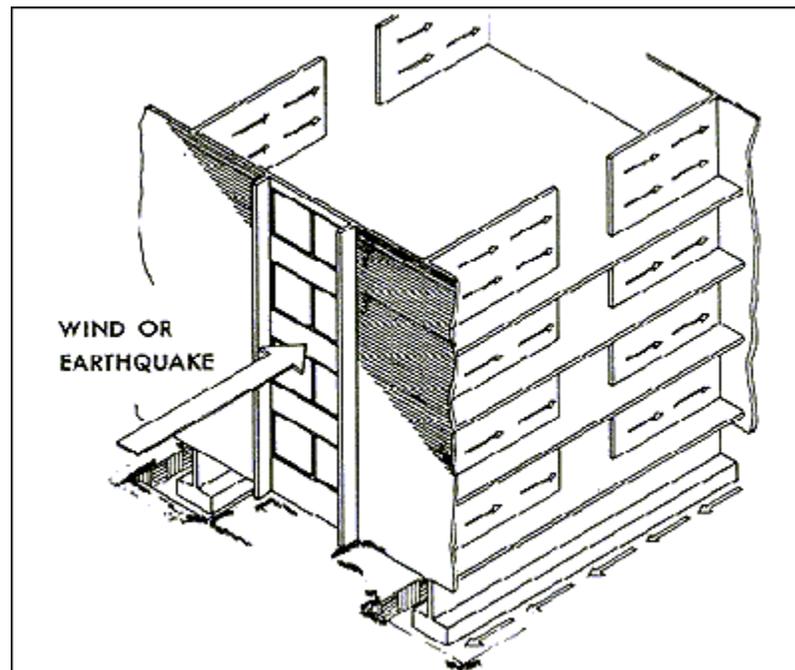


Technical Notes 24C - The Contemporary Bearing Wall - Introduction to Shear Wall Design
[Sept./Oct. 1970] (Reissued May 1988)

INTRODUCTION

The general design concept of the contemporary bearing wall building system depends upon the combined structural action of the floor and roof systems with the walls. The floor system carries vertical loads and, acting as a diaphragm, lateral loads to the walls for transfer to the foundation. Lateral forces of wind and earthquake are usually resisted by shear walls which are parallel to the direction of the lateral load. These shear walls, by their shearing resistance and resistance to overturning, transfer the lateral loads to the foundation. See Fig. 1.



Shear Wall Action

FIG. 1

It is the purpose of this *Technical Notes* to discuss some of the factors involved in the design of brick masonry shear walls and to present some of the available test data regarding their strength. Other issues of *Technical Notes* will contain examples relating to the design of brick masonry shear walls.

LATERAL FORCES

The principal lateral forces to be considered in the design of shear walls are wind pressure and earthquake. Most building codes and engineering practice standards specify that wind and earthquake may be assumed never to occur simultaneously.

Wind Pressure. Building codes usually specify design wind load requirements which should be considered minimum. For additional information on wind pressure, the designer may refer to the *American Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*, A58.1 - 1955, and "Wind Forces on Structures", *ASCE Transactions*, Vol. 126, Part II, 1961.

Earthquake. Unlike wind pressure, earthquake forces on a structure are considered a function of the mass and stiffness of the structure. Generally speaking, subject to dynamic phenomena, the greater the weight and rigidity of the structure - the greater are the forces which must be resisted by the structure. It is, therefore, current engineering practice to design "box systems" (structures without complete vertical load-carrying space frames) to resist the greater lateral forces. In this type of structural system the lateral forces are resisted by the shear walls. The *Uniform Building Code*, 1970 edition, contains the following requirements pertaining to minimum earthquake forces for structures:

Every structure shall be designed and constructed to withstand minimum total lateral seismic forces assumed to act nonconcurrently in the direction of each of the main axes of the structure in accordance with the following formula:

$$V = Z K C W$$

where:

V = total lateral load or shear at the base

Z = numerical coefficient dependent upon the zone of seismic activity:

Z = 1/4 for Zone 1 (minor damage);

Z = 1/2 for Zone 2 (moderate damage);

Z = 1 for Zone 3 (major damage)

K = numerical coefficient from Table 1

C = numerical coefficient dependent on fundamental period of vibration of the structure (in seconds) in the direction considered

W = total dead load of the structure.

TABLE 1
Horizontal Force Factor "K" for
Buildings or other Structures

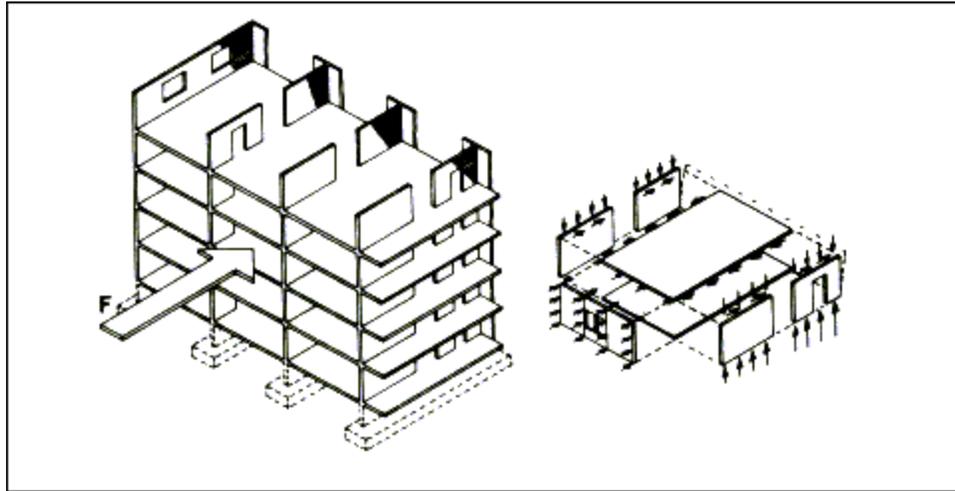
Type or Arrangement of Resisting Elements	Value of K
All building framing systems except as hereinafter classified	1.00
Buildings with a box system	1.33
Buildings with a dual bracing system consisting of a ductile moment resisting space frame and shear walls using the following design criteria: (1) The frames and shear walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the shear walls and frames (2) The shear walls acting independently of the ductile moment resisting portions of the space frame shall resist the total required lateral forces (3) The ductile moment resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force	0.80
Buildings with a ductile moment resisting space frame designed in accordance with the following criteria: The ductile moment resisting space frame shall have the capacity to resist the total required lateral force	0.67
Elevated tanks plus full contents, on four or more cross-braced legs and not supported by a building	3.00
Structures other than buildings	2.00

For additional information on earthquake forces and design, the designer may refer to the following publications:

1. *Uniform Building Code*, 1970 edition, International Conference of Building Officials.
2. *Reinforced Brick Masonry and Lateral Force Design*, Harry C. Plummer and John C. Blume, Structural Clay Products Institute, 1953.
3. *Seismic Design for Buildings*, (Department of the Army, TM 5-809-10; Department of the Navy, NAVDOCKS P-355; Department of the Air Force, AFM 88-3, Chapter 13), March 1966.
4. *Earthquake Engineering Research*, The Committee on Earthquake Engineering Research - Division of Engineering - National Research Council, National Academy of Engineering, National Academy of Sciences, 1969.

DISTRIBUTION OF LATERAL FORCES

Diaphragms. Horizontal distribution of lateral forces to shear walls is achieved by the floor and roof systems acting as diaphragms (see Fig. 2).



Diaphragm Action

FIG. 2

To qualify as a diaphragm, a floor and roof system must be able to transmit the lateral forces to the shear walls without exceeding a deflection which would cause distress to any vertical element. The successful action of a diaphragm also requires that it be properly tied into the supporting shear walls. The designer should insure this action by appropriate detailing at the juncture between horizontal and vertical structural elements of the building.

Diaphragms may be considered as analogous to horizontal (or inclined, in the case of some roofs) plate girders. The roof or floor slab constitutes the web; the joists, beams and girders function as stiffeners; and the walls or bond beams act as flanges.

Diaphragms may be constructed of materials such as concrete, wood or metal in various forms. Combinations of such materials are also possible. Where a diaphragm is made up of units such as plywood, precast concrete planks or steel deck units, its characteristics are, to a large degree, dependent upon the attachments of one unit to another and to the supporting members. Such attachments must resist shearing stresses due to internal translational and rotational actions.

The stiffness of a horizontal diaphragm affects the distribution of the lateral forces into the shear walls. No diaphragm is infinitely rigid or flexible. However, for the purpose of analysis, diaphragms may be classified into three groups: rigid, semirigid or semiflexible, and flexible.

A *rigid diaphragm* is assumed to distribute horizontal forces to the vertical resisting elements in proportion to their relative rigidities (see Fig. 3).

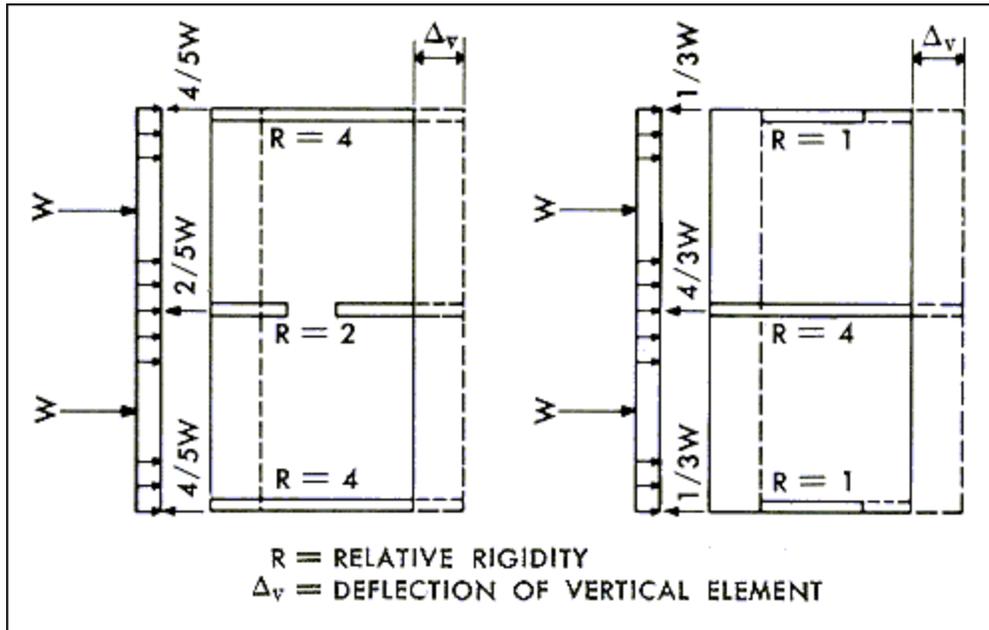


FIG. 3

Semirigid or *semiflexible diaphragms* are those which have significant deflections under load, but which also have sufficient stiffness to distribute a portion of the load to the vertical elements in proportion to the rigidities of the vertical resisting elements. The action is analogous to a continuous beam system of appreciable stiffness on yielding supports (see Fig. 4). The support reactions are dependent upon the relative stiffness of both diaphragm and the vertical resisting element.

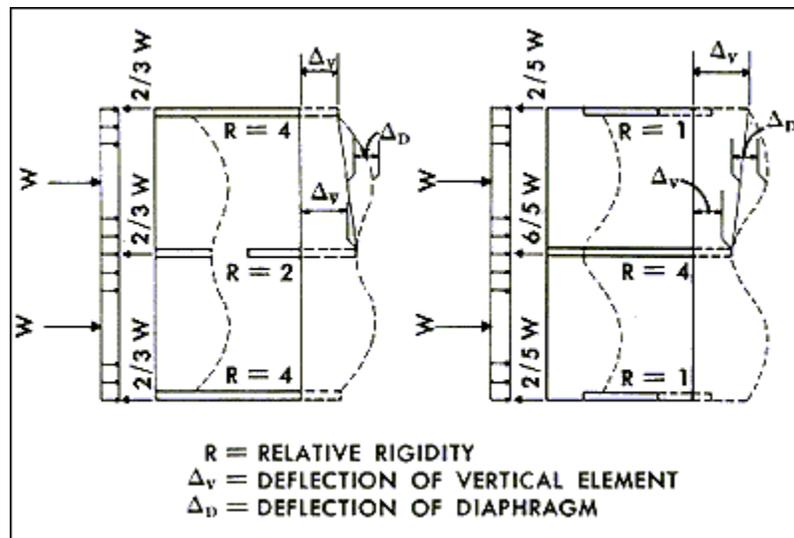


FIG. 4

A *flexible diaphragm* is analogous to a shear deflecting continuous beam or series of beams spanning between supports. The supports are considered non-yielding, and the relative stiffness of the vertical resisting elements compared to that of the diaphragm is great. Thus, a flexible diaphragm is considered to distribute the lateral forces to the vertical resisting elements on a tributary area basis (see Fig. 5).

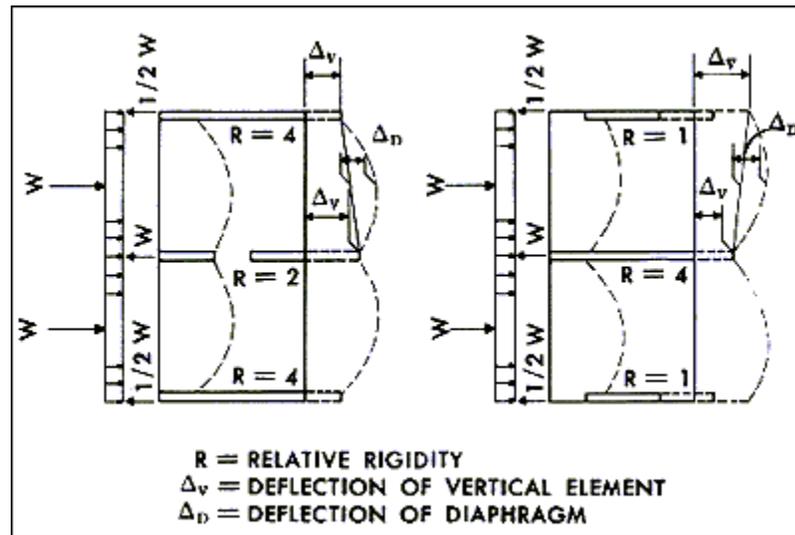
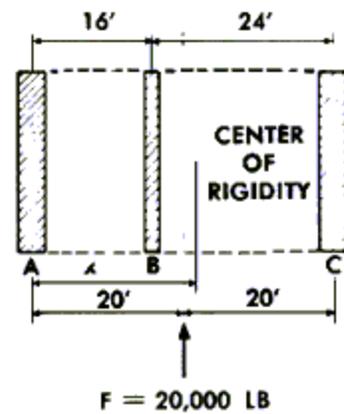


FIG. 5

Where the center of rigidity of a shear wall system does not coincide with the center of application of the lateral force, the distribution of the rotational forces due to a torsional moment on the system must also be considered. Where rigid or semirigid diaphragms are used, it may be assumed that the torsional forces are distributed to the shear walls in direct proportion to their relative rigidities and their distance from the center of rigidity (see Fig. 6). In the design provisions for earthquake forces of the 1970 *Uniform Building Code*, shear resisting elements are required to resist an arbitrary torsional moment equivalent to the story shear acting with an eccentricity of not less than five per cent of the maximum building dimension at that level. A flexible diaphragm is not considered capable of distributing torsional stresses.



Relative Rigidities:

Wall A	0.33
Wall B	0.22
Wall C	0.45

$$x = (0.22)(16) + (0.45)(40) = 21.52 \text{ ft}$$

$$\text{Torsional moment} = (20,000)(21.52 - 20)$$

$$= 30,400 \text{ ft-lb}$$

If F were applied at center of rigidity, it would be distributed as follows:

$$\text{Wall A: } (0.33)(20,000) = 6600 \text{ lb}$$

$$\text{Wall B: } (0.22)(20,000) = 4400 \text{ lb}$$

$$\text{Wall C: } (0.45)(20,000) = 9000 \text{ lb}$$

Since F is not applied at center of rigidity, forces due to torsional moment are distributed to walls in proportion to their relative rigidities and the distance from the center of rigidity.

$$F_B = F_A \left(\frac{0.22}{0.33} \right) \left(\frac{5.52}{21.52} \right) = 0.17 F_A$$

$$F_C = F_A \left(\frac{0.45}{0.33} \right) \left(\frac{18.48}{21.52} \right) = 1.17 F_A$$

$$21.52F_A + 5.52F_B + 18.48F_C = 30,400 \text{ ft-lb}$$

$$21.52F_A + 5.52(0.17F_A) + 18.48(1.17F_A) = 30,400$$

$$44.06F_A = 30,400$$

$$F_A = 690 \text{ lb}$$

$$F_B = 117 \text{ lb}$$

$$F_C = 808 \text{ lb}$$

Due to torsional moment, forces on walls A and B are increased, while force acting on wall C is decreased. However, to be on safe side in designing the walls, no reduction should be made. Therefore, the walls should be designed to resist the following forces:

$$\text{Wall A} = 6600 + 690 = 7290 \text{ lb}$$

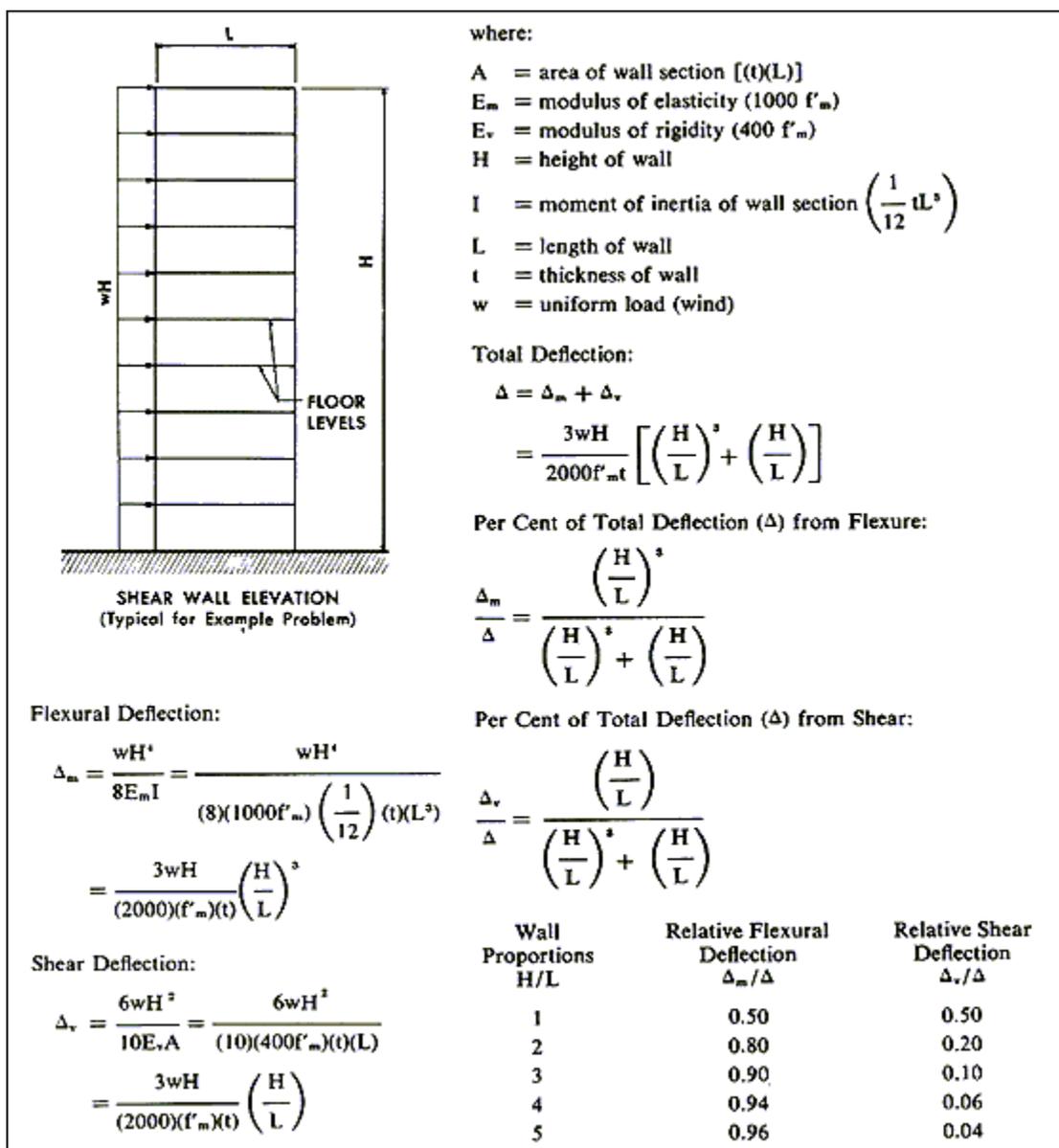
$$\text{Wall B} = 4400 + 117 = 4517 \text{ lb}$$

$$\text{Wall C} = 9000 \text{ lb}$$

Torsional Movement on a Shear Wall System

FIG. 6

When dealing with a rigid diaphragm and distributing the horizontal forces to vertical resisting elements in proportion to the relative rigidities, the relative rigidity of the shear wall is dependent upon the shear and flexural deflections. However, for the proportions of the shear walls in most high-rise buildings, the flexural deflection greatly exceeds the shear deflection, in which case only flexural rigidity need be considered in determining the relative stiffness of the shear walls. For determination of relative shear and flexural deflections, see Fig. 7.



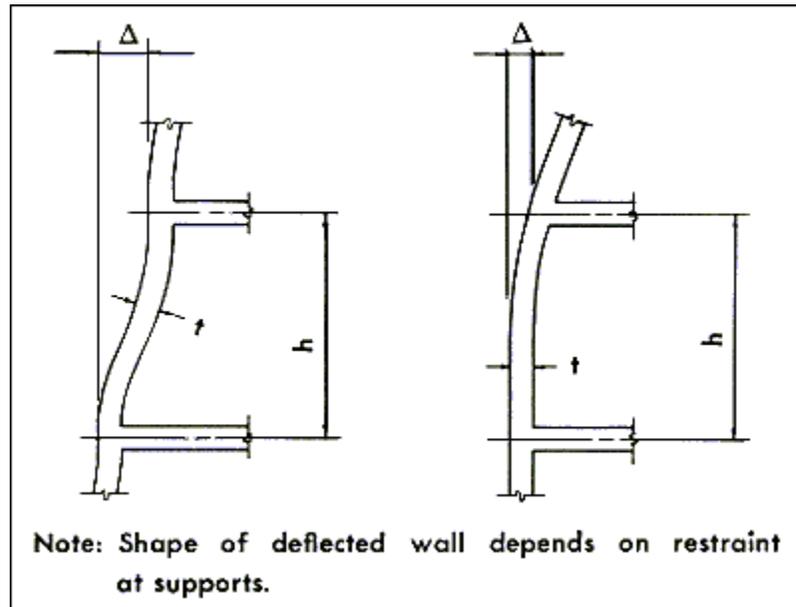
Relative Shear and Flexural Deflections Determined

for a Uniformly Loaded Cantilever Member of Rectangular Section

FIG. 7

A rigorous analysis of the lateral load distribution to the shear wall is sometimes very time-consuming and frequently unjustified by the results. Therefore, in many cases a design based on reasonable limits may be used. For example, the load may be distributed by first considering the diaphragm as rigid and then by considering it flexible. If the difference is not great, the shear wall can then be safely designed for the maximum applied load.

Diaphragm Deflection. As previously indicated, deflection is another factor that must be considered in designing a horizontal diaphragm. As shown in Fig. 8, diaphragm deflection should be limited to prevent excessive stresses in the walls which are perpendicular to the shear walls. The following formula has been suggested by the Structural Engineers Association of Southern California for allowable deflection of horizontal diaphragms in buildings having masonry or concrete walls:



Diaphragm Deflection Limitation

FIG. 8

$$\Delta = \frac{h^2 f}{0.01 Et}$$

where: Δ = allowable deflection between adjacent supports of wall, in inches

h = height of wall between adjacent horizontal supports, in feet

t = thickness of wall, in inches

f = allowable flexural compressive stress of wall material, in pounds per square inch

E = modulus of elasticity of wall material, in pounds per square inch

The application of these limits on deflection must be used with engineering judgment. For example, continuity at floor level is assumed, which in many cases is not present due to through-wall flashing. In this situation the deflection may be based on the allowable compressive stress in the masonry, assuming a reduced cross section of wall. The effect of reinforcement which may be present in a reinforced brick masonry wall or as a tie to the floor system in a non-reinforced or partially reinforced masonry wall is not considered. It should also be pointed out that the limit on deflection is actually a limit on differential deflection between two successive floor or diaphragm levels.

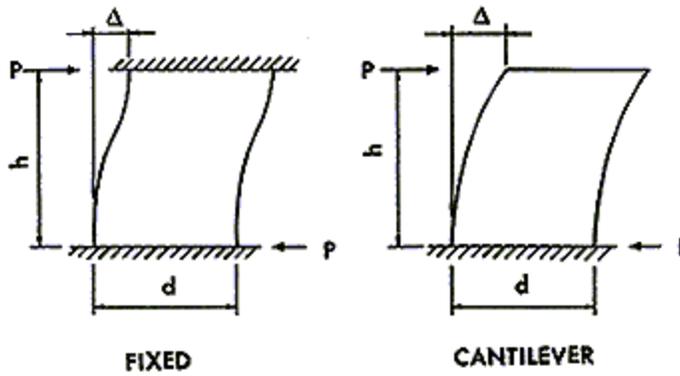
Maximum span-to-width or depth ratios for diaphragms are usually used to indirectly control diaphragm deflection. Normally, if the diaphragm is designed with the proper ratio, the diaphragm deflection will not be critical. As a guide for horizontal diaphragm proportions, Table 2, taken from the State of California Administrative Code, Title 21, Public Works, may prove useful.

TABLE 2
Roof or Floor Diaphragms-Maximum
Span-Width Ratios

Diaphragm Construction	Maximum Span-Width Ratio <i>l/b</i>	
	Masonry and Concrete Walls	Wood and Light Steel Walls
Concrete	Limited by Deflection	
Steel deck (continuous sheet in a single plane)	4:1	5:1
Steel deck (without continuous sheet)	2:1	2 1/2 :1
Poured reinforced gypsum roofs	3:1	
Plywood (nailed all edges)	3:1	4:1
Plywood (nailed to supports only - blocking may be omitted between joists)	2 1/2 :1 ¹	3 1/2 :1
Diagonal sheathing (special)	3:1 ¹	
Diagonal sheathing (conven- tional construction)	2:1 ¹	3 1/2 :1 2 1/2 :1

¹The use of diagonal sheathed or unblocked plywood diaphragms for buildings having masonry or reinforced concrete walls shall be limited to one-story buildings or to the roof of a top story.

Rigidity of Shear Walls. Where shear walls are connected by a rigid diaphragm so that they must deflect equally under horizontal load, the proportion of total horizontal load at any story or level carried by a perpendicular shear wall is based on its relative rigidity or stiffness. The rigidity of a shear wall is inversely proportional to its deflection under unit horizontal load. The total deflection of the shear wall can be determined from the sum of the shear and moment deflections. Equations for the deflection of fixed and cantilevered walls or piers are shown in Fig. 9.



$$\text{Rigidity} = \frac{1}{\Delta}$$

$$\text{Deflection} = \Delta = \Delta_m + \Delta_v$$

$$\text{For fixed wall, } \Delta_m = \frac{Ph^3}{12E_m I}$$

$$\text{For cantilever wall, } \Delta_m = \frac{Ph^3}{3E_m I}$$

$$\text{For fixed or cantilever wall, } \Delta_v = \frac{1.2Ph}{E_v A}$$

where:

- Δ_m = moment deflection, inches
- Δ_v = shear deflection, inches
- P = horizontal force, pounds
- h = height of wall, inches
- I = moment of inertia of wall, inches⁴
- A = area of wall, square inches
- E_m = modulus of elasticity, psi
- E_v = modulus of rigidity, psi

$$\text{If } E_m = 3 \times 10^6 \text{ psi}$$

$$E_v = 0.4 E_m$$

$$t = 12 \text{ in.}$$

$$P = 10^6 \text{ lb}$$

For fixed wall:

$$\Delta = 0.0278 \left(\frac{h}{d}\right)^3 + 0.0833 \left(\frac{h}{d}\right)$$

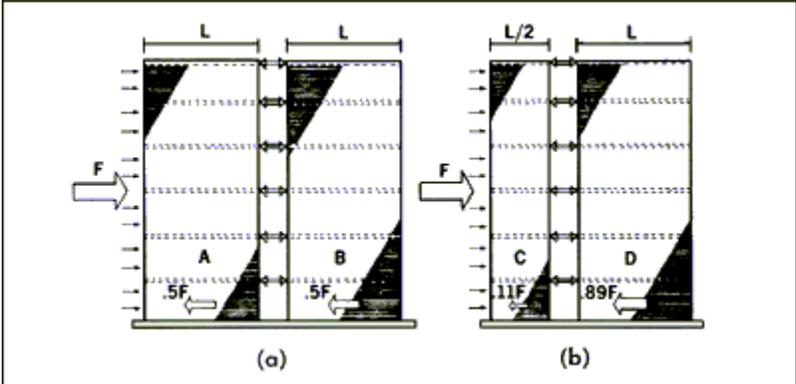
For cantilever wall:

$$\Delta = 0.1112 \left(\frac{h}{d}\right)^3 + 0.0833 \left(\frac{h}{d}\right)$$

Calculation of Wall Deflections

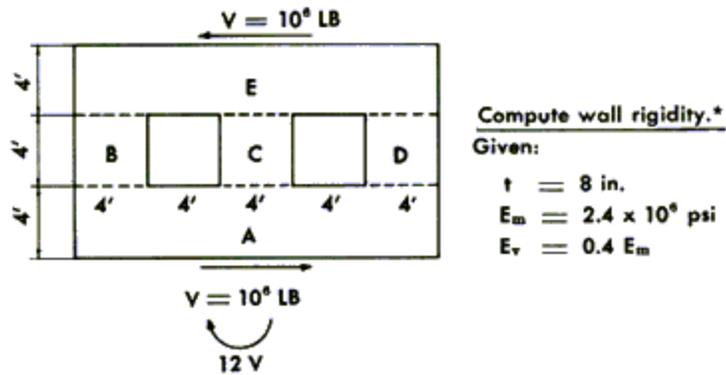
FIG. 9

Where a shear wall contains no openings, the computations for deflection and rigidity are quite simple. In Fig. 10(a), the shear walls are of equal length and rigidity, and each takes one half of the total load. In Fig. 10(b), wall C is one half the length of wall D and it, therefore, receives less than one eighth of the total load. Where shear walls contain openings such as doors and windows, the computations for deflection and rigidity are more complex. However, approximate methods have been developed which may be used. See Fig. 11.



Distribution of Wind Load

FIG. 10



Compute wall rigidity.*

Given:

$$t = 8 \text{ in.}$$

$$E_m = 2.4 \times 10^6 \text{ psi}$$

$$E_r = 0.4 E_m$$

$$\Delta \text{ of Solid Wall. } \frac{h}{d} = \frac{12}{20} = 0.60$$

For cantilever wall (see Fig. 6):

$$\Delta = \left[0.1112 \left(\frac{h}{d} \right)^3 + 0.0833 \left(\frac{h}{d} \right) \right] \left[\frac{3.0}{2.4} \right] \left[\frac{12}{8} \right]$$

$$= 0.138 \text{ in.}$$

$$R = \frac{1}{\Delta} = \frac{1}{0.138} = 7.22$$

$$\Delta \text{ of 4-ft Mid Strip of Wall. } \frac{h}{d} = \frac{4}{20} = 0.20$$

$$\Delta = \left[0.1112(0.2)^3 + 0.0833(0.2) \right] \left[\frac{3.0}{2.4} \right] \left[\frac{12}{8} \right]$$

$$= 0.0328 \text{ in.}$$

$$\Delta \text{ of Piers B, C and D. } \frac{h}{d} = \frac{4}{4} = 1.0$$

For fixed pier (see Fig. 6):

$$\Delta_B = \left[0.0278(1)^3 + 0.0833(1) \right] \left[\frac{3.0}{2.4} \right] \left[\frac{12}{8} \right]$$

$$= 0.208 \text{ in.}$$

$$R_B = \frac{1}{\Delta_B} = \frac{1}{0.208} = 4.81; R_C = 4.81; R_D = 4.81$$

$$\Delta_{BCD} = \frac{1}{(3)(4.81)} = 0.0694$$

$$\Delta(\text{TOTAL}) = 0.138 - 0.0328 + 0.0694 = 0.175 \text{ in.}$$

$$R = \frac{1}{0.175} = 5.71$$

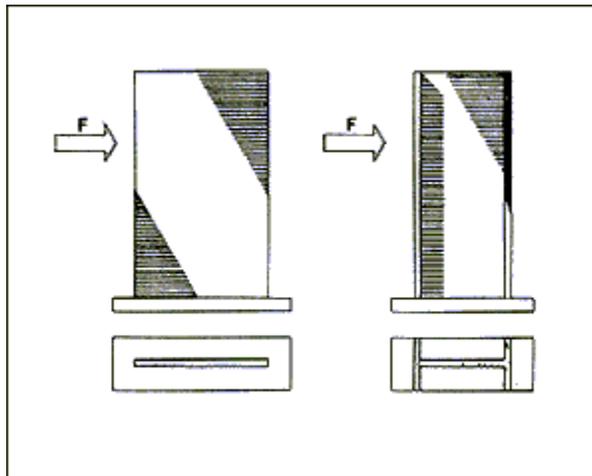
Wall is 79 per cent $\left(\frac{5.71}{7.22} \times 100 \right)$ as rigid as solid wall.

* Based on approximate method outlined in "Seismic Design for Buildings", Departments of Army, the Navy and the Air Force, March 1966.

Calculation of Wall Rigidity

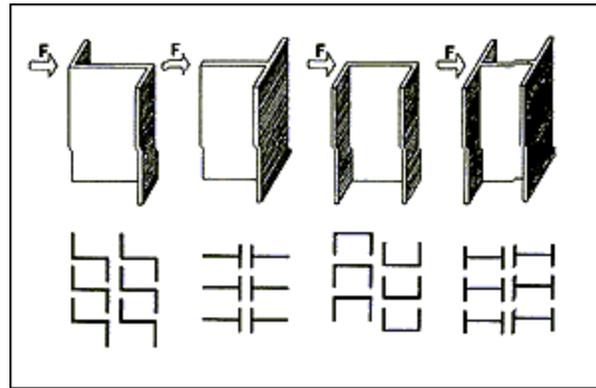
FIG. 11

To increase the stiffness of shear walls as well as their resistance to bending, intersecting walls or flanges may be used. Very often in the design of buildings, Z, T, U, and I-shape sections develop as natural parts of the design. See Figs. 12 and 13. Shear walls with these shapes, of course, have better flexural resistance. The 1969 BIA Standard, *Building Code Requirements for Engineered Brick Masonry*, (Sec. 4.7.12A) limits the effective flange width that may be used in calculating flexural stresses. In the case of symmetrical T or I sections, the effective flange width may not exceed one sixth of the total wall height above the level being analyzed. In the case of unsymmetrical L or C sections, the width considered effective may not exceed one sixteenth of the total wall height above the level being analyzed. In either case, the overhang for any section may not exceed six times the flange thickness (see Figs. 14 and 15). It is, of course, necessary to insure that the shear stress at the intersection of the walls does not exceed the permissible shear stress. This will depend on the method used in bonding the two walls together.



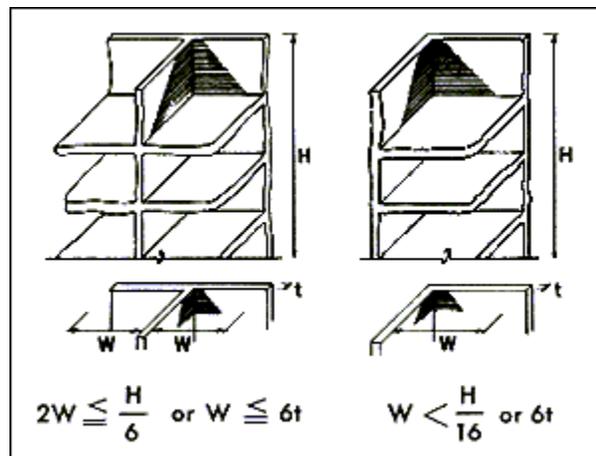
Shear Walls of Equivalent Stiffness

FIG. 12



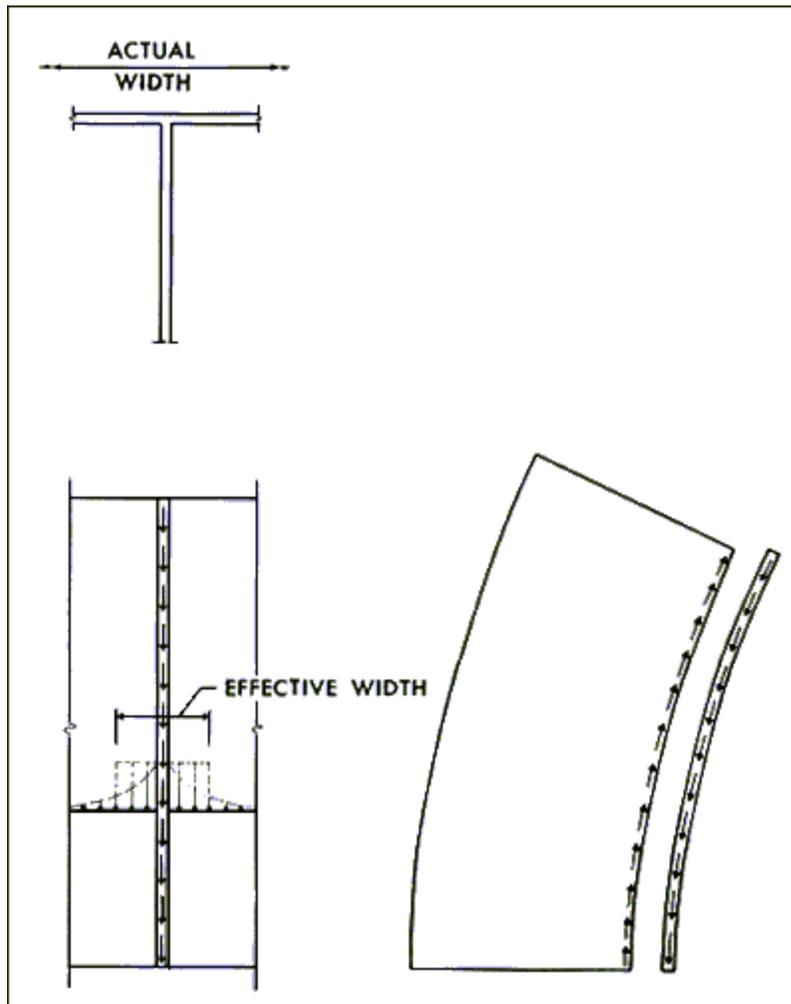
Shear Walls With Flanges

FIG. 13



Effective Flange Width

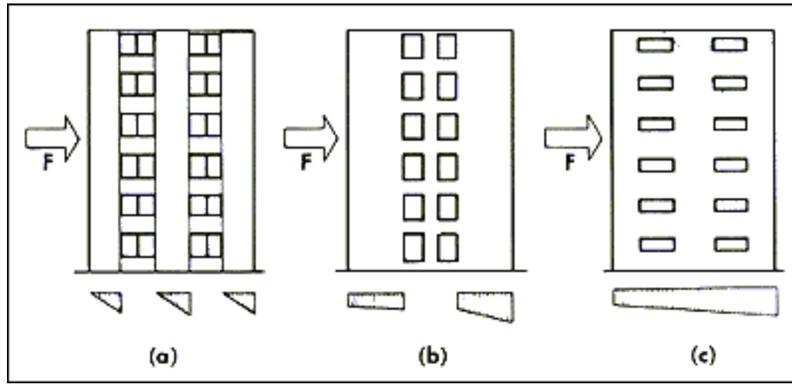
FIG. 14



Effective Flange Width

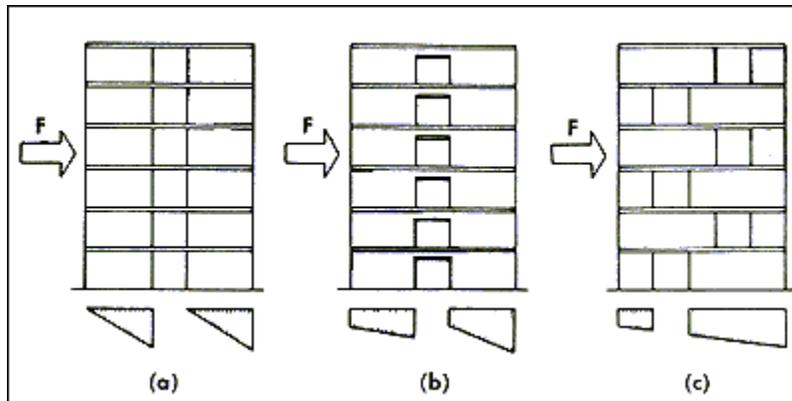
FIG. 15

Coupled Shear Walls. Another method that may be used to increase the stiffness of a bearing wall structure and reduce the possibility of tension developing in shear walls due to wind parallel to the wall is the coupling of collinear shear walls. Figures 16 and 17 indicate the effect of coupling on the stress distribution in the wall due to parallel forces. A flexible connection between the walls is assumed in Figs. 16(a) and 17(a), so that the walls act as independent vertical cantilevers in resisting the lateral loads. Figures 16(b) and 17(b) assume the walls to be connected with a more rigid member which is capable of shear and moment transfer so that a frame-type action results. This can be accomplished with a steel, reinforced concrete or reinforced brick masonry section. The plate type action, which is indicated in Figs. 16(c) and 17(c), assumes an extremely rigid connection between walls, such as full story height walls or deep rigid spandrels.



End Shear Walls

FIG. 16



Interior Shear Walls

FIG. 17

SHEAR STRENGTH

Test Data. The present standard racking test, described in ASTM E 72-68, *Method of Conducting Strength Tests of Panels for Building Construction*, provides only a relative measure of the shearing or diagonal tension resistance of a wall. Results of this test method are consequently valid only for comparison purposes and are not suggested for determination of design values.

In this method of test, horizontal movement of the wall specimen (8 by 8 ft), due to the horizontal racking load at the top of one end, is prevented by a stop block at the bottom of the other end. To counteract rotation of the specimen due to this overturning couple, tie rods are used near the loaded edge of the wall specimen. Under racking load these rods superimpose an indeterminate compressive force which suppresses the critical diagonal tensile stresses and increases the load required to rack the specimen. A summary of the racking data for non-

reinforced brick masonry walls tested in accordance with ASTM E 72 is given in Table 3. A typical mode of failure for a 4-in. brick wall subject to racking is shown in Fig. 18.

¹Tested in accordance with ASTM E 72.

²Walls not loaded to failure.

³"Structural Properties of Six Masonry Wall Constructions", H. L. Whittemore, A. H. Stang and D. E. Parsons, National Bureau of Standards Report BMS5, 1938.

⁴"Structural Properties of a Brick Cavity-Wall Construction". H. L. Whittemore. A. H. Stang, D. E. Parsons, National Bureau of Standards Report BMS23, 1939.

⁵"SCR brick^{*} Wall Tests", C. R. Monk, Jr., Structural Clay Products Research Foundation, Research Report No. 1, June 1953.

⁶"Compressive, Transverse and Racking Strength Tests of Four-Inch Brick Walls", Structural Clay Products Research Foundation, Research Report No. 9, August 1965.

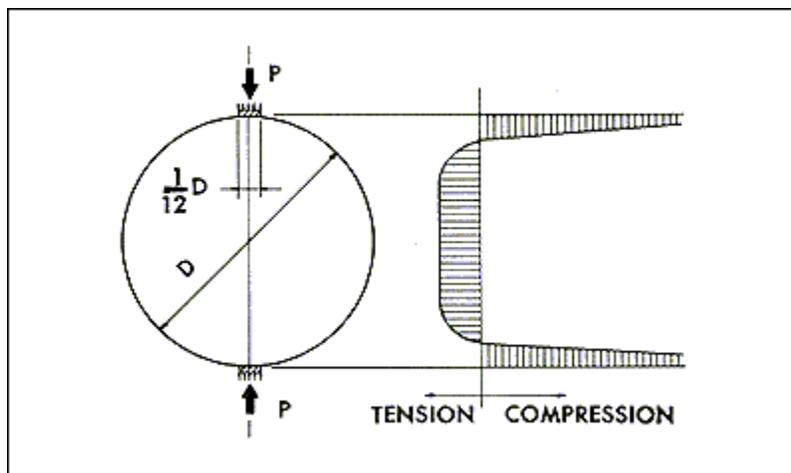
^{*}Reg. U.S. Pat. Off., SCPI



Typical Mode of Failure in Racking

FIG. 18

Circular Shear Specimens. Based on experimental work done at the Balcones Research Center of the University of Texas by Professors Neils Thompson and Frank Johnson, the Research Division of the Structural Clay Products Institute conducted a series of diagonal tensile tests on circular brick masonry specimens. In these tests, a 15-in. diameter specimen is tested in compression with the line of load at 45 deg to the bed joints. As shown in Fig. 19 the diametrical stresses are largely tensile over the central 80 per cent of the specimen. The tensile stress is approximately constant for about 60 per cent of the diameter and may be calculated by the following equation:



Stress Distribution in Tensile Splitting Test

FIG. 19

$$f'_t = \frac{2P}{.8Dt}$$

where: P = load at rupture, in pounds

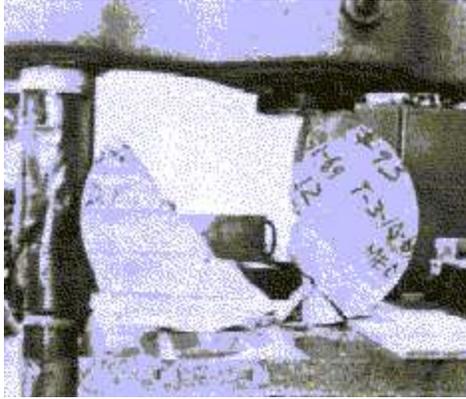
D = diameter of specimen, in inches

t = thickness of specimen, in inches

The test results for 133 specimens built with 27 types of brick and type S portland cement-lime mortar of ASTM C 270, *Specifications for Mortar for Unit Masonry*, are summarized in Table 4. A typical mode of failure for a circular brick specimen is shown in Fig. 20. While there was no consistent relationship between diagonal tensile strength and brick properties, such as initial rate of absorption, it appeared that brick with the weakest bond characteristics as shown by flexural strength values also yielded the lowest diagonal tensile strengths.

¹"Small Scale Specimen Testing", Progress Report No. 1, SCPI-SCPRF, October 1964.

²All specimens built with type S mortar.



Small-Scale Diagonal Tension Test

FIG. 20

The test results for 20 circular specimens built with one type of brick and four types of mortars are summarized in Table 5. As indicated, the mortar type had a marked effect on the diagonal tensile strength of circular back masonry specimens.

¹"Small Scale Specimen Testing", Progress Report No. 1. SCPI-SCPRF, October 1964.

²C = portland cement; L = hydrated lime (type S); S = sand.

³28-day briquets.

⁴All specimens built with 3/8-in. joints and brick having an average compressive strength of 11,771 psi and an initial rate of absorption of 10.6 g per min per 30 sq in.

Square Shear Specimens. The Brick Institute of America has continued to study the diagonal tensile or shear strength of brick masonry in an effort to develop both test methods and design information. Working with the National Bureau of Standards and the Research Division of John A. Blume and Associates (San Francisco), a test method has been developed that has several advantages over the ASTM E 72 racking test procedure. As previously stated, in the E 72 procedure the hold-down tie rods required to prevent overturning of the specimen under load produce an indeterminate bearing condition at the bottom edge of the specimen, thus preventing an analytic determination of the stress within the wall specimen itself.

In the alternate test procedure, which will be submitted to ASTM as a proposed alternate to the ASTM E 72 procedure, the specimens are nominally 4 ft by 4 ft as opposed to the 8-ft square specimen in the E 72 procedure. In this method, the test results are susceptible to stress analysis. In addition, they are more reproducible and thus more reliable for comparison and design data purposes.

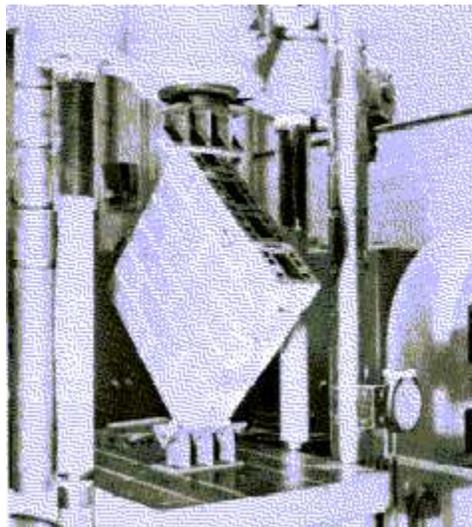
The square specimen is placed in the testing frame so as to be loaded in compression along a diagonal, thus producing a diagonal tension failure with the specimen splitting apart along the loaded diagonal (see Figs. 21 and 22). The results of both small scale (2 ft square) and full scale (4 ft square) tests of 4, 6 and 8-in. thick brick masonry specimens are summarized in Table 6 and typical shear stress-strain curves are displayed in Fig. 23. All specimens were constructed with type S mortar. The shearing stress V'_m is determined by the equation

$$v'_m = \frac{0.07F}{tl}$$

where: F = diagonal compressive force or load, in pounds

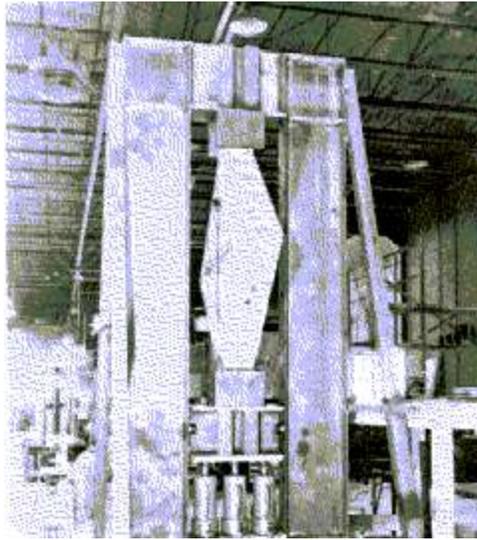
t = thickness of wall specimen, in inches

l = length of a side of a square specimen, in inches



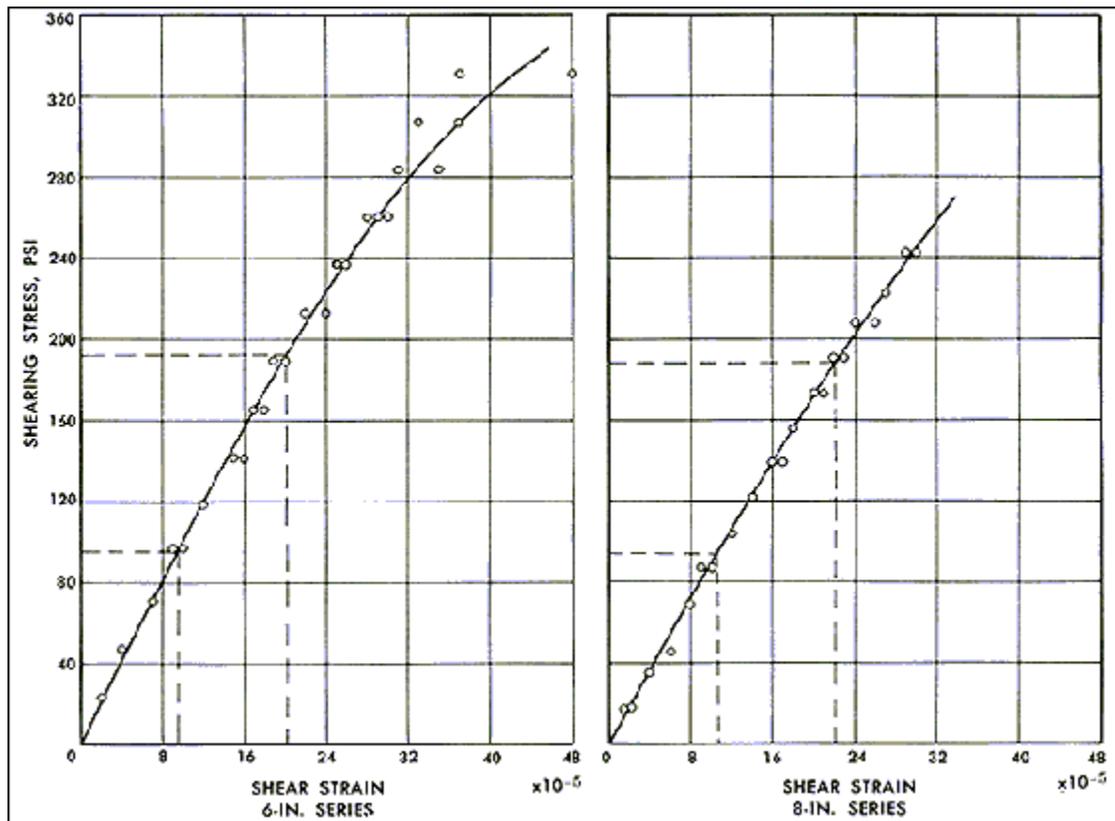
Wallette Shear Test Setup

FIG. 21



Full Scale Shear Setup

FIG. 22



Typical Shear Stress-Strain Curves for 6 and 8 Inch Walls

FIG. 23

12 ft square.

24 ft square.

Effects of Normal Loads. The Brick Institute of America has also investigated the effects of compressive loads normal to the bed joints on the shearing strength of plain (non-reinforced) brick masonry walls. Table 7 summarizes the results of these tests with the normal compressive load on the wall varying from 0 (unloaded) to 375 psi. The specimens were built with one type of brick utilizing type S mortar and inspected workmanship. All specimens were two wythes, 8 in. in thickness and bonded with metal ties.

¹Built with type S mortar, inspected workmanship and metal-tied.

fb = 11,100 psi

Allowable Stresses. The allowable shear stresses for non-reinforced masonry provided in the 1969 SCPI Standard, *Building Code Requirements for Engineered Brick Masonry*, are shown in Table 8, and for reinforced masonry in Table 9.

The Standard also provides a basis for the design of biaxially loaded shear walls. Section 4.7.12.1 states:

"In non-reinforced shear walls, the virtual eccentricity (e) about the principal axis which is normal to the length (l) of the shear wall shall not exceed an amount which will produce tension. In non-reinforced shear walls subject to bending about both principal axes, $(e_l / l + e_t / t)$ shall not exceed $(t / 3)$ where e_t = virtual eccentricity about the principal axis which is normal to the thickness (t) of the shear wall. Where the virtual eccentricity exceeds the values given in this section, shear walls shall be designed in accordance with Section 4.7.9 or 4.7.11". (Reinforced or partially reinforced walls.)

Provision is also made in the standard for increasing the shear capacity of the wall by taking into consideration compressive loads on the wall. Section 4.7.12.3 states:

"The allowable shearing stresses in non-reinforced and reinforced shear walls shall be taken as the allowable stresses given in Tables 3 and 4 (Tables 8 and 9 of this *Technical Notes*) ,respectively, plus one fifth of the average compressive stress due to dead load at the level being analyzed. In no case, however, shall the allowable shear stresses exceed the maximum values given in Tables 3 and 4.

"In computing the shear resistance of the wall, only the web shall be considered."

TABLE 8
Allowable Shear Stresses in Non-Reinforced Brick Masonry

Description	Allowable Stresses, psi	
	Without Inspection	With Inspection
Shear M or S mortar v_m	$0.5\sqrt{f'_m}$, but not to exceed 40	$0.5\sqrt{f'_m}$, but not to exceed 80
N mortar v_m	$0.5\sqrt{f'_m}$, but not to exceed 28	$0.5\sqrt{f'_m}$, but not to exceed 56

REFERENCES

1. *Technical Notes on Brick Construction*, BIA, (a monthly series).
2. Gross, J. G.; Dikkers, R. D.; and Grogan, J. C.; *Recommended Practice for Engineered Brick Masonry*, SCPI, November 1969.
3. Allen M. H. and Watstein, D.; *Compressive, Transverse and Shear Strength Tests of Six and Eight-Inch Single-Wythe Walls Built with Solid and Heavy-Duty Hollow Clay Masonry Units*, Research Report No. 16, SCPI, September 1969.
4. Blume, J. A. and Prolux, J.; *Shear in Grouted Brick Masonry Wall Elements*, Western States Clay Products Association, August 1968.