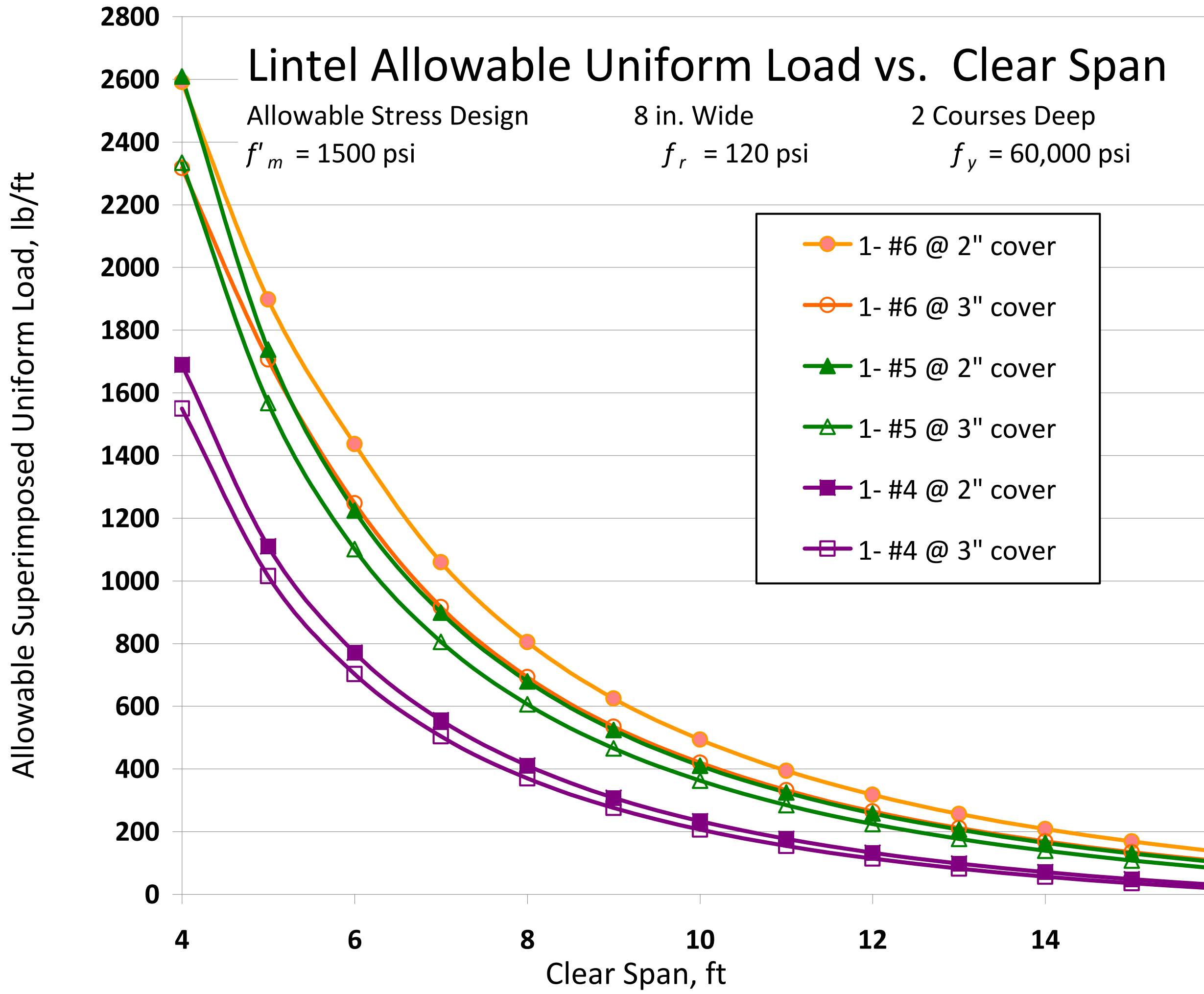

Figure 2.4J—Capacity of 8 x 16 in. (203 x 406 mm) Lintel (See Appendix D for Metric Conversion)

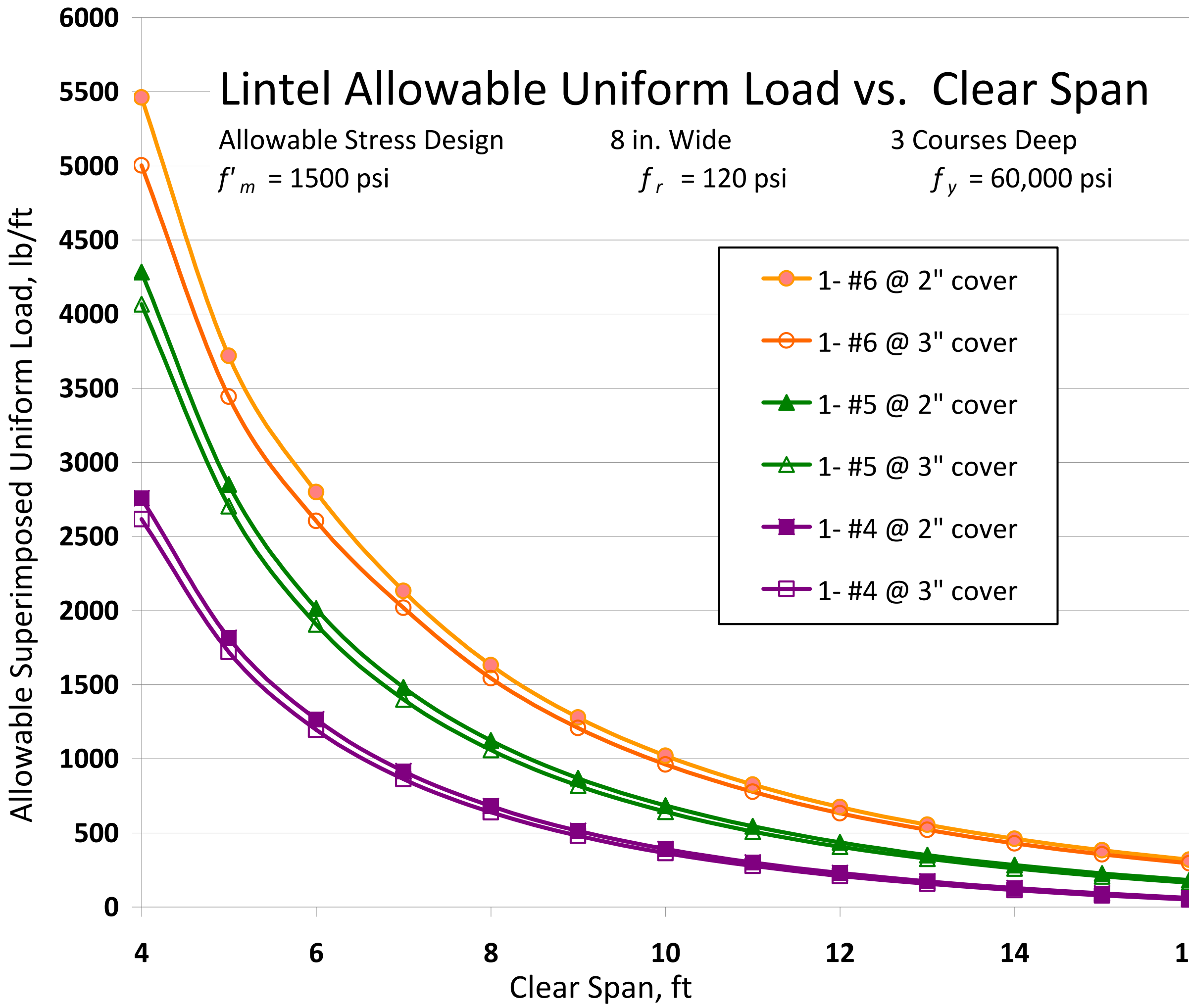

Figure 2.4K—Capacity of 8 x 24 in. (203 x 610 mm) Lintel
(See Appendix D for Metric Conversion)

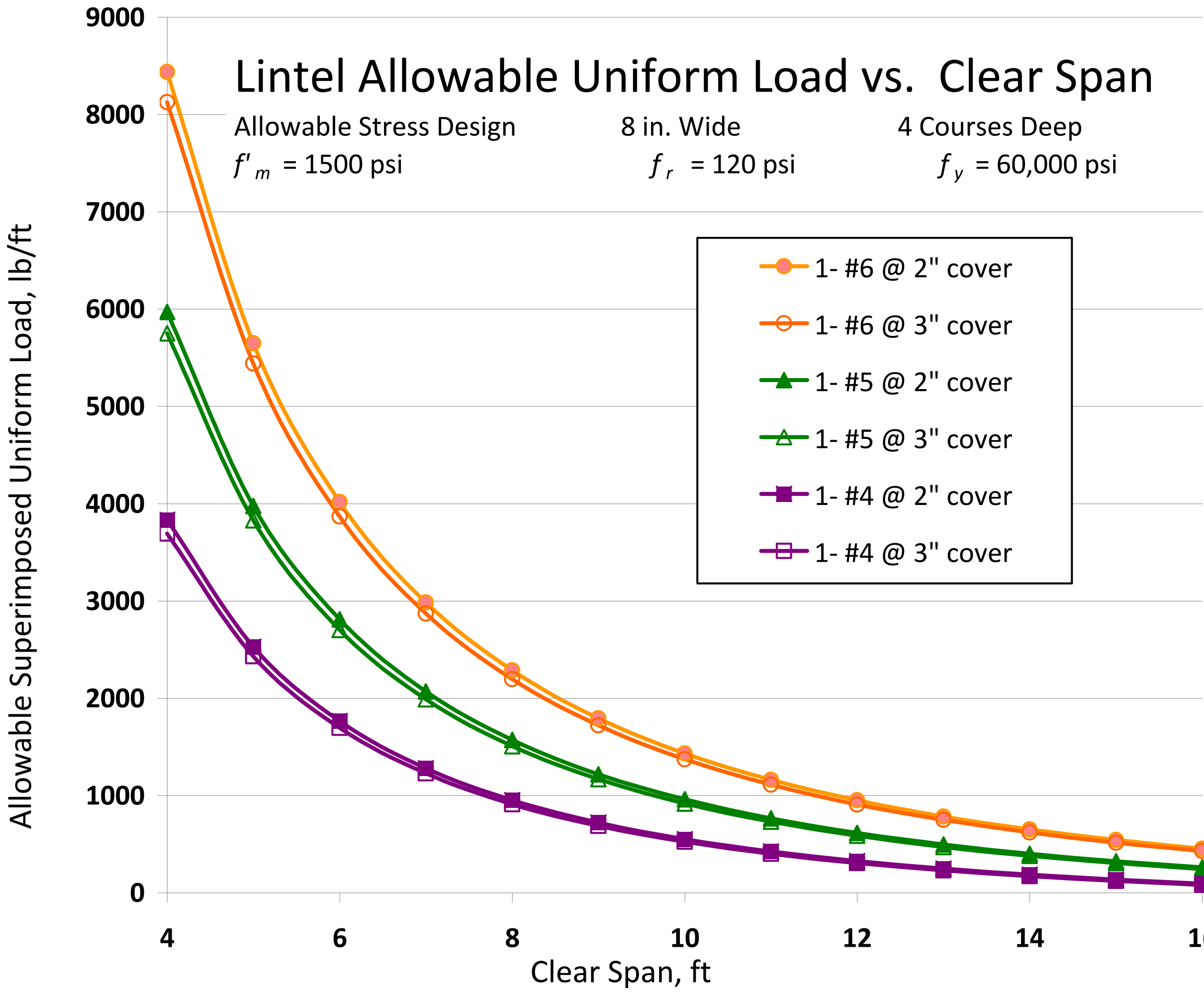

Figure 2.4L—**Capacity of 8 x 32 in. (203 x 813 mm) Lintel**
(See Appendix D for Metric Conversion)

2.3.4.5 Shear Wall Connections

The connections between masonry shear walls and their supporting elements are critical to the satisfactory performance of the building to ensure the transfer and distribution of loads. To use the design aids in this section (Figures 2.4Q and 2.4R), wind loads determined by ASCE 7 (ref. 4) are multiplied by 1.3 and earthquake loads are divided by 1.4 per the IBC alternative basic load combinations. More information on anchor bolts can be found in *TEK 12-3B Design of Anchor Bolts Embedded in Concrete Masonry* (ref. 13).

Example 2.4D—Anchor Bolt Design

The connection at the top of the shear wall shown in Figure 2.4E is to be designed for a 6,000 lb (26.7 kN) in-plane shear force and a 20 psf (1.0 kPa) out-of-plane wind pressure as illustrated in Figure 2.4M. The bond beam to which the spandrel is anchored is continuous for the full length of the 30 ft (9.1 mm) bay spacing.

As previously discussed, the default standard is to pre-punch ½ in. (13 mm) holes in the spandrel at a spacing of 17 in. (432 mm) and a gauge separation of 2.5 in. (64 mm) as shown in Figure 2.1U. To minimize the potential for spandrel buckling, the maximum spacing of the anchors should not exceed 34 in. (864 mm); indicating that every other set of bolt holes would contain at least one anchor.

Consider using one ½ in. (13 mm) headed anchor bolt at the maximum spacing of 34 in. (864 mm) and an embedment of 5.5 in. (140 mm). Over the 30 ft (9.1 m) bay this would provide 10 bolts. Note that if 2 bolts were provided within 2-½ in. (64 mm) of each other (two bolts spaced at 17 or 32 in. (432 or 864 mm) on center), the spheres of influence will intersect and individual anchor bolt capacities will be reduced as discussed later in the section.

The shear force applied to each bolt is calculated as:
Shear force per bolt = 1.3 x 6000 lbs / 10 bolts = 780 lbs/bolt (3.47 kN/bolt)

The tension force applied to each bolt (assuming the spandrel supported the top half of the 16 ft (4.9 m) vertical span plus the two foot section above the spandrel) is calculated as:

Tension force per bolt = $(1.3)(20\,psf)\left(\frac{16\,ft}{2}+2\,ft\right)\left(\frac{34\,in.}{12\,in./\,ft}\right)$ = 737 lbs/bolt (3.28 kN/bolt)

From Figure 2.4N, for an applied shear load of 780 lbs/bolt, the maximum allowable tensile load per bolt is approximately 1,800 lbs (8.0 kN), which is significantly larger than the applied tension load of 737 lbs/bolt (3.28 kN/bolt). Hence the proposed anchorage design is sufficient provided that a minimum edge distance for each headed anchor is 5.5 in. (140 mm) or greater. Alternatively, if bent-bar anchors were selected, Figure 2.4O would be used instead. Note that the capacity of bent-bar anchors is less than headed anchors; however, in this case they would work at the same spacing and embedment. For both types of anchors the location of the anchor should be alternated between the top and bottom holes of the flange to provide torsional resistance for the spandrel.

In cases where anchor bolts are spaced closely together, the projected breakout areas under tension or shear load that overlap must be reduced proportionally; and as a result, anchor strength is reduced proportionally. Additional guidance on determining overlapping breakout areas is provided in TEK 12-3B, *Design of Anchor Bolts Embedded in Concrete Masonry* (ref. 13).

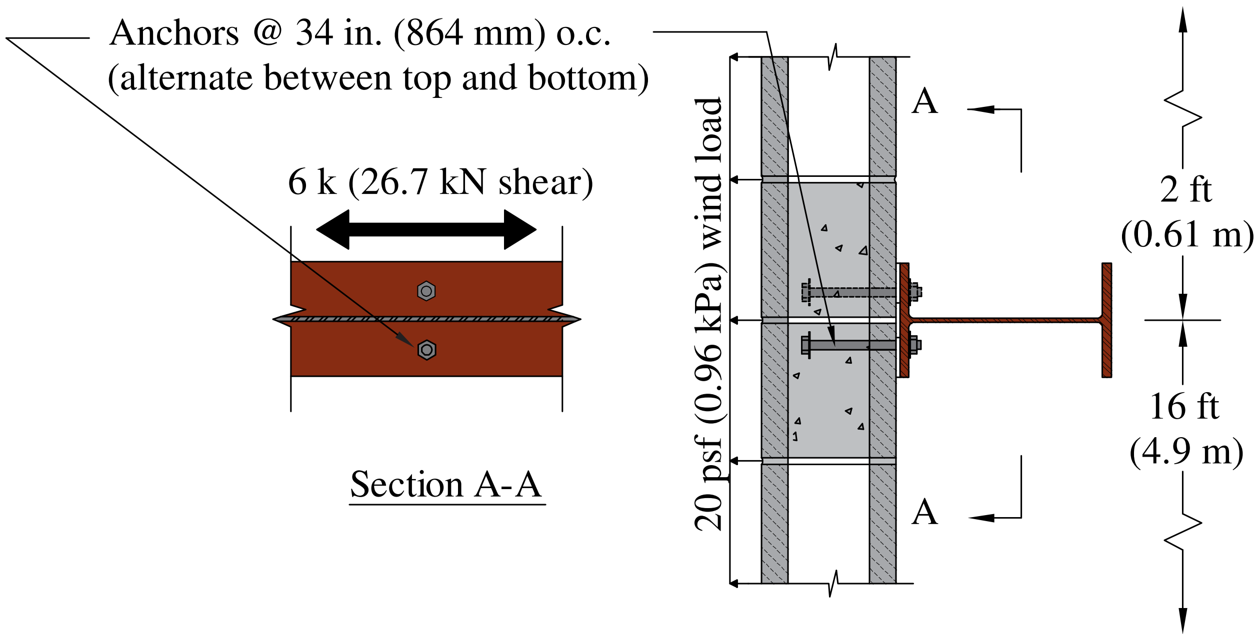

Figure 2.4M—Shear Wall Connection for Example 2.4D

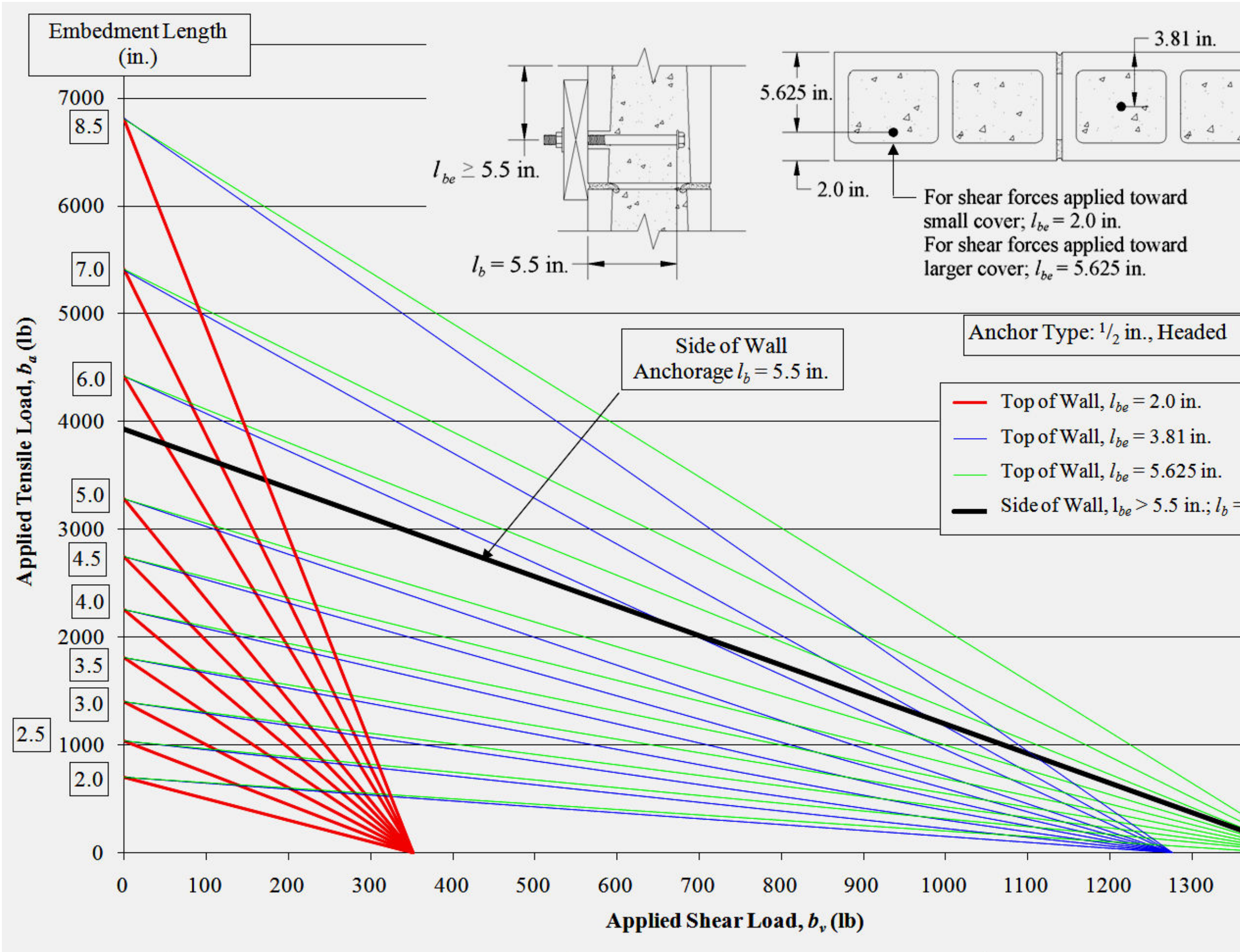

Figure 2.4N—Anchor Bolt Shear and Tension Capacities for ½ in. (13 mm) Headed Anchor Bo mm) CMU - f'_m = 1500 psi (10.3 MPa), f_y = 60 ksi (414 MPa); ASD with 1/3 Stress Increase for IBC Load Combinations (See Appendix D for Metric Conversions)

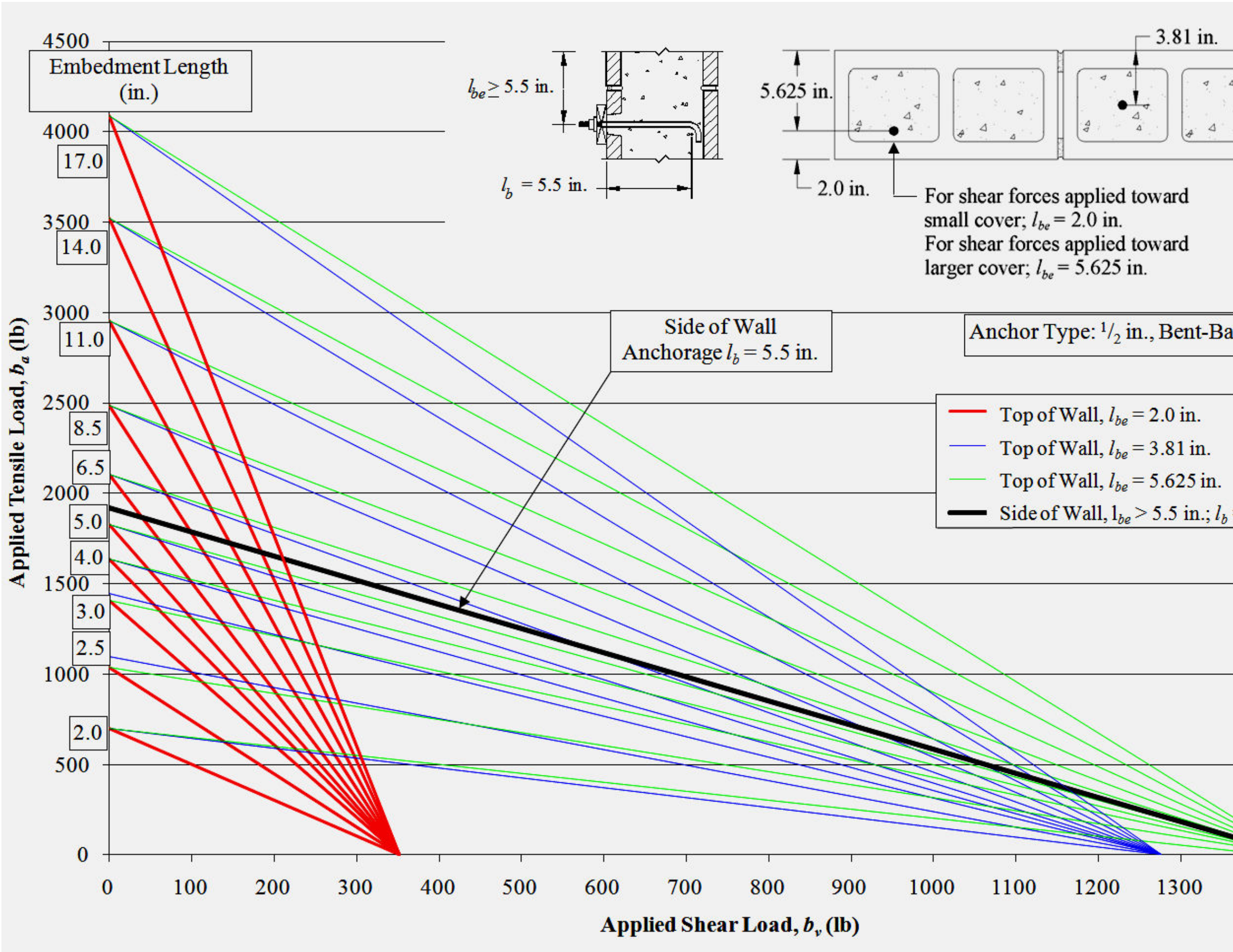

Figure 2.4O—Anchor Bolt Shear and Tension Capacities for ½ in. (13 mm) Bent-Bar Bolt in an 8 f'_m = 1500 psi (10.3 MPa), f_y = 60 ksi (414 MPa); ASD with 1/3 Stress Increase for IBC Alterna Combinations

(See Appendix D for Metric Conversions)

Chapter 3

Construction

3.0 General

Typically, construction of metal buildings with concrete masonry walls proceeds as follows:

1. Concrete footing placement.
2. Concrete masonry foundation wall construction to grade.
3. Concrete slab placement.
4. Steel erection.
5. Concrete masonry wall construction.

It may be advantageous to vary from this sequence to meet the needs of a particular project. For example, one exception to this construction sequence is the case of loadbearing end walls, where erection of the steel supported by the masonry will occur after the masonry wall is in place.

Coordination between the various trades is vital in order for the construction process to proceed efficiently. Preconstruction conferences are an excellent means for the various contractors and subcontractors to coordinate construction scheduling and to avoid conflicts and unforeseen delays.

3.1 Foundation Construction

Possibly the most important part of any structure is the foundation or footing system which transfers loads from the building into the supporting soil. When properly built and sized, footings will span areas of weak soil so that settlement and excessive cracking is avoided.

For masonry walls, footings are typically strip concrete or solid masonry footings. These footings must be located beneath the frost level to prevent damage and heaving caused by freezing of water in the soil.

Footings should be placed on undisturbed native soil, unless this soil is unsuitable, weak or soft. Unsuitable soil should be removed and replaced with compacted soil, gravel or concrete. Similarly, tree roots, construction debris and ice should be removed prior to placing the footings.

Footings must be carefully aligned so that the concrete masonry wall will be near the center of the footing. The top of the footing must also be relatively level and true to facilitate the construction of the concrete masonry walls. Note that although the top surface of concrete footings should be placed relatively level, it should generally not be troweled smooth. Instead, slightly roughened footing surfaces contribute to the bond between the mortar and the concrete.

Footings should generally be at least as deep as the wall thickness and roughly twice as wide as the wall thickness. A minimum of two No. 4 (M# 13) bars should be used to help avoid cracking and differential settlement. Larger or more heavily reinforced footings may be required on weak soil or to carry heavy loads.

Concrete for footings should have a minimum specified strength of 2,500 psi (17.2 MPa). Adding excessive water to the concrete mix in the field should be avoided because the strength of the concrete can be significantly decreased.

3.2 Concrete Masonry Construction

Concrete masonry construction is straightforward, while allowing masonry walls to be constructed in a variety of ways using different techniques, materials and methods. The figures in sections 2.1 through 2.3 show typical construction details for reinforced concrete masonry walls for metal buildings. Explanatory notes are also provided regarding specific items relating to typical wall construction. Masonry construction should comply with the requirements of *Specification for Masonry Structures* (TMS 602/ACI 530.1/ASCE 6) (ref. 6).

Prior to laying the first course of masonry, the top of the footing must be cleaned of mud, dirt, ice or other materials which can reduce the bond between the mortar and the footing. This can usually be accomplished using brushes or brooms, although excessive oil or dirt may require cleaning solutions or sand blasting of the concrete surface.

Masons typically lay the corners of the wall first so that alignment is easily maintained. This also allows the mason to plan where openings are necessary for windows and doors or to fit the building's plan. Note that the most economical and best use of the material is to make masonry openings modular (multiples of 8 in. (203 mm)) in size and location from corners and each other to minimize cutting of units and waste. See NCMA *TEK 5-12 Modular Layout of Concrete Masonry* (ref. 13) for more information on this topic.

To make up for surface irregularities in the footing, the first course of masonry is set on a varying thickness mortar bed joint which can range from $^1/_4$ to $^3/_4$ in. (6 to 19 mm) in thickness. This initial bed joint should be a full bed mortar joint. Mortar should not excessively protrude into cells that are to be grouted.

All other mortar joints should be $^3/_8$ in. (10 mm) thick with a code allowed tolerance of ±⅛ in. (3 mm) and face shell mortar bedding. In partially grouted construction, webs adjacent to the grouted cells are mortared to restrict grout from flowing into ungrouted areas. Head joints must be filled solidly for a thickness equal to the face shell thickness of the units.

In general, tooled concave joints are recommended because they provide the greatest resistance to water penetration and the timing for the tooling should be delayed until the mortar is thumbprint hard. This provides some resistance to the tooling so that the mortar is compressed against the surrounding masonry units forming a tighter seal. Type S mortar is typically used for reinforced masonry wall construction particularly in high wind areas. Type N is an acceptable alternate as it has better workability and lower shrinkage potential, but lower plain masonry flexural strength and durability. Type M mortar is seldom needed or used. Although it provides high compressive strengths, Type M mortar also has a higher shrinkage potential. Type M is usually stiffer mortar and tends to be difficult to use. Type O mortar is not appropriate for most structural applications.

For more information on mortar, grout, masonry manufacturing standards, terms, construction and inspection, see the following NCMA TEK (ref. 13) available free on line on sponsoring member web sites:

- *TEK 1-1E ASTM Specifications for Concrete Masonry Units*
- *TEK 1-4 Glossary of Concrete Masonry Terms*
- *TEK 3-1C All-Weather Concrete Masonry Construction*
- *TEK 3-2A Grouting Concrete Masonry Walls*
- *TEK 3-6B Concrete Masonry Veneers*
- *TEK 3-8A Concrete Masonry Construction*
- *TEK 5-1B Concrete Masonry Veneer Details*
- *TEK 5-2A Clay & Concrete Masonry Banding Details*
- *TEK 5-3A Concrete Masonry Foundation Wall Details*
- *TEK 5-6A Concrete Masonry Curtain and Panel Wall Details*
- *TEK 5-7A Floor and Roof Connections to CM Walls*
- *TEK 5-8B Detailing Concrete Masonry Fire Walls*
- *TEK 5-9A Concrete Masonry Corner Details*
- *TEK 5-10AConcrete Masonry Radial Walls*
- *TEK 5-12 Modular Layout of Concrete Masonry*
- *TEK 5-13 Rolling Door Details for Concrete Masonry Construction*
- *TEK 5-14 Concrete Masonry Hurricane and Tornado Shelters*
- *TEK 5-15 Details for Half-High Concrete Masonry Units*
- *TEK 5-16 Aesthetic Design with Concrete Masonry*
- *TEK 9-1A Mortars for Concrete Masonry*
- *TEK 9-2B Self-Consolidating Grout for Concrete Masonry*
- *TEK 9-3A Plaster and Stucco for Concrete Masonry*
- *TEK 9-4A Grout for Concrete Masonry*
- *TEK 18-3B Concrete Masonry Inspection*
- *TEK 18-5A Masonry Mortar Testing*
- *TEK18-6 Structural Testing of Concrete Masonry Assemblages*
- *TEK 18-8B Grout Quality Assurance*

3.3 Construction Details

Details of the connection between horizontal steel girts and concrete masonry walls are illustrated in the figures in Chapter 2. Shim plates should be used at connections between the girt and masonry to allow for camber in the girt and other construction tolerances. The steel girt should never be pulled to the masonry wall by tightening the anchor bolts.

APPENDIX A

Notations

A_s = effective cross-sectional area of reinforcement, in.^{2} (mm^2)
b = width of section, in. (mm)
b_a = axial force on anchor bolt, lb (N)
b_v = shear force on anchor bolt, lb (N)
D = dead load
d = distance from the extreme compression fiber to the centroid of the tension reinforcement, in. (mm)
d_b = nominal diameter of anchor bolt, in. (mm)
e_b = projected leg extension of bent bar anchor, measured from inside edge of anchor at bend to farthest point of anchor in the plane of the hook, in. (mm)
f'_m = specified compressive strength of masonry, psi (MPa)
f_r = modulus of rupture, psi (MPa)
F_s = allowable tensile or compressive stress in reinforcement, psi (MPa)
f_s = calculated tensile or compressive stress in reinforcement, psi (MPa)
F_v = allowable shear stress, psi (MPa)
f_v = calculated shear stress, psi (MPa)
f_y = specified yield strength of reinforcement, psi (MPa)
h = effective height of masonry element, in. (mm)
l_b = effective embedment length of anchor bolts, in. (mm)
l_{be} = anchor bolt edge distance, measured in direction of load, from edge of masonry to center of the cross section of anchor bolt, in. (mm)
l = length of shear segment, in. (mm)
l_n = lintel clear span, ft (m)
M = maximum calculated bending moment at section under consideration, lb-in., (N-mm)
P = applied axial load, lb (N)
t = thickness of masonry element, in. (mm)
V = applied shear force, lb (N)
W = wind load
ρ = reinforcement ratio
ω = coefficient to be applied to wind loads determined by Chapter 6 of ASCE 7 when using IBC alternative basic load combinations

Definitions

Aggregate: An inert granular or powdered material such as natural sand, manufactured sand, gravel, crushed stone, slag, fines and lightweight aggregate, which, when bound together by a cementitious matrix forms concrete, grout or mortar.

Anchor: Metal rod, tie, bolt or strap used to secure masonry to other elements. May be cast, adhered, expanded or fastened into masonry.

Anchor bolts: Bolts set in concrete or masonry, used to anchor structural members to the foundation or wall.

Axial load: The load exerted on a wall or other structural element and acting parallel to the element's axis. Axial loads typically act in a vertical direction, but may be otherwise depending on the type and orientation of the element.

Bay: The space between frame center lines or primary supporting members in the longitudinal direction of the building. Also called stanchion spacing.

Bay-end: The distance between the centerline of the first interior frame to the inside of the end wall panel.

Bay-interior: The distance from centerline to centerline of two interior columns.

Beam: A structural member, typically horizontal, designed to primarily resist flexure.

Block: A solid or hollow unit larger than brick-sized units. (See also "Concrete block, concrete masonry unit, masonry unit")

Bond: (1) The arrangement of units to provide strength, stability or a unique visual effect created by laying units in a prescribed pattern. See reference 6 for illustrations and descriptions of common masonry bond patterns. (2) The physical adhesive or mechanical binding between masonry units, mortar, grout and reinforcement. (3) To connect wythes or masonry units.

Bond beam: (1) The grouted course or courses of masonry units reinforced with longitudinal bars and designed to take the longitudinal flexural and tensile forces that may be induced in a masonry wall. (2) A horizontal grouted element within masonry in which reinforcement is embedded.

Bond beam block: A hollow unit with depressed webs or with "knock-out" webs (which are removed prior to placement) to accommodate horizontal reinforcement and grout.

Bond breaker: A material used to prevent adhesion between two surfaces.

Bond, running: The placement of masonry units such that head joints in successive courses are horizontally offset at least one-quarter the unit length. Centering head joints over the unit below, called center or half bond, is the most common form of running bond. A horizontal offset between head joints in successive courses of one-third and one-quarter the unit length is called third bond and quarter bond, respectively.

"C" Section: A member cold-formed from steel coil in the shape of a "C", used primarily in bearing frame end walls and framed openings.

Cantilever: A member structurally supported at only one end through a fixed connection. The opposite end has no structural support.

Cavity: A continuous air space between wythes of masonry or between masonry and its backup system. Typically greater than 2 in. (51 mm) in thickness.

Cell: The hollow space within a concrete masonry unit formed by the face shells and webs. Also called core.

Column: (1) In structures, a relatively long, slender structural compression member such as a post, pillar, or strut. Usually vertical, a column supports loads that act primarily in the direction of its longitudinal axis. (2) In masonry, for the purposes of design, an isolated vertical member whose horizontal dimension measured at right angles to the thickness does not exceed 3 times its thickness and whose height is greater than 4 times its thickness.

Component: A part used in a Metal Building System.

Compressive strength of masonry: Maximum compressive force resisted per unit of net cross-sectional area of masonry, determined by testing masonry prisms or as a function of individual masonry units, mortar and grout in accordance with ref. 6. (See also "Specified compressive strength of masonry.")

Concrete: A composite material that consists of a water reactive binding medium, water and aggregate (usually a combination of fine aggregate and coarse aggregate) with or without admixtures. In portland cement concrete, the binder is a mixture of portland cement, water and may contain admixtures.

Concrete block: A hollow or solid concrete masonry unit. Larger in size than a concrete brick.

Concrete masonry unit: Hollow or solid masonry unit, manufactured using low frequency, high amplitude vibration to consolidate concrete of stiff or extremely dry consistency.

Connector: A mechanical device for securing two or more pieces, parts or members together; includes anchors, wall ties and fasteners. May be either structural or nonstructural.

Connector, tie: A metal device used to join wythes of masonry in a multi-wythe wall or to attach a masonry veneer to its backing.

Control joint: A continuous unbonded masonry joint that is formed, sawed or tooled in a masonry structure to regulate the location and amount of cracking and separation resulting from dimensional changes of different parts of the structure, thereby avoiding the development of high stresses.

Coping: The materials or masonry units used to form the finished top of a wall, pier, chimney or pilaster to protect the masonry below from water penetration.

Core: (See "Cell.")

Course: A horizontal layer of masonry units in a wall or, much less commonly, curved over an arch.

Crack control: Methods used to control the extent, size and location of cracking in masonry including reinforcing steel, control joints and dimensional stability of masonry materials.

Curing: (1) The maintenance of proper conditions of moisture and temperature during initial set to develop a required strength and reduce shrinkage in products containing portland cement. (2) The initial time period during which cementitious materials gain strength.

Diaphragm: A roof or floor system designed to transmit lateral forces to shear walls or other lateral load resisting elements.

Dimension, actual: The measured size of a concrete masonry unit or assemblage.

Dimension, nominal: The specified dimension plus an allowance for mortar joints, typically $^3/_8$ in. (9.5 mm). Nominal dimensions are usually stated in whole numbers. Width (thickness) is given first, followed by height and then length.

Dowel: A metal reinforcing bar used to connect masonry to masonry or to concrete.

Drag strut: a structural member that transfers axial loads between shear resisting elements. Bond beams, top plates, joists, girders, and truss chords may be used as drag struts, provided connections at each end of the drag strut are capable of transferring loads.

Drip: A groove or slot cut beneath and slightly behind the forward edge of a projecting unit or element, such as a sill, lintel or coping, to cause rainwater to drip off and prevent it from penetrating the wall.

Drying shrinkage: The change in linear dimension of a concrete masonry wall or unit due to drying.

End wall: exterior wall of a building perpendicular to the roof ridge and parallel to the rigid frame bents of a metal building.

Face: (1) The surface of a wall or masonry unit. (2) The surface of a unit designed to be exposed in the finished masonry.

Face shell: The outer wall of a hollow concrete masonry unit.

Face shell mortar bedding: Hollow masonry unit construction where mortar is applied only to the horizontal surface of the unit face shells and the head joints to a depth equal to the thickness of the face shell. No mortar is applied to the unit cross webs. (See also "Full mortar bedding.")

Fastener: A device used to attach components to masonry, typically nonstructural in nature.

Fire resistance: A rating assigned to walls indicating the length of time a wall performs as a barrier to the passage of flame, hot gases and heat when subjected to a standardized fire and hose stream test. For masonry, fire resistance is most often determined based on the masonry's equivalent thickness and aggregate type.

Flange: The projecting edge of a structural member.

Flange brace: A brace from flange of column or spandrel to a girt, purlin, or masonry wall to provide lateral support and stability.

Flashing: A thin impervious material placed in mortar joints and through air spaces in masonry to prevent water penetration and to facilitate water drainage.

Footing: A structural element that transmits loads directly to the soil.

Full mortar bedding: Masonry construction where mortar is applied to the entire horizontal surface of the masonry unit and the head joints to a depth equal to the thickness of the face shell. (See also "Face shell mortar bedding.")

Gauge/Gage: (1) In metal products, a number designating a specific thickness of metal sheet, or diameter of wire, cable or fastener shank tabulated in a standardized series, each of which represents a decimal fraction of an inch (or millimeter). (2) Distance in inches (or millimeters) between adjacent lines of holes or fasteners.

Girt: A horizontal structural member that is attached to side wall or end wall columns and supports paneling.

Grout: (1) A plastic mixture of cementitious materials, aggregates, water, with or without admixtures initially produced to pouring consistency without segregation of the constituents during placement. (2) The hardened equivalent of such mixtures.

Grout, self-consolidating: Highly fluid and stable grout used in high lift and low lift grouting that does not require consolidation or reconsolidation.

Grouted masonry: (1) Masonry construction of hollow units where hollow cells are filled with grout, or multi-wythe construction in which the space between wythes is solidly filled with grout. (2) Masonry construction using solid masonry units where the interior joints and voids are filled with grout.

Grouting, high lift: The technique of grouting masonry in lifts for the full height of the wall.

Grouting, low lift: The technique of grouting as the wall is constructed, usually to scaffold or bond beam height, but not greater than 4 to 6 ft (1,219 to 1,829 mm),

Haunch: Also Knee. The deepened portion of a column or rafter, designed to accommodate the high stress where column and rafter intersect and connect.

Hair-Pin: Reinforcing bar used to help transfer anchor bolt shear (due to column thrust) to concrete floor mass. The "U" shaped hair-pin wraps around the anchor bolts inside the slab.

Height of wall: (1) The vertical distance from the foundation wall or other similar intermediate support to the top of the wall. (2) The vertical distance between intermediate supports.

Hollow masonry unit: A unit whose net cross-sectional area in any plane parallel to the bearing surface is less than 75 % of its gross cross-sectional area measured in the same plane.

Hook: Reinforcing steel which terminates in a bend, normally 90 degrees plus an extension of at least 12 bar diameters, to increase reinforcement development capability.

Intermediate Bay: A distance between two main frames within a building, other than end frames.

Joint: The surface at which two members join or abut. If they are held together by mortar, the mortar-filled volume is the joint.

Joint reinforcement: Steel wires placed in mortar bed joints (over the face shells in hollow masonry). Multi-wire joint reinforcement assemblies have cross wires welded between the longitudinal wires at regular intervals.

Lap: (1) The distance two bars overlap when forming a splice. (2) The distance one masonry unit extends over another.

Lap splice: The connection between reinforcing steel generated by overlapping the ends of the reinforcement.

Lateral support: The means of bracing structural members in the horizontal span by columns, buttresses, pilasters or cross walls, or in the vertical span by beams, spandrels, floors, foundations, or roofs.

Lintel: A beam placed or constructed over a wall opening to carry the superimposed load.

Lintel block: A U-shaped masonry unit, placed with the open side up to accommodate horizontal reinforcement and grout to form a continuous beam. Also called a solid bottom bond beam unit.

Loadbearing: (See “Wall, loadbearing.”)

Main or primary framing: Steel frames which support secondary framing members such as girts, purlins or eave struts.

Masonry: An assemblage of masonry units, joined with mortar, grout or other accepted methods.

Metal Building System: An integrated set of components and assemblies, including but not limited to frames that are built-up structural steel members, secondary members that are cold formed steel or steel joists and cladding components, specifically designed to support and transfer loads and provide a complete or partial building shell. These components and assemblies are manufactured in a manner that permits plant and or field inspection prior to assembly or erection.

Metric: The Systeme Internationale (SI), the standard international system of measurement. Hard metric refers to products or materials manufactured to metric-specified dimensions. Soft metric refers to products or materials manufactured to English specified dimensions, then converted into metric dimensions.

Modular coordination: The designation of masonry units, door and window frames, and other construction components that fit together during construction without customization.

Modular design: Construction with standardized units or dimensions for flexibility and variety in use.

Mortar: (1) A mixture of cementitious materials, fine aggregate water, with or without admixtures, used to construct unit masonry assemblages. (2) The hardened equivalent of such mixtures.

Mortar bed: A horizontal layer of mortar used to seat a masonry unit.

Mortar joint, bed: The horizontal layer of mortar between masonry units.

Mortar joint, head: The vertical mortar joint placed between masonry units within the wythe.

Mortar joint profile: The finished shape of the exposed portion of the mortar joint. Common profiles include:
Concave: Produced with a rounded jointer, this is the standard mortar joint unless otherwise specified. Recommended for exterior walls because it easily sheds water.
Raked: A joint where 1/4 to 1/2 in. (6.4 to 13 mm) is removed from the outside of the joint.
Struck: An approximately flush joint.

Nonloadbearing: (See "Wall, nonloadbearing.")

Panel: In a building, (1) a portion of a surface flush with or recessed from, or sunk below the surrounding area, sometimes set off by distinct molding or other decorative measure. (2) a usually flat and rectangular piece of construction material made to form part of a surface.

Pier: An isolated column of masonry or a bearing wall not bonded at the sides to associated masonry. For design, a vertical member whose horizontal dimension measured at right angles to its thickness is not less than three times its thickness nor greater than six times its thickness and whose height is less than five times its length.

Plain masonry: (See "Unreinforced masonry.")

Plaster: (See "Stucco.")

Project specifications: The written documents that specify project requirements in accordance with the service parameters and other specific criteria established by the owner or owner's agent.

Purlin: A horizontal structural member that supports roof covering.

Quality assurance: The administrative and procedural requirements established by the contract documents and by code to assure that constructed masonry is in compliance with the contract documents.

Reinforced masonry: (1) Masonry containing reinforcement in the mortar joints or grouted cores used to resist stresses. (2) Unit masonry in which reinforcement is embedded in such a manner that the component materials act together to resist applied forces.

Reinforcing steel: Steel embedded in masonry in such a manner that the two materials act together to resist forces.

Rigid frame: A clear span structure, characterized by tapered columns, tapered haunches and rafter beams.

Sash block: A block specially formed for the jamb of windows or doors, generally with a vertical slot to receive window frames, etc.

Shear wall segment: portion of a shear wall between openings extending between horizontal diaphragms and/or spandrels that can be considered to resist in-plane shear.

Shrinkage: The decrease in volume due to moisture loss, decrease in temperature or carbonation of a cementitious material.

Side wall: an exterior wall of a building parallel to the roof ridge and perpendicular to the rigid frame bents of a metal building.

Simply supported: A member structurally supported at top and bottom or both sides through a pin-type connection, which assumes no moment transfer.

Spandrel: A horizontal structural beam or girt that spans between two or more supports to resist vertical and or horizontal loads. In metal buildings, a common application is to support the lateral loads at the top of a concrete masonry wall.

Specified compressive strength of masonry, f'_m: Minimum masonry compressive strength required by contract documents, upon which the project design is based (expressed in terms of force per unit of net cross-sectional area).

Stirrup: Shear reinforcement in a flexural member.

Structural Steel Members: Load carrying members, may be hot rolled sections, cold formed shapes, or built-up shapes.

Stucco: A combination of cement and aggregate mixed with a suitable amount of water to form a plastic mixture that will adhere to a surface and preserve the texture imposed on it.

Thermal movement: Dimension change due to temperature change.

Tie: (See "Connector, tie.")

Tolerance: The specified allowance in variation from a specified size, location, or placement.

Tooling: Compressing and shaping the face of a mortar joint with a tool other than a trowel. See "Mortar joint profile" for definitions of common joints.

Unreinforced masonry: Masonry in which the tensile resistance of the masonry is taken into consideration and the resistance of reinforcement, if present, is neglected. Also called plain masonry.

Veneer, adhered: Masonry veneer secured to and supported by the backing through adhesion.

Veneer, anchored: Masonry veneer secured to and supported laterally by the backing through anchors and supported vertically by the foundation or other structural elements.

Veneer, masonry: A masonry wythe that provides the finish of a wall system and transfers out-of-plane loads directly to a backing, but is not considered to add load resisting capacity to the wall system.

Wainscot: Wall material, used in the lower portion of a wall, that is different than the material in the rest of the wall.

Wall, bonded: A masonry wall in which two or more wythes are bonded to act as a composite structural unit.

Wall, cavity: A multi-wythe noncomposite masonry wall with a continuous air space within the wall (with or without insulation), which is tied together with metal ties.

Wall, curtain: (1) A nonloadbearing wall between columns or piers. (2) A nonloadbearing exterior wall vertically supported only at its base, or having bearing support at prescribed vertical intervals. (3) An exterior nonloadbearing wall in skeleton frame construction. Such walls may be anchored to columns, spandrel beams or floors, but not necessarily built between columns.

Wall, foundation: A wall below the floor nearest grade serving as a support for a wall, pier, column or other structural part of a building and in turn supported by a footing.

Wall, loadbearing: Wall that supports vertical load in addition to its own weight. By code, a wall carrying vertical loads greater than 200 lb/ft (2.9 kN/m) in addition to its own weight.

Wall, multi-wythe: Wall composed of 2 or more masonry wythes.

Wall, nonloadbearing: A wall that supports no vertical load other than its own weight. By code, a wall carrying vertical loads less than 200 lb/ft (2.9 kN/m) in addition to its own weight.

Wall, panel: (1) An exterior nonloadbearing wall in skeleton frame construction, wholly supported at each story. (2) A nonloadbearing exterior masonry wall having bearing support at each story.

Wall, partition: An interior wall without structural function.

Wall, reinforced: (1) A masonry wall reinforced with steel embedded so that the two materials act together in resisting forces. (2) A wall containing reinforcement used to resist shear and tensile stresses.

Wall, shear: A wall, bearing or nonbearing, designed to resist lateral forces acting in the plane of the wall.

Wall, single wythe: A wall of one masonry unit thickness.
Wall tie: A metal connector that connects wythes of masonry.
Wall tie, veneer: A wall tie used to connect a facing veneer to the backing.
Web: The portion of a hollow concrete masonry unit connecting the face shells.
Weep: An opening left (or cut) in mortar joints or masonry face shells to allow moisture to exit the wall. Usually located immediately above flashing.
Workability: The ability of mortar or grout to be easily placed and spread.
Wythe: Each continuous vertical section of a wall, one masonry unit in thickness.

APPENDIX B

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    - *5-7A Floor and Roof Connections to Concrete Masonry Walls (2001)*

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5-10A Concrete Masonry Radial Walls (2006)
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6-2B R-Values and U-Values for Single Wythe Concrete Masonry Walls (2009)
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APPENDIX C

Design Examples Using Structural Masonry Design System Software

This section addresses the capabilities of Version 5.0 of *Structural Masonry Design System* software (ref. 15) in regard to designing the walls and shear segments for metal building systems. Additional information regarding the software can be found in TEK 14-17A *Software for the Structural Design of Concrete Masonry* (ref. 13). Following are several design examples demonstrating how the wall sections designed using the design aids within this manual can be designed using the wall software.

Example C2.4A Out-of Plane Flexure Design Using NCMA Structural Design Software – Using version 5.0 of the software, select "Design" from the menu at the top, then "Design Basis", then "Design Codes", then "2009 IBC Code", then "ASD with Alternative Basic Load Combinations". Select the "Concrete Masonry" tab, and use all of the default values (f'_m = 1500 psi). Under the "Reinforcement" tab, use the default "Grade 60 steel". For unit size, use the default values. Click "OK" and the design basis information will be displayed.

Now select "Design" from the pull down menu at the top again and this time select "Wall Design (out-of-plane loads)" or the icon with the same name and a new screen will appear with the "Design Data" tab on the top. Select "Compute using load data", "Reinforced Masonry Wall", "Interaction Diagram" and "Grout Spacing" (for "Family"). Use default values for other entries.

Select the "Construction Data" tab, enter H = 16 ft 0 in., select "Partial grout, running bond", and for "Reinforcement and grout spacing" leave the default of 8 in.

Under the "Load Data" tab, click the "Wind" tab and enter "20" psf for both "w_1" and "w_2", the others all leave zero. "h2" should be automatically filled in as "192" in. as the height of the wall from the 16 ft entered on the Construction Data tab above. If it isn't, go ahead and add it in. Also under the "Dead" tab enter for "P", "100" lbs/ft for the 2 ft of masonry extended above the spandrel. For "e" leave the default as "0".

Select "OK" and observe the interaction diagrams for No. 4 bars at various spacings. The largest spacing of reinforcement that works is 24 in. Hence, one solution would be No. 4 bars @ 24 in. However, a more economical choice might be a larger bar at a larger spacing.

Try No. 5 bars and view the interaction diagram. This is done by again selecting "Design" from the menu at the top and select "Edit Design" on the screen that appears. Change the reinforcement on the "Design Data" tab from #4 to #5. Then click "OK". It appears that a 40 in. spacing will work. Go back and click on "Design" from the menu at the top and select "Edit Design" on the screen that appears. The "Design Data" tab will be on top of the new screen that appears as described above. Select "None" under "Display results/Family". Then go to the "Construction Data" tab and select "40 in. on center" under "Reinforcement and grout spacing" and click OK which will bring up a new interaction diagram with the new values. The wall appears to work because all loading “dots” fall inside the contour (see Figure C.1).

Now to see the design calculations, go back and select “Design” from the menu at the top and select “Edit Design” on the screen that appears. On the “Design Data” tab and under Display Results”, select “Design Calculations”, click “OK” and notice that the wall does, in fact, work (See Figure C.2). The design is now complete. Note No. 5 bars @ 48 in. appears to fall right on the line on the interaction diagram but when displaying the design calculations, it indicates that it is just short, so No. 5s @ 48 in. does not work. The interaction diagram is preliminary only and for final results the design calculations should be used for the official results. Similarly, No. 6 bars @ 64 in. will also work as will No. 7 bars @ 80 in.

Notice that increasing the amount of grout to a solid grouted wall does very little to increase the capacity. The reason is that the compression area of the masonry falls within the face shell in this case so there is no difference in the way the wall behaves – the masonry compression is entirely within the face shell in both cases. The only difference is that the solid grouted wall weighs more and therefore has more axial compressive force at the critical section thereby giving it a slightly higher flexural capacity. The amount, however; is seldom enough to warrant the expense of the additional grout.

MASONRY | **DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY**
The National Concrete Masonry Association — Brick Industry Association — Western States Clay Product Association — International Code Council
Masonry 5.0 (Release 5.0.0)

Prjct: Concrete Masonry Walls	Name: No 5 Out-of-Plane Example
Topic: for Metal Buildings	Date: 11-18-2010
Page:	Chkd: DWG

Reinforced Wall Interaction Diagram
Using the 2009 IBC ASD Code with Alternative Basic Loads

Masonry Unit Size = 8 in. (Partially grouted)
Reinforcement Spacing = 40 in.
Fy = 60 ksi   fm' = 1500 psi
As = 0.31 sq.in.   x = 3.813 in.   H = 192 in. (16 ft)

Critical Sections Forces
⊕ Load combinations w/o stress increase
● Load combinations with stress increase

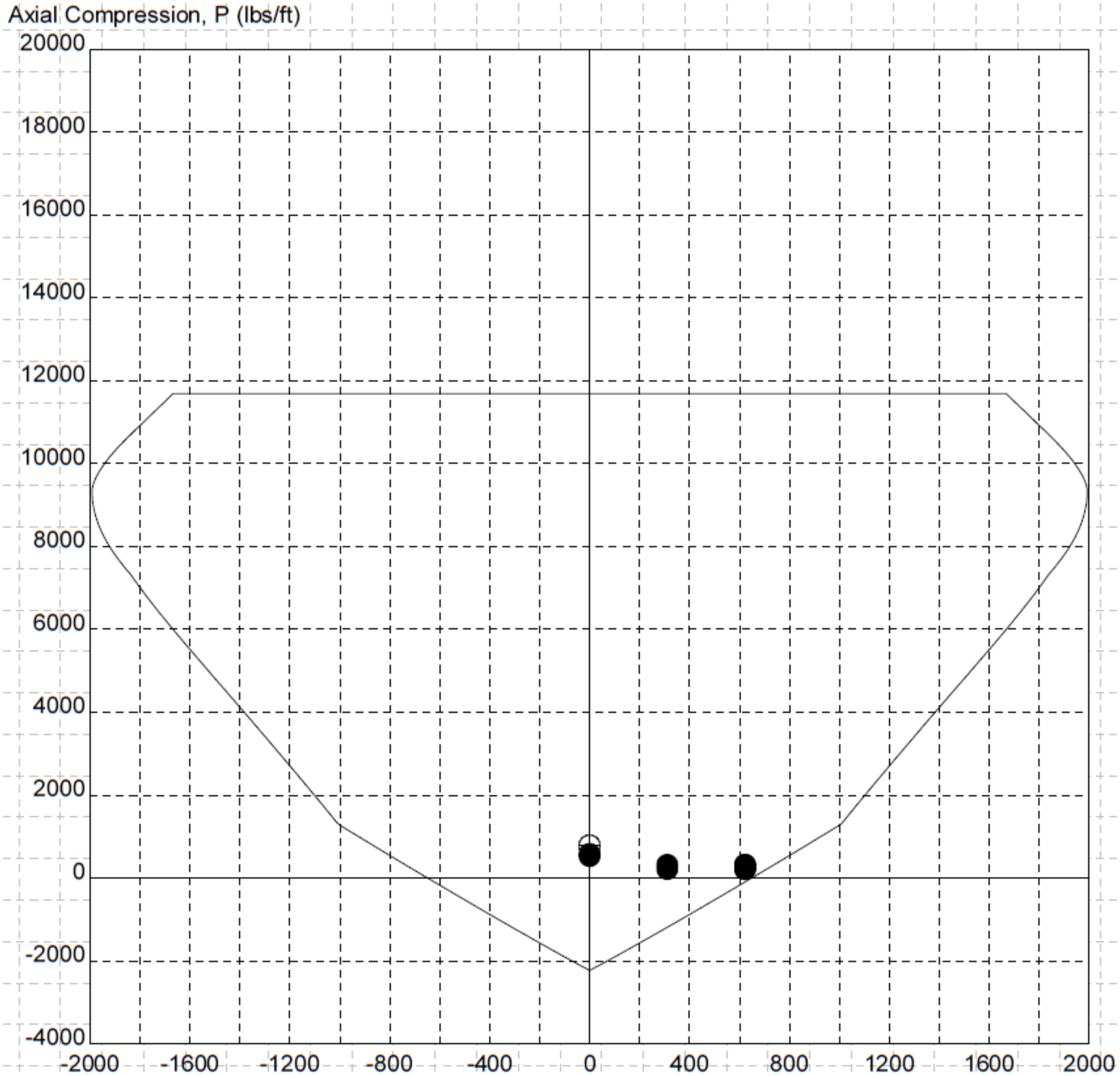


Figure C.1—Structural Masonry Design System Software Out-of-Plane Interaction Diagram

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY

The National Concrete Masonry Association — Brick Industry Association — Masonry 5.0
Western States Clay Product Association — International Code Council — (Release 5.0.0)

Prjct: Concrete Masonry Walls	Name: No 5 Out-of-Plane Example
Topic: for Metal Buildings	Date: 11-18-2010
Page:	Chkd: DWG

Design of a Reinforced Masonry Wall with Out-of-Plane Loads
Using the 2009 IBC ASD Code with Alternative Basic Loads

Material and Construction Data

8 in. CMU, Partial grout, running bond
Wall Weight = 42.6 psf
Mortar,
CMU Density = 115 pcf
f'_m = 1500 psi (Specified)
$E_m = 900f'_m$ = 1350000 psi

Wall Design Details

Thickness = 7.625 in.
Height = 192 in. (Simply Supported Wall, Effective height = H)
x = 3.813 in.
#5 Bars, Fy = 60000.0
Reinforcement Spacing = 40 in. On-Center

Wall Design Section Properties

A_o = 142.6 in^2 on design width
I_o = 1122 in^4 on design width
S_o = 294.4 in^3 on design width
R_o = 2.805 in on design width

Wall Average Section Properties

A_{avg} = 170.8 in^2 on design width
I_{avg} = 1184 in^4 on design width
R_{avg} = 2.633 in on design width

Wall Support: Simply Supported Wall

Specified Load Components

Load	P (lb)	e (in)	W1(psf)	W2 (psf)	L (lb/ft)	h1 (in)	h2 (in)
Dead	100	0	0	0	0	0	192
Live	0	0	0	0	0	0	192
Soil	0	0	0	0	0	0	192
Fluid	0	0	0	0	0	0	192
Wind	0	0	20	20	0	0	192
Seismic	0	0	0	0	0	0	192
Roof	0	0	0	0	0	0	192
Rain	0	0	0	0	0	0	192
Snow	0	0	0	0	0	0	192

Figure C.2—Structural Masonry Design System Software Design Calculations p. 1

MASONRY

DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY

The National Concrete Masonry Association Brick Industry Association Masonry 5.0
Western States Clay Product Association International Code Council (Release 5.0.0)

Prjct: Concrete Masonry Walls
Topic: for Metal Buildings
Page:

Name: No 5 Out-of-Plane Example
Date: 11-18-2010
Chkd: DWG

w = 1.3

Controlling Load Cases

Section Forces with Controlling Flexure and Axial Load--2D/3 + L + F + H + wW

$x/H = 0.520$ from bottom of wall

$V = -8.32$ lb/ft

$M_L = 9968.03$ lb-in./ft

$P = 284.9$ lb/ft at $e_n = 0$ in

$M_T = M_L + Pe_n = 9968.03$ lb-in/ft

Moment Capacity = 11333.3 lb-in/ft (944.445 lb-ft/ft) at this axial load

Shear Capacity = 1104.59 lb/ft

The wall is adequate for these critical section forces.

Section Forces with Controlling Shearing Force--D + L + F + H + wW

$x/H = 0.000$ from bottom of wall

$V = 208$ lb/ft

$M_L = 0$ lb-in./ft

$P = 781.6$ lb/ft at $e_n = 0$ in

$M_T = M_L + Pe_n = 0$ lb-in/ft

Moment Capacity = 12981.7 lb-in/ft at this axial load

Shear Capacity = 1104.59 lb/ft

The wall is adequate for these critical section forces.

These were found to be load cases that controlled the design.
The flexural, shear and axial forces shown are those occurring
at the critical section for the case controlled by flexure and
at the critical section for the case controlled by shear.

The following design calculations are for the section with controlling bending moment

Section Design Forces Used

$V = -8.32$ lb/ft (Computed from Loads)

$M_L = 9968.03$ lb-in./ft (Computed from Loads)

$P = 284.9$ lb/ft at $e = 0$ in (Computed from Loads)

Computed Design Values

Note: 1/3 stress increase was used

Effective Width = 40 in.

Web Width = 8.313 in. on effective width

Allowable Shearing Force = 1105 lb/ft

Figure C.2 (cont.)—Structural Masonry Design System Software Design Calculations p. 2

MASONRY	DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY The National Concrete Masonry Association · Brick Industry Association · Masonry 5.0 Western States Clay Product Association · International Code Council · (Release 5.0.0)
Prjct: Concrete Masonry Walls Topic: for Metal Buildings Page:	Name: No 5 Out-of-Plane Example Date: 11-18-2010 Chkd: DWG

The wall is adequate in shear

Required A_s = 0.2675 in^2 each reinforced cell (0.08025 in^2/ft) OK
d = 3.813 in.
n = 21.48
$k_{balanced}$ = 0.3092
$j_{balanced}$ = 0.8969
k = 0.2505
j = 0.9165
P_{max} (Compression) = 5.196e+004 lbs (1.559e+004 lbs/ft) OK
P_{max} (Tension) = 9920 lbs (2976 lbs/ft) OK

The wall has adequate capacity.

Development and Splice Lengths for Longitudinal Reinforcement
K = 3.1250 in.
Required Development Length: l_d = 25.17 in.
Required Lap Splice Length: = 45.00 in.
Some codes may require epoxy-coated reinforcement to have longer development and splice lengths.

Figure C.2 (cont.)—Structural Masonry Design System Software Design Calculations p. 3

Example C2.4B Shear Wall Design Using Masonry Structural Design System Software – Using version 5.0 of the software, select "Design" from the menu at the top, then "Design Basis", then "Design Code", then "2009 IBC Code", then "ASD with Alternative Basic Load Combinations". Select the "Concrete Masonry" tab, and use all of the default values (f'_m = 1500 psi). Under the "Reinforcement" tab, use the default "Grade 60 steel". For unit size, use the default values. Click "OK" and the design basis information will be displayed.

Now select "Design" from the pull down menu at the top again and this time select "Wall Design (in-plane loads)" or the icon with the same name and a new input screen with the "Design Data" tab will appear on the top. Select "Compute Loads", "Reinforced Masonry Wall", "Normal Shear Wall" (i.e. not a special or intermediate reinforced shear wall), and "Interaction Diagram". Use default values for other entries.

Select the Construction Data tab enter H = 16 ft - 0 in., L = 8 ft - 0 in, "Partial grout, running bond", and for "Reinforcement and grout spacing" use the default of "8 in on center". Under the "Reinforcement" tab, do not use an end-zone (enter "0" cells) and select No. 3 bars (default) in the middle zone. Under the "Load Data" tab, add the weight of the 2 ft of masonry above the shear wall under the "Dead" tab determined as follows: (2 ft x 8 ft x 50 psf)/ 1000 lbs/kip = 0.8 kips). Click on the "Wind" tab and enter V = 6 k. The shear force is not increased by the factor of 1.3 because the software does this internally. All other values are zero.

Click "OK" and a message indicates the wall is not adequate. Click "OK" on the pop-up screen and the interaction diagram will appear with four dots falling outside the interaction diagram. Use the "Edit data" icon and return to the "Reinforcement" tab, and increase the "Middle zone" bar size to No. 6. Click on the "Construction Data" tab and for "Reinforcement and grout spacing" select "32 in. on center" (this would be four No. 6 bars in the 8 ft long wall). Click "OK", and observe that the interaction diagram shows a design stronger than is needed. At this point the bar size can be reduced or the spacing increased. Go back to "Edit data", select the "Construction data" tab and increase the bar spacing to 48 in. (three bars in the 8 ft long wall) and select "OK". This also works. If you select any bar spacing from 48 to 80 in., notice that the interaction diagram does not change. The reason for that is that in all cases between 48 to 80 in., three bars are needed in the 8 ft wall section in order not to exceed the reinforcement spacing listed. Because the applied load dots fall inside the interaction diagram (barely), the wall is OK (see Figure C.3).

Return to the "Design Data" tab and under "Display results" select "Design Calculations", leave the default "Section with critical flexural force" and click "OK". Observe that the input data is what is intended. Note that the moment capacity (1497.6 k-in.) exceeds the design moment for the controlling case (1631.65 k-in.) for the design axial load (see highlight in Figure C.4). The design is OK for combined flexure and axial load.

Now return to the "Design Data" tab and select "section with critical shear force." Click "OK" and observe that f_v = 0.0111 ksi and F_v = 0.04667 ksi for no shear reinforcement (See highlight in Figure C.5). Hence the design is adequate for shear without shear reinforcement. If a No. 6 bar is used, then maximum spacing of 48 inches (three vertical bars) is required for this section.

Note: a smaller bar diameter may be preferable to facilitate easier handling by the mason contractor. If a No. 5 bar is used, then a maximum spacing of 24 in. is required for the section (5 vertical bars).

Also note that in Example 2.4B using the design charts of this manual resulted in 1 more No. 6 bar required for the partially grouted 8 ft shear wall segment. The reason for this is that the design charts are more conservative and encompass a range of conditions while the software design is for the exact condition.

The software permits the design of **fully grouted walls** with little added effort. Return to the "Construction Data" tab and select "Full grout, running bond." Under the "Design Data" tab, select "Design for critical flexural force" and display "Interaction diagram." Also because the 2 ft section above the spandrel is also to be solidly grouted, increase the dead load "P" to "1.28 kips": (2 ft. x 8 ft. x 80 psf / 1,000 lbs/kip). Click "OK" and observe that the wall with No. 6 @ 88 in. works. A spacing of 88 in. is the spacing with reinforcement in the end cells of a 96 in. shear segment. Looking at the interaction diagram, all loading cases (solid black dots) fall on or inside the interaction diagram. To confirm that this design works, use edit data, return to "Design Data", and select "Design calculations". Click "OK", and observe that the wall does work. Return to "Design Data", and display "Design for critical shear," display "Design Calculations" and click "OK". Note that f_v = 0.0111 ksi and F_v = 0.04667 ksi for no shear reinforcement (same as fully grouted), and no shear reinforcement is needed. Note that this reinforcement selection does not work in the partially grouted construction. The added weight of the solid grouted wall makes just enough difference for it to work.

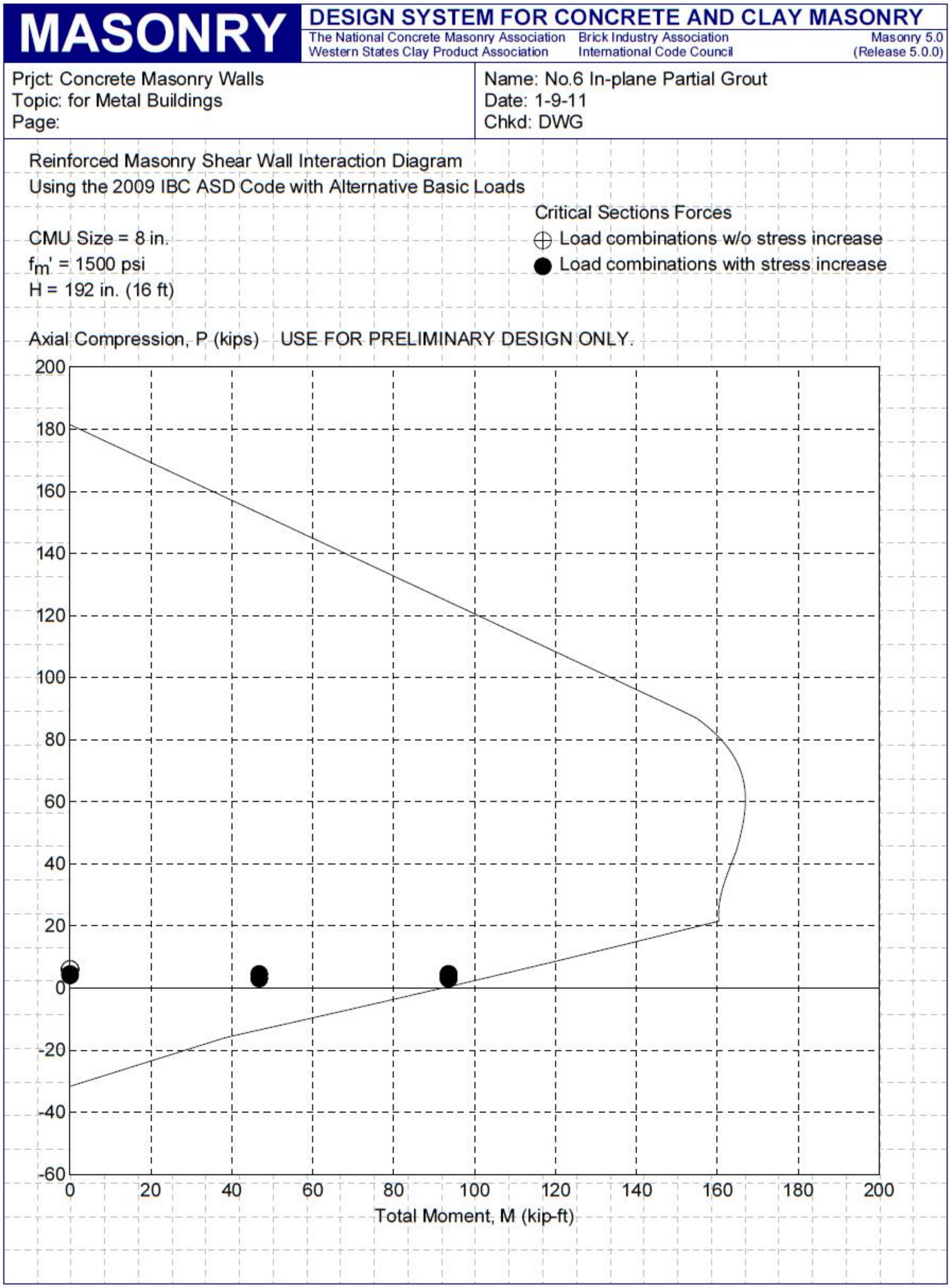


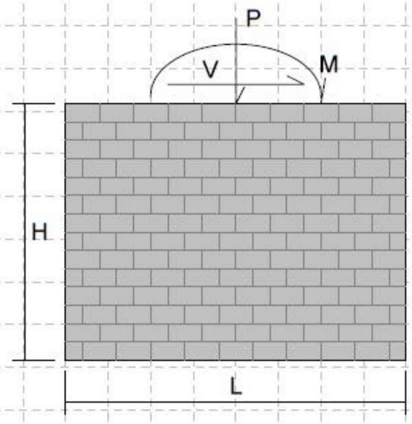
Figure C.3—Structural Masonry Design System Software In-Plane Partially Grouted Interaction Diagram

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY
The National Concrete Masonry Association Brick Industry Association Masonry 5.0
Western States Clay Product Association International Code Council (Release 5.0.0)

Prjct: Concrete Masonry Walls	Name: No.6 In-plane Par Gr Flexure
Topic: for Metal Buildings	Date: 1-9-11
Page:	Chkd: DWG

Working Stress Design of a Reinforced Concrete Masonry Shearwall
Using the 2009 IBC ASD Code with Alternative Basic Loads

Material and Construction Data
8 in. units, Partial grout, running bond
Wall Weight = 41.03 psf
Mortar,
Masonry Density = 115 pcf
f'_m = 1500 psi (Specified)
$E_m = 900f'_m$ = 1350000 psi



Shear Wall Design Details
Thickness = 7.625 in.
Height = 192 in. (16 ft)
Length = 96 in. (8 ft)
x = 3.813 in.
Endzone Length: S_o = 0 in = 0 ft (#3 Bars)
Middlezone Grouted Cell Spacing: 48 in OC
Middlezone Steel Spacing: 48 in OC (#6 Bars)
This shear wall has an aspect ratio of 2.00 and may be a deep beam following the provisions of ACI 318.
A_o = 363 sq.in.
I_o = 3.431e+005 in^4
R_o = 2.662 in.

Location of Reinforcing Bars

Bar	Ri	RCi	As
1	92	44	0.44
2	48	0	0.44
3	4	-44	0.44

The Specified Loads Acting

	P (k)	M (k-in)	V (k)
Dead	0.8	0	0
Live	0	0	0
Soil	0	0	0
Fluid	0	0	0
Wind	0	0	6
Seismic	0	0	0
Roof	0	0	0
Rain	0	0	0
Snow	0	0	0

Figure C.4—Structural Masonry Design System Software
In-Plane Partial Grouted Calculations - Section with Critical Flexure p. 1

MASONRY	DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY The National Concrete Masonry Association · Brick Industry Association · Western States Clay Product Association · International Code Council Masonry 5.0 (Release 5.0.0)
Prjct: Concrete Masonry Walls Topic: for Metal Buildings Page:	Name: No.6 In-plane Par Gr Flexure Date: 1-9-11 Chkd: DWG

w = 1.3

Controlling load case for bending moment: 2D/3 + L + F + H + wW
Controlling x/H ratio for bending moment: 0
M = 1497.6 kip-in (124.8 kip-ft)
V = 7.8 kips
P = 4.03658 kips
Controlling bending moment capacity: 1631.63 kip-in (135.97 kip-ft)

Controlling load case for shearing force: D + L + F + H + wW
Controlling x/H ratio for shearing force: 0
M = 1497.6 kip-in (124.8 kip-ft)
V = 7.8 kips
P = 6.05184 kips

The following design calculations are for the section with controlling bending moment

Section Design Forces Used
V = 7.8 kips (Computed from Loads)
M = 1497.6 kip-in (Computed from Loads)
P = 4.037 kips (Computed from Loads)

Computed Design Values
Note: 1/3 stress increase was used

Wall Flexural Design Data
Maximum P = 133.3 kips (MSJC 2.3.3.2)
M = 1632 kip-in (136 kip-ft) for Design P

Wall Shear Design Data
Design as a normal shear wall.
f_v = 0.01112 ksi (MSJC 2.3.5.2.1)
M/Vd = 1498 / (7.8 * 92) = 2.087
F_v = 0.04667 ksi without reinforcement (MSJC 2.3.5.2.2)
F_v = 0.07746 ksi with reinforcement (MSJC 2.3.5.2.3)
Av/s = 7.8 / (2.4e+004 * 92) = 0 (MSJC 2.3.5.3)

Development and Splice Lengths for EndZone Longitudinal Reinforcement
K = 1.8750 in.
Required Development Length: l_d = 15.10 in.
Required Lap Splice Length: = 27.00 in.

Figure C.4 (cont.)—Structural Masonry Design System Software In-Plane Partial Grouted Calculations - Section with Critical Flexure p. 2

MASONRY

DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY

The National Concrete Masonry Association | Brick Industry Association | Masonry 5.0
Western States Clay Product Association | International Code Council | (Release 5.0.0)

Prjct: Concrete Masonry Walls	Name: No.6 In-plane Par Gr Flexure
Topic: for Metal Buildings	Date: 1-9-11
Page:	Chkd: DWG

Development and Splice Lengths for MidZone Longitudinal Reinforcement

K = 3.4375 in.

Required Development Length: l_d = 42.84 in.

Required Lap Splice Length: = 54.00 in.

Some codes may require epoxy-coated reinforcement to have longer development and splice lengths.

Figure C.4 (cont.)—Structural Masonry Design System Software In-Plane Partial Grouted Calculations - Section with Critical Flexure p. 3

MASONRY | **DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY**

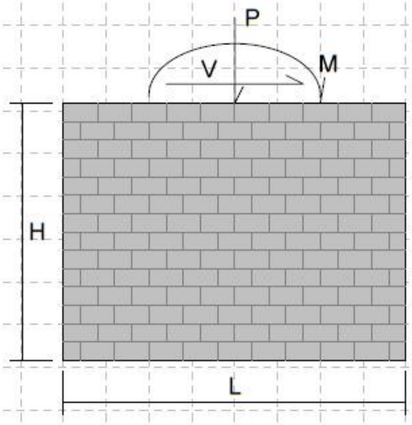
The National Concrete Masonry Association Brick Industry Association Masonry 5.0
Western States Clay Product Association International Code Council (Release 5.0.0)

Prjct: Concrete Masonry Walls	Name: No.6 In-plane Par Gr Shear
Topic: for Metal Buildings	Date: 1-9-11
Page:	Chkd: DWG

Working Stress Design of a Reinforced Concrete Masonry Shearwall
Using the 2009 IBC ASD Code with Alternative Basic Loads

Material and Construction Data

8 in. units, Partial grout, running bond
Wall Weight = 41.03 psf
Mortar,
Masonry Density = 115 pcf
f'_m = 1500 psi (Specified)
$E_m = 900f'_m$ = 1350000 psi



Shear Wall Design Details

Thickness = 7.625 in.
Height = 192 in. (16 ft)
Length = 96 in. (8 ft)
x = 3.813 in.
Endzone Length: S_o = 0 in = 0 ft (#3 Bars)
Middlezone Grouted Cell Spacing: 48 in OC
Middlezone Steel Spacing: 48 in OC (#6 Bars)
This shear wall has an aspect ratio of 2.00 and may be a deep beam following the provisions of ACI 318.
A_o = 363 sq.in.
I_o = 3.431e+005 in^4
R_o = 2.662 in.

Location of Reinforcing Bars

Bar	Ri	RCi	As
1	92	44	0.44
2	48	0	0.44
3	4	-44	0.44

The Specified Loads Acting

	P (k)	M (k-in)	V (k)
Dead	0.8	0	0
Live	0	0	0
Soil	0	0	0
Fluid	0	0	0
Wind	0	0	6
Seismic	0	0	0
Roof	0	0	0
Rain	0	0	0
Snow	0	0	0

Figure C.5—Structural Masonry Design System Software
In-Plane Partial Grouted Calculations - Section with Critical Shear p. 1

MASONRY	DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY The National Concrete Masonry Association · Brick Industry Association · Western States Clay Product Association · International Code Council · Masonry 5.0 (Release 5.0.0)
Prjct: Concrete Masonry Walls Topic: for Metal Buildings Page:	Name: No.6 In-plane Par Gr Shear Date: 1-9-11 Chkd: DWG

w = 1.3

Controlling load case for bending moment: 2D/3 + L + F + H + wW
Controlling x/H ratio for bending moment: 0
M = 1497.6 kip-in (124.8 kip-ft)
V = 7.8 kips
P = 4.03658 kips
Controlling bending moment capacity: 1631.63 kip-in (135.97 kip-ft)

Controlling load case for shearing force: D + L + F + H + wW
Controlling x/H ratio for shearing force: 0
M = 1497.6 kip-in (124.8 kip-ft)
V = 7.8 kips
P = 6.05184 kips

The following design calculations are for the section with controlling shearing force

Section Design Forces Used
V = 7.8 kips (Computed from Loads)
M = 1497.6 kip-in (Computed from Loads)
P = 6.052 kips (Computed from Loads)

Computed Design Values
Note: 1/3 stress increase was used

Wall Flexural Design Data
Maximum P = 133.3 kips (MSJC 2.3.3.2)
M = 1710 kip-in (142.5 kip-ft) for Design P

Wall Shear Design Data
Design as a normal shear wall.
f_v = 0.01112 ksi (MSJC 2.3.5.2.1)
M/Vd = 1498 / (7.8 * 92) = 2.087
F_v = 0.04667 ksi without reinforcement (MSJC 2.3.5.2.2)
F_v = 0.07746 ksi with reinforcement (MSJC 2.3.5.2.3)
Av/s = 7.8 / (2.4e+004 * 92) = 0 (MSJC 2.3.5.3)

Development and Splice Lengths for EndZone Longitudinal Reinforcement
K = 1.8750 in.
Required Development Length: l_d = 15.10 in.
Required Lap Splice Length: = 27.00 in.

Figure C.5 (cont.)—Structural Masonry Design System Software In-Plane Partial Grouted Calculations - Section with Critical Shear p. 2

MASONRY	DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY The National Concrete Masonry Association Brick Industry Association Masonry 5.0 Western States Clay Product Association International Code Council (Release 5.0.0)
Prjct: Concrete Masonry Walls Topic: for Metal Buildings Page:	Name: No.6 In-plane Par Gr Shear Date: 1-9-11 Chkd: DWG

Development and Splice Lengths for MidZone Longitudinal Reinforcement

K = 3.4375 in.

Required Development Length: l_d = 42.84 in.

Required Lap Splice Length: = 54.00 in.

Some codes may require epoxy-coated reinforcement to have longer development and splice lengths.

Figure C.5 (cont.)—Structural Masonry Design System Software In-Plane Partial Grouted Calculations - Section with Critical Shear p. 3

Lap-splice Design

Note that the software also displays a lap-splice length of 45 in. for the No. 5 reinforcement which is the same value obtained using Table 2.4B.

Lintel Design Example Using NCMA Structural Design Software –Using version 5.0 of the software, under "Design Basis", select "Design Code", then "2009 IBC Code, Allowable Stress Design". (Note that most lintels are designed for gravity load only, not wind or earthquake, so the 1/3 stress increase associated with alternative basic load combinations in the IBC Code does not apply. However, there is no need to change the software if it has already been set for the alternative basic load combinations. The software will automatically check all load combinations including those with no stress increase. If no wind or seismic loads are included under the loads tab, the critical load combination will obviously be without the 1/3 stress increase).

Select the "Concrete Masonry" tab, and use all of the default values (f'_m = 1500 psi). Under the "Reinforcement" tab, select the default "Grade 60" reinforcing steel. For unit size, select the default values. Click OK and the design basis information will be displayed.

Now select the "lintel" icon and a new screen with the "Design Data" tab on top will appear. Select "Compute Loads", "Reinforced Masonry Lintel" and "Not exposed to weather". Use default cover and x = 3 in. (press the F1 key for help on regarding these dimensions or drawing in Figure C.6). Use default values for other entries under this tab.

Select the "Construction Data" tab and select "8 in. wall", "running bond, ungrouted". A wall weight of 33 psf is displayed. For lintel construction preferences enter "2" for "Minimum courses" and "2" for "Maximum courses". Enter dimensions H = 4 ft. 0 in., L = 15 ft. 4 in, B = 8 in.

Under "Load Data", the only load is self weight of the wall and the lintel, so no entry is needed as these are calculated automatically. Select "OK" and observe if the output reflects the intended input values. If the default No. 4 bars are used, two bars are required.

Because we want only one bar, using the "edit data" icon, return to the "Design Data" tab and change the bar size to "No. 5". Select "OK" and observe that only one bar is needed (see Figure C.6). This matches the solution presented in Example 2.4C using the design aids of this manual.

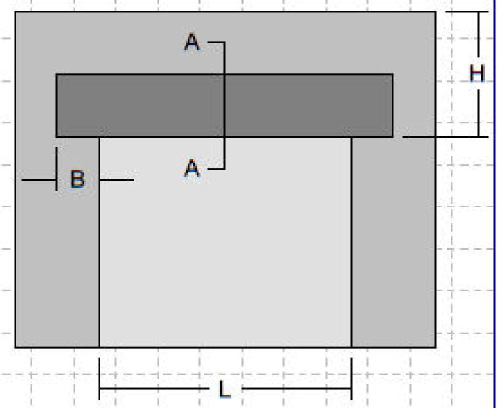
Note also that design shear (1590 lb) is less than capacity without shear reinforcement (3636 lb), so shear is adequate without stirrups. Had the shear capacity not been adequate, the software would have automatically designed and printed out the shear reinforcement requirements. Also notice that the working stresses used were 24,000 psi for f_s and 500 psi for f_b which are without the 1/3 stress increase. The deflection of this beam is 0.26 in. which is less than l/600 = (16 ft x 12 in./ft)/600 = 0.32 in. Therefore this lintel can be used to support unreinforced masonry per the MSJC (ref. 1).

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY

The National Concrete Masonry Association Brick Industry Association Masonry 5.0
Western States Clay Product Association International Code Council (Release 5.0.0)

Prjct: Concrete Masonry Walls	Name: Lintel Design Example
Topic: for Metal Buildings	Date: 11-20-10
Page:	Chkd: DWG

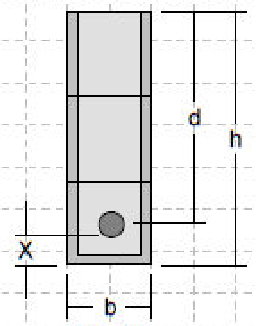
Working Stress Design of a Concrete Masonry Lintel
Using the 2009 IBC ASD Code



Specified Wall Construction Data
- 8 in Masonry Units
- Running Bond Construction
- Ungrouted Construction
- Clear Span = 15 ft - 4.00 in
- Bearing Length = 8 in
- Height Above Opening = 4 ft - 0.00 in
- Minimum Courses Considered = 2
- Maximum Courses Considered = 2
- Maximum Number of Bars Considered = Must Fit

Specified Load Data

Load	Uniform (lb/ft)	Joist (lb)	Point A (lb)	PointB (lb)	Point C (lb)
Dead	0	0	0	0	0
Live	0	0	0	0	0
Roof	0	0	0	0	0
Snow	0	0	0	0	0
Rain	0	0	0	0	0
So	0	0	0	0	0
Sl		0			
Sw	0				



Sect AA

The lintel is considered to be a simply-supported beam when calculating the design forces using these specified loads.

Parameters for Load Calculations
- L_e = 192 in
- Lintel W = 109.22 lb/ft
- Wall W = 89.4899 lb/ft
- Arching was not used
- Load combination D controlled moment
- Load combination D controlled shear

Design Forces Used
- V = 1590 lbs (Computed from Loads)
- M = 76304.5 lb-in (Computed from Loads)

Material Strength Properties
- f_y = 60000 psi (Specified)
- Mortar: ,

Figure C.6—Structural Masonry Design System Software Lintel Design Printout - p. 1

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY
The National Concrete Masonry Association Brick Industry Association Masonry 5.0
Western States Clay Product Association International Code Council (Release 5.0.0)

Prjct: Concrete Masonry Walls Topic: for Metal Buildings Page:	Name: Lintel Design Example Date: 11-20-10 Chkd: DWG

Unit Compressive Strength = 1900 psi
f'_m = 1500 psi (From Tables)
$E_m = 900f'_m$ = 1350000 psi

Working Stresses Used
f_s = 24000 psi
f_b = 500 psi

Required Cross-Sectional Dimensions
b = 7.625 in. (Specified)
x = 3 in. (Specified)
h = 15.63 in. (2 courses used)
d = 12.31 in. (Computed for bars and dimensions specified)

Check Shear Capacity
Maximum Permitted V = 3636 lbs without stirrups
OK -- Shear capacity is acceptable

Compute Required Steel Area
Computing balanced conditions
K_b = 1 / [1 + 24000/[21.4815(500)]] = 0.3092
J_b = 1 - 0.3092/3 = 0.8969
d_b = sqrt[(2(76304.5))/(500(0.3092)0.8969(7.625))] = 12.01 in

Section is under-reinforced -- F_s controls

Conditions for this section
K = 0.3027
J = 1 - 0.3027/3 = 0.8991
A_s = 0.287 sq.in (Required)

Use 1 #5 Bars (A_s Provided = 0.31 sq.in)
Final K = 0.3123
M_{max} = 80862.8 lb-in
I_{cr} = 621.939 in^4
M_{cr} = 37231.4 lb-in
I_{eff} = 831.268 in^4
Maximum deflection = -0.2611 in.
Maximum allowable deflection at L_e/240 = 0.8 in.
Maximum allowable deflection at L_e/360 = 0.5333 in.
Maximum allowable deflection at L_e/600 = 0.32 in.
Maximum allowable deflection = 0.32 in. when supporting unreinforced masonry (MSJC 1.13.3.1)

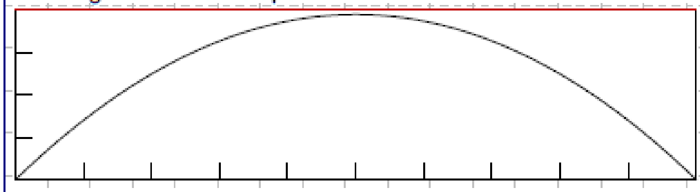
Figure C.6 (cont.)—Structural Masonry Design System Software Lintel Design Printout p. 2

MASONRY DESIGN SYSTEM FOR CONCRETE AND CLAY MASONRY

The National Concrete Masonry Association Brick Industry Association Masonry 5.0
Western States Clay Product Association International Code Council (Release 5.0.0)

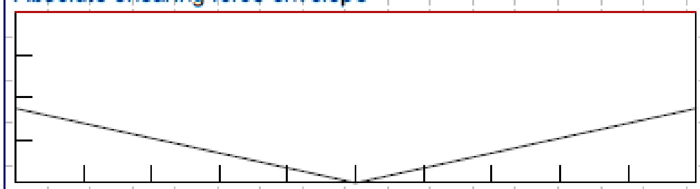
Prjct: Concrete Masonry Walls	Name: Lintel Design Example
Topic: for Metal Buildings	Date: 11-20-10
Page:	Chkd: DWG

Bending moment envelope



Bending moment capacity
with the reinforcement provided

Absolute shearing force envelope



Maximum shearing force
without shear reinforcement

Development and Splice Lengths for Longitudinal Reinforcement

K = 2.6875 in.

Required Development Length: l_d = 29.27 in.

Required Lap Splice Length: = 45.00 in.

Some codes may require epoxy-coated reinforcement to have longer development and splice lengths.

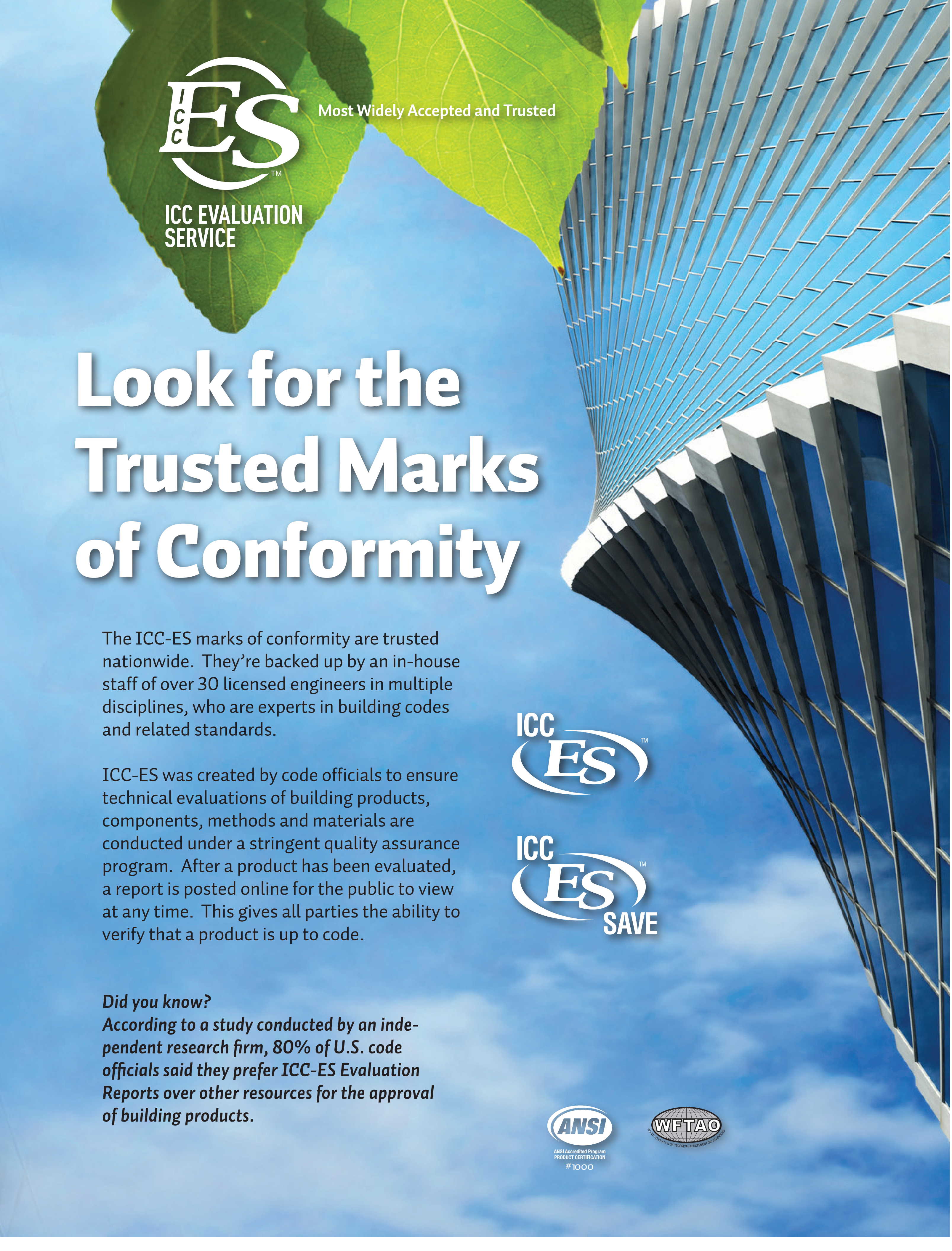
Figure C.6 (cont.)—Structural Masonry Design System Software Lintel Design Printout p. 3

APPENDIX D

Metric Conversions

Quantity	Inch-Pound Units	Metric Units	Multiply Inch-Pound Units by:
Length	ft in.	m mm	0.3048* 25.4*
Area	ft^2 $in.^2$	m^2 mm^2	0.09290304* 645.16*
Volume	ft^3 $in.^3$	m^3 mm^3	0.0283168 16,387.064*
Mass	lb	kg	0.4535924
Density	lb/ft^3	kg/m^3	16.01846
Force	lb k	N kN	4.448222 4.448222
Force/Unit Length	lb/ft lb/in.	N/m N/mm	14.593904 0.1751269
Force/Unit Area	psf	Pa	47.88026
	psf	kPa	0.04788026
	psi	kPa	6.89476
	psi	MPa	0.00689476
	ksi	MPa	6.89476
Mass/Unit Area	psf	kg/m^2	4.882428

*Denotes exact conversion.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Code officials can accept IAS accreditation as evidence that the inspection programs of metal building manufacturers are in compliance with requirements in Chapter 17 of the *International Building Code®* (IBC®).

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- Personnel requirements
- Product traceability
- Process control
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- Technical standards

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(continued)

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- Working with an organization you can trust and one that shares your commitment to code compliance

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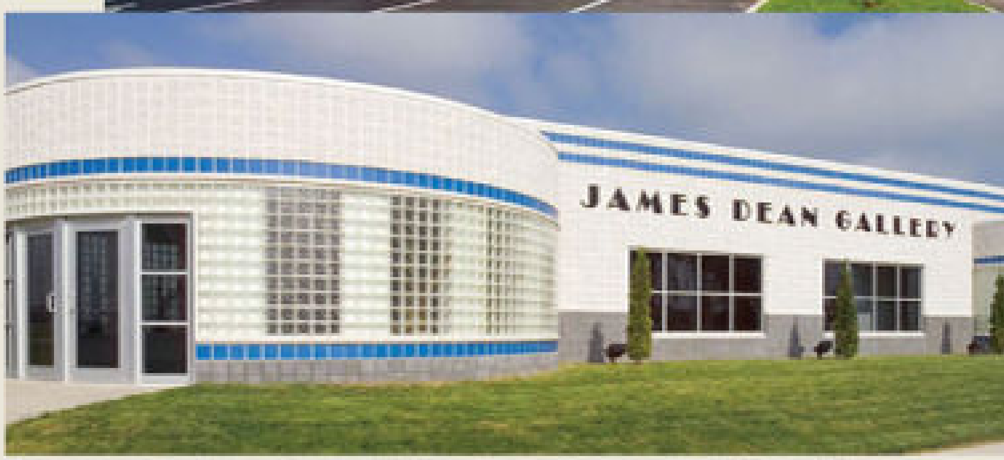

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