SEISMIC DESIGN OF MASONRY STRUCTURES

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Topic 12 deals with the seismic design of masonry structures.

In this first slide, we see examples of different applications of masonry : on the left, a low-rise bearing-wall building of reinforced masonry; in the center, a high-rise bearing-wall building of reinforced masonry; and on the right, stone and clay unit veneer over a frame structure.

Note that this topic, while complete, does <u>not</u> specifically utilize the examples in Chapter 9 of the FEMA 451, *NEHRP Recommended Provisions: Design Examples*. The instructor should review Chapter 9 of FEMA 451 to determine if additional materials should be incorporated into this topic. An individual using these training materials for independent study also should read that chapter carefully.

NEHRP Recommended Provisions Masonry Design

- Context in the ***NEHRP Recommended Provisions***
- Masonry behavior
- Reference standards
- Seismic resisting systems
- Component design
- Quality assurance
- Summary

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This is a list of topics covered in this review module on masonry design according to the *NEHRP Recommended Provisions*, which is developed for the Federal Emergency Management Agency (FEMA) by the Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences (NIBS).

Objectives of Module

- **Basics of masonry behavior**
- **Basics of masonry specification**
- **The MSJC code and specification and their relationship to the *NEHRP Recommended Provisions* documents**
- **Earthquake design of masonry structures and components using the 2005 MSJC code and specification**
- **Example of masonry shear wall design**

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Because many in the intended audience may not have studied masonry recently, the module begins with a review of the basic components of masonry and of the basic behavior of wall-type structures. It then addresses the specification of masonry units, mortar, grout, and accessory materials. It continues with the rudiments of the mechanical behavior of masonry. It then reviews the Masonry Standards Joint Committee (MSJC) Code and Specification, which is the fundamental technical resource behind the *NEHRP Recommended Provisions*. It concludes with the design and detailing of a reinforced masonry shear wall.

Context in the *NEHRP Recommended Provisions*

- **Design seismic loads**
  - **Load combinations** **Chap. 5**
  - **Loads on structures** **Chap. 5**
  - **Loads on components & attachments** **Chap. 6**
- **Design resistances** **Chap. 11**
  - **Strength design (mostly references the 2002 MSJC)**

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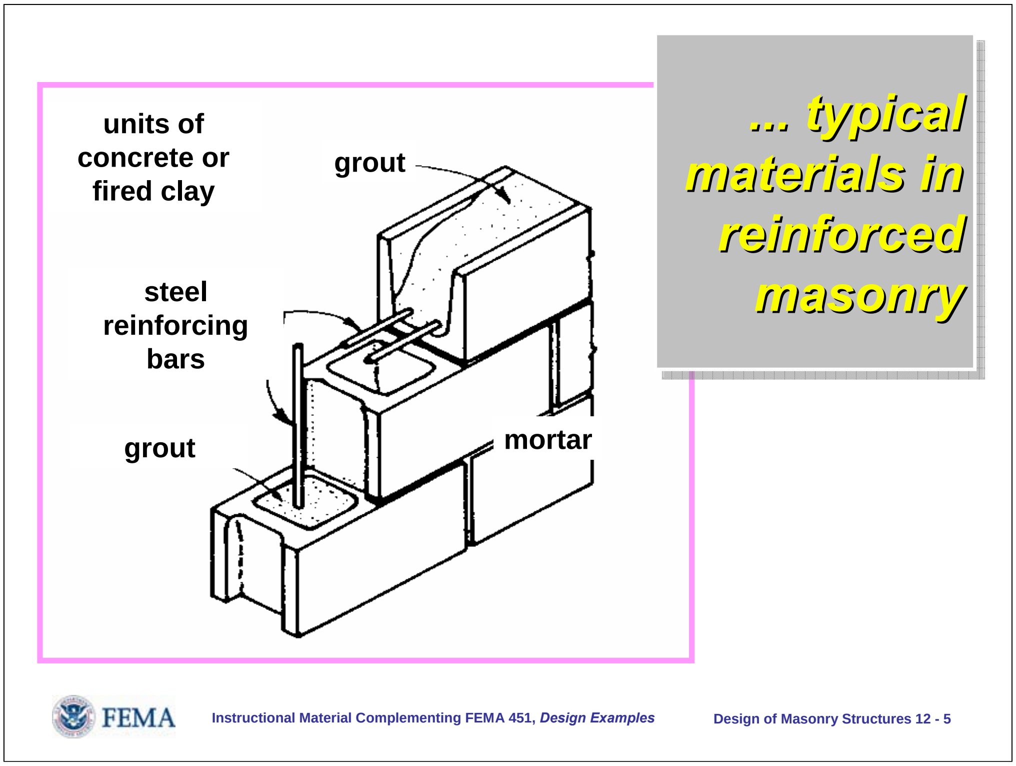
Provisions Chapters 5 and 6 are used to determine load combination and total base shear, seismic load analysis, and additional minimum levels of design force requirements for components and attachments. Chapter 11 is used to select masonry designs with adequate strength to meet demand and specifies detailing for ductility. Note that the load combination from the *Provisions* **must** be used.



Modern reinforced masonry is commonly composed of hollow concrete or clay masonry units, jointed together by cementitious mortar. Deformed reinforcement is placed vertically and horizontally within voids in the masonry, which are then filled with grout, a fluid concrete-like mixture.

Essential Elements of Simplified Design for Wall-type Structures

- **Starting point for design**
- **Design of vertical strips in walls perpendicular to lateral loads**
- **Design of walls parallel to lateral loads**
- **Design of lintels**
- **Simplified analysis for lateral loads**
- **Design of diaphragms**
- **Detailing**

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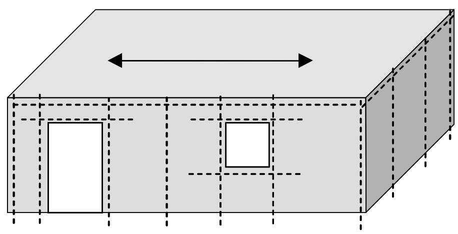
Most practicing engineers are very familiar with the behavior of frame-type structures. Many, however, may never have formally studied the behavior of wall-type structures. For that reason, it is appropriate to review that behavior. The following series of slides presents the basic starting point for design of wall-type structures and their components.

Starting Point for Wall-type Masonry Structures

No beams or columns

(Example of direction of span)



Vertical reinforcement of #4 bars at corners and jambs

Horizontal reinforcement of two #4 bars in bond beam at top of wall, and over and under openings (two #5 bars with span > 6 ft)

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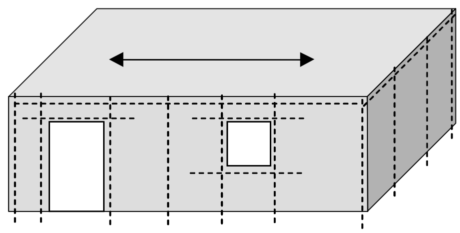
The starting point for low-rise, wall-type masonry structures is shown in this slide.

Vertical reinforcement is placed at corners, at jambs (edges of openings), at the bottoms of lintels, and at bond beams (intersections of walls and horizontal diaphragms. In regions of relatively low design loads from earthquake or wind, #4 bars are sufficient. In regions with higher design loads, #5 bars are probably required. While this starting point must be confirmed by design calculations, it is generally valid.

Horizontal reinforcement is placed at the bottoms of lintels and in bond beams at the level of the horizontal diaphragms.

Essential Function of Walls in Resisting Gravity Loads



Bearing walls resist axial loads (concentric and eccentric) as vertical strips

Nonbearing walls resist concentric axial load as vertical strips

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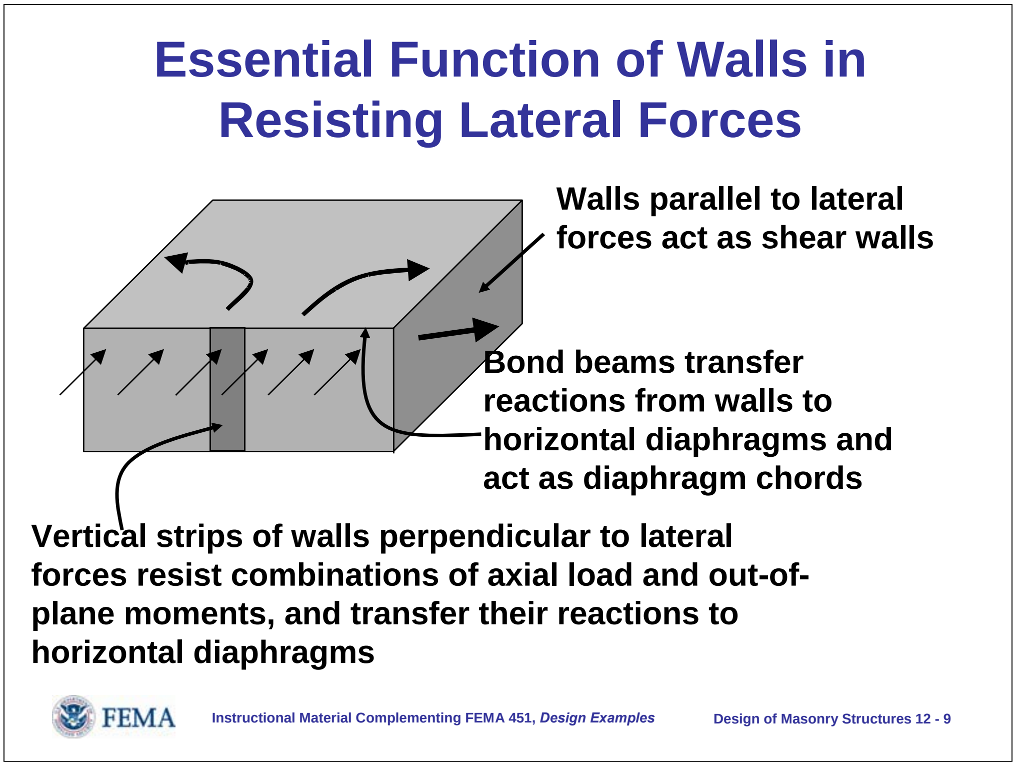
This slide shows how a wall-type building resists gravity loads. Nonbearing walls resist concentric axial loads from their own weight only. Bearing walls resist concentric axial loads from their own weight, and possibly eccentric axial loads from the reactions of roof elements. Both types of wall can be idealized as vertically spanning strips simply supported at the level of floor slab and horizontal diaphragms.

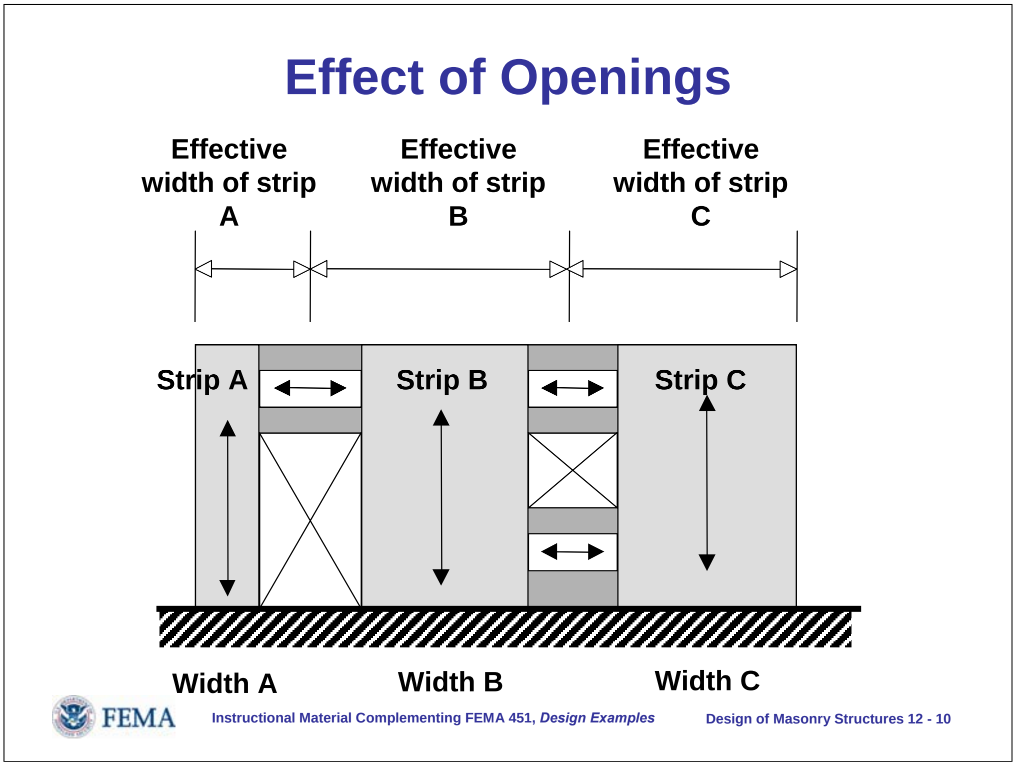


This slide shows how wall-type structures resist lateral loads.

Vertically spanning strips in the walls oriented perpendicular to the direction of lateral load resist combinations of axial load (from self-weight and roof bearing) and moments from out-of-plane wind.

Reactions from those strips are transferred to horizontal diaphragms that must resist in-plane shears and moments. The load transfer to diaphragms and the in-plane resistance are accomplished with the help of the bond beams that act as diaphragm chords.

The diaphragms, in turn, transfer their reactions to walls oriented parallel to the direction of lateral load, which then act as shear walls.



When walls oriented perpendicular to the direction of lateral load have door or window openings, these interrupt the path of vertical force transmission and change the design concept to a combination of horizontally spanning and vertically spanning strips.

The openings' vertical strips resist the loads that act directly on them and also reactions from the horizontal strips that they support. Strip B, for example, resists loads acting on its own width and also loads from the right half of the horizontal strips at the door and from the left half of the horizontal strips at the window. Strip B therefore resists loads tributary to an effective width extending from the center of the opening to the left, to the center of the opening to the right.

Effect of Openings

Openings increase original design actions on each strip by a factor equal to the ratio of the effective width of the strip divided by the actual width:

$$Actions\ in\ Strip\ B = Original\ \ Actions\left(\frac{Effective\ Width\ B}{Actual\ Width\ B}\right)$$

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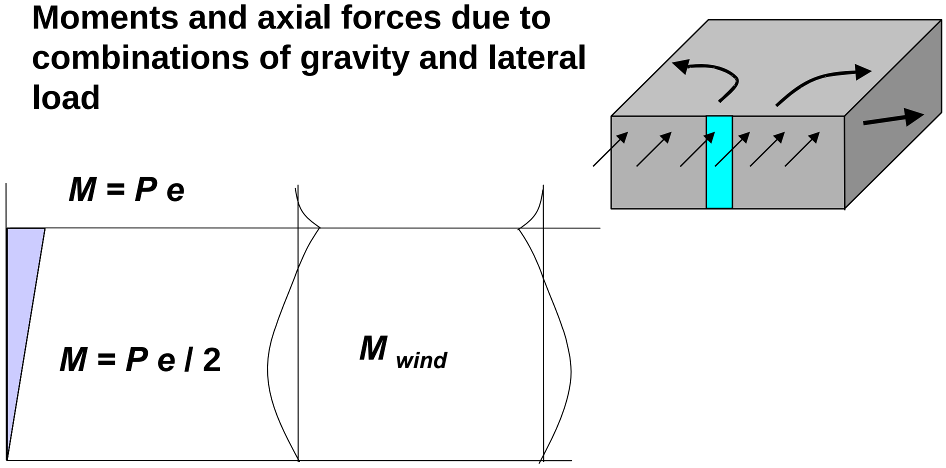
Openings in effect increase the original design actions on each strip by a factor equal to the ratio of the effective width of the strip divided by the actual width. This is precise for wind loads. It is usually conservative for seismic loads because the masses associated with doors and windows are often less than those associated with walls.

Basically, because the openings don’t change the loads on the wall, the wall’s required vertical reinforcement remains the same, and the designer must simply move that steel horizontally so that it does not coincide with the openings.

Design of Vertical Strips in Perpendicular Walls

Moments and axial forces due to combinations of gravity and lateral load



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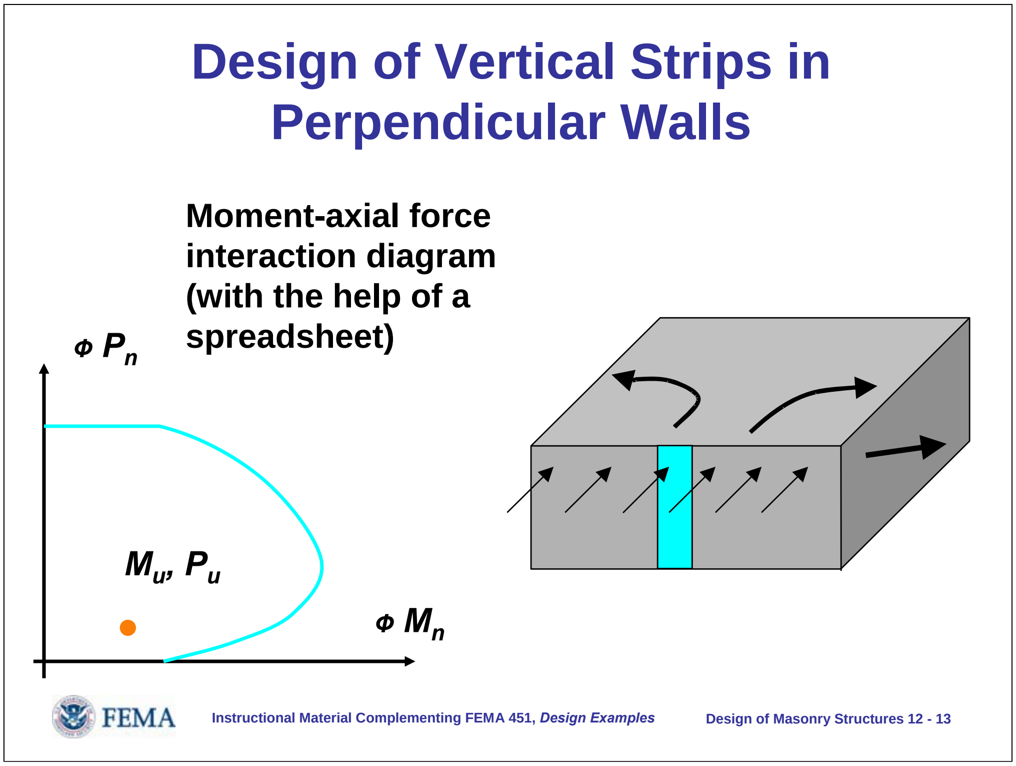
Vertically spanning strips in walls perpendicular to the direction of lateral load are subjected to axial load and possibly to eccentric gravity load from the roof.

If the strips are assumed to be simply supported at the level of the slab, the moment diagram due to eccentric gravity load varies linearly from its maximum value at the roof, to zero at the slab. These moments must be combined with moments from out-of-plane wind and earthquake forces.

This example shows the effect on the moment diagram due to a parapet and also shows that wind can act in either direction.



Design of the vertical strips consists simply of comparing the combination of factored design moment and axial load, with the design capacity expressed in terms of a moment-axial force interaction diagram, including the effects of Φ factors.

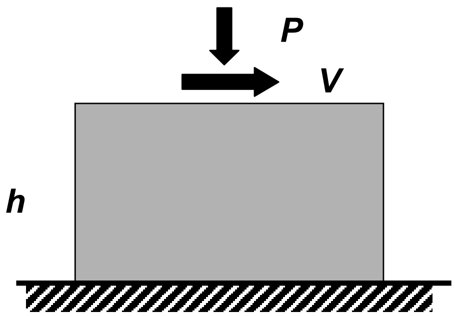
In such walls, axial loads are usually quite low, and the out-of-plane flexural capacity of the strips is essentially proportional to their flexural reinforcement.

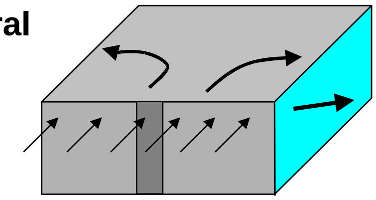
Moment-axial force interaction diagrams can be computed by hand or by spreadsheet. Such walls usually have only a single layer of reinforcement at their midplane.

Because such a single layer usually cannot be supported laterally, it is not considered effective in resisting compressive stress. It is permitted to be included in computing maximum tensile reinforcement.

Design of Parallel Walls

Moments, axial forces, and shears due to combinations of gravity and lateral loads





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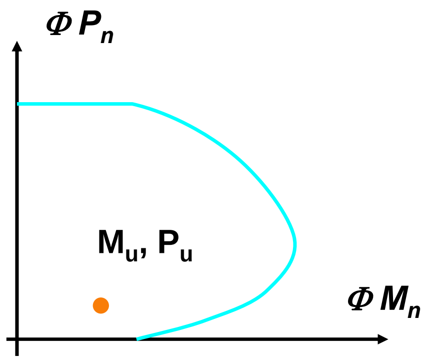
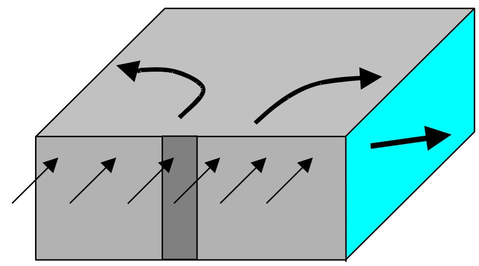
Parallel walls must be designed to resist shears from the diaphragms plus moments and axial forces.

Design of Parallel Walls

Moment-axial force interaction diagram (with the help of a spreadsheet)



Sufficient lateral capacity comes from wall density.



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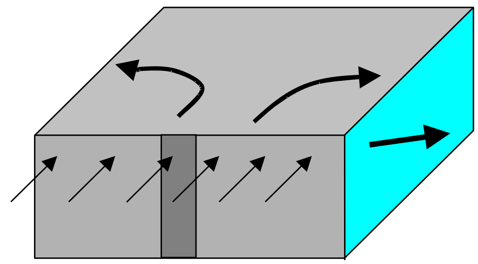
Flexural design of shear walls is expressed in terms of the relationship between combinations of factored moment and axial force and a moment-axial force interaction diagram.

The easiest way to generate such a diagram is by use of a spreadsheet in which the position of the neutral axis is moved from one side of the cross-section to the other; forces in masonry and reinforcement are computed; and the resulting axial force and moment are calculated.

Design of Parallel Walls

Shearing resistance:



$$V_n = V_m + V_s$$

$$V_m = \left[4.0 - 1.75 \left(\frac{M_u}{V_u\, d_v} \right) \right] A_n \sqrt{f'_m} + 0.25\, P_u$$

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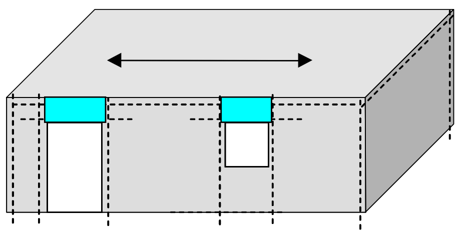
The shear design of shear walls is quite similar to that of reinforced concrete shear walls. Nominal resistance is taken as the summation of resistance from masonry plus the resistance from shear reinforcement. Nominal resistance due to masonry is considered to vary with the aspect ratio of the element. Capacity design is required for shear walls -- they must either be designed for a design strength at least equal to 1.25 times the shear associated with development of the nominal flexural strength or for a nominal strength at least equal to 2.5 times the required shear strength.

Because the nondimensional aspect ratio need not be taken greater than 1.0, nominal resistance due to masonry varies from 2.25 to 4.0 times the product of the area and the square root of the specified compressive strength of the masonry.

Design of Lintels

Moments and shears due to gravity loads:

(Example of direction of span)



$$M_u = \frac{w\,\ell^2}{8}$$

$$V_u = \frac{w\,\ell}{2}$$

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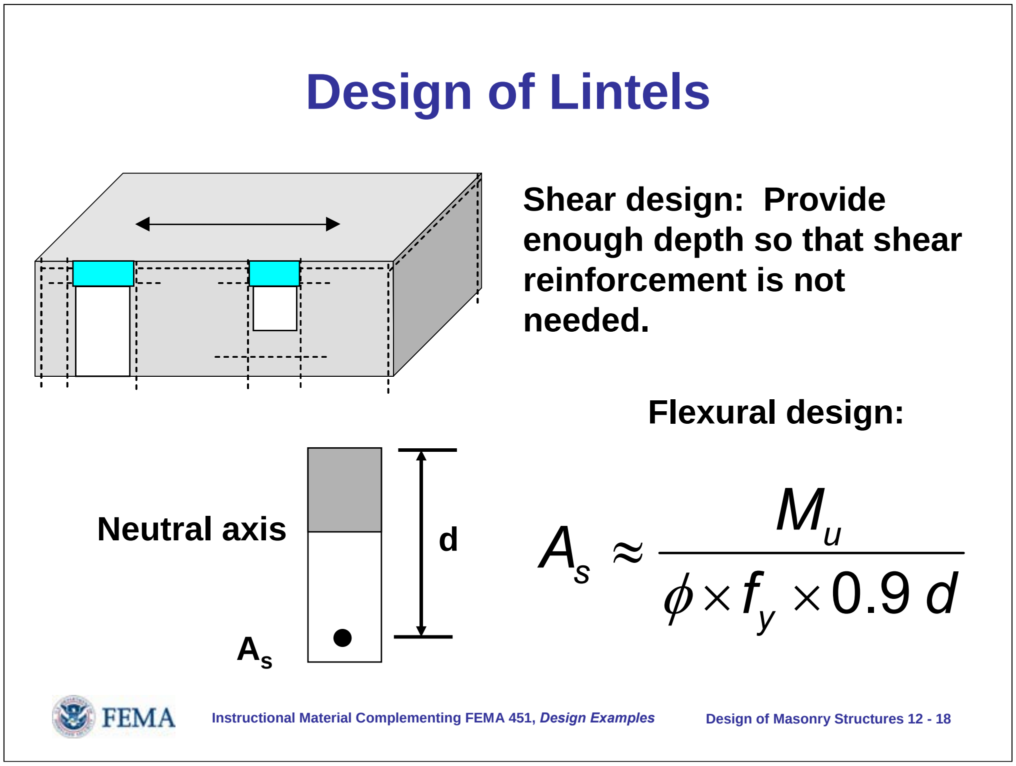
Lintels are beams that define openings in a wall.

Design of lintels is actually easier than the design of beams in general because the cross-sectional dimensions of lintels are defined by the size of the openings in the wall.

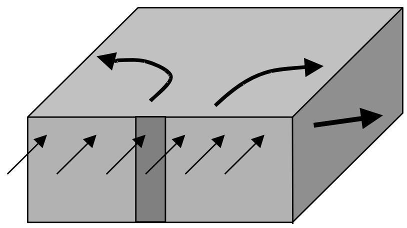
Moments and shears due to factored gravity loads are calculated.



Design of the lintel for shear is quite simple. Because shear reinforcement is impractical for masonry lintels, the designer simply checks that the architecturally defined depth and width of the lintel give sufficient cross-sectional area so that shear reinforcement is unnecessary. Lintels should be grouted solid.

Flexural design is also quite simple. Approximating the internal lever arm (distance between resultants of tensile and compressive forces in the section) as 0.9 d, the required area of longitudinal reinforcement is calculated as the factored design moment divided by the product of the lever arm and the specified yield strength of the reinforcement.

Distribution of Shears to Shear Walls



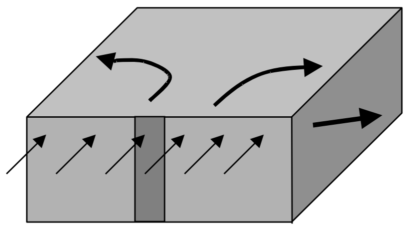
- **Classical approach**
  - **Determine whether the diaphragm is “rigid” or “flexible”**
  - **Carry out an appropriate analysis for shears**

In designing low-rise masonry buildings for lateral load, it is also necessary to compute the distribution of lateral loads to shear walls.

The classic approach is to first determine whether the diaphragm is “rigid” or “flexible” compared to the lateral force resisting system and then to carry out the appropriate analysis for wall shears. In the next few slides, the appropriate analysis for each case is briefly explained.

At the end, however, the designer is encouraged to reduce design effort by simplifying the analysis for each case and finally by bounding the shears from each case.

Classical Analysis of Structures with Rigid Diaphragms



- Locate center of rigidity
- Treat the lateral load as the superposition of a load acting through the center of rigidity and a torsional moment about that center of rigidity

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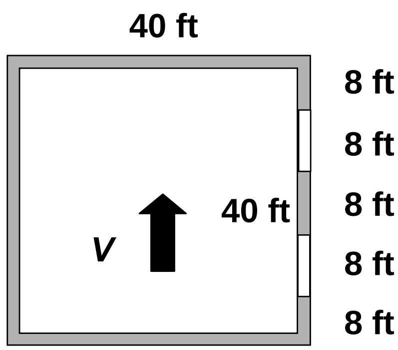
In a structure with a rigid diaphragm, the classic approach is first to locate the “center of rigidity,” or shear center of the plan. The shear center is the point through which lateral forces must be applied so that the building will not twist in plan.

The lateral load is then decomposed into a load acting through the center of rigidity and a torsional moment about the center of rigidity.

The lateral load produces direct shears on shear walls oriented parallel to the direction of the lateral load; the torsional moment produces torsional shears on all shear walls. For each wall, direct shears and torsional shears are added to get the design shear. This process is tedious.

Simplified Analysis of Structures with Rigid Diaphragms



- Consider only the shearing stiffness, which is proportional to plan length
- Neglect plan torsion

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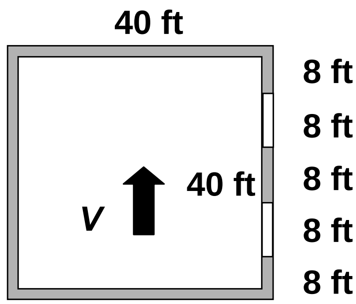
This process can be simplified considerably by neglecting plan torsion. This assumption is generally valid if the building has several walls oriented in each plan direction and well-distributed about the plan perimeter. It is not valid for garage-type buildings with one side almost completely open.

For low-rise buildings, it also is possible to neglect flexural deformations because they are quite small. Considering shearing deformations only and assuming uniform wall thickness and story height, the shearing stiffness of different walls is simply proportional to their plan lengths.

Simplified Analysis of Structures with Rigid Diaphragms



$$V_{left} = \frac{40\ ft}{(40+8+8+8)\ ft} \times V_{total} = \frac{5}{8} V_{total}$$

$$V_{right} = \frac{(8+8+8)\ ft}{(40+8+8+8)\ ft} \times V_{total} = \frac{3}{8} V_{total}$$

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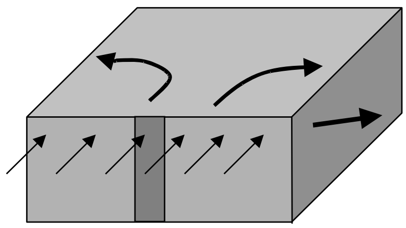
Consider the building plan shown above in which the wall on the left has a plan length of 40 ft and the wall on the right is composed of three segments with a plan length of 8 ft each.

The left and right walls together have a total stiffness proportional to their total plan length of 64 ft. The wall on the left represents 40/64 (or 5/8) of that stiffness and, therefore, resists 5/8 the applied shear. The wall on the right resists the remaining 3/8 of the applied shear.

Classical Analysis of Structures with Flexible Diaphragms



- Distribute shears according to tributary areas of the diaphragm independent of the relative stiffnesses of the shear walls

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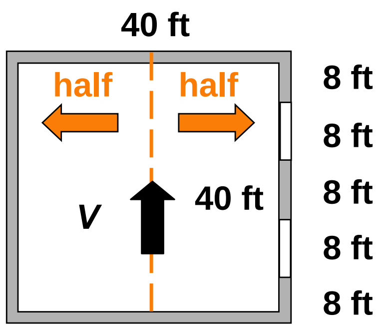
For buildings with flexible diaphragms, the diaphragm is idealized as a simply supported beam acting in the horizontal plane and resting on the shear walls. Because the shear walls are very stiff compared to the diaphragm, the shears on the shear walls depend on the tributary areas of the diaphragm that each shear wall supports.

Classical Analysis of Structures with Flexible Diaphragms



$$V_{left} = \frac{1}{2} V_{total}$$

$$V_{right} = \frac{1}{2} V_{total}$$

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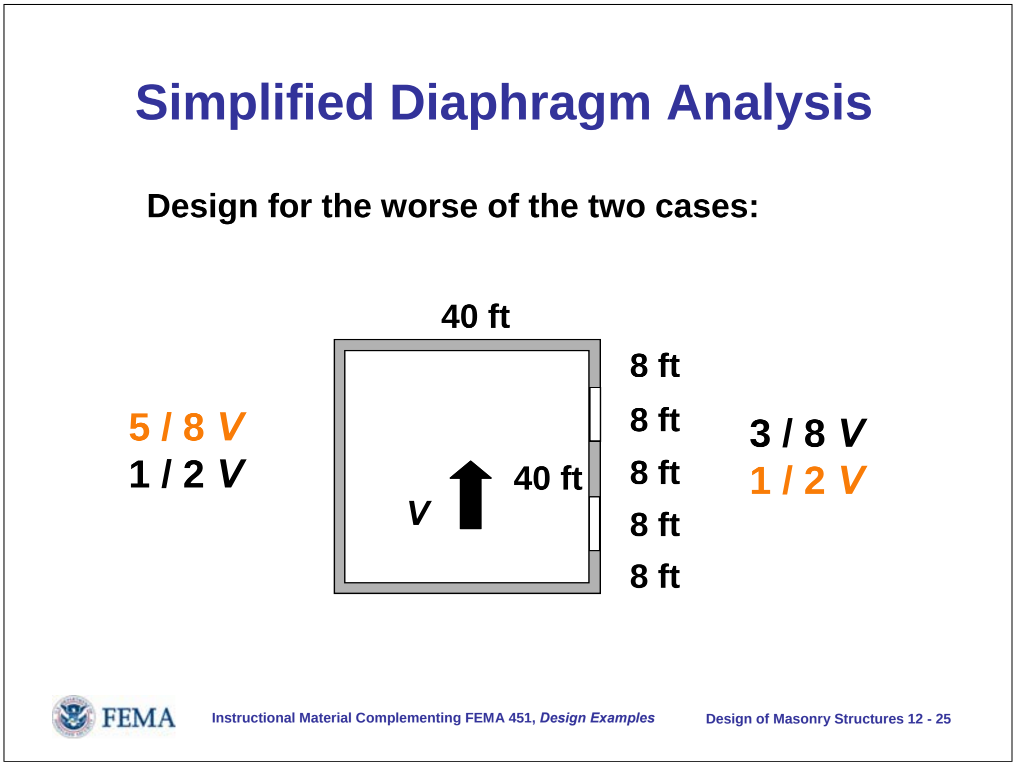
For the same building studied above but with a flexible diaphragm, the left and right shear walls each resist 1/2 the total lateral load.



Further, it is possible to avoid the need to classify the diaphragm as “rigid” or “flexible.” Simply design each wall for the more critical of the simplified rigid-diaphragm case and the flexible-diaphragm case.

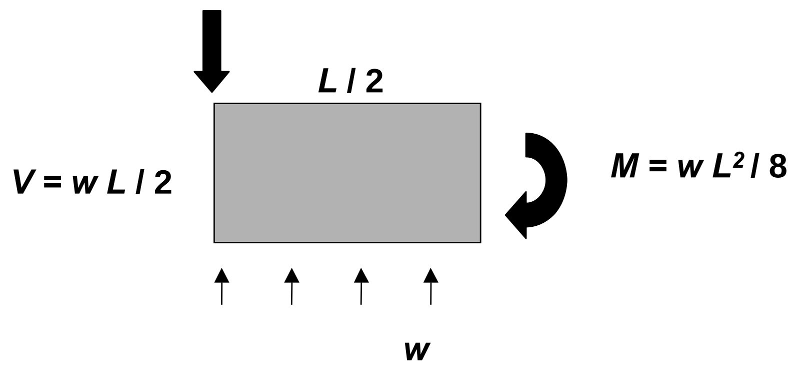
For this example, the left-hand wall had a shear of 2/3 V for the rigid-diaphragm case and a shear of 1/2 V for the flexible-diaphragm case. It could therefore be designed for the more severe of the two design shears or 2/3 V.

The right-hand wall would be similarly designed for the worse of 1/3 V (rigid) and 1/2 V (flexible) or 1/2 V.

Even though these two design shears sum to more than V, the design is conservative and valid. It might be too conservative for a few cases in which event the diaphragm stiffness would have to be evaluated.

Diagram Design

- **Diaphragm shears are resisted by total depth or by cover (for plank diaphragms). Diaphragm moments are resisted by diaphragm chords in bond beams.**



If the diaphragm is continuous, diaphragm shears are resisted by the total depth of the diaphragm. If the diaphragm is discontinuous, diaphragms are resisted by cover only. Flexural resistance comes from forces in diaphragm chords separated by the internal lever arm (distance between chords)

Details

- Wall-diaphragm connections
- Design of lintels for out-of-plane loads between wall-diaphragm connections
- Connections between bond beam and walls
- Connections between walls and foundation

After designing the out-of-plane strips, the in-plane shear walls and the lintels, additional details would have to be addressed: wall-diaphragm connections, design of lintels for out-of-plane loads between wall-diaphragm connections, connections between bond beam and walls, and connections between walls and foundations.

Masonry Behavior

- On a local level, masonry behavior is nonisotropic, nonhomogeneous, and nonlinear.
- On a global level, however, masonry behavior can be idealized as isotropic and homogeneous. Nonlinearity in compression is handled using an equivalent rectangular stress block as in reinforced concrete design.
- A starting point for masonry behavior is to visualize it as very similar to reinforced concrete. Masonry capacity is expressed in terms of a specified compressive strength, f_m', which is analogous to f_c'.

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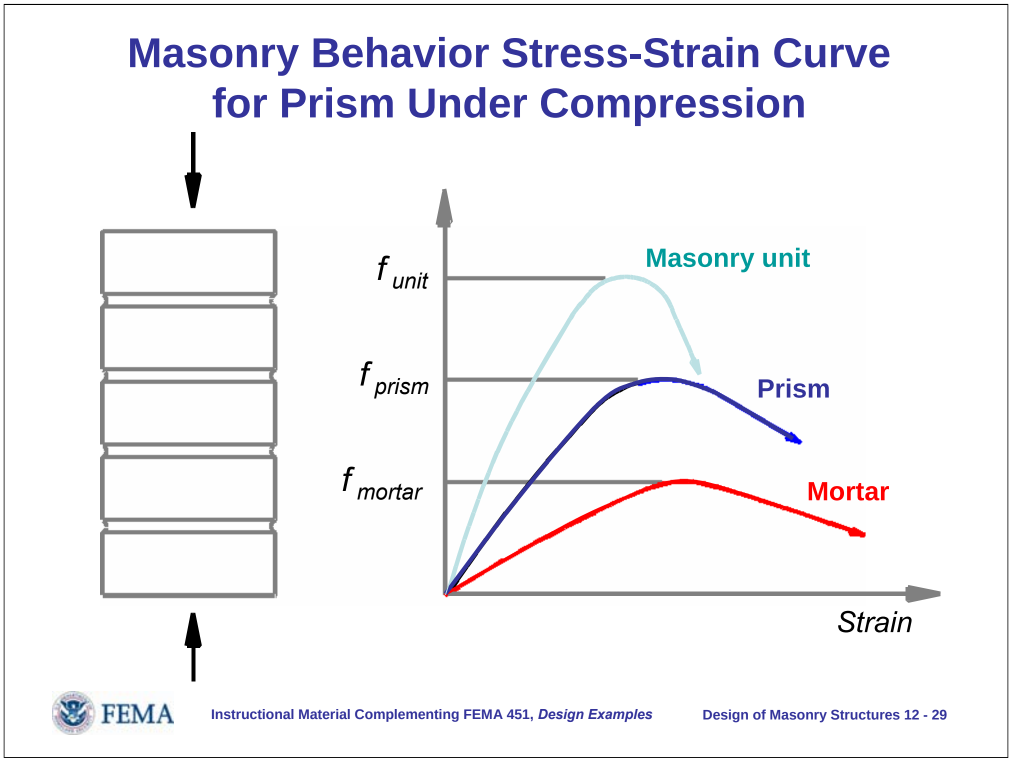
In the context of the MSJC Code and Specification as referenced by the *NEHRP Recommended Provisions*, masonry is composed of units held together by mortar. The masonry is usually reinforced (either by prescription or by design methodology), and the reinforcement is surrounded by grout. The result is an integral material very similar to reinforced concrete.



Compressive stress-strain behavior is evaluated using a masonry "prism" composed of units bonded by mortar and filled with grout (if it is intended to represent grouted construction). The individual behavior of units, mortar, and grout is not nearly as important as the behavior of the composite.

Review Masonry Basics

- **Basic terms**
- **Units**
- **Mortar**
- **Grout**
- **Accessory materials**
  - **Reinforcement (may or may not be present)**
  - **Connectors**
  - **Flashing**
  - **Sealants**
- **Typical details**

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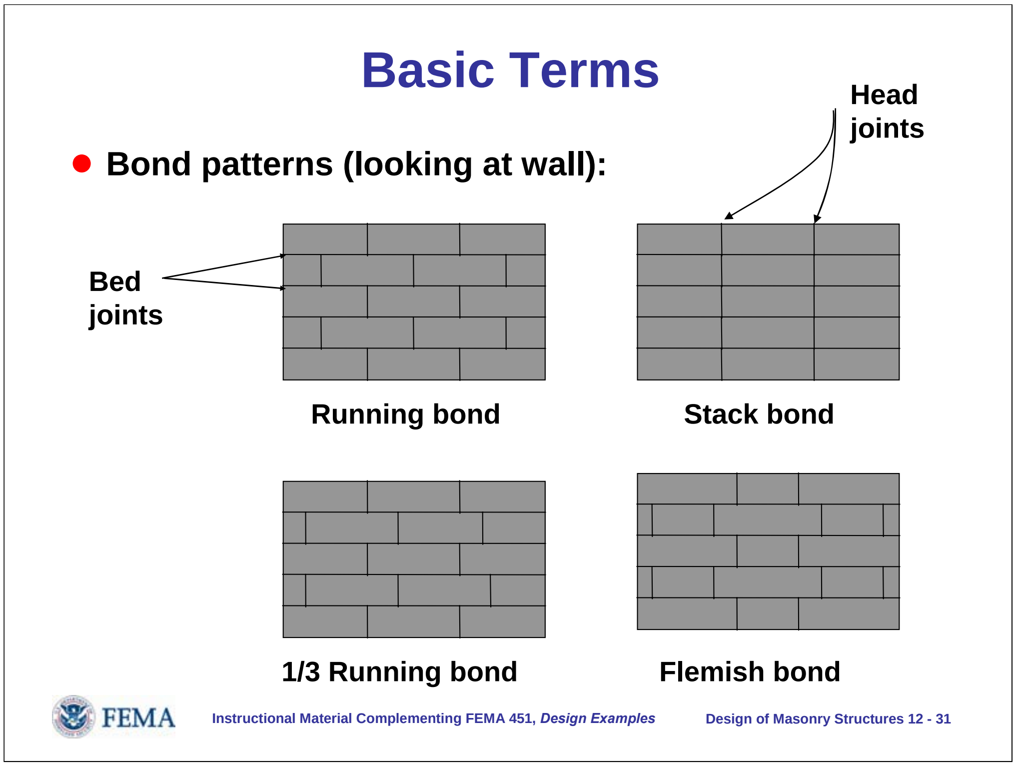
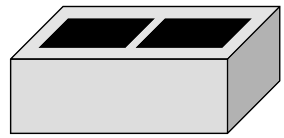
Let's start by reviewing masonry basics. Masonry is made up of units, mortar, grout, and accessory materials. The mortar holds the units together as well as apart, compensating for their dimensional tolerances. The grout is a fluid concrete mixture used to fill voids in the masonry and to anchor deformed reinforcement.



Masonry units can be laid in different bond patterns. The most common of these is 1/2 running bond, often called simply “running bond.” Horizontal bed joints are continuous; vertical head joints alternate courses, and the head joint of one course aligns with the middle of the unit on the adjacent courses. In stack bond (referred to in the MSJC Code and Specification as “other than running bond”), the head joints are continuous between adjacent courses.

Masonry Units

- Concrete masonry units (CMU):
  - Specified by ASTM C 90
  - Minimum specified compressive strength (net area) of 1900 psi (average)
  - Net area is about 55% of gross area
  - Nominal versus specified versus actual dimensions
  - Type I and Type II designations no longer exist



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Masonry units have three basic systems of dimensions: nominal, specified, and actual.

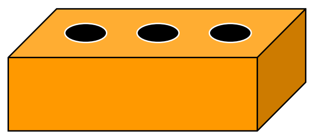
Nominal dimensions are used to lay out a structure. A common size C90 unit has nominal dimensions of 8 by 8 by 16 inches.

Specified dimensions are nominal dimensions minus one-half the thickness of a joint on all sides of the unit. Since masonry joints are normally 3/8-in. thick, the specified dimensions of a nominal 8 x 8 x 16-in. unit are 7-5/8 by 7-5/8 by 15-5/8 inches.

Actual dimensions are what the unit actually measures and should lie within the specified dimensions, plus or minus the specified dimensional tolerance.

Masonry Units

- **Clay masonry units:**
  - **Specified by ASTM C 62 or C 216**
  - **Usually solid, with small core holes for manufacturing purposes**
  - **If cores occupy ≤ 25% of net area, units can be considered 100% solid**



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Clay masonry units are specified by ASTM C62 (building brick) or C216 (facing brick). They are usually solid and may have small core holes to facilitate drying and firing. Because clay masonry units usually have more than enough compressive strength, the core holes are ignored unless they occupy more than 25% of the area of the units.

Masonry Mortar

- Mortar for unit masonry is specified by ASTM C 270
- Three cementitious systems
  - Portland cement – lime mortar
  - Masonry cement mortar
  - Mortar cement mortar

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Mortar for unit masonry is specified by ASTM C270. In specifying mortar, the designer must make three decisions:

The first decision involves the cementitious system to be used. Three cementitious systems are available: portland cement-lime mortar; masonry cement mortar; and mortar cement mortar.

Masonry Mortar

- **Within each cementitious system, mortar is specified by type (M a S o N w O r K):**
  - **Going from Type K to Type M, mortar has an increasing volume proportion of portland cement. It sets up faster and has higher compressive and tensile bond strengths.**
  - **As the volume proportion of portland cement increases, mortar is less able to deform when hardened.**
  - **Types N and S are specified for modern masonry construction.**

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Within each cementitious system, the designer must specify the mortar type. Mortar type describes the amount of cement in the mortar compared to the amount of other constituents.

The designations for mortar type were intentionally selected as alternating letters in the phrase “mason work,” avoiding the connotations that might be associated with designations such as “A,” “B,” “C,” and “D.”

Going from Type K to Type M, mortar has an increasing volume proportion of portland cement or other cements. It sets up faster and has higher compressive and tensile bond strengths. As the volume proportion of portland cement increases, mortar is less able to deform when hardened.

Types N and S are specified for modern masonry construction.

Masonry Mortar

- Under ASTM C270, mortar can be specified by proportion or by property.
- If mortar is specified by proportion, compliance is verified only by verifying proportions. For example:
  - Type S PCL mortar has volume proportions of 1 part cement to about 0.5 parts hydrated mason's lime to about 4.5 parts mason's sand.
  - Type N masonry cement mortar (single-bag) has one part Type N masonry cement and 3 parts mason's sand.

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Under ASTM C270, mortar can be specified by proportion or by property. The proportion specification is the default.

When mortar is specified by proportion, a Type S PCL mortar has volume proportions of 1 part cement to about 0.5 parts hydrated mason's lime to about 4.5 parts mason's sand. A Type N masonry cement mortar (using the most common single-bag case) has one part Type N masonry cement and about 3 parts mason's sand.

Note that the amount of water is not specified. This is because the water should be adjusted by the mason in the field to achieve good workability.

Masonry Mortar

- Under ASTM C270, mortar can be specified by proportion or by property:
  – Proportion specification is simpler -- verify in the field that volume proportions meet proportion limits.
  – Property specification is more complex: (1) establish the proportions necessary to produce a mortar that, tested at laboratory flow, will meet the required compressive strength, air content, and retentivity (ability to retain water) requirements and (2) verify in the field that volume proportions meet proportion limits.

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"Flow" is a standard ASTM measurement of the workability of a mortar. Mortar in the field typically has a flow of 130 to 135. ASTM specifications have no requirements for the properties of mortar mixed to field flow. ASTM property specifications are based on mortar with a so-called "laboratory flow" of 110 representing the characteristics of mortar after some of the water has been absorbed from it by the surrounding units.

To specify mortar by proportion according to ASTM C270 is relatively simple. The required proportions are given in the specification, and compliance is verified by verifying that the mortar is being batched using those proportions.

To specify mortar by property according to ASTM C270, one must evaluate, at laboratory flow of 110, the compressive strength, air content, and retentivity of different mortars and then decide on volume proportions that will meet the required criteria. Finally, one must verify in the field that the mortar s being batched using those proportions.

Masonry Mortar

- **The proportion specification is the default. Unless the property specification is used, no mortar testing is necessary.**
- **The proportion of water is not specified. It is determined by the mason to achieve good productivity and workmanship.**
- **Masonry units absorb water from the mortar decreasing its water-cement ratio and increasing its compressive strength. Mortar need not have high compressive strength.**

The proportion specification is the default. Unless the property specification is used, no mortar testing is necessary.

Some suggest that the words "by proportion" be added to the end of specifications to emphasize that compliance with proportion specifications involves no testing whatsoever.

The proportion of water is not specified. It is determined by the mason to achieve good productivity and workmanship.

Masonry units absorb water from the mortar decreasing its water-cement ratio and increasing its compressive strength. Mortar need not have high compressive strength.

Grout

- Grout for unit masonry is specified by ASTM C 476
- Two kinds of grout:
  - Fine grout (cement, sand, water)
  - Coarse grout (cement, sand, pea gravel, water)
- ASTM C 476 permits a small amount of hydrated lime, but does not require any. Lime is usually not used in plant – batched grout.

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Grout for unit masonry is specified by ASTM C 476, which addresses two kinds of grout: fine grout which is composed of cement, sand and water and coarse grout, which is composed of cement, sand, pea gravel and water. The fairly common practice of specifying grout as concrete is acceptable but only if the resulting mixture proportion conforms to C476.

ASTM C 476 permits a small amount of hydrated lime but does not require any. Lime is usually not used in plant-batched grout because lime is not available in such plants.

Grout

- **Under ASTM C476, grout can be specified by proportion or by compressive strength:**
  - **Proportion specification is simpler. It requires only that volume proportions of ingredients be verified.**
  - **Specification by compressive strength is more complex. It requires compression testing of grout in a permeable mold (ASTM C 1019).**

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Under ASTM C476, grout can be specified by proportion or by compressive strength.

The proportion specification is simpler. It requires only that volume proportions of ingredients be verified.

Specification by compressive strength is more complex. It requires compression testing of grout in a permeable mold (ASTM C 1019).

Grout

- **If grout is specified by proportion, compliance is verified only by verifying proportions. For example:**
  - **Fine grout has volume proportions of 1 part cement to about 3 parts mason's sand.**
  - **Coarse grout has volume proportions of 1 part cement to about 3 parts mason's sand and about 2 parts pea gravel.**
- **Unless the compressive-strength specification is used, no grout testing is necessary.**

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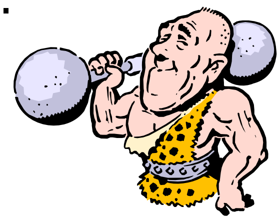
If grout is specified by proportion, compliance is verified only by verifying proportions.

For example, fine grout has volume proportions of 1 part cement to about 3 parts mason's sand. Coarse grout has volume proportions of 1 part cement to about 3 parts mason's sand and about 2 parts pea gravel.

Unless the compressive-strength specification is used, no grout testing is necessary.

Grout

- **The proportion of water is not specified. The slump should be 8 to 11 in.**
- **Masonry units absorb water from the grout decreasing its water-cement ratio and increasing its compressive strength. High-slump grout will still be strong enough.**



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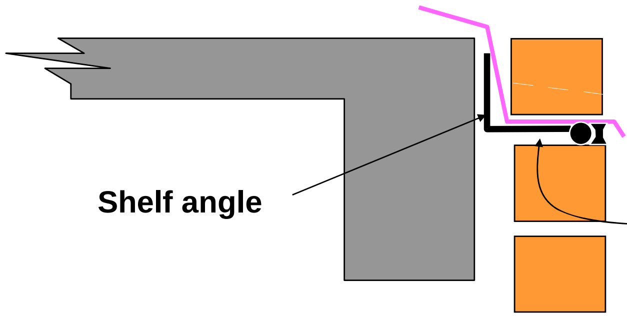
The proportion of water in grout is not specified. The slump should be 8 to 11 inches.

Masonry units absorb water from the grout, decreasing its water-cement ratio and increasing its compressive strength. High-slump grout will still be strong enough.

Accessory Materials

Horizontally oriented expansion joint under shelf angle:



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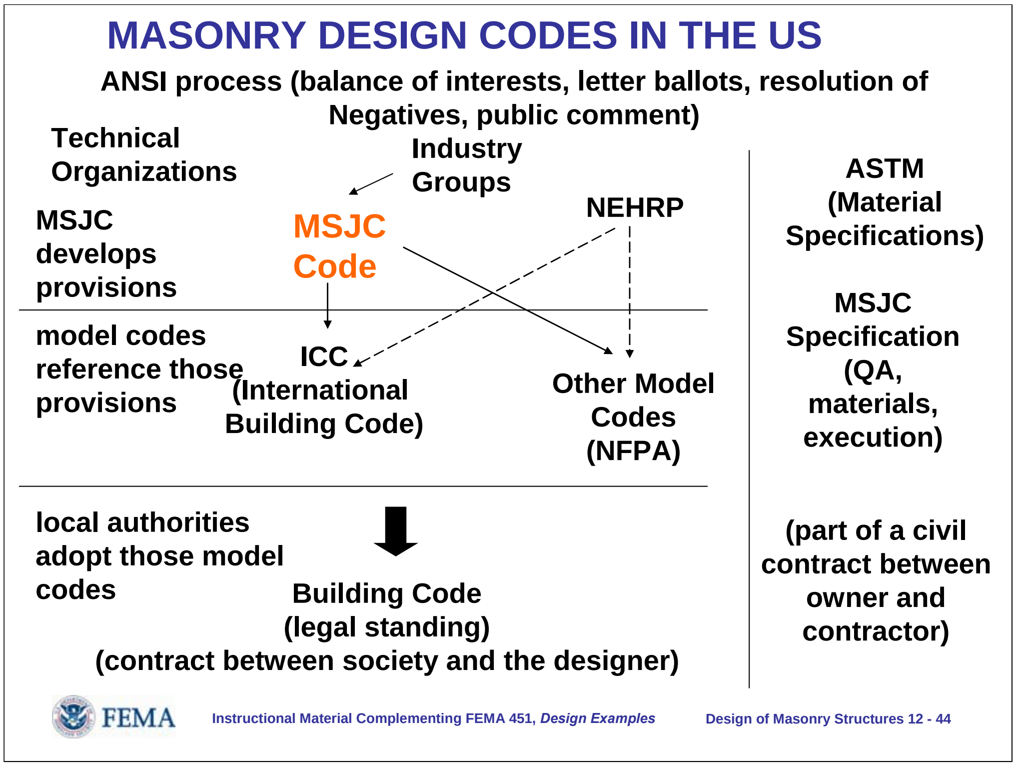
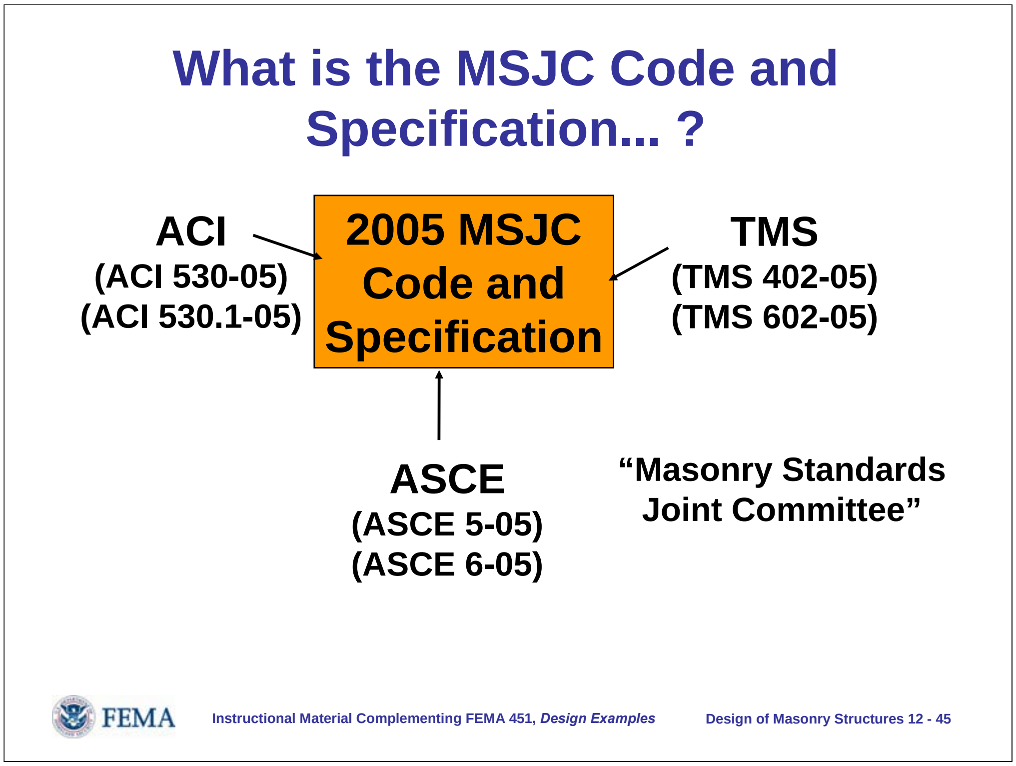
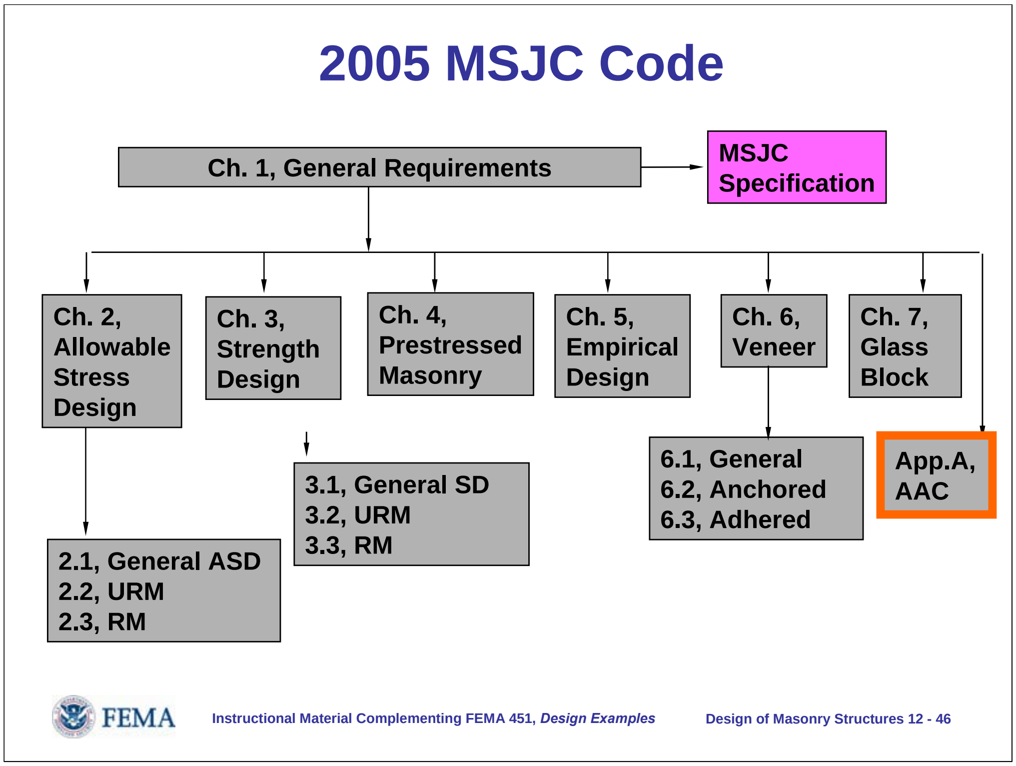
One of the most important details in a masonry building is the expansion joint under shelf angles in clay masonry veneer. Clay masonry expands over time; concrete and concrete masonry shrink. If the veneer is laid without the expansion joint, it will end up supporting the building even though it is not made to resist the resulting compression.



Through the National Earthquake Hazard Reduction Program, the U.S. federal government develops the resource documents or guidelines for technical organizations and model codes. Thus, the *NEHRP Recommended Provisions* are just that -- recommendations only.



The 2003 *NEHRP Recommended Provisions* reference the MSJC Code and Specification. That document is developed under ANSI-consensus rules by the Masonry Standards Joint Committee, which is sponsored jointly by The Masonry Society, The American Concrete Institute, and The American Society of Civil Engineers. The latest version is the 2005 edition.



This slide shows the organization of the MSJC Code. It is linked to the MSJC Specification. The 2005 edition includes a new mandatory-language appendix dealing with autoclaved aerated concrete (AAC) masonry.

Relation Between Code and Specification

- **Code:**
  - **Design provisions are given in Chapters 1-7 and Appendix A**
  - **Sections 1.2.4 and 1.14 require a QA program in accordance with the specification**
  - **Section 1.4 invokes the specification by reference.**
- **Specification:**
  - **Verify compliance with specified f_m'**
  - **Comply with required level of quality assurance**
  - **Comply with specified products and execution**

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 47

The MSJC Code references and is intended to be used with the MSJC Specification.

In the Code, design provisions are given in Chapters 1 through 7 and Appendix A. Sections 1.2.4 and 1.14 require a QA program in accordance with the Specification. Section 1.4 invokes the Specification by reference.

The Specifications requires verification of compliance with specified f_m'; compliance with the required level of quality assurance; and compliance with the specified products and execution.

Role of f_m'

- **Concrete:**
  - **Designer states assumed value of f_c'**
  - **Compliance is verified by compression tests on cylinders cast in the field and cured under ideal conditions**
- **Masonry**
  - **Designer states assumed value of f_m'**
  - **Compliance is verified by "unit strength method" or by "prism test method"**

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 48

In designing and specifying masonry according to the MSJC Code, the role of f'_m is analogous to that of f'_c for concrete.

For concrete, the designer states an assumed value of f_c' and compliance is verified by compression tests on cylinders cast in the field and cured under ideal conditions.

For masonry, the designer states an assumed value of f_m' and compliance is verified either by the "unit strength method," or by the "prism test method."

Verify Compliance with Specified f_m'

- Unit strength method (Spec 1.4 B 2):
  - Compressive strengths from unit manufacturer
  - ASTM C 270 mortar
  - Grout meeting ASTM C 476 or 2,000 psi
- Prism test method (Spec 1.4 B 3):
  - Pro -- can permit optimization of materials
  - Con -- require testing, qualified testing lab, and procedures in case of non-complying results

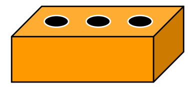
Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 49

The MSJC Code offers two ways of demonstrating compliance with the specified f'_m. The simplest is the "unit strength method." Using compressive strengths for standard ASTM units obtained by the unit manufacturer as part of production quality control, mortar meeting ASTM C270 and grout meeting ASTM C476 or having a compressive strength of at least 2000 psi, conservative values for f'_m can be taken from Tables 1 and 2 of the Specification.

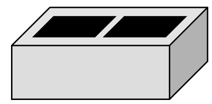
Alternatively, prisms can be constructed and tested by ASTM C1314.

Example of Unit Strength Method (Specification Tables 1, 2)

- **Clay masonry units (Table 1):**
  - **Unit compressive strength ≥ 4150 psi**
  - **Type N mortar**
  - **Prism strength can be taken as 1500 psi**



- **Concrete masonry units (Table 2):**
  - **Unit compressive strength ≥ 1900 psi**
  - **Type S mortar**
  - **Prism strength can be taken as 1500 psi**



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For example, in clay masonry, if units have a compressive strength of at least 4150 psi and ASTM C270 Type N mortar is used, the prism strength can be taken as 1500 psi.

In concrete masonry, if units have a compressive strength of at least 1900 psi (which happens to be the minimum for C90 units) and ASTM C270 Type S mortar is used, the prism strength can be taken as 1500 psi.

A specified prism strength of 1500 psi is very common for masonry.

Application of Unit Strength Method (Spec Tables 1, 2)

- **Design determines required material specification:**
  - **Designer states assumed value of f_m'**
  - **Specifier specifies units, mortar and grout that will satisfy "unit strength method"**
- **Compliance with f_m' can be verified with no tests on mortar, grout, or prisms**

Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 51

If compliance with the specified f'_m is verified by the unit strength method and compliance with ASTM C270 (mortar) and ASTM C476 (grout) is verified by proportion), no job-specific testing whatsoever is required.

Comply with Specified Products and Execution

- **Products -- Specification Article 2:**
  - **Units, mortar, grout, accessory materials**
- **Execution -- Specification Article 3**
  - **Inspection**
  - **Preparation**
  - **Installation of masonry, reinforcement, grout, prestressing tendons**

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 52

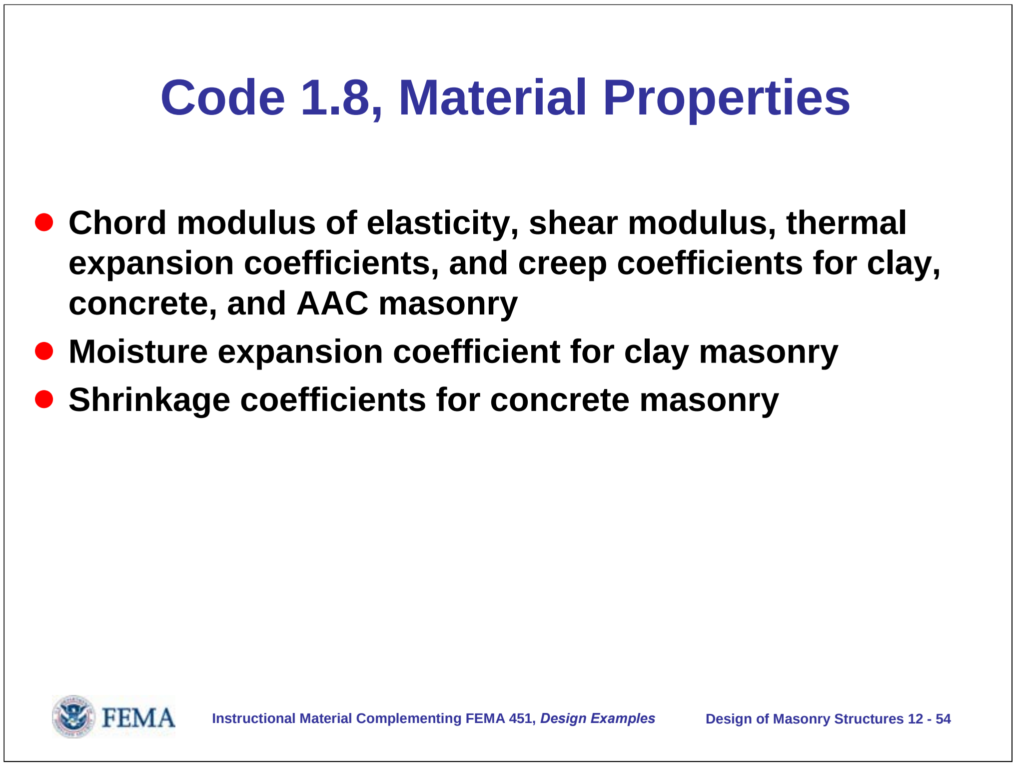
Finally, the MSJC Specification requires compliance with the specified products (Article 2), which means units, mortar, grout and accessory materials, and with the specified execution (Article 3), which means inspection, preparation and installation of masonry, reinforcement, grout and prestressing tendons.

Organization of MSJC Code Chapter 1

- 1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions
- 1.7 Loading
- 1.8 Material properties
- 1.9 Section properties
- 1.10 Deflections
- 1.11 Stack bond masonry
- 1.12 Corbels
- 1.13 Details of reinforcement
- 1.14 Seismic design requirements
- 1.15 Quality assurance program
- 1.16 Construction

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 53

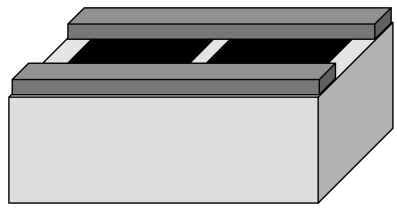
Chapter 1 of the MSJC Code is an “umbrella” chapter. It gives basic requirements that govern over the other provisions. As with other slides, the sections marked in orange are emphasized in the slides that immediately follow.



Code Section 1.8 deals with material properties to be used for design. It specifies the values of chord modulus of elasticity, shear modulus, thermal expansion coefficients, and creep coefficients for clay, concrete, and AAC masonry. It also gives moisture expansion coefficients to be used for clay masonry and shrinkage coefficients to be used for concrete masonry.

Code 1.9, Section Properties

- **Use minimum (critical) area for computing member stresses or capacities**
  - **Capacity is governed by the weakest section; for example, the bed joints of face-shell bedded hollow masonry**



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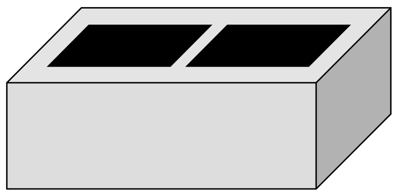
The MSJC Code uses two different ways of computing section properties. To compute member stresses or capacities, use the weakest section. For hollow unit masonry bedded on the outside only (face-shell bedding), this is the area of the face shells only.

Code 1.9, Section Properties

- Radius of gyration and member slenderness are better represented by the average section; for example, the net area of units of face-shell bedded masonry



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For computing slenderness-related properties, use the average section (the net area of units of face-shell bedded masonry).

Organization of MSJC Code
Chapter 1

- **1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions**
- **1.7 Loading**
- **1.8 Material properties**
- **1.9 Section properties**
- **1.10 Deflections**
- **1.11 Stack bond masonry**
- **1.12 Corbels**
- **1.13 Details of reinforcement**
- **1.14 Seismic design requirements**
- **1.15 Quality assurance program**
- **1.16 Construction**

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Section 1.13 of the MSJC Code deals with details of reinforcement.

Code 1.13, Details of Reinforcement

- Reinforcing bars must be embedded in grout; joint reinforcement can be embedded in mortar
- Placement of reinforcement
- Protection for reinforcement
- Standard hooks

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Reinforcing bars must be embedded in grout; joint reinforcement can be embedded in mortar. Minimum distances between reinforcement and the insides of cells or void spaces are specified as are minimum cover distances for protection of reinforcement from corrosion. Standard hooks are defined.

Organization of MSJC Code Chapter 1

1.1 – 1.6 Scope, Contract documents and calculations, special systems, reference standards, notation, definitions

1.7 Loading

1.8 Material properties

1.9 Section properties

1.10 Deflections

1.11 Stack bond masonry

1.12 Corbels

1.13 Details of reinforcement

1.14 Seismic design requirements

1.15 Quality assurance program

1.16 Construction

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Section 1.14 of the MSJC Code deals with seismic design requirements.

Code 1.14, Seismic Design

- Applies to all masonry except
  - Glass unit masonry
  - Veneers
- Seeks to improve performance of masonry structures in earthquakes
  - Improves ductility of masonry members
  - Improves connectivity of masonry members
- Different requirements for AAC masonry

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 60

Section 1.14 applies to all masonry except glass unit masonry and veneers. It seeks to improve performance of masonry structures in earthquakes by improving the ductility of masonry members and the connectivity of masonry members.

Seismic requirements for autoclaved aerated concrete (AAC) masonry are somewhat different and are not addressed here.

Code 1.14, Seismic Design

- **Define a structure's Seismic Design Category (SDC) according to ASCE 7-02**
  - **SDC depends on seismic risk (geographic location), importance, underlying soil**
- **SDC determines**
  - **Required types of shear walls (prescriptive reinforcement)**
  - **Prescriptive reinforcement for other masonry elements**
  - **Permitted design approaches for LFRS (lateral force-resisting system)**

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In Code Section 1.14, the structure's Seismic Design Category (SDC) is defined according to ASCE 7-02. The SDC depends on seismic risk (geographic location), importance, and underlying soil.

The SDC determines the required types of shear walls (prescriptive reinforcement); the prescriptive reinforcement required for other masonry elements; and the permitted design approaches for LFRS (lateral force-resisting system).

Code 1.14, Seismic Design

- Seismic design requirements are keyed to ASCE 7-02 Seismic Design Categories (from A up to F).
- Requirements are cumulative; requirements in each “higher” category are added to requirements in the previous category.

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 62

Seismic design requirements are keyed to ASCE 7-02 Seismic Design Categories (from A up to F). Requirements are cumulative; requirements in each “higher” category are added to requirements in the previous category.

Code 1.14, Seismic Design

- **Seismic Design Category A:**
  - **Drift limit = 0.007**
  - **Minimum design connection force for wall-to roof and wall-to-floor connections**
- **Seismic Design Category B:**
  - **Lateral force resisting system cannot be designed empirically**

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 63

In Seismic Design Category A, a drift limit of 0.007 and a minimum design connection force are imposed for wall-to-roof and wall-to-floor connections.

In Seismic Design Category B, the lateral force resisting system cannot be designed empirically.

Code 1.14, Seismic Design

- **Seismic Design Category C:**
  - **All walls must be considered shear walls unless isolated**
  - **Shear walls must meet minimum prescriptive requirements for reinforcement and connections (ordinary reinforced, intermediate reinforced, or special reinforced)**
  - **Other walls must meet minimum prescriptive requirements for horizontal or vertical reinforcement**

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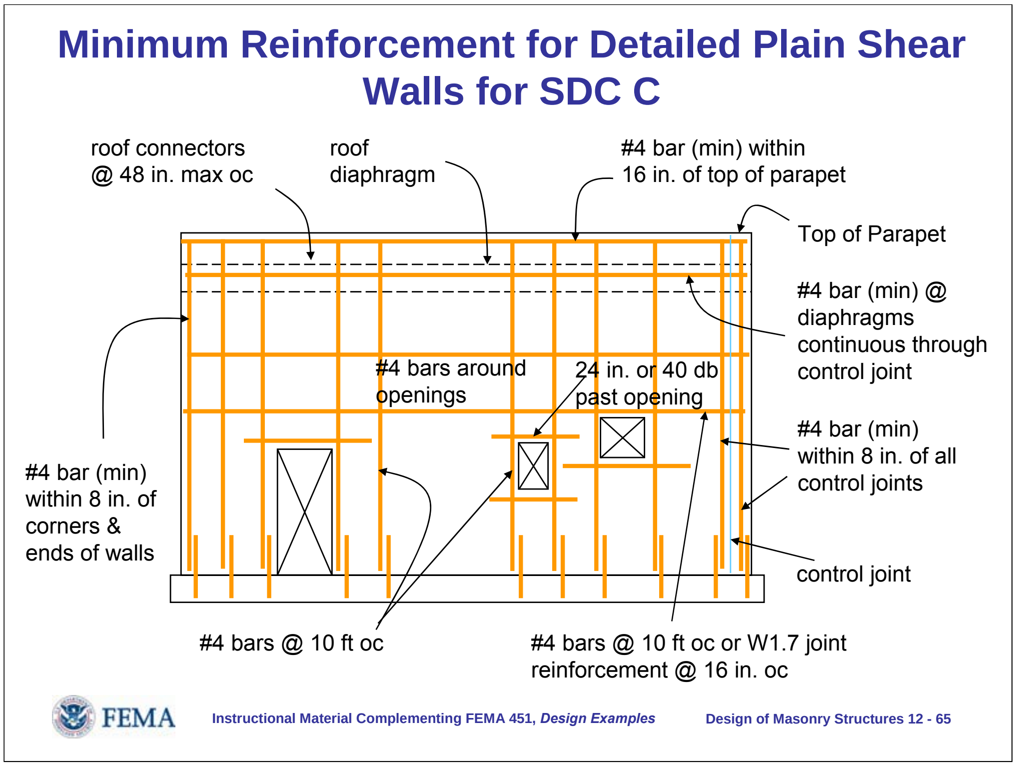
In Seismic Design Category C, all walls must be considered shear walls unless isolated. Shear walls must meet minimum prescriptive requirements for reinforcement and connections (ordinary reinforced, intermediate reinforced, or special reinforced) and other walls must meet minimum prescriptive requirements for horizontal or vertical reinforcement.



This slide illustrates the minimum reinforcement requirements for detailed plain masonry shear walls and for all shear walls in SDC C.

Code 1.14, Seismic Design

- **Seismic Design Category D:**
  - **Masonry that is part of the lateral force-resisting system must be reinforced so that $\rho_v + \rho_h \geq 0.002$, and ρ_v and $\rho_h \geq 0.0007$**
  - **Type N mortar and masonry cement mortars are prohibited in the lateral force-resisting system**
  - **Shear walls must meet minimum prescriptive requirements for reinforcement and connections (special reinforced)**
  - **Other walls must meet minimum prescriptive requirements for horizontal and vertical reinforcement**

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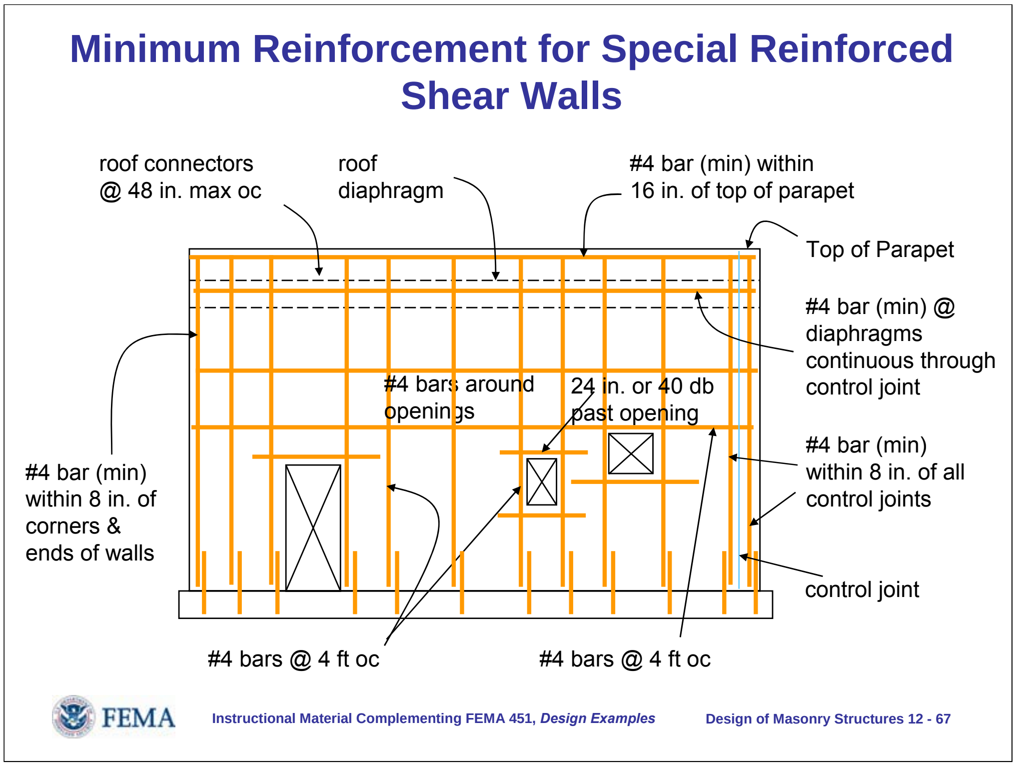
In Seismic Design Category D, masonry that is part of the lateral force resisting system must be reinforced so that $\rho_v + \rho_h \geq 0.002$, and ρ_v and $\rho_h \geq 0.0007$. Type N mortar and masonry cement mortars are prohibited in the lateral force resisting system. Shear walls must meet minimum prescriptive requirements for reinforcement and connections (special reinforced) and other walls must meet minimum prescriptive requirements for horizontal and vertical reinforcement.



This slide illustrates the minimum reinforcement requirements for special reinforced masonry shear walls.

Code 1.14, Seismic Design

- **Seismic Design Categories E and F:**
  - **Additional reinforcement requirements for stack-bond masonry**

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In Seismic Design Categories E and F, additional requirements are imposed for stack-bond masonry because it is inherently weaker in flexure across continuous head joints.

Minimum Reinforcement, SW Types

SW Type	Minimum Reinforcement	SDC
Empirically Designed	none	A
Ordinary Plain	none	A, B
Detailed Plain	Vertical reinforcement = 0.2 in.^{2} at corners, within 16 in. of openings, within 8 in. of movement joints, maximum spacing 10 ft; horizontal reinforcement W1.7 @ 16 in. or #4 in bond beams @ 10 ft	A, B
Ordinary Reinforced	same as above	A, B, C
Intermediate Reinforced	same as above, but vertical reinforcement @ 4 ft	A, B, C
Special Reinforced	same as above, but horizontal reinforcement @ 4 ft, and ρ = 0.002	any

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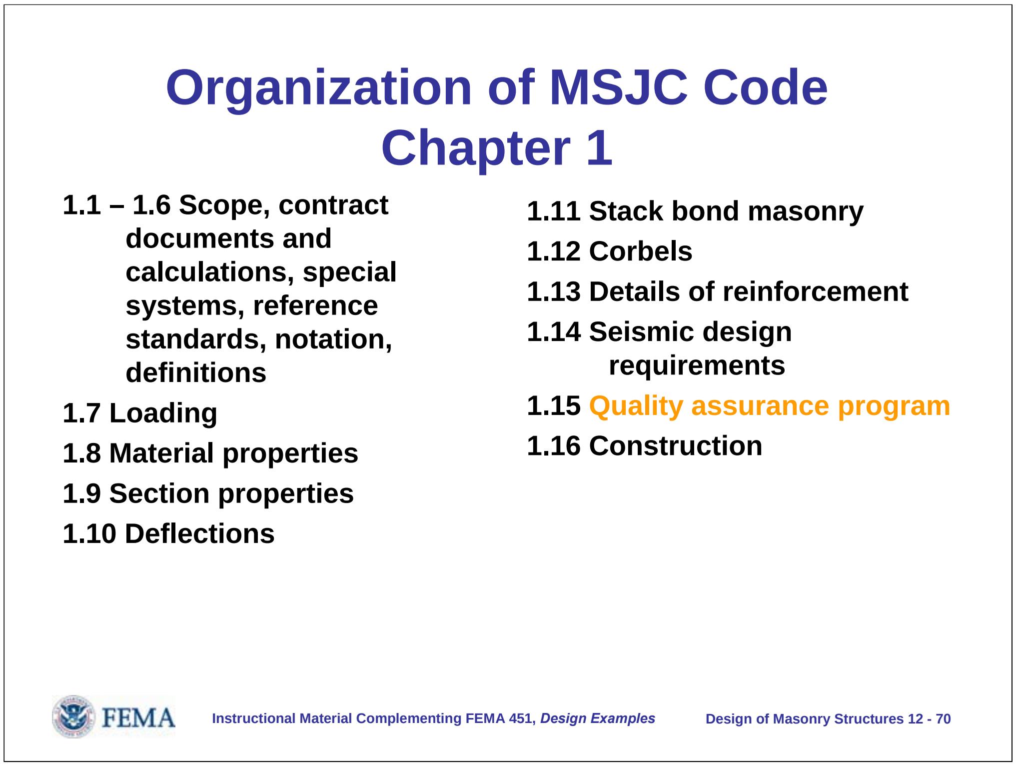
This slide summarizes the requirements for different shear wall types and the Seismic Design Categories in which each type is permitted to be used.



Section 1.15 of the MSJC Code requires a quality assurance program. Unlike some older model codes that permitted a normal requirement for continuous inspection to be waived provided that allowable stresses were divided by two, the MSJC Code requires some level of quality assurance for every masonry job.

Code 1.15, Quality Assurance

- Requires a quality assurance program in accordance with the MSJC Specification:
  - Three levels of quality assurance (A, B, C)
  - Compliance with specified f_m'
  - Increasing levels of quality assurance require increasingly strict requirements for inspection, and for compliance with specified products and execution

Section 1.15 of the MSJC Code requires a quality assurance program in accordance with the Specification. The section contemplates three levels of quality assurance (A, B, C): compliance with specified f_m', increasing levels of quality assurance with increasingly strict requirements for inspection, and for compliance with specified products and execution.

Code 1.15, Quality Assurance

- **Minimum requirements for inspection, tests, and submittals:**
  - **Empirically designed masonry, veneers, or glass unit masonry**
    - Table 1.14.1.1 for nonessential facilities
    - Table 1.14.1.2 for essential facilities
  - **Other masonry**
    - Table 1.14.1.2 for nonessential facilities
    - Table 1.14.1.3 for essential facilities

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Section 1.15 of the MSJC Code gives minimum requirements for inspection, tests, and submittals.

For empirically designed masonry, veneers or glass unit masonry, Table 1.14.1.1 addresses nonessential facilities and Table 1.14.1.2, essential facilities.

For other masonry, Table 1.14.1.2 addresses nonessential facilities, and Table 1.14.1.3, essential facilities.

Organization of MSJC Code Chapter 1

- **1.1 – 1.6 Scope, contract documents and calculations, special systems, reference standards, notation, definitions**
- **1.7 Loading**
- **1.8 Material properties**
- **1.9 Section properties**
- **1.10 Deflections**
- **1.11 Stack bond masonry**
- **1.12 Corbels**
- **1.13 Details of reinforcement**
- **1.14 Seismic design requirements**
- **1.15 Quality assurance program**
- **1.16 Construction**

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Section 1.16 of the MSJC Code addresses construction requirements.

1.16, Construction

- Minimum grout spacing (Table 1.16.2)
- Embedded conduits, pipes, and sleeves:
  - Consider effect of openings in design
  - Masonry alone resists loads
- Anchorage of masonry to structural members, frames, and other construction:
  - Show type, size, and location of connectors on drawings

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That section specifies a minimum grout spacing (Table 1.16.2).

With respect to embedded conduits, pipes and sleeves, it requires that the effects of openings be considered in design and that masonry alone be considered effective in resisting loads.

With respect to anchorage of masonry to structural members, frames and other construction, it requires that the type, size, and location of connectors be shown on drawings.

... Organization of MSJC Code Chapter 3, Strength Design (SD)

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- Φ-factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

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Now let's move to Chapter 3 of the MSJC Code dealing with strength design. It is this chapter that first addresses the fundamental basis for strength design.

Fundamental Basis for Strength Design

- Factored design actions must not exceed nominal capacities, reduced by Φ factors
- Quotient of load factor divided by the Φ factor is analogous to safety factor of allowable-stress design, and should be comparable to that safety factor.

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Factored design actions must not exceed nominal capacities, reduced by Φ-factors. The quotient of load factor divided by Φ-factor is analogous to safety factor of allowable stress design and should be comparable to that safety factor.

Organization of MSJC Code Chapter 3, Strength Design

- Fundamental basis
- Loading combinations
- Design strength
- Φ factors
- Deformation requirements
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

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Chapter 3 next addresses loading combinations.

Code 3.1.2, Loading Combinations for SD

- From governing building code
- From ASCE 7-02

If the governing code (e.g., IBC, NFPA 5000, or a state code) specifies strength loading combinations, they must be used. If none are specified, the strength loading combinations must be taken from ASCE 7-02. From the standpoint of the *NEHRP Recommended Provisions*, the combinations of *Provisions* Chapters 5 and 6 would govern.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading
- Design strength
- *Φ* factors
- Deformation requirements
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 79

Chapter 3 of the MSJC Code now addresses design strength.

Code 3.1.3, Design Strength for SD

- **Design strength must exceed required strength**
- **Extra caution against brittle shear failure:**
  - **Design shear strength shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength**
  - **Nominal shear strength need not exceed 2.5 times required shear strength**

The design strength must exceed required strength.

To give additional protection against brittle shear failure, the design shear strength must exceed the shear corresponding to the development of 1.25 times the nominal flexural strength (capacity design). The nominal shear strength need not, however, exceed 2.5 times the required shear strength.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- Φ factors
- Deformation requirements
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

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Chapter 3 of the MSJC Code now address ϕ factors (capacity reduction factors).

Code 3.1.4, Strength-reduction Factors for SD

Action	Reinforced Masonry	Unreinforced Masonry
Combinations of flexure and axial load	0.90	0.60
Shear	0.80	0.80
Anchorage and splices of Reinforcement	0.80	---
Bearing	0.60	0.60

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The table of this slide summarizes the capacity reduction factors used by the MSJC Code. For reinforced masonry, they are quite similar to those of ACI 318.

Code 3.1.4, Strength-reduction Factors for SD

Capacity of Anchor Bolts as Governed by	Strength-reduction Factor
Steel yield and fracture	0.90
Masonry breakout	0.50
Pullout of bent-bar anchors	0.65

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This slide summarizes capacity reduction factors for the design of anchor bolts.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- *Φ* factors
- Deformation requirements
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

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Chapter 3 of the MSJC Code now addresses deformation requirements.

Code 3.1.5, Deformation Requirements

- Drift limits from ASCE 7-02
- Deflections of unreinforced masonry (URM) based on uncracked sections
- Deflections of reinforced masonry (RM) based on cracked sections

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The MSJC Code imposes the drift limits of ASCE 7-02. It requires that deflections of unreinforced masonry be based on uncracked sections and that deflections of reinforced masonry be based on cracked sections.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- *Φ* factors
- Deformation requirements
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Chapter 3 of the MSJC Code now address anchor bolts.

Code 3.1.6, Anchor Bolts

- **Tensile capacity governed by:**
  - **Tensile breakout**
  - **Yield of anchor in tension**
  - **Tensile pullout (bent-bar anchor bolts only)**
- **Shear capacity governed by:**
  - **Shear breakout**
  - **Yield of anchor in shear**
- **For combined tension and shear, use linear interaction**

FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 87

Tensile capacity is governed by tensile breakout, by yield of the anchor in tension, and by tensile pullout (for bent-bar anchor bolts only).

Shear capacity is governed by shear breakout and by yield of the anchor in shear.

Combined tension and shear are conservatively handled using a linear interaction relationship.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- Φ factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Reinforced masonry
- Unreinforced masonry

Chapter 3 of the MSJC Code now addresses required compressive strength.

Code 3.1.7.1.1, Compressive Strength of Masonry

- For concrete masonry, 1,500 psi $\leq f_m' \leq$ 4,000 psi
- For clay masonry, 1,500 psi $\leq f_m' \leq$ 6,000 psi

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The specified compressive strength of concrete masonry must be between 1500 and 4000 psi because that is the range of compressive strengths used in the test data justifying this approach. For clay masonry, corresponding limits are 1500 and 6000 psi.

Code 3.1.7.1.2, Compressive Strength of Grout

- **For concrete masonry, $f_m' \leq f_g' \leq$ 5,000 psi**
- **For clay masonry, $f_g' \leq$ 6,000 psi**

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For similar reasons, the specified compressive strength of grout for concrete masonry is required to be between f'_m and 5000 psi and for clay masonry, to be less than 6000 psi.

If grout is specified by proportion, it will automatically lie within these limits.

Organization of MSJC Code Chapter 3

- **Fundamental basis**
- **Loading combinations**
- **Design strength**
- **Deformation requirements**
- ***Φ* factors**
- **Anchor bolts**
- **Bearing strength**
- **Compressive strength**
- **Modulus of rupture**
- **Strength of reinforcement**
- **Unreinforced masonry**
- **Reinforced masonry**

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Chapter 3 of the MSJC Code now addresses the modulus of rupture of masonry. For reinforced masonry, this is irrelevant for computing capacity but still relevant in computing effective moments of inertia for deflection calculations.

Code 3.1.8.2, Modulus of Rupture

- **In-plane and out-of-plane bending**
  - **Table 3.1.8.2.1**
  - **Lower values for masonry cement and air-entrained portland cement-lime mortar**
  - **Higher values for grouted masonry**
  - **For grouted stack-bond masonry, f_r = 250 psi parallel to bed joints for continuous horizontal grout section**

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In contrast to earlier versions of the MSJC Code, the 2005 edition specifies identical values of the modulus of rupture for in-plane and out-of-plane bending (Table 3.1.8.2.1). Modulus of rupture values are lower for masonry cement and air-entrained portland cement-lime mortar and are higher for grouted masonry. For grouted stack-bond masonry, f_r = 250 psi parallel to bed joints.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- Φ factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 93

Chapter 3 of the MSJC Code now addresses the strength of reinforcement.

Code 3.1.8.3, Strength of Reinforcement

- **$f_y \leq$ 60 ksi**
- **Actual yield strength shall not exceed 1.3 times the specified value**
- **Compressive strength of reinforcement shall be ignored unless the reinforcement is tied in compliance with Code 2.1.6.5**

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The specified yield strength of reinforcement is not to be taken in excess of 60 ksi.

The actual yield strength is not to exceed 1.3 times the specified value (to avoid excesses of actual flexural capacity, which can lead to excessive shear demand).

The compressive strength of reinforcement is to be ignored unless the reinforcement is tied in compliance with Code 2.1.6.5.

Organization of MSJC Code Chapter 3

- Fundamental basis
- Loading combinations
- Design strength
- Deformation requirements
- Φ factors
- Anchor bolts
- Bearing strength
- Compressive strength
- Modulus of rupture
- Strength of reinforcement
- Unreinforced masonry
- Reinforced masonry

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Reinforced masonry, in the context of the MSJC Code and Specification, is designed assuming that flexural tension is resisted entirely by flexural reinforcement.

Code 3.3, Reinforced Masonry

- **Masonry in flexural tension is cracked**
- **Reinforcing steel is needed to resist tension**
- **Similar to strength design of reinforced concrete**

Masonry in flexural tension is considered to be cracked. Flexural tension is to be resisted entirely by reinforcing steel. Design is similar to strength design of reinforced concrete.

Code 3.3, Reinforced Masonry

3.3.2 Design assumptions

3.3.3 Reinforcement requirements and details, including maximum steel percentage

3.3.4 Design of piers, beams and columns:

- Nominal axial and flexural strength
- Nominal shear strength

3.3.5 Design of walls for out-of-plane loads

3.3.6 Design of walls for in-plane loads

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Section 3.3 of the MSJC Code deals in detail with the assumptions underlying strength design of reinforced masonry:

Section 3.3.3 addresses reinforcement requirements and details, including maximum steel percentage.

Section 3.3.4 addresses design of piers, beams and columns, including nominal axial and flexural strength and nominal shear strength

Section 3.3.5 addresses design of walls for out-of-plane loads.

Section 3.3.6 addresses design of walls for in-plane loads.

Let's look at each of these in more detail.

Code 3.3.2, Design Assumptions

- **Continuity between reinforcement and grout**
- **Equilibrium**
- **ε_{mu} = 0.0035 for clay masonry, 0.0025 for concrete masonry**
- **Plane sections remain plane**
- **Elasto-plastic stress-strain curve for reinforcement**
- **Tensile strength of masonry is neglected**
- **Equivalent rectangular compressive stress block in masonry, with a height of 0.80 f_m' and a depth of 0.80 c**

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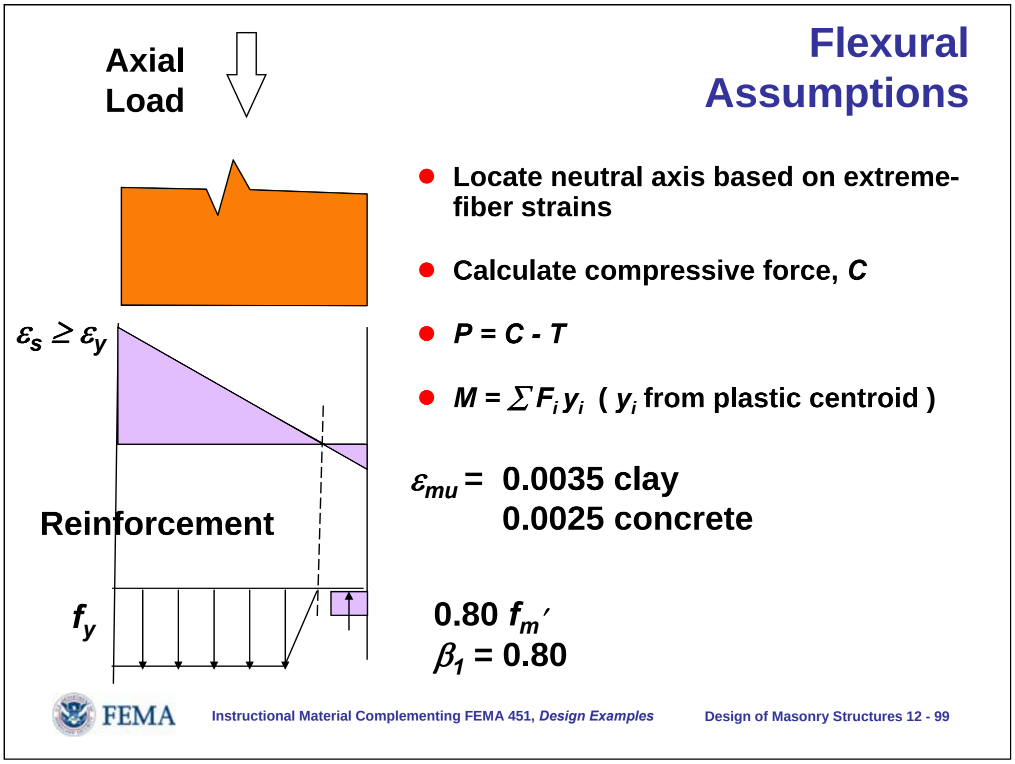
The basic assumptions of flexural design in reinforced masonry are quite similar to those of reinforced concrete.

Continuity between reinforcement and grout is assumed. Equilibrium must be satisfied. The maximum usable strain is to be taken as ε_{mu} = 0.0035 for clay masonry and 0.0025 for concrete masonry.

Plane sections are assumed to remain plane. An elastoplastic stress-strain curve is used for for reinforcement; and the tensile strength of masonry is to be neglected.

The compressive stress block is idealized as an equivalent rectangle with a height of 0.80 f_m' and a depth of 0.80 c.



This slide summarizes how those flexural assumptions are applied. As will be discussed subsequently, sections are required to be tension-controlled so the strain gradient varies across the depth of the cross-section from the maximum useful strain in the masonry to a steel strain at least equal to yield. Reinforcement is considered to be elastoplastic. The equivalent rectangular stress block is taken as shown.

The axial force in the cross-section is computed as the difference between the compressive and the tensile forces, and the moment is computed as the summation of each of those forces times its respective distance from the plastic centroid (usually the centerline) of the cross-section.

For concrete and masonry design, the plastic centroid is defined as the line of action of the resultant force in the cross-section corresponding to a uniform compressive strain equal to the maximum useful strain in the concrete or masonry.

Code 3.3.3, Reinforcement Requirements and Details

- Bar diameter ≤ 1 / 8 nominal wall thickness
- Standard hooks and development length:
  - Development length based on pullout and splitting
- In walls, shear reinforcement must be bent around extreme longitudinal bars
- Splices:
  - Lap splices based on required development length
  - Welded and lap splices must develop 1.25 f_y

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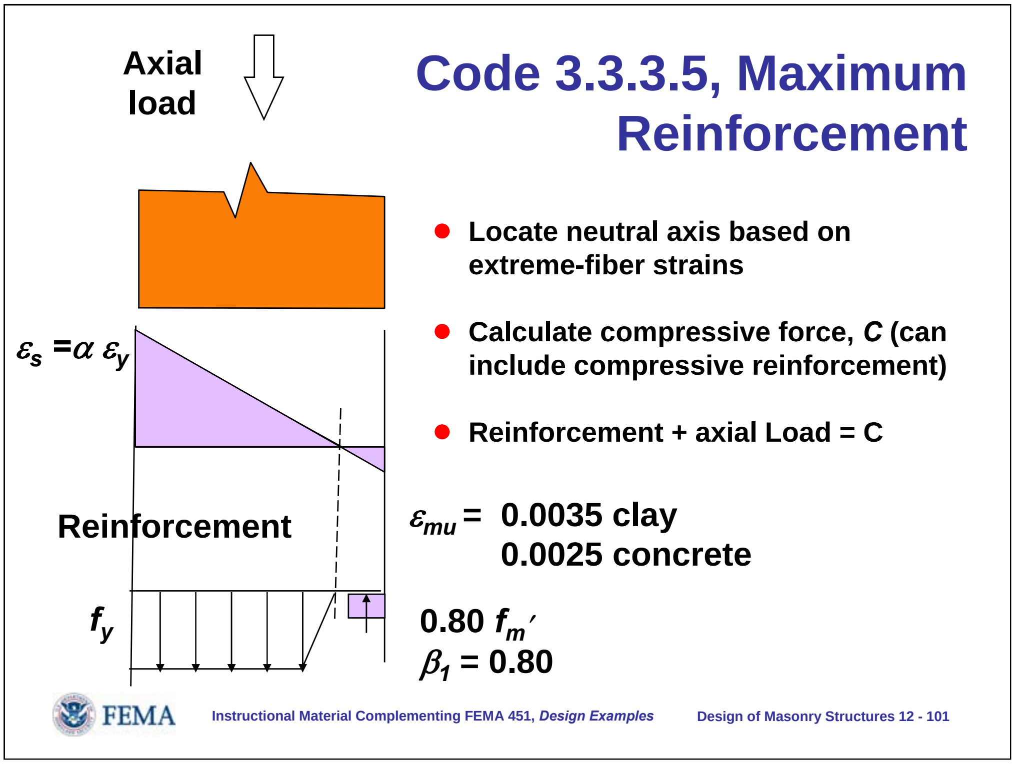
Section 3.3.3 of the MSJC Code addresses reinforcement requirements and details.

The diameter of reinforcement in eighths of an inch must not exceed 1/8 the nominal wall thickness. For example, in a nominal 8-in. wall, reinforcement must not be larger in diameter than #8 (nominal diameter 8/8 inch). This is intended to prevent failure by the masonry by splitting along the bar.

Standard hooks are defined. Development length is based on pullout and on splitting (bar-to-cover and bar-to-bar). In walls, shear reinforcement must be bent around extreme longitudinal bars (intended to increase the effectiveness of the hooks).

Required splice length is based on required development length; welded and lap splices must develop 1.25 f_y .

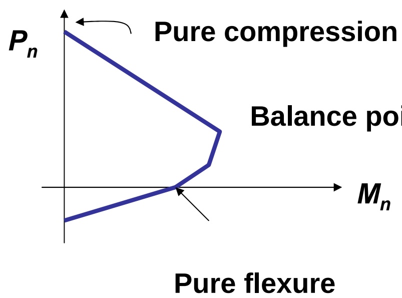


This slide illustrates the principle behind the maximum reinforcement limitations of Section 3.3.3.5 of the MSJC Code. Assumed is a critical strain gradient, whose maximum value on the compression side of the element is the maximum useful strain in masonry and whose maximum value on the tension side of the element is a multiple of the specified yield strain in that reinforcement. That multiple depends on the expected flexural ductility demand on the element. In contrast to the calculation of flexural capacity, the contribution of compressive reinforcement can be considered.

That critical gradient defines the location of the neutral axis and the dimensions of the compressive stress block. That, in turn, gives the maximum compressive capacity of that block. The combination of tensile steel area (acting at a stress limited by f_y) and axial compression must not exceed that compressive capacity. The intent of this provision is to prevent compressive crushing of that block and, therefore, ensure tension-controlled behavior.

Code 3.3.4, Design of Beams, Piers, and Columns

- Capacity under combinations of flexure and axial load is based on the assumptions of Code 3.3.2 (interaction diagram)



Using the assumptions of Section 3.3.2 of the MSJC Code, capacity of reinforced masonry elements under combinations of flexure and axial load is described using a moment-axial force interaction diagram, just as for reinforced concrete.

Such a diagram can be computed by hand for key points, or computed using a spreadsheet for various assumed positions of the neutral axis.

Code 3.3.4, Design of Beams, Piers, and Columns

- Slenderness is addressed by multiplying axial capacity by slenderness-dependent modification factors

$$\left[1-\left(\frac{h}{140r}\right)^2\right] \quad \text{for } \frac{h}{r} \leq 99$$

$$\left(\frac{70r}{h}\right)^2 \quad \text{for } \frac{h}{r} > 99$$

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The 2005 MSJC Code does not use moment magnifiers. Slenderness is addressed by multiplying axial capacity by slenderness-dependent modification factors. Since axial loads are usually quite low compared to axial capacity in concentric compression, slenderness effects are usually not significant.

Code 3.3.4, Nominal Shear Strength

- $V_n = V_m + V_s$
- V_n shall not exceed:
  - $M / V d_v \leq 0.25$ $V_n \leq 6 A_n \sqrt{f_m'}$
  - $M / V d_v \geq 1.0$ $V_n \leq 4 A_n \sqrt{f_m'}$
  - Linear interpolation between these extremes
  - Objective is to avoid crushing of diagonal strut

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Section 3.3.4 of the MSJC Code addresses nominal shear strength of reinforced masonry in a way that is quite similar to design of reinforced concrete. Nominal shear resistance is taken as the summation of resistance from masonry, plus resistance from shear reinforcement. To prevent diagonal crushing of masonry, upper limits are imposed on V_n (and thereby on V_s).

Code 3.3.4, Nominal Shear Strength

- V_m and V_s are given by:

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u \qquad (3-21)$$

$$\left(\frac{M_u}{V_u d_v}\right) \leq 1.0$$

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v \qquad (3-22)$$

Nominal shear resistance due to masonry depends on the aspect ratio of the element, and varies from 2.25 to 4 times the product of the square root of f'_m, and the cross-sectional area of the element.

Nominal shear resistance due to shear reinforcement is computed similarly to reinforced concrete as the product of the number of layers of shear reinforcement crossing a 45-degree crack, the cross-sectional area of each layer of reinforcement, and the specified tensile yield strength of the reinforcement. The MSJC Code uses an efficiency factor to account for the fact that shear reinforcement does not yield uniformly over the height of a wall element.

Code 3.3.4.2, Requirements for Beams

- $P_u \le 0.05\, A_n\, f_m'$
- $M_n \ge 1.3\, M_{cr}$
- Lateral bracing spaced at most 32 times beam width
- Nominal depth not less than 8 in.

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Section 3.3.4 2 of the MSJC Code addresses requirements for beams, defined as elements whose design axial load is low. To prevent brittle fracture of tensile reinforcement following flexural cracking, nominal capacity is required to be at least equal to 1.3 times the computed cracking capacity. To prevent lateral-torsional buckling, lateral bracing is required to be spaced at most 32 times the beam width. To ensure reasonable flexural capacity, the total depth (nominal dimension) cannot be less than 8 in.

Code 3.3.4.3, Requirements for Piers

- **Isolated elements (wall segments are not piers)**
- $P_u \leq 0.3\, A_n\, f_m'$
- **Nominal thickness between 6 and 16 in.**
- **Nominal plan length between 3 and 6 times the nominal thickness**
- **Clear height not more than 5 times the nominal plan length**

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In the context of the MSJC Code and Specification, a "pier" is an isolated structural element resisting axial compression and meeting certain geometric criteria. Wall segments are normally not piers.

For piers, design axial load cannot exceed 0.3 $A_n\, f_m'$. The nominal thickness must be between 6 and 16 in. The nominal plan length must be between 3 and 6 times the nominal thickness. The clear height must not be more than 5 times the nominal plan length.

Code 3.3.4.4, Requirements for Columns

- **Isolated elements (wall segments are not columns)**
- $\rho_g \geq 0.0025$
- $\rho_g \leq 0.04$**, and also meet Code 3.3.3.5**
- **Lateral ties in accordance with Code 2.1.6.5**
- **Solid-grouted**
- **Least cross-section dimension ≥ 8 in.**
- **Nominal depth not greater than 3 times the nominal width**

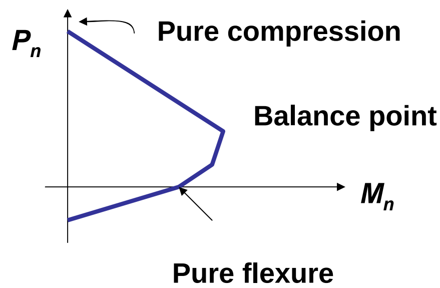
Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 108

In the context of the MSJC Code and Specification, a “column” is an isolated structural element resisting axial compression, and meeting certain geometric criteria. Wall segments are normally not columns.

Columns must have a ratio of total longitudinal reinforcement to gross cross-sectional area between 0.0025 and 0.04. They must meet maximum flexural reinforcement requirements of Section 3.3.3.5 of the MSJC Code. Longitudinal reinforcement must be supported laterally (tied) in accordance with Section 2.1.6.5 of the MSJC Code. Their least cross-sectional dimension must be at least 8 in., and their nominal depth must not exceed 3 times their nominal width.

Code 3.3.5, Design of Walls for Out-of-plane Loads

- **Capacity under combinations of flexure and axial load is based on the assumptions of Code 3.3.2 (interaction diagram)**



Walls are designed under out-of-plane loads using a strength interaction diagram.

Code 3.3.5, Design of Walls for Out-of-plane Loads

- Maximum reinforcement by Code 3.3.3.5
- Procedures for computing out-of-plane moments and deflections (moment magnifier, vary depending on axial load)
- Nominal shear strength by Code 3.3.4.1.2

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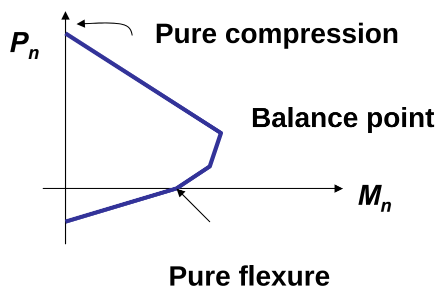
Maximum reinforcement requirements are given by Section 3.3.3.5 of the MSJC Code. Slenderness effects are addressed (for walls loaded out of plane only) by computing a magnified moment based on computed out-of-plane deflections.

Nominal shear strength is given by Section 3.3.4.1.2.

Code 3.3.6, Design of Walls for In-plane Loads

- Capacity under combinations of flexure and axial load is based on the assumptions of Code 3.3.2 (interaction diagram)



Flexural design of walls for in-plane loads is again handled using a strength interaction diagram. Because such walls have multiple layers of reinforcement over their depth, hand computations are tedious, and hand approximations (see design example at the end of this module) or computer spreadsheet calculations are preferable.

Code 3.3.6, Design of Walls for In-plane Loads

- **Maximum reinforcement by Code 3.3.3.5**
- **Vertical reinforcement not less than one-half the horizontal reinforcement**
- **Nominal shear strength by Code 3.3.4.1.2**

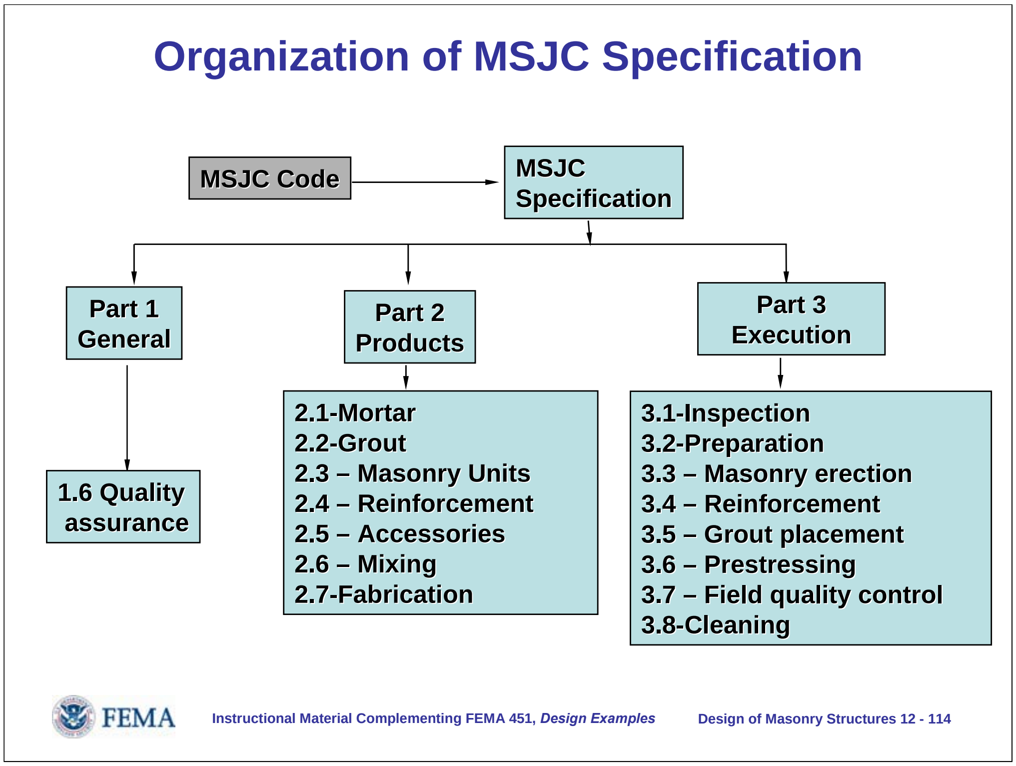
FEMA Instructional Material Complementing FEMA 451, *Design Examples* Design of Masonry Structures 12 - 112

Code Section 3.3.6 addresses design of walls for in-plane loads. Maximum longitudinal (flexural) reinforcement is specified by Section 3.3.3.5. Nominal shear strength is calculated by Section 3.3.4.1.2. Vertical reinforcement must be at least one-half the horizontal reinforcement. Usually this will not govern since a certain amount of vertical reinforcement is needed to resist out-of-plane loads (see the example at the end of this module).

Code 3.3.6, Alternative Approach to Maximum Reinforcement

- **For walls expected to have flexural ductility in plane, provide confined boundary elements in hinging regions (this is another way of preventing toe crushing)**
- **Detailing requirements for boundary elements have yet to be developed**

During the 2005 MSJC cycle, Section 3.3.6 was developed as a "place-holder" to permit the possibility of designing walls with confined boundary elements rather than limiting the amount of flexural reinforcement in them. That section requires detailing requirements for boundary elements to be developed based on experiment. As of 2005, such detailing requirements are not yet available.



The MSJC Specification is referenced by and linked to the MSJC Code. The Specification has three articles governing general provisions, products, and execution, respectively.

Strength Design of Reinforced Masonry Shear Walls

- **Compute factored design moments and shears for in- and out-of-plane loading.**
- **Given practical thickness for wall, design flexural reinforcement as governed by out-of-plane loading.**
- **Design flexural reinforcement as governed by in-plane loading and revise design as necessary.**
- **Check shear capacity using capacity design if required.**
- **Check detailing.**

Now let's look at a specific application of the masonry design provisions that we have seen in this module -- the strength design of reinforced masonry shear walls.

That design involves the following steps, listed here and addressed in more detail in subsequent slides.

Compute factored design moments and shears for in- and out-of-plane loading.

Given practical thickness for wall, design flexural reinforcement as governed by out-of-plane loading.

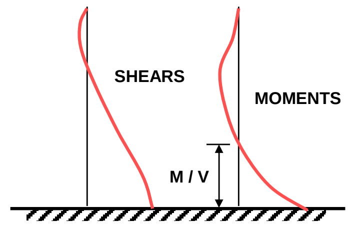
Design flexural reinforcement as governed by in-plane loading, revise design as necessary.

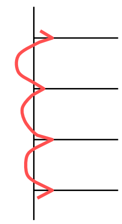
Check shear capacity, using capacity design if required.

Check detailing.

Compute Factored Design Moments and Shears

- **Factored design moments and shears for in-plane loading depend on actions transferred to shear walls by horizontal diaphragms at each floor level.**
- **Factored design moments and shears for out-of-plane loading depend on wind or earthquake forces acting between floor levels**





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The first step is computation of factored design shears and moments.

Factored design moments and shears for in-plane loading depend on actions transferred to shear walls by horizontal diaphragms at each floor level.

Factored design moments and shears for out-of-plane loading depend on wind or earthquake forces acting between floor levels.

Design Flexural Reinforcement as Governed by Out-of-plane Loading

- **Practical wall thickness is governed by available unit dimensions:**
  - **8- by 8- by 16-in. nominal dimensions**
  - **Specified thickness = 7-5/8 in.**
  - **One curtain of bars, placed in center of grouted cells**
- **Practical wall thickness = 7-5/8 in.**
- **Proportion flexural reinforcement to resist out-of-plane wind or earthquake forces**

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The next step is to design the flexural reinforcement as governed by out-of-plane loading, which usually will govern over the reinforcement necessary to resist in-plane loading.

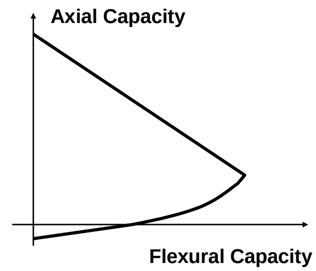
The practical wall thickness is governed by available unit dimensions. Using concrete masonry units, nominal dimensions of 8 by 8 by 16 in. are most common. This implies a specified thickness of 7-5/8 in., and a single curtain of reinforcement, placed in the center of grouted cells.

The specified wall thickness is 7-5/8 in.

Proportion flexural reinforcement to resist out-of- plane wind or earthquake forces. Because axial loads are small, the necessary reinforcement can be estimated by dividing the maximum factored design moment from out-of-plane loads by the product of the specified steel yield strength and the internal lever arm.

Design Flexural Reinforcement as Governed by In-plane Loading

- **Construct moment – axial force interaction diagram**
  - **Initial estimate (more later)**
  - **Computer programs**
  - **Spreadsheets**
  - **Tables**



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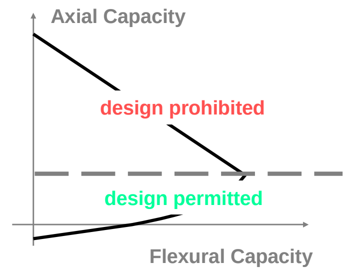
The next step is to design flexural reinforcement as governed by in-plane loading. A moment-axial force interaction diagram can be computed by hand or by spreadsheet. As an initial estimate for hand calculations, the approach of Cardenas and Magura can also be used, and is discussed in a few more slides.

Strict Limits on Maximum Flexural Reinforcement

- **Objective -- Keep compressive stress block from crushing:**
  - **Walls must be below balance point.**
  - **Maximum steel percentage decreases as axial load increases, so that design above balance point is impossible.**



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As noted previously, the 2005 MSJC Code places strict limits on maximum flexural reinforcement. To keep the compressive stress block from crushing, the maximum steel percentage decreases as axial load increases so that design above the balance point is impossible.

Revise Design as Necessary

- **If flexural reinforcement required for out-of-plane moments is less than or equal to that required for in-plane moments, no adjustment is necessary. Use the larger amount.**
- **If flexural reinforcement required for out-of-plane moments exceeds that required for in-plane moments, consider making the wall thicker so that in-plane flexural capacity does not have to be increased. Excess in-plane capacity increases shear demand.**

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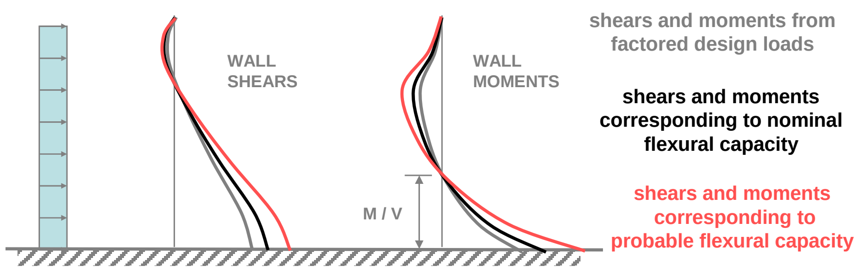
After determining the reinforcement required to resist out-of-plane loads, and the reinforcement required to resist in-plane loads, the greater of the two requirements governs. If out-of-plane requirements considerably exceed in-plane requirements, consider making the wall thicker. In-plane over-capacities in flexure can lead to an unacceptable increase in shear demand if capacity design is used (see subsequent example).

Check Shear Capacity (1)

- **Elastic structures or those with considerable shear overstrength:**
  - **Compute factored design shears based on factored design actions.**
- **Inelastic structures:**
  - **Compute design shears based on flexural capacity**



In checking shear capacity, the first step is to estimate the design shear.

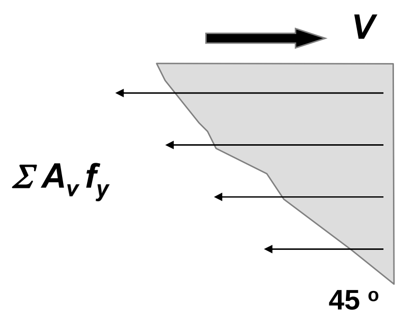
If the structure is essentially elastic, factored design shears should be computed based on factored design actions.

If the structure is expected to have significant inelastic flexural deformation, then capacity design should be used. Design shears should be computed based on flexural capacity.

In the above figure, the gray diagrams represent shears and moments from factored design loads. The black diagrams represent shears and moments corresponding to nominal flexural capacity. They are the gray diagrams,divided by the capacity reduction factor for flexure. The red diagrams represent the shears and moments corresponding to the probable flexural capacity. They are the black diagrams, multiplied by the ratio of the probable yield strength of the flexural reinforcement divided by the specified yield strength and also multiplied by the ratio of the probable area of reinforcement divided by the required reinforcement.

Check Shear Capacity (2)

- $V_n = V_m + V_s$
- V_m depends on $(M_u / V_u d_v)$ ratio
- $V_s = (0.5)\ A_v f_y$ (note efficiency factor)



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As discussed previously, shear resistance is calculated similarly to that of reinforced concrete.

Shear Resistance from Masonry, V_m (1)

- V_m depends on $(M_u / V_u d_v)$ ratio and axial force
- $(M_u / V_u d_v)$ need not be taken greater than 1.0

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u\, d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u$$

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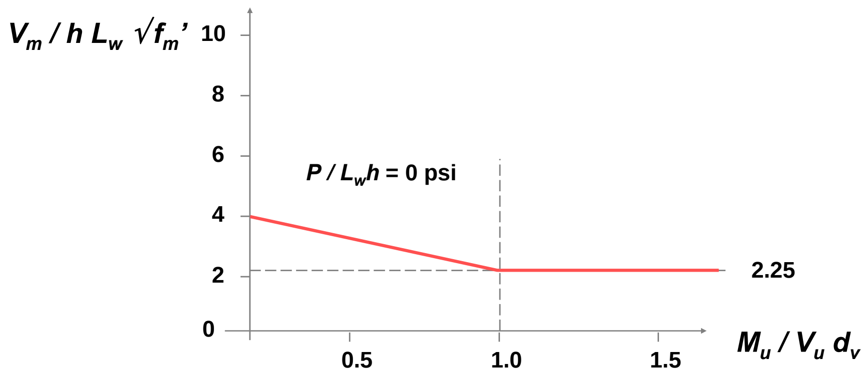
As discussed previously, V_m depends on aspect ratio.

Shear Resistance from Masonry, V_m (2)



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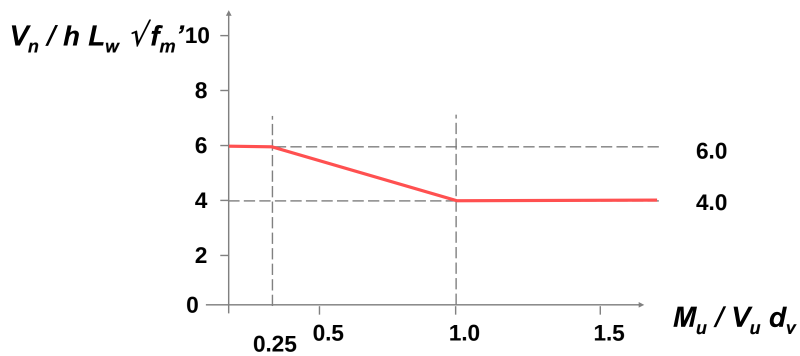
Shear resistance from masonry varies with aspect ratio as shown in this graph.

Total Shear Resistance, V_n (1)

- Resistance from masonry (V_m) plus resistance from reinforcement (V_s)
- Upper limit on V_m depends on ($M_u / V_u d_v$) ratio



As discussed previously, total shear resistance is limited to prevent diagonal crushing of masonry.

Check Detailing

- Cover
- Placement of flexural and shear reinforcement
- Boundary elements not required:
  - Ductility demand is low
  - Maximum flexural reinforcement is closely controlled

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Finally, the designer must check detailing, including cover and placement of flexural and shear reinforcement. Boundary elements are not required because ductility demand is usually low and maximum flexural reinforcement is closely controlled.

Detailing (1)

- **Cover:**
  - **Automatically satisfied by putting reinforcement in grouted cells**
- **Placement of flexural and shear reinforcement:**
  - **Minimum flexural reinforcement and spacing dictated by Seismic Design Category**
  - **Flexural reinforcement placed in single curtain. Typical reinforcement would be at least #4 bars @ 48 in.**
  - **Place horizontal reinforcement in single curtain. Typical reinforcement would be at least #4 bars @ 48 in.**
  - **Add more flexural reinforcement if required, usually uniformly distributed.**

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Detailing requirements are usually not difficult to satisfy.

Cover requirements are automatically satisfied by putting reinforcement in grouted cells.

Flexural and shear reinforcement must comply with the minimum flexural reinforcement and spacing dictated by the Seismic Design Category.

Flexural reinforcement is normally placed in single curtain.

Typical reinforcement would be at least #4 bars @ 48 in.

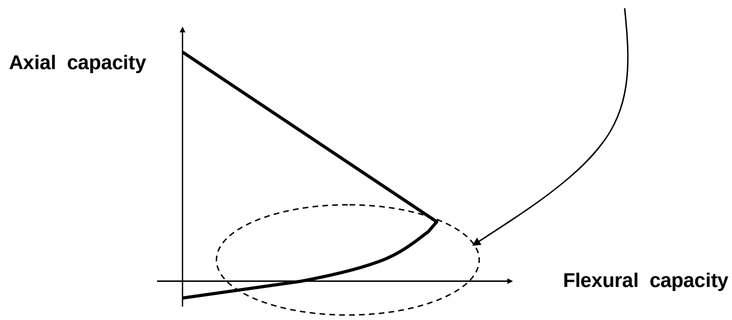
Horizontal reinforcement is also placed in the same single curtain.

Typical reinforcement would be at least #4 bars @ 48 in.

Additional reinforcement (usually uniformly distributed) can be added if required to resist in-plane flexure.

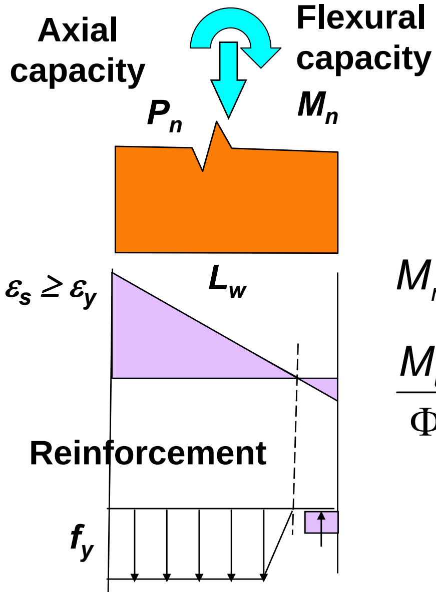
Flexural Strength of Lineal Walls (1)

- **Approximation to moment-axial force interaction diagram for low axial load**



Now let's look briefly at Cardenas and Magura's approximation for flexural strength.

Flexural Strength of Lineal Walls (2)



- Sum moments about centroid of compressive stress block

$$M_n \approx 0.9 A_s\, f_y \frac{0.9 L_w}{2} + P_n \frac{0.9 L_w}{2}$$

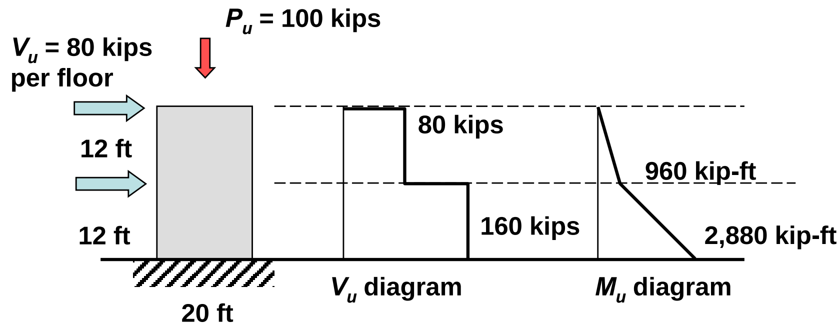
$$\frac{M_u}{\Phi} \approx 0.41\, A_s\, f_y\, L_w + 0.45 \frac{P_u}{\Phi} L_w$$

- Given M_u and P_u, solve for A_s

At nominal capacity, internal forces are in equilibrium with axial capacities and flexural capacities, referred to the plastic centroid. Using the free body at left, sum moments about the centroid of the compressive stress block. The moment M_n is resisted by tensile reinforcement (acting over approximately 90% of the depth of the section and acting at about half of that 90% away from the compression resultant) and by applied axial load (acting at about half of 90% of the depth away from the compression resultant). Given M_u and P_u, solve for A_s.

Design Example (1)

- **Carry out the preliminary design of the masonry shear wall shown below. Use f_m' = 1500 psi.**



Now let's use those principles to carry out the preliminary design of a masonry shear wall. It measures 20 ft in plan; has two stories; and has factored lateral loads of 80 kips on each floor. Factored axial load is 100 kips. Factored design shear and moment diagrams are shown to the right of the slide.

Design Example (2)

- **Assume out-of-plane flexure is OK.**
- **Check in-plane flexure using initial estimate.**

$$\frac{M_u}{\Phi} \approx 0.41\, A_s\, f_y\, L_w + 0.45\, \frac{P_u}{\Phi}\, L_w$$

$$\frac{2880\ kip - ft \times 12\ in./ft}{0.90} \approx 0.41\, A_s \times 60\ ksi \times 240\ in. + 0.45\, \frac{100\ kips}{0.9} \times 240\ in.$$

$$38,400 \approx 5,900\, A_s + 12,000$$

$$A_s \approx 4.47\ in.^2$$

- **This is equivalent to #5 bars @ 12 in.**

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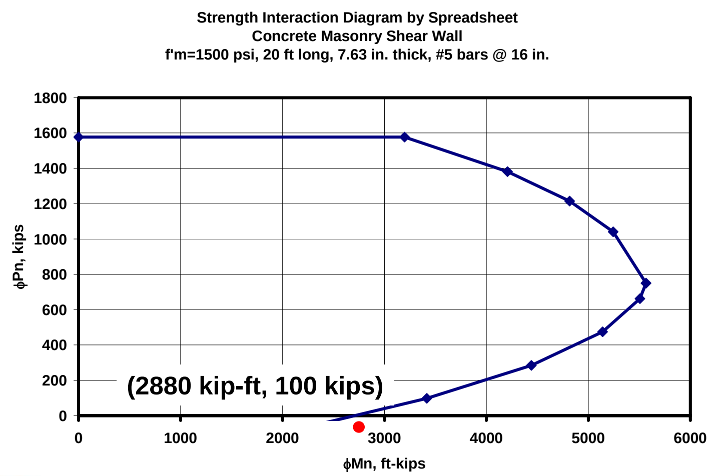
Assuming that out-of-plane flexure is satisfied, the required area of flexural reinforcement can be calculated. Using the approximate formula derived previously, the required steel area is predicted as 4.47 in.^{2}. This is equivalent to #5 bars @ 12 in.

Design Example (3)

- **Refine flexural reinforcement using spreadsheet-based interaction diagram -- use #5 bars @ 16 in.**



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Using a spreadsheet-based program, calculate the interaction diagram for different assumed steel areas. The diagram shown is for #5 bars @ 16 in. The combination of factored actions (2880 kip-ft, 100 kips) lies within the diagram of design capacities.

Design Example (4)

- **Now check shear:**

$$\frac{M_u}{V_u\, d} = \frac{2{,}880\ kip-ft}{160\ kips \cdot 20 \cdot 0.8\ ft} = 1.13$$

$$V_m = 2.25\sqrt{f'_m}\ h\, L_w + 0.25P$$

$$V_m = 2.25\sqrt{1500} \cdot 7.63\ in. \cdot 240\ in. + 0.25 \cdot 100\ kips$$

$$V_m = 159.6\ kips + 25.0\ kips = 184.6\ kips$$

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Now check the shear capacity. Using a fully grouted wall, calculate the design shear capacity from masonry alone. By convention, *d* is taken as 0.8 L_w .

Design Example (5)

- **Compute required shear reinforcement, including capacity design:**

$$V_u = 160\ kips\left(\frac{1.25\,M_n}{M_u}\right)$$

$$V_u = 160\ kips\left[\frac{1.25 \times 3427\ kip-ft \times \left(\frac{1}{0.9}\right)}{2880\ kip-ft}\right] = 160\ kips \times 1.65$$

$$V_n^{required} = \frac{V_u}{\Phi} = \frac{V_u}{0.8} = \frac{160\ kips \times 1.65}{0.8} = 2.07\,V_u \leq 2.5\,V_u$$

Now continue the shear design, including the effects of capacity design. The factored design shear is required to be 1.25 times the shear associated with the development of the nominal flexural capacity of the section. In this problem, the nominal flexural capacity is the design flexural capacity at the given factored axial load of 100 kips, or 3427 kip-ft (from the interaction diagram), divided by the *Φ* factor of 0.9. The shear associated with that flexural capacity is the calculated design shear of 160 kips multiplied by the ratio of 1.25 times the nominal flexural capacity, divided by the factored moment. The design shear is then 1.65 times 160 kips. The nominal capacity need not exceed 2.5 times the design shear however.

Design Example (6)

- **Compute required shear reinforcement including capacity design:**

$$V_s^{required} \geq \frac{V_u}{\Phi} - V_m = \frac{160 \times 1.65}{0.8} - 184.6 = 145.4\ kips$$

$$V_s^{required} = 2 \times A_v f_y \frac{d}{s}$$

$$A_v^{required} = \frac{V_s^{required} s}{d\ f_y} = \frac{145.4\ kips \cdot 16\ in.}{0.8 \cdot 240\ in. \cdot 60\ ksi} = 0.202\ in.^2$$

- **Use #4 bars every 16 in.**

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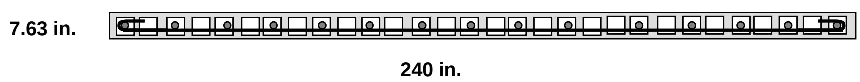
The required shear capacity from steel is the design shear of 1.65 times 160 kips divided by the Φ factor of 0.8 and minus the available nominal capacity from masonry, or 145.4 kips. This is consistent with shear reinforcement of 0.202 in.^{2} every 16 in. Use #4 bars every 16 in. horizontally.

Design Example (7)

- Now finish detailing:
  - Use #5 bars @ 16 in. vertically
  - Use #4 bars @ 16 in. horizontally
  - Hook #4 horizontal bars around end #5 vertical bars



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Finally, finish detailing the wall. Vertical reinforcement is placed in grouted cells. Horizontal reinforcement is placed in the space made available by knocking out part of the cross-webs of the units. Shear reinforcement is required to be hooked (180-degree hooks) around the extreme-fiber flexural reinforcement.

Web sites for more information

- BSSC = www.bssconline.org
- TMS = www.masonrysociety.org
- ACI = www.aci-int.org
- ASCE / SEI = www.seinstitute.org
- MSJC = www.masonrystandards.org

Further information on the topics presented here is given in these web sites.

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- Some of the material was originally prepared by Prof. Klingner for The Masonry Society and is used with their permission.