V–6. Methods of Structural Design for Interaction Forces on Masonry Walls

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ABSTRACT

The structural design of a masonry wall subjected to load and moment has been a major concern of designing engineers. The approach has been to use the old classic reinforced concrete unit equations and to consider each load and moment independently.

This paper presents several methods for the analysis and design of walls subjected to vertical load and moment and compares the results for each of the methods. The unity equation is considered to be the limiting feature for design in accordance with the building code and current practice. The various examples of design illustrate the results obtained. The designer should take into consideration the variation in results and should evaluate and use the method in keeping with his judgment.

INTRODUCTION

In most structures, many of the walls are subjected to both vertical load and bending load. This is particularly true of bearing walls that carry the loads of floors and roofs above and are subjected to lateral forces due to wind or earthquake. Lateral loads may also be imposed upon walls due to earth pressure and water pressure, and moments may be caused in the wall by means of imposed eccentric loads or moments on walls themselves. Walls acting as shear resisting elements for lateral forces will be subjected to overturning forces and shear inducing moments on the wall in addition to the normal vertical load. (See Figure 1.)

UNITY EQUATION

The basis of design for combined load and moment is the unity equation. This is a carry-over from the design of reinforced concrete elements based upon working stress design and formula 18 of the 1956 ACI code. This is a simplified equation compared with previous equations in the 1951 ACI code.

The unity equation is the sum of the ratios of the actual stress to the allowable stress whether due to axial compression or flexural compression. The allowable value of flexural compression is 1/3 f′m while the allowable value for axial compression in walls is 0.2 f′m times the h/t reduction factor. The allowable value for axial compression is less because of creep, plastic flow, and the sustained load on the total cross section. In flexural compression, only the exterior wythe or face shell is stressed to the maximum. If there is no bending moment or flexural stresses, the limit is the axial load and the limiting ratio equal to one. If there is no vertical load and only bending moment, the limiting condition is flexure and the ratio is equal to one. Any combination of these two, load and moment, follows a straight line variation between them. (See Figure 2.)

Unity equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

where

f_a = actual unit stress due to the load determined from total axial load and effective cross-sectional area

$f_a = \frac{P}{bt}$ psi (walls)

bt = actual cross-sectional solid area of wall

F_a = maximum allowable unit stress if the member were carrying axial load only, psi

F_a = [0.2 f′m or 1/2 (0.2 f′m)] psi × R (walls)

R = h/t reduction factor, decimal $\leq$ 1.00

F_b = maximum allowable flexural stress if member were carrying bending load only = 0.33 f′m or 1/2 (0.33 f′m) psi

f_b = actual bending stress and must not exceed

$$\left(1 - \frac{f_a}{F_a}\right) F_b \text{ psi or}$$

$$f_b = (1 - C)\, F_b \text{, where } C = \frac{f_a}{F_a}$$

$$f_b = \frac{M}{bd^2} \times \frac{2}{jk}$$

When the loads or moments are due to temporary instantaneous forces such as wind, earthquake, etc., the combination of the ratios can be made equal to 1.33 instead of one. This is allowing a 1/3 increase in stresses due to temporary short term loading.

Some engineers, in the interest of conservativism, only increase by 1/3 the stress that is caused by the temporary short term loading. Therefore, if the moment is caused by earthquake or wind, the F_b, which is the allowable flexural compression stress, is increased by 1/3. In any case, the resulting design should never be less than when no temporary earthquake or wind loads are on the structure.

METHODS OF DESIGN FOR INTERACTION OF LOAD AND MOMENT

For the present state-of-the-art any method of design should comply with the two basic requirements of:

a. Plane sections remain plane after bending.
b. Stress is proportional to strain which is proportional to the distance from the neutral axis.

There are a number of methods in current use, four of which are presented in this paper.

Method 1.

This method is useful when considering walls where the load is perpendicular to the plane of the wall. It takes into account each force and analyzes each independently. The stresses for vertical load are considered and then independently the stress for moment is considered.

Using the stresses caused by each of the conditions the unity equation is checked. The reinforcing steel is then designed as if there were flexural stress only and no vertical load on the wall. (See Figure 3 for an illustration of Method 1.)

Modified Method 1.

The modification of Method 1 above assumes that the vertical load counteracts the tension caused by moment on the wall up to the point where the tension stress exceeds the vertical load compression stress. This limiting condition is when the eccentricity of the load is equal to t/6 or $\ell/6$, as in Figure 4. The initial determination of flexural stress can be by assuming a homogeneous section and using the familiar equation

$$f_b = \frac{M}{S} \quad \text{or} \quad f_b = \frac{Mc}{I}.$$

When the virtual eccentricity is in excess of the kern eccentricity there is tension on the section. When the tension stress exceeds the compression stress or the allowable tensile stress for the masonry, consider each condition of vertical load and moment independently and proceed similar to Method 1.

Method 2.

This method evaluates the sum of the vertical forces and the sum of the moments to equal zero. The limiting compression stress in the masonry is assumed to be the sum of the actual axial compressive stress plus maximum allowable flexural compressive stress that will satisfy the unity equation. With this value and the applied loads or moments the statics of the section are evaluated based upon sum of the vertical forces equal zero and sum of the moments equal zero.

The size of the compressive stress block is determined and, by ratios of distances and the theory of straight line stress and strain variation, the stress in the steel is calculated. From this, the required area of steel is computed based upon the tension force divided by the stress in the steel.

This method has many advantages because it can take full advantage of the compressive stress capacity of the masonry and thus recognize that perhaps reinforcing steel may not be needed. Under these circumstances, even though the vertical eccentricity is in excess of the kern eccentricity, or e exceeds t/6 or $\ell/6$, the resultant compression force in the masonry may be coincidental with the virtual eccentric vertical load. If they are coincidental, there would be no tension force and thus no requirement for tension steel. (See Figure 5.)

Method 3.

In the design of piers and shear walls which are subjected to wind or earthquake lateral shear forces are induced into the walls. These shear forces produce overturning moments on the wall and may be acting in either direction. The vertical load on these walls due to floors and structure above produce axial compressive stress. The overturning moment produces flexural compressive stress in either direction. Accordingly, when the section is analyzed, it may require tension steel, and if it does, it would be required on either end of the wall.

Considering that the steel is there anyway, many times it is beneficial to take into account the presence of this steel acting as compression steel with the masonry to resist the high compressive forces. The steel on the other end of the wall resists the tension force. This analysis is similar to a beam with an axial load and moment on it. It takes into account that the steel can assist the masonry in compression and is included in the design as a transformed area of steel equal to $(2n-1)$, which is similar to the technique used in the elastic design of compression steel in concrete.

The limiting allowable stress in the masonry in compression is obtained using the unity equation. Figure 6 is a graphical representation of Method 3 and the inclusion of the steel in one end as compression steel to assist the masonry.

Method 4.

This is a method that has been in frequent use. It applies the classic consideration of a homogeneous section to determine the stresses on that section due to axial load and moment, or $f = P/A \pm Mc/I$. The evaluation of the moment of inertia of the section is based upon an uncracked homogeneous section.

If the vertical axial load or stress, P/A, is greater than the flexural stress, Mc/I or M/S, it is a section that does act as an homogeneous section because there is compression on the total area. Under these circumstances, no tension steel would be required and the section would perform as a compression element.

If the axial compressive stress, P/A, is less than the flexural stress, Mc/I, then there would be tension on the section and should be reinforced to resist this tension force. (See Figure 7.)

The axial stress and flexural stress as determined by P/A and Mc/I would be checked against the maximum allowable stresses to insure compliance with the unity equation. This method is useful in determining requirements for sections that have flanges and intersecting walls on them.

In this method the axial and the flexural stresses are determined for an homogeneous section and the resulting total compression and total tension stresses computed.

The method then assumes that the total tension force thus obtained may be resisted by the reinforcing steel stressed at the maximum allowable steel stress. Furthermore, an ad-

justment is made recognizing the steel is located at a more advantageous position than the centroid of the assumed masonry tensile stresses.

EQUATIONS AND ILLUSTRATIONS OF EACH METHOD

Each of the methods of design will be shown using the following example as shown in Figure 8.

Given a 9.5″ thick reinforced grouted brick shear wall with Grade SW brick, f′m = 1800 psi, n = 16.7, intermediate grade reinforcing steel, f_s = 20,000 psi with a total uniform vertical load of 95 kips, and a moment due to lateral (wind or seismic) load of 450 ft kips.

Method 1. (See Figure 9.)

Proceed as follows and solve for:

a. Kern distance, $e_k = \frac{\ell}{6}$ or $\frac{I}{A_y} = \frac{S}{A}$; $e_k = \frac{96''}{6} = 16''$.

b. Virtual eccentricity, $e = \frac{M}{P}$; $e = \frac{450 \times 12}{95} = 56.8''$.

c1. If $e \leq e_k$ minimum reinforcement is required.

c2. If $e > e_k$ design reinforcement for flexural stresses.

$$e > e_k;\ 56.8'' > 16''.$$

d1. Actual axial stress,

$$f_a = \frac{P}{\ell t};$$

$$f_a = \frac{95 \times 1000}{96 \times 9.5} = 104 \text{ psi.}$$

d2. Flexural stress assuming uncracked section,

$$f_b = \frac{Mc}{I} = \frac{M}{S} = \frac{6M}{t\ell^2};$$

$$f_b = \frac{6 \times 450{,}000 \times 12}{9.5 \times 96^2} = 370 \text{ psi.}$$

d3. The flexural tensile stress, 370 psi, exceeds the axial compressive stress of 104 psi and the allowable masonry tensile stress of 12 psi. Therefore, the section is assumed cracked.

e. h/t reduction factor,

$$R = \left[1 - \left(\frac{h}{40t}\right)^3\right]$$

$$= \left[1 - \left(\frac{180}{40 \times 9.5}\right)^3\right];$$

$$R = 0.894.$$

f. Maximum allowable axial compressive stress,

$$F_a = 0.2\, f'm\, R = 0.2 \times 1800 \times 0.894 = 322 \text{ psi.}$$

Ratio of axial stress,

$$C = \frac{f_a}{F_a} = \frac{104}{322} = 0.323.$$

g. Maximum allowable flexural compressive stress,

$$F_b = 1/3\, f'm = 1/3\,(1800) = 600 \text{ psi.}$$

h. Maximum allowable flexural compressive stress that will satisfy the unity equation

$$f_b = (1 \text{ or } 1.33 - C)\, F_b$$

$$= (1.33 - 0.32)\, 600 = 604 \text{ psi.}$$

i. Compute flexural coefficient,

$$K = \frac{M}{F} = \frac{M}{bd^2} = \frac{12{,}000M}{bd^2}$$

$$K = \frac{12{,}000 \times 450}{9.5 \times 88^2} = 73.4$$

j. With the flexural coefficient K = 73.4 and the maximum allowable flexural stress f_b = 604 psi, determine the steel ratio p from the K vs p diagram found in *Reinforced Masonry Engineering Handbook,* J. E. Amrhein, Third Edition, diagram F6, page 335.

Enter Diagram F6 with K = 73.4 and f = 604 psi, read p = 0.0030, steel stress of f_s = 26,700 psi governs f_b = 590 psi.

k. Determine the area of tension steel,

$$As = pbd = 0.003 \times 9.5 \times 88 = 2.5 \text{ sq in.}$$

Modified Method 1.

The flexural tensile stress, 370 psi, from the equation f_b = Mc/I exceeds the axial compressive stress, f_a = P/A = 104 psi, therefore this method is not applicable.

Method 2.

Evaluation of forces based on static equilibrium of $\Sigma F_v = 0$ and $\Sigma M = 0$. Given: (See Figure 10.) Length of wall (ℓ inches); thickness of wall (t inches); distance to steel (d inches); distance to steel (d′ inches); axial load (P pounds); compression force (C pounds); tension force (T pounds); moment (M foot pounds); steel stress (f_s psi); masonry stress (f_m psi); height of wall (h feet); compression force ($C = 1/2\, t\, kd\, f_m$); and tension force ($T = C - P$).

Development of Equations for Method 2.

Taking the sum of the moments about the centerline or axis of the vertical load:

$$C\left(\frac{\ell}{2} - \frac{kd}{3}\right) + T\left(\frac{\ell}{2} - d'\right) - M = 0; \text{ but, } T = C - P;$$

$$C\left(\frac{\ell}{2} - \frac{kd}{3}\right) + (C - P)\left(\frac{\ell}{2} - d'\right) - M = 0.$$

Substituting for $C = 1/2\, t\, kd\, f_m$,

$$(1/2\, t\, kd\, f_m)\left(\frac{\ell}{2} - \frac{kd}{3}\right) + (1/2\, t\, kd\, f_m - P)\left(\frac{\ell}{2} - d'\right) - M = 0$$

$$1/4\, t\, f_m \ell kd - 1/6\, t\, f_m (kd)^2 + 1/2\, t\, kd\left(\frac{\ell}{2} - d'\right) f_m - P\left(\frac{\ell}{2} - d'\right) - M = 0$$

$$1/4\ t\ f_m \ell kd - 1/6\ t\ f_m (kd)^2 + 1/4\ t\ f_m \ell kd - 1/2\ t\ f_m d'kd - P\left(\frac{\ell}{2} - d'\right) - M = 0.$$

Change signs and combine terms,

$$1/6\ t\ f_m (kd)^2 - 1/2\ t\ f_m (\ell - d')\ kd + P\left(\frac{\ell}{2} - d'\right) + M = 0.$$

Solving this quadratic equation, $a\chi^2 + b\chi + c = 0$, let $a = 1/6\ t\ f_m$, $b = -1/2\ t\ f_m\ (\ell - d')$, note: $(\ell - d') = d$; $c = P\left(\frac{\ell}{2} - d'\right) + M$; $x = kd$.

Using the binominal formula to solve the quadratic equation,

$$kd = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

therefore

$$kd = \frac{-b - \sqrt{b^2 - 4ac}}{2a}$$

$$kd = \frac{1/2\ t\ f_m(\ell - d')}{2 \times 1/6\ t\ f_m} - \frac{\sqrt{(1/2\ tf_m(\ell - d'))^2 - 4(1/6\ tf_m)\left[P\left(\frac{\ell}{2} - d'\right) + M\right]}}{2 \times 1/6 t f_m}$$

a. Determine the maximum allowable compressive masonry stress, f_m,
Actual axial stress

$$fa = \frac{P}{t\ell} = \frac{95{,}000}{9.5 \times 96} = 104 \text{ psi.}$$

Maximum allowable axial stress,

$$F_a = 0.20\ f'm \left[1 - \left(\frac{h}{40t}\right)^3\right]$$

$$= 0.20\ (1800) \left[1 - \left(\frac{180}{40 \times 9.5}\right)^3\right] = 322 \text{ psi.}$$

Maximum allowable flexural stress,

$$F_b = 1/3\ f'm = 1/3\ (1800) = 600 \text{ psi.}$$

Maximum allowable flexural stress that will satisfy the unity equation,

$$f_b = F_b\left[1.00 \text{ or } 1.33 - \frac{f_a}{F_a}\right] = 600\left[1.33 - \frac{104}{322}\right]$$

$$= 604 \text{ psi.}$$

Maximum allowable combined compressive masonry stress,

$$f_m = f_a + f_b = 104 + 604 = 708 \text{ psi.}$$

b. Solve for kd,

$$a = 1/6\ t\ f_m = 1/6 \times 9.5 \times 708 = 1121.00.$$

$$b = -1/2\ t\ f_m(\ell - d') = -1/2 \times 9.5 \times 708\ (96 - 8)$$
$$= -295{,}944.$$

$$c = P\left(\frac{\ell}{2} - d'\right) + M$$

$$= \left[95\left(\frac{96}{2} - 8\right) + 450 \times 12\right]1000$$

$$= 9{,}200{,}000.$$

$$kd = \frac{-b - \sqrt{b^2 - 4ac}}{2a}$$

$$= \frac{+295{,}944 - \sqrt{(295{,}944)^2 - 4 \times 1121 \times 9.2 \times 10^6}}{2 \times 1121}$$

$$= 36''.$$

c. Solve for compressive force C,

$$C = 1/2\ t\ kd\ f_m$$
$$= 1/2 \times 9.5 \times 36.0 \times 708 = 121{,}068 \text{ lbs.}$$

d. Solve for tension force by sum of vertical force equal zero,

$$T = C - P = 121{,}068 - 95{,}000 = 26{,}068 \text{ lbs.}$$

e. Solve for stress in steel,

$$k = \frac{kd}{d} = \frac{36}{88} = 0.409$$

$$f_s = \left(\frac{1-k}{k}\right) n\ f_m$$

$$= \left(\frac{1 - 0.409}{0.409}\right) 16.7(708) = 17{,}080\text{psi}$$

f. Solve for area of reinforcing steel,

$$A_s = \frac{T}{f_s} = \frac{26{,}068}{17{,}080} = 1.53 \text{ sq in.}$$

Method 3. (See Figure 11.)

Design and Analysis Including Compression Steel. This method is similar to an element subjected to axial load plus moment utilizing both compression and tension steel.

a. Calculate constants, Virtual eccentricity $e = \frac{M}{P}$

$$e_v = \frac{450 \times 12}{95} = 56.8''$$

Eccentricity from tension steel

$$e = \frac{12M}{P} + d'' = \frac{12 \times 450}{95} + 40 = 96.8''$$

$$E = \frac{e}{12} = \frac{96.8}{12} = 8.07 \text{ ft}$$

$$\frac{e}{d} = \frac{96.8}{88} = 1.10$$

b. Determine flexural coefficient K for balanced design. From Table E6 found in *Reinforced Masonry Engineering Handbook,* J. E. Amrhein, Third Edition, Table E6, page 313, for f_b = 800 psi and f_s = 26,700 psi (one third increase) K_b = 118.7.

c. Dimensional coefficient,

$$F = \frac{td^2}{12{,}000} = \frac{9.5 \times 88^2}{12{,}000} = 6.13.$$

d. Determine if compression steel is required,

$$M = P\,E = 95 \times 8.07 = 766.65 \text{ ft kips}$$

Moment capacity at balanced design, $KF = 118.7 \times 6.13 = \underline{727.63}$ ft kips

39.02 ft kips.

Compression steel is required based on balanced stresses.

e. Determine reinforcing steel required for compression. From Table M2 found in *Reinforced Masonry Engineering Handbook,* J. E. Amrhein, Third Edition, Table M2, page 381, coefficient c for compressive reinforcement where

$$n = 16.7, \frac{d'}{d} = \frac{8}{88} = 0.091, f_s = 26{,}000 \text{ psi and } f_b = 800 \text{ psi.}$$

From Table M2, *ibid.*

$$c = 1.436, A_s' = \frac{M - KF}{cd} = \frac{39.02}{1.436 \times 88} = 0.31 \text{ sq in.}$$

f. Determine steel required for tension. Find i from Table M1 if applicable from *Reinforced Masonry Engineering Handbook,* J. E. Amrhein, Third Edition, Table M1, page 380. In this case $\frac{e}{d}$ is outside range of Table M1 but

$$i = \frac{1}{1 - \frac{jd}{e}},$$

where j from Table E6 = 0.889,

$$i = \frac{1}{1 - \frac{0.889 \times 88}{96.8}} = 5.22.$$

Also, from Table E6 a = 1.48, $A_s = \frac{M}{adi} = \frac{766.65}{1.48 \times 88 \times 5.22} = 1.13$ sq in.

The area of tension steel exceeds the required area of compression steel. Therefore, use area of tension steel at each end of wall.

Method 4.

Section assumed homogeneous for combined loads.

a. Solve for f_m and f_t as in figure 12.

$$f_a = \frac{P}{t\ell} = \frac{95{,}000}{9.5 \times 96} = 104 \text{ psi,}$$

$$f_b = \frac{M}{S} = \frac{6M}{t\ell^2} = \frac{6 \times 450 \times 12000}{9.5 \times 96^2} = 370 \text{ psi}$$

$$f_t = f_a - f_b = -266 \text{ psi}$$

$$f_m = f_a + f_b = 474 \text{ psi}$$

b. Check unity equation as in the above examples where F_a = 322 psi, F_b = 600 psi

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{104}{327} + \frac{370}{600} = 0.33 + 0.62$$

$$= 0.95 \leqslant 1.33 \therefore \text{O.K.}$$

c. Determine the total tension force (See Figure 13).

$$k\ell = \frac{f_m}{f_m + f_t} \times \ell,$$

$$\ell - k\ell = \frac{f_t}{f_m + f_t} \times \ell,$$

$$\text{Tension force} = 1/2\, f_t \times b\,(\ell - k\ell).$$

$$k\ell = \frac{474}{474 + 260} \times 96 = 61.5,$$

$$\ell - k\ell = 96 - 61.5 = 34.5''$$

$$\text{Tension force} = 1/2\, f_t b\,(\ell - k\ell)$$

$$= 1/2 \times 266 \times 9.5\,(34.5) = 43{,}590 \text{ lbs.}$$

d. Determine area of steel. The area of steel may be determined by dividing the tension force by the allowable tension stress which can be increased by one third if the force is due to wind or earthquake.

$$A_s = \frac{T}{f_s} \text{ or } \frac{T}{1.33\, f_s}, A_s = \frac{43{,}590}{26{,}700} = 1.63 \text{ sq in.}$$

e. Moment resistance of tension steel (See Figure 14).

The moment of the tension force, T, about the neutral axis is:

$$\text{Mom}_{N.A.} = T\left[\frac{2}{3}(\ell - k\ell)\right]$$

If the reinforcing steel is moved from the centroid of the stress triangle, two thirds of the distance from the neutral axis, to the actual location, d', from the edge of the wall to the jamb steel, then the tension force can be reduced because the moment arm is increased.

The equivalent tension force, T_{eq}, required is:

$$T_{equiv.} = T\left[\frac{2}{3}(\ell - k\ell)\right] \times \frac{1}{\ell - k\ell - d'}.$$

The adjusted area of steel would be $A_{s\ eq.} = \frac{T_{equiv.}}{1.33\, f_s}$.

$$T_{equiv.} = 43{,}590\left[\frac{2}{3}(34.5)\frac{1}{96 - 61.5 - 8}\right]$$

$$= 37{,}833 \text{ lbs.}$$

$$A_s = \frac{T_{equiv.}}{1.33 f_s} = \frac{37{,}833 \text{ lbs}}{26{,}700 \text{ psi}} = 1.42 \text{ sq in.}$$

f. The above area of steel is based on the maximum allowable stress in the reinforcing. This assumption should

be checked against the conditions stated at the beginning: that plane sections remain plane after bending and that stress is proportional to strain.

On the basis of the above stresses, the stress in the steel would be as follows:

$$f_s = n\, f_m \left(\frac{\ell - k\ell - d'}{k\ell}\right)$$

$$= 16.7(474)\left(\frac{34.5 - 8}{61.5}\right) = 3{,}411 \text{ psi.}$$

Accordingly, the area of steel determined in step e, A_s = 1.42 sq in. is not in compliance with the criteria.

CONCLUSIONS

Of the four methods reviewed herein, Method 1 is most conservative because it neglects the effect of the axial load in reducing the amount of tensile steel required. A_s = 2.50 sq in.

Method 2 is a statical analysis which takes full advantage of the maximum allowable compressive stress in the masonry. A_s = 1.53 sq in.

Method 3 takes into account the beneficial effect of compression steel and results in an area of steel of: A_s = 1.13 sq in.

Method 4 in the example results in an area of steel, A_s = 1.42 sq in. This is close to Method 2, however, the method is unorthodox for it assumes minimum compressive stress in the masonry and maximum stress in the steel which is not in keeping with the straight line elastic theory.

The authors would urge research into the actual performance of reinforced masonry elements subjected to combined loadings. The original assumptions may not be in keeping with performance and new assumptions and theory may have to be developed.

It is of growing importance that research into the performance of reinforced masonry elements subjected to extreme loadings be conducted. The principles of ultimate strength design or capacity design must be developed for reinforced masonry systems that will better predict the reinforcing needs, ductility and the performance of the systems.

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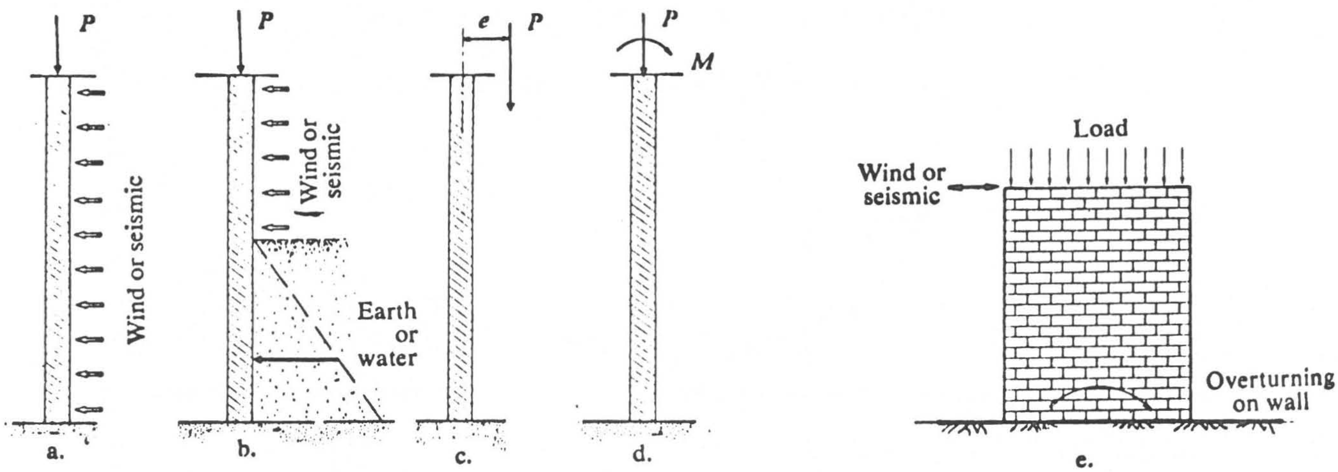


Figure 1. Combined load and moment on a wall.

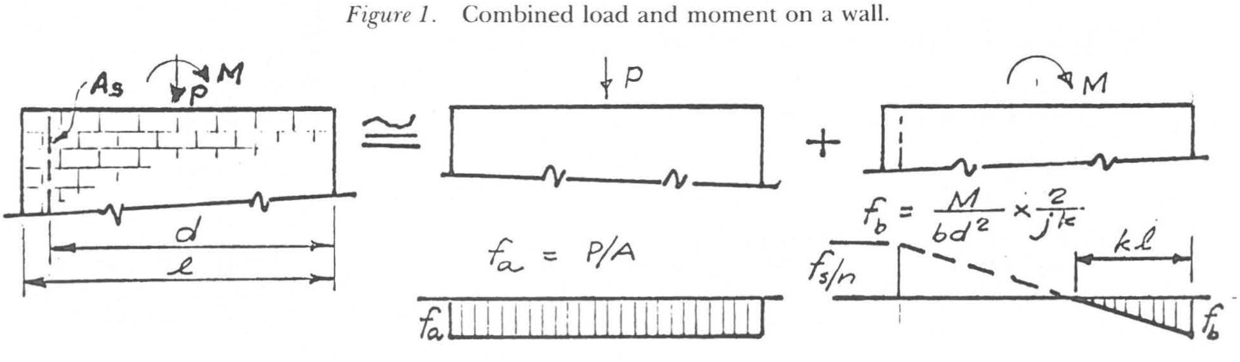


Figure 9.

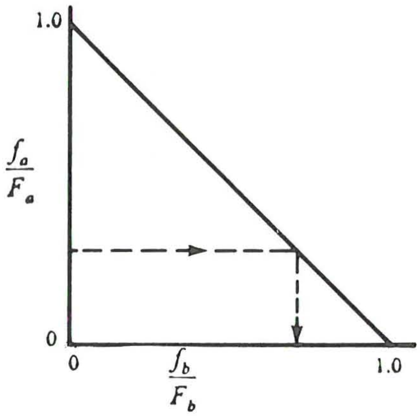


Figure 2. Graphic represention of unity equation.

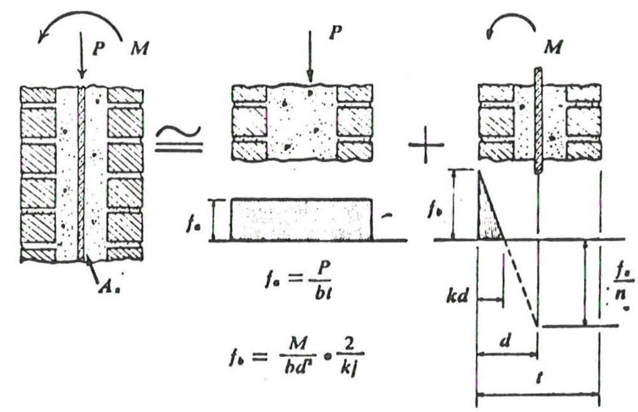


Figure 3. Graphical representation of interaction analysis by Method 1.

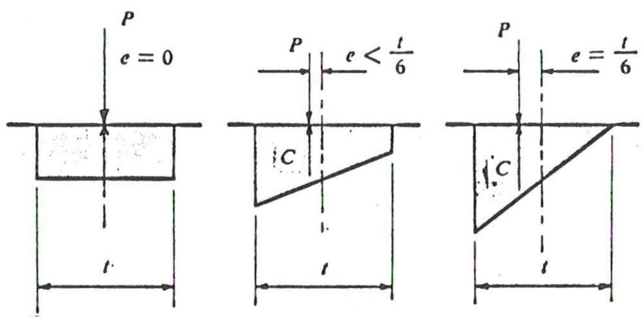
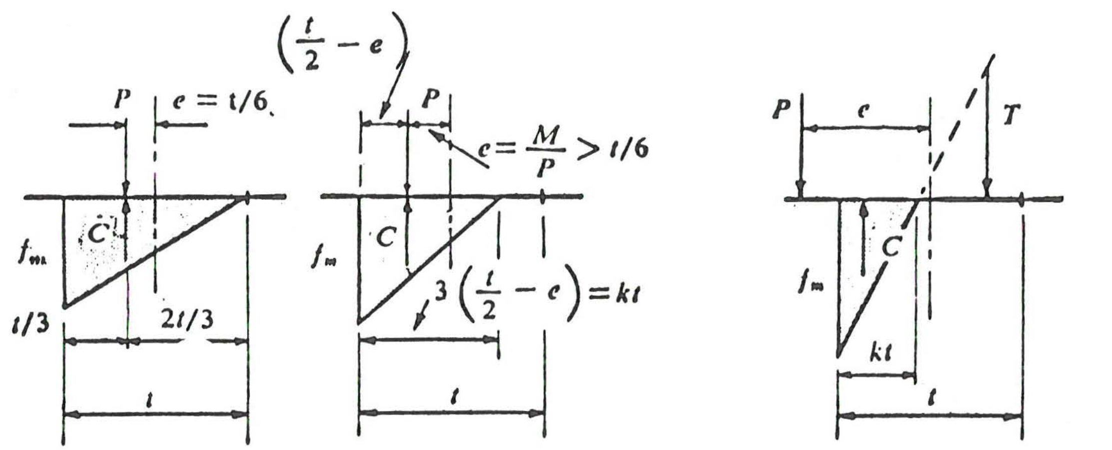


Figure 4. Graphical representation for Modified Method 1.



a) **Eccentricity equals kern distance, $e_k = t/6$**

b) **Eccentrictiy is greater than kern distance but is coincidental with compression resultant**

c) **Eccentricity is greater than the eccentricity of the compression resultant**

Figure 5. Graphical representation of Method 2 with conditions of increasing eccentricity of load on a wall.

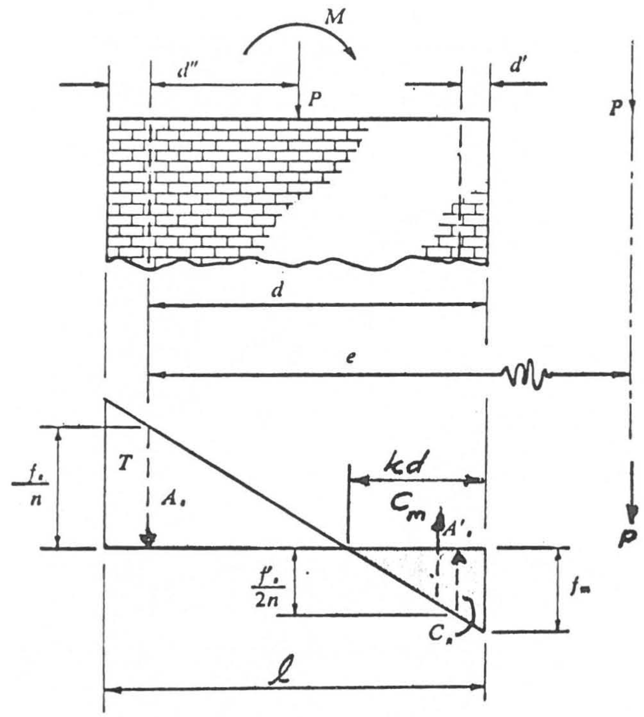


Figure 6. Graphical representation of Method 3 including compression steel.

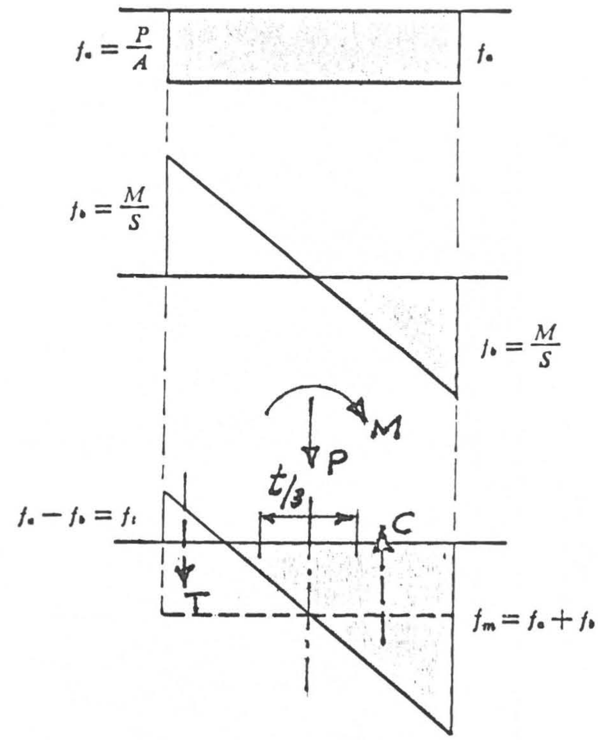


Figure 7. Graphical representation of Method 4.

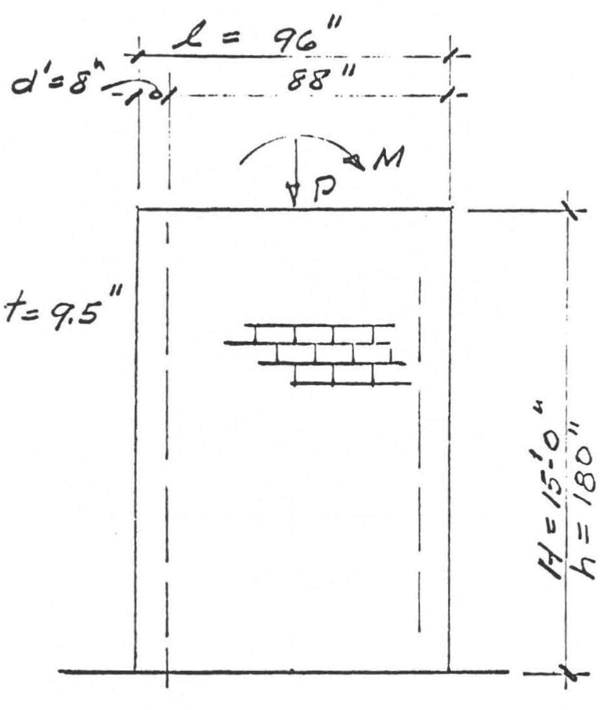


Figure 8. Brick wall subjected to axial load and moment.

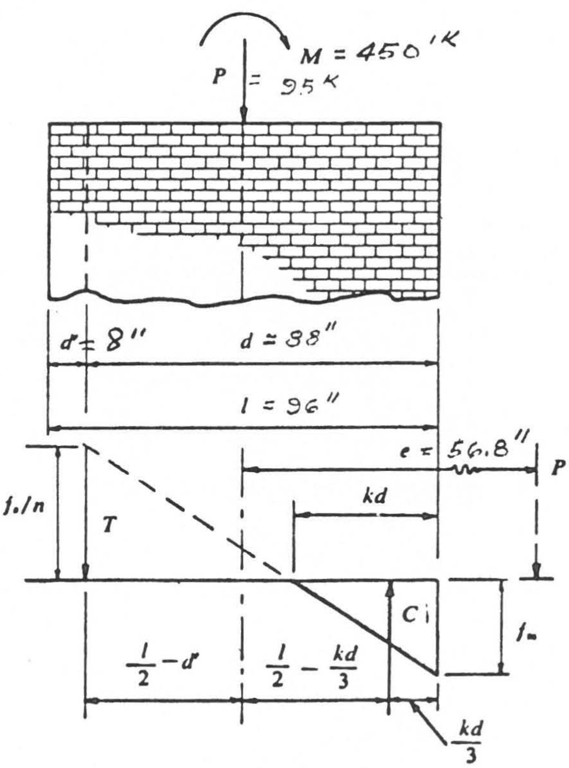


Figure 10.

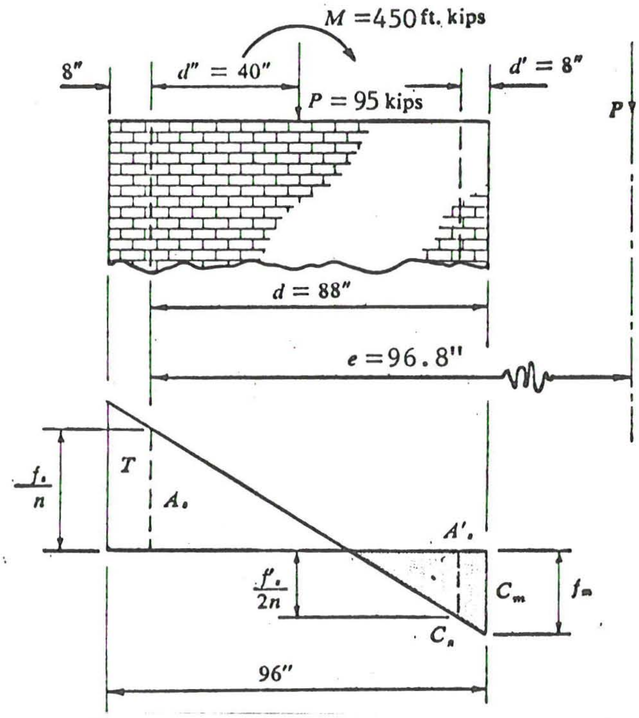


Figure 11. Shear wall with load and moment, tension and compression steel.

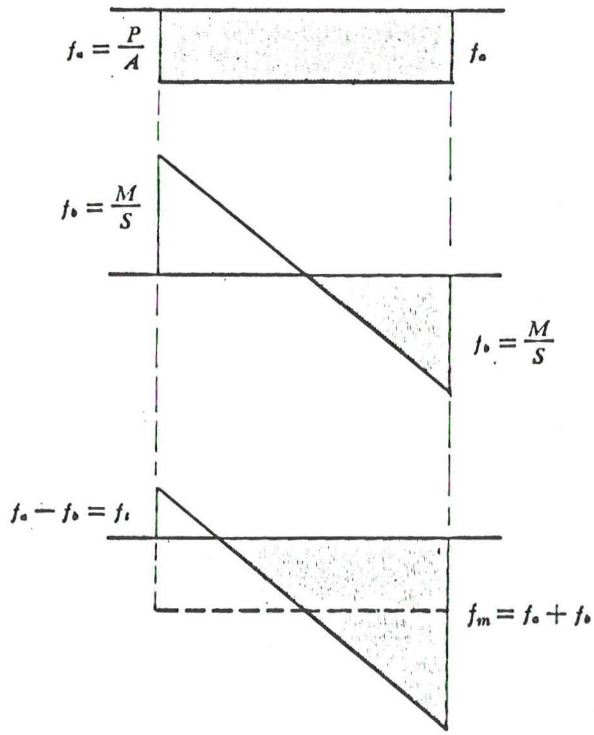


Figure 12.

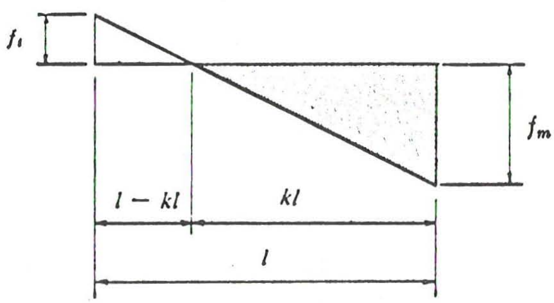


Figure 13.

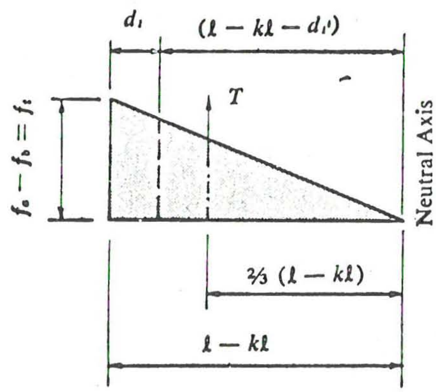
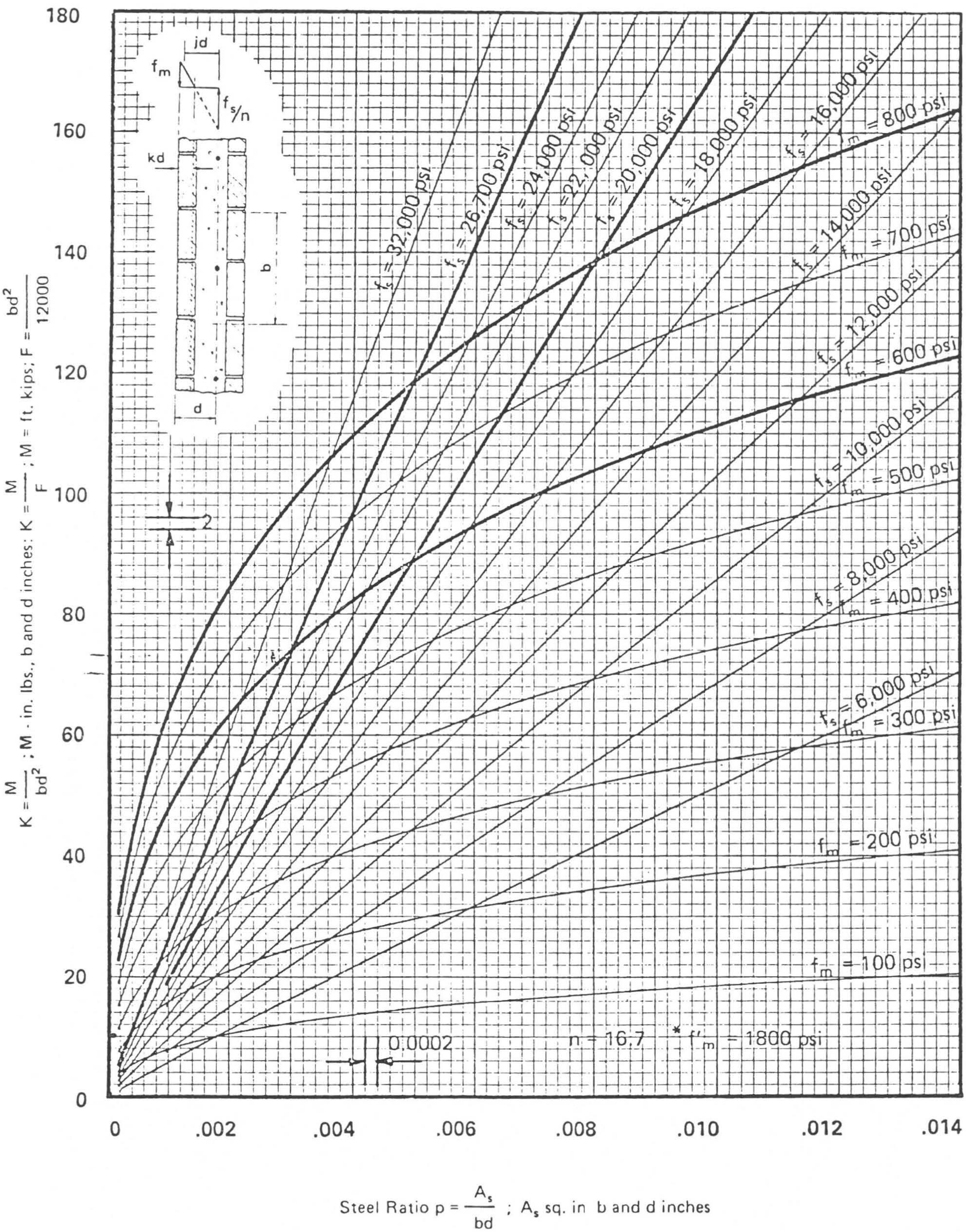


Figure 14.

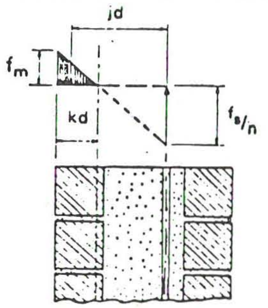
DIAGRAM F-6 K[1] vs p for Various Masonry and Steel Stresses, * n = 16.7



[1] To convert K psi to K, kilo pascals multiply the value psi by 6.8948

From *Reinforced Masonry Engineering Handbook,* Amrhein, J. E., Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, CA 90057, Third Edition, 1978, p. 335.

TABLE E-6—Coefficients for Flexural Design



f'_m = 1,800 psi

f_m = 600 psi

f_s = 20,000 psi

n = 16.7

a = 1.48 (Approximately balanced design)

$M = Kbd^2$

$M = KF$

$$K = \frac{1}{2} kjf_m$$

$$n = \frac{E_s}{E_m} = \frac{30{,}000{,}000}{1000f'_m}$$

$$p = \frac{A_s}{bd} = \frac{K}{f_s j}$$

$$k = \frac{1}{1 + f_s/nf_m}$$

$$j = 1 - \frac{k}{3}$$

$$a = \frac{f_s}{12000} \times (\text{average } j)$$

$$A_s = \frac{M}{ad} \text{ or } A_s \frac{NE}{adi}$$

* Solid units, Grade SW; 3000 psi on gross area

Increase for wind or earthquake
1.33 f_m = 800 psi
1.33 f_s = 26,700 psi

f_m	f_s	K	p	np	k	j	2/jk	1.33 f_s	1.33 f_m	1.33 K
60	20000	1.4	.0001	.001	.048	.984	42.60	26700	80	1.9
70	20000	1.9	.0001	.002	.055	.982	36.90	26700	93	2.5
80	20000	2.5	.0001	.002	.063	.979	32.62	26700	107	3.3
90	20000	3.1	.0002	.003	.070	.977	29.30	26700	120	4.1
100	20000	3.8	.0002	.003	.077	.974	26.64	26700	133	5.0
110	20000	4.5	.0002	.004	.084	.972	24.46	26700	147	6.0
120	20000	5.3	.0003	.005	.091	.970	22.65	26700	160	7.1
130	20000	6.2	.0003	.005	.098	.967	21.11	26700	173	8.2
140	20000	7.1	.0004	.006	.105	.965	19.80	26700	187	9.4
150	20000	8.0	.0004	.007	.111	.963	18.66	26700	200	10.7
160	20000	9.1	.0005	.008	.118	.961	17.66	26700	213	12.1
170	20000	10.1	.0005	.009	.124	.959	16.79	26700	227	13.5
180	20000	11.2	.0006	.010	.131	.956	16.00	26700	240	15.0
190	20000	12.4	.0007	.011	.137	.954	15.30	26700	253	16.6
200	20000	13.6	.0007	.012	.143	.952	14.68	26700	267	18.2
210	20000	14.9	.0008	.013	.149	.950	14.11	26700	280	19.8
220	20000	16.2	.0009	.014	.155	.948	13.59	26700	293	21.6
230	20000	17.5	.0009	.016	.161	.946	13.12	26700	307	23.4
240	20000	18.9	.0010	.017	.167	.944	12.69	26700	320	25.2
250	20000	20.3	.0011	.018	.173	.942	12.29	26700	333	27.1
260	20000	21.8	.0012	.019	.178	.941	11.92	26700	347	29.1
270	20000	23.3	.0012	.021	.184	.939	11.58	26700	360	31.1
280	20000	24.9	.0013	.022	.190	.937	11.27	26700	373	33.1
290	20000	26.4	.0014	.024	.195	.935	10.97	26700	387	35.2
300	20000	28.0	.0015	.025	.200	.933	10.70	26700	400	37.4
310	20000	29.7	.0016	.027	.206	.932	10.44	26700	413	39.6
320	20000	31.4	.0017	.028	.211	.930	10.20	26700	427	41.8
330	20000	33.1	.0018	.030	.216	.928	9.98	26700	440	44.1
340	20000	34.8	.0019	.031	.221	.926	9.76	26700	453	46.4
350	20000	36.6	.0020	.033	.226	.925	9.56	26700	467	48.8
360	20000	38.4	.0021	.035	.231	.923	9.38	26700	480	51.2
370	20000	40.2	.0022	.037	.236	.921	9.20	26700	493	53.6
380	20000	42.1	.0023	.038	.241	.920	9.03	26700	507	56.1
390	20000	44.0	.0024	.040	.246	.918	8.87	26700	520	58.6
400	20000	45.9	.0025	.042	.250	.917	8.72	26700	533	61.2
410	20000	47.8	.0026	.044	.255	.915	8.57	26700	547	63.8
420	20000	49.8	.0027	.046	.260	.914	8.43	26700	560	66.4
430	20000	51.8	.0028	.047	.264	.912	8.30	26700	573	69.1
440	20000	53.8	.0030	.049	.269	.910	8.18	26700	587	71.8
450	20000	55.9	.0031	.051	.273	.909	8.06	26700	600	74.5
460	20000	57.9	.0032	.053	.278	.908	7.94	26700	613	77.2
470	20000	60.0	.0033	.055	.282	.906	7.83	26700	627	80.0
480	20000	62.1	.0034	.057	.286	.905	7.73	26700	640	82.8
490	20000	64.3	.0036	.059	.290	.903	7.63	26700	653	85.7
500	20000	66.4	.0037	.062	.295	.902	7.53	26700	667	88.5
510	20000	68.6	.0038	.064	.299	.900	7.44	26700	680	91.4
520	20000	70.8	.0039	.066	.303	.899	7.35	26700	693	94.4
530	20000	73.0	.0041	.068	.307	.898	7.26	26700	707	97.3
540	20000	75.2	.0042	.070	.311	.896	7.18	26700	720	100.3
550	20000	77.5	.0043	.072	.315	.895	7.10	26700	733	103.3
560	20000	79.7	.0045	.075	.319	.894	7.02	26700	747	106.3
570	20000	82.0	.0046	.077	.323	.893	6.95	26700	760	109.4
580	20000	84.3	.0047	.079	.326	.891	6.88	26700	773	112.4
590	20000	86.7	.0049	.081	.330	.890	6.81	26700	787	115.5
600	20000	89.0	.0050	.084	.334	.889	6.74	26700	800	118.7
600	19000	91.7	.0055	.091	.345	.885	6.55	25300	800	122.2
600	18000	94.5	.0060	.100	.358	.881	6.35	24000	800	126.0
600	17000	97.5	.0065	.109	.371	.876	6.15	22700	800	130.0
600	16000	100.7	.0072	.121	.385	.872	5.96	21300	800	134.3
600	15000	104.1	.0080	.134	.401	.867	5.76	20000	800	138.8
600	14000	107.7	.0089	.149	.417	.861	5.57	18700	800	143.7
600	13000	111.6	.0100	.168	.435	.855	5.37	17300	800	148.8
600	12000	115.8	.0114	.190	.455	.848	5.18	16000	800	154.4
600	11000	120.3	.0130	.217	.477	.841	4.99	14700	800	160.4
600	10000	125.1	.0150	.251	.501	.833	4.80	13300	800	166.8
600	9000	130.3	.0176	.293	.527	.824	4.61	12000	800	173.7
600	8000	135.9	.0209	.348	.556	.815	4.42	10700	800	181.2
600	7000	142.0	.0252	.421	.589	.804	4.23	9300	800	189.3
600	6000	148.5	.0313	.522	.626	.792	4.04	8000	800	198.0
600	5000	155.6	.0400	.668	.667	.778	3.86	6700	800	207.5

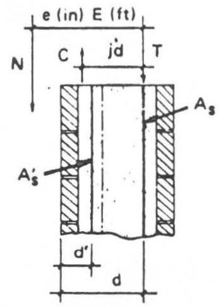
To convert psi to mega pascals, multiply the value psi by 0.006895

From *Reinforced Masonry Engineering Handbook*, Amrhein, J. E., Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, CA 90057, Third Edition, 1978, p. 313.

TABLE M-1—Coefficients i for Flexural Members Subject to Bending and Axial Loads

For use of Table see Example 11 Q.

Values of $i = \dfrac{1}{1 - \dfrac{jd}{e}}$



Tension steel $A_s = \dfrac{M}{adi}$ or $\dfrac{NE}{adi}$ See tables E for values of a

Compression steel $A'_s = \dfrac{M - KF}{cd}$ or $\dfrac{NE - KF}{cd}$

See Tables M-2 for values of c

Enter table with known values of $\dfrac{e}{d}$ and j; select value of i.

For guide in estimating j in investigation, follow the zigzag line. j may also be determined exactly by using table L-1, L-2 and L-3 by knowing the physical characteristics of the member; b, d, d', A_s, n or f'_m.

e/d	j																				
	0.75	0.76	0.77	0.78	0.79	0.80	0.81	0.82	0.83	0.84	0.85	0.86	0.87	0.88	0.89	0.90	0.91	0.92	0.93	0.94	0.95
1.15	2.87	2.95	3.03	3.11	3.20	3.29	3.38	3.48	3.59	3.71	3.83	3.96	4.11	4.26	4.43	4.60	4.79	5.00	5.23	5.48	5.75
1.16	2.83	2.90	2.98	3.05	3,13	3.22	3.31	3.41	3.52	3.62	3.74	3.87	4.00	4.14	4.30	4.46	4.64	4.84	5.04	5.27	5.52
1.17	2.79	2.85	2.92	3.00	3.08	3.16	3.25	3.34	3.44	3.55	3.66	3.77	3.90	4.03	4.18	4.33	4.50	4.68	4.87	5.09	5.32
1.18	2.75	2.81	2.88	2.95	3.02	3.11	3.19	3.28	3.37	3.47	3.58	3.69	3.81	3.93	4.07	4.21	4.39	4.54	4.72	4.92	5.13
1.19	2.70	2.77	2.83	2.90	2.98	3.05	3.13	3.22	3.31	3.40	3.50	3.61	3.72	3.84	3.97	4.10	4.25	4.41	4.58	4.76	4.96
1.20	2.67	2.73	2.79	2.86	2.93	3.00	3.08	3.16	3.24	3.33	3.43	3.53	3.64	3.75	3.87	4.00	4.14	4.28	4.44	4.62	4.80
1.22	2.60	2.65	2.71	2.77	2.84	2.91	2.98	3.05	3.13	3.21	3.30	3.39	3.49	3.59	3.70	3.81	3.94	4.07	4.21	4.36	4.52
1.24	2.53	2.58	2.64	2.70	2.76	2.82	2.88	2.95	3.03	3.10	3.18	3.26	3.35	3.44	3.54	3.65	3.76	3.88	4.00	4.13	4.28
1.26	2.47	2.52	2.57	2.62	2.68	2.74	2.80	2.86	2.93	3.00	3.07	3.15	3.23	3.32	3.41	3.50	3.60	3.71	3.82	3.94	4.06
1.28	2.41	2.46	2.51	2.56	2.61	2.67	2.72	2.78	2.84	2.91	2.98	3.05	3.12	3.20	3.28	3.37	3.46	3.56	3.66	3.76	3.88
1.30	2.36	2.41	2.45	2.50	2.55	2.60	2.65	2.71	2.77	2.83	2.89	2.96	3.03	3.10	3.17	3.25	3.33	3.42	3.51	3.61	3.71
1.32	2.32	2.36	2.40	2.44	2.49	2.54	2.59	2.64	2.69	2.75	2.81	2.87	2.93	3.00	3.07	3.14	3.22	3.30	3.38	3.47	3.57
1.34	2.27	2.31	2.35	2.39	2.44	2.48	2.53	2.58	2.63	2.68	2.74	2.79	2.85	2.92	2.98	3.05	3.12	3.19	3.27	3.35	3.44
1.36	2.23	2.27	2.31	2.35	2.39	2.43	2.47	2.52	2.57	2.62	2.67	2.72	2.78	2.83	2.89	2.96	3.02	3.09	3.16	3.24	3.32
1.38	2.19	2.23	2.26	2.30	2.34	2.38	2.42	2.46	2.51	2.56	2.60	2.65	2.71	2.76	2.82	2.88	2.94	3.00	3.07	3.14	3.21
1.40	2.15	2.19	2.22	2.26	2.30	2.33	2.37	2.41	2.46	2.50	2.55	2.59	2.64	2.69	2.75	2.80	2.86	2.92	2.98	3.04	3.11
1.42	2.12	2.15	2.18	2.22	2.25	2.29	2.33	2.37	2.41	2.45	2.49	2.54	2.58	2.63	2.68	2.73	2.79	2.84	2.90	2.96	3.02
1.44	2.09	2.12	2.15	2.18	2.22	2.25	2.28	2.32	2.36	2.40	2.44	2.49	2.53	2.57	2.62	2.67	2.72	2.77	2.82	2.88	2.94
1.46	2.06	2.09	2.12	2.15	2.18	2.21	2.25	2.28	2.32	2.35	2.39	2.43	2.48	2.52	2.56	2.61	2.66	2.71	2.75	2.81	2.86
1.48	2.03	2.06	2.09	2.12	2.15	2.18	2.21	2.24	2.28	2.31	2.35	2.39	2.43	2.47	2.51	2.55	2.60	2.64	2.69	2.74	2.79
1.50	2.00	2.03	2.05	2.08	2.11	2.14	2.17	2.21	2.24	2.27	2.31	2.34	2.38	2.42	2.46	2.50	2.54	2.59	2.63	2.68	2.73
1.55	1.94	1.96	1.99	2.01	2.04	2.07	2.10	2.12	2.15	2.18	2.21	2.25	2.28	2.31	2.35	2.38	2.42	2.46	2.50	2.54	2.58
1.60	1.88	1.90	1.93	1.95	1.98	2.00	2.03	2.05	2.08	2.10	2.13	2.16	2.19	2.22	2.25	2.29	2.32	2.35	2.39	2.42	2.46
1.65	1.83	1.85	1.88	1.90	1.92	1.94	1.97	1.99	2.01	2.04	2.06	2.09	2.11	2.14	2.17	2.20	2.23	2.26	2.29	2.33	2.36
1.70	1.79	1.81	1.83	1.85	1.87	1.89	1.91	1.93	1.96	1.98	2.00	2.03	2.05	2.07	2.10	2.13	2.15	2.18	2.21	2.24	2.27
1.75	1.75	1.77	1.78	1.80	1.82	1.84	1.86	1.88	1.90	1.92	1.94	1.97	1.99	2.01	2.04	2.06	2.08	2.11	2.14	2.16	2.19
1.80	1.71	1.73	1.75	1.76	1.78	1.80	1.82	1.84	1.86	1.87	1.89	1.91	1.93	1.95	1.98	2.00	2.02	2.04	2.07	2.09	2.12
1.85	1.68	1.70	1.71	1.73	1.75	1.76	1.78	1.80	1.81	1.83	1.85	1.87	1.89	1.91	1.93	1.95	1.97	1.99	2.01	2.03	2.06
1.90	1.65	1.67	1.68	1.70	1.71	1.73	1.74	1.76	1.78	1.79	1.81	1.83	1.84	1.86	1.88	1.90	1.92	1.94	1.96	1.98	2.00
1.95	1.62	1.64	1.65	1.67	1.68	1.70	1.71	1.73	1.74	1.76	1.77	1.79	1.81	1.82	1.84	1.86	1.88	1.89	1.91	1.93	1.95
2.0	1.60	1.61	1.63	1.64	1.65	1.67	1.68	1.69	1.71	1.72	1.74	1.76	1.77	1.78	1.80	1.82	1.84	1.85	1.87	1.89	1.91
2.1	1.56	1.57	1.58	1.59	1.60	1.62	1.63	1.64	1.65	1.67	1.68	1.69	1.71	1.72	1.74	1.75	1.77	1.78	1.80	1.81	1.83
2.2	1.52	1.53	1.54	1.55	1.56	1.57	1.58	1.59	1.61	1.62	1.63	1.64	1.65	1.67	1.68	1.69	1.71	1.72	1.73	1.75	1.76
2.3	1.48	1.49	1.50	1.51	1.52	1.53	1.54	1.55	1.57	1.58	1.59	1.60	1.61	1.62	1.63	1.64	1.65	1.67	1.68	1.69	1.70
2.4	1.45	1.46	1.47	1.48	1.49	1.50	1.51	1.52	1.53	1.54	1.55	1.56	1.57	1.58	1.59	1.60	1.61	1.62	1.63	1.64	1.66
2.5	1.43	1.44	1.44	1.45	1.46	1.47	1.48	1.49	1.50	1.51	1.51	1.52	1.53	1.54	1.55	1.56	1.57	1.58	1.59	1.60	1.61
2.6	1.41	1.41	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.48	1.49	1.49	1.50	1.51	1.52	1.53	1.54	1.55	1.56	1.57	1.58
2.7	1.39	1.39	1.40	1.41	1.41	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.47	1.48	1.49	1.50	1.51	1.52	1.53	1.53	1.54
2.8	1.37	1.37	1.38	1.39	1.39	1.40	1.41	1.41	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.47	1.48	1.49	1.50	1.51	1.51
2.9	1.35	1.36	1.36	1.37	1.37	1.38	1.39	1.39	1.40	1.41	1.42	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.47	1.48	1.49
3.0	1.33	1.34	1.34	1.35	1.36	1.36	1.37	1.38	1.38	1.39	1.40	1.40	1.41	1.42	1.42	1.43	1.44	1.44	1.45	1.46	1.46
3.5	1.27	1.28	1.28	1.29	1.29	1.30	1.30	1.31	1.31	1.32	1.32	1.33	1.33	1.34	1.34	1.35	1.35	1.36	1.36	1.37	1.37
4.0	1.23	1.23	1.24	1.24	1.25	1.25	1.25	1.26	1.26	1.27	1.27	1.27	1.28	1.28	1.29	1.29	1.29	1.30	1.30	1.31	1.31
4.5	1.20	1.20	1.21	1.21	1.21	1.22	1.22	1.22	1.23	1.23	1.23	1.24	1.24	1.24	1.25	1.25	1.25	1.26	1.26	1.26	1.27
5.0	1.18	1.18	1.18	1.19	1.19	1.19	1.19	1.19	1.20	1.20	1.20	1.21	1.21	1.21	1.22	1.22	1.22	1.22	1.23	1.23	1.23
6.0	1.14	1.15	1.15	1.15	1.15	1.15	1.16	1.16	1.16	1.16	1.16	1.17	1.17	1.17	1.17	1.18	1.18	1.18	1.18	1.18	1.19
7.0	1.12	1.12	1.12	1.13	1.13	1.13	1.13	1.13	1.14	1.14	1.14	1.14	1.14	1.14	1.15	1.15	1.15	1.15	1.15	1.16	1.16
8.0	1.10	1.10	1.11	1.11	1.11	1.11	1.11	1.11	1.12	1.12	1.12	1.12	1.12	1.12	1.13	1.13	1.13	1.13	1.13	1.13	1.14
9.0	1.09	1.09	1.09	1.09	1.10	1.10	1.10	1.10	1.10	1.10	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.12	1.12
10.0	1.08	1.08	1.08	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
15.0	1.05	1.05	1.05	1.05	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.07	1.07	1.07	1.07
20.0	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
25.0	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04
30.0	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
35.0	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03

From *Reinforced Masonry Engineering Handbook*, Amrhein, J. E., Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, CA 90057, Third Edition, 1978, p. 380.

TABLE M-2—Coefficients c for Compression Reinforcement

$$c = \frac{f_s\,(2n-1)(1-d'/d)(k-d'/d)}{12000n\,(1-k)}$$

where f_m and f_s are allowable stresses, psi.[1]

Enter table with known value of d'/d; f_s; n; f_m

Select value of c; compute $A'_s = \frac{M - KF}{cd}$

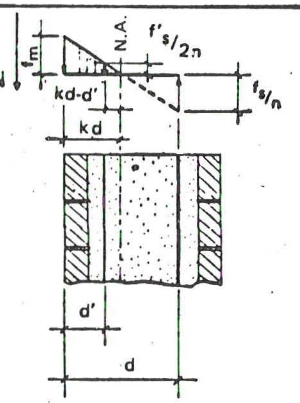
M — ft. kips, d — inches

K from table E for balance section $A_s = \frac{M}{ad}$

F from table D

$$k = \frac{1}{1 + \frac{f_s}{nf_m}}$$

*For d'/d values greater than 0.26 the effect of the compression steel becomes increasingly negligible.



n	f'm	fm	d'/d = .02	.04	.06	.08	.10	.12	.14	.16	1.8	.20	.22	.24	.26
		f_s = 20,000 psi[1]													
40	½(1500)	250	1.516	1.390	1.269	1.151	1.037	.927	.821	.779	.121	.527	.436	.350	.268
33	½(1800)	300	1.496	1.372	1.251	1.134	1.121	.912	.807	.705	.608	.515	.425	.340	.258
20	1500	500	1.497	1.373	1.263	1.136	1.024	.915	.811	.710	.613	.520	.431	.346	.265
16.7	1800	600	1.492	1.369	1.249	1.133	1.021	.913	.809	.708	.612	.519	.431	.346	.265
15	2000	667	1.485	1.362	1.243	1.127	1.016	.908	.804	.704	.608	.516	.128	.343	.263
12	2500	833	1.471	1.349	1.231	1.116	1.006	.899	.796	.697	.602	.511	.423	.340	.260
11	2700	900	1.450	1.329	1.212	1.099	.989	.884	.782	.684	.589	.199	.412	.329	.250
10	3000	900	1.307	1.192	1.081	.973	.869	.769	.673	.580	.491	.405	.324	.245	.171
		f_s = 24,000 psi[1]													
40	½(1500)	250	1,503	1.365	1.231	1.102	.978	.857	.742	.630	.524	.421	.324	.230	.141
33	½(1800)	300	1.483	1.346	1.214	1.085	.962	.842	.728	.617	.511	.410	.313	.220	.132
20	1500	500	1.484	1.348	1.216	1.088	.965	.847	.732	.622	.517	.416	.319	.227	.139
16.7	1800	600	1.480	1.344	1.213	1.086	.963	.845	.731	.622	.517	.416	.320	.228	.141
15	2000	667	1.472	1.337	1.206	1.080	.958	.840	.727	.618	.513	.413	.317	.226	.139
12	2500	833	1.458	1.324	1.195	1.069	.948	.832	.719	.611	.508	.408	.314	.223	.137
11	2700	900	1.438	1.305	1.176	1.052	.932	.816	.705	.598	.495	.397	.303	.213	.128
10	3000	900	1.294	1.167	1.045	.926	.812	.702	.596	.495	.397	.304	.215	.130	.049
		f_s = 26,700 psi[1]													
40	½(1500)	333	2.019	1.852	1.689	1.532	1.380	1.234	1.092	.956	.825	.700	.580	.465	.355
33	½(1800)	400	1.995	1.829	1.667	1.511	1.361	1.215	1.075	.940	.810	.615	.566	.452	.343
20	1500	667	1.997	1.831	1.671	1.515	1.365	1.221	1.081	.946	.813	.693	.574	.461	.352
16.7	1800	800	1.990	1.825	1.665	1.511	1.361	1.217	1.078	.944	.815	.692	.573	.460	.352
15	2000	890	1.981	1.817	1.658	1.504	1.355	1.211	1.073	.939	.811	.688	.570	458	.350
12	2500	1110	1.960	1.797	1.639	1.487	1.339	1.197	1.060	.928	.801	.679	.563	.451	.345
11	2700	1200	1.934	1.772	1.616	1.465	1.319	1.178	1.042	.911	.785	.664	.549	.438	.333
10	3000	1200	1.742	1.589	1.440	1.297	1.159	1.025	0.896	.772	.654	.540	.431	.326	.227
		f_s = 32,000 psi[1]													
40	½(1500)	333	2.002	1.818	1.640	1.468	1.302	1.142	.987	.839	.697	.560	.430	.306	.187
33	½(1800)	400	1.978	1.795	1.618	1.447	1.282	1.123	.970	.823	.682	.546	.417	.293	.176
20	1500	667	1.980	1.798	1.672	1.452	1.288	1.130	.977	.831	.690	.555	.427	.304	.187
16.7	1800	800	1.973	1.792	1.617	1.447	1.284	1.126	.975	.829	.689	.555	.426	.304	.187
15	2000	890	1.965	1.784	1.611	1.441	1.278	1.121	.970	.825	.685	.552	.424	.302	.186
12	2500	1110	1.943	1.764	1.592	1.425	1.263	1.108	.958	.814	.676	.544	.417	.297	.182
11	2700	1200	1.917	1.740	1.568	1.403	1.243	1.089	.940	.798	.661	.529	.404	.284	.170
10	3000	1200	1.725	1.556	1.393	1.235	1.013	.936	.795	.660	.530	.495	.287	.173	.066

1 To convert psi to kilo pascals, kPa, or mega pascals, MPa, multiply the value psi by 6.8948 or .0068948.

From *Reinforced Masonry Engineering Handbook*, Amrhein, J. E., Masonry Institute of America, 2550 Beverly Boulevard, Los Angeles, CA 90057, Third Edition, 1978, p. 381.