Ce 479
Reinforced Masonry
Fall 2004

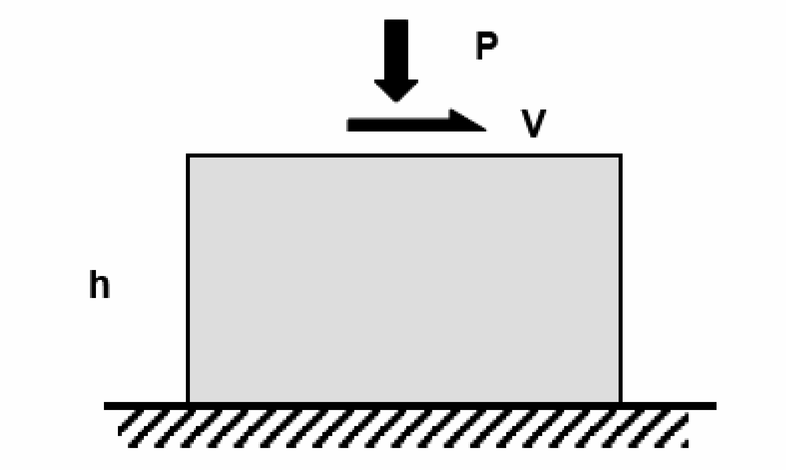
DESIGN OF REINFORCED MASONRY SHEAR WALLS

Introduction

Next, let us consider the behavior and design of reinforced masonry shear walls.

Design Steps

Reinforced masonry shear walls must be designed for the effects of:
1) gravity loads from self-weight, plus gravity loads from overlying roof or floor levels; and
2) moments and shears from in-plane shear loads
Actions are shown below. Either strength or allowable-stress design can be used.



DESIGN OF REINFORCED SHEAR WALLS USING STRENGTH PROCEDURES

Flexural capacity of reinforced shear walls using strength procedures is calculated using moment-axial force interaction diagrams as discussed in the lecture on masonry beam-columns. In contrast to the elements addressed in that lecture, a shear wall is subjected to flexure in its own plane rather than out-of-plane. It therefore usually has multiple layers of flexural reinforcement. Computation of moment-axial force interaction diagrams for shear walls is much easier using a spreadsheet.

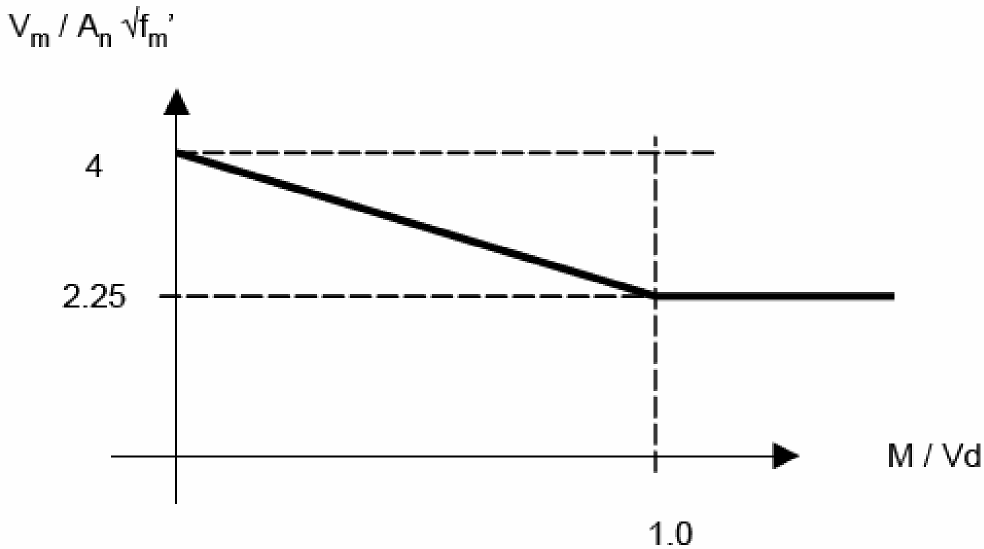
From the 2002 MSJC Code, Section 3.2.4.1.2, nominal shear strength is the summation of shear strength from the masonry and shear strength from the shear reinforcement.

$$V_n = V_m + V_s$$

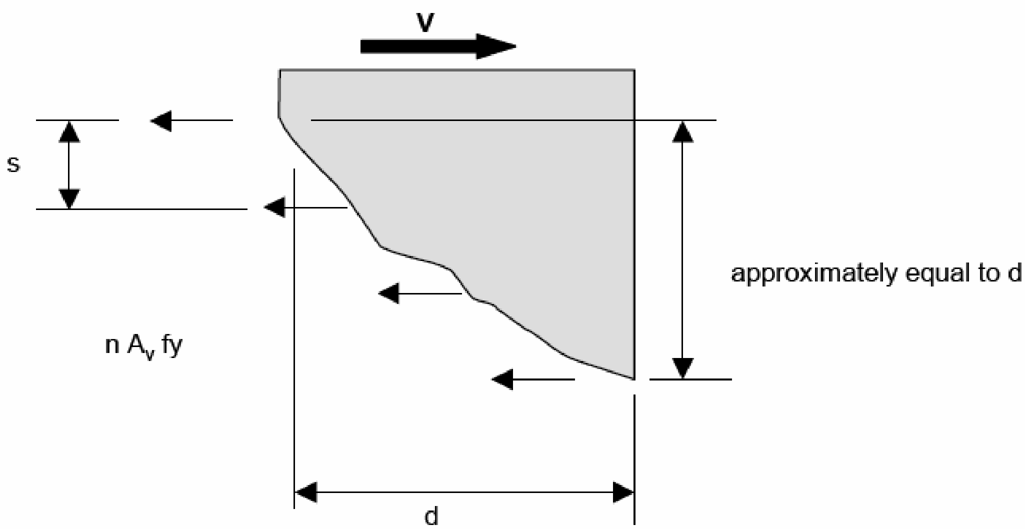
From the 2002 MSJC Code, Section 3.2.4.1.2.1,

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right]A_n\sqrt{f_m'} + 0.25P$$

As *(M/Vd_v)* increases, V_m decreases. Because *(M/Vd_v)* need not be taken greater than 1.0 (2002 MSJC Code, Section 3.2.4.1.2.1), the most conservative (lowest) value of V_m is obtained with *(M/Vd_v)* equal to 1.0.



Just as in reinforced concrete design, this model assumes that shear is resisted by reinforcement crossing a hypothetical failure surface oriented at 45 degrees:



The nominal resistance from reinforcement is taken as the area associated with each set of shear reinforcement, multiplied by the number of sets of shear reinforcement crossing the hypothetical failure surface. Because the hypothetical failure surface is assumed to be inclined at 45 degrees, its projection along the length of the member is approximately equal to *d*, and number of sets of shear reinforcement crossing the potential failure crack can be approximated as (d/s).

$$V_s = A_v f_y n$$

$$V_s = A_v f_y \left(\frac{d}{s}\right)$$

The actual failure surface may be inclined at a larger angle with respect to the axis of the wall, however. Also, all reinforcement crossing the failure surface may not yield. For both these reasons, the assumed resistance is decreased by an efficiency factor of 0.5. From the 2002 MSJC Code, Section 3.2.4.1.2.2 (Page C-34)

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

Finally, because shear resistance really comes from a truss mechanism in which horizontal reinforcement is in tension, and diagonal struts in the masonry are in compression, crushing of the diagonal compressive struts is controlled by limiting the total shear resistance V_n, regardless of the amount of shear reinforcement:

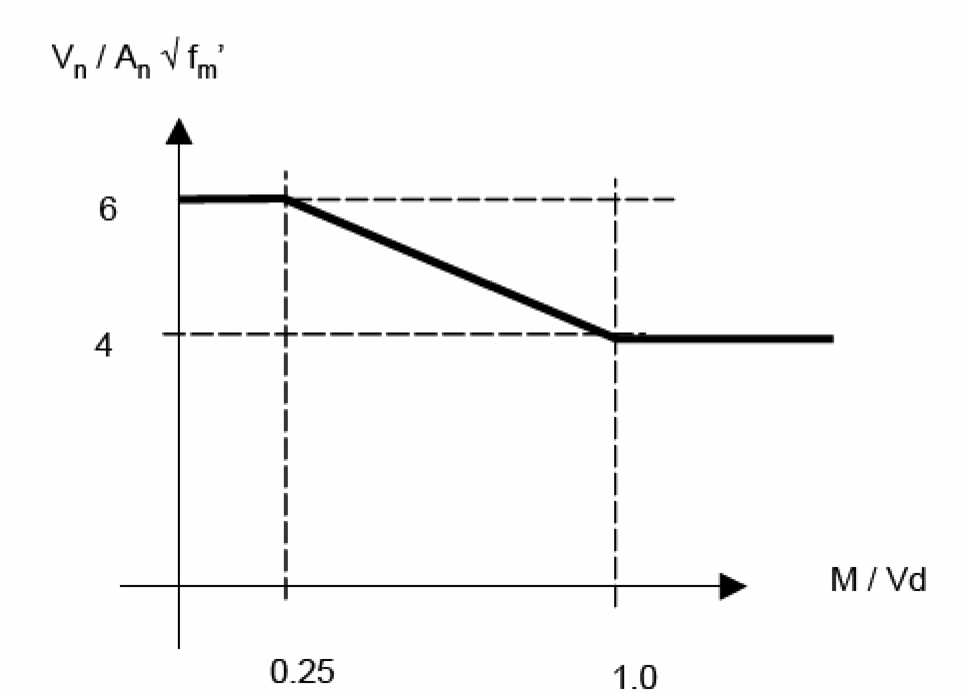
For (M/Vd) ≤ 0.25

$$V_n = 6A_n\sqrt{f_m^{'}} \quad ;$$

and for *(M/Vd)* > 1.00,

$$V_n = 4A_n\sqrt{f_m^{'}} \quad .$$

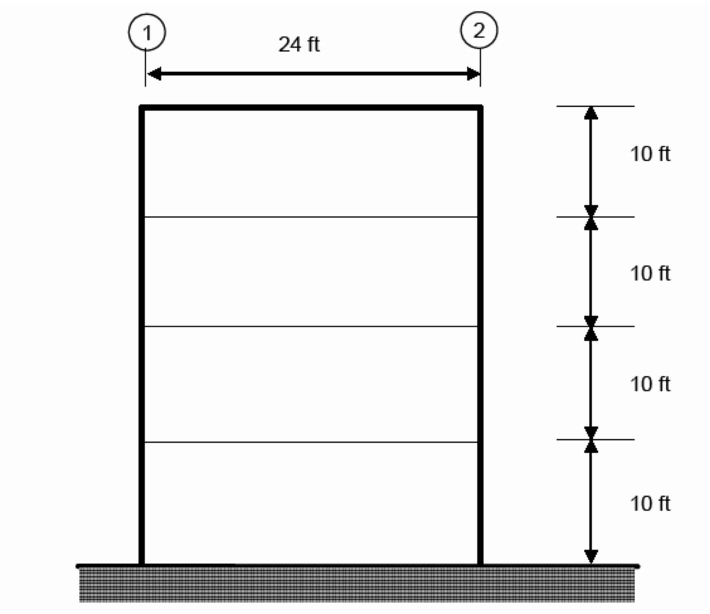
Interpolation is permitted between these limits.



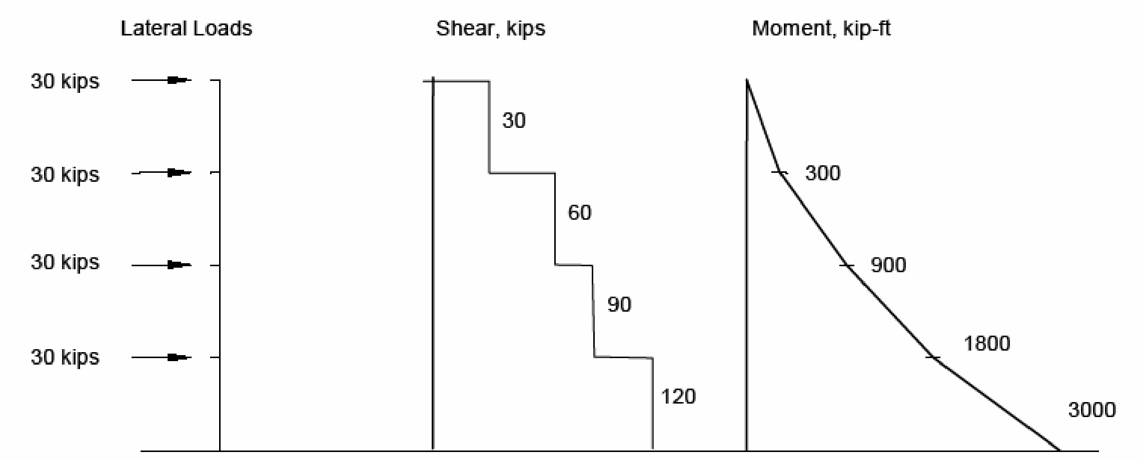
If these upper limits on V_n are not satisfied, the cross-sectional area of the section must be increased.

Example: Strength Design of Reinforced Clay Masonry Shear Wall

Consider the masonry shear wall shown below:



Design the wall. Unfactored in-plane lateral loads at each floor level are due to earthquake, and are shown below, along with the corresponding shear and moment diagrams.



Assume an 8-in. nominal clay masonry wall, grouted solid, with Type S PCL mortar. The total plan length of the wall is 24 ft (288 in.), and its thickness is 7.5 in. Assume an effective depth d of 285 in.

	Clay Masonry
Unit Strength	*6,600*
Mortar	*Type S*
f'_m *or* f_g (psi)	*2,500*

Reinforcement = Grade 60; $E_s = 29 \times 10^6$ psi

Unfactored axial loads on the wall are given in the table below.

Level	DL	LL
(Top of Wall)	(kips)	(kips)
4	90	15
3	180	35
2	270	55
1	360	75

Use ASCE 7-02 -Basic Strength Load Combination 7- : 0.9*D* + 1.0*E*

Check shear for assumed wall thickness. By Section 3.2.4.1.2 of the 2002 MSJC Code,

$$V_n = V_m + V_s$$

M = 3,000 x 12 x 1,000 in.-lb = 36.0 x 10^6 in.-lb

V = 120,000 lb and $d_v = 285$ *in*

$$M / Vd_v = \frac{36 \times 10^6 \text{ in.-lb}}{120{,}000 \text{ lb } (285 \text{ in.})} = 1.05$$

<u>The 2002 MSJC Code requires that this check be carried using unfactored loads.</u> The MSJC is considering changing this to factored loads for the 2005 Code.

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right] A_n \sqrt{f'_m} + 0.25P$$

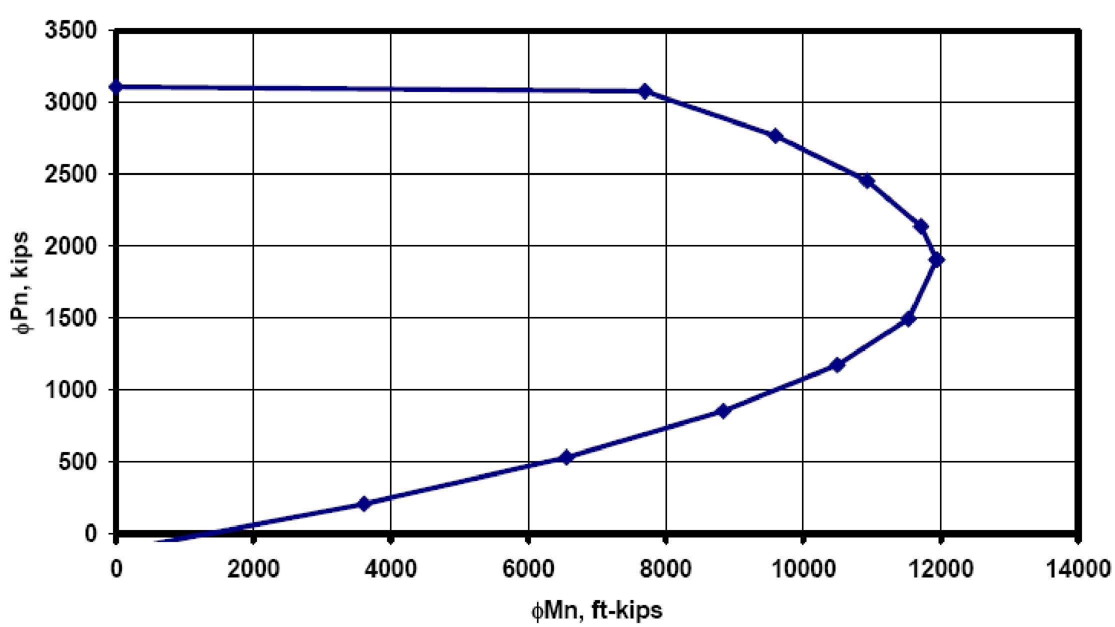
$$V_m = [4.0 - 1.75(1.0)]\ 7.5\ in. \times 285\ in.\left(\sqrt{2500\ \text{psi}}\right) + 0.25\ (360{,}000\ \text{lb})$$

$$V_m = 240.5 + 90.0\ kips = 330.5\ kips$$

$$\phi V_n > V_u \qquad \phi = 0.80 \qquad V_n = V_m = 330.5\ kips$$

$$0.80(330.5\ kips) = 264.4\ kips \geq V_u = 120\ kips$$

Shear design is satisfactory so far, even without shear reinforcement. Code Section 3.1.3 will be checked later. Now check flexural capacity using a spreadsheet-generated moment-axial force interaction diagram. Try #5 bars @ 4 ft.



At a factored axial load of 0.9D, or 0.9 x 360 kips = 324 kips, the design flexural capacity of this wall is about 4000 ft-kips, and the design is satisfactory for flexure.

Now check Code Section 3.1.3. of MSJC 2002. First try to meet the capacity design provisions of that section. At an axial load of 324 kips, the nominal flexural capacity of this wall is 4000 ft-kips, divided by the strength reduction factor of 0.9, or 4,444 ft-kips. The ratio of this nominal flexural capacity to the factored design moment is 4,444 divided by 3,000, or 1.48. Including the additional factor of 1.25, that gives a ratio of 1.85.

3.1.3 *Design strength*

Masonry members shall be proportioned such that the design strength equals or exceeds the required strength. Design strength is the nominal strength multiplied by the strength reduction factor, ϕ, as specified in Section 3.1.4.

The design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the member, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

3.1.3.1 *Seismic design provisions* — At each story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls. Along each column line at a particular story level, at least 80 percent of the lateral stiffness shall be provided by lateral-force-resisting walls.

Exception: Where seismic loads are determined based on a seismic response modification factor, R, not greater than 1.5, piers and columns are permitted to be used to provide seismic load resistance.

$$\phi V_n \geq 1.85 V_u$$

$$V_n \geq \frac{1.85}{\phi} V_u = \frac{1.85}{0.8} V_u = 2.31\, V_u = 2.31 \times 120 = 277.5\ kips$$

The wall is satisfactory without shear reinforcement. Prescriptive seismic reinforcement (for example, for SDC D) will probably require #5 bars horizontally @ 24 in.

$$V_n = V_m + V_s = 330.5\ kips + A_v\ f_y\ \frac{d}{s} = 330.5 + 0.31\ in.^2 \times 60{,}000\ psi\ \frac{285\ in.}{24\ in.}$$

$$V_n = 330.5 + 220.9\ kips = 551.4\ kips$$

Prescriptive seismic reinforcement is sufficient for shear. Use #5 bars at 2 ft.

Summary:

Use #5 @ 4 ft vertically, #5 @ 2 ft horizontally.

MINIMUM AND MAXIMUM REINFORCEMENT RATIOS FOR FLEXURAL DESIGN BY THE STRENGTH APPROACH

The strength design provisions of the 2002 MSJC Code include requirements for minimum and maximum flexural reinforcement.

Minimum Flexural Reinforcement by 2002 MSJC Code

The 2002 MSJC Code has no global requirements for minimum flexural reinforcement. For design of shear walls for in-plane loads, however, Section 3.2.6.2 requires that the vertical reinforcement be not less than one-half the horizontal reinforcement.

Maximum Flexural Reinforcement by 2002 MSJC Code

In an innovative departure from previous codes for masonry and concrete, the 2002 MSJC Code has a maximum reinforcement requirement (Section 3.2.3.5) that is intended to ensure ductile behavior over a range of axial loads. As compressive axial load increases, the maximum permissible reinforcement percentage decreases. For compressive axial loads above a critical value, the maximum permissible reinforcement percentage drops to zero, and design is impossible unless the cross-sectional area of the element is increased.

For walls subjected to in-plane forces, for columns, and for beams, the provisions of the 2002 MSJC Code set the maximum permissible reinforcement based on a critical strain condition in which the masonry is at its maximum useful strain, and the extreme tension reinforcement is at 5 times its yield strain. For walls subjected to out-of-plane forces, the critical strain gradient has a strain in the extreme tension reinforcement of 1.3 times its yield strain. Less restrictive requirements are imposed on members not required to undergo inelastic deformations.

The critical strain condition for walls loaded in-plane, and for columns and beams, is shown below, along with the corresponding stress state. The parameters for the equivalent rectangular stress block are the same as those used for conventional flexural design. The height of the stress block is $0.80 f_m'$, and the depth is $0.80\,c$. The reinforcement is conservatively assumed to be at $1.25\,f_y$, to allow for possible steel overstrength and strain hardening. This assumption is conservative because its effect is to reduce the maximum amount of reinforcement that can be used.