concrete, the type of reinforcement, arrangement of reinforcing steel and special details such as joints and connections in shear walls are also presented.

ACKNOWLEDGEMENTS

The writer wishes to express his gratitude to Dr. Zenon A. Zielinski, Professor of Civil Engineering, Concordia University for his advice and guidance, who helped immensely in the author's writing of this report.

Thanks are also extended to Dr. H.K.Ha, Assistant Professor, for reviewing, guiding and correcting this report.

My final acknowledgement is to my wife, Tajwar, for her continuous encouragement and patience.

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NOTATIONS

A_c = area of columns

A_{c_1}, A_{c_2} = areas of wall sections

a = distance of applied load

B = width of frame

b = clear span of beam

E_c = Young's modulus of elasticity of concrete

E_s = Young's modulus of elasticity of steel

F_g, F_m, F_n, F_s = functions used in Equations A and B dependent on the type of loading

g = ratio $\dfrac{I_b \text{ at top of frame}}{I_b \text{ at bottom of frame}}$ as used in Table 2.1

H = total height of wall

h = height of column, i.e., storey height

I_b = moment of inertia of a connecting beam

I_{c_1}, I_{c_2} = moment of inertia of wall sections

I_w = moment of inertia of wall

K_B = rotational stiffness of shear wall support

K_f = lateral point load applied at top of frame to cause unit deflection in its line of action

K_i = factor representing both K_ω and K_f

K_ω = lateral point load applied at the top of a shear wall to cause unit deflection in its line of action; also the stiffness of a shear wall without openings

$K_{\omega o}$ = stiffness of a shear wall with openings

ℓ = distance between centroidal axes of walls or columns or span of beams

m = g or s in equation of Table 2.1

K_1, K_2 = wall bending stress factors

n = ratio $\frac{A_c \text{ at top of frame}}{A_c \text{ at bottom of frame}}$ as used in Table 1

P = interaction load at the top of the frame

R = support reaction coefficient for a propped cantilever

S = ratio $\frac{I_c \text{ at top of frame}}{I_c \text{ at bottom of frame}}$ as used in Table 1

W = total applied lateral load (subscript denotes load on specific wall)

X, Y, Z = coordinate directions or displacement in these directions

α = a variable used for shear wall with opening - see Section 3.4

β_c = c/ℓ

β_o = D/h

ν_ω = dimensionless parameter which relates the rotational stiffness of the wall to that of the foundation, i.e.,

$$\text{ratio } \frac{K_{BH}}{4E_\omega I_\omega}$$

Δ = total deflection at top of structure

ΔA = top deflection due to axial deformation

ΔB = top deflection due to bending deformation

θ = a deformation at top of structure with torsion

λ = ratio of column-to-beam stiffness

$$\left(\frac{E_c I_c / h}{E_b I_b / \ell}, \text{ etc.} \right)$$

μ = variable used for shear wall with openings (see Section 3.4)

U = internal stored energy or energy absorbed

f_{cu} = average flexural compressive stress of unconfined concrete at ultimate

$q = p\, f_y/f'_c$ = reinforcement index

$q_u = p\, f_y/f_{cu}$ = useful limit of q for tensile reinforcement

$q_u' = p'\, f'_y/f_{cu}$ = useful limit of q for comp. reinforcement

CHAPTER 1

INTRODUCTION

1.1 DEFINITION OF SHEAR WALL

A reinforced concrete shear wall in a multi-storey reinforced concrete building is essentially a deep, slender cantilever beam, to resist lateral forces when frame systems alone are insufficient or when it is convenient to make partitions load-bearing.

Shear walls can have various cross-sections such as: rectangular, I-shaped, box, which are usually found as elevator walls, stairwells and central unit core. The shear walls support the vertical load in addition to their function to stiffen the frames to resist lateral loads due to winds, earthquake and blast.

Although interior and exterior concrete walls have been used to stiffen structures as long as reinforced concrete itself has been in use. The modern concept of shear walls designed as vertical slender cantilever was first utilized in 1948 in housing projects in New York City and in Chicago in buildings designed for wind forces, to augment the lateral resistance of the frame.

1.2 ELEMENTS RESISTING SEISMIC FORCES

1.2.1 Reinforced Concrete Shear Walls

Reinforced concrete walls cast integrally with columns and girders are effective structural elements providing resistance to lateral forces. Among these reinforced concrete walls, those which are given enough thickness and reinforcement are called the "shear walls" or the "quake-resisting walls". The stiffness and strength of reinforced concrete walls is much higher than that of open frames. They can carry seismic forces also acting on other portions of the building through the slab connected to them. Thus they can resist seismic forces with the co-operation of other frames or other walls. An arrangement of shear walls is illustrated in Figure 1.1. Openings in a shear wall are permissible, but the stiffness and strength may be considerably reduced due to openings.

1.2.2 Walled Frames and Frames With Reinforced Concrete Bracings

Walled frames which consist of girders and wide columns can also be used as an effective antiseismic element. The exterior wall which may act as an antiseismic element is shown in Figure 1.2. The resisting capacity of the walled frames to lateral forces falls between those of shear walls and ordinary open frames. As the deflection

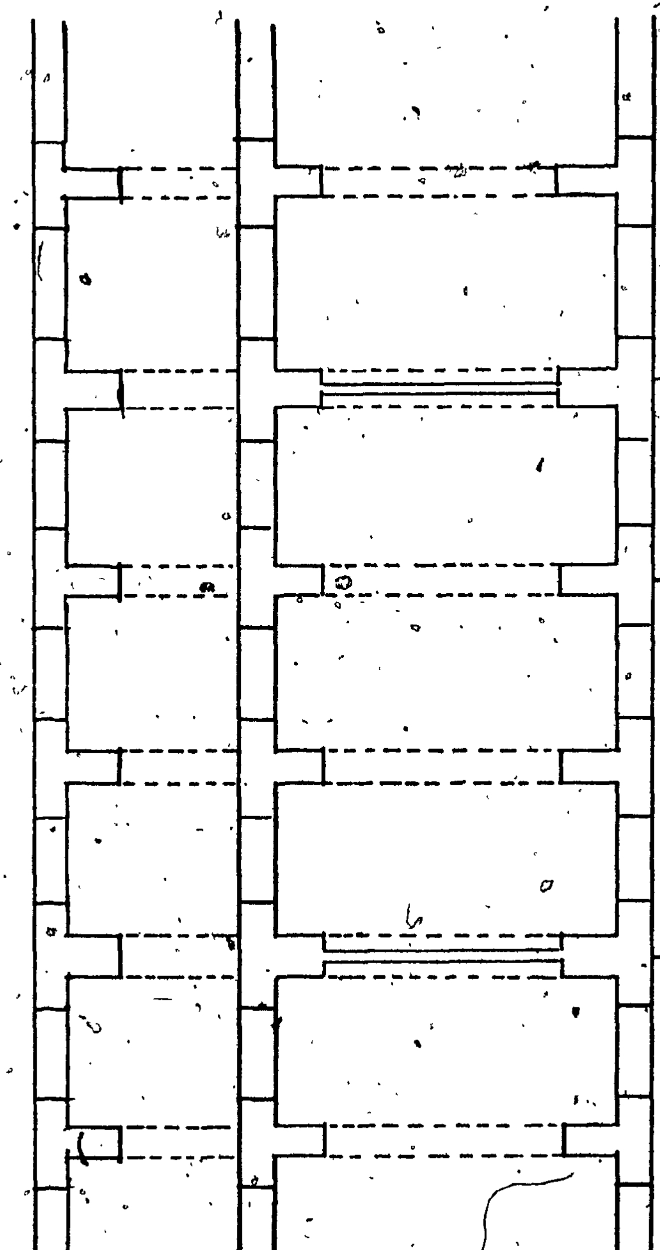


FIG. 1.1 Frame With Shear Wall

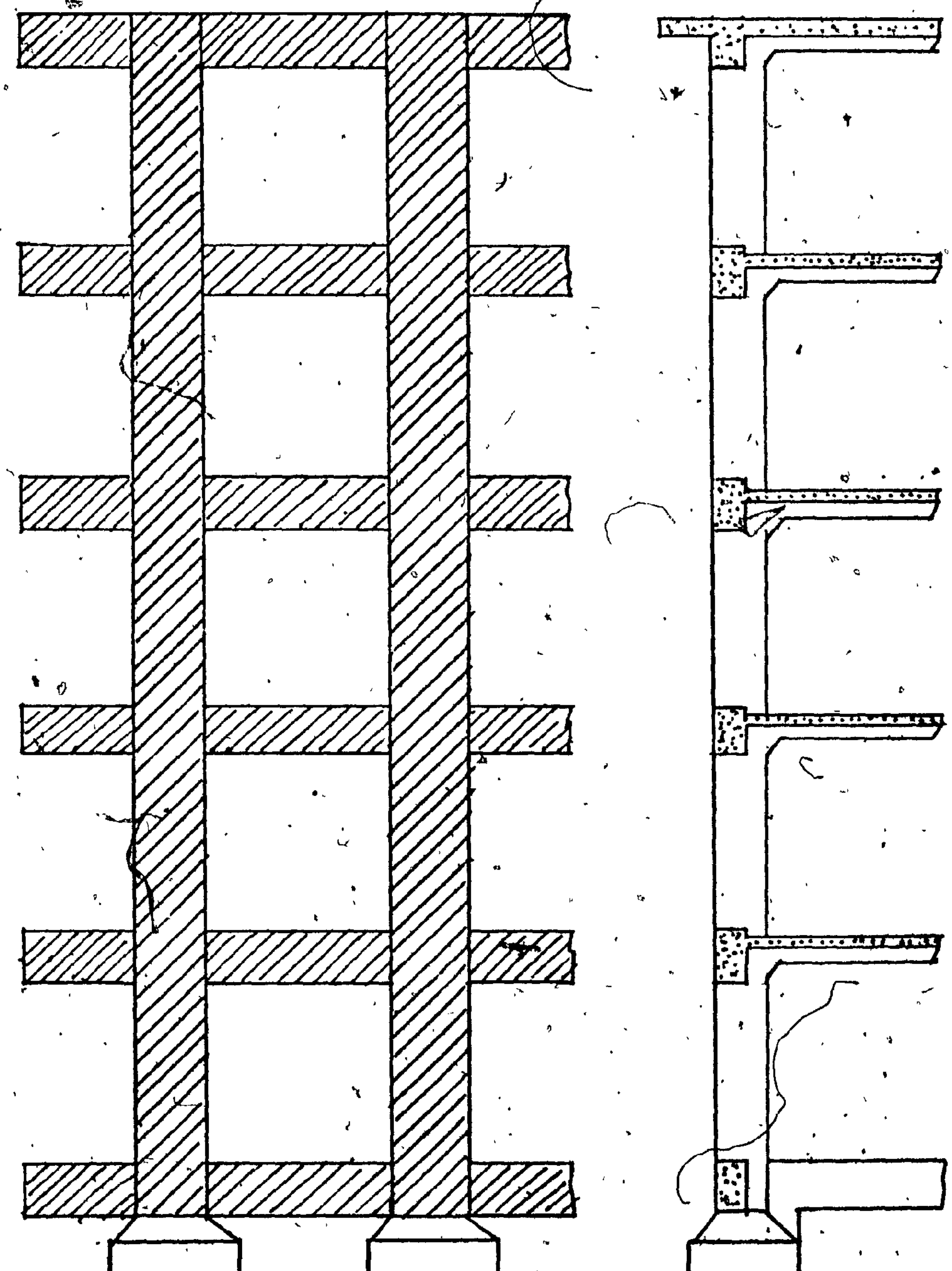


FIG. 1.2 Walled Frame With Openings

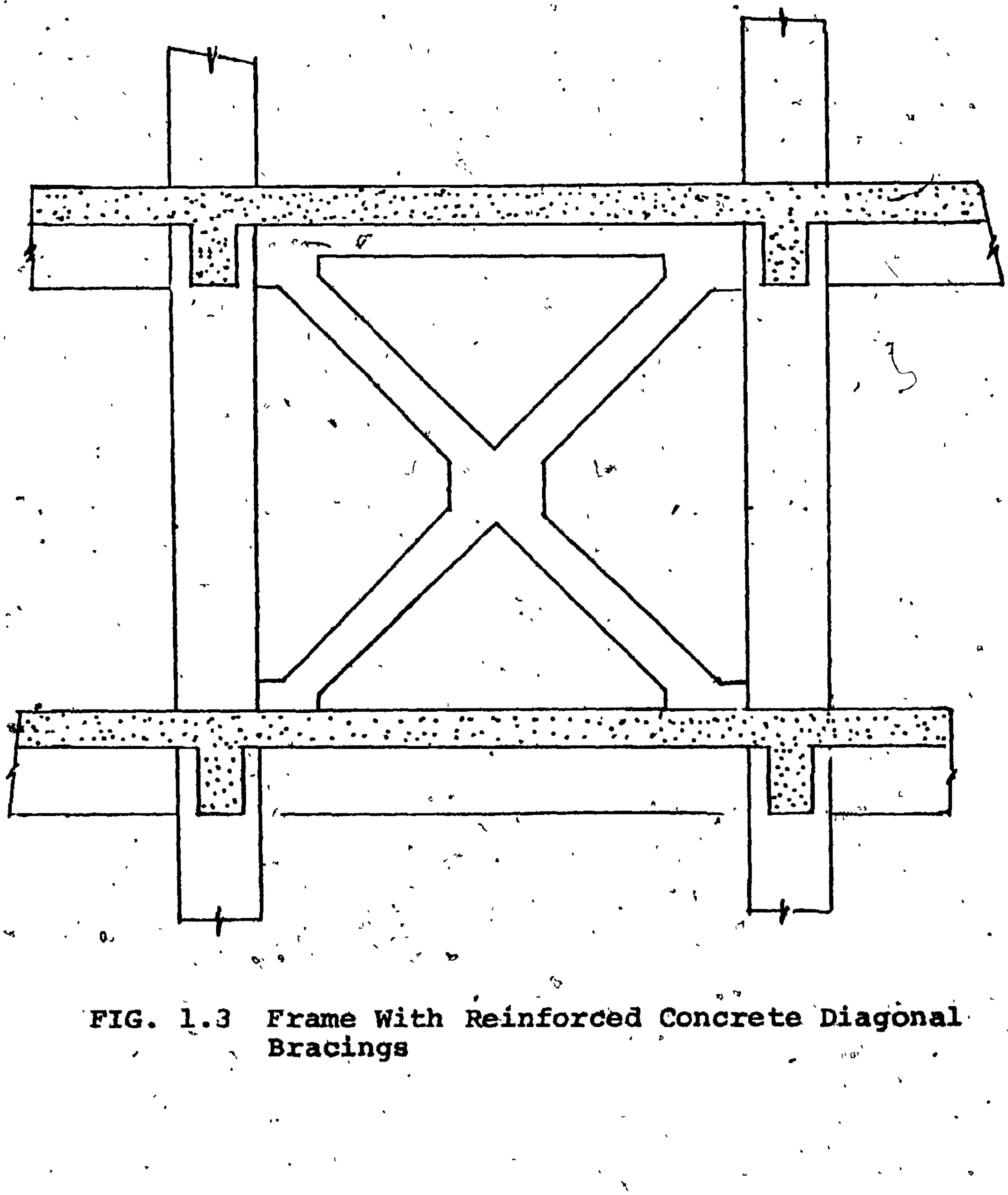


FIG. 1.3 Frame With Reinforced Concrete Diagonal Bracings

pattern of the walled frame due to lateral forces is similar to that of ordinary frame, both the upper and lower parts are almost equally effective, while the upper part of the shear wall is not so effective because of large deflection due to cantilever action.

Reinforced concrete braced frames which have enough strong reinforced concrete diagonal bracings may introduce similar effect as full reinforced concrete shear walls (Fig. 1.3). However, in these frames, attention should be paid to the concentration of stresses at the end of bracings and anchorage of reinforcing bars.

1.2.3 Masonry Walls With Embedded Reinforced Concrete Frames

In general, ordinary masonry walls made of concrete blocks are not considered as antiseismic elements. If they are properly reinforced with steel bars in order to prevent the collapse during earthquakes, they can act as monolithic shear walls.

1.3 CLASSIFICATION OF SHEAR WALLS IN TERMS OF WORK

The SEAOC [23] introduced factor K describing the ductility of shear walls. The value of K depends on the types of frames resisting the lateral loads. The structures with very high ductility such as space frames have the lowest value of K, while structures with non-ductile lateral force resisting elements like solid shear walls have high values of K.

Depending on ductility, shear walls can be classified as follows [16]:

(a) Shear Shear-Walls

(b) Moment Shear-Walls

(c) Ductile-Moment Shear Walls

In all three classes, a shear wall acts as a cantilever by itself alone, or in combination with other shear walls, or in combination with beam-column space frames linked by connecting beams or slabs.

(a) The shear-shear-wall is a wall whose primary deformation is due to shear strain and whose primary energy absorbing capacity is shear strain energy when the wall is acted upon by lateral forces in the plane of the wall.

This type of shear wall has a low H/D ratio and would be designed as a deep beam. Such walls are common in concrete block, in precast and in poured-in-place concrete walls in low-rise structures. It is suggested that for shear shear-walls the K factor=1.33 [16] should be used in earthquake formula given by the SEAOC Code. [23]

(b) The Moment shear-wall is a wall whose primary mode of deformation is bending and whose primary energy absorbing capacity is bending strain energy when the wall is acted upon by lateral forces in the plane of the wall.

Its behaviour is characteristically that of a flexural beam. This type of shear wall is commonly found in high-rise shear-wall structures. In order to qualify as a Moment Shear-Wall, the wall should have shear deformations which are 10% or less of the total deformation (flexural plus shear) under the action of lateral loads [16]. It is necessary to carry out a structural analysis of the building taken as a whole; this analysis will reveal, among other things, the ratio of shear deformation to total deformation in each shear wall to be used for the classification to be made.

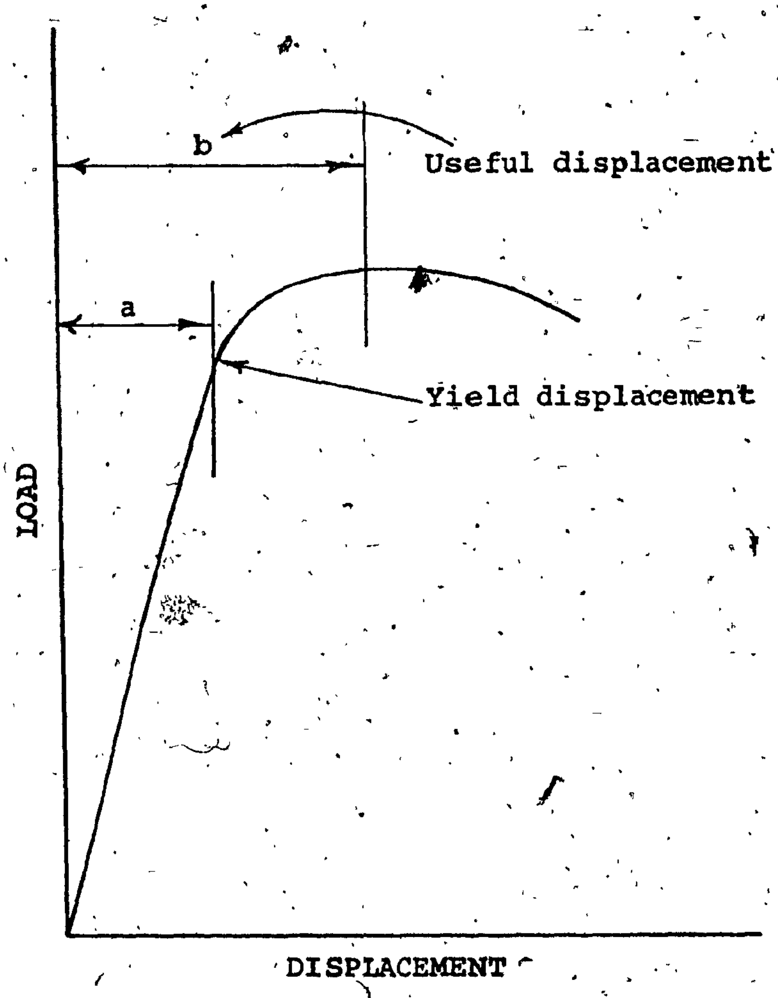


FIG. 1.4 Load-Displacement Curve

(c) The Ductile Moment-shear-wall is a moment shear-wall which has a ductility factor of three or greater. [16]

Ductility of a structure is its ability to undergo increasing deformation beyond the initial yield deformation while still sustaining load. This is illustrated by simple load-deflection curve in Fig. 1.4.

Ductility factor is defined as the ratio of the maximum permissible or useful inelastic deflection or displacement to the initial yield deflection. In Figure 1.4, the ductility factor would be the ratio b/a. It is obvious that the determination of the value of the ductility factor for a reinforced concrete moment-resisting frame is a complex problem involving bending and shear deformations of heterogeneous members consisting of concrete and ductile reinforcing steel. The treatment of this complex problem is presented in Reference [1].

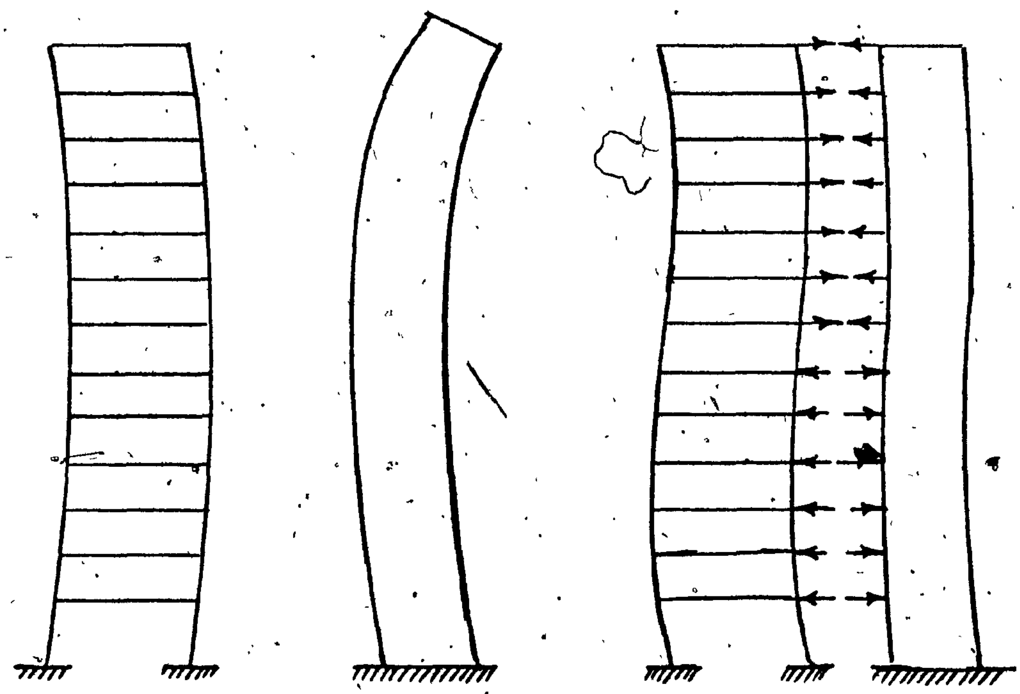
In consideration of what K factors are appropriate to such walls, a minimum ductility factor of 4, for ductile moment-resisting space frame has been recommended by the Portland Cement Association [1], and that in the SEAOC Code, [23], a ductile moment resisting space-frame with an assumed minimum ductility factor of 4, is given a K value of 0.67. Hence, a Ductile-Moment shear-wall with a ductility factor of 4 or more should qualify for a K value of 0.67.

1.4 ANALYSIS FOR LATERAL LOADS

Analysis for lateral loads of buildings containing shear walls was carried out initially, in the '50's, by assigning [12,13] all the lateral loads to the shear walls, since it was felt that the very big difference in stiffness between the shear walls and the frame allow the shear walls to accept the total lateral loads. This inaccurate assumption may have been conservative for the computation of shear wall moments; it is, however, not conservative for the frame, and particularly in the upper parts of the building.

Formal procedures for shear wall-frame interaction were first introduced in the early '60's, resulting in substantial increase of the overall stiffness of the combined system. [2,4]

Most of the recent prominent ultra-high-rise reinforced concrete buildings were built without any additional cost for the lateral resistance. The high lateral rigidity was achieved as a result of shear wall-frame interaction [2],[3].



(a) Rigid frame shear mode deformation

(b) Shear wall bending mode deformation

(c) Interconnected frame and shear wall (Equal deflections at each storey level)

FIG. 1.5 Deformation Modes

A shear wall deflects predominantly in a bending mode, i.e., as a cantilever, as shown in Fig. 1.5 (b). Elevator shafts, stairwells and reinforced concrete walls normally exhibit this behaviour.

It is not always easy to differentiate between modes of deformation. For example, a shear wall weakened by a row (or rows) of openings can tend to act like a rigid frame and conversely an infilled frame will tend to deflect in a bending mode.

When all vertical units of a structure exhibit the same behaviour under lateral loads, i.e., if they are all rigid frames or all shear walls, the analysis is comparatively simple. The load can be distributed to the units directly in proportion to their stiffnesses. The difference in behaviour under lateral loads, in combination with the in-plane rigidity of the floor slabs, causes non-uniform interacting forces to develop when walls and frames are present. This makes the analysis more difficult.

CHAPTER 2

METHODS OF SHEAR WALL ANALYSIS

2.1 ANALYTICAL METHODS

Prior to the early 1960's little attention was paid to the development of analytical techniques for shear walls. Early papers by European authors introduced a period of increased activity in shear wall research which led to the first international conference concerned with the subject in 1966. [24] Since that time considerably more research information has become available.

2.1.2 Continuum Approach

This technique was first applied to the analysis of coupled shear walls by Beck [18], but probably the most comprehensive treatment has been by Rosman [19] who has extended the original analysis to take into account the rotations of the cross-sections of the relatively wide walls, the axial forces in the walls and the effects of different foundation conditions.

In its most basic form, the theory assumes that elastic structural properties of the coupled wall system remain constant throughout, that both walls are founded in a common, stiff footing and that the point of contraflexure

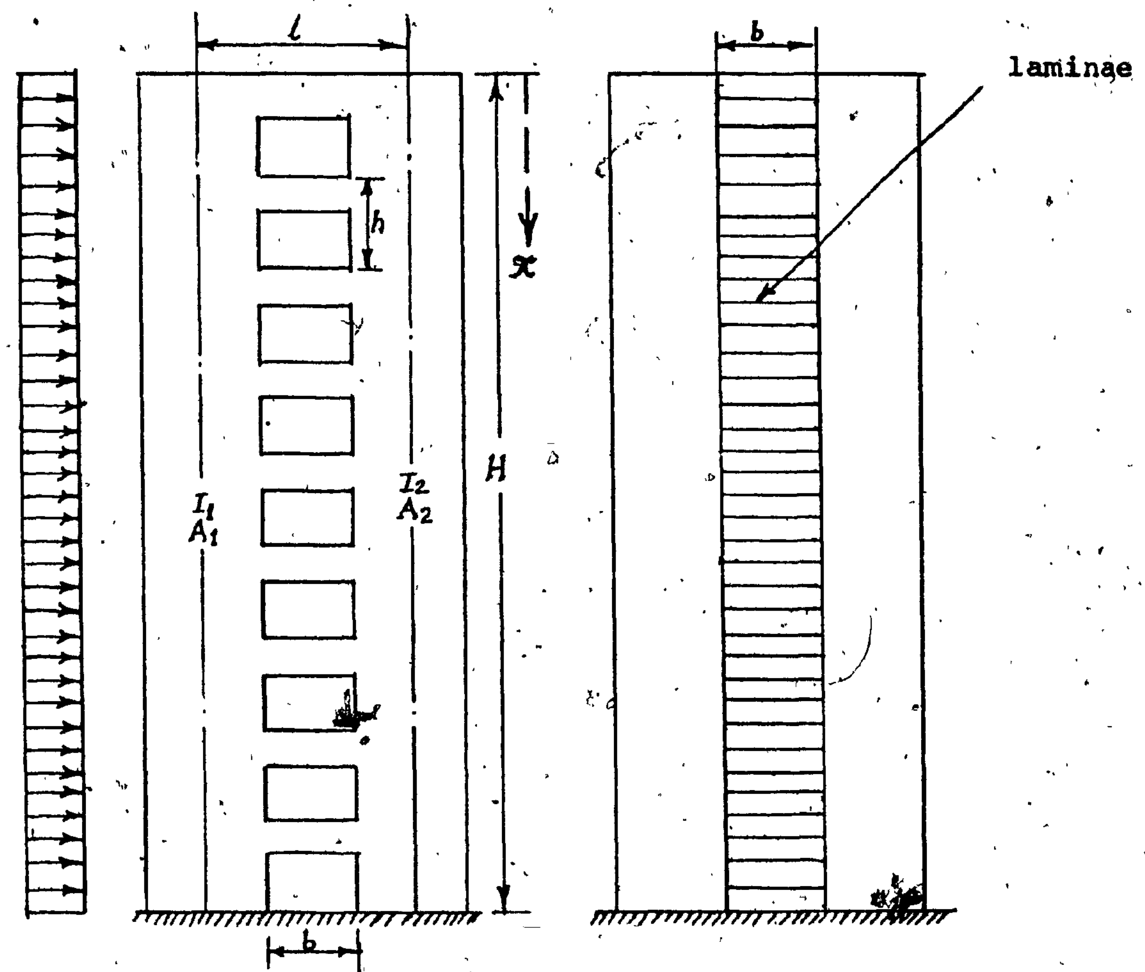


FIG. 2.1 Coupled Shear Walls

of all beams are at midspan.

For this method, the individual connecting beams, Fig. 2.1(a), are replaced by a continuous connection of laminae, Fig. 2.1 (b). Under the horizontal loading, the wall deflects and induces shear forces in the lamina. A second order differential equation is set up and solved to give shear moments and deformations throughout the wall. Several papers [5],[18],[19] use this approach with different choices of variables,all yielding essentially the same results.

The continuum approach can be used to illustrate the basic behaviour of coupled wall systems. Hand analysis is feasible and it can be programmed for a small computer.

One such approach is given by Coull and Choudhry [6] in which curves are presented for the rapid evaluation of the stresses and maximum deflections in any system of coupled shear walls. The curves are derived from the continuum theory, in which discrete systems of connecting beams are replaced by an equivalent continuous medium.

In a coupled shear wall structure, Fig. 2.2, the individual connecting beams of stiffness EI_p are replaced by an equivalent continuous medium or laminas of stiffness EI_p/h per unit height. It is assumed that the connecting beams do not deform axially under the action of the lateral loading and both walls will deflect equally with a point of contra flexure at the mid-point of each connecting beam.

The axial force in the wall T, and the bending moments in walls 1 and 2 are given by

$$T = \frac{2\beta}{\alpha 4} \{1 + \frac{\sinh\alpha H - \alpha H}{\cos\alpha H} \sinh\alpha x - \cosh\alpha x + \tfrac{1}{2}\alpha^2 x^2\} \quad (2.1)$$

and

$$M_1 = (\tfrac{1}{2}\omega x^2 - T\ell)\frac{I_1}{I} \quad (2.2)$$

$$M_2 = (\tfrac{1}{2}\omega x^2 - T\ell)\frac{I_2}{I} \quad (2.3)$$

The complete stress distribution at any section, (which consists in reality of a superposition of a uniform axial stress and a linear bending stress) may be defined from an alternative superposition of two pure bending stress distributions; (a) a bending stress based on the assumption that wall systems act as a single composite cantilever, the neutral axis being situated at the centroid of the two wall elements and (b) two linear stress distributions obtained on the assumption that the wall acts completely independently with the neutral axis at the centroid of each wall.

For the two walls shown in Fig. 2.2, the stress distribution at any section, under the action of the bending moments M_1 and M_2 and the axial force T, is shown

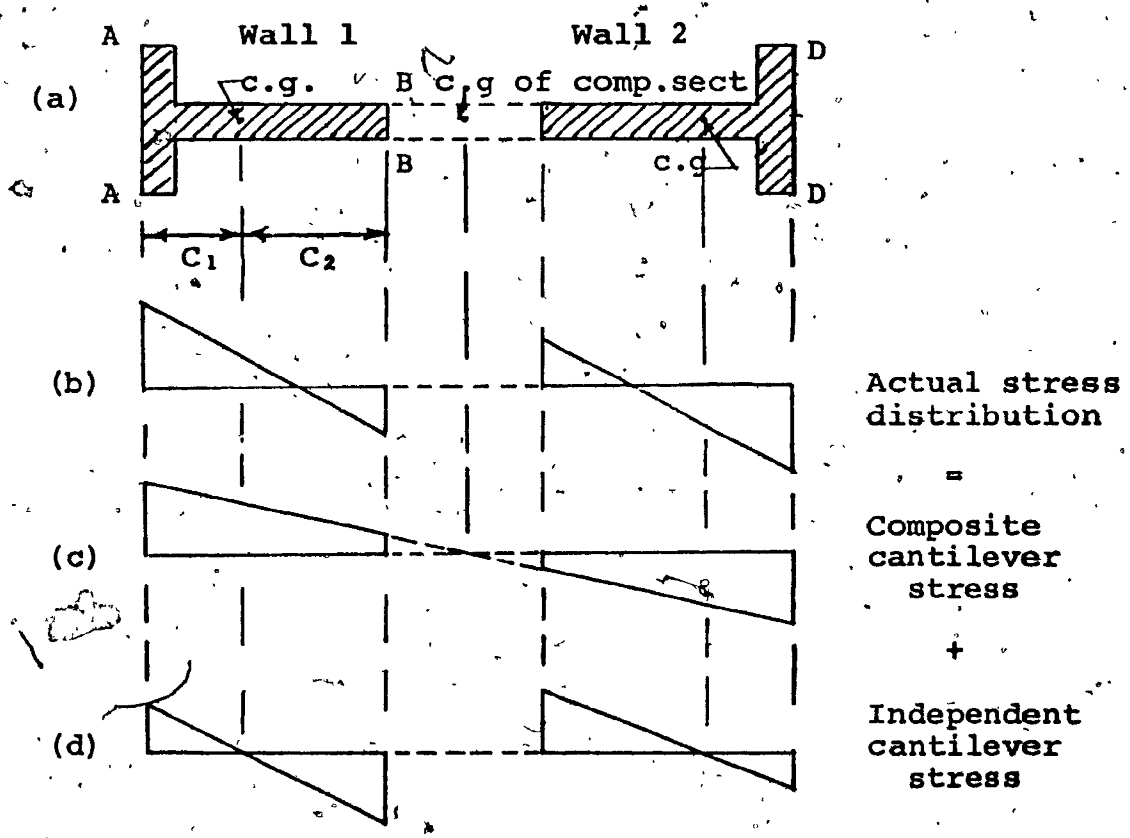


FIG. 2.2 Superposition of Stress Distribution due to Composite and Individual Cantilever Action to Give True Stress Distribution in Walls

$$\sigma A = (\tfrac{1}{2}\omega x^2 - T\ell)\,\frac{C_1}{I} + \frac{T}{A_1} \qquad (2.4)$$

$$\sigma B = -(\tfrac{1}{2}\omega x^2 - T\ell)\,\frac{C_2}{I} + \frac{T}{A_1} \qquad (2.5)$$

Suppose that K_1 is the percentage of the load carried by independent cantilever action, and that K_2 is the percentage carried by composite cantilever action, then the two component stress can be considered as follows:

(a) <u>Composite cantilever action [6] (Fig. 2.2(c))</u>

The total bending moment at any section is equal to

$$M = (\tfrac{1}{2}\omega x^2)\,\frac{K_2}{100}$$

and the extreme fiber stress in wall I will be

$$\sigma A = \frac{\omega x^2}{2I'}\left(\frac{A_2\ell}{A} + C_1\right)\frac{K_2}{100}$$

$$\sigma B = \frac{\omega x^2}{2I'}\left(\frac{A_2\ell}{A} - C_2\right)\frac{K_2}{100}$$

where I' is the moment of inertia of the composite cantilever given by

$$I' = I_1 + I_2 + \frac{A_1 A_2}{A}\,\ell^2$$

Similar expressions again hold for wall 2.

(b) Individual cantilever action [6] (Fig. 2.2(d))

The walls will deflect equally and the loads carried will be proportional to the moment of inertia if the axial deformation of the connecting beams is ignored. The bending moments in walls 1 and 2 are given by

$$M_1 = \frac{1}{2}\,\omega x^2 \frac{I_1}{I}\,\frac{K_1}{100}$$

$$M_2 = \frac{1}{2}\,\omega x^2 \frac{I_2}{I}\,\frac{K_1}{100}$$

and the extreme fiber stresses in wall 1 become

$$\sigma A = \frac{1}{2}\,\omega x^2 \frac{C_1}{I}\,\frac{K_1}{100}$$

$$\sigma B = -\frac{1}{2}\,\omega x^2 \frac{C_2}{I}\,\frac{K_1}{100}$$

Similar expressions again hold for wall 2.

2.1.1.1 Relation Between Shear Stress Factors

The proportions of composite and individual cantilever action required to produce the true stress distribution at any position are functions of the geometrical parameter α and the height ratio x/H and K_2 is expressed as

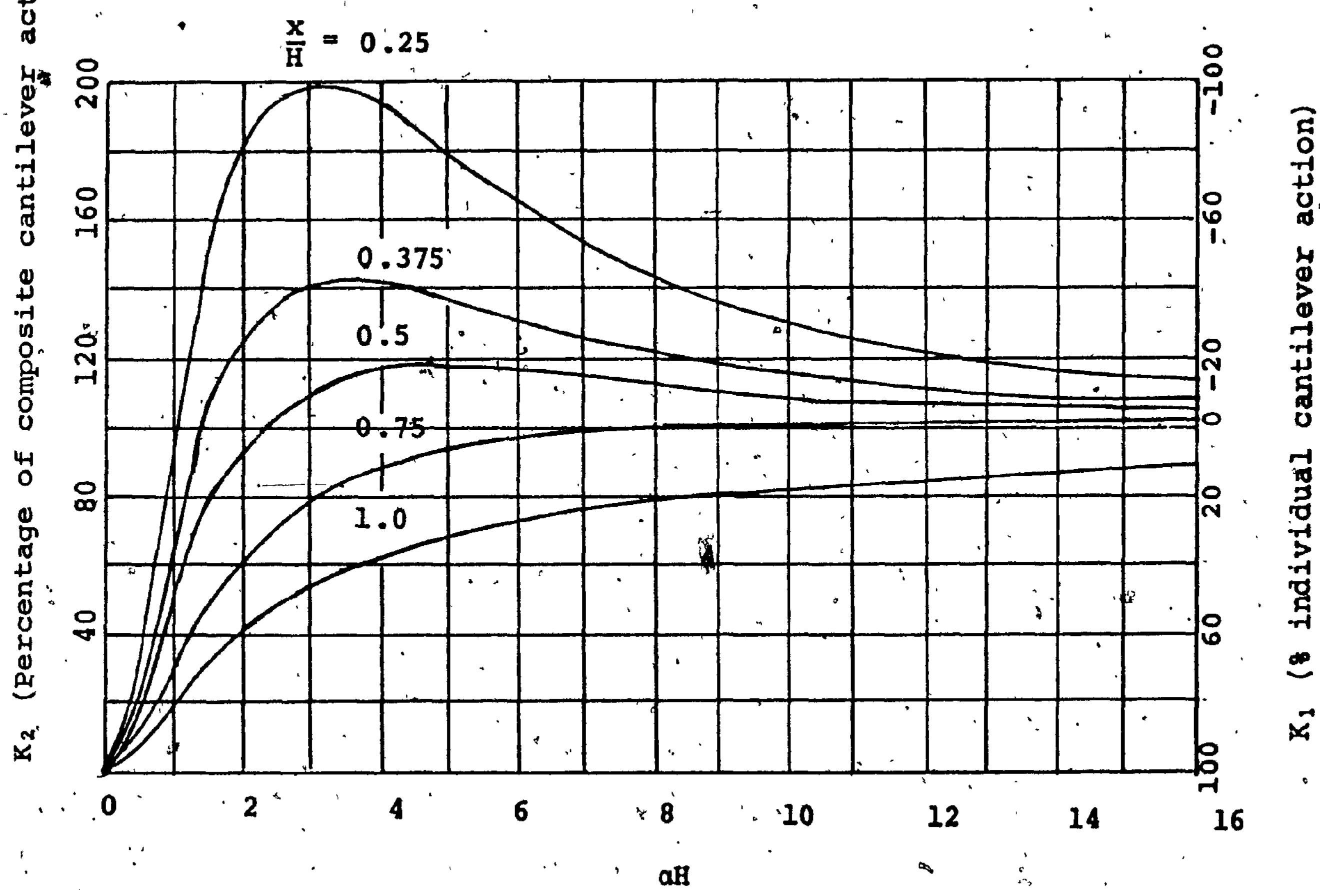


FIG. 2.3 Variation of Wall Bending Stress Factors K_1 and K_2 (Adopted from Reference [6])

$$K_2 = \frac{200}{(\alpha H)^2 (x/H)} \{1 + \frac{\sinh\alpha H - \alpha H}{\cosh\alpha H} \sinh(\alpha H\, x/H) - \cosh(\alpha H\, x/H) + \frac{1}{2}(\alpha H)^2 (x/H)^2\}$$

and $$K_1 = 100 - K_2$$

Fig. 2.3 shows the variation of K_2 for a number of height ratios, for a range of values of the parameter α covering all practical cases.

2.1.1.2 Stresses in connecting beams [6]

The shear force per unit height in the equivalent continuous system of connecting laminas may be expressed as

$$q = \omega \frac{H}{\ell} \frac{1}{\mu} K_3$$

where

$$\mu = 1 + \frac{A}{A_1 A_2} \frac{I}{\ell^2}$$

and

$$K_3 = \frac{\sinh\alpha H - \alpha H}{\alpha H \cosh\alpha H} \cosh(\alpha H\, x/H) - \frac{\sinh\alpha H\, x/H}{\alpha H} + \frac{x}{H}$$

The shear forces depend on the geometrical parameters μ and α, and the height ratio $\frac{x}{H}$. Curves showing the variation of the factor K_3 with the parameters αH and the height ratio $\frac{x}{H}$ are shown in Fig. 2.4.

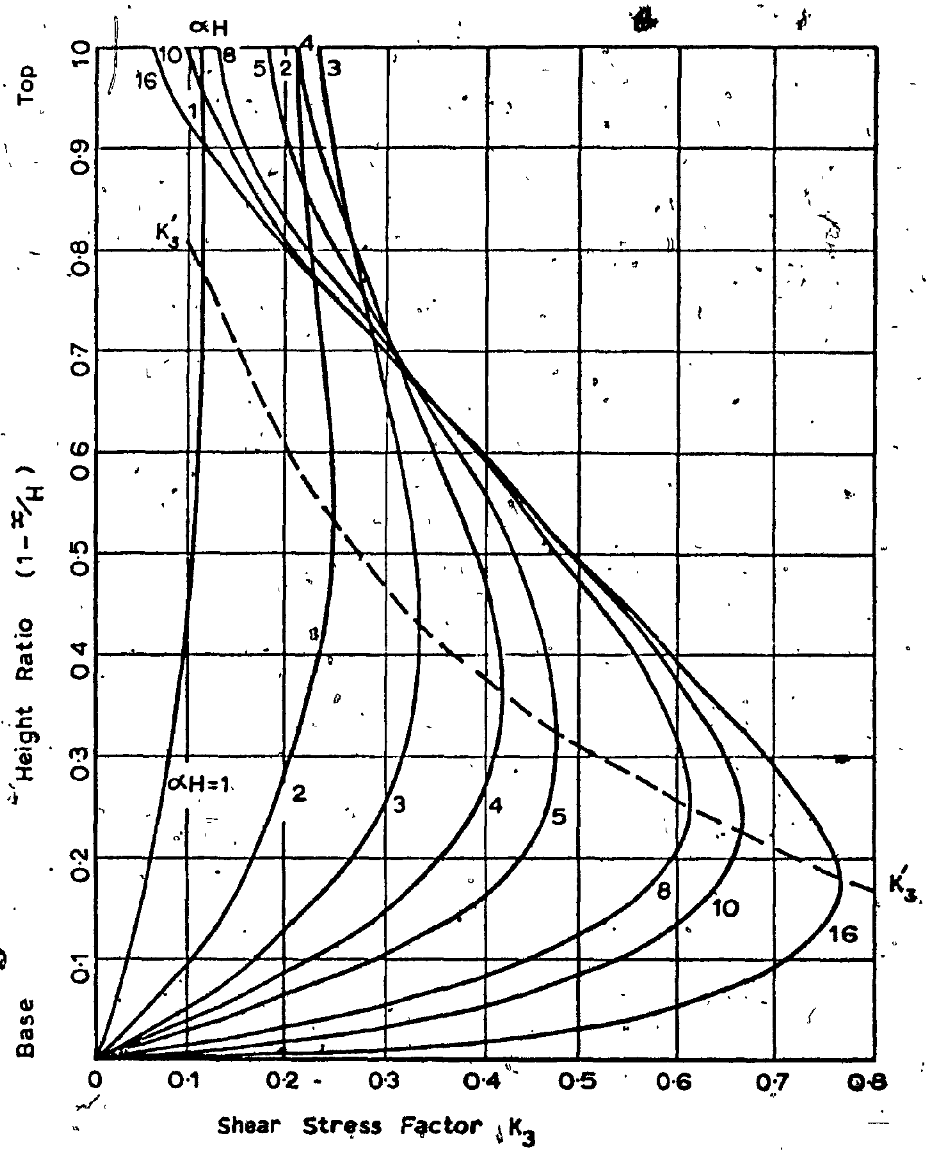


FIG. 2.4 Variation of Connecting Beam Stress Factor, K_3

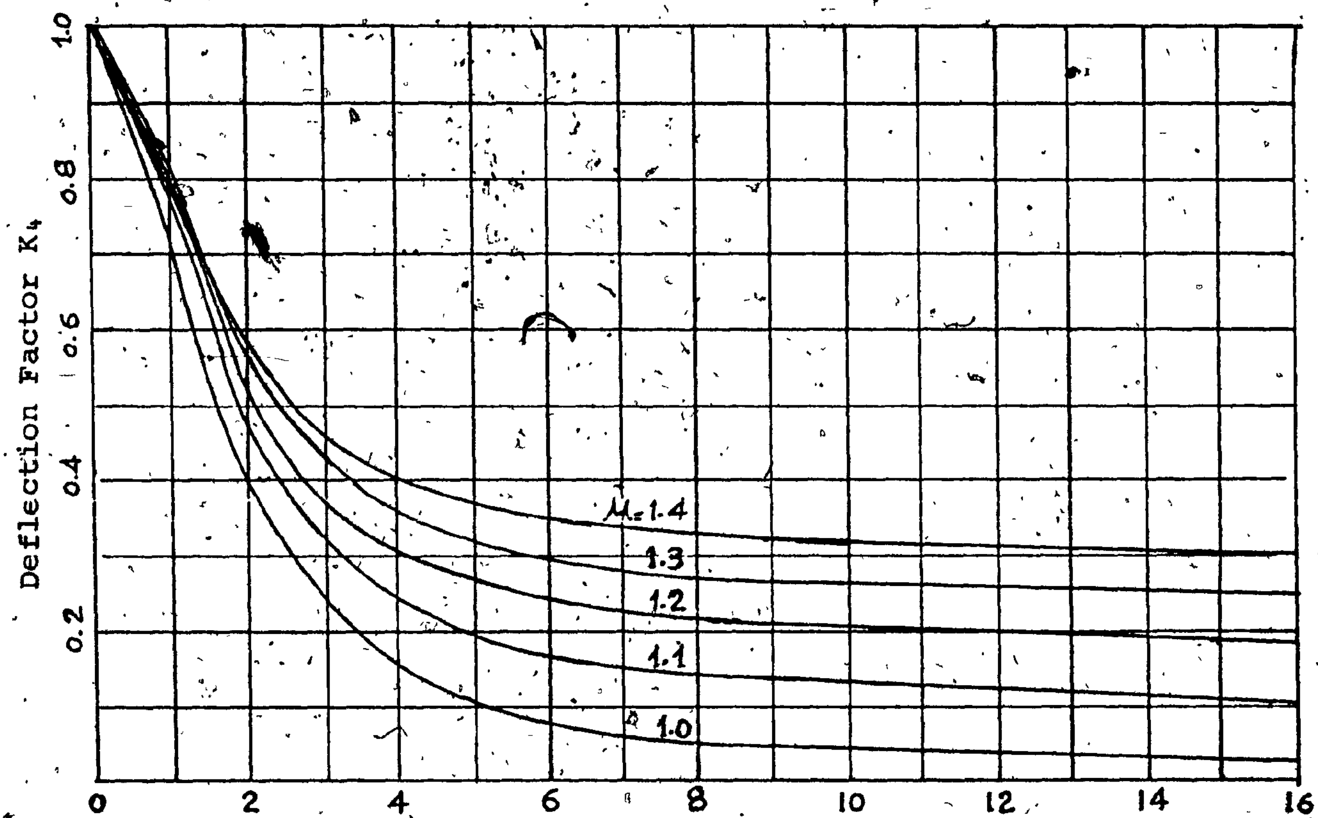


FIG.2.5 Variation of Deflection Factor K_4
(Adopted From Reference [6])

2.1.1.3 Deflections [6]

The moment curvature relationship for each wall is given by

$$EI \frac{d^2y}{dx^2} = \frac{1}{2} \omega x^2 - T\ell$$

where y is the deflection at any height. On integration

$$Y_m = \frac{1}{8} \frac{\omega H^4}{EI} K_4$$

where

$$K_4 = \frac{\mu-1}{\mu} - \frac{8}{\mu} \left[\frac{\alpha H \sinh\alpha H - \cos\alpha H + 1}{(\alpha H)^4 \cos\alpha H} - \frac{1}{2(\alpha H)^2}\right]$$

The variation of the factor K_4 with the parameters αH and μ is shown in Fig. 2.5.

2.1.2 Loading

Solutions are available for uniformly distributed loads, triangularly distributed loads and for concentrated point loads at the top of the building. The most readily available and convenient are those developed by Coull and Choudhury [5,6].

2.2 EFFECT OF OPENINGS IN SHEAR WALLS

Openings can have an important effect on the behaviour of shear walls. The openings are normally in vertical rows and in the common case of a single row of openings, Fig.3.2(a) a useful parameter for assessing the effect of the openings is αH where

$$\alpha = \sqrt{\frac{12\ I_b}{hb^3}\left(\frac{\ell^2}{I_{c_1}+I_{c_2}} + \frac{A_{c_1}+A_{c_2}}{A_{c_1}A_{c_2}}\right.}$$

H = total height of wall

I_b = moment of inertia of a connecting beam

h = story height

b = clear span of beams

ℓ = distance between centroidal axes of the wall sections

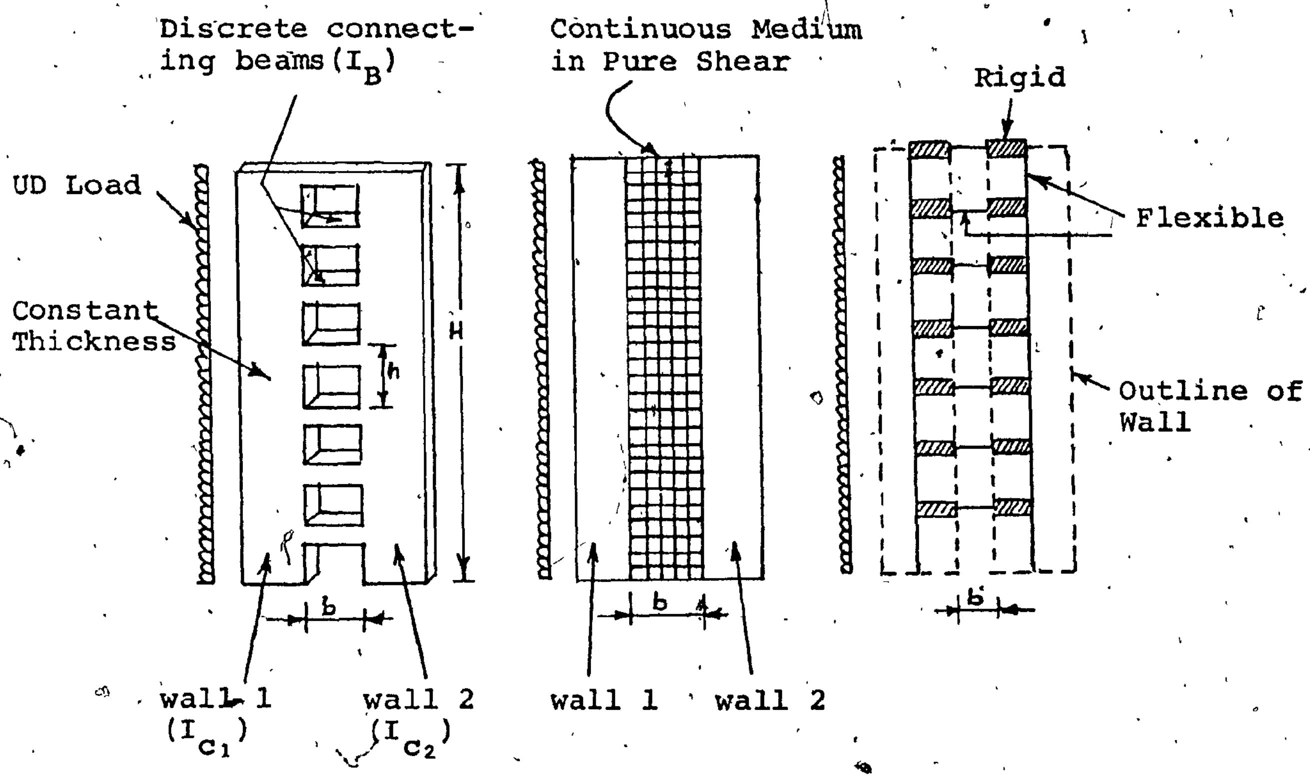
I_{c_1}, I_{c_2} = moment of inertia of wall sections

A_{c_1}, A_{c_2} = area of wall sections

α = a variable for shear wall with openings

For αH that is greater than 8, the wall tends to behave like a single cantilever. For low αH, e.g., less than 4, the behaviour is more like two connected walls and frame action is more prominent.

With a single row of openings the effect of openings on the stiffness can be assessed by comparing Kw and Kw_0, which are given as



(a) Shear wall with single row of openings (b) Idealization for shear connection method (c) Frame idealization

FIG. 2.6 Wall with a Single Row of Openings

$$Kw = \frac{3E\,I_w}{H^3}$$

and

$$Kw_0 = \frac{3E(I_{C_1}+I_{C_2})}{H^3 K_4}$$

where

Kw and Kw_0 = stiffnesses of the wall without and with openings, respectively

E = Young's modulus of elasticity

I_w = moment of inertia of wall without openings

$$K_4 = 1 - \frac{3}{\mu}\left(\frac{1}{3} + \frac{\sinh(\alpha H)}{(\alpha H)^3\cosh(\alpha H)} - \frac{1}{(\alpha H)^2}\right)$$

$$\mu = 1 + \frac{(A_{C_1}+A_{C_2})(I_{C_1}+I_{C_2})}{A_{C_1}\;A_{C_2}\;b^2}$$

The stiffness and the distribution of stress in a wall will normally be appreciably affected by the presence of openings. Coull and Choudhury's chart references [5 and 6] are useful for problems with a single row or two symmetrical rows of openings.

Shear walls with openings can be idealized as illustrated in Fig. 3.2(c) using a plane frame analysis program.

2.3 SHEAR WALLS ACTING WITH FRAMES

Since the late 1940's the use of shear walls to resist lateral load in high-rise buildings has been extensive. Many frame structures cannot be efficiently designed to satisfy lateral load provisions without the aid of shear walls.

The main function of a shear wall for the type of structure being considered here, is to increase the rigidity for lateral load resistance. Shear walls also resist vertical load and the difference between a column and a shear wall may not always be obvious. The distinguishing features are:

(1) The shear wall has a much higher moment of inertia than a column; and

(2) the shear wall has a width which is less than the span of the adjacent beam, which is not negligible.

2.3.1 Simplified Method of Estimating Lateral Load Distribution [4].

Two simplified methods of determining the interaction of frames and shear walls, based on neglecting of torsion effect are available.

(1) MacLeod introduced the component stiffness method [4].

(2) Charts are given by Khan and Sbarounis [2] and PCA's Advance Engineering Bulletin No.14 [3].

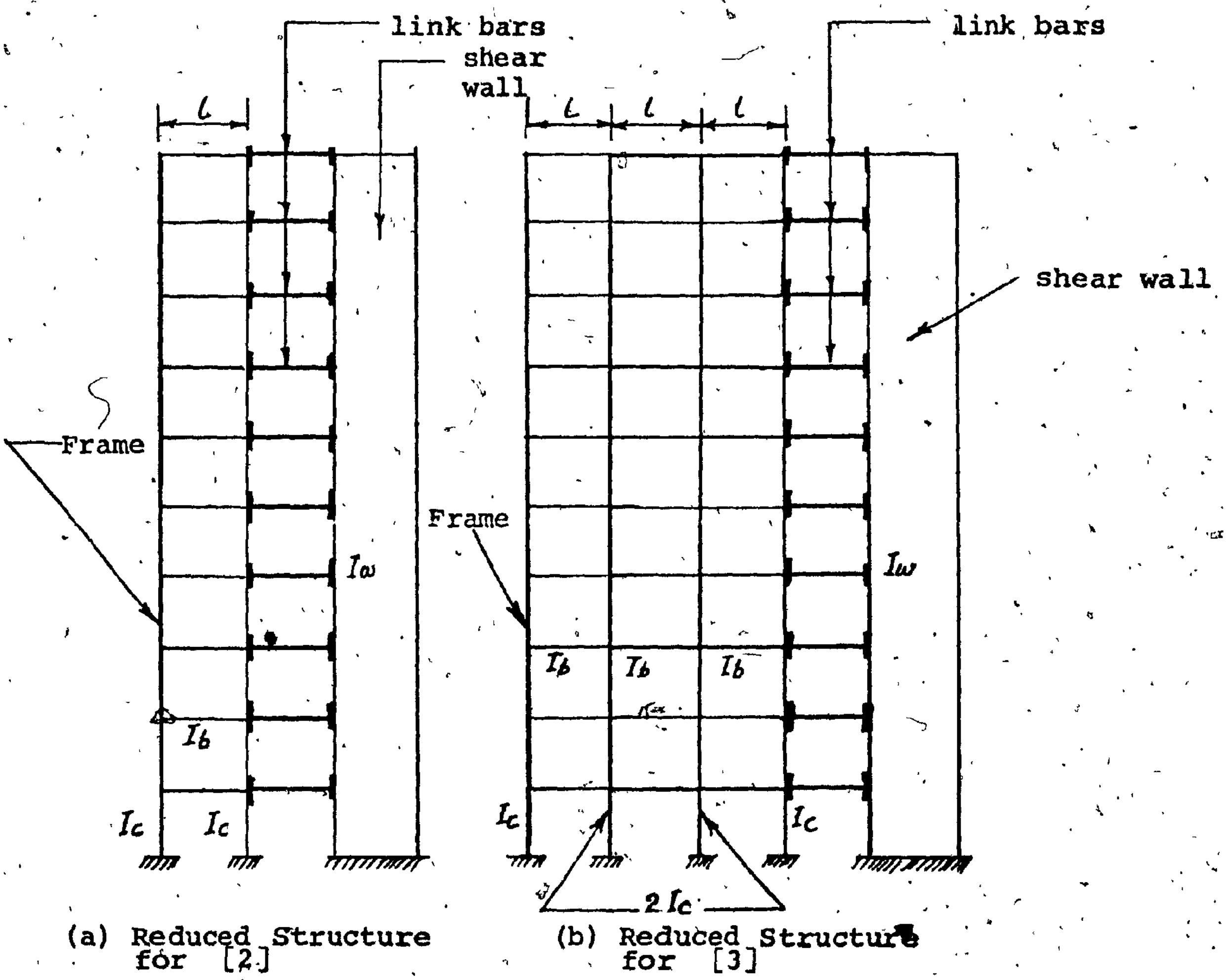


FIG. 2.7 Reduced Structure

2.3.1.1 Design charts [2 and 3]

In order to use these charts the structure must be reduced to a single frame and a single wall by the addition of the properties of the separate vertical units. The stiffnesses (I_W) of all the shear walls are summed up to give an equivalent single wall. Frames could be a single bay frame, or a three-bay frame, as shown in Fig. 2.7.

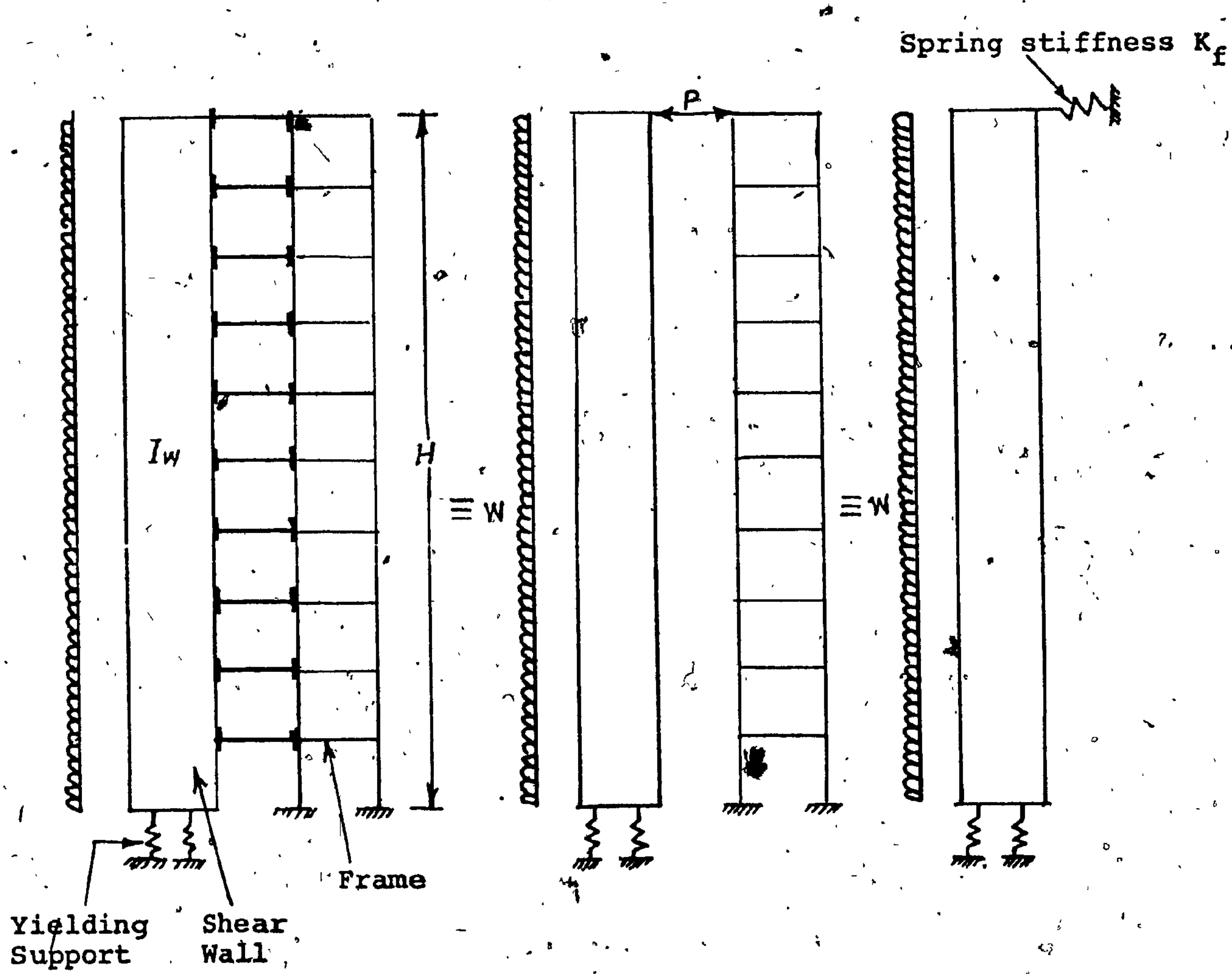
Both the frames shown in Fig. 2.7 are "proportioned". A proportioned frame may be defined as one that has points of contraflexure at all mid-beam sections under lateral load. The frame being proportioned and the column axial deformation being negligible are basic assumptions made by reducing a multibay frame to a single or three-bay frame.

The procedure for reducing the number of bays is: at each storey level, sum all column moments of inertia (I_c) and beam rotational stiffnesses (I_b/ℓ). The equivalent stiffnesses to be used in the substitute frame. The procedure is described by Khan and Sbarounis [2].

Having thus reduced the problem, the shear on the frame, moment on the shear wall and deflection can be found by using the charts.

2.3.1.2 The component stiffness method [4]

The component stiffness method has more flexibility than the charts method, but it lacks accuracy if the wall is more flexible than the frame ($K_w/K_f < 1$).



(a) Structure (b) Interaction at top only (c) Frame modelled by spring

FIG. 2.8 Idealization for Equation C

The main assumption is that the frame takes constant shear, i.e., the interaction force between the frame and wall can be represented by a concentrated force at the top.

Consider the single bay frame and shear wall loaded in plane by the uniform distributed load shown in Fig.2.8(a). If the frame shear is assumed to be constant, the system can be treated as a wall supported at the top by a spring, Fig. 2.8(c). The spring stiffness K_f is defined as the lateral point load applied at the top of the frame to cause unit deflection in its line of action. K_f can be calculated using equation A and B of Table 2.1 and the top deflection equation in Table 2.2 notation.

Kw is defined as the lateral point load required to cause unit deflection at the top of the wall (similarly to K_f).

Table 2.2 (Equation C) gives relationships between P/W, νw and Kw/K_f for different loading cases. Similar expressions for other load cases can easily be established. P is the interaction load at the top of the frame, i.e., the constant shear; W is the total applied lateral load and νw is a dimensionless parameter which relates the rotational stiffness of the wall to that of the foundation, i.e.,

$$\nu w = \frac{K_B H}{4 E_W I_W}$$

where K_B is the rotational stiffness of the shear wall support. If the rotation at the base of the shear wall is to be neglected, the term with ν_W in Equation C should be omitted. However, the effect of shear wall base rotation can significantly affect the distribution of load between shear wall and frames and Equation C can be used as a simple method of assessing this factor.

Table 2.1 shows Equations B and A for top deflection of rigid regular frames.

Equation B - for bending deformation

$$\Delta_B = \frac{Wh^2H}{12\Sigma(E_cI_c)} [F_s(1-\beta_D)^3 + F_g(1-\beta_C)^3 \; 2\lambda]$$

where

Δ_B = deflection at top of frame due to bending of member

W = total lateral load

h = storey height

H = total height

E = Young's modulus

ΣI_c = sum of moments of inertia of columns at first-storey level

F_s, F_g = functions of s and g, dependent on type of loading

S = ratio $\dfrac{I_c \text{ at top of frame}}{I_c \text{ at bottom of frame}}$

g = ratio $\dfrac{I_b \text{ at top of frame}}{I_b \text{ at bottom of frame}}$

Linear variation of I_c and I_b with height. If E varies use EI instead of I

β_D = D/h where D is beam depth

β_c = c/ℓ where c is column width and ℓ is distance between column centre lines

$\lambda = \dfrac{\Sigma(E_c I_c/h)}{2\Sigma(E_b I_b/\ell)}$ i.e., summation over width of structure at first-storey level

I_b = moment of inertia of beam at bottom of structure

<u>Equation A</u> - For axial load deformation

$$\Delta_A = \frac{WH^3F_n}{E_c A_c B^2}$$

where

ΔA = deflection at top of frame due to axial deformation of exterior columns

F_n = function of n, dependent on the type of loading

n = ratio $\dfrac{\text{Area of exterior column at top of frame}}{\text{Area of exterior column at bottom of frame}}$ (linear variation of A_c with height)

A_c = area of exterior columns at first-storey level

B = total width of frame

Δ = ΔB = ΔA = Total deflection

TABLE 2.1 EQUATIONS B AND A FOR TOP DEFLECTION OF RIGID REGULAR FRAMES [4]

Load Condition	$F_s\,(m=s)$ or $F_g\,(m=g)$
Point load at top	$\frac{\log_e m}{m-1}$
Uniformly distributed	$\frac{1}{1-m} + \frac{m\log_e m}{(1-m)^2}$
Triangular (Earthquake)	$\frac{\log_e m}{m-1} - \frac{3/2+2m-m^2/2-\log_e m}{(m-1)^3}$

(B)

Load Condition	F_n
Point load at top	$\frac{1.4n+3n^2-2n^2}{(1-n)^3}$
Uniformly distributed	$\frac{2.9n+18n^2-11n^3+6n^3\log_e n}{6(1-n)^4}$
Triangular (Earthquake)	$\frac{2}{3}\left(\frac{2\log_e n}{n-1} + \frac{5(1-n+\log_e n)}{(n-1)^2} + \frac{\frac{9}{2}-6n+\frac{3n^2}{2}+3\log_e n}{(n-1)^3} + \frac{-\frac{11}{6}+3n-\frac{3n^2}{2}+\frac{n^3}{3}-\log_e n}{(n-1)^4} + \frac{-\frac{25}{12}+4n-3n^2+\frac{4n^3}{3}-\frac{n^4}{4}-\log_e n}{(n-1)^5}\right)$

(A)

TABLE 2.2 EQUATION C, RELATIONSHIP BETWEEN P/W. REFERENCE [4]

Load Condition	Equation C
Point load at top	$\frac{P}{W} = \frac{1 + \frac{3}{4\nu\omega}}{1 + \frac{3}{4\nu\omega} + \frac{K\omega}{K_f}}$
Uniformly distributed	$\frac{P}{W} = \frac{\frac{3}{8}(1 + \frac{1}{\nu\omega})}{1 + \frac{3}{4\nu\omega} + \frac{K\omega}{K_f}}$
Triangular (Earthquake)	$\frac{P}{W} = \frac{\frac{11}{20} + \frac{1}{2\nu\omega}}{1 + \frac{3}{4\nu\omega} + \frac{K\omega}{K_f}}$

Notations

P = interaction force at top

W = total applied lateral load

$\nu_\omega = \dfrac{K_b H}{4E_w I_w}$

K_b = rotational stiffness of shear wall support

H = total height of wall

E = Young's modulus

I_ω = moment of inertia of wall

$K_\omega = \dfrac{3E\, I_w}{H^3}$ (with constant I_w)

K_f = point load at top of frame to cause unit deflection in its line of action, i.e.,

$$\frac{P}{\Delta} \text{ or } \frac{P}{\Delta_B\ \Delta_A}$$

since top deflection $\Delta = \dfrac{P}{K_f}$

2.3.1.3 The Component Stiffness Method for Structures With Torsion [4]

By making the asusmption that the frames take constant shear, the structure shown in Fig. 2.9(a) may be idealized as in Fig.2.9(d). Two degrees of freedom which correspond to the deformation Δ and θ at the top of the structure Fig. 2.9(f) can be assigned and the structure

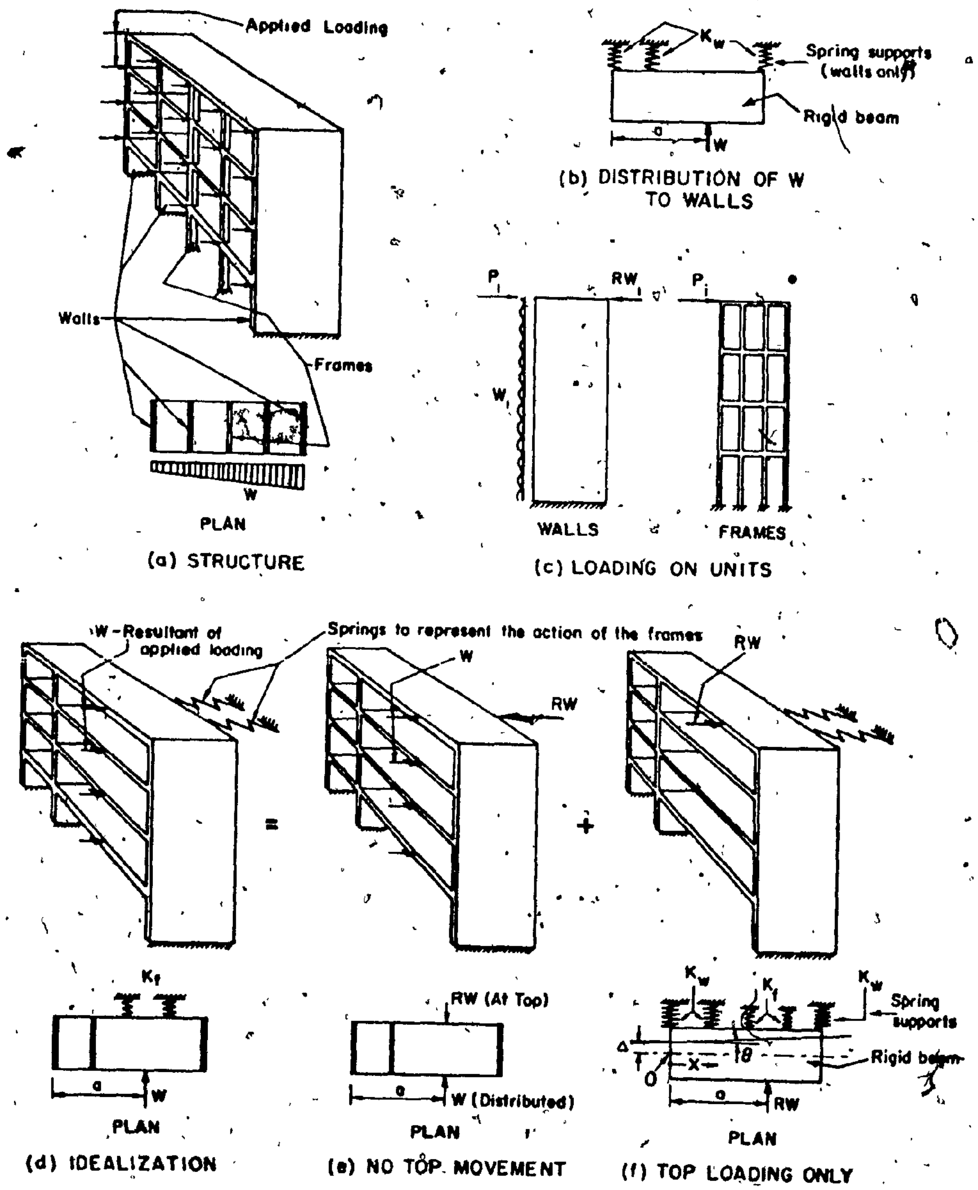


FIG. 2.9 Component Stiffness With Torsion (Adopted from Reference [4])

solved by the stiffness method. The analysis is carried out by adding the results from Systems 1 and 2 (Fig. 2.9(e) and Fig. 2.9(f)). Lateral movement at the top of the System 1 (Fig.2.9(e) is prevented by a force that equals RW where R is the support reaction coefficient for a propped cantilever and W is the total lateral load. RW acts in the line of and in the opposite direction to the resultant of W at the top of the structure. For an analysis of System 2, (Fig.2.9(f)) the roof slab can be considered as a rigid beam on spring supports. The spring stiffnesses are K_w and K_f as defined in Section 2.3.1.2.

The equations for determining the unknowns Δ and θ are set up as follows. Transverse equilibrium gives

$$\Sigma K_i(\Delta + X_i\theta) = RW$$

and therefore

$$\Sigma K_i\Delta + \Sigma K_iX_i\theta = RW \qquad (2.6)$$

and taking moment about 0

$$\Sigma K_i(\Delta + X_i\theta) = RW_a$$

and therefore

$$\Sigma K_iX_i\Delta + \Sigma K_iX_i^2\theta = RWa \qquad (2.7)$$

The above K_i represents both the wall stiffnesses, $K\omega$, and the frame stiffnesses, K_f, and a is the distance of applied load from origin.

After solving equations (2.6) and (2.7) for Δ and θ, the loads on the springs are

$$P_i = K_i(\Delta + X_i\theta) \tag{2.8}$$

If there are only two walls, simple statics gives the distribution of the total lateral load, W. With more than two walls, Equations (2.6) and (2.7) must be re-established, neglecting the frame springs and solving with W as the only loading, Fig.2.9(b). The resulting deformations are fictitious but can be used to calculate W_i, i.e., the proportion of W to each wall, according to Equation (2.8) by substituting W_i for P_i.

In other words, the process of analysis for more than two walls is:

(A) <u>Analyze System 1</u>

1. Find the proper cantilever reaction coefficient, R, for the given loading.

2. Calculate the portion of W tributary to each wall by applying the system of Fig.2.9(b).

In both these calculations, the frames should be ignored.

(B) Analyze System 2

Analyze System 2 as a rigid beam on spring supports (including the frames) to find the top loads, P_i, on each unit as shown in Fig. (2.9) (f).

(C) Add the Results for the Two Systems

Care should be taken with the signs of the forces. On a given unit, $R\omega_i$ is always opposite in direction to ω_i and positive P_i from Equation (2.3) will be in the same direction as W. The final loadings on the units are illustrated in Fig. (2.9) (c). A situation could occur where some values of P_i would be in the opposite direction to W, e.g., when the effect of torsion is pronounced.

The above approach can be extended to cover any problems where deformation in the longitudinal direction is also possible. Under these circumstances, three simultaneous equations have to be solved.

CHAPTER 3

STRENGTH, DUCTILITY AND ENERGY ABSORPTION OF CONCRETE SHEAR WALLS

3.1 GENERAL DESCRIPTION OF TESTED CONCRETE SHEAR PANELS [13]

In this Section, a brief description of the test results [13] of reinforced concrete shear panels is made, without openings, describing loading, the mode of failure and the influence of panel variations, as shown in Fig.3.1.

The wall is loaded through a distributing member (either a beam or floor diaphragm) at the top of the wall. Both tension and compression columns are provided, and the wall is supported on an essentially rigid foundation. Face walls may act as the bounding columns.

The important variables are:

(a) Loading
(b) Materials
(c) Panel design (thickness, proportions, and reinforcing)
(d) Tension column design
(e) Compression column design, and
(f) Method of construction

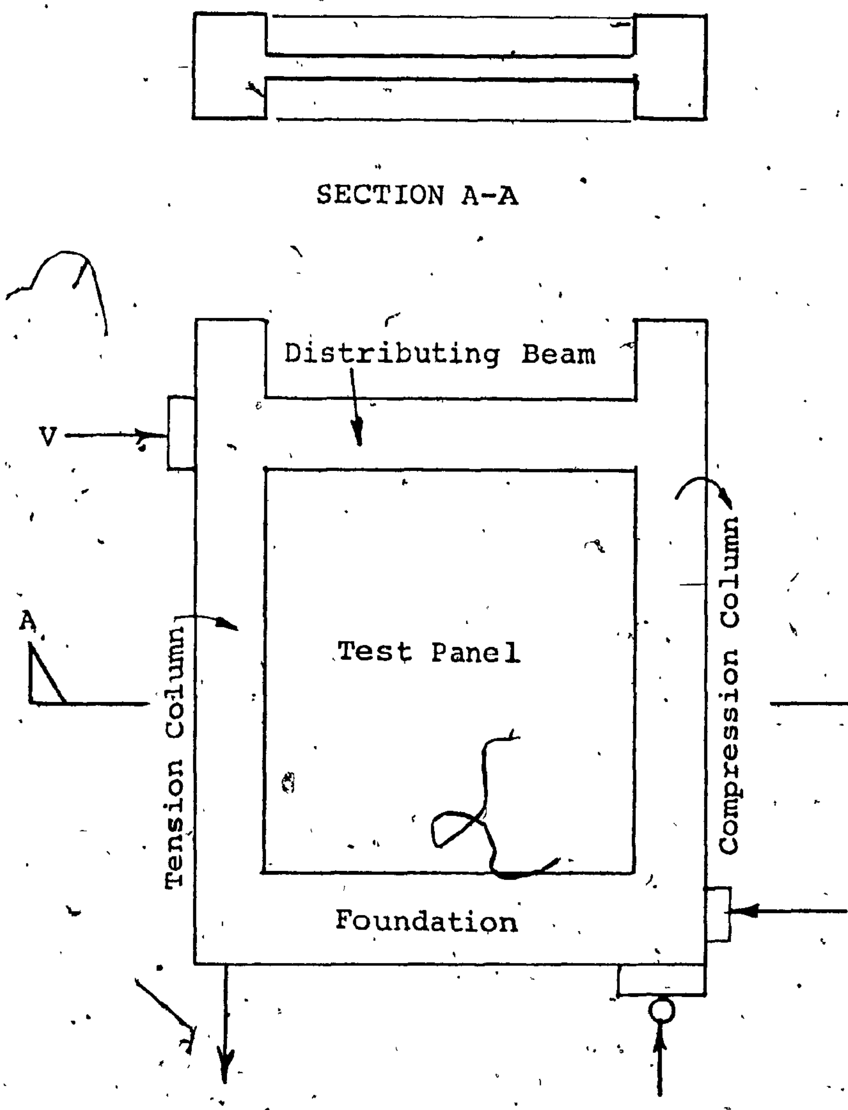


FIG. 3.1 Typical Wall Panel and Method of Loading

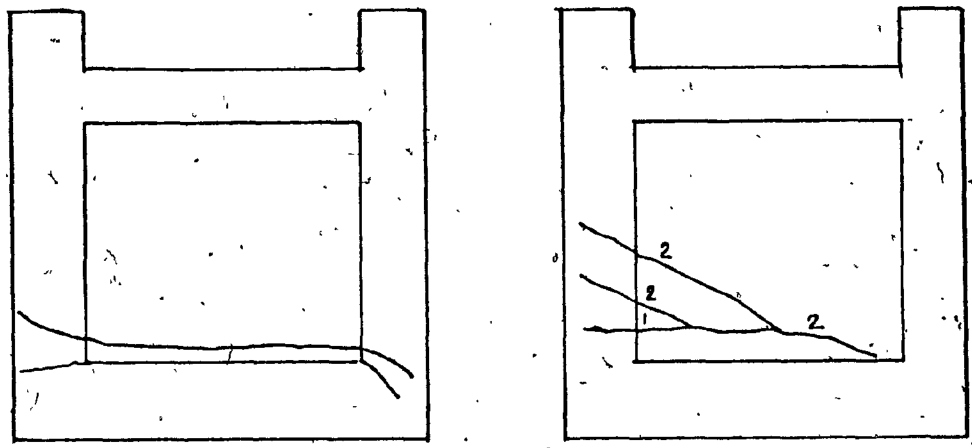
3.1.1 Modes of Wall Failure

The wall had several modes of failure. Steel had a negligible influence until concrete cracked. Reinforcing steel then controlled the opening of the crack and its propagation.

The first possible mode of failure is characterised as tension column failure. If there is insufficient steel at the junction of the tension column and foundation to take up the load when the concrete cracks, the crack opens and rapidly runs along the junction of the panel and the foundation to the compression column, as shown in Figs.3.2 and 3.3.

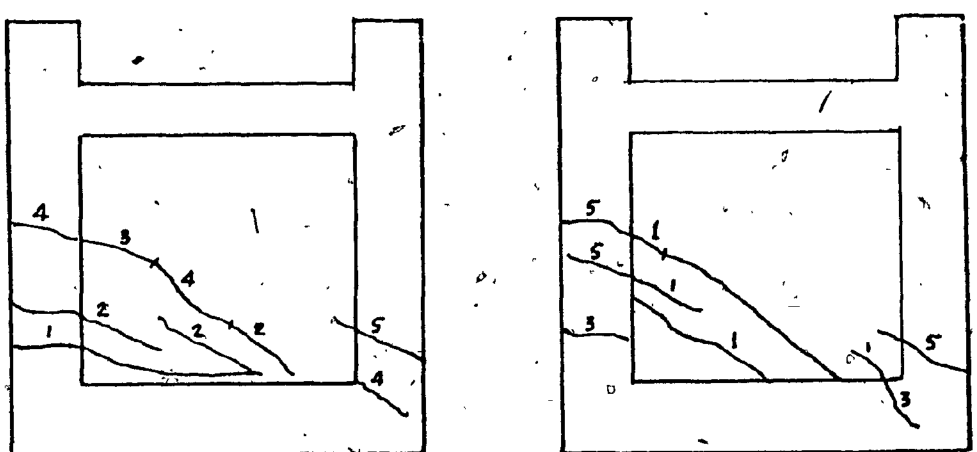
The second possible mode of failure involves the panel cracking diagonally in the tensile stress region. If the panel is unreinforced the crack follows the pattern shown in Fig. 3.2, and the first crack spreads rapidly to the foundation and tension column. An additional increase in shear load then produces cracks along the foundation to the compression column and along the tension column to the beam. The shear load will increase further with more and more panel cracking until the compression column cracks at the foundation, and finally shears off.

If the panel is moderately reinforced ($p \geq 0.0025$), the failure mode is modified as a function of the amount of panel steel. The steel produces general diagonal cracking



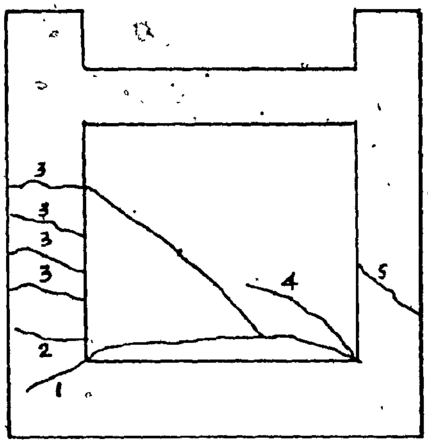
Column Reinf. 0.5%   Column Reinf. 0.705%

Tension Failure in Columns
(Load Applied at Upper Left Corner)

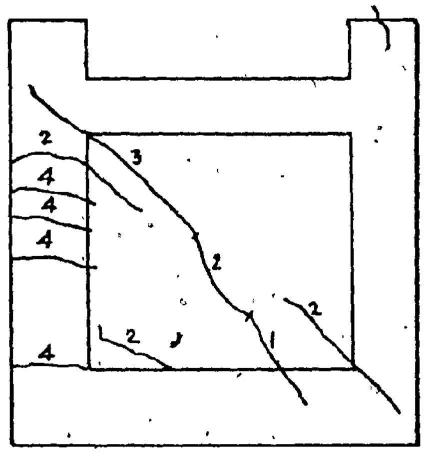


Column Reinf. 1.68%   Column Reinf. 3.3%

FIG. 3.2 Cracking Pattern for Unreinforced Wall Panels With Varied Per. of Steel in the Columns

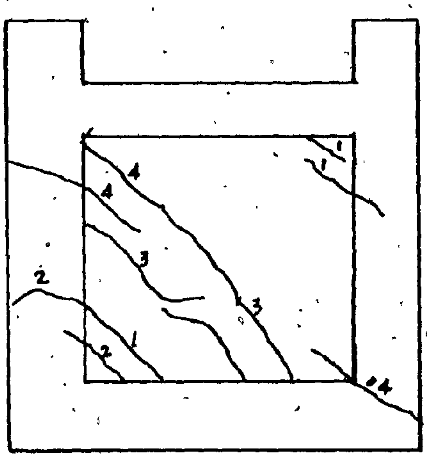


0.5% Panel Reinforcement
1.1% Column Reinforcement

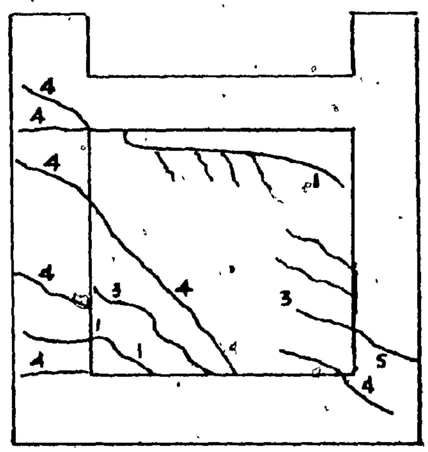


0.5% Panel Reinforcement
2.2% Column Reinforcement

DIAGONAL CRACKING



0.5% Panel Reinforcement
3.3% Column Reinforcement



0.25% Panel Reinforcement
3.3% Column Reinforcement

FIG. 3.3 Cracking Pattern for Reinforced Wall Panels With Varied Per. of Steel in the Column (Load Applied at Upper Left Corner)

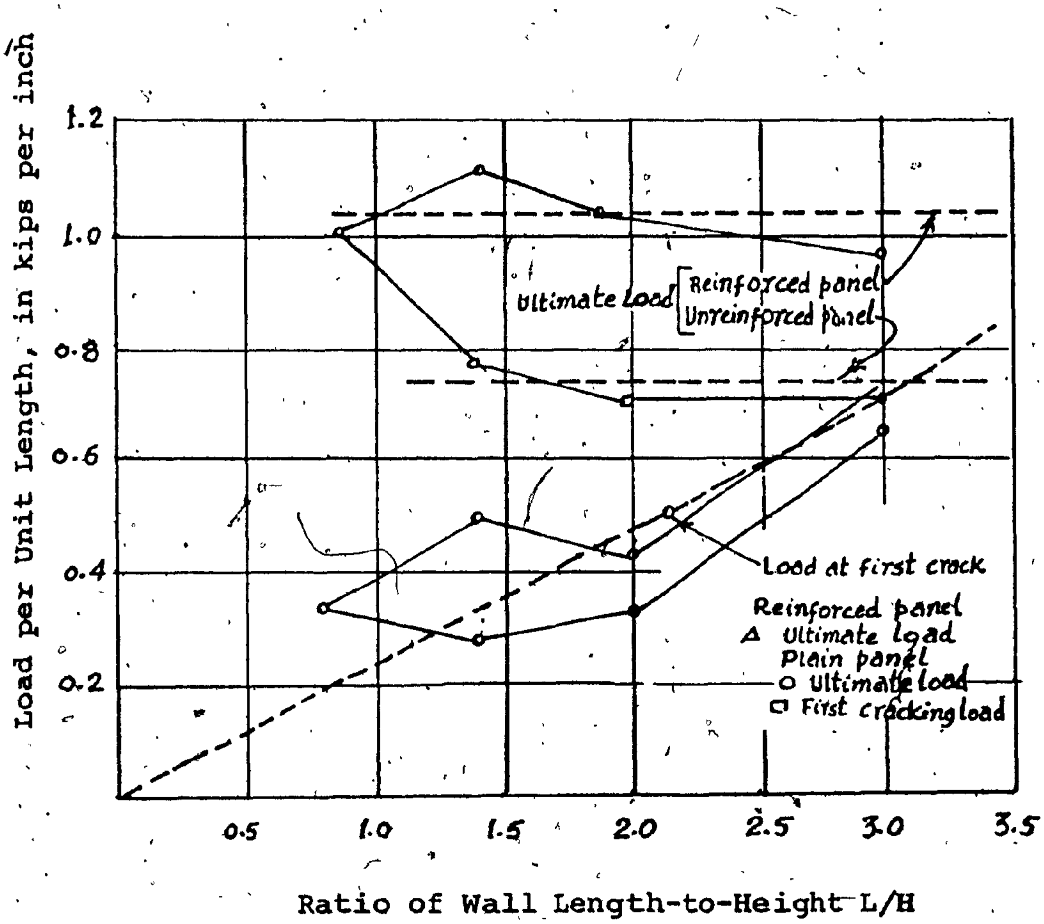


FIG. 3.4 Shear Load Per Unit Length of Wall at First Crack and Ultimate Load for Specimens With Varying Panel Proportions [13]

as shown in Fig. 3.3, instead of a single major crack as with unreinforced panel.

Panel reinforcement has a pronounced influence on the cracked wall behaviour. At the ultimate load a major part of the vertical panel steel is apparently at its yield point and a few of the horizontal bars are in the post-yield condition approaching actual failure. Ultimate load occurs with the compression column shearing off in the same manner as with unreinforced panels.

3.1.2 Affect of Length-to-Height Ratio on Shear Walls

The panel proportions (Length, L, height H and thickness, t) and the panel reinforcement directly influence the cracking load of the shear walls.

Tests [13] were conducted for panel variations and the results are shown in Fig. 3.4. The length-to-height ratio (L/H) has a direct and pronounced influence. The crack pattern changes as the (L/H) ratio increases. The crack becomes progressively higher in the wall, finally approaching a pure diagonal crack. The study of load deflection curve [13] shows that as L/H increases, the load at the first crack, or the load at a major break in the load deflection curve approaches the ultimate load.

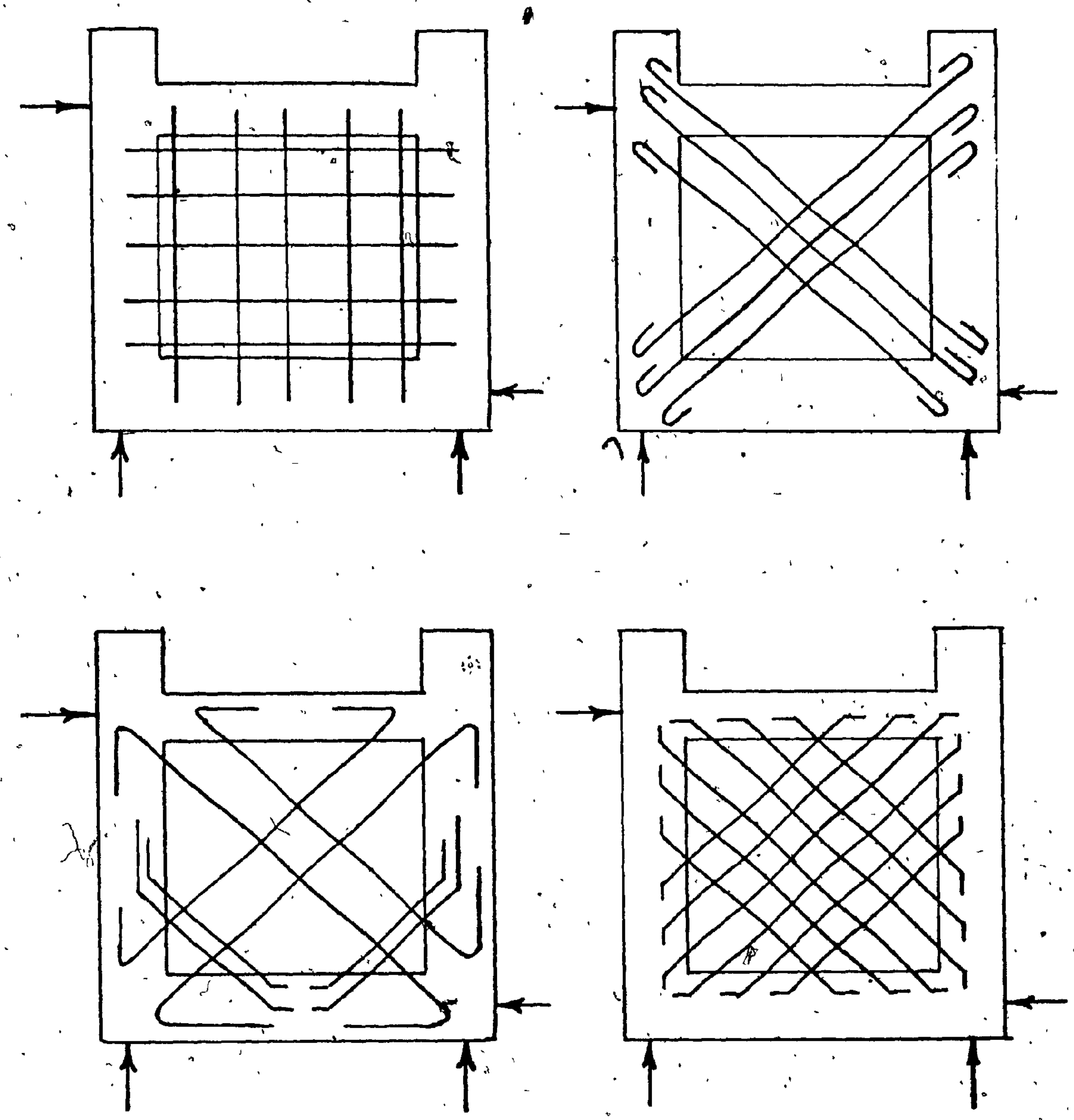


FIG. 3.5 Variation in Panel Reinforcement

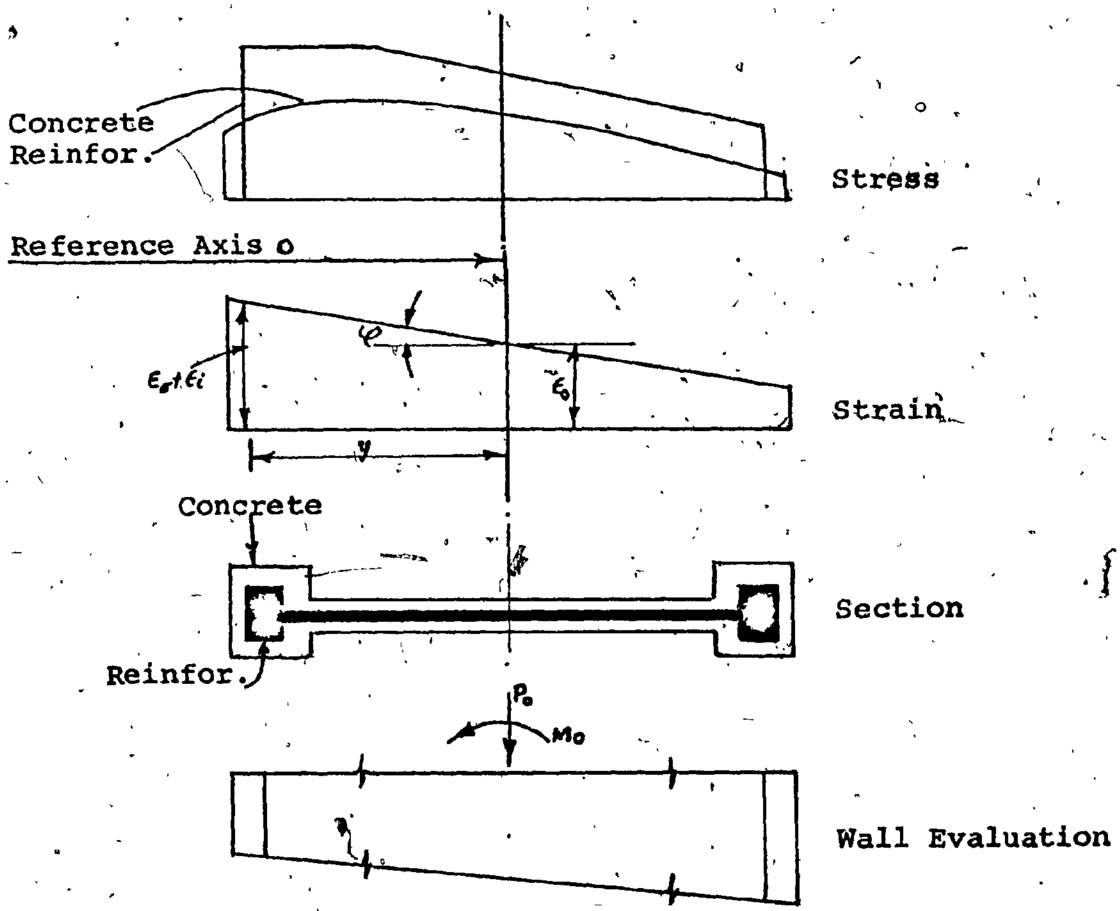
3.1.3 Variation in Panel Reinforcement

Tests [13] were conducted by varying the panel reinforcement both horizontally and vertically, as shown in Fig. 3.5, and it was proven that all walls have essentially the same behaviour before cracking. Panel reinforcing is effective only after cracking begins. As the panel reinforcing increases, the number of cracks before the ultimate load increases and the individual width of each crack decreases.

In order to obtain the most efficient reinforcement system, the diagonal reinforcing was varied from that which was distributed uniformly to bands concentrated at the diagonals, and special corner reinforcing was added in the region of the highest stress. In the analysis [13] it was found that diagonal reinforcing was less effective than rectangular reinforcing. Special corner steel was ineffective in improving wall characteristics. Heavy concentration of steel on the panel diagonals produce large shrinkage cracks along the panel diagonals and decrease the wall rigidity.

3.2 AXIAL FORCE - MOMENT INTERACTION DIAGRAM

A series of analytical studies to investigate the strength, stiffness and ductility of shear wall sections was carried out by Salse and Fintel [10]. The mathematical



Strain and Stress Distribution
Non-linear Beam Theory

FIG.3.6 Non-Linear Beam Theory Assumptions model used for the development of the interaction diagram of a shear wall section is based on non-linear beam theory. The mathematical model for the "computer test series" has the obvious shortcoming of simulating only monotonic loading, since no model of possible effects on concrete due to cyclic loading has yet been developed.

The nonlinear beam theory assumes the strains to vary linearly across the Section, while the stresses in both the concrete and reinforcement vary nonlinearly according to their actual stress-strain relationship, as shown in Fig. 3.6.

Two sections were investigated by Mark Fintel [7]: A rectangular section with uniformly distributed reinforcement and reinforcement bunched at the ends and an I-section with concentration of reinforcement in the flanges. For the steel reinforcement there is a good stress-strain information available, for the stress-strain characteristics of concrete, a complex equation was developed based on tests [10]. The accurate representation of both the ascending and descending of stress-strain curves was the most important consideration.

MAXIMUM CAPACITY ENVELOPE

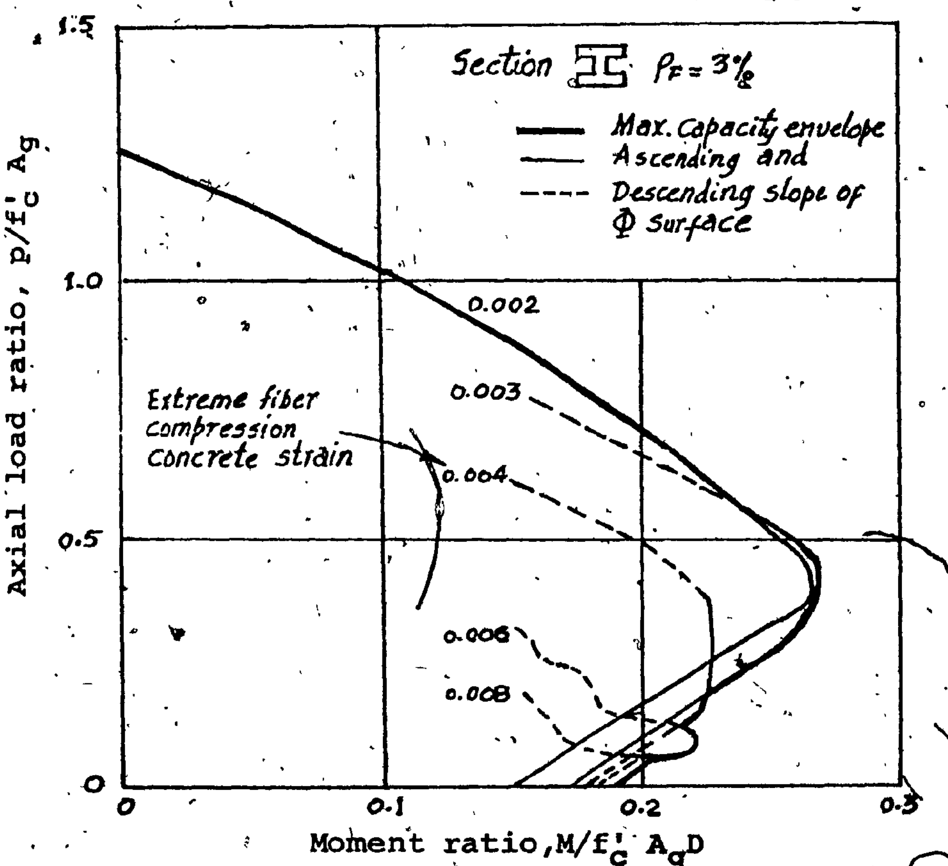


FIG. 3.7 Load-Moment Interaction Envelope

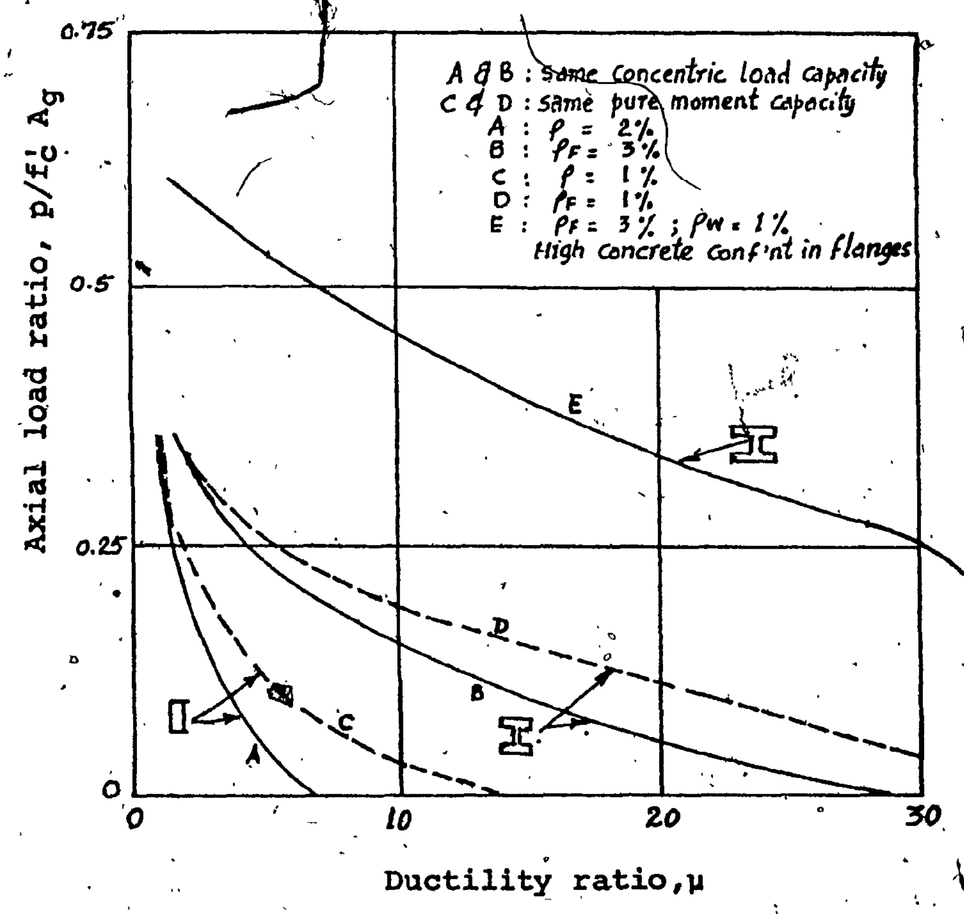


FIG. 3.8 Ductility vs. Axial Load

The load-moment interaction diagram shown in Fig. 3.7, is the envelope of maximum capacity, at whatever strain this may occur. It has been found that the strain of the concrete at which maximum capacity occurs increases as the eccentricity increases, from about 0.002 for the compression controlled branch of the interaction diagram to three-to-four times as much for tension controlled branches. The reason for this increase: as the steel stretches in yielding, the concrete strain increases progressing along the descending branch; the strain hardening of the steel causes an increase of the strength of the section. The deformation capacity of the section from the onset of yielding of reinforcement until its final rupture or until the compression failure of the concrete, is the section ductility.

The section ductility was also investigated as related to the axial load level. The plot in Fig. 3.8 of sectional ductility against axial loads shows that I-section with confined flanges has a substantial ductility, even at balanced load level.

3.3 STRENGTH AND BEHAVIOUR OF REINFORCED CONCRETE SHEAR WALLS

A reinforced concrete shear wall in a multistorey reinforced concrete building is essentially a deep slender cantilever beam. It resists the bending moments and the

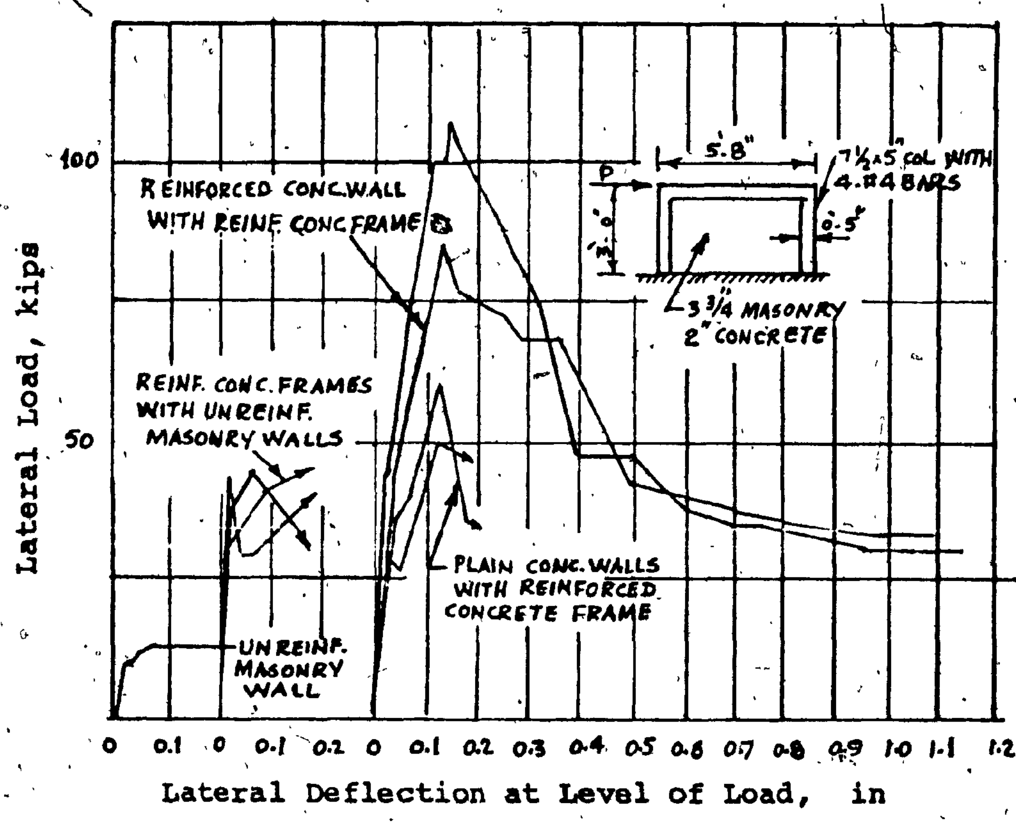


FIG. 3.9 Load-Deflection Curves for One Storey Shear Wall (Adapted from [13])

shearing and axial forces to which it is subjected through essentially the same type of action for reinforced concrete frame members. The strength and behaviour of a reinforced concrete shear wall can be estimated on the same basis of reinforced concrete frame members with some modifications to take account of the shape of the shear wall as compared with an ordinary beam.

Two characteristics of the reinforced concrete shear wall demand special emphasis in view of the possible interpretation of the behaviour of a shear wall in the traditional frame building as an isolated one-storey high panel acting "in parallel" with the enclosing frame one is that the reinforced concrete shear wall extending monolithically through several stories may be subjected to high bending moments, as well as high shearing forces. In fact, flexure rather than shear may govern the strength of the wall. The other characteristic is that the reinforced concrete wall acts together with the surrounding frame as a single structural unit and not as an isolated panel.

The interaction between the frame and the wall is illustrated in Fig. 3.9, by the load deflection curves measure in tests of one-storey shear walls [11]. All the curves in Fig. 3.9 refer to one-storey walls having the same overall dimensions and loaded as indicated at the upper left-hand corner of the frame as shown in the figure. The enclosing reinforced concrete frame was composed of members measuring

7.5 by 5 in. in cross-section and reinforced with four No. 4 deformed bars. The wall thickness was 3.75 in. for the masonry walls and 2 in. for the concrete walls. The concrete strength was about 3,000 psi.

The first set of two curves in Fig. 3.9 is for masonry walls without enclosing frames. These unreinforced walls failed in a brittle manner and carried relatively little load. The second set of three curves refer to masonry walls enclosed in reinforced concrete frames. These walls resisted the lateral force P of about 40 kips. The frame would resist a later load of about 13 kips acting independently. The sum of the larger measured strength of the masonry wall (6 kips) and the estimated strength of the frame totals 19 kips, a quantity far short of the 40 kips carried by the combined wall and the frame. Evidently, the reinforced concrete frame with the masonry wall cannot be considered as an individual frame and an individual wall acting "in parallel", but must be considered as a single structural unit. It is easy to see that although an unreinforced masonry wall possesses little tensile strength, it can act effectively in compression. Ideally, the action of the masonry wall may be considered as that of a diagonal strut extending from the corner where the load acts to the opposite corner. In this capacity, it possesses considerable strength. As the tensile strength of the wall is exceeded and cracks develop, parallel struts form so that the frame never acts completely as a rigid rectangular frame but behaves as a braced frame with diagonal members.

It follows that if the addition of a masonry wall inside a frame results in a mode of behaviour which is close to that of a beam, rather than that of a frame and wall working in parallel, a reinforced concrete frame with a monolithic plain or reinforced concrete wall tends to act as a single unit. Examples of the behaviour of such members are represented by the third group of curves in Fig. 3.9. Of this set, the lower two refer to plain concrete panels and the upper two to reinforced concrete panels ($p' = 0.005$ in both the horizontal and vertical direction.) Again both the strength and the behaviour of these walls bear no resemblance to the strength and behaviour of the enclosing frame. These walls with enclosing frames are essentially reinforced concrete beams, two with web reinforcement and two without, subjected to very high shearing forces in relation to the bending moment.

The load deflection curves in Fig. 3.9 may not represent the behaviour of shear walls in multistorey buildings, because the test specimens were loaded so that the shear-to-moment ratio was very high. An interesting example of the behaviour of a three-storey frame with and without a masonry filler wall was obtained in the course of the tests [14] to the destruction of the Old Dental Hospital in Johannesburg, South Africa. The end frames of

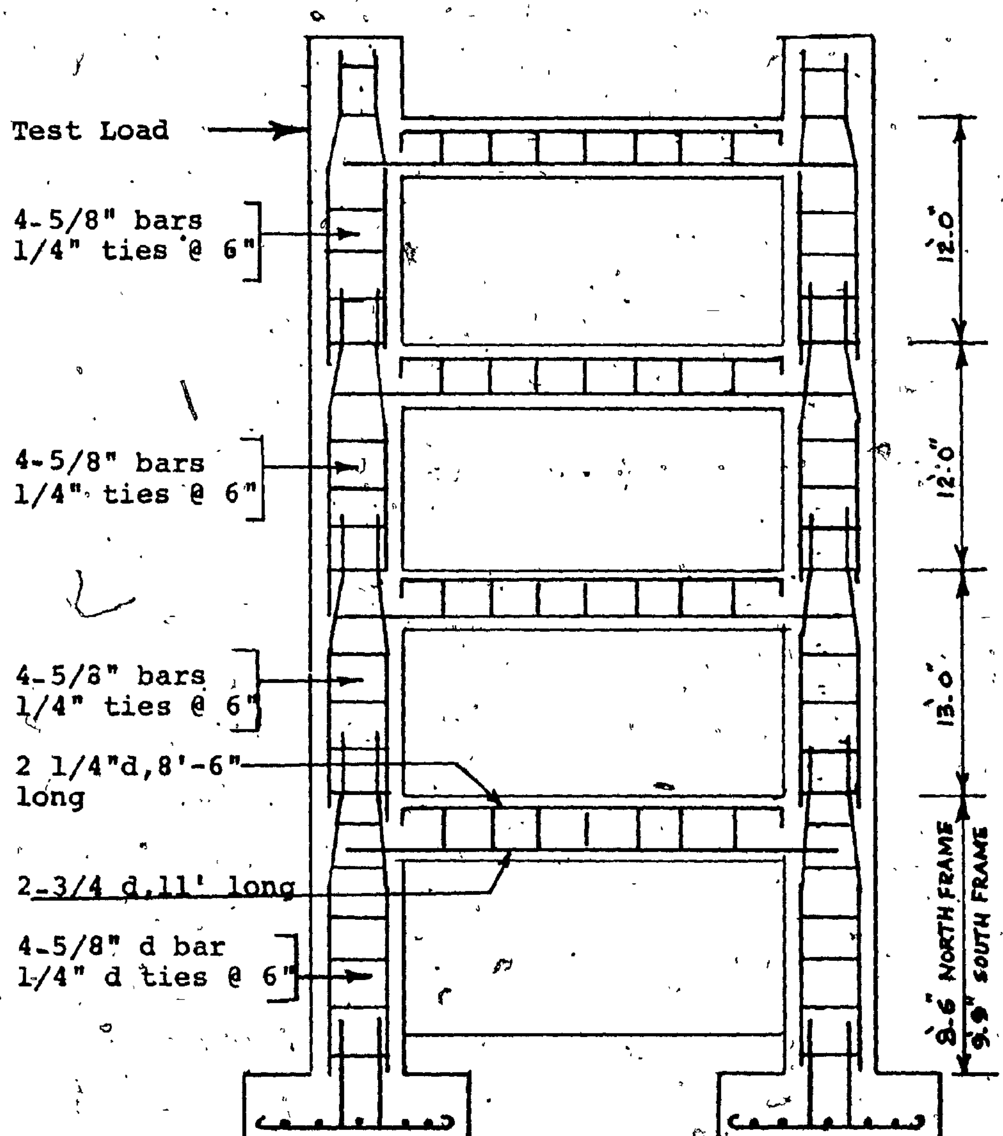


FIG. 3.10 End Frames of Test Building Loaded Laterally at Ceiling Beam of Third-Storey

the north and south wings of the building were separated from the structure and were loaded laterally along the ceiling beam of the third storey, as indicated in Fig.3.10.

The dimensions of the north and south frames and the arrangement of the reinforcement are shown in Fig. 3.10. It should be mentioned that the frames were not designed to resist lateral loads. Consequently, the top reinforcement in the beams was not extended into the columns. The average concrete strength in the frames, based on 4 in. cores, was 4,550 psi (cube strength). The yield stress of the plain reinforcing bars ranged from 41,300 to 44,800 psi. The south frame was tested to failure after the masonry walls had been removed. The north frame was tested with the 4.5 in. masonry walls in place. The unreinforced masonry was of poor quality. Tests of samples indicated compressive strength ranging from 390 to 500 psi.

The load deflection curves for the north and south frames are shown in Fig. 3.11. The south frame which did not have the masonry filler walls carried only about 20 percent of the load carried by the north frame. Despite the poor masonry, the north frame functioned as a single structural unit and carried much greater load than the sum of the capacities of the wall and the frame. Its mode of failure was associated with beam action, a condition for which it had not been designed. It failed in a tension splice in the first-storey column (which was designed as

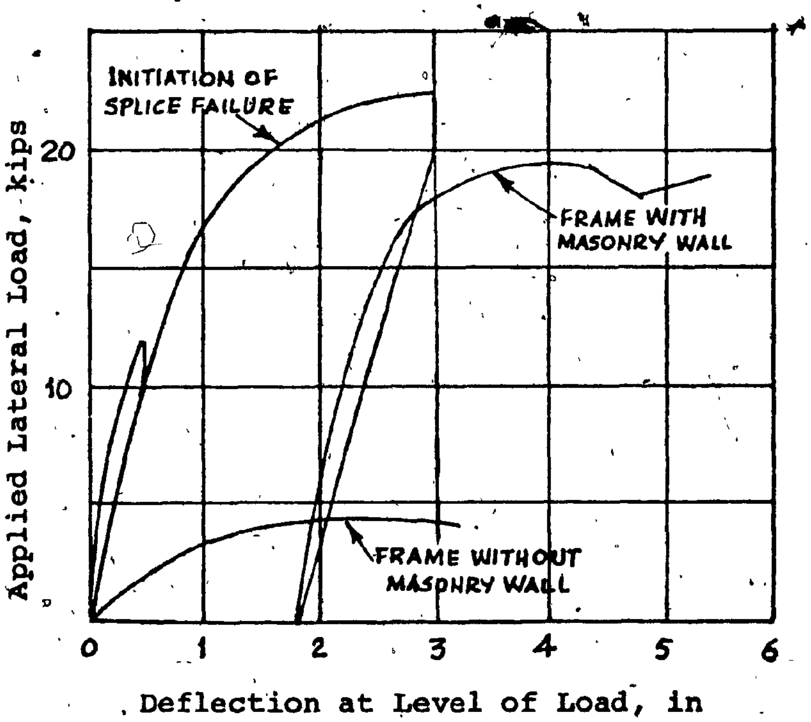


FIG. 3.11 Load-Deflection Curve for Three-Storey Frame With and Without Masonry Wall Shown in Fig. 4.2 (Adapted from [14])

a compression splice) in a region of maximum bending moment.

Another example of this integral action is of a model of multistorey shear wall tested at the Mutt Laboratory of the University of Tokyo. The reinforced concrete model was loaded by a concentrated load and supported at the ends. The crack pattern is typical for reinforced concrete beams loaded over short spans. The whole system should be considered as a single "waffle", beam, rather than a frame with shear walls.

If tall slender shear walls are treated as webs of waffle beams or I-beams and are reinforced adequately in shear, they will act as cantilever beams fixed at the ground. Their behaviour in the ductile range will be controlled by tensile yielding of the vertical reinforcement concentrated in the flanges (or columns) as well as of the vertical distributed reinforcement in the wall, itself. It is evident that the strength and deformation characteristics of reinforced concrete shear walls are governed by the same criteria as those fore-frame members.

3.4 ENERGY-ABSORBING CAPACITY

A good measure of the energy-absorbing capacity of a reinforced concrete section subjected to earthquake effects in the area under the $M - \phi$ curve. The $M - \phi$ curve to the cross-sectional properties is shown in Fig. 3.12. The $M - \phi$

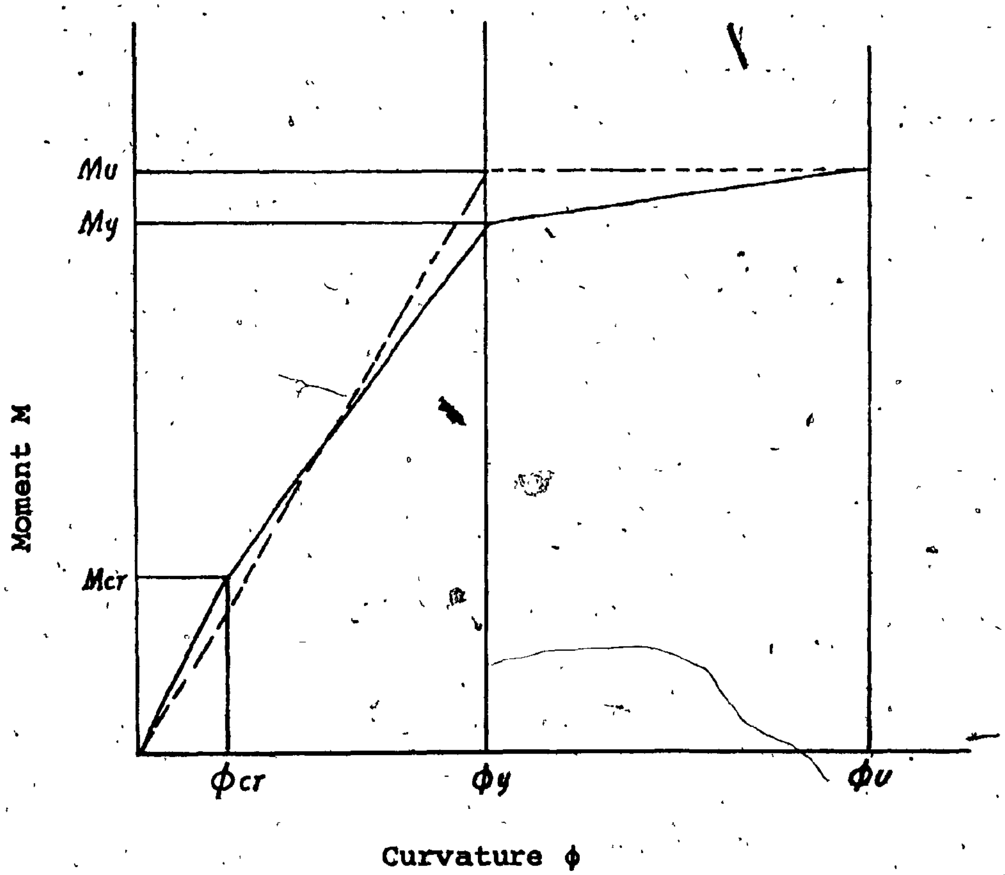


FIG. 3.12 Moment Curvature Relationship for a Moderately Reinforced Section

curve for a moderately reinforced cross-section exhibits three distinctly different stages as shown in Fig. 3.12. The first stage corresponds to an uncracked section and $M - \phi$ curve is essentially linear. The appearance of the first hairline cracks introduces the second stage, M_{cr}. The third stage begins at yielding of the tension steel and ends when the useful limit of strain is reached in compressed concrete.

Sensitivity of the Energy-Absorbing Capacity

If the effect of the tensile strength of the concrete is ignored the energy absorbing capacity per unit of length for a reinforced concrete section can be written as follows:

$$U = \frac{1}{2} M_y \phi_y + \frac{1}{2}(M_u + M_y)(\phi_u - \phi_y)$$

where U, the energy absorbed, is the area under the $M - \phi$ curve. For a moderately reinforced section, it can be assumed that

$$M_y = M_u$$

and

$$M_y = A_s f_y jd \qquad (3.3)$$

$$M_u = A_s F_{su} d(1-0.4K_u) \qquad (3.4)$$

$$\frac{M_y}{M_u} = \frac{j}{1 - 0.4Ku} = \frac{1 - K/3}{1 - 0.4pf_y/f_{cu}}$$

which is close enough to unity not to warrant complicating the equations by considering M_y and M_u separately. Therefore

$$U = M_u(\phi u - \frac{\phi y}{2})$$

The two curvatures, ϕu and ϕy are defined as follows

$$\phi u = \frac{\varepsilon_{cu}}{q_{ud}}$$

$$\phi y = \frac{\varepsilon_y}{(1-K)d}$$

The expression for U can be further simplified by assuming that $1 - K = 1 - q_u$.

Thus the expression for the energy-absorbing capacity of a reinforced section in tension

$$U = bdf_{cu}(\varepsilon_{cu}-\varepsilon_y/2)(1-0.4qu)$$

CHAPTER 4

DESIGN OF CONCRETE SHEAR WALLS

4.1 DESIGN OF SHEAR WALLS

All portions of a shear wall should be designed to resist the combined effects of axial load, bending and shear determined from a rational analysis of structural system.

Flexural reinforcement should be provided in accordance with the requirement of the ACI Building Code [20]. Wall with proportions such that a linear strain distribution does not apply should be designed as short cantilever.

Design of shear walls for shear should be in accordance with Section 11.16, "Special Provisions for Walls", proposed revision of ACI 318.63 [20].

Minimum amounts of reinforcement in both the vertical and horizontal directions should be those required by flexural calculations or those specified in the provision of shear strength.

In addition to provision of the necessary amount of reinforcement, it is essential that reinforcement details in every shear wall receive careful attention to insure optimum performance.

Contrary to the common opinion and the misleading name, the strength of shear wall is governed by flexure and not by shear, except for very low and long walls. Shear walls of multistorey buildings behave like slender cantilevers only extremely heavy reinforced columns at the ends of thin walls (dumb-bells) can force a shear failure under heavy lateral loads. Laboratory tests for shear walls in progress verify the above points.

4.2 WORKING STRESSES AND LOADING COMBINATIONS

Design codes for earthquake-resistant structures used allowable design stresses one third greater than normal design stresses. In no case, however, are the normal design requirements for dead and live load reduced because of lateral forces in combination.

Wind forces design is also allowed a one-third increase in allowable unit stresses as for earthquake forces, and in the same combination. Wind and earthquake are never assumed to occur simultaneously. Also, wind or earthquake is normally assumed to occur parallel to each major horizontal axis of a building but only on one of these axes at a time. The vertical component of the earthquake motion has generally been ignored.

Higher unit stresses (more than 1-1/3 times normal) have been proposed for California earthquake codes, along the greater seismic coefficients. Japanese codes do use greater allowable unit stresses and there is a logical reason for them. It can be shown that the reserve capacity of structural members to resist severe earthquake loading will depend upon the additional stress required to cause yielding and failure.

Another question is whether live load should be included with dead load in computing earthquake forces. It is customary to compute the earthquake forces using dead load only, or with the dead load and a reduced average live load except for warehouses, which have higher average live loading. In most buildings, the design live load is not realized over the whole area, in fact, the actual average live load can be very low or even negligible. In spite of the low average live loading, there may be heavy live loads at certain points on a floor, such as a concentrated assembly of file cabinets over a girder span. It is therefore, logical to use the lateral force from the small average live load in general, and when designing members to combine the stresses caused by such lateral forces with those from the live load specified for the floor. This latter live load should be reduced, provided by building codes. The one-third increase in allowable unit stresses applies only to the combination with the lateral forces.

4.3 WORKING STRESS AND ULTIMATE STRENGTH DESIGN

The lateral loads specified for design by the SEAOC Code are based on a working stress procedure using a one-third increase in the working stresses except for anchorage by bond when lateral loads combined with vertical loads control the design. The use of the ACI design requirements, is possible with this procedure and will lead to satisfactory results. The alternative method of design in the ACI Code, the so-called ultimate-strength design procedure, may also be used. The earthquake lateral loading effects determined in accordance with the SEAOC Code or other applicable code should be modified by the load factor shown in Section A604, ACI Code, for wind or earthquake and the most critical load combination should be used for proportioning members. Under no circumstances however, should a combination of load factors be used such that the overall factor of safety or load factor with earthquake effects considered, based on the actual capacity of the members as determined by ultimate-strength procedure, would be less than 1.5. The load factor is normally obtained by the working stress method as indicated by the ratio of the yield stress of intermediate-grade reinforcement to 1.33 times the allowable design stress.

The ultimate-strength procedure for design of building frames is useful and valuable since it results in a more nearly uniform factor of safety than can be achieved by the working stress concept. However, in the use of either procedure, it is important to follow the recommendations, regarding the amount, location and the arrangement of reinforcement in order to achieve the necessary ductility, as well as strength.

4.4 STRENGTH OF CONCRETE AND REINFORCEMENT

The designer has a freedom of choice in selecting the combination of materials, he believes to be most economical. However, concrete with a compressive strength in standard cylinders of less than 3,000 psi may not have the requisite strength in bond or shear to take full advantage of the design provisions.

In general, it will be desirable to use concrete strength of 4,500 to 6,000 psi, in beams or columns; attention must be given to the upper limits of the allowable steel percentage.

Intermediate-grade steel, having a minimum yield point of 40,000 psi is generally used in flexural and compression members. Structural-grade steel may be used provided that the working or yield stresses are adjusted correspondingly to account for its minimum yield-point stress of 33,000 psi.

High-grade billet steel, or high strength alloy steels may be used with appropriate values of minimum specified yield point stress," and with corresponding changes in the permissible working stress, providing tests show the required ductility by ASTM.

The use of high-strength reinforcement will generally be found to be governed by the specifications concerning bending properties, since it is necessary in many instances to have hooks or bends to provide for the proper continuity of the steel. The specification for spiral reinforcement should be as per ACI code. The minimum strength of transverse reinforcement should correspond to that of intermediate-grade steel bars.

4.5 RECOMMENDED ARRANGEMENT OF REINFORCEMENT IN SHEAR WALLS

Shear walls or curtain walls in buildings are equally important in an earthquake resistant, no wall in an unimportant wall. Reinforcing details in every wall must receive careful attention, not only to ensure against unsightly cracking but also to make optimum use of the inherent energy-absorbing capacity of the wall even though its contribution to the lateral strength of the building is ignored in the computation. Of course, the contribution of walls to the stiffness of a structure should be taken into consideration in the determination of shears.

4.5.1 Minimum Wall Reinforcement

Minimum wall reinforcement should be provided in accordance with the requirement of the ACI Building Code. The following arrangements of wall reinforcement are offered as a guide.

Every wall must be properly designed for seismic forces or shears normal and parallel to its plane.

Welded wire fabric may be used for wall reinforcement in accordance with the requirement of the ACI Code.

4.5.2 Wall Reinforcement at Corner and Junctures

It is particularly important that the corners and junctures of intersecting walls be adequately tied together to ensure unity of action.

Recommended reinforcing details are shown in Fig. 4.1.

It should be noted that all horizontal bars extend nearly to the far face of the joining wall when they are bent in a right-angle around a No. 6 vertical bar and extended 24-bar diameter, but not less than 12 inches.

TABLE 4.1 MINIMUM WALL REINFORCEMENT REQUIREMENT

Wall Thickness in	Horizontal Reinforcement Spacing in		Vertical Reinforcement Spacing in	
	#3	#4	#3	#4
6	7	13	12	16
8	5	10	9	16
10	9	16	12	16
12	7	13	12	16

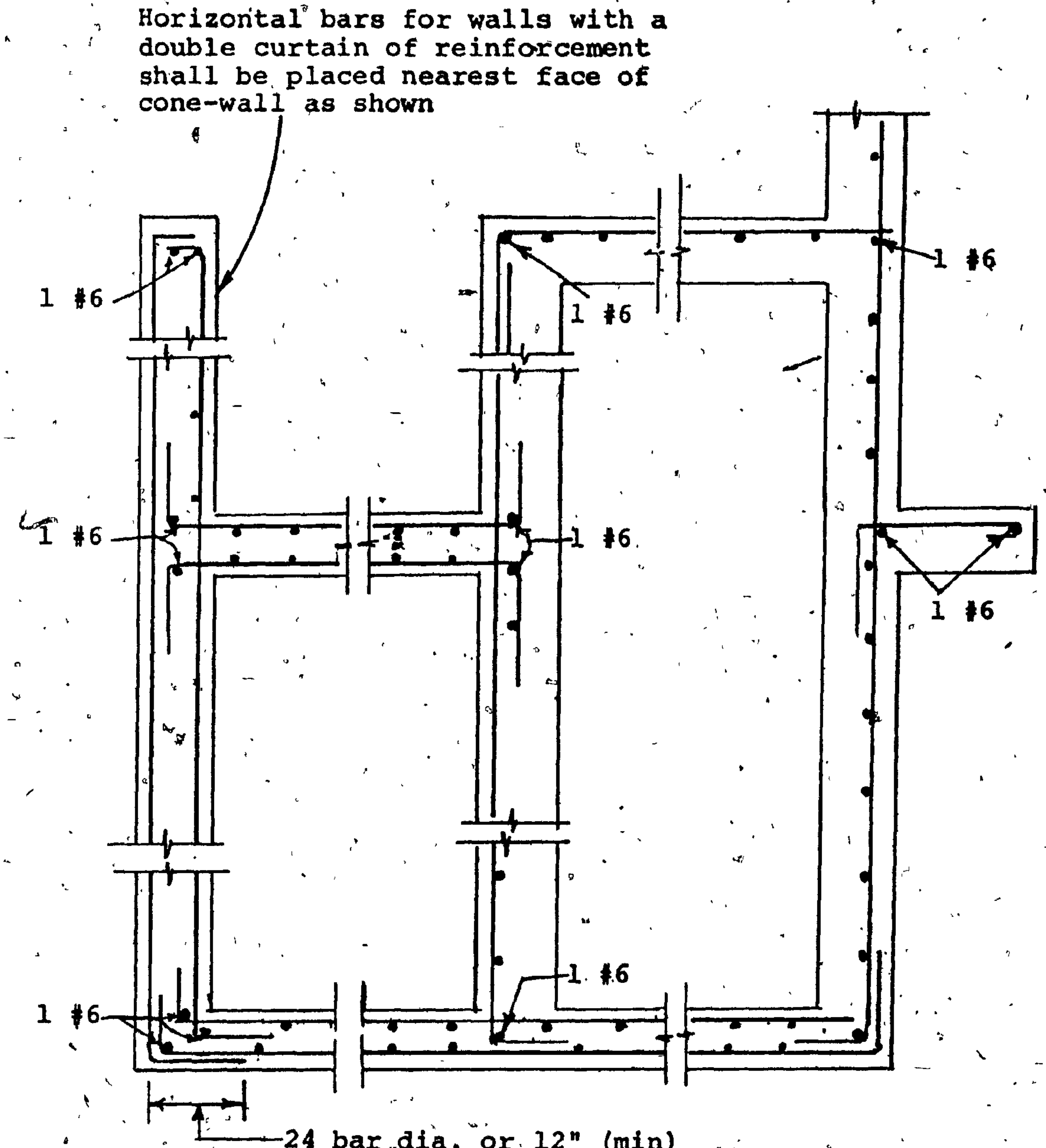


FIG. 4.1 Plan View Showing Typical Wall Reinforcement Detail

4.5.3 Wall Reinforcement Around Openings

Additional reinforcement should be provided at the sides and corners of openings in walls similar to those in floors, as shown in Figure 4.2. If the area of reinforcement in either direction is interrupted by the opening and is greater than the area of the trimmer bar, additional bars should be provided to equal the area of the interrupted bars and should be extended beyond the sides of the opening not less than 30 in. diameter or 16 in.

4.5.4 Splices for Wall Reinforcement

Where necessary to splice wall bars, they should be lap-spliced a minimum of 24 in. diameter or 12 in. Splices in adjacent bars should be staggered a minimum of 18 in. Recommended details for splicing wall reinforcement at vertical and horizontal construction joints are shown in Fig.4.3.

4.5.5 Shear Wall Reinforcement

The minimum amount of reinforcement in walls designed to resist shearing forces caused by earthquake motions should be 0.25 percent of the wall cross-section in both the vertical and horizontal direction and the spacing of the bars should not exceed that given in Table 4.1.

The maximum designed bending moment and shear, which occurs at the bottom of the wall unless larger openings exist

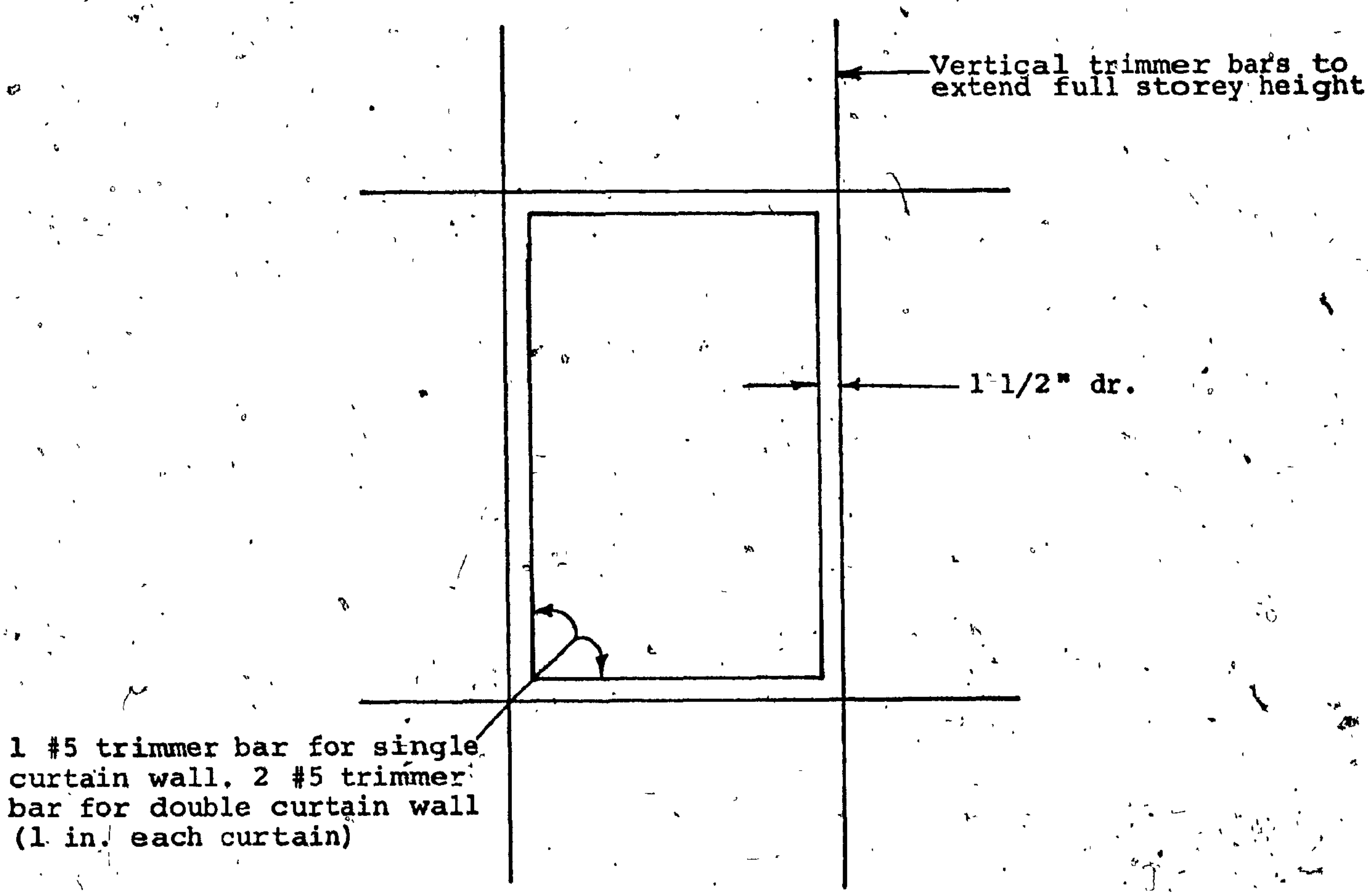


FIG. 4.2 Supplementary Reinforcement to be Provided at Wall Openings

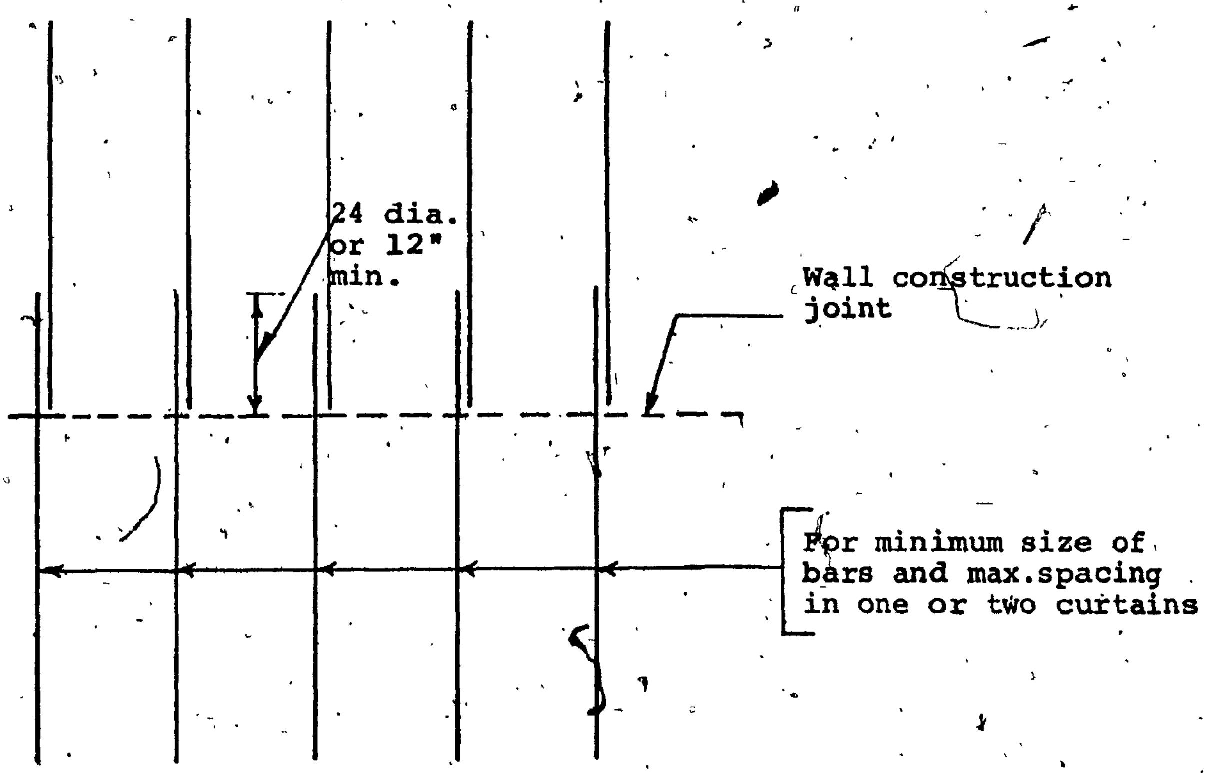


FIG. 4.3 Recommended Reinforcement Detail at Horizontal and Vertical Wall Construction Joints.

in the lower stories, should be increased by a factor of 1.5 and compared with the resistance at cracking of the uncracked wall estimated from the following equations

$$M_{cr} = 6 \frac{I}{C} \sqrt{f'_c} \qquad (4.1)$$

$$V_c = 4bL \sqrt{f'_c} \qquad (4.2)$$

where

M_{cr} = flexural cracking moment

I = moment of inertia of shear wall

c = distance from neutral axis to extreme fiber in tension

V_c = cracking shear

b = minimum width of wall

L = length of wall in horizontal direction

If the cracking resistance of the wall is not sufficient to counteract 1½ times the design moment and/or shear, special reinforcement should be provided in the wall such that the following expressions yield the desired shear and moment

$$M = \frac{1}{3} A_s f_y L \qquad (4.3)$$

where

A_s = total cross-sectional area of the vertical reinforcement distributed over the length of the wall and

$$N = 1.9\ bL\sqrt{f'_c} + A_s f_y \qquad (4.4)$$

where

A_s = total cross-sectional area of horizontal reinforcement distributed uniformly over the height of the wall equal to half its length

The amount of reinforcement indicated above may be reduced in accordance with the values of design moments and shear along the height of the wall to the minimum of 0.25 percent.

4.6 EXPANSION AND CONSTRUCTION JOINTS

Construction joints are stopping places in the process of placing concrete and are required because it is impractical to place concrete in a continuous operation except for small structures. Construction joints should be so designed and built as to prevent movement and to be essentially as strong as where there is no joint. Expansion joints are designed to allow for free movement of parts of a building because of size or shape. Expansion joints designed if considered necessary.

The most desirable location for construction joints should be detailed by the engineer and shown on the drawings.

Vertical construction joints are required in buildings