'THE COMPRESSIVE STRENGTH OF BRICK MASONRY WALLS WITH REFERENCE TO WALL/FLOOR SLAB INTERACTION'

A Thesis submitted for the Degree of

Doctor of Philosophy

of the

UNIVERSITY OF EDINBURGH

by

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November 1980

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ABSTRACT

The object of this thesis is to establish a theoretical approach for estimating the compressive strength of brick masonry walls taking into account the interaction between the brick supporting walls and reinforced concrete slab floors.

Chapter 1 introduces the problem of brickwork in compression and reviews previous investigations carried out in this field.

The determination of capacity reduction factors and accordingly the load bearing capacities for walls bent in double curvature is discussed in Chapter 2.

In Chapter 3, the capacity reduction factors and wall bearing capacities are discussed for two types of single curvature bending.

A simplified method for the determination of joint moments and eccentricities is developed in Chapter 4 which covers all usual joint configurations and locations, exterior or interior.

Chapter 5 presents a description of the test programme carried out and the analyses of the test results are presented in Chapter 6.

The application of the theories presented in computing joint eccentricities and loadbearing capacities is presented in Chapter 7 together with comparisons made with existing design codes as applied to practical design cases.

Finally, in Chapter 8, the general conclusions arising from this research are listed together with some suggestions for future work.

ACKNOWLEDGEMENTS:

I wish to thank Professor A W Hendry, Ph.D., D.Sc., FRSE, FICE, FI Struct E, for the opportunity to undertake this research under his personal supervision.

Thanks are also due to Mr R S Elder, Department Superintendent and all the Technical Staff of the Department of Civil Engineering and Building Science for their excellent co-operation, and to Mrs B McWilliam for her invaluable assistance. The skilful presentation of the thesis manuscript is the work of Mrs S Crilly.

Financial support for the experimental work was provided by the Brick Development Association and the British Ceramic Research Association.

Finally, I would like to thank the Government of Iraq for their generous approval in granting me leave of absence from employment to pursue this work and providing the necessary financial support.

PRINCIPAL NOTATIONS:

b	=	Wall width
c	=	Wall curvature
C_f	=	Coefficient (1/12)
C_s	=	Moment Coefficient for two way slabs, depending on configuration
f_k	=	Characteristic compressive strength of masonry
E_s	=	Modulas of elasticity of concrete
E_w	=	Modulas of elasticity of brickwork
e	=	Eccentricity of wall load
ε	=	e/t (relative eccentricity)
ε_f	=	Eccentricity at failure = $1/2\ (1 - {}^{P_L}/f_k.t)$
G	=	Dead load of wall per storey
H	=	Actual wall height
H_s	=	Length of equivalent column
H/t	=	Slenderness ratio
I_s	=	Second moment of area of floor slab
I_w	=	Second moment of area of brick wall
K	=	$\sigma_{br.}b.t.\ H^2/(EI)w$
$\bar{K}$	=	$\dfrac{2(EI)_s}{(EI)w} \cdot \dfrac{H}{L}$ (Single curvature)
$\bar{K}$	=	$\dfrac{(EI)s}{(EI)w} \cdot \dfrac{H}{L}$ (Double curvature)
L	=	Floor slab span
L_1	=	Longer floor span on one side of joint
L_2	=	Shorter floor span on the other side of joint
m	=	Ratio of short to long span of floor
m_1	=	Ratio of short span to long span of larger slab floor on one side of the joint
m_2	=	Ratio of the short span to long span of the shorter slab floor on the other side of the joint

$\bar{M}$	=	$wL^2/12$ (Fixed end moment)
$\bar{M}_R$	=	Rigid frame moment
M_o	=	Actual joint moment
n	=	Ordinal number of a floor slab reckoned from the top of a building (i.e. roof slab, n = 1)
P	=	Compressive load in wall
P_E	=	Eular critical load
P_u	=	Compressive force acting on the wall immediately above a floor slab
P_L	=	Compressive force acting on the wall immediately below a floor slab
R	=	A factor depending on wall eccentricity, wall slenderness, and type of wall curvature
S	=	Twice the length from wall end to end of the equivalent column
t	=	Actual wall thickness
t_{eff}	=	Effective wall thickness
U	=	Deflection of wall (or equivalent column)
U_o	=	Maximum deflection of wall (or equivalent column)
W	=	Total uniformly distributed load of slab dead and live load

$$\left.\begin{array}{lll} Z & = & PH^2/(EI)_w \\ \Omega & = & (H/t).\omega \\ \Phi & = & (H/t).\emptyset \end{array}\right) \quad \text{where: } H = 1/2 \text{ actual wall height for walls bent in double curvature and the full wall height for walls bent in single curvature}$$

β	=	Capacity Reduction factor = $P/\sigma_{b.r.}b.t. = Z/K$
σ	=	Wall stress
$\sigma_{b.r.}$	=	Prism compressive strength
σ_{max}	=	Maximum compressive stress in wall
ω	=	Angle of rotation of wall relative to the compressive force line
ξ	=	Wall strain
Φ_b	=	Ultimate wall end rotation corresponding to buckling failure
ψ	=	P_u/P_L

$\bar{\psi}$	=	$1 + \psi$
γ	=	Modification factor for stress failure equations (short walls)
γ	=	$0.5H/(0.5H - 1.5t)$ (Double curvature)
γ	=	$H/(H - 1.5t)$ (Single curvature)
γ_m	=	Partial safety factor
λ	=	Stiffness of wall/slab joint
$\bar{\lambda}$	=	$2(EI)_s/\lambda L$
α	=	$\bar{K}/2$
ϕ	=	Actual wall end rotation
θ_s	=	Slab end rotation
θ_j	=	Joint rotation
θ_w	=	Wall end rotation
η	=	$P_L/f_k.t$

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CHAPTER 1 - BRICKWORK IN COMPRESSION:

1.1 INTRODUCTION:

Investigation into the load bearing strength of brickwork was undertaken from time to time many decades ago, but it is only as recently as the late forties that loadbearing brickwork has come into use on the basis of engineering principles. The revived interest in loadbearing brickwork was prompted by the realisation that structural brickwork could, under favourable circumstances, be an economic proposition for high-rise structures, exemplified aptly in the erection of multi-storey buildings in loadbearing brickwork in Switzerland, Canada, USA, UK and elsewhere.

In the early stages of the investigation into the compressive strength of brickwork, research programmes were concerned mainly in gathering reliable experimental data on which to base the first Code of Practice and subsequently to update its provisions where these were found to be unduly conservative. Little effort was made at this point to achieve a theoretical solution which could describe the behaviour of brickwork in compression.

From observations of the failure mode of centrally compressed walls, it has been noted that failure is usually initiated by vertical cracking of the bricks.(1) It is also widely known that since masonry constructed with mortars composed of portland cement and hydrated lime possesses little tensile strength perpendicular to the course joints, such masonry should, as far as possible, be loaded concentrically in order to maximise its compressive loadbearing capacity. However, in practice, eccentricity of the gravity loading is virtually impossible to avoid, since bending of the floors imposes a displacement of the reaction from the centric axis of the wall. In addition, if the floor end rotations are restrained, a bending moment is induced in the wall which causes the/

the same effect on the wall bearing capacity as misalignment of the vertical load. Thus, the influence of eccentricity and slenderness are coupled in practice and, assuming that both are known, the capacity of the wall may be expressed as a fraction of the capacity of a short concentrically loaded wall, this factor may be defined as a stress, or capacity reduction factor.

Since the bearing strength of masonry, having low tensile strength, decreases rapidly with increasing eccentricity (2), the accurate assessment of the wall moment at a wall/floor joint is important. However, this assessment is difficult since the moment is influenced by the degree of fixity at the joint, the floor loading, the wall loading, the method of construction, and most significantly on the relative stiffnesses of the floor slab and wall. In addition, long term effects including creep may influence the magnitude of the wall eccentricity. As a result of these difficulties, little information is available to assist the designer in estimating the wall eccentricities which should be used in design.

If a structure composed of loadbearing masonry walls and reinforced concrete floor elements is idealized as a plane frame structure, several problems still exist in making an accurate assessment of the wall eccentricities. These problems may be discussed in relation to the rigidity of the wall/floor slab connections. If the wall/floor joints are considered to be fully rigid, the determination of the distribution of moment throughout the frame, for a given loading, is complicated by the fact that the wall stiffness may be dependent on the wall loading. Thus, if the eccentricities of the wall loadings cause tension cracking, the rigidity of the frame is reduced and a redistribution of moment takes place. Since the extent of wall cracking is not known in advance, an accurate analysis may require several iterations in which the effect of cracking on the moment redistribution is repeatedly

revised/

revised. This nonlinearity further restricts the validity of the solution obtained to the loading magnitude initially considered and a complete nonlinear analysis is, therefore, required for each loading of interest. Further refinements in the analysis, such as considering the secondary effects of wall bending deformations on the extent of cracking add to the complexity of the analysis.

At the other extreme, even if the wall/floor slab connection is considered to be perfectly pinned, the magnitude of the eccentricity of the floor reaction is not well defined. A common assumption is that the pressure distribution under the bearing area is triangular,[3] although if the floor slab span is reduced, a uniform pressure distribution may be assumed. However, even for a wall supporting a roof slab, either of these assumptions is an oversimplification which at best can only be valid for some situations. A more serious criticism of these assumptions is that there is little information to indicate that the errors involved are on the safe side. The fact that many loadbearing masonry structures designed on these basis have performed satisfactorily is not in itself a proof of the validity of these design eccentricities.

For a semi-rigid wall/floor slab connection, the moment-rotation behaviour of the joint must be accurately known, and unless it can be represented as a linear function which remains constant, a non-linear iterative solution procedure will be required irrespective of the possibility of wall cracking. In this case also, details of the joint construction, properties of the concrete floor slab, masonry and mortar, and the precompression in the joint will undoubtedly influence the moment-rotation behaviour.

1.2 REVIEW OF PREVIOUS WORK:

Past research[2] on brick masonry compression elements with well defined end conditions indicates that test results agree well with the theoretical/

theoretical predictions based on neglecting the tensile strength of the masonry. The complexity of theoretical investigations of the bearing capacity of linear elastic walls having no tensile strength depends on whether or not slenderness effects are considered. Investigations by Kazinczy(4) in 1933-34 and Nylander(5) in 1944 ignored the effects of second order deflection resulting from the compressive loading in determining wall bearing capacities. These additional deflections give rise to changes in the effective cross-sectional area of the wall with a resulting reduction in wall bearing capacity. Thus, more recent studies have included the effect of the additional deflection on the behaviour of the masonry wall. Unfortunately, consideration of the additional deflections coupled with the assumption of zero tensile strength leads to a complex differential equation. The basic equation was solved by Angervo(6) in 1954 and Chapman and Slatford(7) in 1957.

Other notable contributions include the development of equations by Monk(8) for cracked or uncracked walls which include the effects of slenderness. The resulting formulae are too complicated for practical use, and as a result, Monk developed a table of stress reduction factors for use in design. These factors were developed after modification of the theoretical equations as a result of a comparison with an extensive series of wall-column tests conducted by the Brick Institute of America. The present design specifications of the BIA(3) include stress reduction factors for both slenderness effects and eccentricity. The major variables involved in evaluating these reduction factors are, wall slenderness ratio, magnitude of the maximum eccentricity, and the type of wall curvature. However, in spite of this detailed elastic design procedure, the magnitudes of the end eccentricities to be used in design are primarily based on engineering judgement.

Chen/

Chen[9] has approached the theoretical problem by using curvature curves to predict the response of eccentrically loaded columns and walls. However, again, this study is primarily related to evaluating the effect of eccentric loading on wall capacities and little information is available to estimate eccentricities that should be used in design.

The effects of the loading condition on the strength of brick and mortar have been the subject of an extensive study carried out by Hendry.[10] The effects of slenderness on the capacity of brick masonry walls have been experimentally investigated by Hendry,[11] Fattal[12] and Yokel et al.[13]

Yokel[14] carried out a series of tests on a number of masonry walls, using both brick and concrete masonry. They suggested that rational analysis can be used to determine a lower limit of the strength of masonry wall under eccentric vertical load. The range of eccentricities in their experimental programme carried from zero to the kern point. A small number of concrete masonry walls were tested by Drysdale et al[15] and they concluded that the present method of designing walls had a very high safety factor. According to the above study, formulation of more rational design procedure is required to permit advantage to be taken of the extra capacity which exists in many loading conditions.

Yorkdale and Allen[16] carried out tests done at the Brick Institute of America on 12 brick walls tested with eccentricities 1/2 and 5/12 of the wall thickness. They concluded that tensile stress at the top quarter point of both short and tall walls, loaded with an eccentricity of 5/12 of the thickness, t, were sufficiently large to indicate formation of cracks in the bed joints at relatively low loads. Walls tested with load eccentricity of t/2 and slenderness ratio of 10, developed tensile strain, indicating cracking at relatively low loads at the level of the top quarter point. For walls with larger slenderness ratio, the cracks were formed at the top four courses.

The principal research efforts related to wall design have been contributed by Sahlin beginning in 1959.[17] Sahlin's work included the development of a design procedure based on the continuity of the wall/floor joint which requires evaluating the moment-rotation behaviour of various joint details. In addition, a comprehensive series of tests on frame structures was performed and the variation of wall eccentricities obtained under increasing wall precompressions. In these tests, the ratio of wall precompression to floor load was held constant, whereas in practice, this ratio is different at various floor levels.

Risager[18] analysed the statical behaviour and the bearing capacity of the simply supported, eccentrically loaded column of linear elastic material without tensile strength on the bases of an estimated deflection curve. In this work, the concept of an equivalent column was presented. The deflection, rotation and section of this equivalent column are identical to the original wall, and accordingly derived generalised expressions for eccentricity, angle of rotation of the wall ends, based on these relations, he derived equations for bearing capacity for uncracked and cracked cross-section and general expressions for wall buckling loads. However, in spite of the detail and importance of Risager's work, the equations developed are not practical for design purposes since the wall end rotation and/or the eccentricity has to be estimated based on very little information available.

Colville[19] developed equations for stress reduction factors and moment rotation relation based on the basic generalised expression of the equivalent column developed by Risager. These equations can be used for design, but require a tedious procedure to evaluate the stress reduction factors due to slenderness and eccentricity which determines the wall bearing capacity. Colville also presented a method to calculate the wall eccentricity by solving the appropriate quadratic/

quadratic equations involved. To verify the proposed equations, tests [20] were conducted on a full scale structure to determine the experimental eccentricities obtained for various floor loading and variable wall precompression.

In spite of the importance of the theories developed (17,18,19), a major obstacle arises in the application of these theories for practical use due to the complex and tedious computations involved. Furthermore, some of the equations developed were not fully explored to provide complete understanding of their application and characteristics. This is because previous authors did not make use of an electronic computer to solve and evaluate various complex equations and expressions defining the behaviour of the wall end rotation corresponding to various eccentricities and slenderness ratios in order to determine the wall bearing capacity. As a result, it is felt that a comprehensive study of existing work, coupled with the application of electronic computers is needed and will result in the development of a theoretical approach that will provide a useful tool in the structural analysis of load-bearing brick masonry walls compressed between floor slabs. Thus, the object of the theoretical part of this thesis, to study and apply existing theories and develop theoretical analysis for wall eccentricities and bearing capacities.

Accordingly, equations for wall eccentricities, capacity reduction factors are developed and presented together with tables and graphs which would present the application of these theoretical solutions to the design of brick masonry walls in compression.

In conjunction with the theoretical part of this work, experimental programmes were carried out to investigate the eccentricity of loading at wall/floor slab joints and to provide data for the verification of the theoretical solutions. Tests were conducted on a half scale, two storey single bay structure and on a full scale three storey/

storey two bay frame. Two ratios of slab/wall flexural rigidities were considered to provide a more thorough investigation of this important parameter.

1.3 OBJECT AND SCOPE:

The basic objective of this study was:

(a) To examine and develop existing theories for wall bearing capacity.

(b) To apply electronic computers to the solution of equations relating to wall bearing capacities.

(c) To develop a simplified method for calculation of joint eccentricities.

(d) To set out a method for the design of masonry walls in compression taking into account the interaction between brick walls and reinforced concrete floor slabs.

(e) To examine the behaviour and strength of brick masonry walls under various floor loadings and wall precompression.

(f) To observe the behaviour and measure the joint eccentricities corresponding to various floor loadings and wall precompression.

(g) To examine the influence of different floor/wall flexural rigidity ratio on the development of joint eccentricity.

CHAPTER 2 - WALL BEARING CAPACITY (DOUBLE CURVATURE BENDING)

2.1 INTRODUCTION:

Conventional methods for the estimation of the bearing capacity of masonry walls subjected to eccentric loading are based on a theoretical model of a hinged ended, brittle column. Real walls in multi-storey buildings are, however, compressed between reinforced concrete slabs through joints which are capable of transmitting bending moments. The transmitted moments on the one hand influence the deflected form of the wall and, on the other, control the end rotation of the wall. It follows, therefore, that a satisfactory method for the design of masonry walls in compression must allow for interactive effects between walls and floor slabs.

Solutions have been produced by Sahlin,(21) Risager(18) and Colville.(19) The theory presented in this and the next chapter represents a development of these theories. As it will be seen, the theoretical results in compressive strength values are in reasonable agreement with experimental results.

2.2 WALL CURVATURE:

The type of wall curvature is governed by the wall end eccentricities which in turn are related to the type and condition of floor loading. Thus, three types of wall bending are considered in this study, see Fig (2.1):

(1) Walls bent in double curvature with equal and opposite end eccentricities, as shown in Fig (2.1.a)

(2) Walls bent in single curvature with equal end eccentricities (Fig (2.1.b))

(3)/

(3) Walls bent in single curvature with zero eccentricity at one end (Fig (2.1.c)).

Double curvature bending is discussed in Chapter 2 and the single curvature bending in Chapter 3.

It should be noted that in practice, there are other possible combinations of end eccentricities. However, since the above three conditions bracket all possible cases, it is felt that further refinements in the consideration of the ratio of end eccentricities are not justified and that it is reasonable in practical design to consider every wall as conforming to one or other of these three cases.

2.3 WALLS BENT IN DOUBLE CURVATURE ($\varepsilon_1/\varepsilon_2 = -1.0$)

The basic assumption of the procedure for considering wall elements bent in double curvature is that the deflection curve can be composed of parabolic arcs. Coupled with this approximation is the introduction of an equivalent column which has a zero eccentricity of axial load at the ends and whose deflection between floors represents that of the actual column, or wall. This approach has been developed by Risager[18] for a wall having equal angles of rotation at both ends. The basic procedure has been generalized to consider walls with different end eccentricities. The tensile strength of the wall is considered to be zero, and it is also assumed that there is no sidesway between floor levels.

2.3.1 Deflected Shape of the Equivalent Column

Figure (2.2a) represents a typical storey of a multi-storey frame structure, in which the walls are bent in double curvature. A free body diagram of the left wall showing the forces applied to the wall is given in Fig (2.2.b). In this diagram, the weight of the wall has been neglected. The simply/

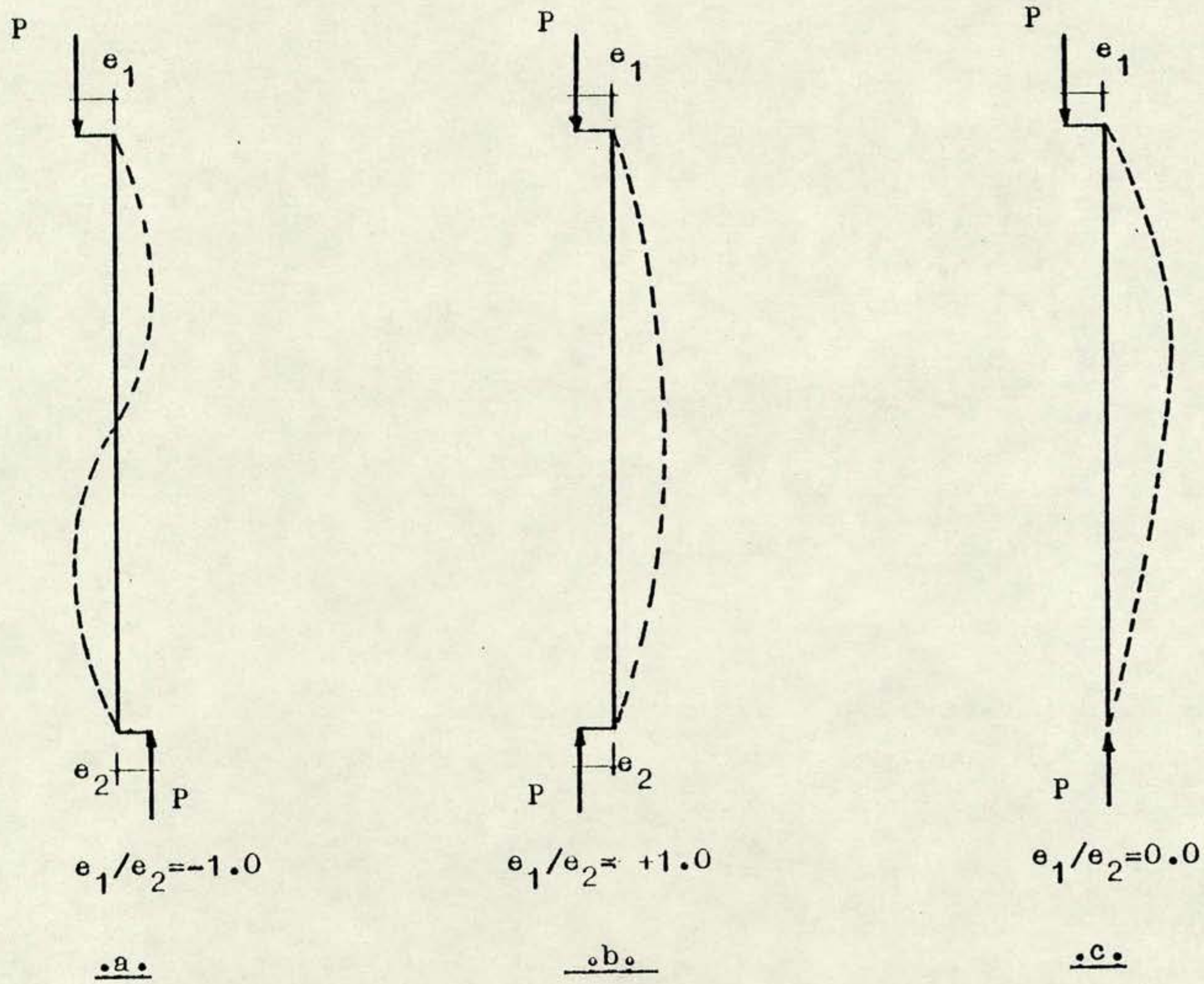

Fig.(2.1)- Wall Curvature

simply supported equivalent column,which has an identical deflection pattern as the actual wall between floor levels, is shown in Fig (2.2.c). As noted by both Risager[18] and Chen,[9] the equivalent column which has the same cross-section and material properties as the actual wall is statically equivalent to the actual wall. By considering Fig (2.2.c) to which corresponds an equivalent column with length H_s greater than H/2. From a statical point of view, wall b, and the part of the system c, lying between the dotted lines (representing floor levels) are completely uniform systems. It must, therefore, hold that $P_s = P$ and $e_s = e$ and the maximum eccentricity will occur at floor level. If the load P_s is increased and the angle of rotation ϕ at the ends at the same time is kept constant, the equivalent column tries to increase its deflection and thereby its angle of rotation ϕ at the floor level. As the equivalent column at floor level is prevented from rotating, an external anti-clockwise moment is created as a result of the reduction of the eccentricity e_s. Thus, provided that the equivalent column does not fail first, there will be a value of P_s at which the eccentricity $e_s = 0$. The length of the equivalent column will then be, $H_s = H/2$ and in such cases, the maximum eccentricity will be between the wall ends. Thus, for walls bent in double curvature, the length of the equivalent column will always be greater than or equal to half the length of the actual wall.

Figure (2.3) shows a wall with full storey height, H, and thickness,t. The width of the wall is b. On the wall acts a load P, and the curves of the deflection for uncracked cross-section is shown in (a), and that of a cracked cross-section/

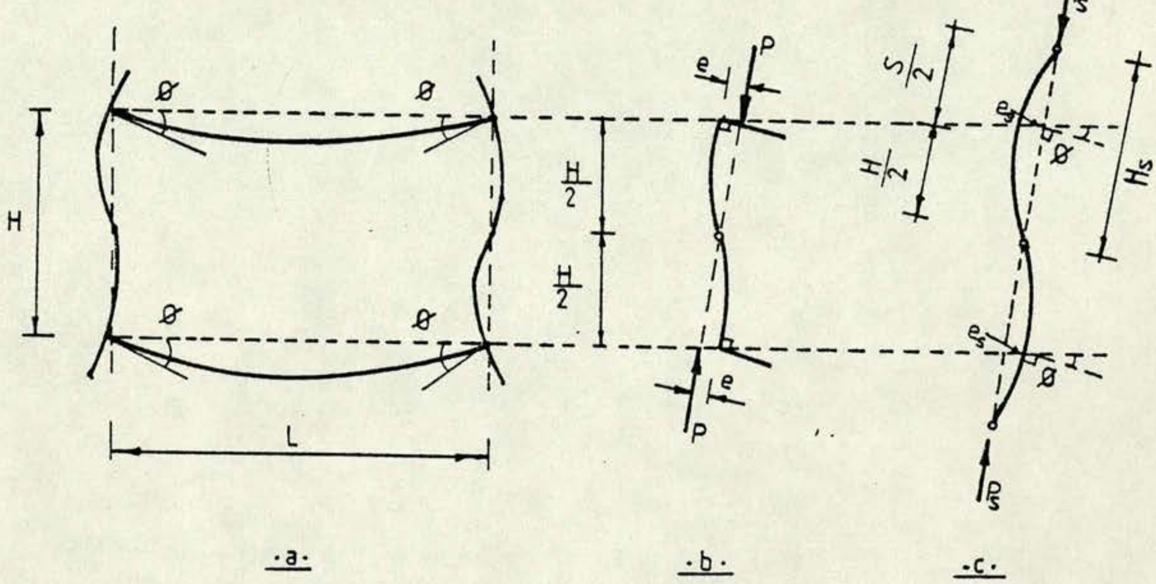

Fig. (2.2)

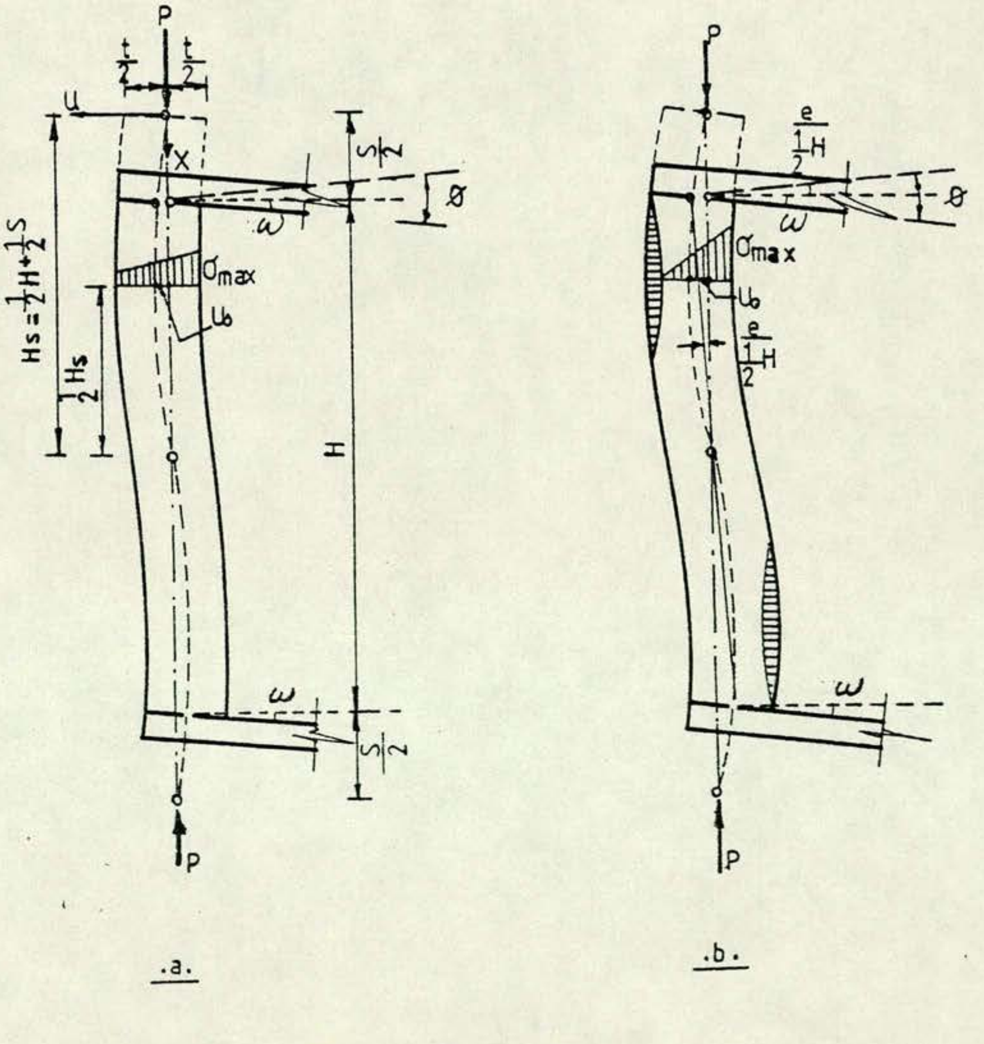

Fig. (2.3)

section in (b). The corresponding equivalent column has the length H_s. Assuming that the point of inflection of the wall is known or can be estimated, those portions of the equivalent column between the points of zero moment are assumed to have a parabolic deflection curve. Also implied in the following equations is the concept that the assumed point of inflection of the wall based on a consideration of the primary moment diagram does not move as a result of secondary deflections. That is, the point of inflection is fixed.

In computing the wall bearing capacity, the principal problem to be solved is to locate the section at which the eccentricity of the axial load, P, is maximum. As the slenderness ratio of the wall is increased, the likelihood that the section of maximum eccentricity lies away from the wall ends is also increased. However, as noted by Colville,[19] if the maximum eccentricity lies at the ends of the actual wall (i.e. at floor levels), the wall slenderness does not affect the wall bearing capacity computed from the ultimate compressive strength. Of course, if the bearing strength is based on stability of the wall, then the variations in wall slenderness ratio will in all cases influence the wall load carrying capacity.

Considering the portion of the equivalent column between points of zero moment (Fig (2.3)), and letting the maximum deflection of the column be U_o, the equations valid for a parabola with the deflection U, and the inclination du/dx may be expressed as[18]

$$U = 4\,U_o\,\frac{x}{H_s}\left(1 - \frac{x}{H_s}\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

In which H_s equals the length of that portion of the equivalent column being considered, and x is the distance from the end of the/

the equivalent column. From equation (2.1), the inclination du/dx is:

$$\frac{du}{dx} = 4\frac{U_o}{H_s}\left(1 - \frac{2x}{H_s}\right) \quad \ldots\ldots\ldots\ldots\ldots (2.2)$$

At x = S/2, the eccentricity of the wall, e, and the rotation of the wall end relative to the compressive force line, ω, may be obtained from equations (2.1) and (2.2) respectively as:

$$e = 2\,U_o \frac{S}{H_s}\,(1 - S/2H_s) \quad \ldots\ldots\ldots\ldots\ldots (2.3)$$

$$\omega = \frac{4\,U_o}{H_s}\,(1 - S/H_s) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.4)$$

Introducing the following notations:

$$\varepsilon = e/t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.5)$$

$$\Omega = \frac{(H/2)}{t}\,\omega \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.6)$$

$$\Phi = \frac{(H/2)}{t}\,\phi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.7)$$

In which H, represents the actual wall height, ϕ is the actual angle of rotation of the wall end (see Fig (2.3)), $H_s = H/2 + S/2$, the following general relations may be developed:

$$\Omega = \frac{4\,U_o}{t}\;\frac{(1 - S/H)}{(1 + S/H)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.8)$$

$$\varepsilon = \frac{4\,U_o}{t}\;\frac{S}{H}\;\frac{1}{(1 + S/H)^2} \quad \ldots\ldots\ldots\ldots (2.9)$$

From Figure (2.3) it will be seen that the angle of rotation at the ends of the wall, ϕ, can be expressed as:

$$\phi = \omega + {}^{e}/(H/2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.10)$$

By multiplying both sides of eq. (2.10) by (H/2)/t, the following relation is obtained:

$$\Phi = \Omega + \varepsilon \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.11)$$

Combining/

Combining eqs. (2.8, 2.9 and 2.11) gives:

$$\Phi = \frac{4\ Uo}{t} \quad \frac{1}{(1 + S/H)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.12)$$

Finally, from eqs. (2.9 and 2.12) we obtain:

$$\frac{S}{H} = \frac{\varepsilon}{\Phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.13)$$

The above equations are valid for both uncracked and cracked stress conditions and will be used to compute wall bearing capacities.

2.3.2 Stress Failure Equations:

2.3.2.1 Uncracked Wall:

For a rectangular wall of width, b, and thickness, t, composed of a linearly elastic material, and loaded within the kern by an eccentric load, P, the maximum compressive stress is given by the following equation[21]

$$\sigma_{max} = \frac{P}{bt} \left(1 + \frac{6e_{max}}{t}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.14)$$

In fact, the stress-strain relationship for masonry is non-linear. However, in reference (3) it is noted that for eccentrically loaded walls, good agreement is obtained between tests and theory using a linear stress-strain curve if failure is assumed to occur when the maximum stress equals 1.5 times the compressive strength. Thus, it is assumed (19) herein, that stress failure will occur when the maximum compressive stress in the wall is 1.5 times the compressive wall strength, that is:

$$\sigma_{max} = 1.5\ \sigma_{b.r.} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.15)$$

Where $\sigma_{b.r.}$ is the prism maximum compressive strength.

Substituting/

Substituting eq. (2.15) into eq. (2.14) and introducing the notation:

$$\beta = \frac{P}{\sigma_{b.r.}\, b.t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.16)$$

Where, P, is the compressive force in the wall, $\sigma_{b.r}.b.t$, is the compressive force of the same concentrically loaded wall, gives:

$$\beta = \frac{3}{2\left(1 + \frac{6\, e_{max}}{t}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.17)$$

The term, β, may be considered to be a stress reduction factor (or most commonly referred to as a capacity reduction factor) whose value depends on the maximum eccentricity of the load, e_{max}. By definition, β, is the ratio of the compressive (eccentric or concentric) force in the wall, to the compressive force of the same concentrically loaded wall, it follows, that, β must always be less than or equal to unity. Thus, eq. (2.17) is re-written:

$$\beta = \frac{3}{2\left(1 + \frac{6\, e_{max}}{t}\right)} \leqslant 1.0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.18)$$

The magnitude of e_{max} that to be used in eq. (2.18) depends on the slenderness of the wall. Thus, when the height of the equivalent column becomes equal to or greater than the actual wall height, i.e. $S/2 \geqslant H/2$, the maximum eccentricity will occur at the wall end (at floor level). Conversely, if the height of the equivalent column is less than the actual wall height, i.e. $S/2 < H/2$, then the maximum eccentricity will occur between the wall ends and $e_{max} = U_o$ is substituted in eq. (2.18). Considering the first case where $S/2 \geqslant H/2$, $e_{max} = e$, we obtain from eq. (2.18):

$$\beta = \frac{3}{2\,(1 + 6\varepsilon)} \leqslant 1.0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.19)$$

Equation (2.19) is based on a compressive stress failure occurring at a floor level. As indicated in reference (17), however, the stress condition at a wall/floor joint is much more complex than indicated by eq. (2.14). As a result, eq. (2.19) is too conservative and it is assumed[19] that compression failures cannot occur within a distance equal to 1.5 times the wall thickness from the joint; thus, equation (2.19) is modified by the factor γ, given as:

$$\gamma = \frac{(H/2)}{(H/2 - 1.5t)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.20)$$

Applying eq. (2.20) into eq. (2.19) we finally get:

$$\beta = \frac{1.5\ (H/2)}{(H/2 - 1.5t)(1 + 6\ \varepsilon)} \leqslant 1.0 \ \ldots\ldots\ldots \quad (2.21)$$

For the case where S/2 < H/2, as stated earlier, the maximum wall eccentricity will occur within the wall ends, substituting $e_{max} = Uo$ into equation (2.18) we obtain[19]

$$\beta = \frac{3}{2\left(1 + \frac{6\ Uo}{t}\right)} \leqslant 1.0 \ \ldots\ldots\ldots\ldots \quad (2.22)$$

From equations (2.12 and 2.13):

$$\frac{Uo}{t} = \frac{\Phi}{4}\left(1 + \varepsilon/\Phi\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.23)$$

Substituting eq. (2.23) into eq. (2.22) gives:

$$\beta = \frac{3}{2\left(1 + \frac{3}{2\Phi}\ (\Phi + \varepsilon)^2\right)} \leqslant 1.0 \ \ldots\ldots\ldots \quad (2.24)$$

Thus, for uncracked walls bent in double curvature, eq. (2.21) may be used to compute the capacity, or stress reduction factor provided that the maximum eccentricity is at floor level (i.e. joint), and eq. (2.24) is used where the wall eccentricity lies at some point removed from that point.

As mentioned in Section (2.3.1), in computing the wall bearing capacity, the main problem to be solved is to locate the section at which the eccentricity of the axial load is maximum, by deciding which of eqs. (2.21) or (2.24) is to be used.

Based on information given in reference (3) to distinguish between short and slender walls, it is suggested that for walls bent in double curvature, a column or wall having a slenderness ratio H/t less than 10 be regarded as a short wall (or column) and hence, the maximum eccentricity will always occur at the floor level and relative joint eccentricity ε is assumed to be greater than the wall end rotation function, Φ. Conversely, if the slenderness ratio is equal to or greater than 10, the maximum eccentricity will occur between floor levels, hence the wall end rotation function Φ is assumed to be greater than the wall eccentricity, ε. H, is the actual full height of the wall or column, thus, based on this assumption, the capacity reduction factor, β, is calculated from equation (2.21) provided that the wall slenderness is less than 10, and from equation (2.24) when the slenderness ratio is equal to or greater than 10.

2.3.2.2 Cracked Wall:

For a rectangular cross-section of a linear elastic material without tensile strength, the width of the compressed zone after tensile cracking occurs will be $3(t/_2 - e)$, and the magnitude of the maximum compressive strength is:(21)

$$\sigma_{max} = \frac{2P}{3b\ (t/2 - e_{max})} \quad \dots\dots\dots\dots\dots\dots \quad (2.25)$$

Once again, two cases must be considered depending on the location of the maximum eccentricity, e_{max}. Considering firstly, walls with slenderness ratio less than 10, the maximum eccentricity is assumed to occur at the floor level, and by applying the assumption of failure at a distance of 1.5 times the wall thickness away from the joint, the following eq. is derived from eqs. (2.15 and 2.16) after substituting into eq. (2.25).

$$\beta = \frac{9}{8}(1 - 2\varepsilon)\frac{(H/2)}{(H/2-1.5t)} \leqslant 1.0 \quad \ldots\ldots\ldots\ldots\ (2.26)$$

Thus, eq.. (2.26) replaces eq. (2.21) when the wall is cracked and $\varepsilon \geqslant 1/6$.

For slender walls, with slenderness ratio equal to or greater than 10, the maximum eccentricity is assumed to occur away from the wall ends, hence, from egs. (2.15, 2.16 and 2.23) the following equation is obtained by substituting the above equations into eq. (2.25):

$$\beta = \frac{9}{8}\left(1 - \frac{(\Phi + \varepsilon)^2}{2\Phi}\right) \leqslant 1.0 \quad \ldots\ldots\ldots\ldots\ (2.27)$$

Eq. (2.27) replaces eq. (2.24) when the wall is cracked.

The crack criterion is set at $\varepsilon \geqslant 1/6$, thus, applying this limit to eq. (2.17) yields to cracking of the wall occurring at a value of $\beta = 0.75$.

2.3.3 Moment-Rotation Equations:

In order to assist in determining the moment distribution in load bearing masonry construction, it is necessary to develop relationships between end moments and wall rotations. These relations are required for both uncracked and cracked walls.

2.3.3.1 Uncracked Wall:

For a simply supported wall, the load required to maintain equilibrium of the equivalent column being considered is

$$P_E = \frac{\pi^2 EI}{H_s^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.28)$$

Equation (2.28) represents Eular buckling load for a simply supported pinned column having zero eccentricity of loading at both ends.letting (19)

$$z = \frac{PH^2}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.29)$$

$$\text{And } K = \sigma_{b.r}.b.t.\ H^2/EI \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.30)$$

For walls bent in double curvature, where H, is taken as half the wall height. Also from reference (3), assuming $\sigma_{b.r.} = E/666$, it follows that, $K = (H/t)^2/222.0$, and equations (2.29) and (2.30) becomes respectively:

$$z = PH^2/4EI \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.31)$$

$$\text{And: } K = \sigma_{b.r}.b.t.\ H^2/4EI \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.32)$$

Dividing equation (2.31) by equation (2.32) gives:

$$\frac{z}{K} = \frac{P}{\sigma_{b.r}.b.t} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.33)$$

It follows that from eq. (2.16)

$$\beta = z/K \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.34)$$

From eqs. (2.28 and 2.31) by equating the flexural rigidity EI of the actual wall to that of the equivalent column since as assumed they have the same cross-section and material properties and by noting that the length of the equivalent column, $H_s = H/2 + S/2$, we obtain:

$$Z = \frac{\pi^2}{(1 + S/H)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.35)$$

Substituting eq. (2.13) into eq. (2.35) gives (19):

$$Z = \frac{\pi^2 \, \Phi^2}{(\Phi + \varepsilon)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.36)$$

Equation (2.36) gives a relationship between load, end eccentricity, and wall end rotation which is valid irrespective of the location of the maximum eccentricity, thus, it is independent of the wall slenderness and will be used for short and slender walls.

In the case of a concentrically loaded wall, with relative eccentricity, ε , equal to zero, substituting $\varepsilon = 0$ into eq. (2.36) yields:

$$Z = \pi^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.37)$$

Thus, there is an upper limit for Z , and eq. (2.36) is re-written as:

$$Z = \frac{\pi^2 \, \Phi^2}{(\Phi + \varepsilon)^2} \leqslant \pi^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.38)$$

Equation (2.38) is presented graphically in Figure (2.4) relating end moment and end rotation for uncracked walls in double curvature. Due to the fact that tedious calculations are involved in computing the value of Z corresponding to various values of wall eccentricity and end rotation, it was possible with the aid of computer programmes written in FORTRAN and run on the IBM 360/370 computer system to compile comprehensive data for the moment rotation equations for both uncracked and cracked walls in double and single curvature. These results are not included herein, instead, the computer programme for each case is included in Appendix (c) for walls in double and single curvature.

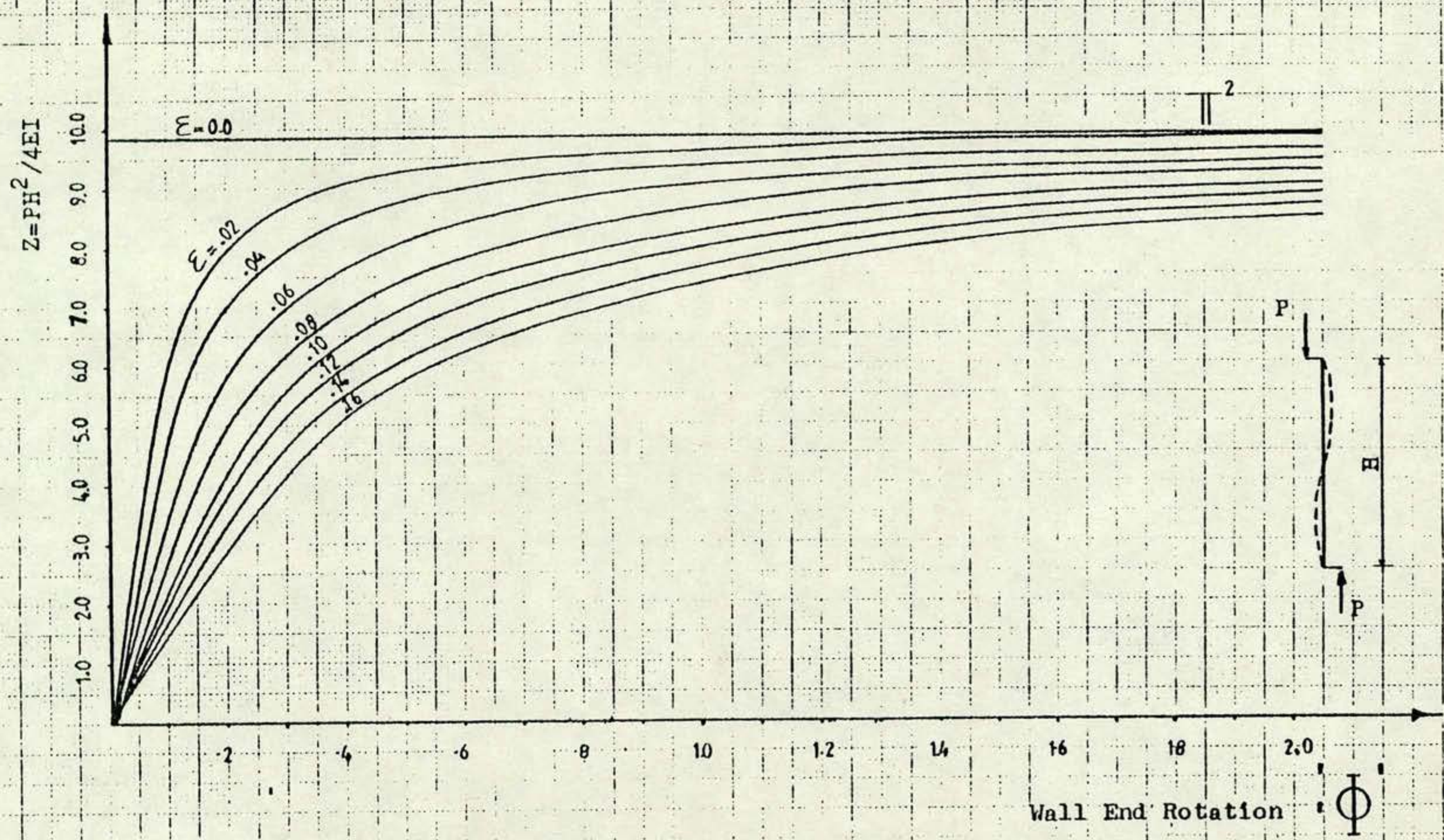

Fig.(2.4) Moment-Rotation Relations 'Uncracked Walls' (Double Curvature, $\mathcal{E}_1/\mathcal{E}_2=-1.0$)

2.3.3.2 Cracked Wall:

As shown earlier, the width of the compressed zone after tensile cracking is 3(t/2-e). The magnitude of the maximum compressive stress is re-written:

$$\sigma_{max} = \frac{2P}{3b\ (t/2 - e_{max})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.25)$$

The curvature, c, of this section is equal to the edge strain divided by the compressed length of the cross-section (18) so that:

$$c = \frac{\xi}{3\ (t/2 - e_{max})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.39)$$

where ξ is the maximum edge strain.

The maximum compressive stress, $\sigma_{max} = E\xi$, then eq. (2.39) becomes:

$$c = \frac{\sigma_{max}}{3E(t/2 - e_{max})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.40)$$

substituting eq. (2.25) into eq. (2.40), we finally obtain:

$$c = \frac{2P}{9Eb\ (t/2 - e_{max})^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.41)$$

The curvature of the same section due to bending is given by (22):

$$c = \frac{M}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.42)$$

The bending moment, M, is equal to the moment induced by the load maintaining equilibrium multiplied by the maximum wall deflection, Uo. By applying eq. (2.28), equation (2.42) finally becomes:

$$c = \pi^2 \frac{Uo}{H_s^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.43)$$

Since the curvature of this section due to the applied compressive force is equal to that due to bending, equating eqs. (2.41) and(2.43) yields:

$$\frac{\pi^2 Uo}{H_s^2} = \frac{2P}{9Eb\,(t/2 - e_{max})^2} \quad \ldots\ldots\ldots\ldots \quad (2.44)$$

By considering firstly, short walls, the maximum eccentricity, e_{max}, is assumed to occur at the wall/floor joint and $e_{max} = e$. By applying eqs. (2.23 and 2.31) into eq. (2.44) and re-arranging, the following equation is obtained:

$$Z = \frac{27\,\pi^2}{8} . \,\Phi\,(1 - 2\varepsilon\,)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.45)$$

For slender walls, the maximum eccentricity will occur away from the wall ends and $e_{max} = Uo$. Once again, by applying eqs. (2.23 and 2.31) into eq. (2.44) and re-arranging, the following is obtained:

$$Z = \frac{27\,\pi^2}{8} . \,\Phi\,(1 - \frac{(\Phi + \varepsilon\,)^2}{2\Phi})^2 \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.46)$$

Thus, eq. (2.45) is valid to compute the moment-rotation for short cracked walls in double curvature and eq. (2.46) for slender walls.

Equations (2.45) and (2.46) are presented in Figure (2.5) for cracked walls bent in double curvature by the use of a computer programme developed for cracked walls in double curvature, the value of Z, corresponding to the corresponding values of ε and Φ is calculated from the appropriate equation depending on the wall slenderness ratio, i.e. as previously assumed in Section 2.3.2.1,

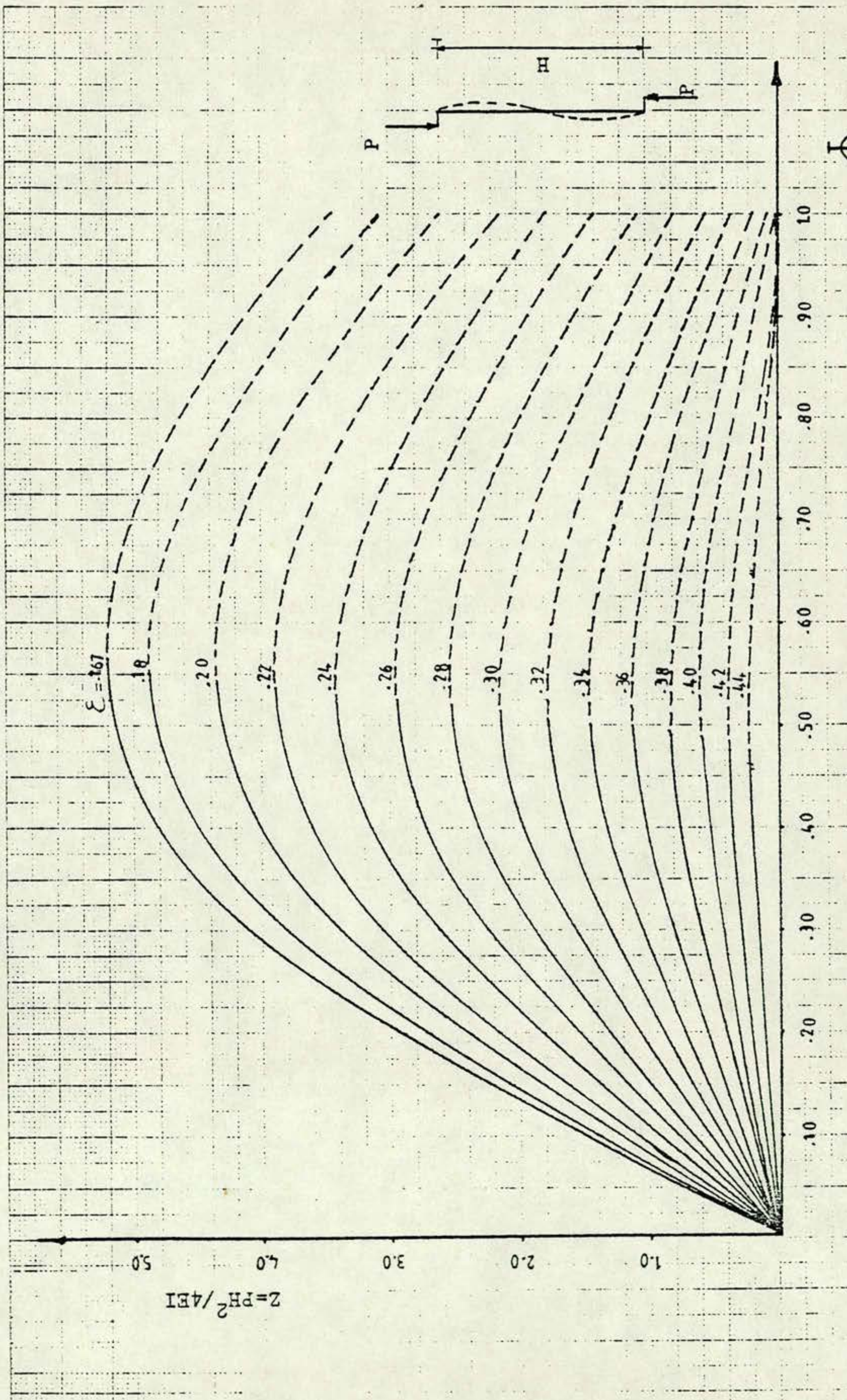

Fig.(2.5) Moment-Rotation Relations 'Cracked Walls' (Double Curvature, $\mathcal{E}_1/\mathcal{E}_2$=-1.0) for short walls, $\varepsilon > \Phi$, and equation (2.45) is used. For slender walls, $\Phi > \varepsilon$, and equation (2.46) is used to compute $\mathcal{Z}$.

2.3.4 Wall Buckling Loads:

Wall buckling loads may be computed from the moment-rotation equation by determining, for various eccentricity values, ε , the corresponding maximum value of $\mathcal{Z}$. Thus, $\mathcal{Z}$ max is equivalent to the wall buckling load. Assuming that cracking of the wall occurs prior to buckling(19), values of $\mathcal{Z}$ max may be obtained from eq. (2.46) by letting $\frac{d\mathcal{Z}}{d\Phi}$ = zero, and computing the value of the ultimate end rotation prior to buckling, Φ_b, corresponding to $\mathcal{Z}_{max}$. Thus, from eq. (2.46) with $\frac{d\mathcal{Z}}{d_\Phi}$ = zero, the following equation is obtained:

$$\Phi_b = \frac{(1 - \varepsilon) \pm \sqrt{4\varepsilon^2 - 2\varepsilon + 1}}{3} \qquad \ldots\ldots\ldots\ldots \quad (2.47)$$

By substituting the value of Φ_b from eq. (2.47) into eq. (2.38) the wall buckling load, $\mathcal{Z}_{max}$, can be computed for uncracked walls in double curvature, short or slender. For cracked walls, the wall buckling load is calculated from eq. (2.45) for short walls and e.q. (2.46) for slender ones. Buckling loads have been computed using eqs. (2.38, 2.45 and 2.46) with the use of computer programme developed for this case. The results are presented in Appendix (c) together with the programme. A plot of the wall buckling loads is presented in Fig. (2.6). The discontinuity in the curve corresponds to the transition from the uncracked to cracked wall.

The ultimate wall end rotation corresponding to buckling, Φ_b , is plotted in Fig. (2.7) as a function of the wall buckling load $\mathcal{Z}_{max}$, and the relative eccentricity ε . This relationship

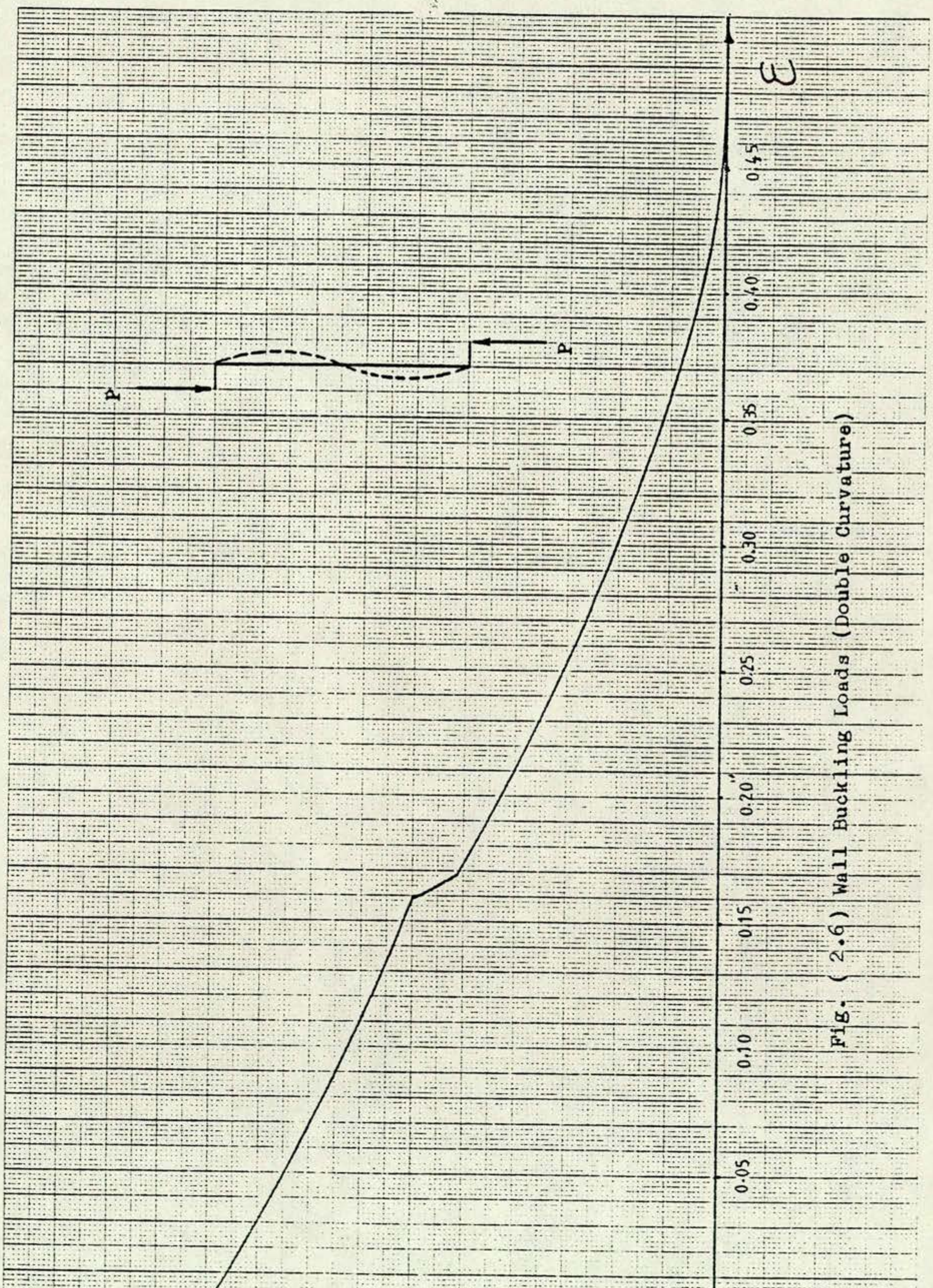

Fig. (2.6) Wall Buckling Loads (Double Curvature)

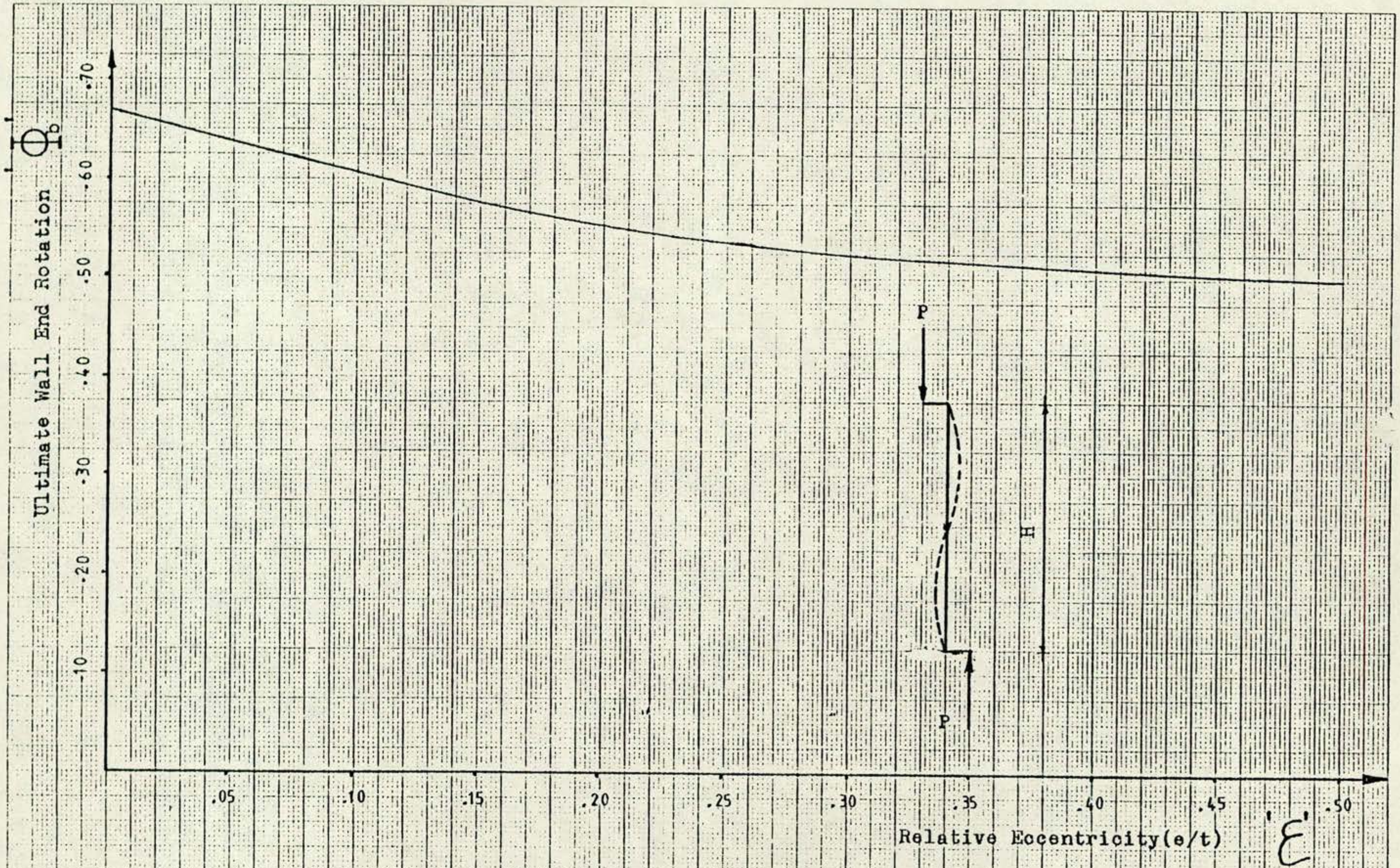

Fig. (2.7) Walls In Double Curvature

will be used in determining the Parameter R, which is a factor depending on the wall eccentricity, wall slenderness, and type of wall curvature as will be presented in Chapter (4) in dealing with joint eccentricities.

2.3.5 Wall Bearing Capacity:

Stress failure of the wall is assumed to occur when the end rotation of the wall is less than the rotation corresponding to buckling failure, Φ_b. For Φ values in excess of Φ_b, buckling failure is assumed to govern. In evaluating the stress failure loads, it is necessary to determine the wall end rotation, Φ. This is accomplished by equating the stress failure equations with the corresponding moment-rotation equation, thus by definition, from equation (2.34):

$$\beta . K = z \quad \ldots\ldots\ldots\ldots \quad (2.48)$$

2.3.5.1 Uncracked Wall:

For an uncracked wall in double curvature, two conditions must be considered, firstly, for short walls with $\varepsilon > \Phi$, and secondly, for slender walls with $\Phi > \varepsilon$. For the first case, substituting eqs. (2.21 and 2.38) into eq. (2.48) yields:

$$\frac{0.75\ H * K}{(0.5H - 1.5t)(1 + 6\varepsilon)} = \frac{\pi^2\ \Phi^2}{(\Phi + \varepsilon)^2} \ldots\ldots \quad (2.49)$$

The solution of this quadratic equation is:

$$\Phi = \frac{- B \pm \sqrt{B^2 - 4AC}}{2A} \quad \ldots\ldots\ldots \quad (2.50)$$

where: $A = \frac{3}{2}(H/2t).K + \frac{3}{2}\pi^2 + 9\pi^2\varepsilon - \pi^2(H/2t) - 6\pi^2\varepsilon(H/2t)$

$B = 3(H/2t).\varepsilon.K$

$C = \frac{3}{2}K\varepsilon^2(H/2t)$

For the case of slender walls, $\Phi > \varepsilon$, the appropriate equations are equations (2.24) and (2.38), from which:

$$\frac{3K}{2(1 + \frac{3}{2\Phi}(\Phi + \varepsilon)^2)} = \frac{\pi^2 \Phi^2}{(\Phi + \varepsilon)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(2.51)}$$

The solution of the above cubic equation was carried out using a computer programme to calculate the wall end rotation for a wide range of end eccentricities and slenderness ratios. These were used in turn to calculate the capacity reduction factors as will be presented later.

Since K, H, t are known, eqs. (2.49) and/or (2.51) may be used to calculate the appropriate Φ value corresponding to any value of end eccentricity, ε. Once the value of Φ is known, the failure load may be computed by determining the capacity reduction factor using either eq. (2.21) or eq. (2.24) provided that Φ is less than Φ_b. If the calculated Φ is equal or greater than Φ_b, then the appropriate equation to be used is eq. (2.38) and Φ_b is substituted for Φ. Solving eq. (2.38) yields the wall buckling load, Z_{max}, whence the capacity reduction factor can be calculated, i.e.

$\beta = Z/K.$

2.3.5.2 Cracked Wall:

For cracked wall in double curvature, once again two conditions must be considered. For short walls, substituting eqs. (2.26 and 2.45) into eq. (2.48) yields:

$$\frac{9}{8} K (1 - 2\varepsilon) \frac{0.5H}{(0.5H - 1.5t)} = \frac{27 \pi^2}{8} . \Phi \quad (1 - 2\varepsilon)^2 \ldots (2.52)$$

Solving eq. (2.52) for the wall end rotation, Φ:

$$\Phi = \frac{K(H/2)}{3\pi^2(1 - 2\varepsilon)(H/2 - 1.5t)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.53)$$

For slender walls, substituting eq. (2.27) and eq. (2.46) into eq. (2.48) yields

$$\frac{9}{8} K \left(1 - \frac{(\Phi + \varepsilon)^2}{2\Phi}\right) = \frac{27 \pi^2 \Phi}{8} \left(1 - \frac{(\Phi + \varepsilon)^2}{2 \Phi}\right)^2 \ldots\ldots (2.54)$$

The solution of the quadratic equation (2.54) is of the form:

$$\Phi = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A}$$

where: $A = 27 \pi^2/16$

$$B = \frac{27 \pi^2}{8} (\varepsilon - 1)$$

And $C = \dfrac{27 \pi^2 \varepsilon^2}{16} + \dfrac{9}{8} K$

Once again, since K, H, t are known, eqs. (2.53) and (2.54) may be used to calculate the appropriate Φ corresponding to any value of end eccentricity, ε. Finally, the failure load is computed by calculating the capacity reduction factor β from eqs. (2.26) or (2.27) depending on the wall slenderness provided that the computed wall end rotation, Φ, is less than the buckling wall end rotation Φ_b. If the calculated Φ is equal to or greater than Φ_b, then, the appropriate wall buckling equation to be used in eq. (2.45) for short walls or eq (2.46) for slender walls after substituting the value of Φ_b for Φ in these equations. Thus, the capacity reduction factor is computed as:-

$\beta \;\; = Z\,/K.$

As may be expected, calculation of the capacity reduction factors is complex and tedious but has been carried out by developing special computer programmes. For uncracked slender walls in double curvature, the solution of the cubic equation required the presentation of a special programme to compute the wall end rotation Φ . The computed real values of Φ (imaginary values ignored) were fed as input data in a second programme where the walls considered were in the uncracked mode. The value of Φ used in this programme were either read from the input data (if the walls are slender) or calculated within the same programme for short walls, following the procedure outlined above, the capacity reduction factors were computed and printed. In the case of cracked walls, another programme was employed to solve the quadratic equation involved and compute the capacity reduction factors in the same manner as outlined earlier. These results, together with the programmes are presented in Appendix (c). It should be pointed out that, the capacity reduction factors are titled as 'stress reduction factors (V) as this term was originally defined at time of preparing these programmes.

As may be seen from these results, the eccentricities considered ranged from zero up to 0.5 for slenderness ratios ranging from 5 up to 80. Those reduction factors most likely to be applicable in practical designs, are shown in Figure (2.8). The discontinuities in the curves correspond to the transition from the uncracked to cracked wall and/or the solution of different equations governing the behaviour of the wall at various stages. Interaction between wall and floor slabs is taken into account through the eccentricity ratio as will be shown in Chapter (4). In determining the capacity reduction factors from either the given tables or Fig (2.8), the slenderness ratio (H/t) is equal to the actual wall height divided by the actual wall thickness, as in computing the capacity reduction factors from the appropriate equations the wall height is halved in these equations; thus, the output print out represents the actual wall slenderness ratio.

In order to check the validity of the stress failure equations and the buckling load equations developed earlier, theoretical prediction of wall bearing capacities are compared with test results conducted by the Brick Institute of America.[3] The capacity reduction factors obtained by tests represent the ratio of the compressive eccentric load in a wall to the concentric axial load for the same type of wall. It is assumed that stress failure of the wall is to occur when the end rotation of the wall is less than the rotation corresponding to buckling failure, Φ_b. For Φ values greater or equal to Φ_b, buckling failure is assumed to govern.

Thus, for walls bent in double curvature with equal but opposite end eccentricity, the experimental and theoretical results are given in Table (2.1) and are also shown in Fig (2.9) in order to facilitate the comparison. Within the limits of experimental accuracy to be expected in this type of testing, agreement between

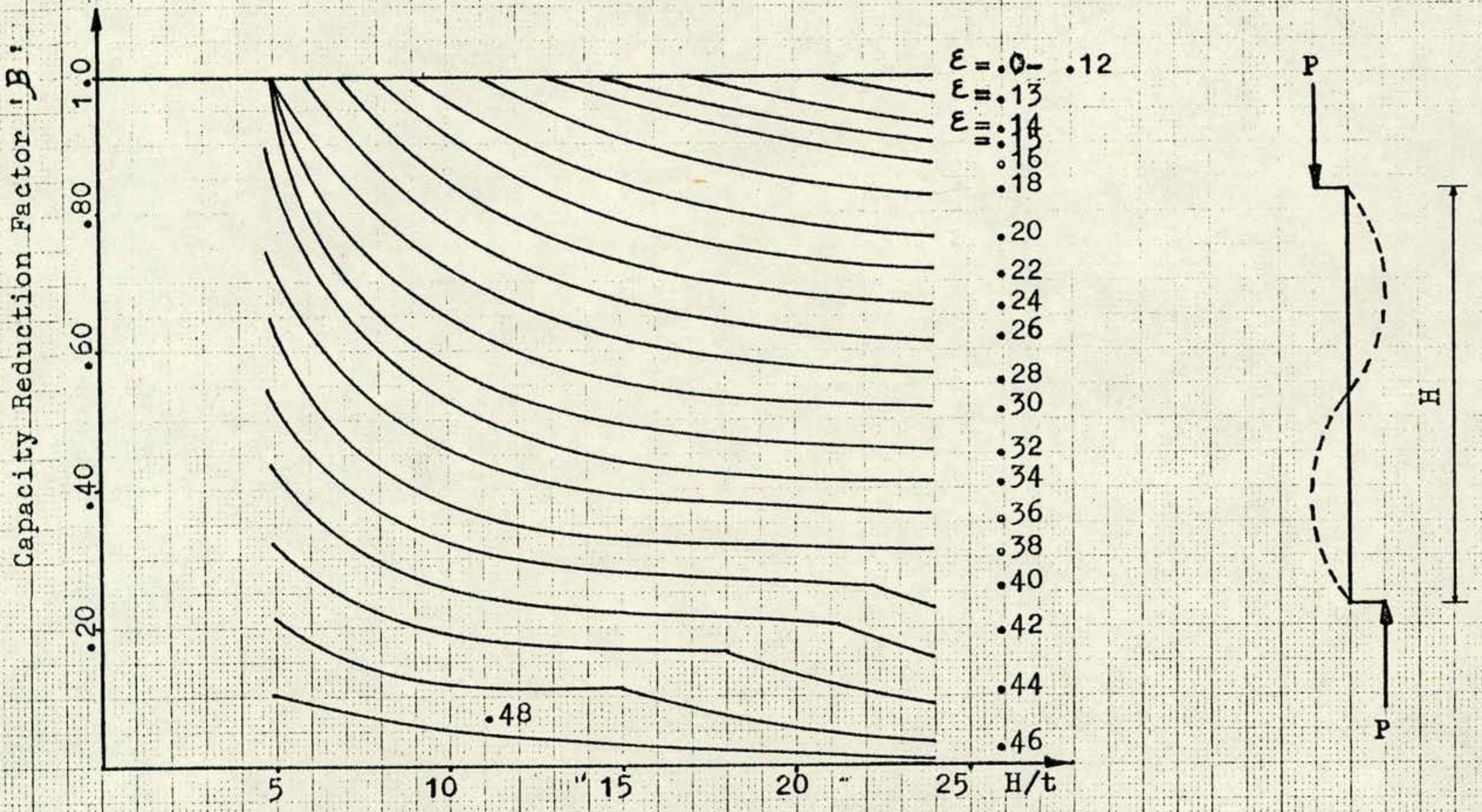

Fig.(2.8) Capacity Reduction Factor Vs. Wall Slenderness Ratio

Walls Bent In Double Curvature.

theory and experiment may be regarded as satisfactory, especially as the theoretical β would be very difficult to achieve at these very high slenderness ratios on account of experimental difficulties.

If the maximum value of slenderness ratio experienced in practice is taken as around 25, then the maximum differences between test and theory occur with $\varepsilon = 1/6$, where the theoretical capacity reduction factor, β, is around 26% greater than the corresponding test values. For eccentricity value, $\varepsilon = 1/3$, this difference is around 13%. Thus, in summary, the capacity reduction factors equations presented may be considered adequate for determining the moment-rotation relations as well as wall bearing capacities.

Table (2.1) - Capacity Reduction Factors - Walls bent in double curvature

H/t	ε	β_{Test}	β_{Theory}	% Diff.
6.6	1/6	0.79	1.00	26.5
22.7	"	0.70	0.86	22.8
46.1	"	0.31	0.55	77.4
51.4	"	0.15	0.44	193.0
6.6	1/3	0.71	0.69	2.9
20.5	"	0.47	0.44	6.8
20.5	"	0.47	0.44	6.8
20.5	"	0.47	0.44	6.8
20.5	"	0.50	0.44	13.6
22.7	"	0.47	0.43	9.3
46.1	"	0.20	0.16	25.0

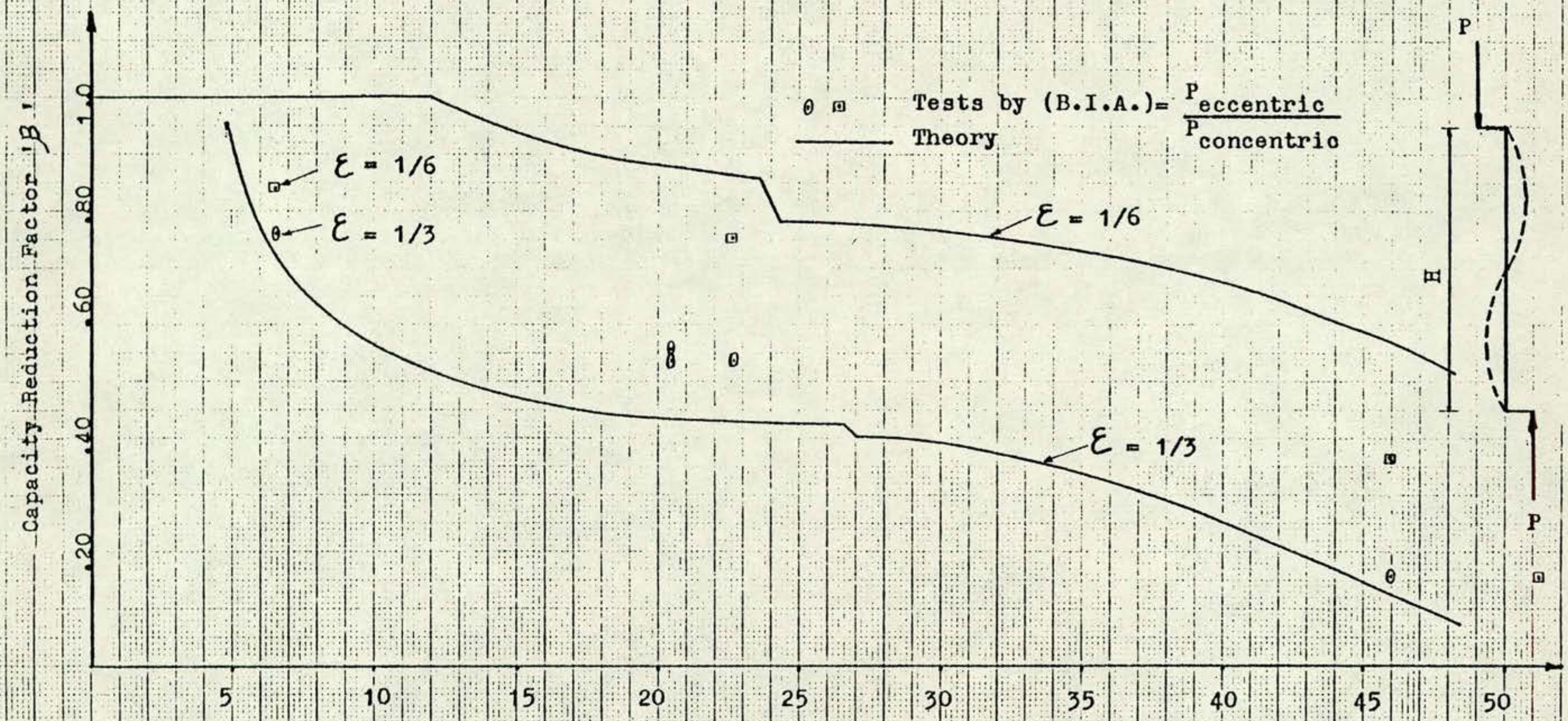

Fig. (2.9) Comparison of C.R.F. Between Theory and Tests by The Brick Institute of America

CHAPTER 3 - WALL BEARING CAPACITY (SINGLE CURVATURE BENDING)

3.1 INTRODUCTION:

The Bearing Capacity of walls bent in single curvature may be divided into two curvature bending categories; firstly, walls bent in single curvature with equal end eccentricities, as shown in Figure (2.1.b) in Chapter (2) where, basically, the maximum compressive stress in the wall occurs at mid-height, hence it is only necessary to consider the condition of uncracked or cracked walls, since the maximum eccentricity will always occur at mid-height of the wall and, therefore, no differentiation is necessary between short or slender walls. The second type of single curvature bending is where the eccentricity of loading at one end equals zero, in this case, the wall curvature is exactly like that of walls bent in double curvature, with the exception that the full height of the wall must be considered as a portion of the equivalent column. Thus, all stress failure equations and moment rotation equations developed for walls in double curvature are valid provided that the full height of the wall is considered in these equations as will be dealt with in this chapter.

3.2 WALLS BENT IN SINGLE CURVATURE WITH EQUAL END ECCENTRICITIES ($\varepsilon_1 / \varepsilon_2 = +1.0$) :

Figure (3.1.a) represents a part of a continuous multi-storey building where the inner walls may be considered bent in single curvature with equal end eccentricities. This type of curvature may be encountered in those cases where one span is assumed to be loaded by live loads on successive floors, and the other spans unloaded and/or, one span has a considerable span difference compared with adjoining spans (usually this difference would be greater than 20%). In such cases and where the structure configuration being analysed dictates such a curvature bending, the wall should be analysed as a wall bending in single curvature between floor levels with equal end eccentricities as shown in Figure (3.1.b). This condition of bending is the most severe case of all types of wall bending considered where the maximum central deflection and wall eccentricity are encountered. In practice, however, it is not generally critical due to the fact that such walls develop small bending moments due to the relatively small eccentricities developed as a result of non-uniform floor loadings and/or span differences.

Figure (3.2) shows a wall with storey height H, width b, and thickness t, the deflection curves for uncracked and cracked cross-sections are shown in (a) and (b) respectively.

The structural solution proposed is based on the assumption of linear elastic material having no tensile strength. It is also assumed that the deflected form of the wall between floor slabs may be represented by parabolic arcs and that there is no sidesway between floor levels since the horizontal thrust generated due to floor loading may be assumed to be counter-balanced by the reinforced concrete floor slabs. Based on these assumptions, the deflected shape of a wall bent in single curvature having equal end eccentricities may be approximated by a parabola having the equation of the form identical to equation (2.1) except that the height of the equivalent column H_s, being replaced by the actual wall height, whence,

$$U = 4\, U_o \, \frac{X}{H} \, (1 - X/H) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

And the wall inclination, ω , is:

$$\omega = \frac{dU}{dX} = \frac{4U_o}{H} (1 - 2X/H) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.2)$$

At the joint, X = 0, and the angle of rotation of the wall end ϕ, is:

$$\phi = \omega_{x=0}$$

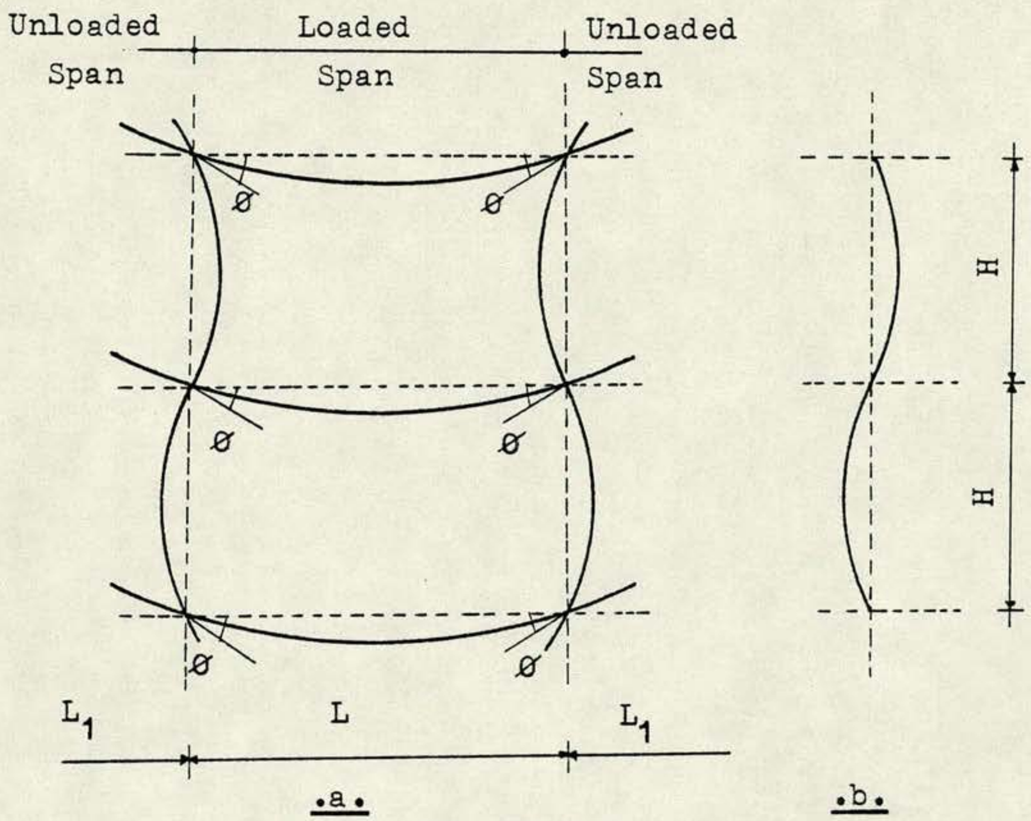
Unloaded
Span
Loaded
Span
Unloaded
Span
Ø
H
H
L1
L
L1
.a.
.b.

Fig.(3.1)

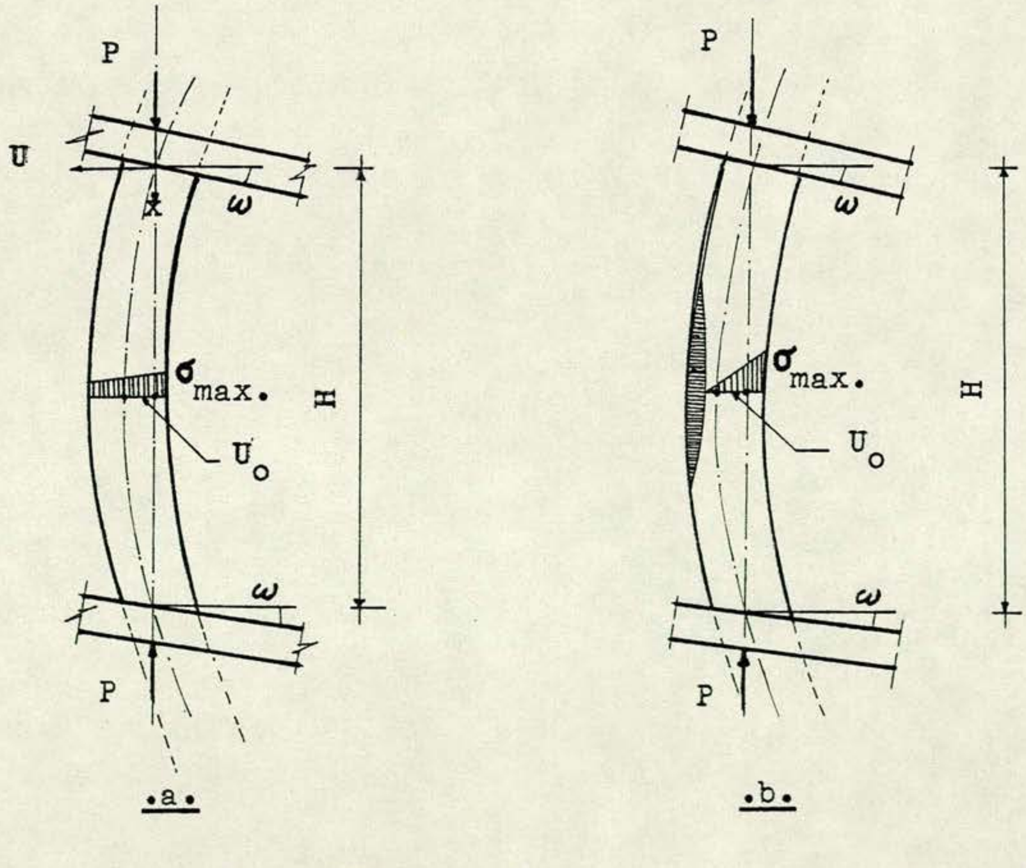
P
U
x
ω
σ max.
U0
H
P
.a.
.b.

Fig.(3.2)

hence:

$$\phi = \frac{4Uo}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.3)$$

Letting:

$$\varepsilon = e/t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4)$$

$$\Phi = \phi \ (H/t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.5)$$

Multiplying equation (3.3) by (H/t) gives:

$$\Phi = 4Uo/t \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.6)$$

3.3 STRESS FAILURE EQUATIONS

3.3.1 Uncracked Wall:

For a rectangular wall of width b, and thickness t, composed of linearly elastic material and loaded within the kern by an eccentric load P, the maximum compressive stress in the wall occurs at mid-height, where the load eccentricity equals (e + Uo), where e, is the end eccentricity of the compressive force P, thus, the maximum compressive stress in the cross-section may be written as (19)

$$\sigma_{max} = \frac{P}{bt} \left(1 + \frac{6}{t} (e + U_o)\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (3.7)$$

Assuming failure of the cross-section will occur when the maximum wall stress is equal to 1.5 times the wall compressive strength, hence,

$$\sigma_{max} = 1.5 \ \sigma_{br} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.8)$$

Re-introducing the capacity reduction factor, β , where:

$$\beta = \frac{P}{\sigma_{b.r} . b.t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.9)$$

Substituting equations (3.7 and 3.8) into equation (3.9) and noting that from equation (3.6) $\Phi = 4U_o/t$, hence equation (3.9) becomes:

$$\beta = \frac{3}{2(1 + 6\varepsilon + \frac{3}{2}\Phi)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.10)$$

Since, by definition, the capacity reduction factor is always less than or equal to unity, equation (3.10) is re-written as:

$$\beta = \frac{3}{2(1 + 6\varepsilon + \frac{3}{2}\Phi)} \leqslant 1.0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.11)$$

Thus, equation (3.11) is used to compute the capacity reduction factors for uncracked walls bent in single curvature having equal end eccentricities irrespective of their slenderness ratios (H/t).

The cracking criterion is set at $\varepsilon \geqq 1/6$, hence, from equation (3.11) with $\Phi = 4\ U_o/t$, at the wall floor joint, x = 0 and $U_o = 0$, it follows that cracking of the wall occurs at a value of $\beta = 0.75$.

3.3.2 Cracked Wall:

In a rectangular section of a linear elastic material with no tensile strength, the width of the compressed zone after tensile cracking occurs is 3(t/2 - e). The magnitude of the maximum compressive stress is (17)

$$\sigma_{max} = \frac{2P}{3b\ (t/2 - e_{max})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.12)$$

Since the maximum compressive stress in the wall occurs at mid-height, the maximum eccentricity e_{max}, equals $(e + U_o)$, hence:

$$\sigma_{max} = \frac{2P}{3b(t/2 - e - U_o)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.13)$$

Substituting equations (3.6, 3.8 and 3.11) into equation (3.13) and re-arranging gives:

$$\beta = \frac{9}{8}\ (1 - 2\varepsilon - \Phi/2) \leqslant 1.0 \quad \ldots\ldots\ldots\ldots\ldots \quad (3.14)$$

Equation (3.14) is used to compute the capacity reduction factors for cracked walls bent in single curvature having equal end eccentricities irrespective of their slenderness ratios.

Because of the location of the section of maximum stress is known to be at mid-height of the wall, only one equation is required to define the capacity reduction factor for each of the uncracked or cracked mode when walls in single curvature with equal end eccentricities are considered.

3.4 MOMENT-ROTATION EQUATIONS:

Once again, in order to determine the moment distribution in load-bearing masonry, it is necessary to develop relationships between wall end moments and end rotations for uncracked and cracked cross-sections. The load required to maintain equilibrium of a pin ended wall or column is given as:

$$P = \pi^2\ EI/H^{\,2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.15)$$

Where E, is the modulas of elasticity and, I, the second moment of area of the brick wall being considered.

Re-introducing the terms:

$$z = PH^2/EI \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.16)$$

And

$$K = \sigma_{b.r.}\, b.t\ H^2/EI \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.17)$$

As mentioned earlier, from Reference (3), assuming $\sigma_{br} = E/666$, the value of K becomes:

$$K = \frac{(H/t)^2}{55.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.18)$$

Dividing equation (3.16) by (3.17) yields:

$$\beta = z/K = \frac{P}{\sigma_{br.}b.t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.19)$$

3.4.1 Uncracked Wall

For an uncracked wall in single curvature, where the compressive force lies within the kern at the wall end, the curvature of the wall end is given by (17):

$$C = \frac{P.U_o}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.20)$$

And the curvature of the wall end due to bending is:

$$C = \frac{M}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.21)$$

Since the maximum moment occurs at mid-height of the wall where this bending moment is equal to the moment induced by the compressive force maintaining equilibrium of the wall multiplied by the maximum central wall deflection, which, at wall mid-height equals the sum of the end eccentricity and wall deflection induced by the force P, thus, equation (3.21) is re-written:

$$C = \frac{P\,(e + Uo)}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.22)$$

Equation (3.20) may be re-written after substituting equation (3.15) as:

$$C = \frac{\pi^2}{H^2} \;.\; Uo \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.23)$$

Equating equations (3.22) and (3.23) yields:

$$\frac{\pi^2\,Uo}{H^2} = \frac{P\,(e + Uo)}{EI}$$

From which:

$$z = \frac{\pi^2\Phi}{(4\,\varepsilon + \Phi)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.24)$$

can be obtained.

For the case of a concentrically loaded wall, with $\varepsilon = o$, hence:

$$Ƶ = \pi^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.25)$$

Thus, an upper limit exists for equation (3.24), whence it is rewritten as:

$$Ƶ = \frac{\pi^2\Phi}{(4\varepsilon + \Phi)} \leqslant \pi^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.26)$$

Equation (3.26) relates the wall load, end eccentricity and end rotation which is valid prior to cracking of the wall.

3.4.2 Cracked Wall:

For cracked walls in single curvature, the curvature of the cross section is equal to the edge strain divided by the compressed length of the cross-section (17) so that:

$$C = \frac{\dot{\xi}}{3(t/2 - e_{max})} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.27)$$

As the maximum compressive stress in the wall occurs at mid-height, hence, $e_{max} = e + Uo$, and equation (3.27) is rewritten:

$$C = \frac{\dot{\xi}}{3(t/2 - e - U_o)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.28)$$

Substituting equation (3.13) into equation (3.28) gives:

$$C = \frac{2P}{9Eb\,(t/2 - e - U_o)^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (3.29)$$

The curvature of the wall end is also defined by equation (3.20) which is valid prior to cracking of the wall, whence: for a cracked section:

$$C = \frac{P\,U_o}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.30)$$

is also valid. By applying equation (3.15) into equation (3.30) we obtain:

$$C = \frac{\pi^2\,U_o}{H^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.31)$$

Equating equations (3.29) and (3.31) gives:

$$\frac{\pi^2 U_o}{H^2} = \frac{2P}{9Eb(t/2 - e - U_o)^2} \quad \ldots\ldots\ldots\ldots\ldots \text{(3.32)}$$

Using equations (3.6, 3.16 and 3.32) we obtain after re-arranging:

$$Z = \frac{27\ \pi^2}{8} \quad \Phi \quad (1 - 2\varepsilon - \Phi/2)^2 \quad \ldots\ldots\ldots\ldots\ldots \text{(3.33)}$$

Thus, equation (3.33) gives the relationship between the wall load P, end eccentricity ε , and wall end rotation Φ which is valid after wall cracking occurs.

The moment rotation equations (3.26) and (3.33) were programmed on an IBM 360/370 series electronic computer to obtain all the parameters involved for various combinations of wall eccentricities and wall end rotations. Part of the results obtained are presented in Figures (3.3) and (3.4) for uncracked and cracked walls respectively. The computer programme is presented in Appendix (C).

3.5 WALL BUCKLING LOADS:

Wall buckling loads may be computed from the moment rotation equations (3.26 and 3.33) for uncracked and cracked walls respectively by determining Z_{max} for various eccentricity values. Since equation (3.26) was derived on the basis that the curvature of the wall end is defined by equation (3.20) which is valid for the case where the compressive force lies within the kern at the end of the wall but outside the kern in some parts of the wall (17). Thus it is assumed that cracking occurs prior to wall buckling and equation (3.33) is used throughout. By setting $\frac{dZ}{d\Phi}$ = zero and solving equation (3.33) the wall end rotation corresponding to wall buckling, Φ_b , is found to be equal to (19):

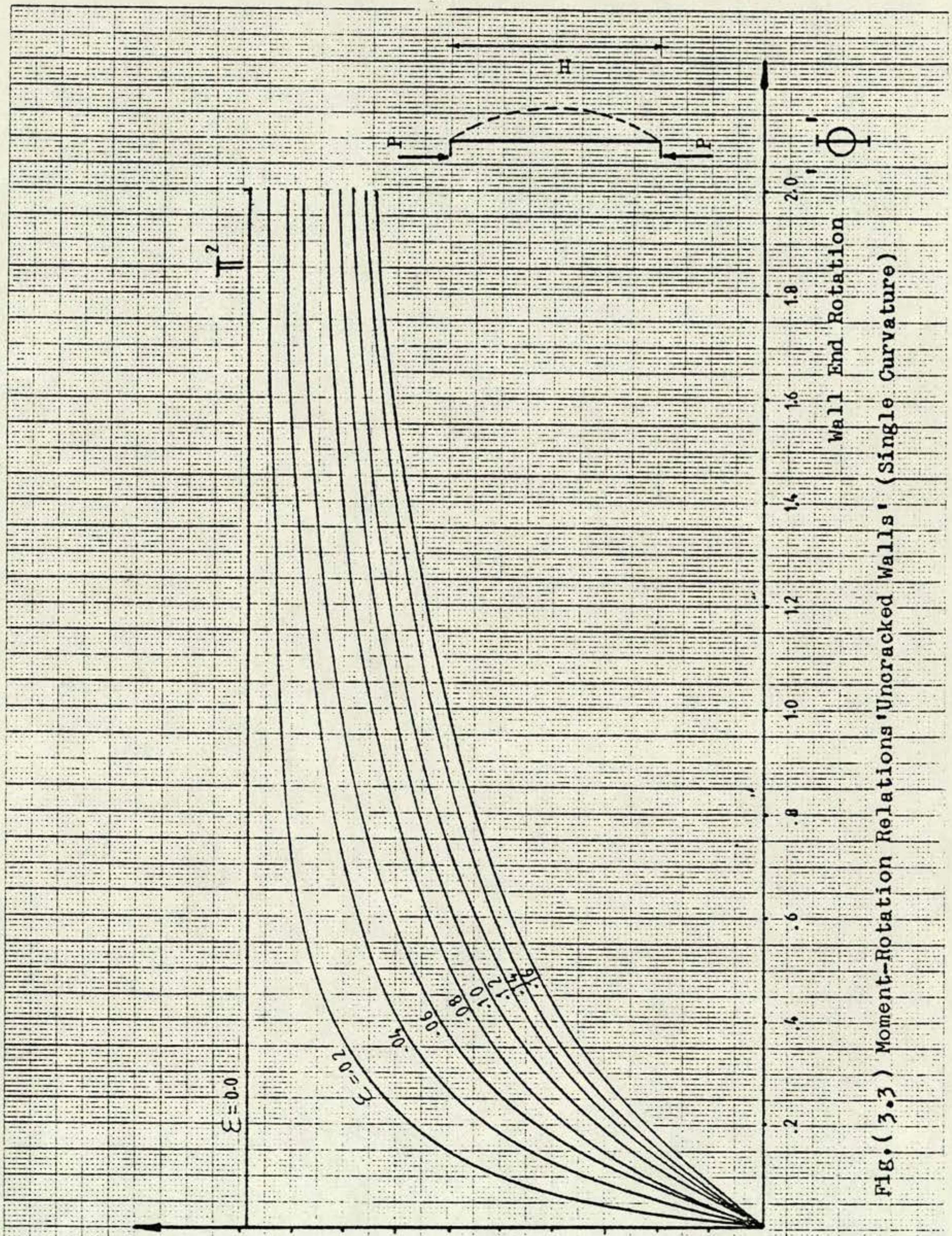

Fig.(3.3) Moment-Rotation Relations'Uncracked Walls' (Single Curvature)

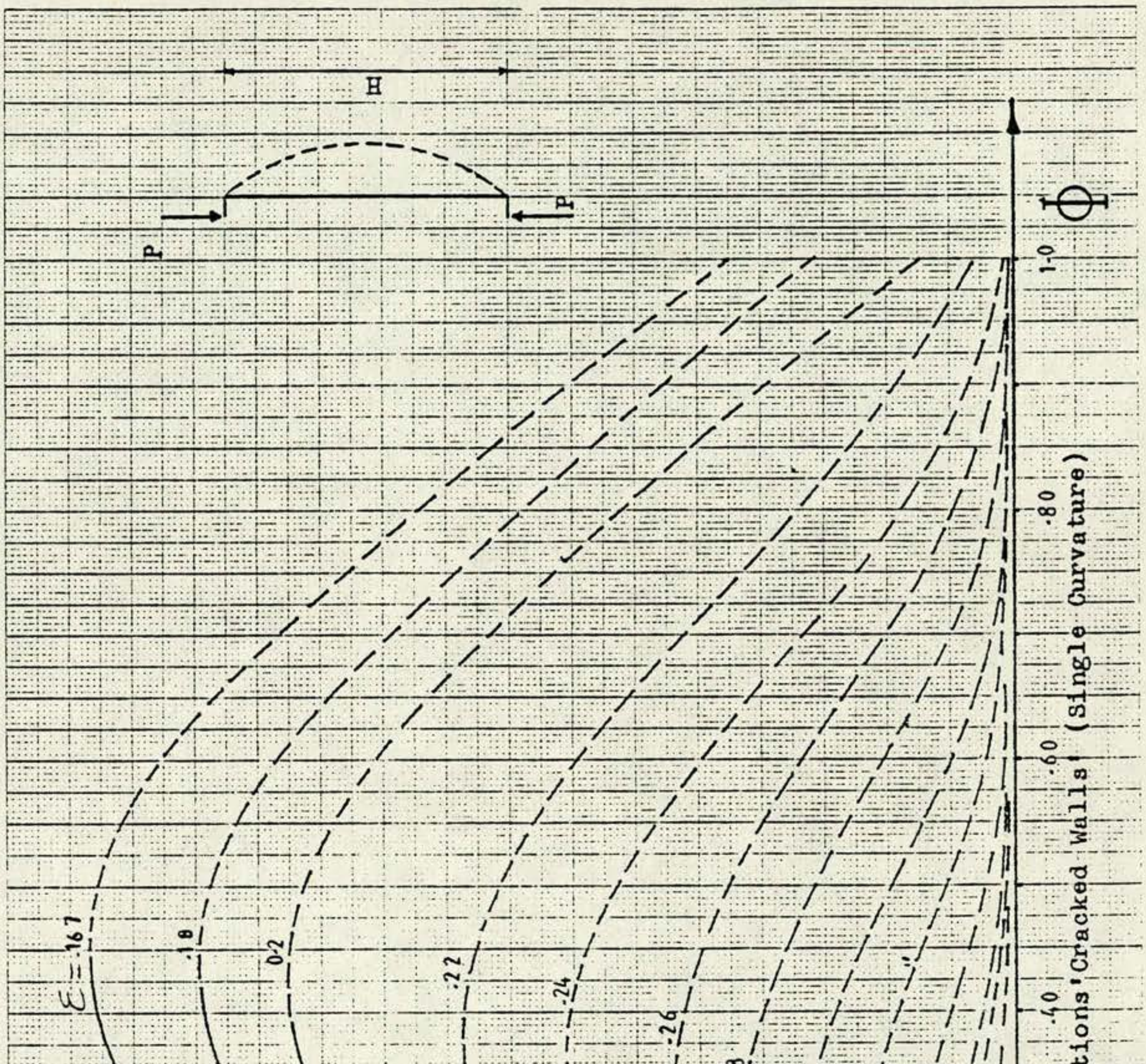

Fig.(3.4) Moment-Rotation Relations Cracked Walls (Single Curvature)

$$\Phi_b = \frac{2}{3}(1 - 2\,\varepsilon) \quad \text{.........................} \quad (3.34)$$

Substituting equation (3.34) into equation (3.33) gives:

$$\mathcal{Z}_{max} = \pi^2 \;(1 - 2\,\varepsilon)^3 \quad \text{..................} \quad (3.35)$$

Buckling loads have been computed using equation (3.35) for various values of end eccentricities, ε, and the results are given in Appendix (C). A plot of equation (3.35) is shown in Figure (3.5).

The ultimate wall end rotation, Φ_b, is presented graphically in Figure (3.6). This relationship will be used in Chapter (4) to evaluate the Parameter R, which is a factor depending on the type of wall curvature, wall eccentricity and slenderness.

3.6 WALL BEARING CAPACITY:

In evaluating the capacity reduction factors, the stress failure of the wall is assumed to occur when the end rotation of the wall is less than the rotation corresponding to buckling failure, Φ_b. For Φ values equal or greater than Φ_b, buckling failure is assumed to govern. In computing the stress failure loads, it is necessary to determine the wall end rotation, Φ. This is accomplished by equating the stress failure equation with the corresponding moment-rotation equation, hence, the interactive relation between walls and floor slabs is taken into account. Thus, from equation (3.19):

$$\beta \,.K = \mathcal{Z} \quad \text{..................................} \quad (3.36)$$

Equation (3.36) is used by substituting the appropriate expression for β and $\mathcal{Z}$ to compute the wall end rotation Φ.

3.6.1 Uncracked Wall:

For an uncracked wall in single curvature with equal end eccentricities, substituting the expression of β and $\mathcal{Z}$ from equations (3.11 and 3.26) into equation (3.36) gives:

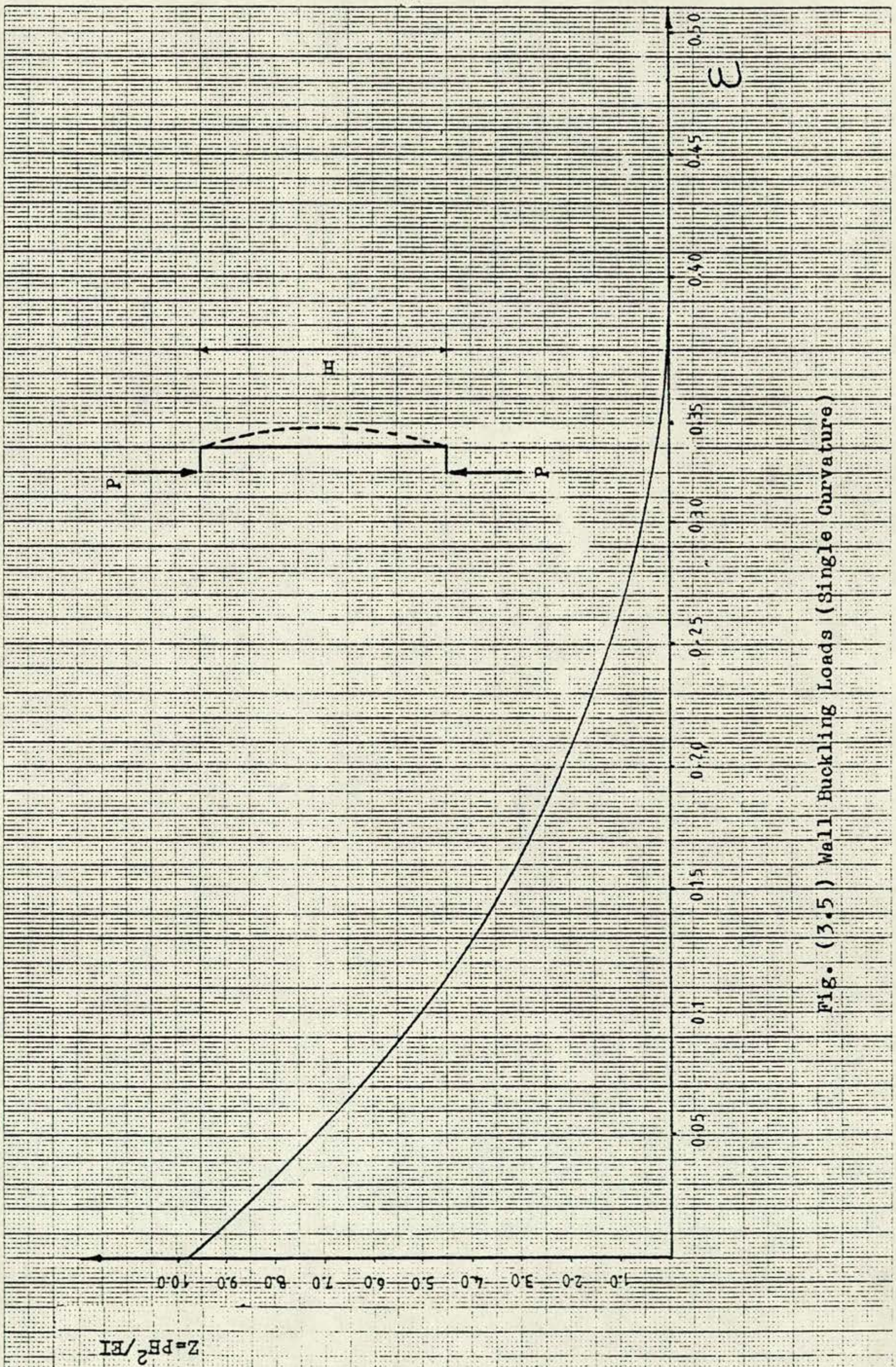

Fig. (3.5) Wall Buckling Loads (Single Curvature)

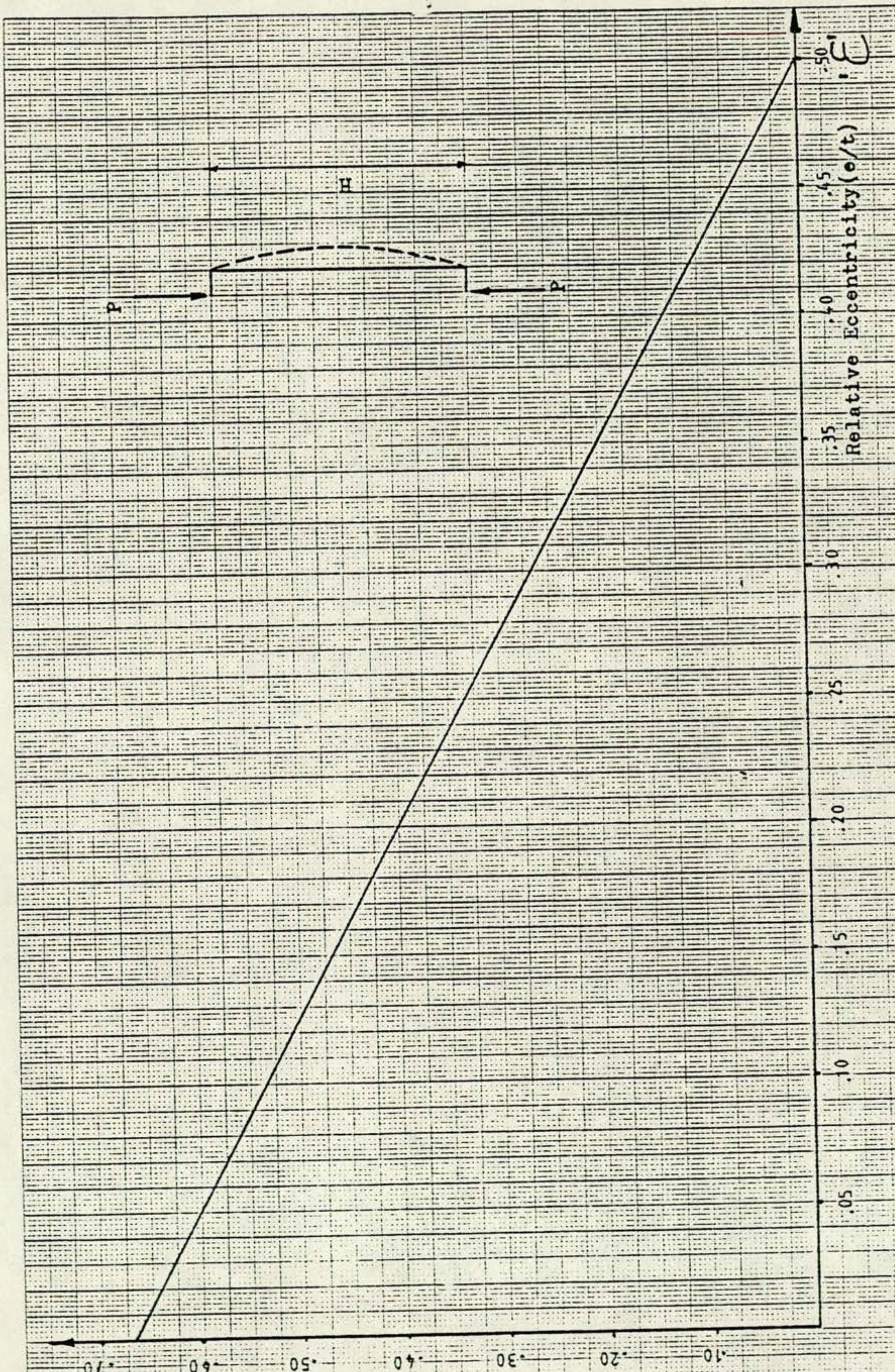

Fig. (3.6) Walls In Single Curvature

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$$\frac{3K}{2(1 + 6\varepsilon + \frac{3}{2}\Phi)} = \frac{\pi^2 \Phi}{(4\varepsilon + \Phi)} \quad \ldots\ldots\ldots\ldots\ldots \quad (3.37)$$

The solution of this quadratic equation is of the form:

$$\Phi = \frac{-B \pm \sqrt{B^2 - 4AC}}{2A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.38)$$

Where:

$A = 3\pi^2$

$B = 12\pi^2\varepsilon + 2\pi^2 - 3K$

$C = -12 K.\varepsilon$

Since K, H and t are known, for any given eccentricity ε, equation (3.38) gives the wall end rotation Φ. Once the value of Φ is known, the capacity reduction factor β, and hence the wall bearing capacity may be computed using equation (3.11) for uncracked cross-section provided that the calculated wall end rotation Φ is less than the ultimate wall end rotation corresponding to buckling, Φ_b, as defined in equation (3.34). If the calculated Φ is equal to or greater than Φ_b, then, Φ_b is substituted for Φ in equation (3.26) to compute Z_{max}, whence, the capacity reduction factor is calculated from equation (3.36), i.e.

$\beta = Z/K.$

3.6.2 Cracked Wall:

For cracked walls bent in single curvature, substituting the expressions of β and Z from equation (3.14) and(3.33) into equation (3.36) gives:

$$\frac{9K}{8}(1 - 2\varepsilon - \Phi/2) = \frac{27\pi^2}{8}\Phi(1 - 2\varepsilon - \Phi/2)^2 \ldots (3.39)$$

Equation (3.39) has the form of solution identical to equation (3.38) and the values of A, B and C are given:

$$A = 27\pi^2/16$$

$$B = \frac{27\pi^2}{4}\varepsilon - \frac{27\pi^2}{8}$$

$$C = \frac{9}{8}K$$

Again, since K, H and t are known, by following the same procedure outlined earlier and by using the appropriate equations for cracked walls in single curvature with equal end eccentricities, the capacity reduction factors and wall bearing capacities may be determined for various values of end eccentricities.

As may be noticed, the procedure of computing the capacity reduction factors is much simpler for the case of walls bent in single curvature and having equal end eccentricities than that for walls bent in double curvature. In spite of this simple procedure, computing the capacity reduction factors for various end eccentricities and wall slenderness ratios can be quite tedious. To avoid this, a computer programme was developed to calculate the capacity reduction factors for this case for both uncracked and cracked cross-section and the results are presented with the programme in Appendix (C).

As can be seen from these results, eccentricities considered ranged from zero up to 0.5, and the corresponding wall slenderness ratios ranged from 5 up to 80. Although in practice it is unlikely that wall slenderness ratios exceed 25, nonetheless these tables would provide useful information on the capacity reduction factors for high wall slenderness ratios. Part of these results are presented in Figure (3.7) to allow easy usage of these capacity reduction factors most likely to be used in

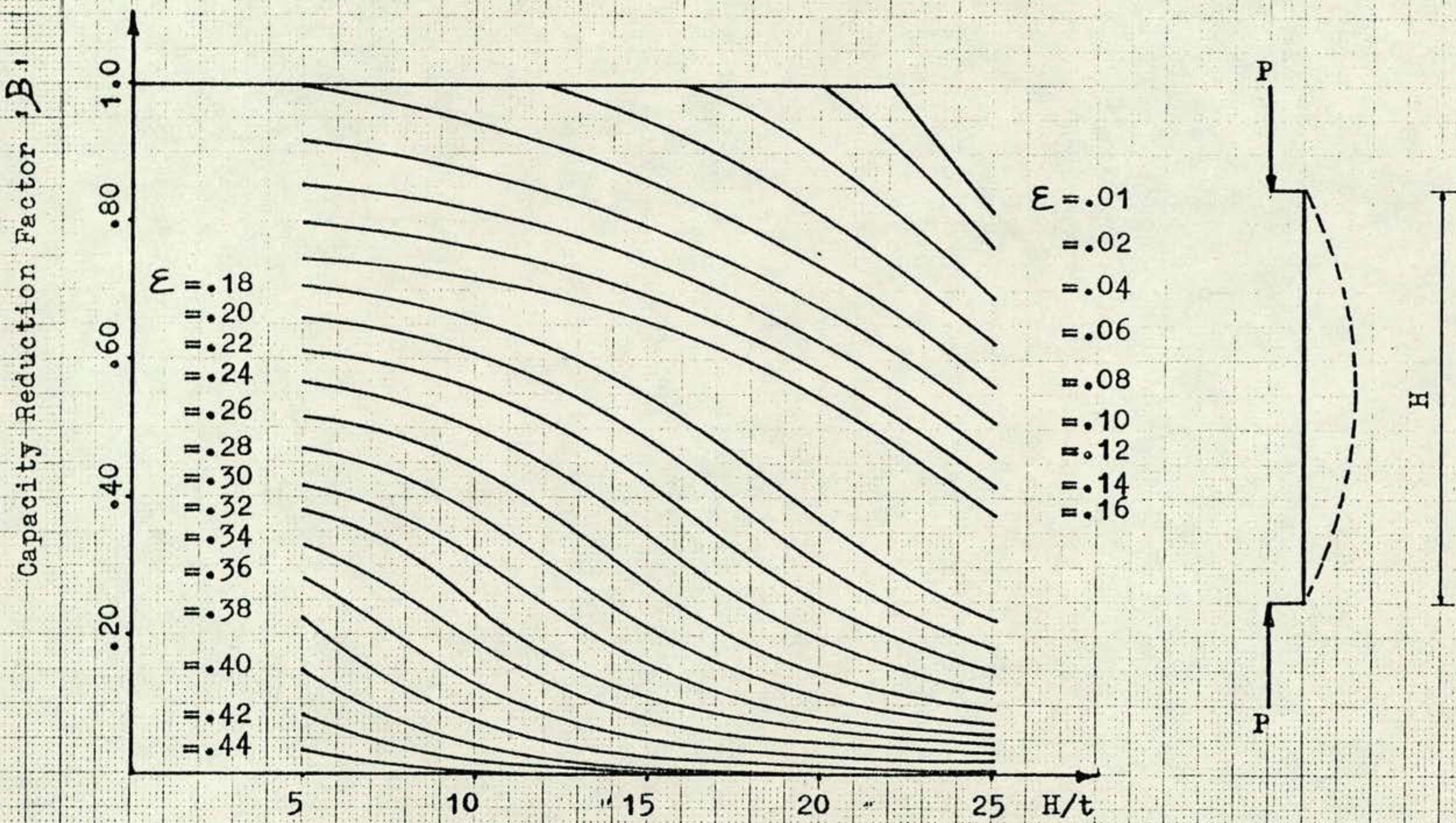


Fig.(3.7)Capacity Reduction Factor Vs. Wall Slenderness Ratio

Walls Bent In Single Curvature.

practical designs. It should be pointed out that these curves are smoother than those presented for walls in double curvature, since only one equation is used to define the wall behaviour in each of the uncracked and cracked conditions.

3.7 WALLS BENT IN SINGLE CURVATURE WITH ZERO ECCENTRICITY AT ONE END ($\varepsilon_1 / \varepsilon_2 = 0.0$)

Figure (3.8) represents a part of a multi-storey building where the walls are bent in single curvature with zero eccentricity at one end. This type of wall curvature is encountered in ground floor walls where all the top floors are bent in double curvature, and in internal walls of continuous multi-storey buildings where all floors above the level under consideration are assumed loaded and the floor at this level loaded to one side of the wall only. Fig (b) represents the left wall, which is subjected to the forces from the floor slabs equivalent to a compressive force P, with end eccentricity e, the weight of the wall is neglected and it is assumed that the horizontal components of the force P are exerted by the floor slabs. Fig (c) represents the equivalent column with the length H_s, where the full wall height is a part of the equivalent column length. The procedure of deriving the relation equations is exactly the same as for walls in double curvature discussed in Chapter (2), except that the height of the equivalent column, H_s, equals to:

$H_s = H + S/2$, hence the following relations are derived.

The deflected shape of the equivalent column may be approximated by a parabola having the equation:

$$U = 4U_o \frac{x}{H_s} \left(1 - \frac{x}{H_s}\right) \qquad (3.40)$$

At the wall/floor joint, the eccentricity is e and the inclination $\frac{du}{dx}$; thus at $x = S/2$ (joint):

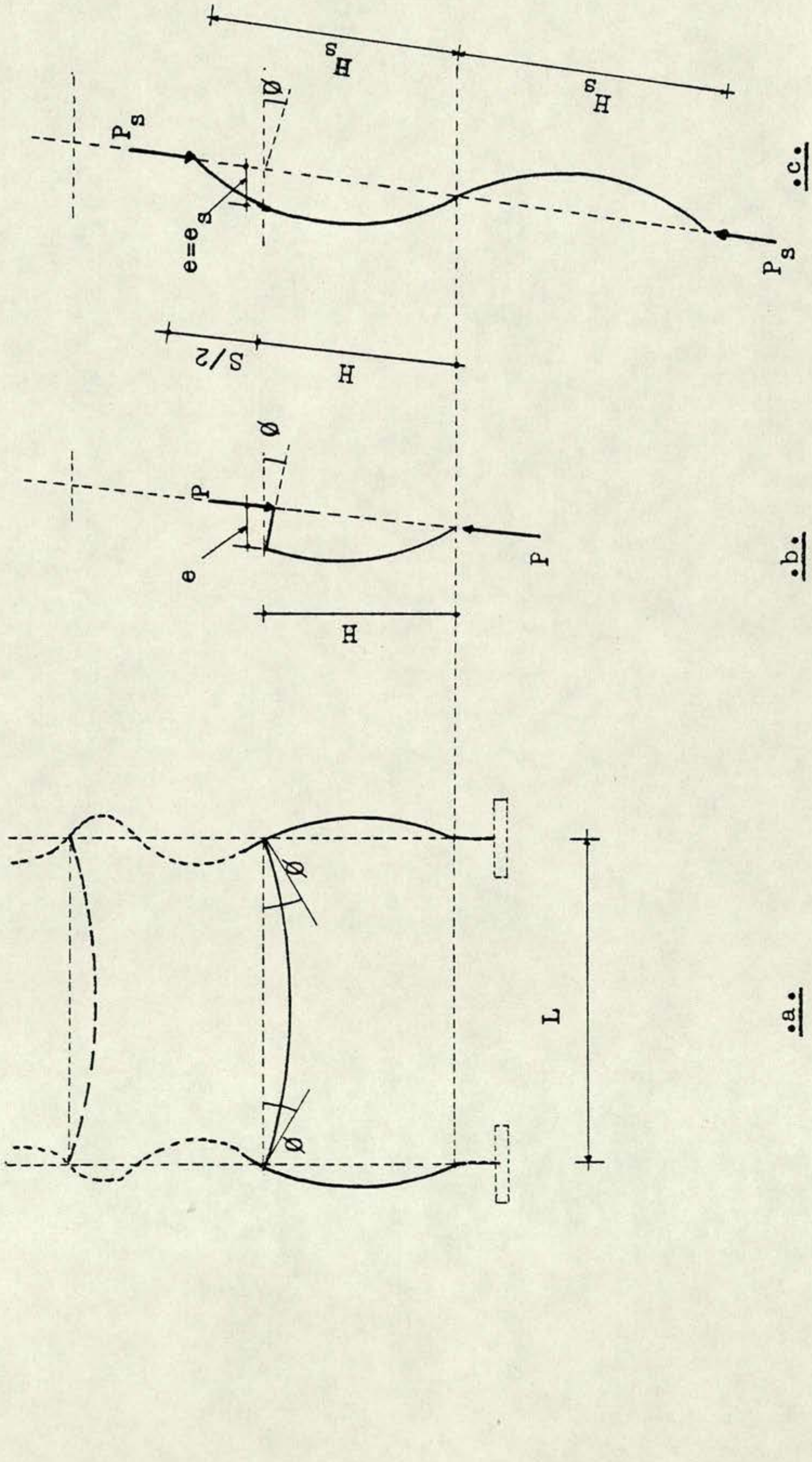


Fig.(3.8)

$$e = \frac{4\ U_o\ S}{2H_s}\left(1 - \frac{S}{2H_s}\right) \quad \text{.......................... (3.41)}$$

$$\omega = \frac{4\ U_o}{H_s}\ (1 - S/H_s) \quad \text{.......................... (3.42)}$$

Reintroducing:

$$\varepsilon = e/t \quad \text{.................................... (3.43)}$$

$$\Omega = \omega \cdot \frac{H}{t} \quad \text{.................................. (3.44)}$$

$$\Phi = \phi\ H/t \quad \text{.................................. (3.45)}$$

The following relations are derived:

$$\Omega = \frac{4U_o}{t}\ \frac{(1 - S/2H)}{(1 + S/2H)^2} \quad \text{....................... (3.46)}$$

$$\varepsilon = \frac{2U_o}{t}\ \frac{S}{H} \cdot \frac{1}{(1 + S/2H)^2} \quad \text{.................. (3.47)}$$

$$\Phi = \frac{4U_o}{t(1 + S/2H)^2} \quad \text{............................ (3.48)}$$

$$\frac{S}{H} = \frac{2\varepsilon}{\Phi} \quad \text{................................. (3.49)}$$

3.8 STRESS FAILURE EQUATION:

The stress failure equations are derived exactly in the same manner as those for walls bent in double curvature by using the above relations (eqs. 3.43 through eq. 3.49) and by noting that the full height of the wall, H, is part of the equivalent column, thus the stress failure equations are summarized for brevity:

3.8.1 Uncracked Wall:

For short walls, with slenderness ratios less than 10:

$$\beta = \frac{1.5H}{(H - 1.5\ t)(1 + 6\varepsilon)} \leqslant 1.0 \quad \text{........... (3.50)}$$

And for slender walls:

$$\beta = \frac{3}{2\left(1 + \frac{3}{2\Phi}(\Phi + \varepsilon)^2\right)} \leqslant 1.0 \quad \text{............ 3.51)}$$

3.8.2 Cracked Wall:

For cracked walls bent in single curvature with zero eccentricity at one end, the following equations are derived:

For short walls:

$$\beta = \frac{9}{8}\ (1 - 2\varepsilon)\ \frac{H}{(H - 1.5t)} \leqslant \quad 1.0 \ldots\ldots\ldots\ldots \qquad (3.52)$$

And for slender walls:

$$\beta = \frac{9}{8}\,(1 - \frac{(\Phi + \varepsilon)^2}{2\Phi}) \leqslant\ 1.0 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad (3.53)$$

As can be seen from the stress failure equations above, equations (3.51) and (3.53) are identical to equations (2.24 and 2.27) for walls bent in double curvature, smaller values of capacity reduction factors are calculated for walls bent in single curvature with zero eccentricity at one end from the application of the value of K, which is for this case given as:

$$K = (H/t)^2\ /55.5 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad (3.54)$$

Thus, the value of K, is four times greater than that for walls bent in double curvature, hence, as will be shown later, for a given (H/t) and eccentricity value, the wall end rotation Φ, is greater for walls bent in single curvature with zero eccentricity at one end than those for double curvature corresponding to the same values of H/t and wall eccentricity, ε.

As assumed in Chapter (2), the limit distinguishing short walls from slender ones is set at 10 for both single and double curvature. Buckling failure is more pronounced for walls in single curvature with zero end eccentricity than for the same wall configuration bent in double curvature. Thus by halving the slenderness ratios for walls in double curvature in the capacity reduction factor equations and using the full wall height slenderness in the case of single curvature, eventually wall buckling will occur in the latter case (i.e. single curvature) prior to that of the first case, (i.e. double curvature).

Equations (3.50) and (3.52) are identical to equations (2.21 and 2.26) for walls in double curvature except that they have been modified by assuming that compression failure of the wall cannot occur within a distance equal to 1.5 times the wall thickness away from the joint, hence, equations (3.50) and (3.52) are modified by the factor H/(H - 1.5t), i.e. by using the full wall height.

3.9 MOMENT-ROTATION EQUATIONS:

The moment rotation equations for walls bent in single curvature with zero eccentricity at one end are identical to those for walls in double curvature, except that, $\mathcal{Z}$ in this case is defined for walls in single curvature from equation (3.16) re-written:

$$\mathcal{Z} = \frac{PH^2}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.16)$$

Equation (3.16) implies that, for a given wall height and thickness, t, the compression force in the wall required to maintain equilibrium of the equivalent column, P, is four times greater for walls in double curvature than that for walls bent in single curvature with zero eccentricity at one end. It follows that by plotting $\mathcal{Z}$ as defined by equation (3.16) versus the wall end eccentricity ε, the moment rotation relations are identical for both cases, similarly, for the plot of the ultimate wall end rotation, Φ_b, versus the wall end eccentricity.

3.9.1 Uncracked wall:

For short or slender walls bent in single curvature with zero eccentricity at one end, the moment-rotation equation for uncracked walls is re-written:

$$Z = \frac{\pi^2 \Phi^2}{(\Phi + \varepsilon)^2} \leqslant \pi^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.55)$$

Equation (3.55) is presented graphically in Fig (3.9).

3.9.2 Cracked Wall:

For cracked walls bent in single curvature with zero eccentricity at one end, two cases must be considered; firstly, for short walls with slenderness ratios less than 10, and for slender walls with greater slenderness ratios, hence, for the first case with $\varepsilon > \Phi$:

$$Z = \frac{27\pi^2}{8} \Phi \quad (1 - 2\varepsilon)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.56)$$

And for the second case:

$$Z = \frac{27\pi^2}{8} \Phi \left(1 - \frac{(\Phi + \varepsilon)^2}{2\Phi}\right)^2 \quad \ldots\ldots\ldots\ldots \quad (3.57)$$

Equations (3.56) and(3.57) are presented graphically in Fig (3.10). The dotted lines represent those values of Z corresponding to the buckling mode (i.e. $\Phi > \Phi_b$.)

3.10 WALL BUCKLING LOADS:

Wall buckling loads can be obtained by setting $\frac{dZ}{d\Phi}$ = zero and computing the wall end rotation corresponding to buckling Φ_b from eq. (3.57), whence:

$$\Phi_b = \frac{(1 - \varepsilon) \pm \sqrt{4\varepsilon^2 - 2\varepsilon + 1}}{3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.58)$$

By substituting the value of Φ_b from equation (3.58) for various eccentricity values into the moment rotation equations(3.55, 3.56 and 3.57) (cracked and uncracked, depending on the value of ε) the wall buckling loads can be computed. Wall buckling loads have been computed for several values of end eccentricity,ε, and a plot of the results is given in Figure (3.11). A plot of the ultimate wall end rotation for various eccentricities is shown in Figure (3.12), and a

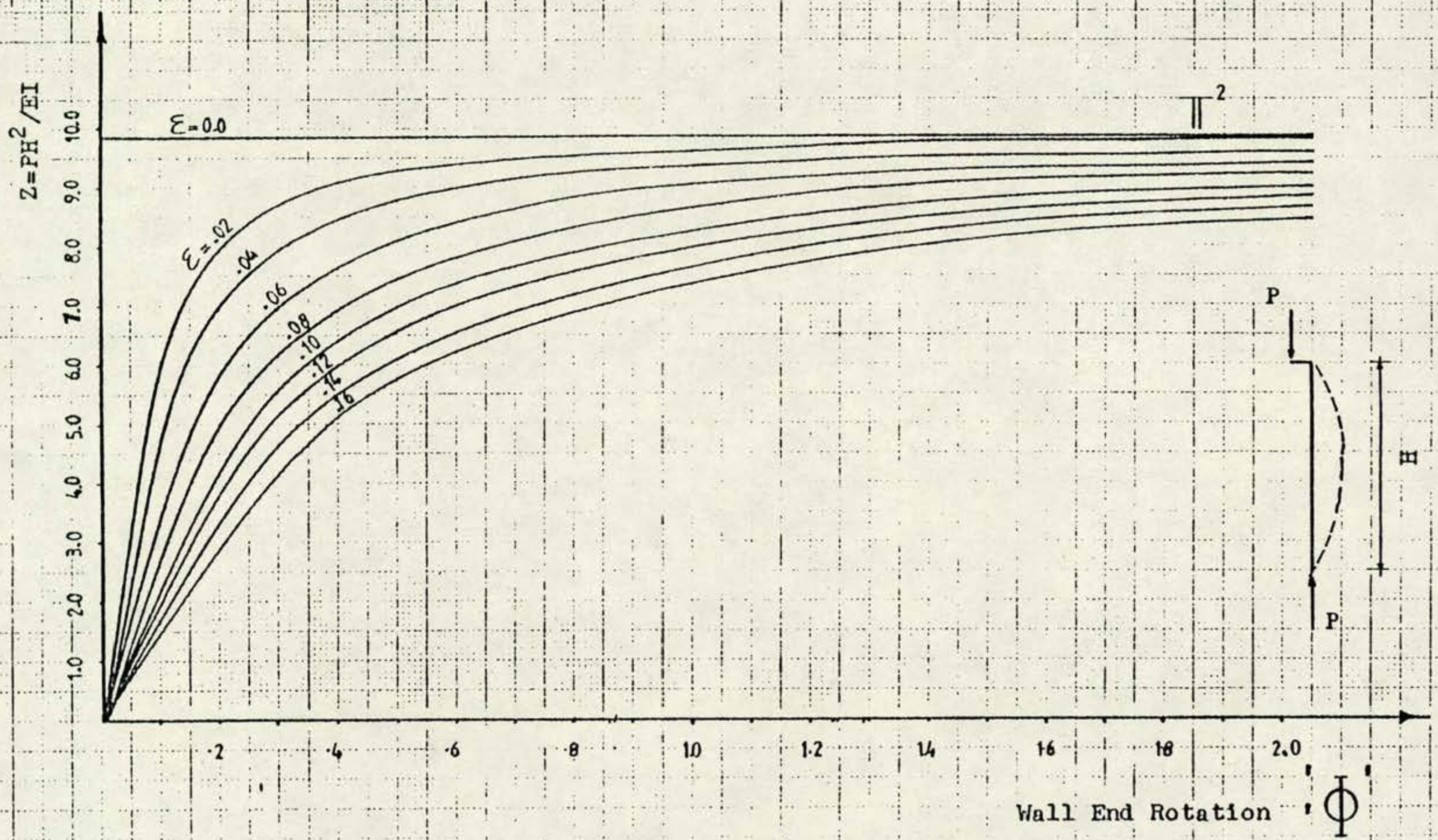


Fig.(3.9) Moment-Rotation Relations'Uncracked Walls' (Single Curvature, $\mathcal{E}_1/\mathcal{E}_2$=0.0)

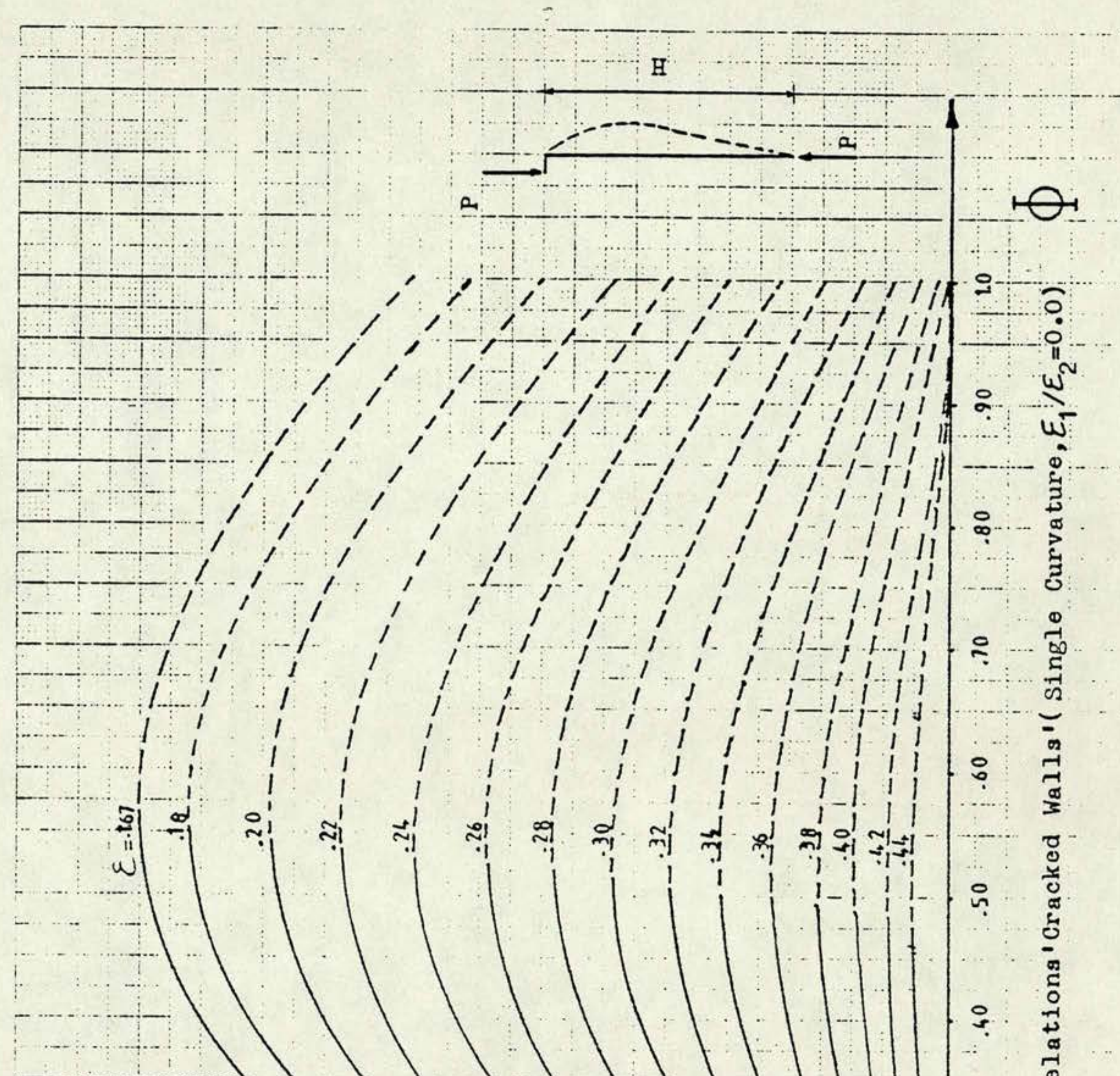


Fig.(3.10) Moment-Rotation Relations'Cracked Walls'(Single Curvature, $\mathcal{E}_1/\mathcal{E}_2$=0.0)

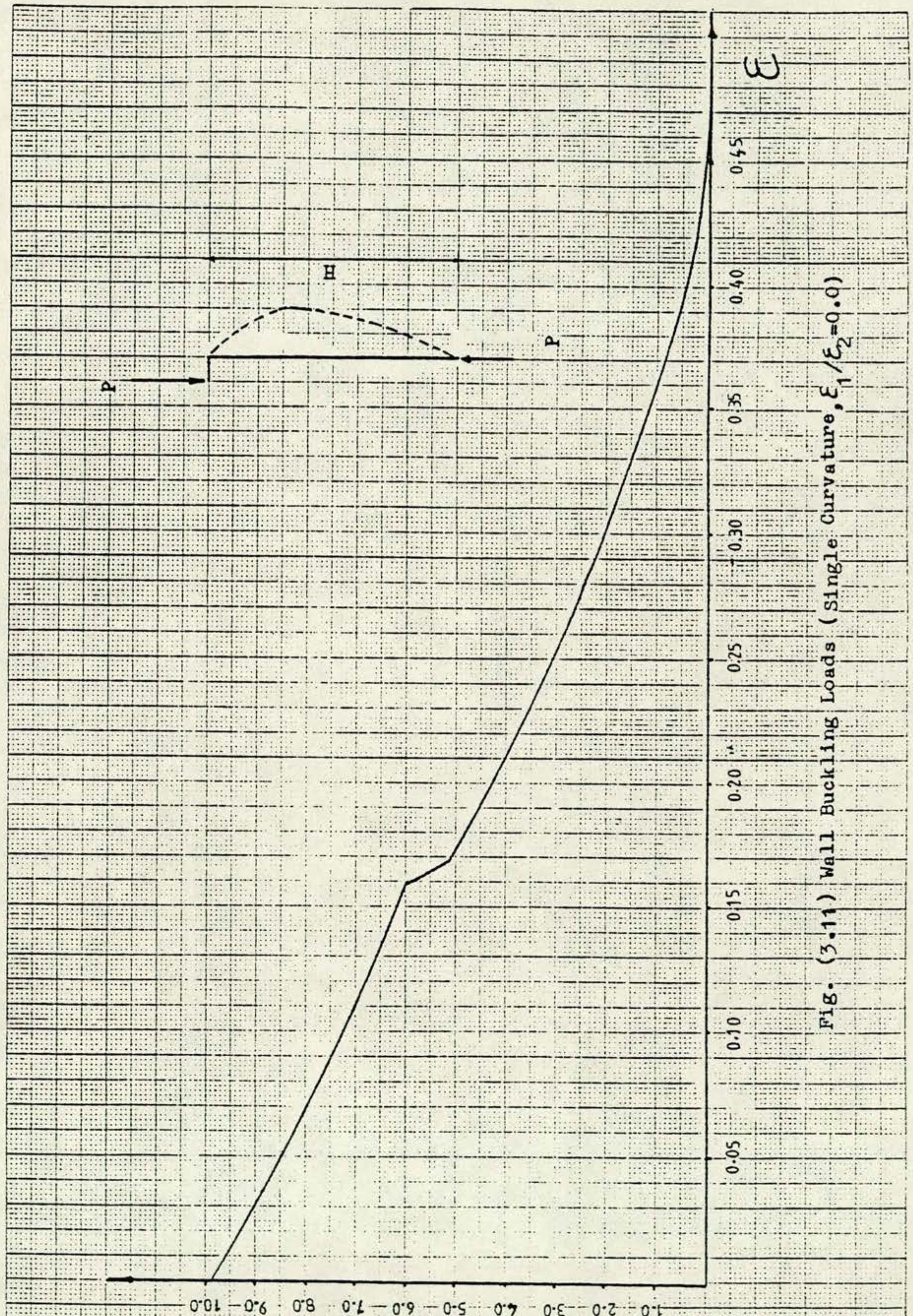


Fig. (3.11) Wall Buckling Loads (Single Curvature, $\varepsilon_1/\varepsilon_2=0.0$)

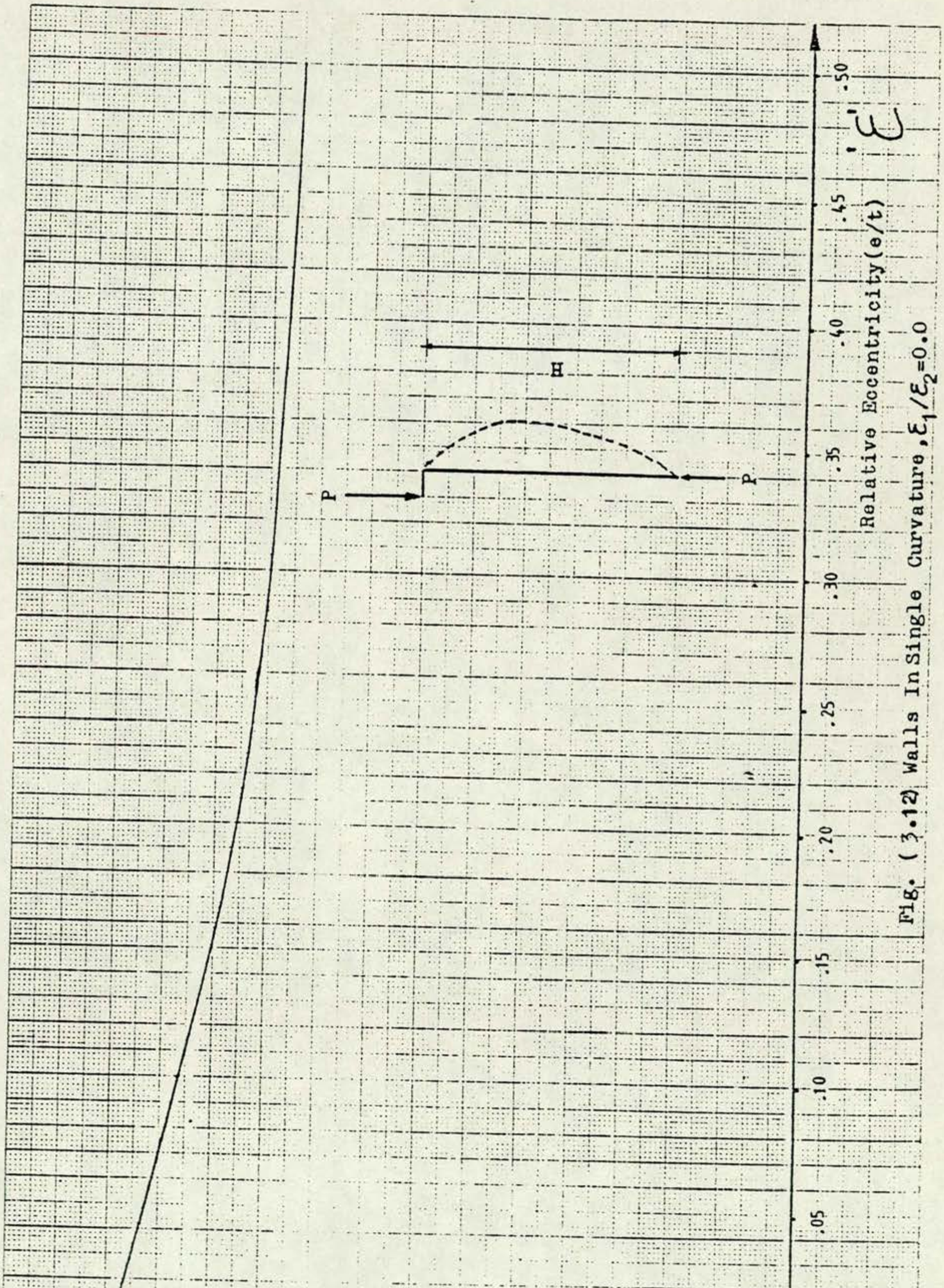


Fig. (3.12) Walls In Single Curvature, $\mathcal{E}_1/\mathcal{E}_2$=0.0

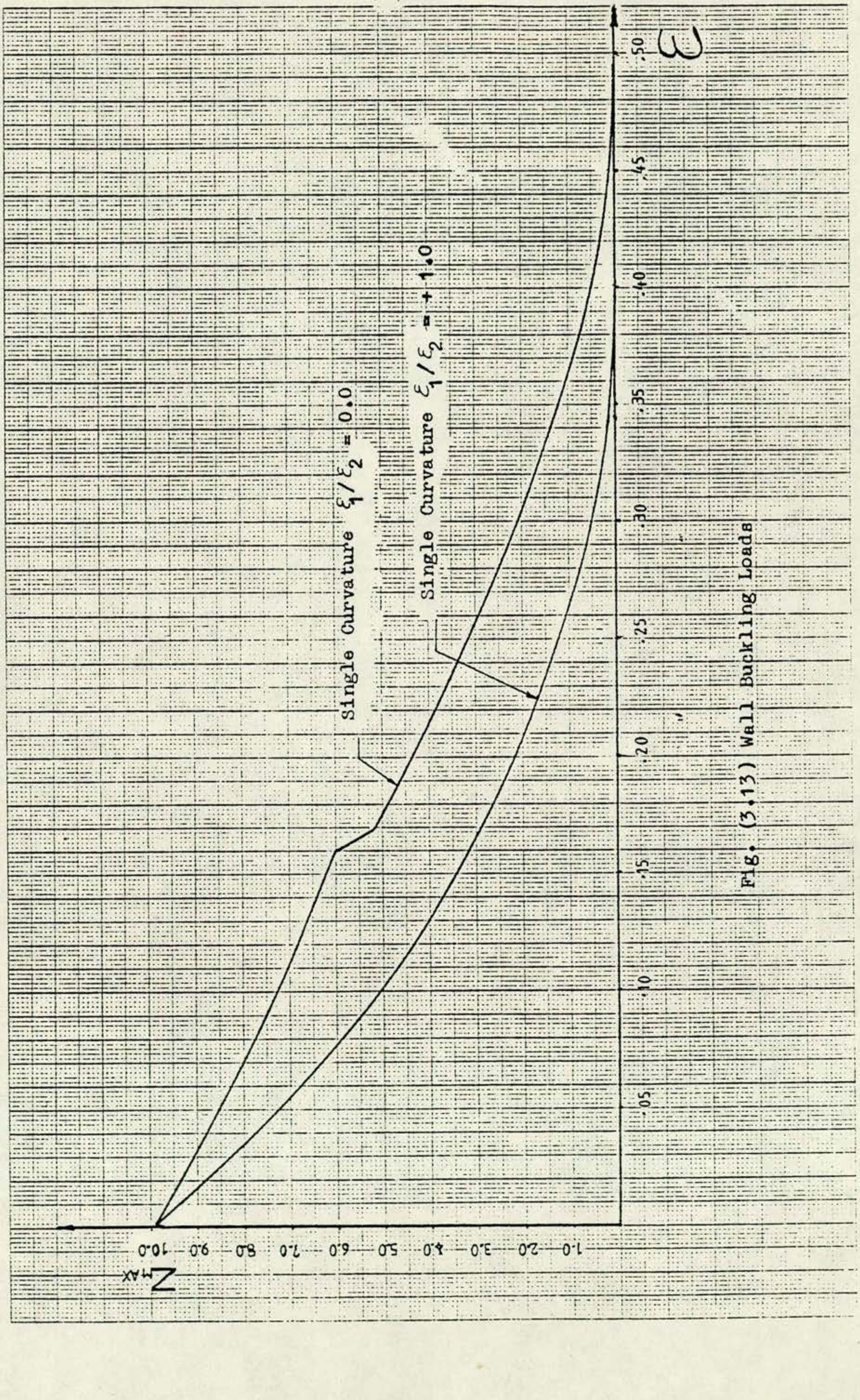


Fig. (3.13) Wall Buckling Loads

comparison of the wall buckling loads for both cases of walls bent in single curvature is shown in Fig (3.13).

3.11 WALL BEARING CAPACITY EQUATIONS:

Once again it is assumed that the stress failure of the wall will occur when the end rotation of the wall is less than the rotation corresponding to buckling failure, Φ_b. For Φ values in excess of Φ_b buckling failure is assumed to govern. In evaluating the stress failure loads, it is again necessary to determine the wall end rotation, Φ. This is achieved by equating the stress failure equations with the corresponding moment rotation equations, whence, by definition from eq. (3.36)

$$\beta . K = Z$$

once the wall end rotation is computed for known values of K, H, t and ε from the appropriate eqs., the failure load may be calculated from the stress failure equations provided that the computed Φ is less than Φ_b. If the calculated Φ is equal to or greater than Φ_b, then, Φ_b is substituted for Φ in the moment rotation equations to obtain the wall buckling loads, Z, whence, the capacity reduction factor is computed:

$$\beta = Z/K.$$

Capacity reduction factors where computed using two computer programmes for uncracked and cracked walls, these programmes together with the results are presented in Appendix (C) for various end eccentricities and wall slenderness ratios and some of these results are shown in Fig (3.14) for walls bent in single curvature with zero eccentricity at one end. The discontinuation in these curves is due to the use of different equations governing the wall behaviour in both the uncracked and cracked mode. Thus, the wall end rotation equations are summarized below:

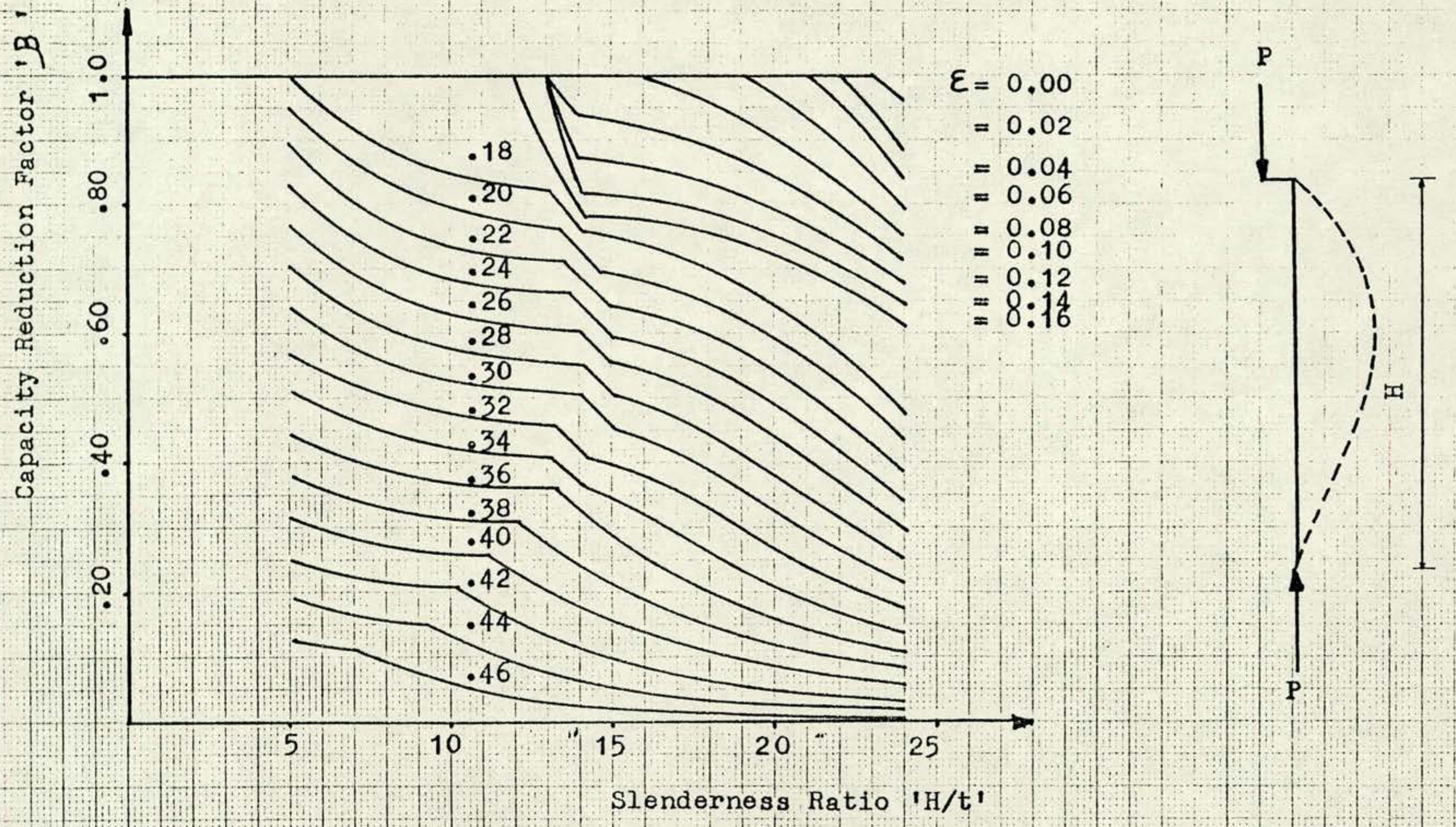


Fig.(3.14)-Capacity Reduction Factor Vs. Wall Slenderness Ratio

Walls Bent In Single Curvature,

3.11.1 Uncracked wall:

For short walls bent in single curvature with zero eccentricity at one end, the wall end rotation is given as:

$$\Phi = - B \pm \sqrt{B^2 - 4Ac} \;/2A \;\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.59)$$

where:

$$A = \frac{3}{2}(H/t)K + \frac{3}{2}\pi^2 + 9\pi^2\varepsilon - \pi^2 (H/t) - 6\pi^2\varepsilon \;(H/t)$$

$$B = 3\,(H/t).\varepsilon\,.\,K.$$

$$C = \frac{3}{2}\,K\,\varepsilon^2(H/t)$$

For slender walls, Φ is calculated from the solution of the cubic equation involved by the application of a computer programme as discussed in Section(2.3.5.1) for walls in double curvature.

3.11.2 Cracked wall:

For short cracked walls, the wall end rotation is given:

$$\Phi = \frac{K.H}{3\,\pi^2(1 - 2_{\varepsilon})(H - 1.5t)} \;\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.60)$$

And for slender walls, Φ , has the form of solution as eq. (3.59) where:

$$A = 27\,\pi^2/16$$

$$B = \frac{27\pi^2}{8}(\,\varepsilon - 1)$$

$$C = \frac{27\pi^2\varepsilon^2}{16} + \frac{9}{8}\,K$$

3.12 COMPARISONS WITH EXPERIMENTAL RESULTS:

The validity of the theoretical solution described in this chapter has been checked by comparisons with the results of a series of tests reported by the Brick Institute of America (3). These tests covered a range of slenderness ratios and eccentricity conditions corresponding to wall deflection in both cases of single curvature. Theoretical and experimental capacity reduction factors are shown in Fig (3.15) for walls bent in single curvature with equal end eccentricity, and in

Fig (3.16) for walls bent in single curvature with zero eccentricity at one end. These results are also presented in Tables (2.1) and (2.2) respectively. As may be seen from these comparisons, for both cases of walls bent in single curvature significant differences occur for H/t values in excess of 20 and large end eccentricities (i.e. $\varepsilon = 1/3$), However, since the capacity reduction factors are quite low, being less than 0.15, differences between test and theory are magnified, limiting H/t values to 20 for this case would eliminate those cases where agreement is not within acceptable limits. Thus, it may be concluded that, within the limits of experimental accuracy to be expected in this type of testing, agreement between theory and experiment may be regarded as satisfactory.

TABLE (3.1) - CAPACITY REDUCTION FACTORS - WALLS BENT IN SINGLE CURVATURE ($\varepsilon_1/\varepsilon_2$ = + 1.0)

H/t	ε	β test	β theory	% difference
3.7	1/6	0.80	0.74	8.1
6.6	"	0.70	0.73	4.3
6.7	"	0.83	0.73	13.7
14.0	"	0.56	0.63	12.5
22.7	"	0.39	0.32	21.8
30.9	"	0.19	0.17	11.7
46.1	"	0.17	0.08	112.5
3.7	1/3	0.39	0.36	8.3
6.6	"	0.31	0.32	3.2
6.7	"	0.40	0.32	25.0
14.0	"	0.13	0.10	30.0
22.7	"	0.12	0.04	200.0
22.7	"	0.17	0.04	325.0
30.9	"	0.06	0.03	100.0
46.1	"	0.03	0.01	200.0

TABLE (3.2) - CAPACITY REDUCTION FACTORS - WALLS BENT IN SINGLE CURVATURE ($\varepsilon_1/\varepsilon_2$ = 0.0)

H/t	ε	β test	β theory	% difference
3.7	.05	1.09	1.00	9.0
6.6	"	1.02	1.00	2.0
6.7	"	1.06	1.00	6.0
14.0	"	0.89	1.00	12.3
21.9	"	0.76	0.97	27.6
22.6	"	0.91	0.91	0.0
22.7	"	0.88	0.90	2.3
24.1	"	0.76	0.80	5.2
30.9	"	0.45	0.49	8.8
46.1	"	0.30	0.22	36.3
6.6	1/6	0.79	0.97	22.7
21.9	"	0.62	0.60	3.3
22.6	"	0.64	0.57	12.2
22.7	"	0.54	0.56	3.7
24.1	"	0.56	0.50	12.0
46.1	"	0.23	0.14	64.0
6.6	1/3	0.58	0.49	18.3
14.0	"	0.41	0.38	7.8
22.7	"	0.30	0.17	76.4
46.1	"	0.08	0.04	100.0

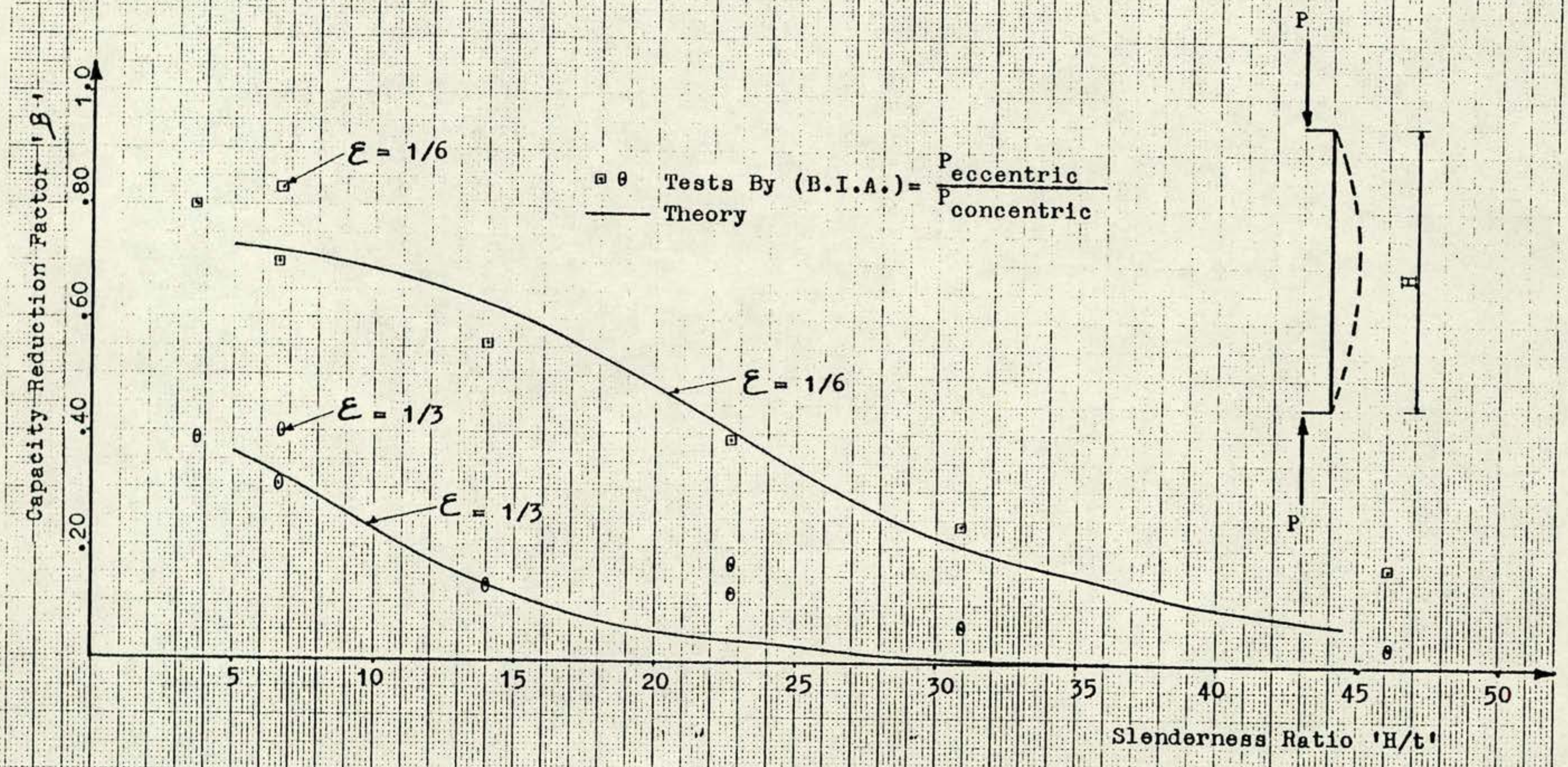


Fig. (3.15)- Comparison Of C.R.F. Between Theory And Tests By The Brick Institute Of America

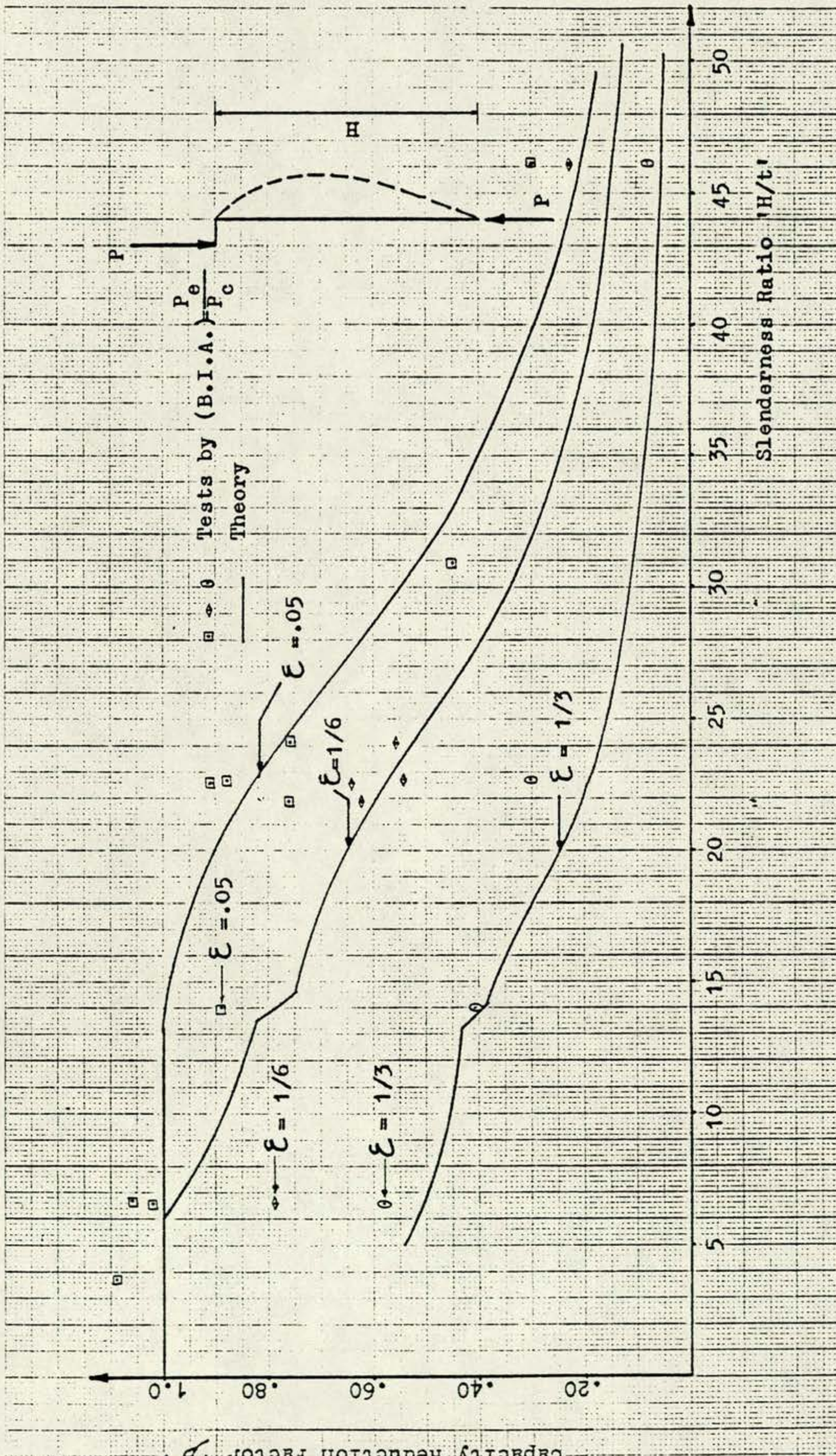


Fig.(3.16)- Comparison of C.R.F. Between Theory and Tests by The Brick Institute of America

CHAPTER 4 - JOINT ECCENTRICITIES

4.1 INTRODUCTION:

A major problem in the design of structural masonry walls is the determination of the eccentricity at a floor/wall joint. This problem is usually resolved in design on an empirical basis although a few papers have been published suggesting methods for the calculation of such eccentricities. None of these, however, has the advantage of direct calculation of the joint eccentricity, some involve tedious trial and error procedures, others involve solution of a quadratic equation or the use of complex expressions.

In this chapter, a simple method for the calculation of the joint eccentricities is developed, the joint being in any location in a single or multi-storey structure, exterior or interior, and the floor slabs may be of any type of reinforced concrete construction, one way or two way spanning, precast or cast in situ, the brickwork may be of solid or hollow construction.

To verify the theoretical analysis, the equations developed have been used to calculate the joint eccentricities and the values are compared with test results. These comparisons showed good agreement between tests and theory.

4.2 JOINT ECCENTRICITIES:

Two major problems arise in the design of structural masonry walls to resist compressive loading; firstly, is the determination of eccentricity of loading at wall/floor slab joints and, secondly, the reduction of the load bearing capacity resulting from eccentricity of loading and wall slenderness. Existing design codes treat these problems independently but in an actual structure they are not independent because of interaction between brick walls and reinforced concrete floor slabs. Thus, joint eccentricities depend on the ratio of stiffnesses of walls and floors and on the characteristics of the joint between them as well as on the wall curvature type. The load bearing capacity of a wall panel is influenced by the same factors so that both problems have to be dealt with on the basis of assumptions and analytical methods which are mutually consistent.

4.3 WALLS SUPPORTING ONE WAY SLAB SYSTEM:

As the eccentricity of the wall loads depends in addition to other factors on the rigidity of the slab and of the supporting wall as well as on the characteristics of the wall/floor joint. The load eccentricity (sometimes referred to as structural eccentricity) may be calculated by considering the continuity condition at the joint (17) thus,

$$\theta_s - \theta_j = \theta_w \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.1)$$

Where θ_s, is the net slab rotation, θ_w is the rotation of the wall end, and θ_j represents the angle of deformation in the joint, which is a measure of the joint rigidity.

Where the floor slab forms a part of a structure of the type shown in Figure (4.1) in which the wall/floor joints are rigid and the walls are bent in double or single curvature, the angle of rotation of the slab at the support is given by (17):

$$\theta_s = \frac{WL^3}{24(EI)_s} - \frac{ML}{2(EI)_s} \quad \ldots\ldots\ldots\ldots \quad (4.2)$$

Where:

$$M = P_L \cdot e_L + P_u \cdot e_u \quad \ldots\ldots\ldots\ldots \quad (4.3)$$

$$P_u = (n-1)\cdot\frac{WL}{2} + (n-1)G \quad \ldots\ldots\ldots \quad (4.4)$$

$$P_L = P_u + WL/2 \quad \ldots\ldots\ldots\ldots \quad (4.5)$$

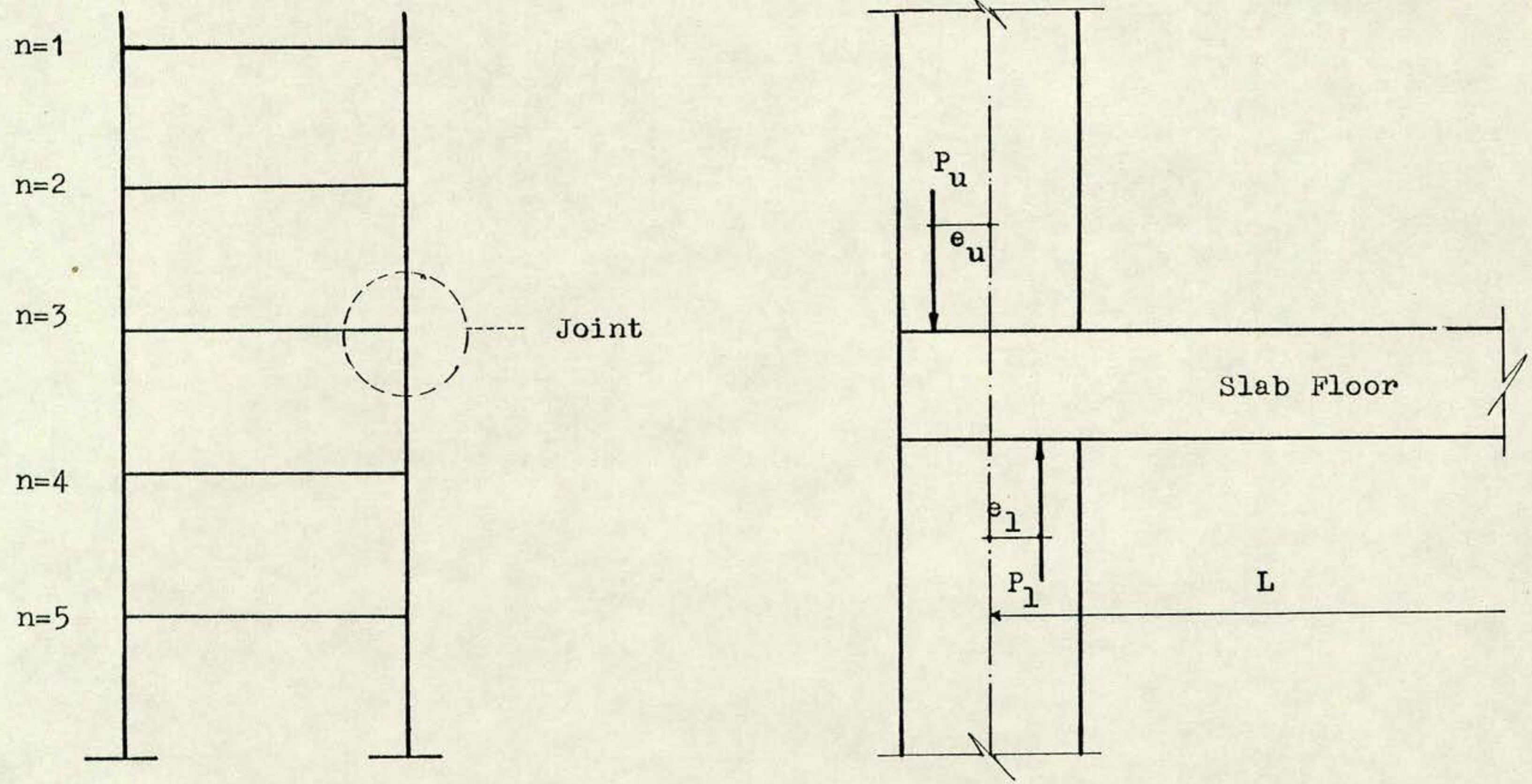


Figure (4.1)

W, is the uniformly distributed load of slab dead and live load, $(EI)_s$ is the slab flexural rigidity, G, the dead load of the wall per storey, and n is the ordinal number of a floor slab reckoned from the topmost storey of a building (i.e., Roof, n = 1).

Substituting equation (4.3) into equation (4.2) gives:

$$\theta_s = \frac{WL^3}{24(EI)_s} - \frac{(P_L \cdot e_L + P_u \cdot e_u)}{2(EI)_s} \quad \ldots\ldots\ldots \quad (4.6)$$

where e_u and e_L is the eccentricity of the compressive force immediately above and below the floor slab respectively.

Assuming the joint moment rotation relation is linear (19), the value of θ_j may be given as:

$$\theta_j = \frac{M}{\lambda} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.7)$$

Where λ is the joint stiffness, M is the joint bending moment as defined by equation (4.3). Thus, as a measure of joint stiffness, for rigid joints, λ approaches infinity, hence the joint rotation approaches zero, in the case of very flexible joints, λ approaches zero, it follows that θ_j approaches infinity, i.e. the joint is deformed and in a failure mode.

Substituting the expression for M from equation (4.3) into equation (4.7) gives:

$$\theta_j = \frac{(P_L \cdot e_L + P_u \cdot e_u)}{\lambda} \quad \ldots\ldots\ldots\ldots\ldots \quad (4.8)$$

The wall end rotation θ_w, may be expressed as a function of the curvature due to bending multiplied by the deflected length of the wall, hence, H, may either represent the full wall height for walls in single curvature, or half the wall height for walls in double curvature, whence,

$$\theta_w = \frac{M}{(\bar{E}I)_w} \cdot H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.9)$$

Since the wall end rotation θ_w is dependent on the wall curvature type, equation (4.9) is re-written to account for this parameter:

$$\theta_w = \frac{M}{(\bar{E}I)_w} \cdot \frac{H}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.10)$$

Considering the wall end rotation, θ_w, of the wall immediately below the joint under consideration where $M = P_L.e_L$, equation (4.10) finally becomes:

$$\theta_w = \frac{P_L.e_L}{(EI)_w} \cdot \frac{H}{R} \qquad (4.11)$$

Equation (4.11) defines the rotation of the wall end as a function of the wall compressive force, P_L, wall eccentricity, e_L, wall flexural rigidity, wall curvature height, and R, which is a factor depending on the joint eccentricity, wall slenderness and type of wall curvature.

Substituting equations (4.6, 4.8 and 4.11) into equation (4.1) we obtain:

$$\frac{P_L.e_L}{(EI)_w} \cdot \frac{H}{R} = \frac{WL^3}{24(EI)_s} - \frac{(P_L \cdot e_L + P_u \cdot e_u)}{2(EI)_s} - \frac{(P_u \cdot e_u + P_L \cdot e_L)}{\lambda} \qquad (4.12)$$

Assuming equal eccentricity in upper and lower walls immediately above and below the joint, hence, $e_L = e_u = e$, and introducing the following notations:

$$\psi = P_u/P_L \qquad (4.13)$$

$$\overline{\lambda} = 2(EI)_s/\lambda L \qquad (4.14)$$

$$\overline{K} = \frac{2(EI)_s}{(EI)_w} \cdot \frac{H}{L} \qquad (4.15)$$

$$\overline{\psi} = 1 + \psi \qquad (4.16)$$

$$\overline{M} = \frac{WL^2}{12} \qquad (4.17)$$

Equation (4.12) may be re-written as:

$$\varepsilon = \frac{\overline{M}\,R}{P_L t\,(\overline{\psi}\,R\,(1 + \overline{\lambda}) + \overline{K})} \qquad (4.18)$$

Assuming a rigid joint, hence $\lambda \to \infty$ it follows that $\overline{\lambda} \to$ zero and equation (4.18) reduces to the form:

$$\varepsilon = \frac{\overline{M}\,R}{P_L t\,(\overline{\psi}\,R + \overline{K})} \qquad (4.19)$$

Equation (4.19) is based on the assumption that the joint is fixed, so that it is valid in calculating joint eccentricities provided that this condition applies and the fixed end moment, $\overline{M}$, could be reached if there was no rotation of the joint as a whole (i.e. if the walls were infinitely stiff). Experiments show that the fixity of the joint in a masonry structure depends primarily on the ratio of the flexural rigidity of the floor slab to that of the supporting wall, thus, depending on this ratio, full joint fixity may never be achieved in actual structures, and consequently, the full end moment, $\overline{M}$, may never be developed. Because equation (4.19) is derived based on equation (4.2) defining the slab end rotation (17) of a simply supported slab (or beam) without including the effect of the wall (or column) stiffness on the development of actual end rotation of the slab, equation (4.19) should, therefore, be expressed as a function of the actual joint moment that would be developed depending on the degree of joint fixity and floor/wall flexural rigidities, and not on the full fixed end moment that may not be fully reached.

In the case of reinforced concrete slabs supported by brick walls, the actual joint moment that could develop depends primarily on two factors besides that due to gravity loading; firstly, on the ratio of the flexural rigidity of the floor slab to that of the wall, and, secondly, on the magnitude of the wall precompression above the floor slab under consideration, as this assumed uniform wall precompression acts as a restraining force in clamping the floor slab. Hence, the higher the wall precompression, the smaller the slab end rotation resulting in larger slab end moments (i.e. joint moment).

If, on the other hand, the magnitude of this wall precompression is small, this phenomenon will vanish and a crack will develop at the floor/wall joint due to loading on the slab. Hence, in this case the slab restraining moment will be small.

Based on this argument, and assuming that the ratio of flexural rigidity of the floor slab to that of the supporting wall is such that the rigid frame moment could be developed, the following relation is introduced:

$$M_o = \overline{M}_R \cdot \psi \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.20)$$

Whereas M_o is the actual moment at the joint (i.e. slab restraining moment), $\overline{M}_R$, is the rigid frame moment obtained by structural analysis, or may be approximated as (23):

$$\overline{M}_R = \frac{WL^2}{12} \left(\frac{3}{3 + \alpha} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.21)$$

α, is the ratio of floor slab stiffness to wall stiffness and is given as:

$$\alpha = \frac{(EI)s}{(EI)w} \cdot \frac{H}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.22)$$

Comparing equations (4.22) with equation (4.15) it follows that

$$\alpha = \frac{\overline{K}}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.23)$$

Where H, represents, half the wall height for walls bent in double curvature and the full height for walls bent in single curvature.

Equation (4.20) relates the actual moment at the joint, M_o, and the floor rigid frame moment, $\overline{M}_R$, depending on the degree of joint fixity as a measure of the total precompression at the joint, thus, when the precompression is equal to zero (i.e. $P_u = o$), as the uppermost slab in a multi-storey building, the joint moment M_o is equal to zero, simulating a hinged support where the slab is free to rotate, as the precompression increases, the value of P_u/P_L increases, and hence the slab restraining moment increases, thus when the value of ψ approaches unity, the joint becomes fully clamped, and M_o is equal to the rigid frame moment, $\overline{M}_R$.

To verify equation (4.20) theoretical values of M_o compared with test results carried out on a full scale two-storey frame (20) where the precompression varied from zero to 0.7 N/mm^2 (101 psi) and these comparisons show good agreement between equation (4.20) and test results for precompression greater than 0.3 N/mm^2 as shown in Table (4.1) but gave overestimated values of M_o for smaller precompression stress, thus, a modification of equation (4.20) is proposed which is compared with test results as will be seen in Chapter (6) and gave close agreement, this proposed modification is given as:

$$M_o = \bar{M}_R\ (\psi / \acute{\psi}) \qquad (4.24)$$

Thus, where the precompression on the joint is less than 0.3 N/mm^2 (44 psi), equation (4.24) should be used to calculate the joint moment, and equation (4.20) is used when the precompression is equal to or greater than 0.3 N/mm^2. The limit of application of equation (4.20) and (4.24) is suggested at 0.3 N/mm^2 precompression. It may be found that using the formulae at values slightly above or below this limit two values of joint moment M_o could be found; in such cases, the average of the two joint moments could be adopted.

TABLE (4.1) COMPARISON BETWEEN THEORETICAL AND EXPERIMENTAL JOINT MOMENTS M_o

Slab U.D.L. Kn/m	Wall Precomp. N/mm^2	Pu (KN)	P_L (KN)	ψ	$\bar{M}_R$* M.KN	$M_o=\bar{M}_R\psi$ ** (theory)	M_o M.KN. (test)	% diff.
2.41	0.3	54.5	58.5	0.93	3.94	3.664	3.215	+ 13.96
2.41	0.5	93.7	97.8	0.96	3.94	3.78	3.351	+ 12.80
2.41	0.7	130.7	134.8	0.97	3.94	3.87	3.54	+ 7.90
3.98	0.3	54.4	61.2	0.89	6.508	5.792	5.395	+ 7.35
3.98	0.5	93.7	100.5	0.93	6.508	6.052	5.599	+ 8.09
3.98	0.7	130.7	137.5	0.95	6.508	6.182	5.741	+ 7.68

* obtained by moment distribution for the frame

**based on $\bar{K} = \frac{2(EI)s}{(EI)_w} \cdot \frac{H}{L} = 0.736$.

The theoretical results based on the previous conclusions and the test results (20) are shown in Table (4.1). Using equation (4.20), equation (4.19) is rewritten to account for these conditions and we obtain:

$$\varepsilon = \frac{M_o R}{P_L t\,(\bar{\psi} R + \bar{K})} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.25)$$

Substituting equations (4.20) and (4.24) into equation (4.25) we finally obtain:-

$$\varepsilon = \frac{M_R . R . \psi}{P_L . t \;\; (\bar{\psi} R + \bar{K})} \;.\; (1/\bar{\psi}) \quad \ldots\ldots\ldots\ldots \quad (4.26)$$

The term $(1/\bar{\psi})$ is applicable where the precompressure stress above the joint is less than 0.3 N/mm^2. The precompression of 0.3 N/mm^2 is associated with effectively rigid joints and equation (4.26) allows for a progressive reduction in joint rigidity at floor levels above that at which this precompression is attained. As mentioned earlier, in the region of the limiting value of precompression, two values of ε will be obtained depending on whether the term $(1/\bar{\psi})$ is used or not. In such cases, the average of the two values may be adopted.

Equation (4.26) is valid irrespective of the joint location and will be used to compute the wall eccentricities at any external joint in any system where the slab is supported in one way direction. For the case of internal joints, equation (4.26) may be modified to compute the wall eccentricities in a one-way slab system where the loading and/ or adjacent spans are different so causing unbalanced moment at the joint, then, the moment $\bar{M}_R$, in the case of unequal spans is either calculated by any structural analysis method or may be approximated as:

$$\bar{M}_R = \frac{W}{12}\,(L_1^{\,2} - L_2^{\,2}\,)\,(\frac{3}{3 + \alpha}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.27)$$

And the expressions for the total compressive force in the wall above and below the joint under consideration are respectively:

$$P_u = (n - 1)\,(\,\frac{W}{2}\,(L_1 \;+ L_2\,) \;+\; G\,) \quad \ldots\ldots\ldots\ldots \quad (4.28)$$

and $$P_L = P_u + \frac{W}{2}\;(L_1 \;+\; L_2\,) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.29)$$

where L_1 is the larger slab span, and L_2 is the smaller slab span.

Considering equation (4.26), it is clear that all the parameters involved are defined by the structure configuration and loading conditions except the parameter, R, which (as mentioned earlier) is a factor depending on wall eccentricity, wall slenderness and type of wall curvature. The evaluation of the term, R, to be used in equation (4.26) is determined from the moment rotation equations presented in Chapters (2) and (3) for walls in double and single curvature respectively.

4.3.1 Walls bent in double curvature:

The relationship between the wall eccentricity, end rotation, gravity loading and the factor R is obtained by multiplying equation (4.11) by (H/2)/t, whence we obtain:

$$\frac{(H/2)}{t} \cdot \theta_w = \frac{(H/2)}{t} \cdot \frac{P_L \cdot e_L}{(EI)_w} \cdot \frac{(H/2)}{R}$$

From which:

$$\Phi = \frac{P_L H^2}{4(EI)_w} \cdot \frac{e_L}{t} \cdot \frac{1}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.30)$$

Can be obtained. Equation (4.30) may be written as:

$$\Phi = \frac{Z\,\varepsilon}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.31)$$

Or may be expressed in the following form:

$$\frac{\partial\Phi}{\partial\varepsilon} = \frac{Z}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.32)$$

4.3.1.1 Uncracked Wall:

The moment rotation relation for uncracked walls bent in double curvature is re-presented as:

$$Z = \frac{\pi^2\Phi^2}{(\Phi+\varepsilon)^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.33)$$

Substituting equation (4.31) into equation (4.33), the term R is equal to:

$$R = \pi\sqrt{Z} - Z \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.34)$$

From which it is evident that R depends on $\mathfrak{Z}$, which is a measure of the wall gravity loading.

4.3.1.2 Cracked Wall:

For cracked walls in double curvature, the moment rotation equation for short walls is:

$$\mathfrak{Z} = \frac{27\pi^2}{8} \Phi(1 - 2\varepsilon)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.35)$$

Substituting equation (4.31) into equation (4.35) and solving for R, we obtain:

$$R = \frac{27\pi^2}{8} \varepsilon (1 - 2\varepsilon)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.36)$$

For cracked slender walls, the moment - rotation equation is given as:

$$\mathfrak{Z} = \frac{27\pi^2}{8} \Phi \left(1 - \frac{(\Phi + \varepsilon)^2}{2\Phi}\right)^2 \quad \ldots\ldots\ldots\ldots \quad (4.37)$$

Substituting equation (4.31) into equation (4.37), R, would require the solution of cubic equation to be evaluated. Thus, in order to compute the joint eccentricities, ε, for walls bent in double curvature, the appropriate expression for R from equations (4.34), (4.36) and that obtained from solving equation (4.37), should be substituted into equation (4.26). By doing so, equation (4.26) would require the solution of a high power root equation which is virtually impossible to be solved by normal computations. Alternatively, by assuming that equation (4.33) is valid for uncracked and cracked walls, and from Figure (4.2), it is possible to obtain a relationship between the wall end rotation, Φ, and the wall eccentricity, ε. This relationship is shown in Figure (4.2) for walls bent in double and single curvature, hence, from Figure (4.2) for uncracked walls in double curvature:

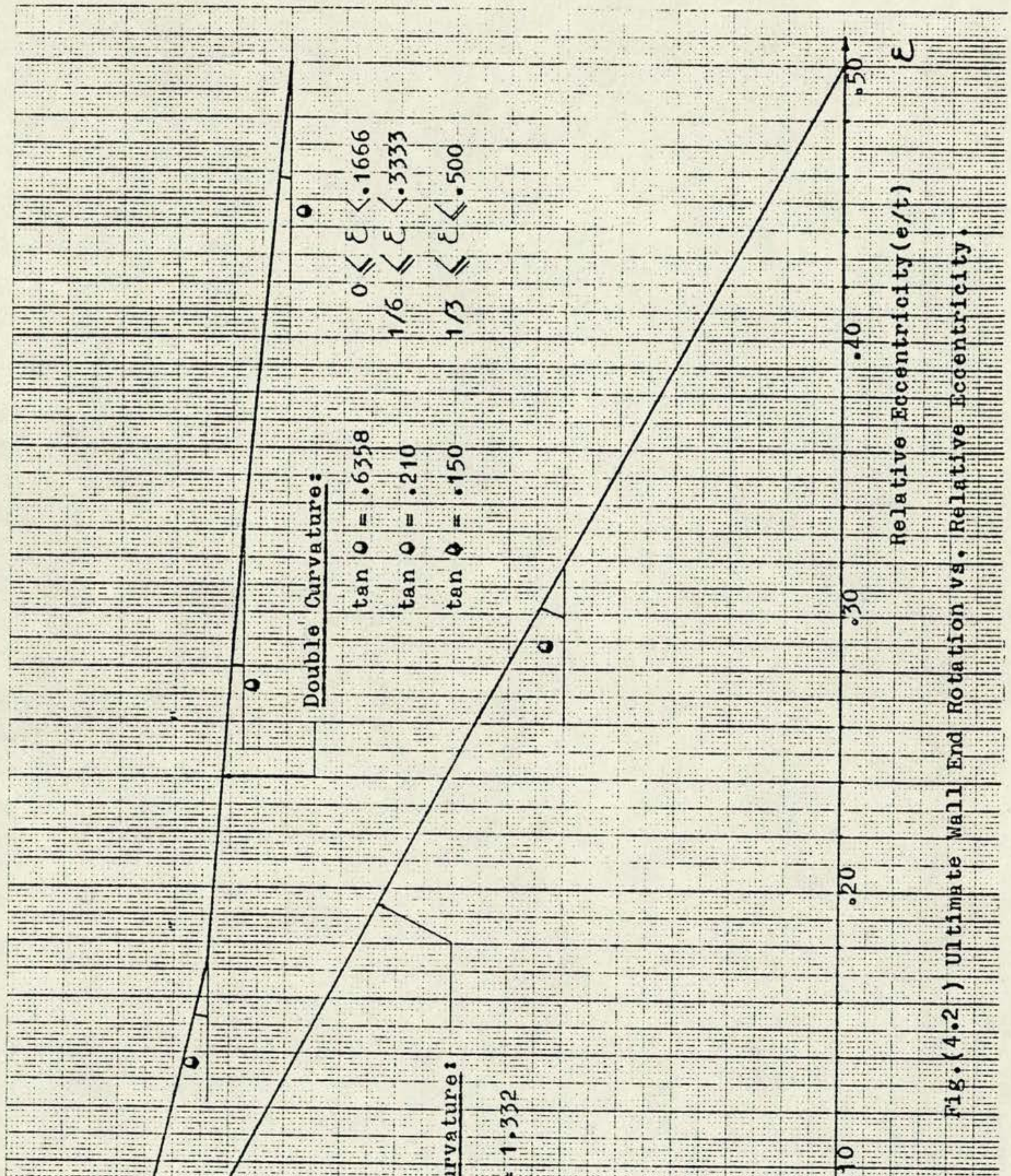


Fig.(4.2) Ultimate Wall End Rotation vs. Relative Eccentricity.

$$\frac{d\Phi}{d\varepsilon} = 0.6358 \;.....\; 0 \leqslant \varepsilon < 1/6 \quad .... \quad (4.38)$$

Substituting the value of ($d\Phi/d\varepsilon$) from equation (4.38) into equation (4.32) we obtain:

$$Z = 0.6358R \quad .... \quad (4.39)$$

Substituting equation (4.39) and (4.38) into equation (4.33) and solving for R, we obtain:

$$R = 2.345 \quad .... \quad (4.40)$$

Thus, R, constant for uncracked walls in double curvature with a value of 2.345.

For cracked walls in double curvature, the slope of ($d\Phi/d\varepsilon$) is given from Figure (4.2):

$$d\Phi/d\varepsilon = 0.210 \quad \text{for } 1/6 \leqslant \varepsilon < 1/3 \;......... \quad (4.41)$$

$$\text{And } d\Phi/d\varepsilon = 0.150 \quad \text{for } 1/3 \leqslant \varepsilon \leqslant 1/2 \;......... \quad (4.42)$$

By taking the average of equations (4.41) and (4.42), hence:

$$d\Phi/d\varepsilon = 0.18 \quad \text{for } 1/6 \leqslant \varepsilon \leqslant 1/2 \;......... \quad (4.43)$$

Thus, equation (4.43) defines the slope of ($d\Phi/d\varepsilon$) for the entire cracked zone.

As previously assumed, that equation (4.33) is valid for the cracked zone, substituting equations (4.32, 4.43) into equation (4.33) and solving for R, gives:

$$R = 1.275 \quad .... \quad (4.44)$$

Thus, the value of R as defined by equations (4.40) and (4.44) may be substituted into equation (4.26) to compute the wall eccentricity, ε, for walls bent in double curvature having equal but opposite end eccentricities.

4.4 WALLS BENT IN SINGLE CURVATURE:

In order to evaluate the parameter R for walls bent in single curvature, two types of curvature must be considered for each of the uncracked and cracked modes; firstly, for the case of walls bent in single curvature with equal end eccentricities, and secondly for walls bent in single curvature, but, with zero eccentricity at one end. In both cases, the deflected wall length equals the full wall height, H, hence multiplying equation (4.11) by H/t gives:

$$\Phi = Z\,\varepsilon/R \qquad \text{.......} \qquad (4.45)$$

Or equation (4.45) may be written as:

$$\frac{d\Phi}{d\bar{\varepsilon}} = \frac{Z}{\bar{R}} \qquad \text{.......} \qquad (4.46)$$

Equations (4.45) and (4.46) will be used to evaluate the parameter R for walls bent in single curvature.

4.4.1 Walls bent in single curvature with equal end eccentricities:

4.4.1.1 Uncracked Wall:

The moment rotation equation of uncracked wall in single curvature with equal end eccentricities is re-presented from Chapter (3) as:

$$Z = \frac{\pi^2\,\Phi}{(4\,\varepsilon + \Phi)} \qquad \text{.......} \qquad (4.47)$$

Substituting equation(4.45) into equation (4.47) and solving for R, we obtain:

$$R = \frac{\pi^2 - Z}{4} \qquad \text{.......} \qquad (4.48)$$

4.4.1.2 Cracked Wall:

The moment rotation equation for the cracked cross-section is:

$$Z = \frac{27\,\pi^2}{8}\,\Phi(1 - 2\varepsilon - \Phi/2)^2 \qquad \text{.......} \qquad (4.49)$$

Thus, by substituting equation (4.45) into equation (4.49), R may be evaluated by solving the cubic equation obtained.

By a similar approach to that adopted for walls bent in double curvature, and by assuming that equation (4.47) is valid for cracked and uncracked walls, from Figure (4.2), $d\Phi/d\varepsilon$ has a constant slope in the uncracked and cracked zones, whence,

$$\frac{d\Phi}{d\varepsilon} = 1.332 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.50)$$

Substituting equation (4.50) into equation (4.46) we obtain:

$$z = 1.332\ R \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.51)$$

Substituting equations (4.50, 4.51) into equation (4.47) and solving for R, gives:

$$R = 1.85 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.52)$$

Thus, the value of $R = 1.85$ could be substituted into the eccentricity equation (4.26) to obtain the joint eccentricity for uncracked and cracked walls bent in single curvature having equal end eccentricities.

4.4.2 Walls bent in single curvature with zero eccentricity at one end:

For walls bent in single curvature having zero eccentricity at one end, the moment-rotation equation for the uncracked and cracked modes are exactly identical to those for walls bent in double curvature. The ultimate wall end rotation corresponding to various values of end eccentricities is also defined by the double curvature slope of $(d\Phi/d\varepsilon)$ of Figure (4.2), hence, the values of R may be taken as those for double curvature. However, since the value of $\overline{K}$, for walls in single curvature as defined by equation (4.15) is twice that for the case of double curvature, to avoid further complication it is assumed that the value of R for walls bent in single curvature with zero eccentricity at one end is taken as defined by equation (4.52) for walls bent in single curvature with equal end eccentricities. This assumption does not alter the computed eccentricities values within unacceptable limits, but simplify the application of using equation (4.26) by substiting R = 1.85 for all cases of uncracked and cracked walls bent in single curvature.

4.5 WALLS SUPPORTING TWO WAY SLAB SYSTEMS:

In the case of brick masonry walls supporting two way slab systems, assuming uniform distribution of floor slab loading at the supporting brick walls (24), it is possible to compute the joint eccentricities provided that the joint moment M_o, calculated based on plate anology, is used in equation (4.26) and the corresponding maximum wall compressive force above and below the joint are evaluated to account for such load distribution.

4.5.1 External Joints

By a similar approach to that of Section (4.3) the following expression for the maximum wall compressive force above and below the joint respectively are derived assuming a uniformly distributed loading at the supporting walls, whence, for walls supporting the short span of the floor slab:

$$P_u = (n - 1)\frac{WL}{3} + (n - 1)\,G \qquad (4.53)$$

and

$$P_L = P_u + \frac{WL}{3} \qquad (4.54)$$

And for walls supporting the long span of the slab:

$$P_u = (n - 1)\frac{WL}{3} \cdot \frac{(3 - m^2)}{2} + (n - 1)\,G \qquad (4.55)$$

and $$P_L = P_u + \frac{WL}{3} \quad \frac{(3 - m^2)}{2} \qquad (4.56)$$

where m is the ratio of short to long span of the floor slab under consideration.

The joint eccentricity ε, may be given as:

$$\varepsilon = \frac{\bar{M}_R . R\ \psi\ (C_s/C_f)}{P_L . t\ (\bar{\psi}R + \bar{K}\ )}\ (1/\bar{\psi})\ \ldots\ldots\ldots\ldots\ldots\ldots \quad (4.57)$$

Where C_s is a factor depending on the slab configuration and type of support, and the value of C_s(24) are presented in Table (4.2). C_f is the coefficient 1/12 constant since the equations were derived based on the full rigid frame moment of a one way slab supported system.

The term $(1/\bar{\psi})$ as mentioned before applies only when the precompression above the joint is less than 0.3 N/mm^2.

The magnitude of $\bar{M}_R$ to be used in equation (4.57) should be the bending moment obtained by structural analysis or may be approximated as given by equation (4.21), and R is as defined by equations (4.40), (4.44) and (4.52).

4.5.2 Internal Joints:-

For internal joints, equation (4.57) may be used to calculate the joint eccentricity provided that the bending moment $\bar{M}_R$ is taken as:

$$\bar{M}_R = \frac{W}{12}\ (L_1^2 - L_2^2)\ \frac{3}{3 + \alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.58)$$

Or $\bar{M}_R$ may be calculated by any structural analysis method.

And the expressions for P_u and P_L, for walls supporting the shorter and longer spans respectively are:

$$P_u = (n - 1)\left(\frac{W}{3}\ (L_1 + L_2\ ) + G\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (4.59)$$

$$P_L = P_u + \frac{W}{3}\ (L_1 + L_2\ ) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.60)$$

And:

$$P_u = (n - 1)\left(\frac{WL_1}{3} . \frac{(3 - m_1^2)}{2} + \frac{WL_2}{3} . \frac{(3 - m_2^2)}{2} + G\right) \ldots (4.61)$$

$$P_L = P_u + \left(\frac{WL_1}{3} . \frac{(3 - m_1^2)}{2} + \frac{WL_2}{3} . \frac{(3 - m_2^2)}{2}\right)$$

$$\ldots\ldots\ldots\ldots\ldots \quad (4.62)$$

TABLE (4.2) MOMENT COEFFICIENTS (C_s)*

Moments	Short span values of m**						Long span all values of m
	1.0	0.9	0.8	0.7	0.6	0.5	
CASE 1 Interior Panels: negative moment at: continuous edge discontinuous edge	 .033 -	 .040 -	 .048 -	 .055 -	 .063 -	 .083 -	 .033 -
CASE 2 One edge discontinuous negative moment at: continuous edge discontinuous edge	 .041 .021	 .048 .024	 .055 .027	 .062 .031	 .069 .035	 .085 .042	 .041 .021
CASE 3 Two edge discontinuous negative moment at: continuous edge discontinuous edge	 .049 .025	 .057 .028	 .064 .032	 .071 .036	 .078 .039	 .090 .045	 .049 .025
CASE 4 Three edge discontinuous negative moment at: continuous edge discontinuous edge	 .058 .029	 .066 .033	 .074 .037	 .082 .041	 .090 .045	 .098 .049	 .058 .029
CASE 5 Four edge discontinuous negative moment at: continuous edge discontinuous edge	 - .033	 - .038	 - .043	 - .047	 - .053	 - .055	 - .033

* From ACI Standard Building Code 318 - 63.

** Interpolate for intermediate values.

Where m_1, is the ratio of shorter side to longer side of the longer slab at one side of the joint, and m_2, is the ratio of shorter to the longer side of the shorter slab at the other side of the joint.

4.6 DETERMINATION OF WALL ECCENTRICITIES AT FAILURE:

The basic relation for computing wall eccentricities is given in equations (4.26) and (4.57) where R is defined by equations (4.40 and 4.44) for walls bent in double curvature and equation (4.52) for walls bent in single curvature. However, it should be pointed out that in all cases when computing the eccentricities at the joint using one of the above equations, these calculated eccentricities must not be greater than the failure eccentricity (19) which is the maximum limiting value of the joint eccentricity, $\varepsilon_{failure}$, defined as:

$$\varepsilon_f = \tfrac{1}{2}(1 - \eta) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.63)$$

Where ε_f is the eccentricity corresponding to joint failure and η is the ratio of the axial compressive force in the wall to the wall compressive strength.

Failure at the joint may also occur if the ultimate negative moment capacity of the slab is attained, thus, the value of $\bar{M}_R$ to be used in the eccentricity equations must be the smaller of the rigid frame moment or the ultimate negative moment capacity of the slab.

In computing the eccentricity at the joints where the slabs are supported by cavity walls, the effective wall thickness could be taken as 2/3 the sum of the actual thicknesses of the two leaves (25).

4.7 COMPARISONS BETWEEN THEORY AND TEST RESULTS:

Little experimental information is available for joint eccentricities in structures supporting one way slab systems and none for structures supporting two way slab systems. Tests carried out on full scale two storey frame (20) where the precompression varied from zero to 0.7 N/mm^2 are compared with theoretical computations for external joint eccentricities in this chapter and is presented in Table (4.3). These comparisons show good agreement between tests and theory. The theoretical equations derived for joint eccentricities are also compared with experimental results carried out on a single-bay, two-storey, half scale model, and a three-storey, two-bay, full size structure, having different floor/wall flexural rigidity ratio and will be discussed in Chapter 6.

TABLE (4.3) COMPARISON BETWEEN THEORETICAL AND EXPERIMENTAL JOINT ECCENTRICITIES

Slab U.D.L. (Kn/m)	Wall Precomp. N/mm^2	Pu (Kn)	P_L (Kn)	ψ	ε_{test}*	ε_{theory}**	Test/theory % difference
2.41	0.3	54.4	58.5	.93	.134	0.131	2.30
2.41	0.5	93.7	97.8	.96	.083	0.08	3.75
2.41	0.7	130.7	134.8	.97	.063	0.0585	7.70
3.98	0.3	54.4	61.2	.89	0.22	0.180	22.22
3.98	0.5	93.7	100.5	.93	.136	0.1265	7.50
3.98	0.7	130.7	137.5	.95	.101	0.0936	7.90

** Based on $\overline{K}$ = 0.736

* Test is obtained by dividing M_o (from tests) by the sum of the total force at the joint (i.e. $\varepsilon_{test} = M_o/(P_u + P_L).t$

CHAPTER 5 - DESCRIPTION OF TEST PROGRAMME

5.1 INTRODUCTION:

To verify the validity of the eccentricity equations developed in Chapter 4, two major test programmes were carried out to obtain sufficient results to allow comparisons between theory and tests.

Since it is generally accepted that the clamping action of wall precompression significantly affects the rigidity of floor/wall joints, and hence the development of wall moments, a series of tests were performed on a single bay, two-storey half scale frame structure and on a two bay, three storey, full size structure. The ratio of flexural rigidities of the floors and walls differed considerably in the two structures in order to study the effect of this variable on the relationship between the wall precompression and the joint rigidity. The major variables in the tests were the magnitude of wall precompression, magnitude of floor live load and sequence of loading the walls and floors.

Different testing methods were used in each case as the size of the test structures influenced the choice of testing equipments and loading frames. The test objectives were to determine the following:

1. Joint moments M_O, corresponding to various floor slab loading and wall precompression.
2. Joint eccentricities ε, developed due to floor slab loading at various magnitudes of wall precompression.
3. Wall end rotations, wall deflections and strain corresponding to various values of floor slab loading and wall precompression.

5.2 HALF SCALE MODEL STRUCTURE:

5.2.1 Description of Test structure:

The general arrangement of the half scale test structure along with pertinent overall dimensions is shown in Figure (5.1). Each wall consisted of 34 courses laid on alternate stretcher and header bond.

Thus, the wall thickness was equal to the length of one half scale brick, an average of 112 mm. The lower course of each wall was laid in stretcher bond, so that the upper course, located below each of the two floor slabs was a header course. The clear height of the wall was 1230 mm for each storey height and the average width was 464 mm throughout.

The reinforced concrete floor slabs were cast in the laboratory and were allowed to cure for 28 days prior to placement on the walls. The average slab thickness was 90 mm and the width 468 mm throughout. The slabs length was 2400 mm, thus the average clear and c/c spans were 2178 mm and 2290 mm respectively.

5.2.2 Material properties and construction procedure:

As mentioned earlier, each wall consisted of 34 courses of half scale bricks laid in alternate stretcher and header bond. The average compressive strength of the bricks, with the frogs filled with 1:1 cement/sand, was between 30.36 N/mm^2 and 35.59 N/mm^2. The mortar was 1:½:4½ (cement:lime:sand) proportioned by volume with average compressive strength between 11.14 N/mm^2 and 11.81 N/mm^2 tested after 28 days water curing.

The reinforced concrete slabs were cast indoors using 1:1.8:3.2 mix by weight with a maximum aggregate size of 16 mm, and were compacted by hand rodding and a poker vibrator. The tops of the slabs were levelled with a wood strike board and were allowed to cure for 28 days

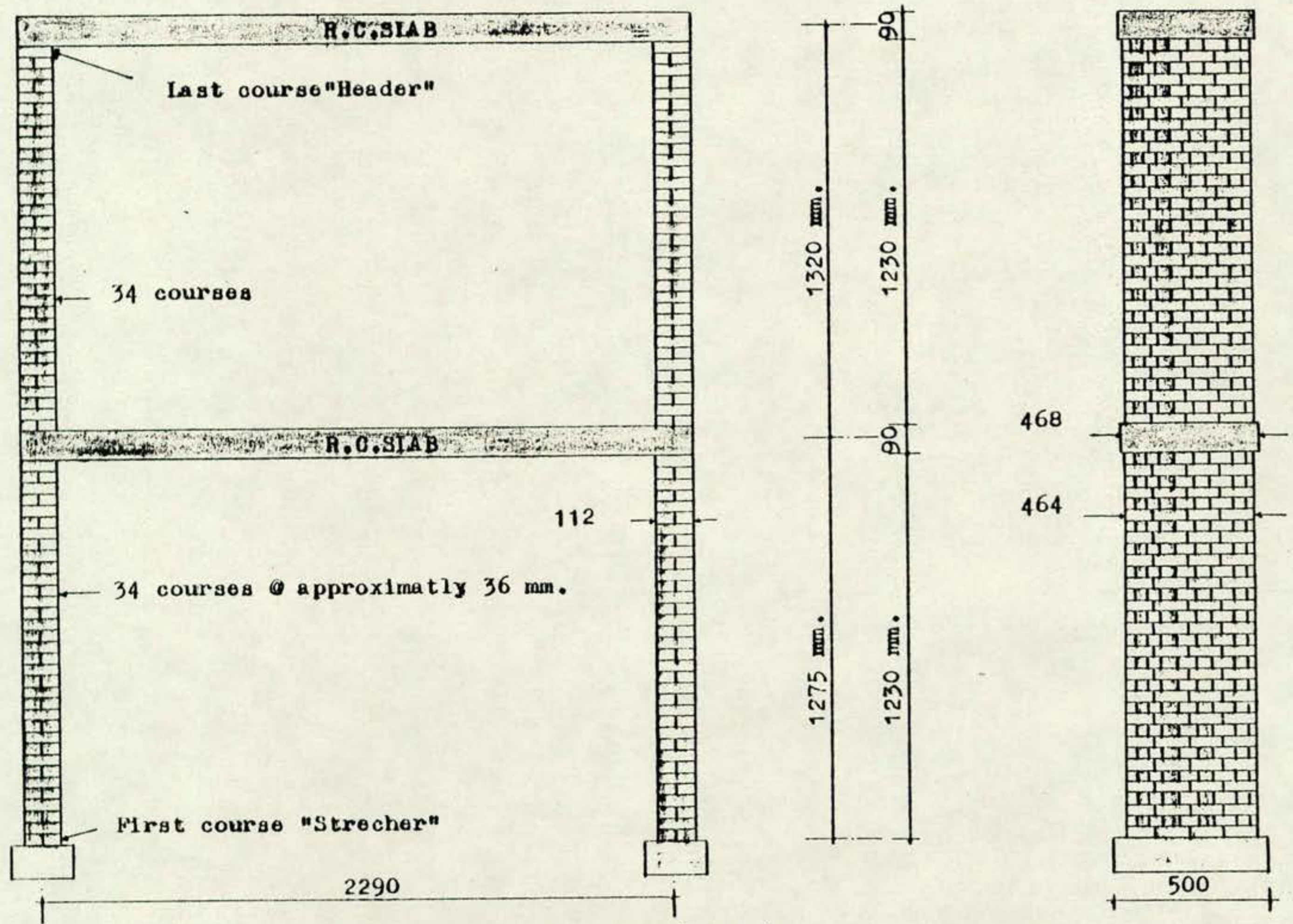

R.C.SIAB
Last course"Header"
34 courses
R.C.SIAB
112
34 courses @ approximatly 36 mm.
First course "Strecher"
2290
1320 mm.
1275 mm.
90
1230 mm.
90
1230 mm.
468
464
500

Fig.(5.1)

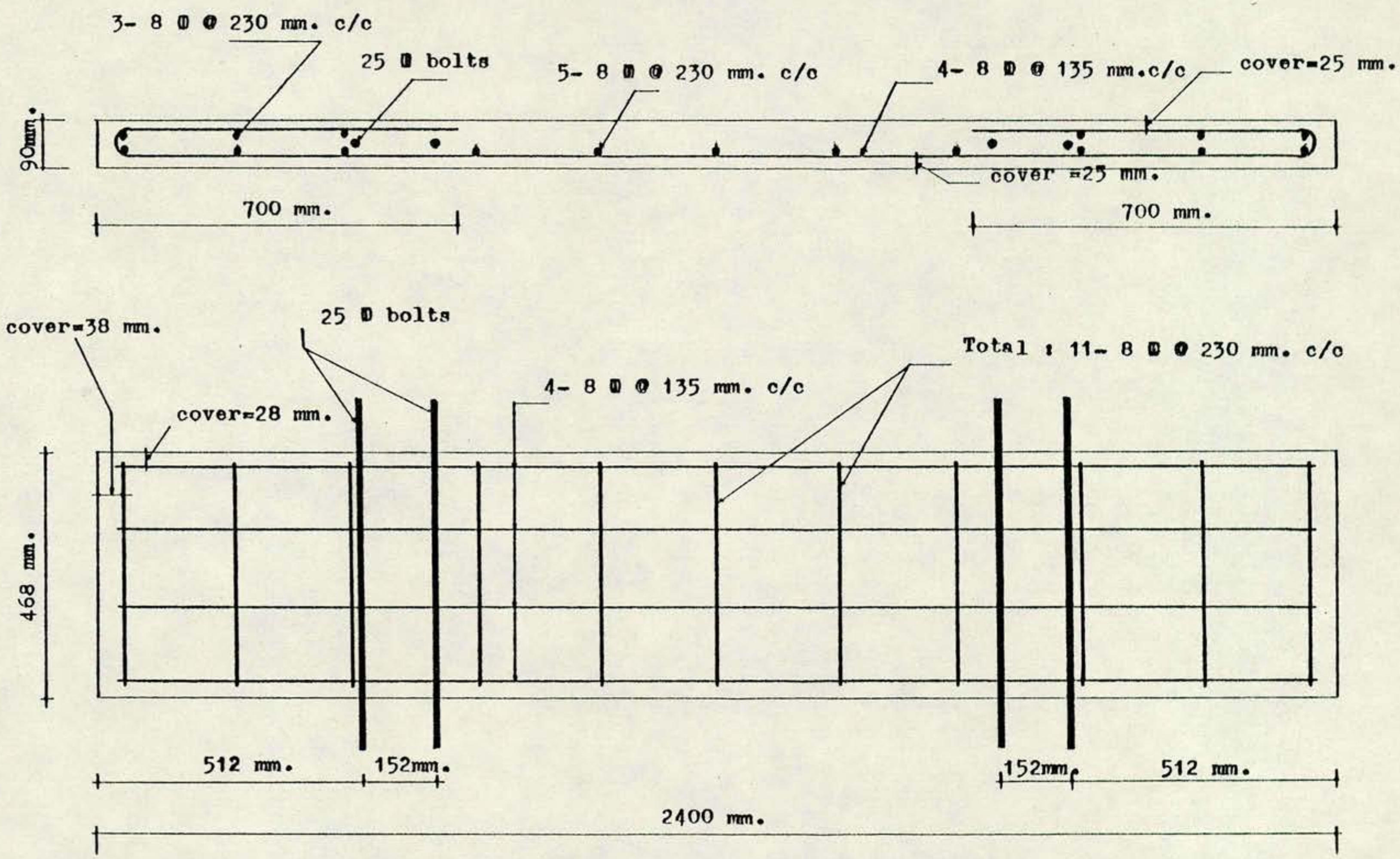


Fig.(5.2) Lower Slab Reinforcement Details.

prior to placement on the walls. The lower and upper slab reinforcement detail is shown in Figure (5.2), with the exception that, the top slab does not contain the 25 mm bolts embedded through the mid-slab depth.

The average cube compressive strength of the floor slabs ranged from 29.74 N/mm^2 and 31.38 N/mm^2 tested after 28 days water curing. The average modulas of elasticity of the concrete cubes was 24.36 KN/mm^2.

The slab reinforcement consisted of hot-rolled, high yield deformed bars (BS 4449), four 8 mm diameter bars placed longtitudinally in the bottom of the slab, with a cover equal to 25 mm and spaced at 135 mm c/c and were bent at the ends to provide for the negative moment on slab top and were extended to one fourth of the span. The temperature and distribution reinforcement consisted of 8 mm diameter bottom bars spaced transversely at 230 mm c/c and identical top bars spaced for one fifth of the span from each end.

Single and double brick prisms were prepared during construction of the walls and allowed to cure in the laboratory. The average modulas of elasticity of single prisms (six-bricks high) was 6.5 KN/mm^2 with a compressive strength of 11.63 N/mm^2. ul

Corresponding values for double brick prisms (eight-bricks high) were 5.04 KN/mm^2 and 8.17 N/mm^2 respectively. The average brick prism density was found to be 17.45 KN/m^3. A summary of the material properties is given in Table (5.1).

TABLE (5.1) MATERIAL PROPERTIES (HALF SCALE MODEL)

Material	Dimensions	28 days compressive strength N/mm^2		Average modulas of elasticity KN/mm^2	Remarks
		Frog upward N/mm^2	Frog downward N/mm^2		
Bricks	110.79 x 53.5 x 32.08 mm	32.56	33.02	-	Frogs filled with 1:1 cement/sand. Average of 12 samples.
Mortar	4" x 4" x 4"	11.35		-	1:½:4½ Cement:Lime:Sand by weight. Average of (5) cubes.
Brickwork					
Single prisms	11.05 x 53.5 mm	11.63		6.50	Average of (6) prisms
Double prisms	229 x 111.0 mm	8.17		5.04	Average of (5) prisms
Concrete cubes	4" x 4" x 4"	30.51		24.36	1:1.8:3.2 Cement:sand:gravel by weight. Average of (4) cubes.

Plate (5.1) shows the lower slab reinforcement. The reinforcement were seated on plastic supporters to ensure a uniform cover of 25 mm throughout. The 25 mm transverse bolts for the lower slab is also shown; these bolts were placed at slab mid-depth as a safety precaution against collapse that may occur due to excessive sidesway movements.

The construction of the brick walls was carried out by using coursing rods and levels and the workmanship was considered equivalent to that which could be obtained in practice. Upon completion of the lower storey walls, the first floor slab was set in place in a fresh mortar bed after seven days. The second storey walls were constructed and allowed to cure for one week before placing the second floor slab on fresh mortar layer. As mentioned earlier, as a safety precaution against collapse that may occur due to excessive sidesway of the structure as a whole, the lower slab was partially restrained from sidesway movements by 4-25 mm diameter bolts that were embedded through the mid-slab depth prior to concrete pouring, these bolts passed through slots in the steel loading frame, the slots were slightly larger than the bolts to ensure undisturbed deflection and rotation of the slab during loading but provide a stability factor if excessive sidesway were to occur. At top slab level, 6 inch collapse prevention channels were fixed to the loading steel rig, and 25 mm bolts passed through these channels to support the top slab laterally if a large sidesway were to occur. A small gap was left at both sides between the top slab and these bolts to eliminate any friction that may cause a restraint on the top slab deflection and rotation. The details of the loading frame are shown in Figure (5.3). Plate (5.2) shows a general view of the test structure and loading frame, and the top slab collapse prevention bolts and channel is shown in Plate (5.3). Plate (5.4) shows the lower slab lateral bolts in position.

5.2.3 Loading Sequence:

All tests were conducted using the dead load position of the structure as the reference position.

Two independent loading systems were used. One system applied axial load to each wall using 2 - 10 ton capacity hydraulic jacks

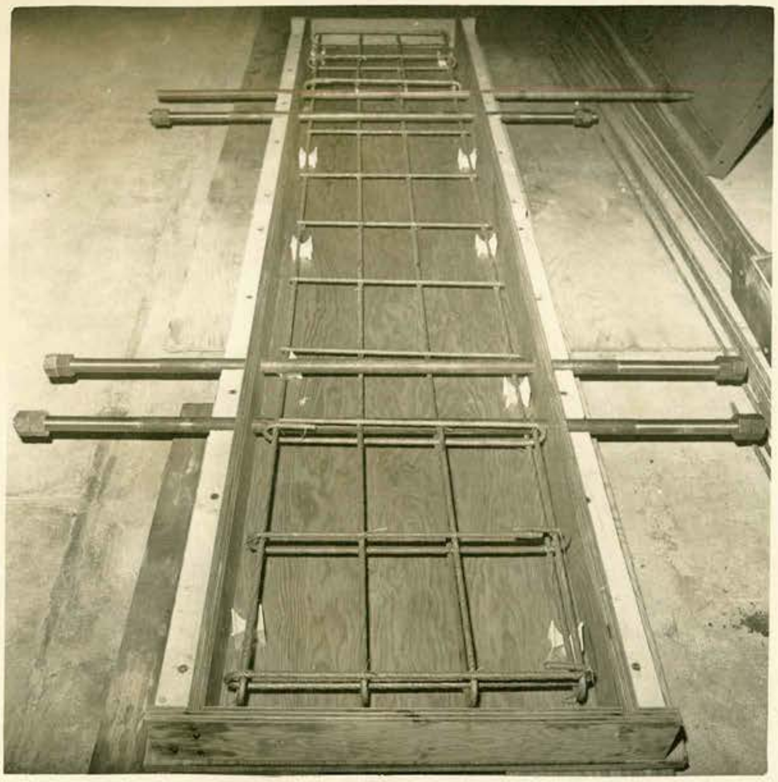


Plate (5.1)

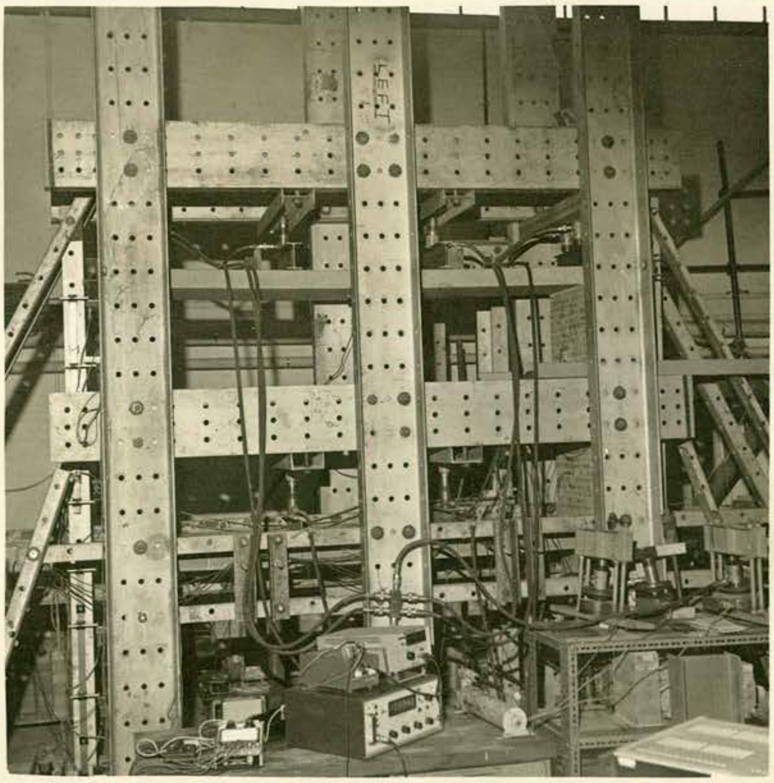


Plate (5.2)

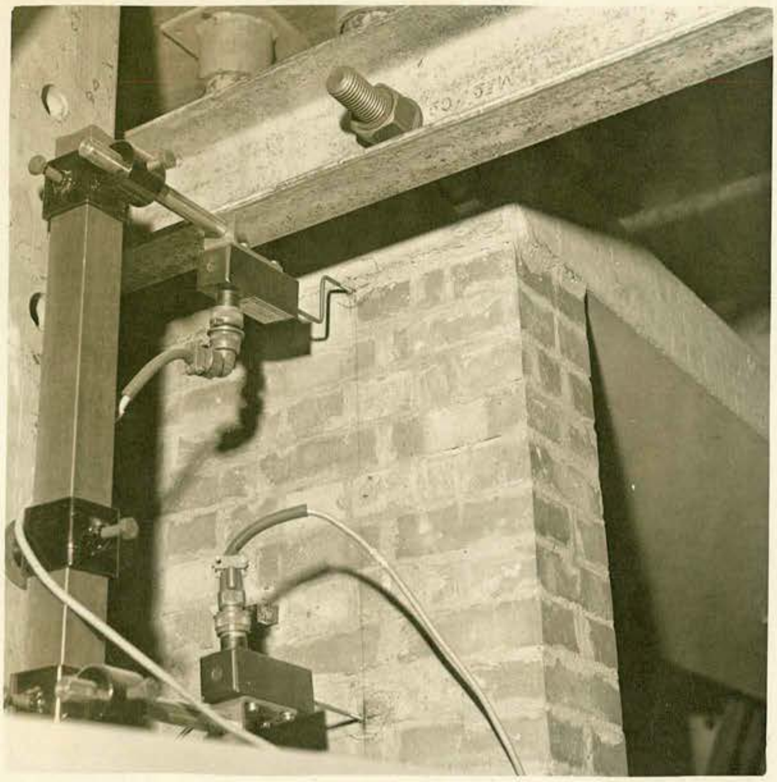


Plate (5.3)

Plate (5.4)

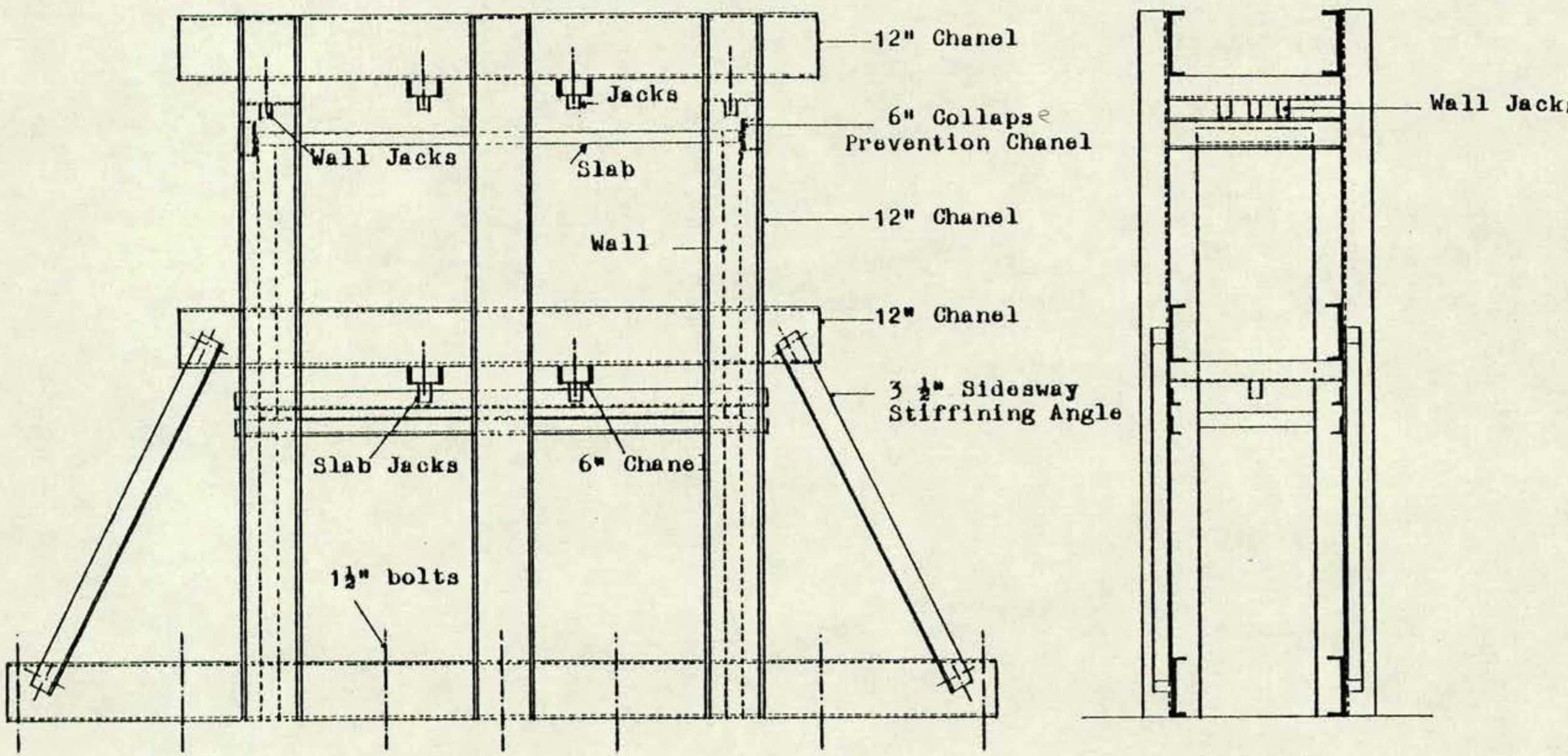


Fig.(5.3) Loading Frame Details.

by means of an electrically operated pressure pump. These jacks were located on the upper surface of the top floor slab. The jacks load was distributed through 4" steel beams resting on a 1" solid steel distribution plate which was seated on a 4 mm rubber pad as shown in Figure (5.4). In the second loading system, load could be applied to either or both of the floor slabs at their third points by 3 ton capacity single load jacks connected to a manually operated hydraulic pump. The jack loads were distributed through 4" steel beams resting on 4 mm rubber pad. The applied loads were measured using load cells located under a single jack connected to each loading system. Both loading systems were calibrated in an Avery Testing Machine. Plates (5.5) and (5.6) show the walls and slabs loading jacks respectively.

The magnitude of active wall precompression (i.e. jack loads excluding dead load of structure) varied from 10 KN/wall up to 80 KN/wall at 10 KN increments.

The two floor loadings were 1.5 KN and 3.0 KN on each load point for the major tests where floors and walls loaded simultaneously; these loadings are considered equivalent to uniformly distributed live loads of 1.75 KN/m and 3.5 KN/m respectively.

In one series of tests, the wall precompression was applied prior to loading the lower floor. In a second series of tests the floor load was applied prior to precompressing the walls. These tests simulate variation in the construction sequence in which the former is representative of a cast-in-situ floor

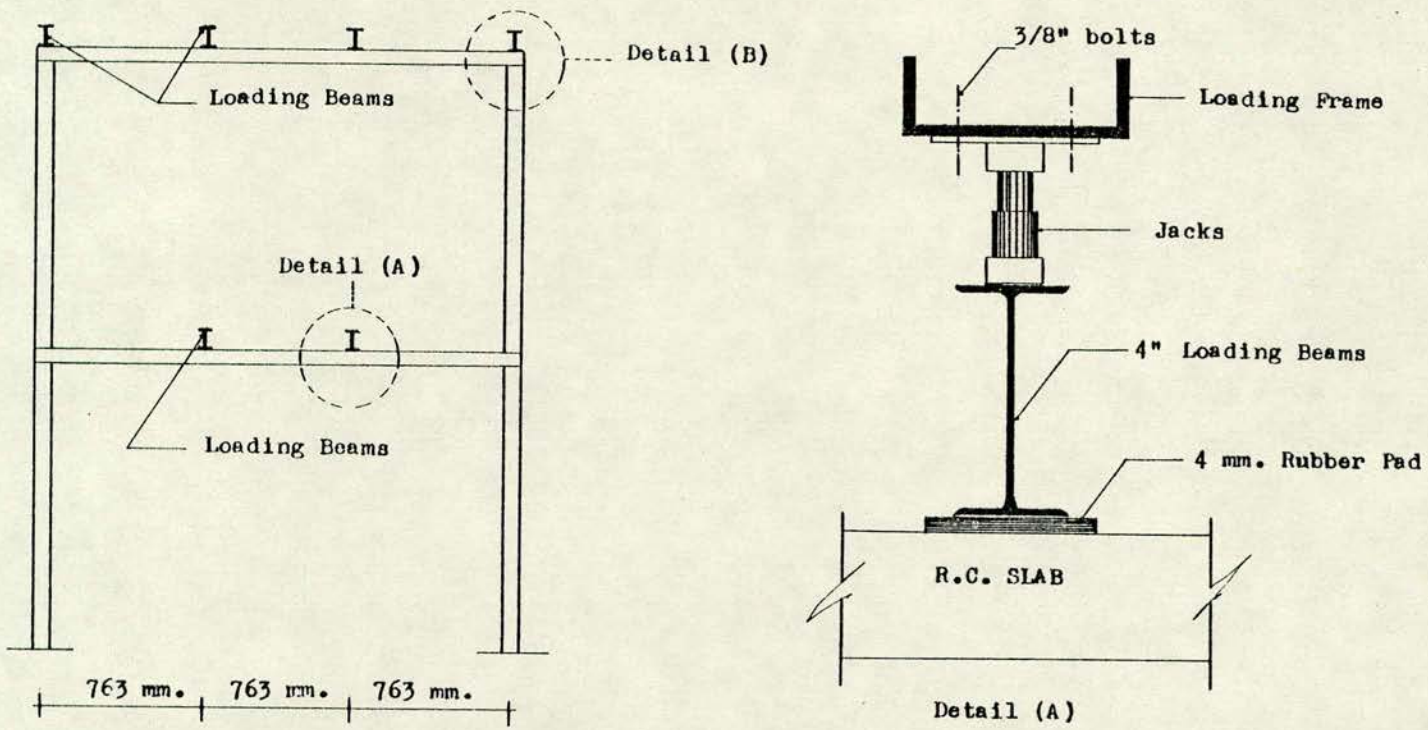


Fig.(5.4) Loading Beams Detail.

N.B. Detail (B) is Similar to detail (A).

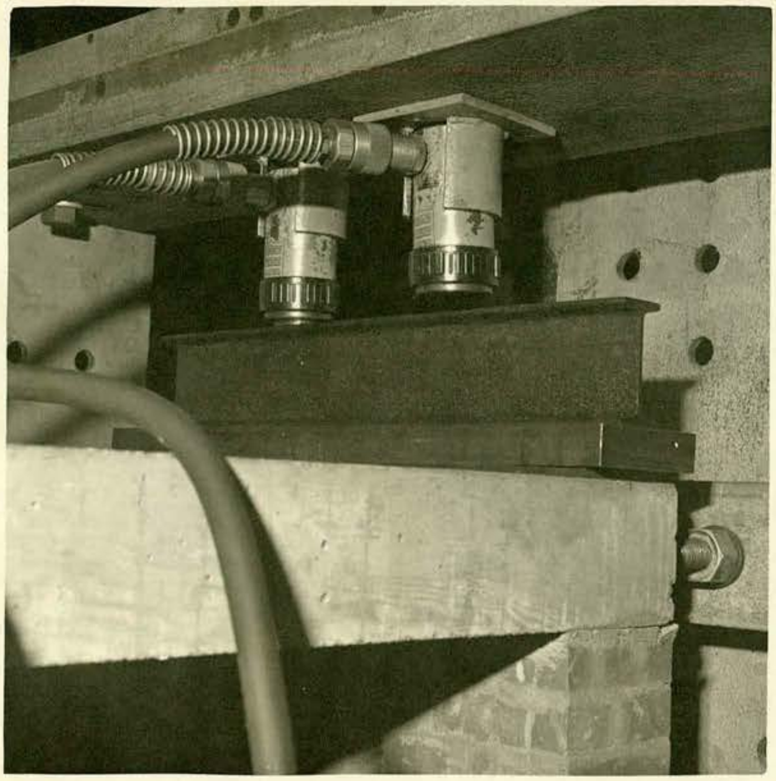


Plate (5.5)

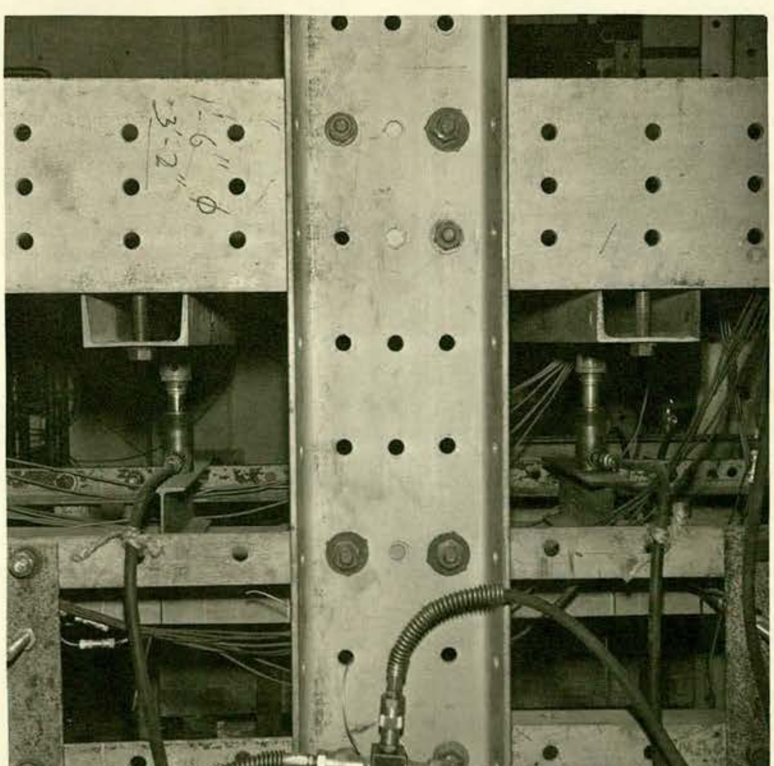


Plate (5.6)

slab, while the latter represents a precast floor system.

A summary of test programme is given in Table (5.2). In each test the following sequence was followed:

1. Read all gauges.

2. Apply either wall or floor load and re-read gauges.

3. Apply remaining wall or floor load and re-read gauges.

Each test was repeated at least twice to ensure reliability of the results. If there was any significant variation in the magnitude of any of the readings the test was repeated until a suitable average was obtained.

TABLE (5.2) - SUMMARY OF TESTS - HALF SCALE FRAME

Test Series	Wall precompression		Floor load KN/point load		Remarks
	Active (KN)	Total (N/mm^2) *	Lower	Upper	
A	0.0	0.0	0.0	0-3	Upper floor loaded only to compute the slab flexural rigidity EI
B	0-80	0-1.567	0.0	0.0	To investigate wall behaviour (def. and rot.) due to wall precompression
C	0.0	0.071	0-4.0	0.0	To determine the joint rigidity of the structure with no wall precompression, joint moments and eccentricity
D,E	0-80	0-1.638	0-3.0	0.0	The wall precompression applied prior to loading slab
F,G	0-80	0-1.638	0-3.0	0.0	Floor loads applied prior to wall precompression
H	0-81.5	0-1.666	1.5	1.5	Floor slabs loaded prior to wall precompression
J	0-83	0-1.697	3.0	3.	Same as (H). Slabs load = 3 KN.
K	0-94.0	0-1.843	3.0	3	Lower and upper floors loaded prior to wall precompression. Each increment of precompression equals the load of one storey above the joint.

* including D.L. of structure, loading beams and jacks.

5.2.4 Instrumentation:

Measurements of deflection, rotation and strain at several locations throughout the structure were recorded in each of the tests conducted. Details of the three basic types of measurements are given in the following.

5.2.4.1 Deflections:

A total of 18 linear displacement transducers reading to 0.01 mm were used. The walls transducers were supported by an independent separate steel frame which was diagonally braced to the laboratory concrete floor to eliminate any horizontal movements as shown in Figure 5.5.

These transducers were fixed to a square hollow steel section 1.5/8" which in turn was fitted into a 1½" square section to enable free movement and positioning before the fixing screw was tightened to lock the transducer in position as shown in plate (5.7).

The upper slab transducer was fixed on the lower floor slab and connected to a stainless steel rod which was fixed at its end to the upper floor slab. The lower slab transducer was fixed on the laboratory concrete floor and similarly connected to a stainless steel rod which was fixed to the underneath side of the lower floor slab. Thus, the net upper slab deflection was dependent on the lower slab deflection as well, hence, the net deflection of the upper slab is obtained as:

Net upper slab deflection = $\mp \Delta$ upper slab $\mp \Delta$ lower slab,

Downward deflection is positive and upward deflection negative.

Since the lower slab transducer was fixed to the floor, the lower slab deflections recorded represent the actual net deflection. Plates (5.8) and (5.9) show the upper slab transducer fixed on to the lower slab floor and the lower slab transducer fixed on to the concrete floor respectively. All transducers were connected to a telytype printer and their locations and numbers are shown in Figure (5.6).

5.2.4.2. <u>Rotation:</u>

Rotation of the wall and floors at their junction was measured using electronic levels reading to 10 secs or 0.5 meter radions (estimate to 0.1 m. rad.). A total of five levels were installed and their locations are shown in Figure (5.7). The electronic levels at lower slab level are shown in Plate (5.10).

5.2.4.3 <u>Strain:</u>

It was initially proposed that wall strains would be recorded using 140 mm vibrating wire gauges for the test structure mounted on both faces of the walls and slabs as shown in Figure (5.8) and Plate (5.11) and 2.5 inches long vibrating gauges for brick prisms. These gauges were connected to a data logger with print out. Each gauge was automatically read 4 times at each loading so that an accurate average value could be obtained. However, during the course of testing, it was evident that strain values obtained in this manner did not necessarily reflect the strain across the wall width, hence, it was decided to ignore these readings as they are unreliable, and demec gauges were used to measure the strain in the brick prisms.

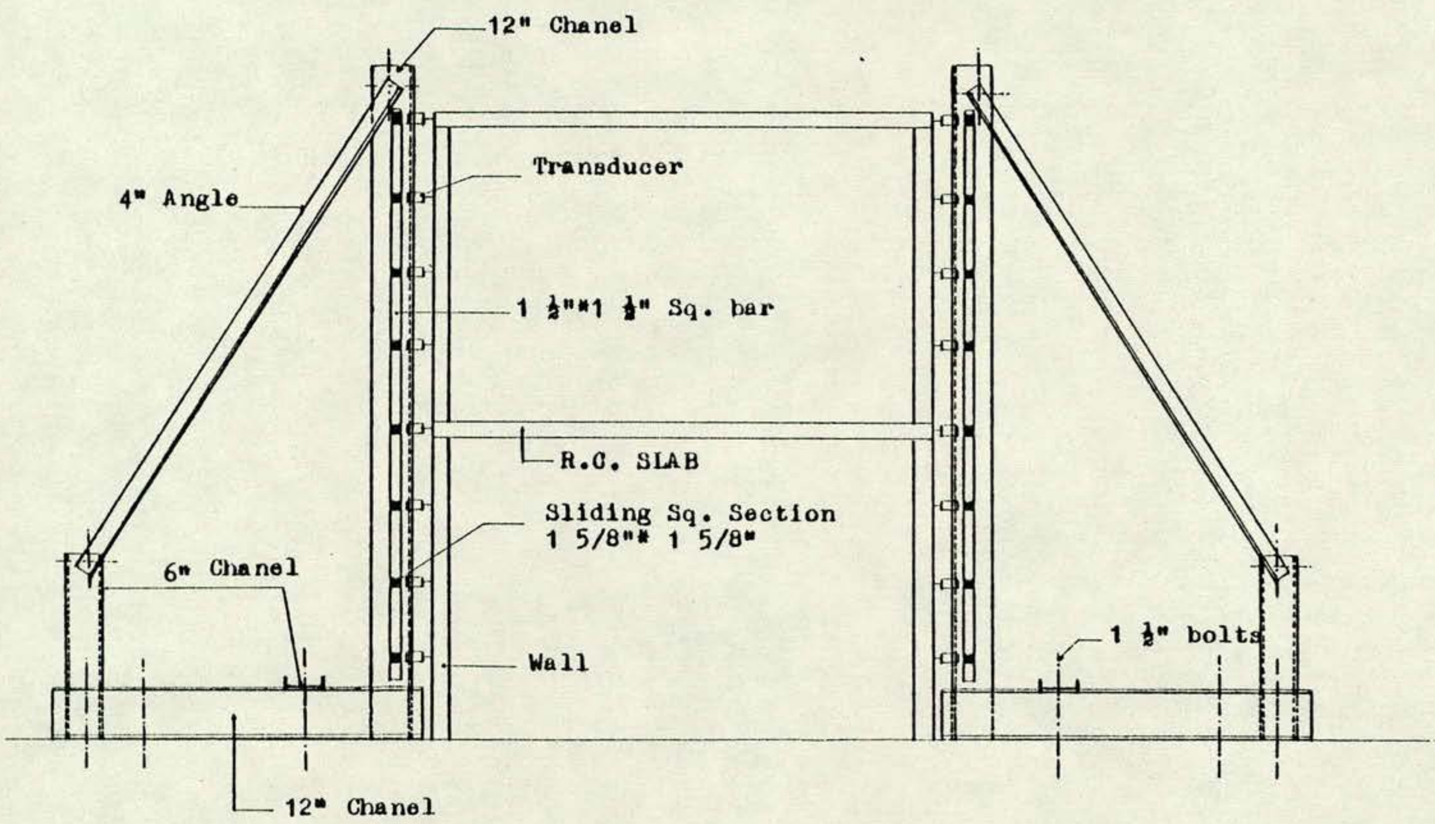


Fig.(5.5) Steel Frame Supporting Wall Transducers.

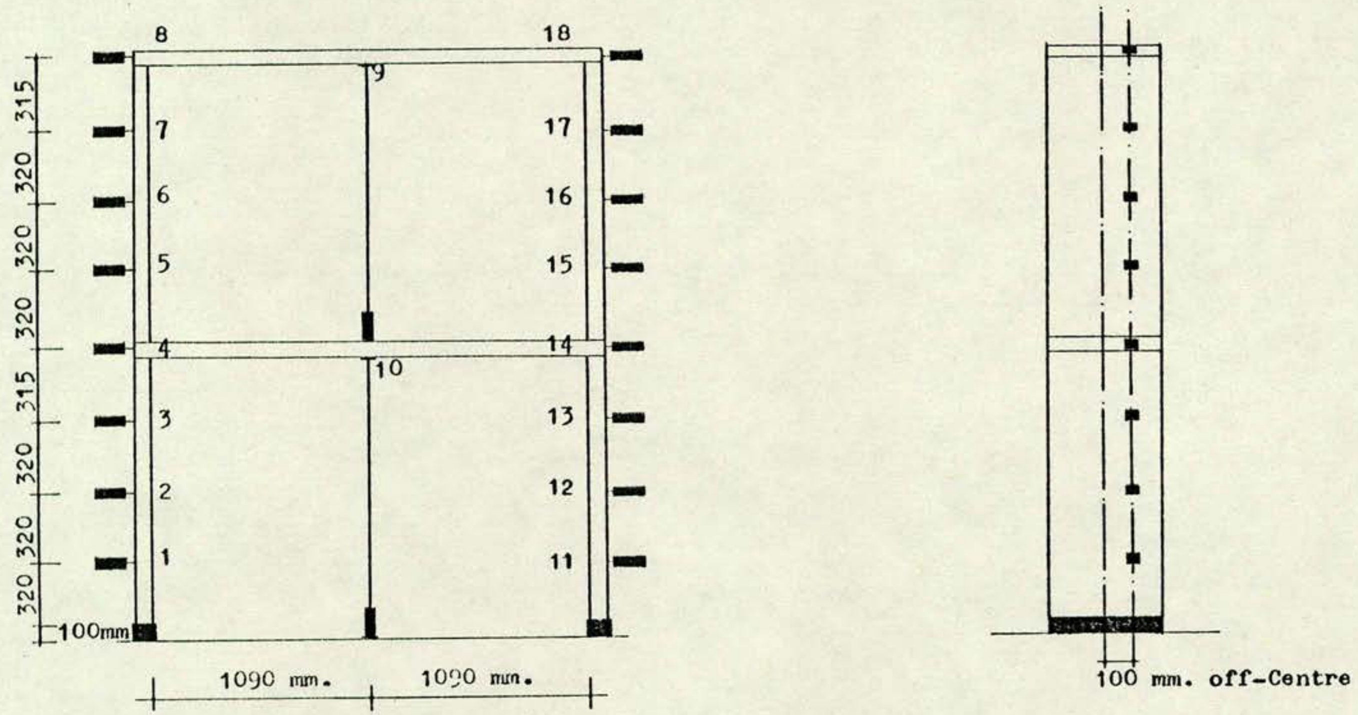


Fig.(5.6) Transducers Numbers & Locations.

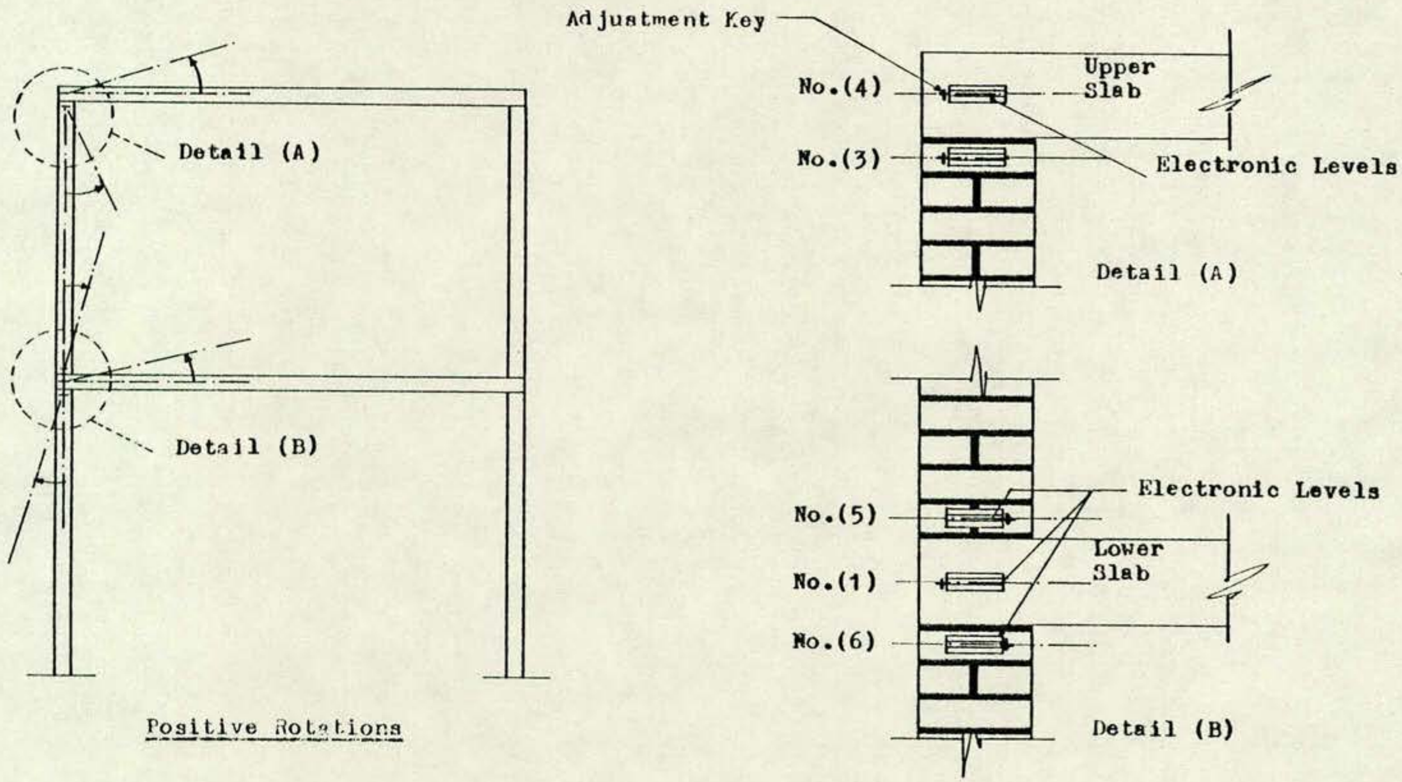


Fig.(5.7) Arrangement of Electronic Levels.

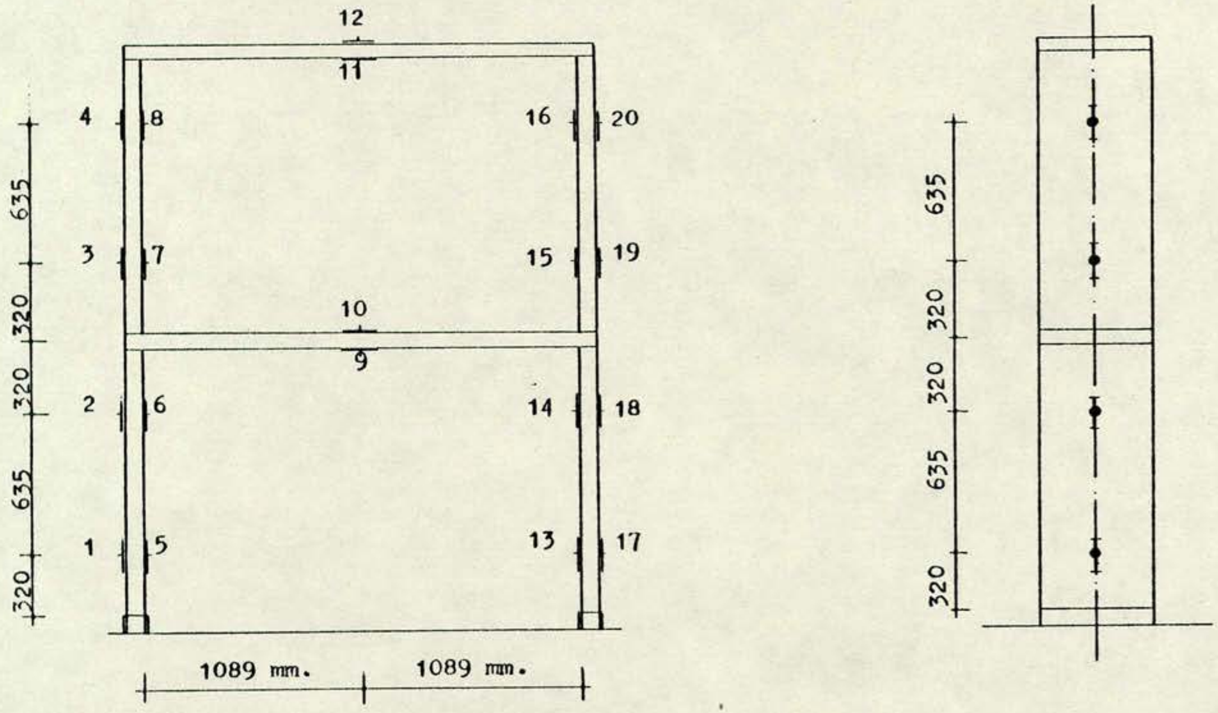


Fig.(5.8) Vibrating Wire Gauges Numbers & Locations.

Plate (5.7)

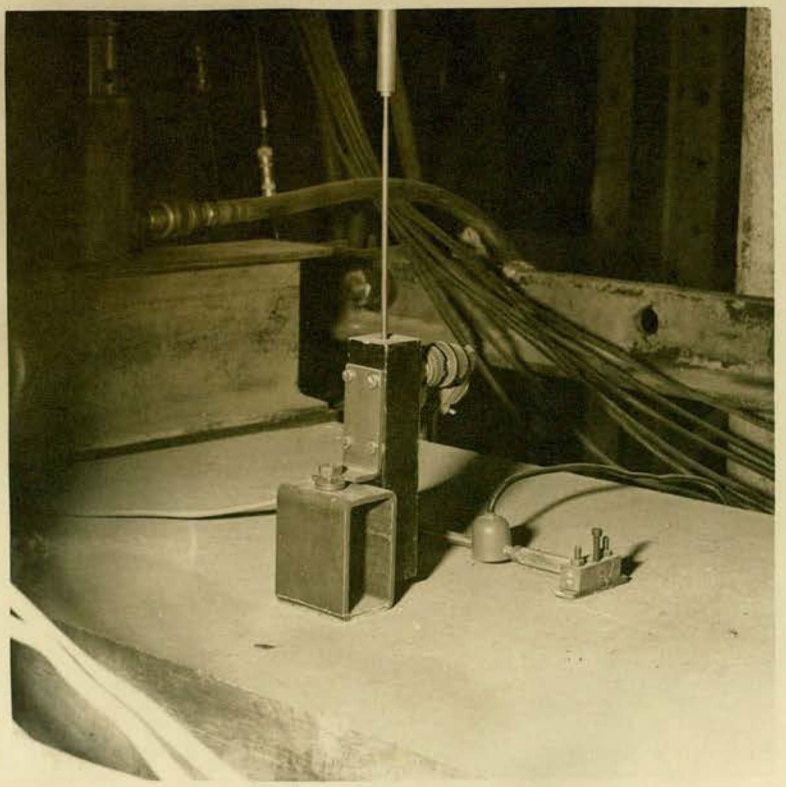


Plate (5.8)

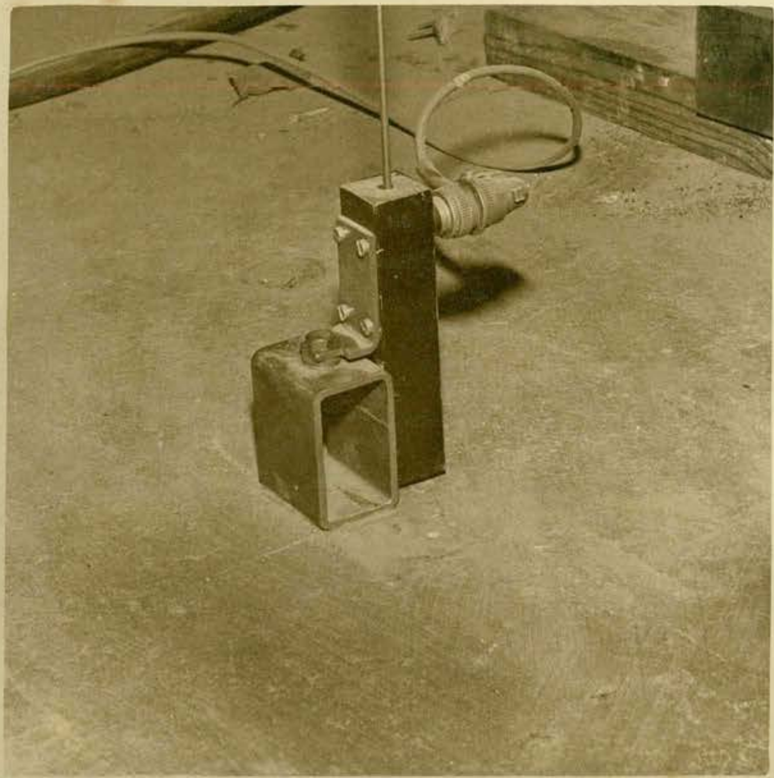


Plate (5.9)

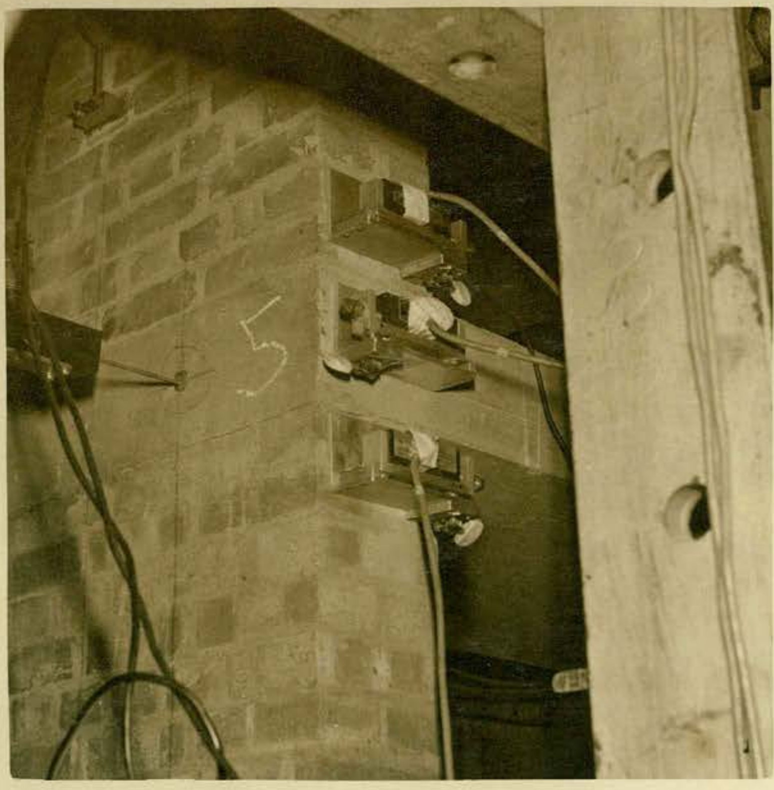


Plate (5.10)

Plate (5.11)

5.3 FULL SCALE TEST STRUCTURE:

5.3.1 Description of test structure:

The two bay, three storey structure is shown in Figure (5.9). The brick walls consisted of 32 courses per storey height and the average height was 2438 mm with average wall thickness equal to 102 mm, thus, the wall thickness equals half a brick. The average width of the test structure is 1219 mm.

This structure was built for an earlier series of tests reported in Reference (26). The Omnia slab reinforcement details are shown in Figure (5.10) together with the dimensions of these precast unit slabs.

5.3.2 Material properties and construction procedure:

The two bay, three storey, structure was constructed (26) using three hole, perforated, wire-cut common bricks of compressive strength 37.9 N/mm^2 with a coefficient of variation of 19%. A 1:1/4:3 (rapid hardening cement:hydrated lime:sand) mortar was used, the average compressive strength of the mortar was 12.7 N/mm^2 at 28 days age.

Brick prisms six-courses high were built and cured on site for quality control. The specimens were capped top and bottom for testing. The average compressive strength of these prisms at an age of 1½ years was 20 N/mm^2 and the modulas of elasticity is taken as 11.7 KN/mm^2. u/

50 mm thick 'Omnia' precast panels equal to the internal dimension of the building bay were lifted and kept in position by props with no bearing on the walls except for the reinforcement protruding at the ends of the panels. Ready mix concrete (1:2:4) was poured

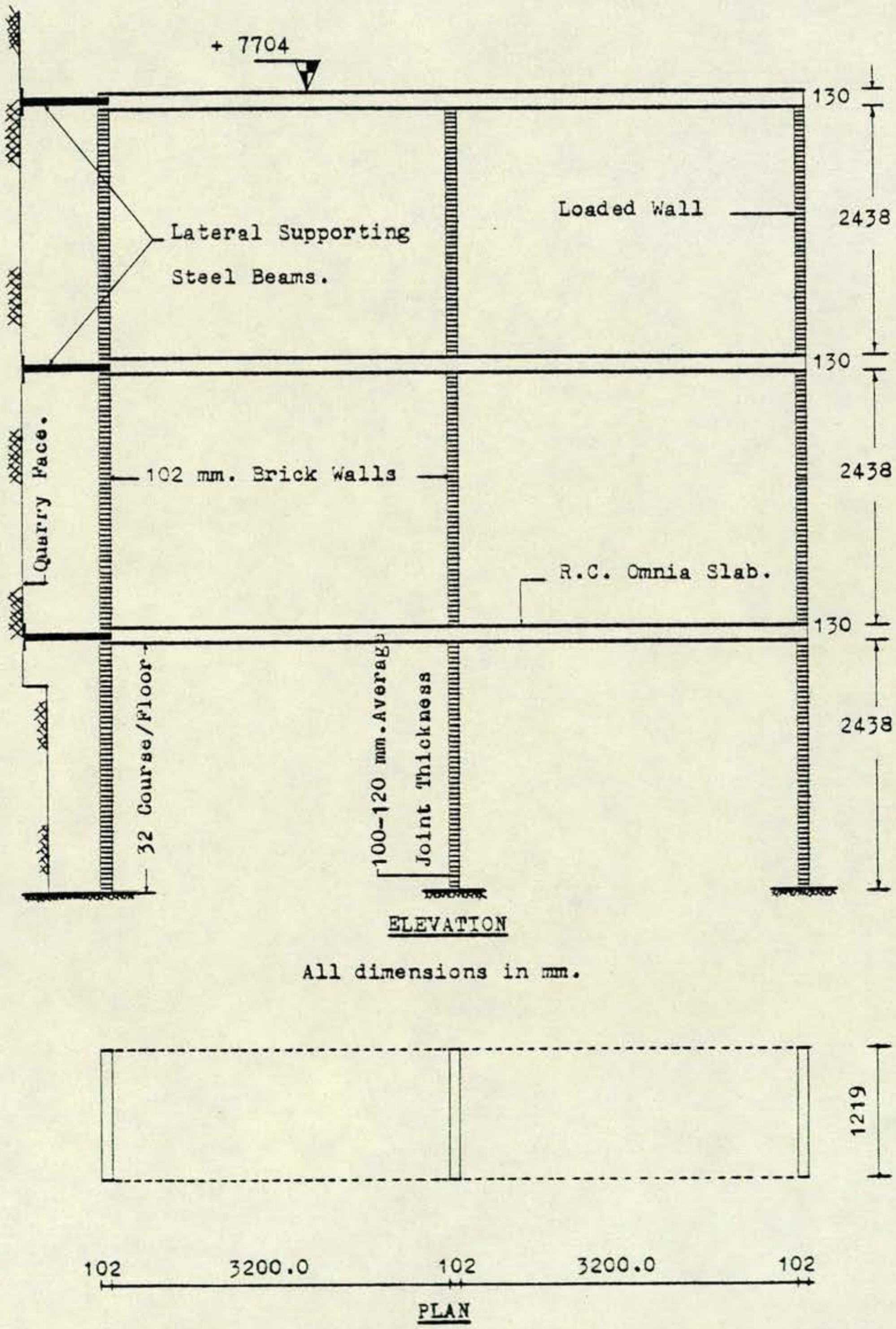


Fig.(5.9)- Test Structure.

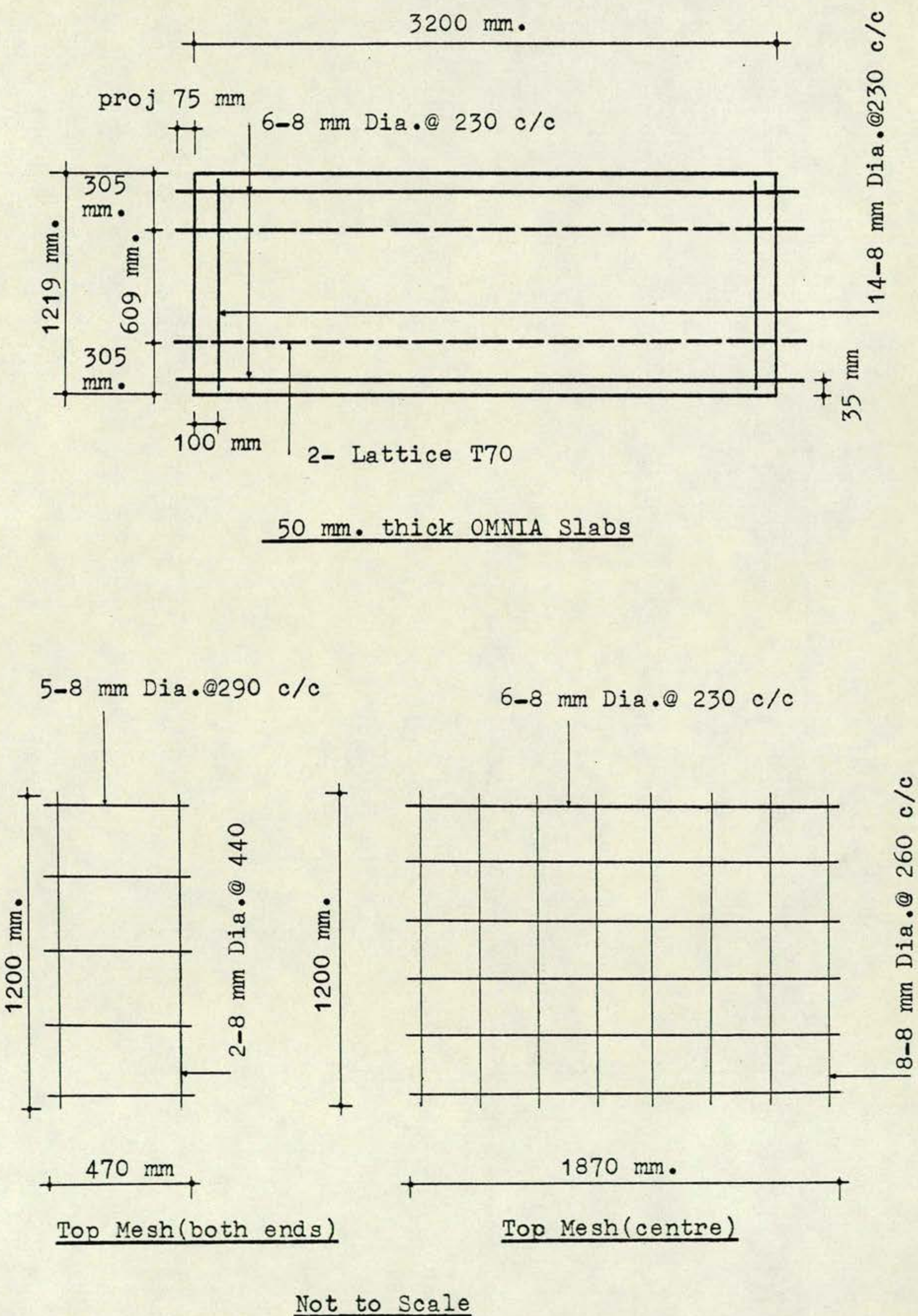


Fig.(5.10)- Omnia Slabs Reinforcement Detail.

on top of the precast slabs to obtain a thickness of 130 mm throughout.

Negative reinforcement was provided as required for continuity at the end and at the central support, before pouring the concrete. By adopting this method of construction, not only considerable saving in time and cost of shuttering was achieved but a good joint similar to a cast-in-situ slab was obtained between the finished slab and wall underneath. The average compressive strength of the concrete cylinders was 25.8 N/mm^2 at an age of 1½ years and the corresponding modulas of elasticity was 25.83 KN/mm^2. A summary of the material properties is given in Table 5.3.

TABLE (5.3) - MATERIAL PROPERTIES - FULL SIZE TEST STRUCTURE

Material	Compressive strength N/mm^2	Modulas of elasticity KN/mm^2	Remarks
Bricks	37.90	-	Three hole, perforated wire-cut common bricks
Mortar	8 - 12.7 7 days - 28 days	-	1:1/4:3 Cement:lime:sand
Brickwork (single prisms)	17.0 - 20.0 28 days - 1½ years	11.70	1:1/4:3 Cement:lime:sand
Concrete	18 - 25.80 7 days - 1½ years	25.83	1:2:4 Cube tested at 7 days age. Cylinder tested after 1½years.

5.3.3 Loading sequence:

Once again two loading systems were used. One system applied axial load to the single exterior wall using two jacks as shown in Figure (5.11). Two load cells were located on the upper surface of the top floor slab (at third floor level). These jacks were connected to the top of a specially assembled steel beam which in turn was connected to the ground anchorage beams by 50 mm high tensile steel bars as shown in Figure (5.12). As the load was applied to the jacks, the top box beam was forced downward against the loading cells which in turn transferred the loads to the wall underneath. The magnitude of the applied loads was measured by the load cells placed between the loading beam and the load distribution plate. This simple method of loading the walls by tensile jacks substituted the conventional method of having a large three storey high steel loading frame which would cost much more and require longer to assemble.

In the second system, load could be applied to the first floor slab at the third points of the floor span (one span loaded only). Applied floor loads were measured using a single load cell and two loading jacks connected to the floor slab loading frame as shown in Figure (5.13).

The magnitude of the active wall precompression considered in the major tests varied from 20 KN and up to 240 KN at 20 KN increment, and the floor loading varied from 3 KN per point load to 15 KN by 3 KN increments. The point loads are considered equivalent to uniformly distributed live loads of 2.42 KN/m to 12.11 KN/m based on continuous beam analysis.

The loading sequence of floor and wall loadings followed the same sequence as in the half scale model. A summary of the test programme is shown in Table (5.4).

A general view of the test structure is shown in Plate (5.12), the outer wall precompression loading frame is also shown, the box beam for wall precompression is shown in Plate (5.13) and the supporting ground anchorage beams in Plate (5.14). As may be seen from Figure (5.9), the floor slabs are supported laterally by steel sections at each level and fixed to the quarry face to prevent possible collapse that may occur due to excessive lateral movements of the floor slabs. In addition, a steel scaffolding frame was erected around the test structure to provide an easy access for installing the instrumentation. It was also arranged that this scaffolding would support the floor slabs if collapse was to occur during testing. The scaffolding struts were placed underneath each floor level leaving 25 mm clearance between them and the floor slabs.

To prevent any tilting or collapse of the wall precompression frame, the box loading beam was tied by a tension bracing system consisting of steel cables with turn buckle adjustment and fixed to the quarry face.

A view of the slab loading frame details is shown in Plate (5.15).

TABLE (5.4) - SUMMARY OF TESTS - FULL SIZE STRUCTURE

Test Series	Wall precompression		Floor load KN/Point load	Remarks
	Active KN	Total * N/mm^2		
A	0.0	0.193	0-15.0	To determine floor slab flexural rigidity EI
B,C,D, E,F	0-240	0.193-2.123	0-15.0	Precompression applied prior to loading floor slab. Floor slab load increments at 3.0 KN.
G,H,J, K,L	0-240.0	0.193-2.123	0-15.0	Floor load applied prior to wall precomp. Floor slab load increments at 3.0 KN.
M	100.0	0.997	0-15.0	Constant precompression applied prior to loading floor slab
N	200.0	1.801	0-15.0	Same as M
P	300.0	2.60	0-15.0	Same as M
R	600.0	5.018	0-37.5	Same as M. Floor slab load continuously increased until failure.

* including dead load of structure, loading beams and jacks.

5.3.4 Instrumentation:

5.3.4.1 Deflections:

A total of 11 linear displacement transducer reading to 0.01 mm were used. The numbers and locations of these transducers are shown in Figure (5.14). The wall transducers were supported by an independent separate steel frame similar to that used for the half scale model structure. A view of the transducers and supporting frame is shown in Plate (5.16).

The slab transducer was supported by an individual scaffolding system which was braced diagonally to eliminate horizontal movements and was fixed to the concrete floor. Plate (5.17) shows the slab transducer in position.

All transducers, electronic levels and vibrating wire gauges were weather protected by covering them with plastic containers as shown in Plate (5.16).

5.3.4.2 Rotation:

Rotation of the wall and floors at their junction was measured using 3 electronic levels as shown in Figure (5.15) and Plate (5.18); these levels were also weather protected by using a plastic container.

5.3.4.3 Strain:

Wall and slab strain were recorded using vibrating wire gauges. The locations and numbers of these gauges are shown in Figure (5.14). As mentioned earlier, it was evident that strain readings obtained were not regarded as reliable and were ignored.

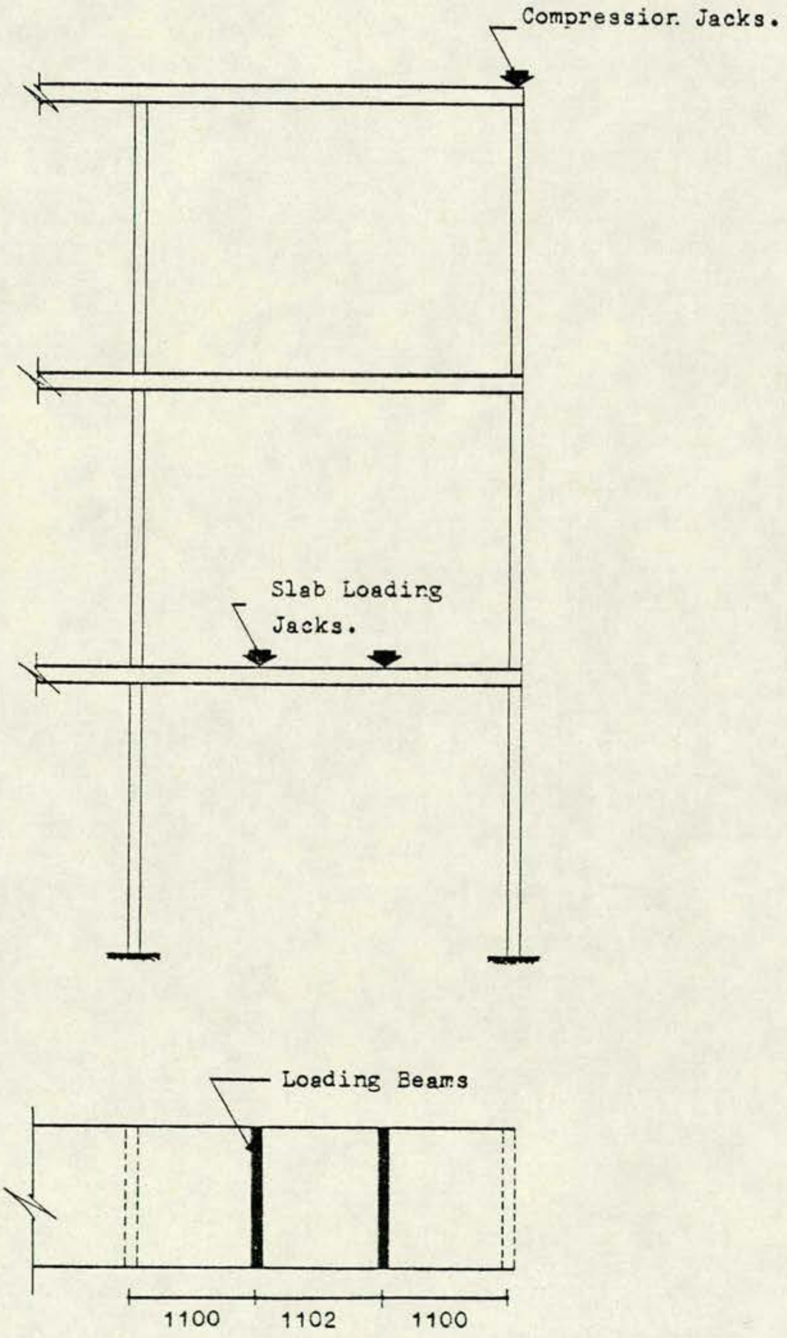


Fig.(5.11)- Test Structure Loading Arrangement.

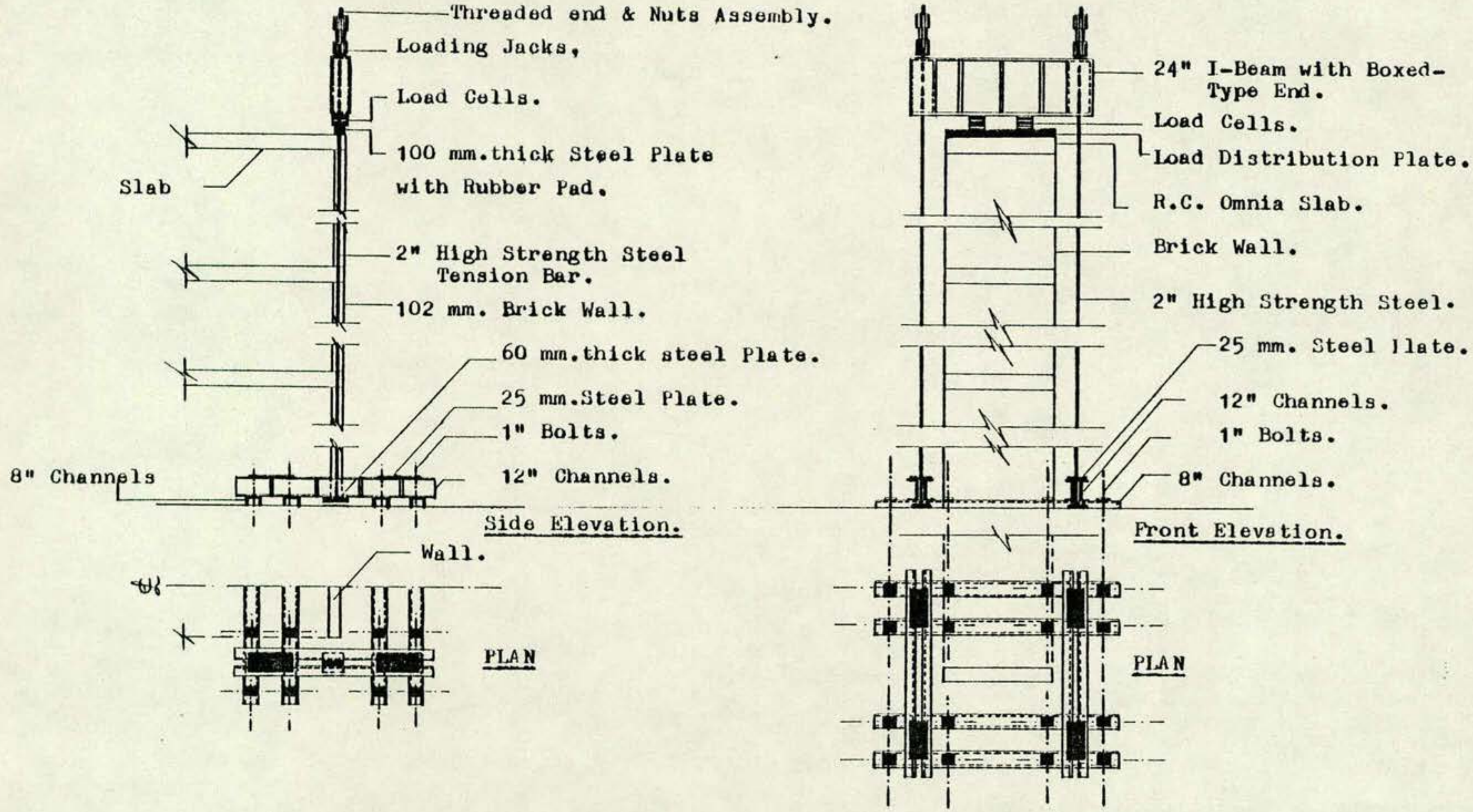


Fig.(5.12)-Loading Frame for Wall precompression.

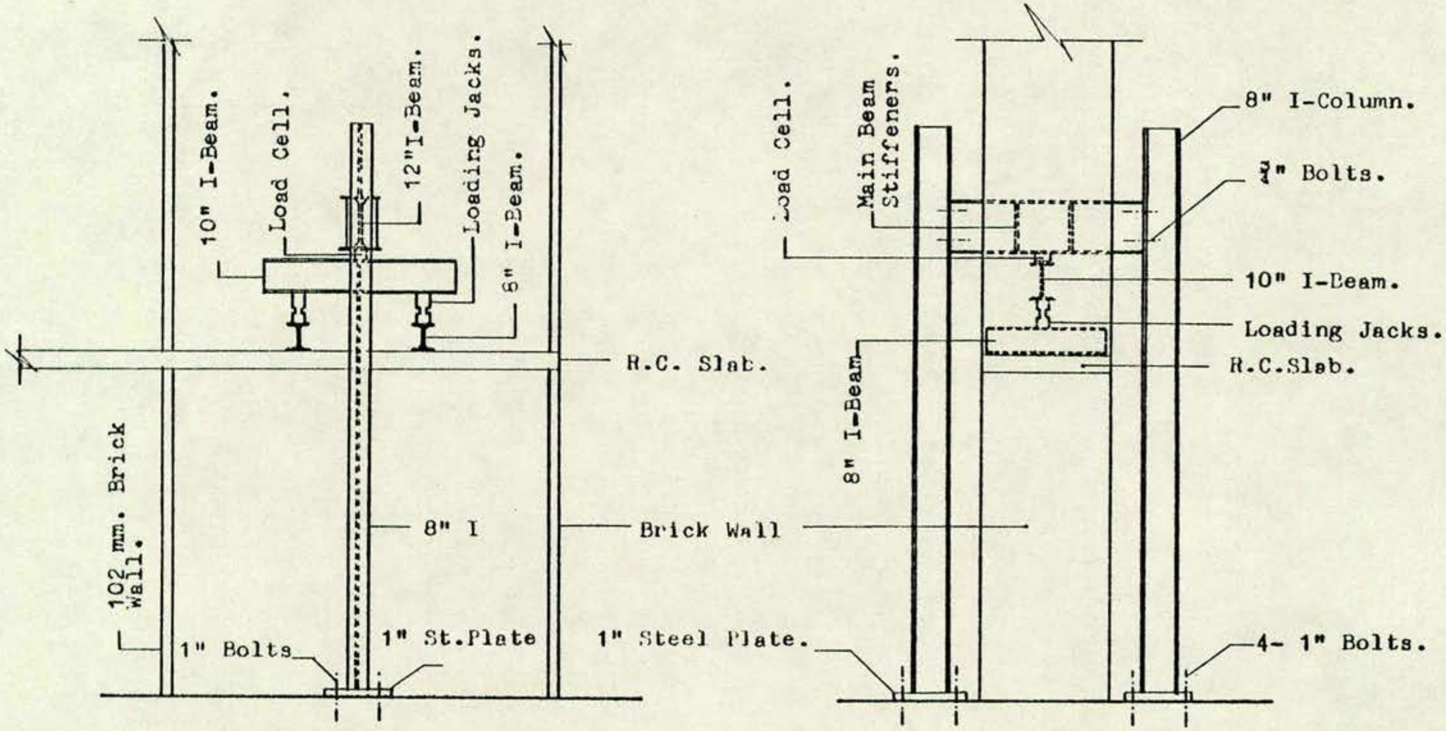
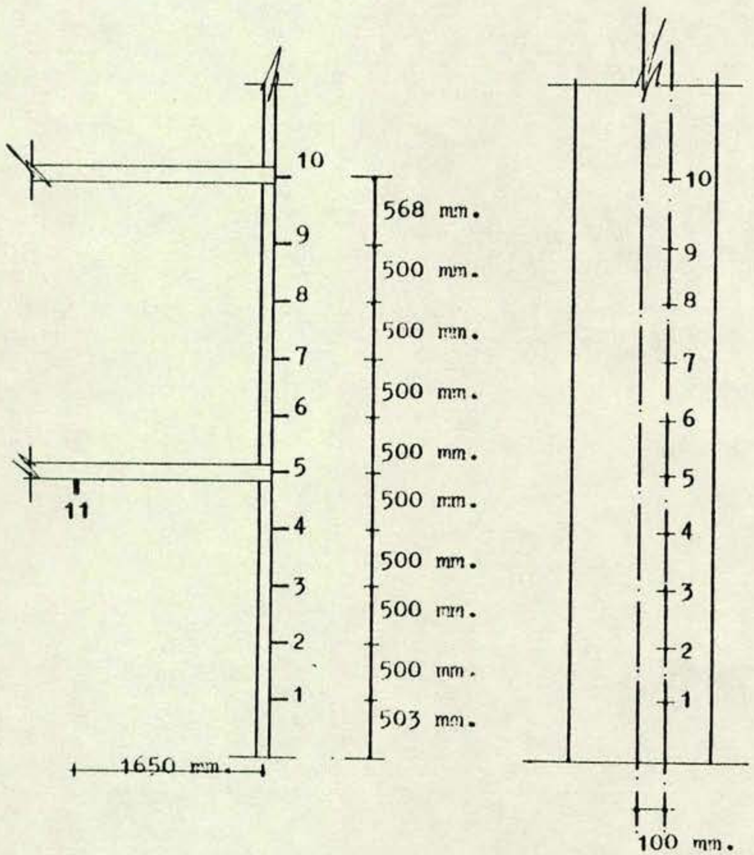
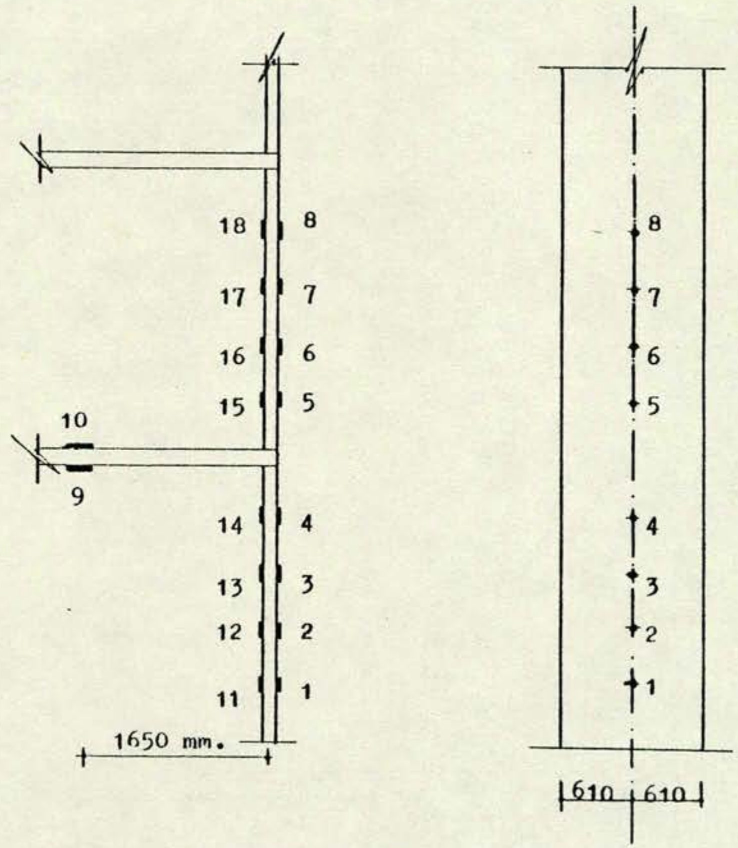


Fig.(5.13)- Slab Loading Frame Details.

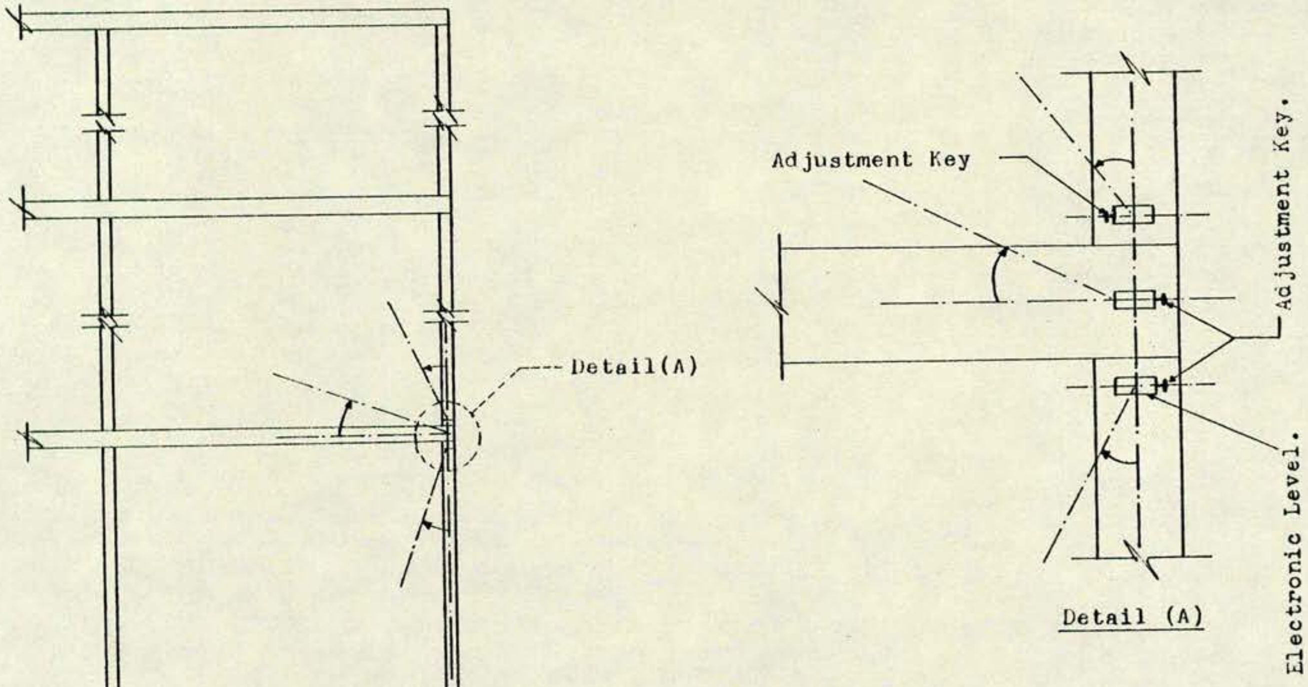


Transducers numbers & Locations.



Vibrating Strain Gauges no.& Locations.

Fig.(5.14)



Positive Rotations

Detail (A)

Fig.(5.15)

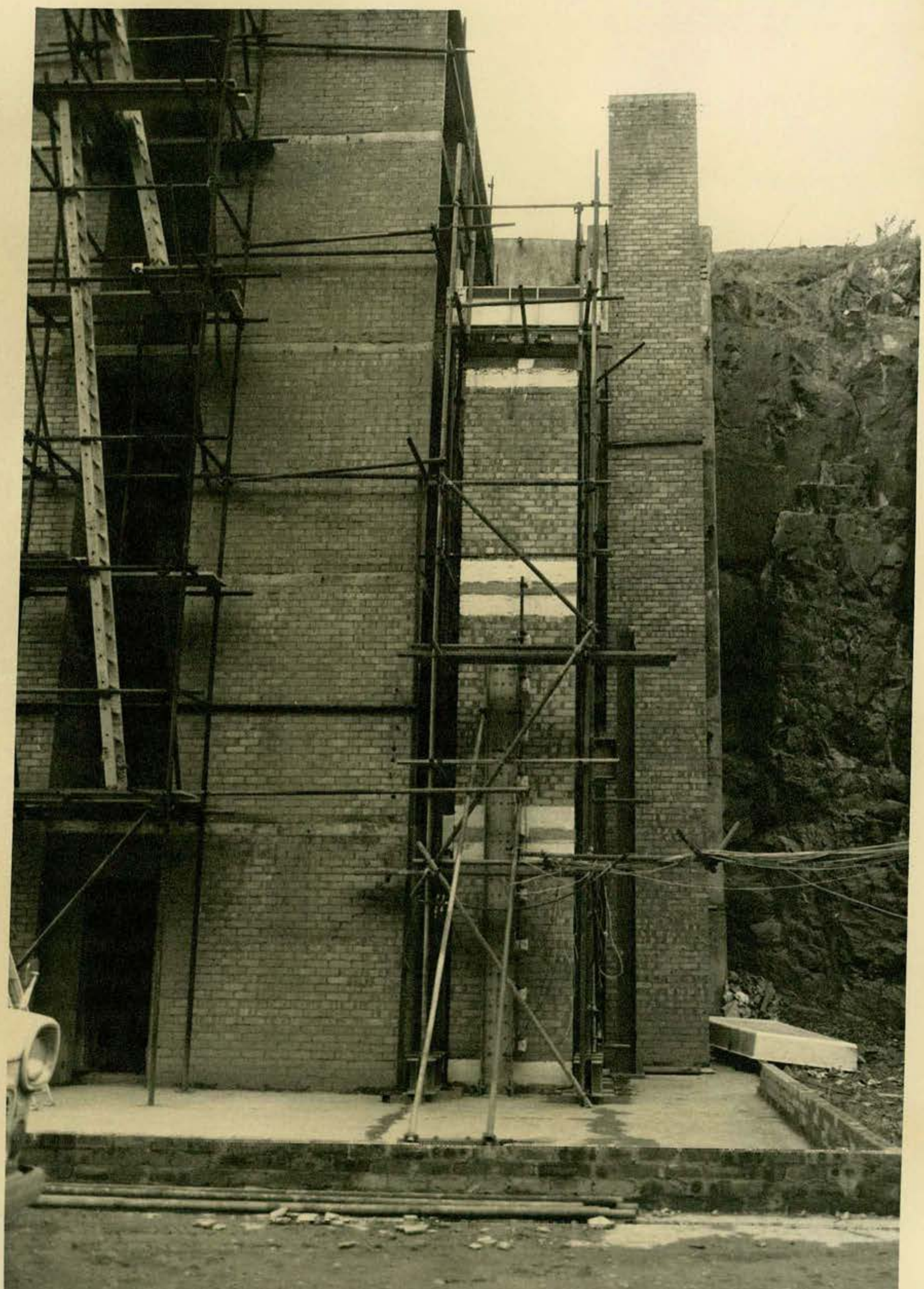


Plate (5.12)

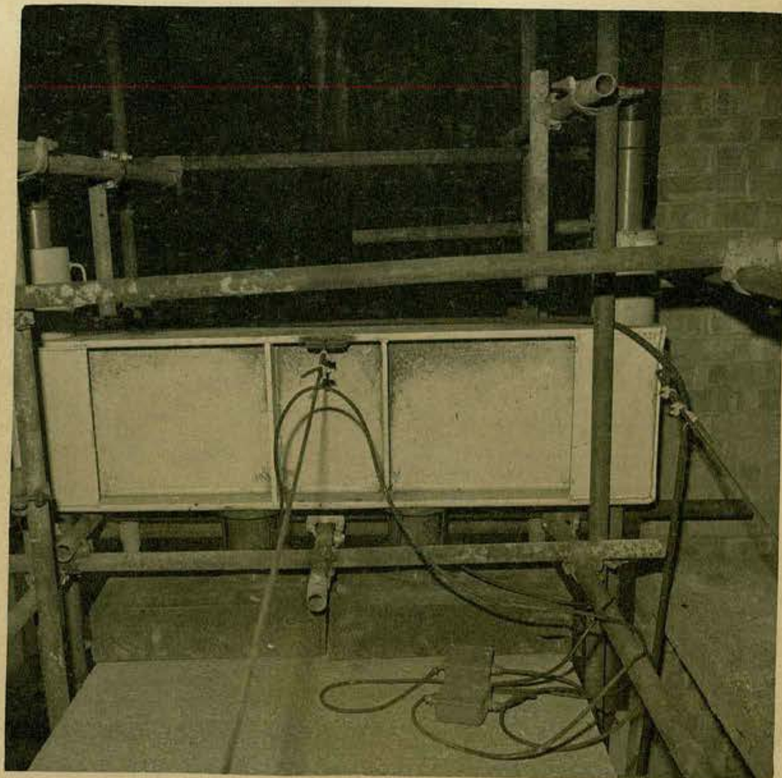


Plate (5.13)

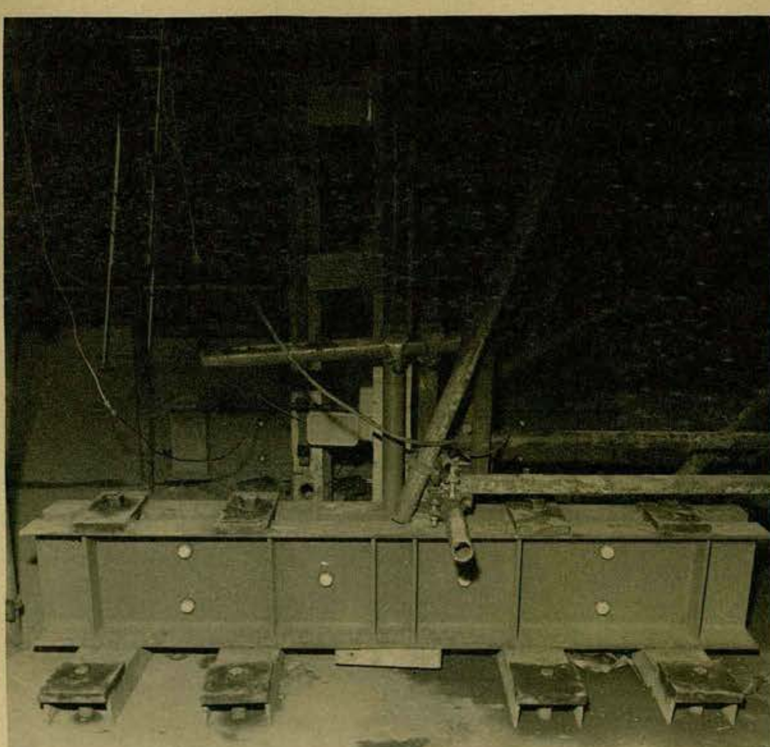


Plate (5.14)

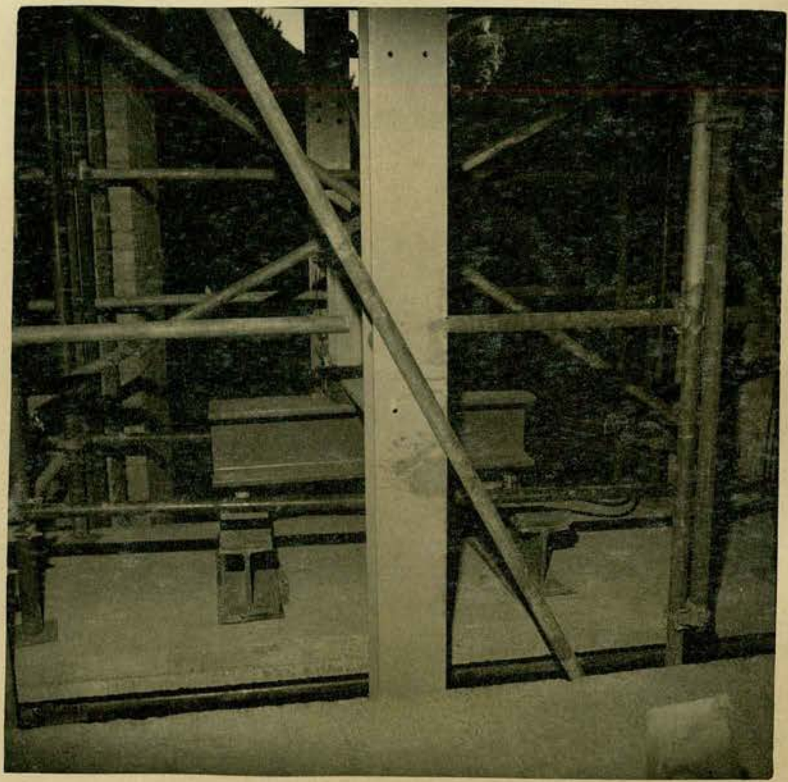


Plate (5.15)

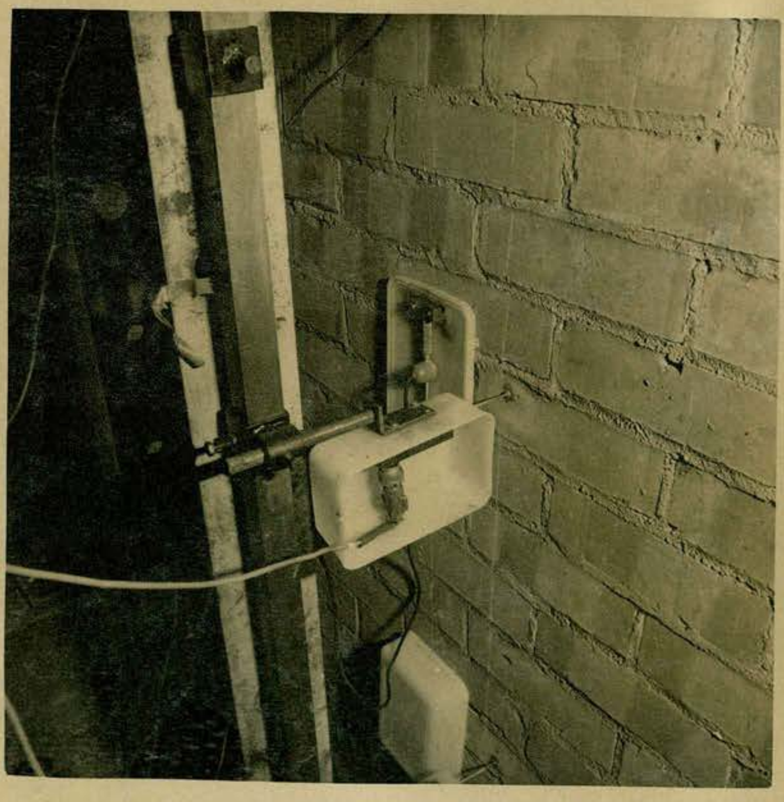


Plate (5.16)

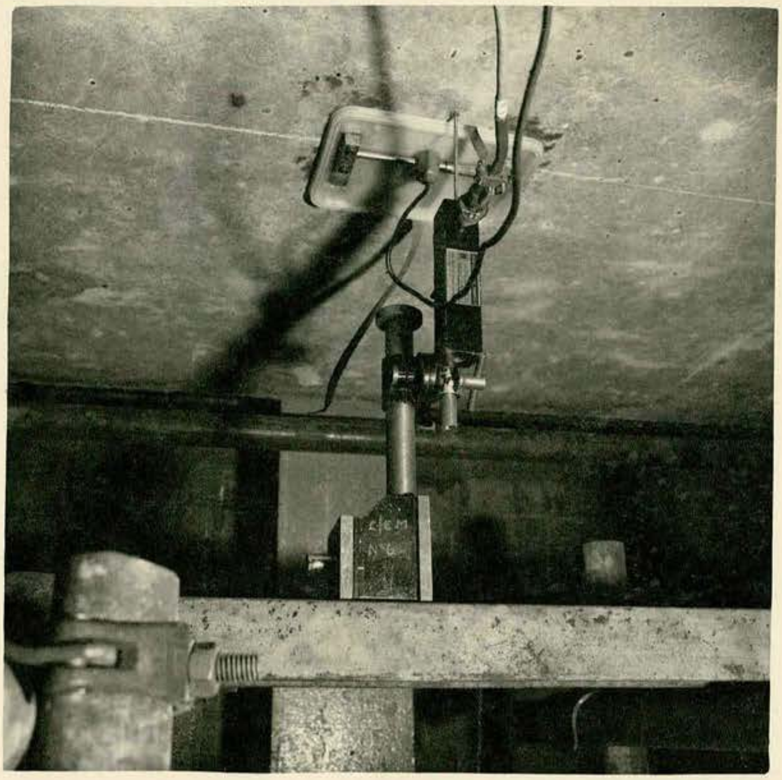


Plate (5.17)

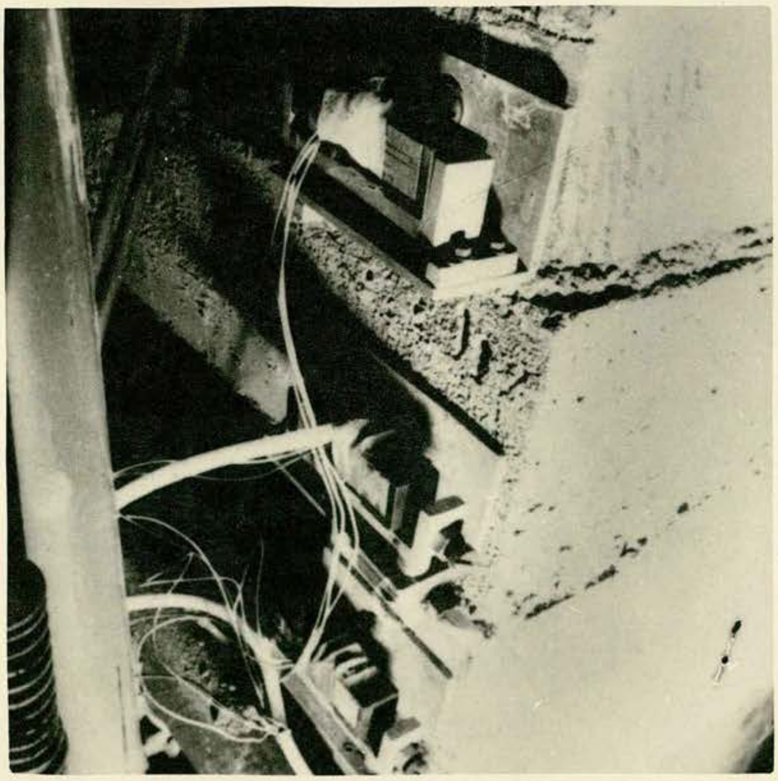


Plate (5.18)

CHAPTER 6 - TEST RESULTS

6.1 INTRODUCTION:-

In this chapter, test results obtained from the half scale model frame, and the full scale test structure discussed in Chapter (5) are presented. From these results, joint eccentricities are computed and compared with the theoretical equations presented in Chapter (4). The test results are presented in two parts, firstly, those corresponding to the half scale model frame, and secondly, those for the full size test structure.

During testing of the half scale model structure, the upper floor slab was loaded only to determine the floor slab flexural rigidity, EI. As the testing was repeated, and the design loads were sometimes exceeded, small cracks on the underneath side of the slab, particularly in the middle third of the span, occurred as a result, and consequently, the central deflection and end rotation of the floor slab became greater for the same magnitude of floor loads resulting in a smaller value of slab flexural rigidity (EI), as will be discussed later.

6.2 HALF SCALE MODEL FRAME:-

6.2.1. Determination of Slab Bending Stiffness:

From test series (A), an estimate of the cracked flexural rigidity of the concrete floor slabs (EI) was possible. Thus, loading only the top floor slab, and since the end restraining moment is so small and may be assumed zero, the following relations are valid from simple mechanics:

$$\Delta_c = \frac{23}{648} \cdot \frac{PL^3}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.1)$$

$$\theta_{end} = \frac{PL^2}{9EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.2)$$

From which, EI, becomes:

$$EI_{\Delta} = 0.0355 \frac{PL^3}{\Delta_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.3)$$

$$EI_{\theta} = 0.111 \frac{PL^2}{\theta_{end}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.4)$$

Values of (EI) calculated from equation (6.3) and (6.4) together with the corresponding values of Δ_c and θ_{end} for various slab loads, P, are presented in Table (6.1).

As mentioned earlier, due to repeated loading and unloading of the upper slab (sometimes the design loads were exceeded), tensile cracking occurred on the underneath side of the slab, particularly in the middle third of the span. As a result, the slab flexural rigidity (EI) was reduced as may be seen from Table (6.1). Thus, an average value of (EI) equal to 337 $KN.m^2$ is adopted and used in all computations involving the reinforced concrete slab flexural rigidity.

From test series (B), the midspan deflection and end rotations of the top slab were measured for several magnitudes of wall pre-compression. Thus, from simple beam theory, the application of equal end moments to a beam of uniform cross-section will induce the following deformations:

$$\Delta_c = ML^2/8EI \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.5)$$

$$\theta_{end} = ML/2EI \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.6)$$

Hence, from equations (6.5) and (6.6), with L = 2.29 metres, the ratio Δ/θ is equal to:

$$\Delta/\theta = L/4 = 572.5 \text{ mm} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.7)$$

However, as may be seen from Table (6.2), the ratio of Δ/θ obtained by tests is smaller than that given by equation (6.7) and does not remain constant.

Since the span length, L, and end moments, M, are the same in both equations (6.5) and (6.6), it is concluded that the reason for the discrepancy between test and theory is due to changes in the flexural rigidity, (EI), of the concrete slab. As a result of these findings, three possibilities exist with respect to equations (6.5) and (6.6), namely, either,

1) Equation (6.5) is incorrect, or,
2) Equation (6.6) is incorrect, or,
3) Equations (6.5) and (6.6) are both incorrect.

Since the magnitudes of the end moments resulting from the application of the wall precompression are not definitely known, it is not possible to say which of these possibilities is correct. However, the reason for this discrepancy between test and theory may be, firstly, that due to loading the upper floor slab in test series (A), the slab was observed to be permanently deflected from its initial position and a slab/wall crack developed and opened up in the joint mortar bed. As the wall precompression was applied in test series (B), the gap between the slab/floor closed rapidly, and hence, the slab end rotations are magnified and the ratio of Δ/θ becomes smaller. The second reason for this discrepancy could be that, as the end moments are increased and the middle of the slab lifts up, tension cracks on the bottom side of the slab tend to close, and with increased wall precompression, tensile cracking occurred on the top side of the slab where there is no tensile reinforcement, thus, causing a reduced flexural rigidity and the ratio of Δ/θ does not comply with that of equation (6.7)

The major reason for the discussion of this data is to show that end moments predicted from equations (6.5) and (6.6) will not be in a very close agreement.

Table (6.1)- Test Series (A)- Half Scale Model.

Slab Load (KN)	Δ_9 (mm)	Δ_{10} (mm)	Δ_{net} (mm)	θ_{slab} $x10^{-3}$ Radians	EI_{Δ} Eq.6.3 $KN.M^2$	EI_{θ} Eq.6.4 $KN.M^2$	% Diff.
First Test							
1.0	+ 1.13	- 0.05	+ 1.08	- 1.65	394.73	353.13	11.78
1.5	+ 1.62	- 0.07	+ 1.55	- 2.20	412.56	397.28	3.84
2.0	+ 2.18	- 0.09	+ 2.09	- 2.80	409.96	416.19	2.01
2.5	+ 2.76	- 0.11	+ 2.65	- 3.40	402.18	428.43	6.52
3.0	+ 3.95	- 0.13	+ 3.82	- 4.50	334.80	388.45	16.02
				Average	390.44	396.69	1.60
Second Test							
1.0	+ 1.40	- 0.05	+ 1.35	- 1.80	315.79	323.70	2.50
1.5	+ 2.04	- 0.08	+ 1.96	- 2.55	326.26	342.75	5.05
2.0	+ 2.67	- 0.11	+ 2.56	- 3.35	333.06	347.86	4.44
2.5	+ 3.30	- 0.13	+ 3.17	- 3.90	336.21	373,50	11.09
3.0	+ 4.18	- 0.15	+ 4.03	- 4.90	317.36	356.74	12.40
				Average	325.73	348.91	7.11
Third Test							
1.0	+ 1.64	- 0.16	+ 1.48	- 2.08	288.05	280.13	2.82
1.5	+ 2.56	- 0.26	+ 2.30	- 3.05	278.03	286.50	3.04
2.0	+ 3.67	- 0.40	+ 3.36	- 4.00	253.76	290.60	14.51
2.5	+ 4.56	- 0.52	+ 4.04	- 4.75	263.81	306.00	15.99
3.0	+ 5.58	- 0.65	+ 4.93	- 6.75	259.42	258.58	0.17
				Average	269.39	285.58	6.00

Table (6.2)- Test Series (B)- Half Scale Model.

Active Wall Precomp. (KN)	Total Wall Precomp. (N/mm^2)	Upper Slab Rotation $x10^{-3}$ (Rad)	Δ_9 Upper Slab Deflection (mm)	Δ_{10} Lower Slab Deflection (mm)	Δ_{net} Net Upper Slab Defl. (mm)	Δ/θ (Theory)	Δ/θ (Test)	% Difference
10	0.196	+ 1.47	- 0.04	- 0.49	- 0.53	572.5	360.54	58.78
20	0.391	+ 4.00	- 0.93	- 0.73	- 1.66	=	415.00	37.95
30	0.587	+ 5.50	- 1.57	- 0.82	- 2.39	=	434.54	31.74
40	0.783	+ 6.60	- 1.95	- 0.84	- 2.79	=	422.77	35.43
50	0.979	+ 7.30	- 2.11	- 0.82	- 2.93	=	401.36	42.64
60	1.175	+ 7.70	- 2.29	- 0.65	- 2.94	=	381.81	49.94
70	1.371	+ 7.90	- 2.28	- 0.59	- 2.87	=	363.29	57.58
80	1.567	+ 7.90	- 2.26	- 0.53	- 2.79	=	353.16	62.10

6.2.2. Slab Deflections and Rotations:

The most significant data recorded during the test programme were the midspan displacement and end rotations of the lower floor slab. In test series (C), only the lower floor was loaded, whereas in test series (D) and (E), the wall precompression were applied prior to loading the lower slab and in test series (F) and (G) the floor slab was loaded prior to precompressing the walls.

In test series (H) and (J), both upper and lower slabs were loaded prior to application of wall precompression, thus, these tests are identical to test series (F) and (G) as far as the deformation of the lower floor slab is concerned and hence these deformations and the corresponding computation of joint eccentricities are not presented, and only the walls deformation are included.

In test series (K), although both slabs were loaded identically to test series (J), the magnitude of wall precompression applied varied from that for all other tests. In this series, the increments of wall precompression applied represents the total force above the joint corresponding to each additional storey height (each increment equals the dead load of, slab, brick wall, loading beams and jacks, and the live load from jacks).

Test data for test series (C), (D), (E), (F), (G) and (K) are presented in tables (B1) through (B6) in Appendix (B).

6.2.3 Moments due to wall precompression:

The slab restraining moments due to application of wall precompression may be obtained from simple mechanics, whence, the relation between end moments, midspan displacement and end rotation are:

$$M = 8EI\ \Delta/L^2 \quad (6.8)$$

$$M = 2EI\ \theta/L \quad (6.9)$$

Where, M is the slab restraining moment due to wall precompression, Δ and θ, are the deformations caused by the application of wall precompression.

Slab restraining moments due to application of wall precompression were calculated using equations (6.8) and (6.9) based on midspan deflection and slab end rotation respectively. In all tests where the floor slab was loaded prior to application of wall precompression, the slab restraining moments developed due to precompressing the wall(s) were greater than those where the wall precompression was applied prior to loading the floor slab. This is because the deformations recorded due to wall precompression in the former tests were greater than those in the latter, particularly at high precompression stress.

6.2.4 Moments due to floor loads:

The slab restraining moments due to floor load application may be computed using the following relations from simple mechanics (20);

1) Based on midspan deflection,

$$M = 0.284\ PL - 8\ EI\ \Delta/L^2 \quad (6.10)$$

2) Based on slab end rotation,

$$M = 2\ PL/9 - 2\ EI\ \theta/L \quad (6.11)$$

Where M, is the slab restraining moment (joint moment) due to floor loads application P, the deformations Δ and θ are those due the slab loads P, which are partially restrained due to the end moments induced at the joints.

Using $E_{wall} = 5.04$ KN/mm^2 from the brick prism test, and assuming uncracked wall, a moment distribution analysis of the frame structure indicates that the rigid frame slab restraining moments would be 0.625 KN.m. and 1.25 KN.m. for floor loads of 1.5 KN/load point and 3.0 KN/load point respectively.

6.2.5. Total Moments:

Total restraining moments are computed using both slab deflection and rotation data, thus, for example, the total restraining moment based on deflection data is obtained by the sum of equations (6.8) and (6.10) and noting that, Δ, in equation (6.8) represents the upward deflection of the slab due to the application of wall precompression, and in equation (6.10) represents the downward deflection due to loading the floor slab. Similarly, total slab restraining moments based on rotation data are obtained from the sum of equations (6.9) and (6.11).

6.2.6 Joint Rigidity:

Determination of the joint fixity is obtained by dividing the slab restraining moments caused by floor loading (i.e. joint moments) obtained from tests by the corresponding rigid frame moments obtained by structural analysis. Thus, this ratio defines the joint rigidity and capability to transfer the slab moments to the supporting walls, or in other words, the capability of the joint to develop end moments as a percentage of that of a fully rigid frame. Joint rigidity plotted as a function of wall precompression is shown in figure (6.1) for the half scale testing frame.

Theoretical joint eccentricities are computed from equation (4.26) and are compared with experimental results in Tables (6.3) through (6.8). To facilitate the comparisons, some of these results are plotted in Figure (6.2).

Computations of the joint eccentricities from test data are obtained by dividing the experimental slab restraining moments by the sum of the load at the base of the wall above the floor and the load at the top of the wall below the floor (i.e. the sum of P_u and P_L).

It should be noted that, in computing ε from equation (4.26), a value of $\overline{K}$ = 0.661 was used which is based on the slab flexural rigidity EI equals to 337 $KN.m^2$.

6.2.7 <u>Wall deflections and rotations</u>:

Deflection of the brick walls under various loading conditions are given in Tables (B7) through (B16) in Appendix (B) and are also presented in Figures (6.3) through (6.12) respectively. As may be noted from these figures, wall deflections are relatively small and the deflection pattern for both walls is not identical as may be expected from theoretical analysis.

Rotation data of the upper and lower brick walls are presented in Table (B17) for test series A, B and C, and in Tables (B18) through (B21) for the other tests in Appendix (B).

Table (6.3)- Test Series (C) - Joint Moments and Eccentricities - Half Scale Model.

Total Wall Prec. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Av.Moment Due to F.L. (Tests) (M_o)	Theoretical Moment Eq.(4.24) $\bar{M}_R x(\psi/\acute{\psi})$	% Diff.	% of Joint Fixity $=(M_o/\bar{M}_R)x100$	Joint Eccent. (Tests)	Joint Eccent. (Theory) Equat. (4.26)	% Diff.
.0715	3.72	7.15	.520	.115	.142	23.47	27.64	.094	.098	4.25
.0715	3.72	7.65	.486	.201	.204	1.50	32.16	.157	.135	16.30
.0715	3.72	8.15	.456	.240	.260	8.30	28.81	.180	.164	9.75
.0715	3.72	8.65	.430	.265	.312	17.70	25.48	.191	.177	7.90
.0715	3.72	9.15	.406	.308	.360	16.90	24.64	.213	.183	16.40

Table (6.4)- Test Series (D)- Half Scale Model.

Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
.267	.153	.190	24.18	.333	.312	6.70	.322	.480	.502	4.50	0.491
.462	.194	.252	29.90	.466	.430	8.30	.448	.660	.682	3.30	0.671
.658	.217	.272	25.34	.489	.507	3.60	.498	.706	.778	10.10	0.742
.854	.232	.267	15.00	.516	.538	4.20	.527	.748	.804	7.40	0.776
1.050	.253	.257	1.58	.539	.559	3.70	.549	.792	.816	3.00	0.804
1.246	.264	.236	11.80	.557	.574	3.00	.565	.821	.810	1.30	0.815
1.442	.264	.216	22.20	.574	.589	2.60	.581	.838	.805	4.00	0.821
1.638	.256	.200	28.00	.580	.600	3.40	.590	.836	.810	3.20	0.823

Table (6.4) - Continued.

Total Wall Prec. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests) *	Joint Eccentricity (Theory)	% Diff.
.267	13.72	17.65	.777	.322	.625	51.52	.0916	.0930 **	1.56
.462	23.72	27.65	.857	.448	.625	71.69	.0778	.0800	3.96
.658	33.72	37.65	.895	.498	.625	79.69	.0623	.0600	2.80
.854	43.72	47.65	.917	.527	.625	84.33	.0515	.0490	5.10
1.050	53.72	57.65	.931	.549	.625	87.85	.0440	.0400	9.45
1.246	63.72	67.65	.942	.565	.625	90.41	.0384	.0340	11.95
1.442	73.72	77.65	.949	.581	.625	92.97	.0342	.0300	11.76
1.638	83.72	87.65	.955	.590	.625	94.41	.0307	.0270	13.70

* Based on $\bar{K}$= 0.66

** Average of two values.

Table (6.5)- Test Series (E)- Half Scale Model.

Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
.267	.153	.190	24.18	.622	.593	4.90	.607	.775	.783	1.03	0.779
.462	.194	.252	29.89	.846	.907	7.20	.876	1.040	1.159	11.44	1.099
.658	.217	.272	25.34	.970	1.020	5.15	.995	1.187	1.292	8.84	1.239
.854	.232	.267	15.08	1.073	1.092	1.77	1.083	1.305	1.359	4.13	1.332
1.050	.253	.257	1.58	1.096	1.128	2.91	1.112	1.349	1.385	2.66	1.367
1.246	.264	.236	11.86	1.117	1.164	4.20	1.140	1.381	1.400	1.37	1.390
1.442	.264	.216	22.22	1.132	1.133	0.08	1.132	1.396	1.349	3.48	1.372
1.638	.256	.200	28.00	1.146	1.102	4.00	1.124	1.402	1.302	7.68	1.352

Table (6.5) - Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
.267	13.72	19.15	.716	0.607	1.25	48.60	.1648	.1650*	0.08
.462	23.72	29.15	.813	0.876	1.25	70.12	.1480	.1480	0.00
.658	33.72	39.15	.861	0.995	1.25	79.60	.1220	.1140	7.00
.854	43.72	49.15	.889	1.082	1.25	86.60	.1040	.0920	13.00
1.050	53.72	59.15	.908	1.112	1.25	88.96	.0880	.0780	12.67
1.246	63.72	69.15	.921	1.140	1.25	91.24	.0766	.0670	13.64
1.442	73.72	79.15	.931	1.132	1.25	90.60	.0660	.0590	11.86
1.638	83.72	89.15	.939	1.124	1.25	89.92	.0580	.0520	11.53

* Average of two values.

Table (6.6) - Test Series (F)- Half Scale Model.

Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
.0715	0.000	0.000	0.00	.159	.173	8.80	.166	.159	.173	8.80	.166
.2670	.191	.144	32.63	.333*	.312*	6.70	.322	.524	.456	14.91	.490
.4620	.294	.226	30.08	.466	.430	8.30	.448	.760	.656	15.85	.708
.6580	.332	.282	17.73	.489	.507	3.60	.498	.821	.789	4.05	.805
.8540	.356	.308	15.58	.516	.538	4.20	.527	.872	.846	3.07	.859
1.0500	.361	.329	9.72	.539	.559	3.70	.549	.900	.888	1.35	.894
1.2460	.373	.339	10.00	.557	.574	3.00	.565	.930	.913	1.86	.921
1.4420	.373	.329	13.37	.574	.589	2.60	.581	.947	.918	3.15	.932
1.6380	.356	.329	8.20	.580	.600	3.40	.590	.936	.929	0.75	.932

* From Test Series (D).

Table (6.6) - Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (F)	Total Moments Test Series (D)	% Diff.
.0715	3.72	7.65	.486	.166	.625	26.56	.166	---	---
.2670	13.72	17.65	.777	.322	=	51.52	.490	.491	.20
.4620	23.72	27.65	.857	.448	=	71.69	.708	.671	5.51
.6580	33.72	37.65	.895	.498	=	79.69	.805	.742	8.49
.8540	43.72	47.65	.917	.527	=	84.33	.859	.776	10.69
1.0500	53.72	57.65	.931	.549	=	87.85	.894	.804	11.19
1.2460	63.72	67.65	.942	.565	=	90.41	.921	.815	13.16
1.4420	73.72	77.65	.949	.581	=	92.97	.932	.821	13.58
1.6380	83.72	87.65	.955	.590	=	94.41	.932	.823	13.30

Table (6.7)- Test Series (G)- Half Scale Model.

Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
.0715	.000	.000	---	0.319	0.285	11.92	0.302	0.319	0.285	11.92	0.302
.267	.235	.221	6.33	0.622*	0.593*	4.89	0.607	0.857	0.814	5.28	0.835
.462	.338	.318	6.28	0.846	0.907	7.21	0.876	1.184	1.225	3.46	1.204
.658	.412	.390	5.64	0.970	1.020	5.15	0.995	1.382	1.410	2.02	1.396
.854	.441	.421	4.75	1.073	1.092	1.77	1.082	1.514	1.513	0.06	1.513
1.050	.441	.426	3.52	1.096	1.128	2.91	1.112	1.537	1.554	1.10	1.545
1.246	.470	.436	7.80	1.117	1.164	4.20	1.140	1.587	1.600	0.82	1.593
1.442	.470	.442	6.33	1.132	1.133	0.08	1.132	1.602	1.575	1.71	1.588
1.638	.441	.436	1.14	1.146	1.102	3.99	1.124	1.587	1.538	3.18	1.562

* From Test Series (E)

Table (6.7) - Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (Ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (G)	Total Moments Test Series (E)	% Diff.
.0715	3.72	9.15	.406	0.302	1.250	24.16	0.302	-----	-----
.267	13.72	19.15	.716	0.607	=	48.60	0.835	0.779	7.25
.462	23.72	29.15	.813	0.876	=	70.12	1.204	1.099	9.59
.658	33.72	39.15	.861	0.995	=	79.60	1.396	1.239	12.67
.854	43.72	49.15	.889	1.082	=	86.60	1.513	1.332	13.62
1.050	53.72	59.15	.908	1.112	=	88.96	1.545	1.367	13.05
1.246	63.72	69.15	.921	1.140	=	91.24	1.593	1.390	14.64
1.442	73.72	79.15	.931	1.132	=	90.60	1.588	1.372	15.77
1.638	83.72	89.15	.939	1.124	=	89.92	1.562	1.352	15.56

Table (6.8)- Test Series (K)- Half Scale Model.

No.of Floors Above Joint	Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity
		M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ			
1	0.130	0.000	0.000	---	0.364	0.285	27.71	0.324	0.364	0.285	27.71	1.25	25.96
2	0.263	0.159	0.215	35.22	0.628	0.578	8.65	0.603	0.787	0.793	0.76	=	48.24
3	0.395	0.235	0.298	26.80	0.758	0.799	5.40	0.778	0.993	1.097	10.47	=	62.28
4	0.562	0.288	0.344	19.44	0.873	0.922	5.60	0.897	1.161	1.266	9.04	=	71.80
5	0.658	0.353	0.385	9.06	0.967	1.020	5.50	0.993	1.320	1.405	6.44	=	79.48
6	0.789	0.412	0.416	0.97	1.040	1.066	2.50	1.053	1.452	1.482	2.06	=	84.24
7	0.921	0.470	0.426	10.32	1.084	1.097	1.20	1.090	1.554	1.523	2.03	=	87.24
8	1.053	0.456	0.431	5.80	1.099	1.123	2.20	1.111	1.555	1.554	0.06	=	88.88
9	1.185	0.441	0.436	1.14	1.108	1.143	3.15	1.125	1.549	1.579	1.93	=	90.04
10	1.316	0.470	0.436	7.79	1.120	1.154	3.00	1.137	1.590	1.590	0.00	=	90.96
11	1.448	0.470	0.442	6.33	1.132	1.154	2.00	1.143	1.602	1.596	0.37	=	91.44
12	1.580	0.485	0.436	11.23	1.143	1.144	0.08	1.143	1.628	1.580	3.03	=	91.44
13	1.711	0.485	0.436	11.23	1.143	1.149	0.52	1.146	1.628	1.585	2.71	=	91.68
14	1.843	0.485	0.436	11.23	1.137	1.149	1.05	1.143	1.622	1.585	2.33	=	91.44

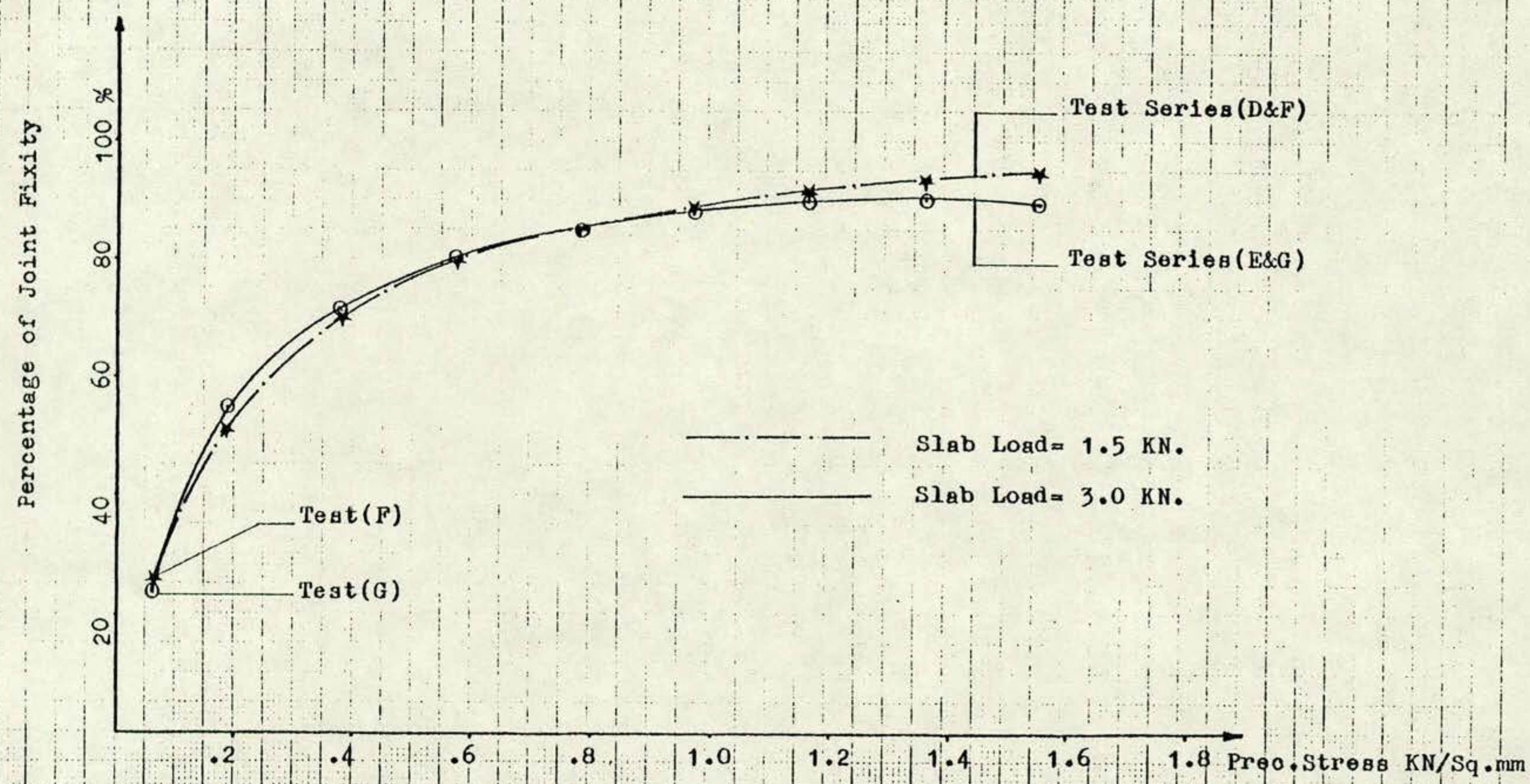


Fig.(6.1)

N.B. Tests Series(D&E),Precompression Applied Prior to Loading Lower Slab(1.5 & 3.0 KN)

Tests Series(F&G),Loading Lower Slab Prior to Application of Precompression.

The Precompression Stress represents the'Active Precompression'from Jacks.

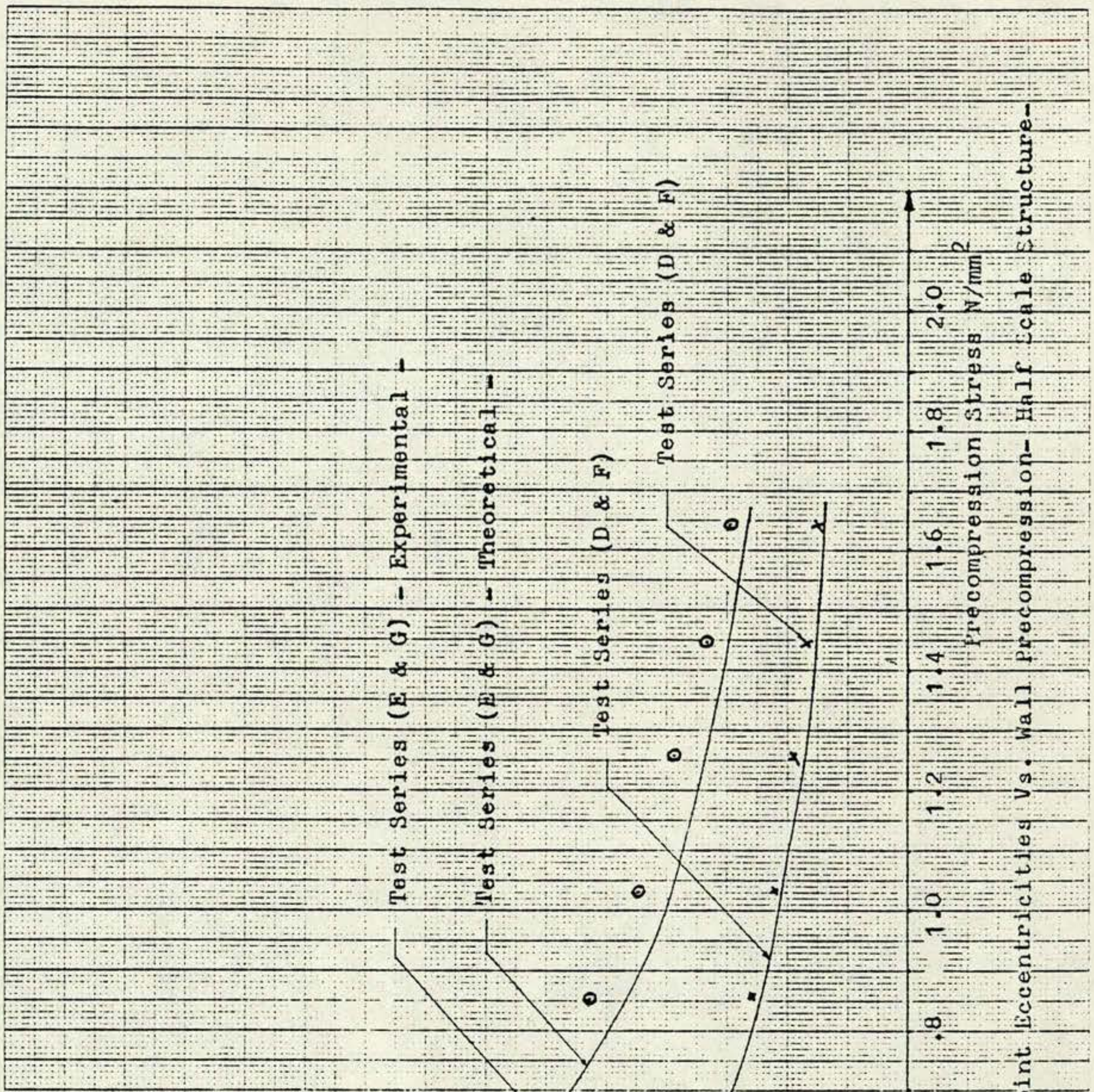
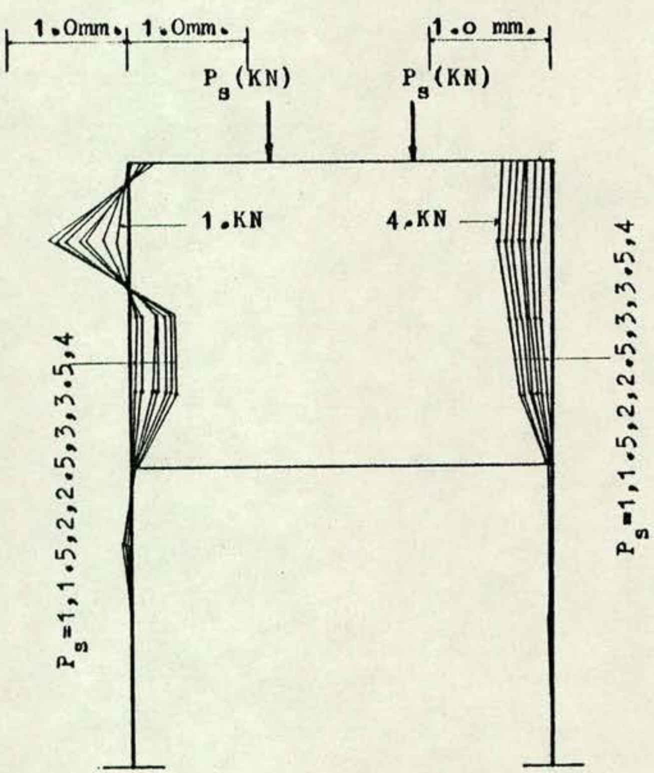
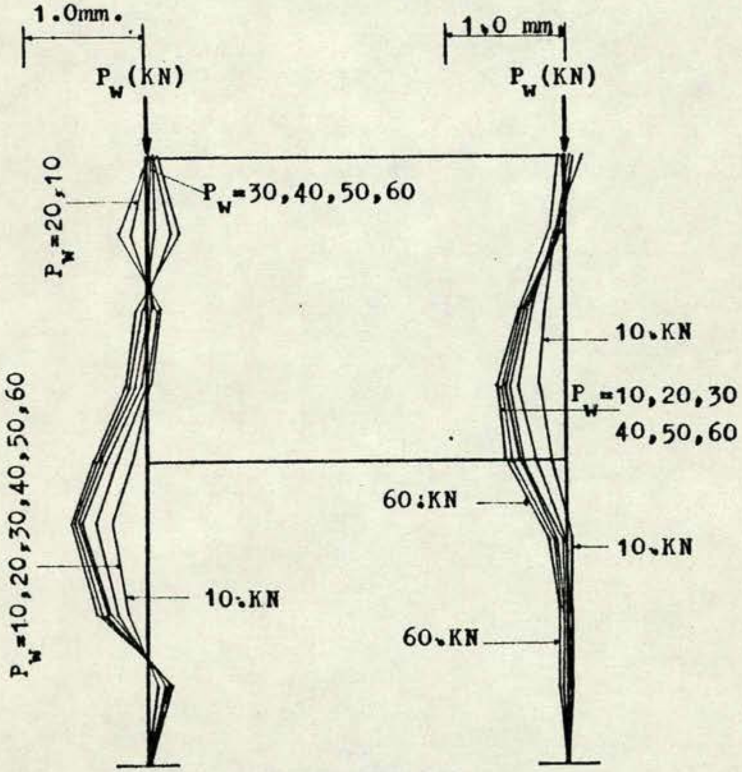


Fig.(6.2)- Joint Eccentricities Vs. Wall Precompression- Half Scale Structure-



Test Series (A)

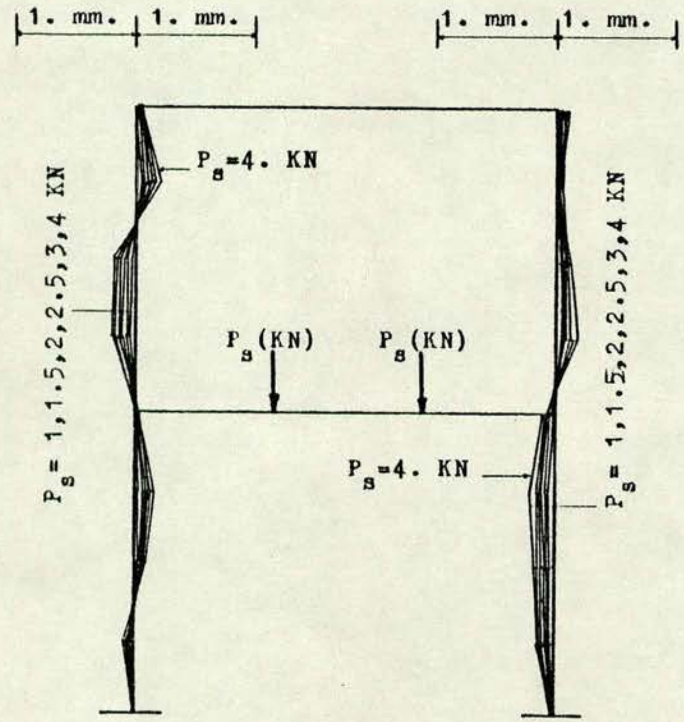
Fig.(6.3)



Test Series (B)

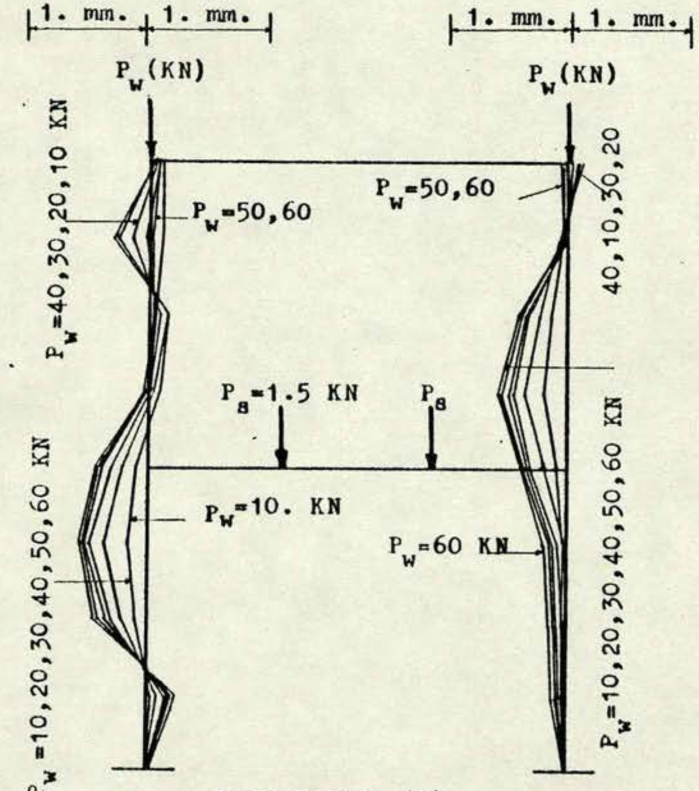
Fig.(6.4)

N.B. Wall DEFLECTION CURVES are Ploted as Straight Lines for an easier tracing.



Test Series (C)

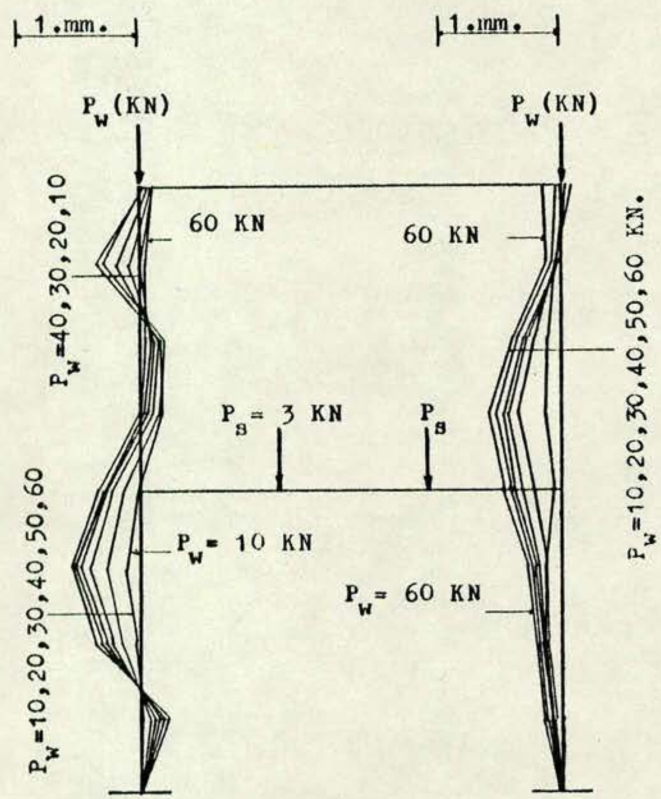
Fig.(6.5)



Test Series (D)

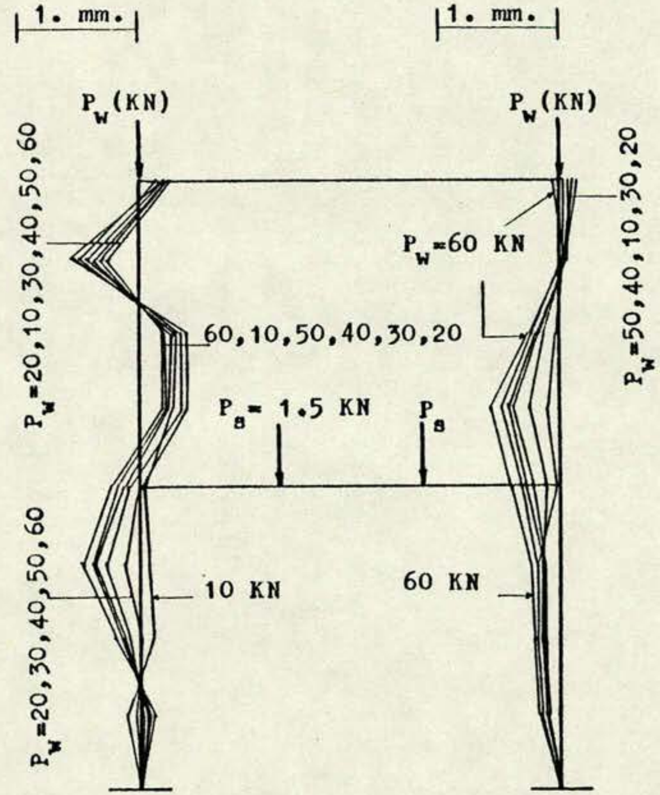
Fig.(6.6)

N.B. For Test Series (D), PRECOMPRESSION Applied Prior to Loading Lower Slab.



Test Series (E)

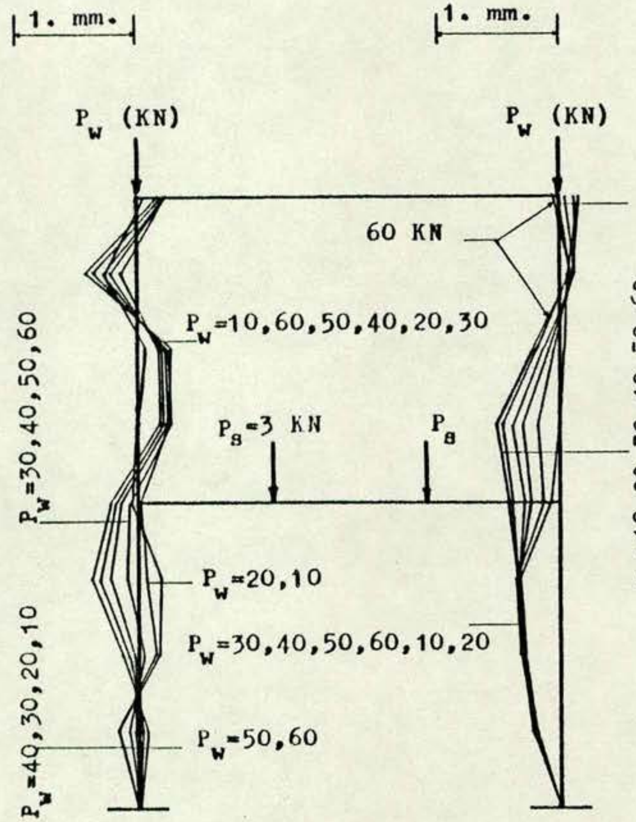
Fig.(6.7)



Test Series (F)

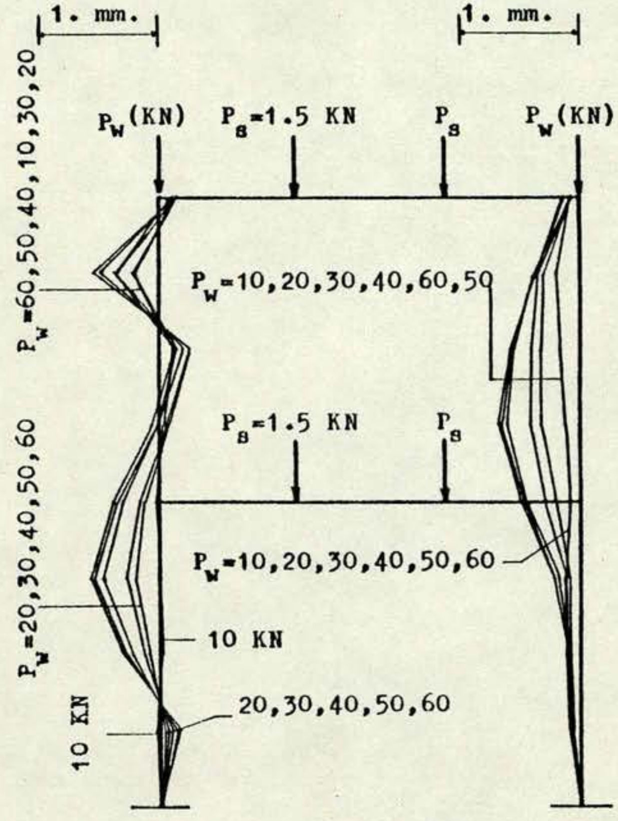
Fig.(6.8)

N.B. For Test Series (E),PRECOMPRESSION Applied Prior to Loading Lower Slab.
For Test Series (F),Loading Lower Slab Prior To Application of PRECOMPRESSION.



Test Series (G)

Fig.(6.9)

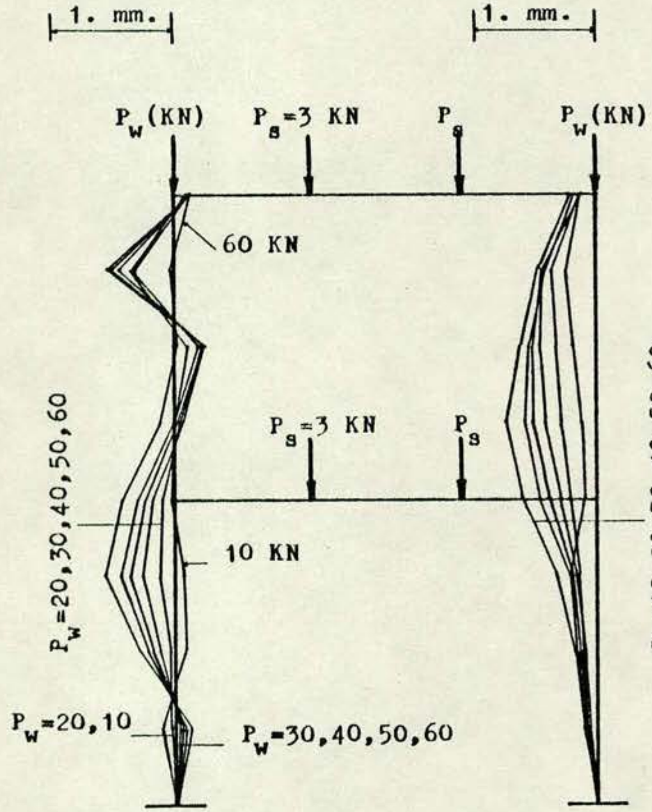


Test Series (H)

Fig.(6.10)

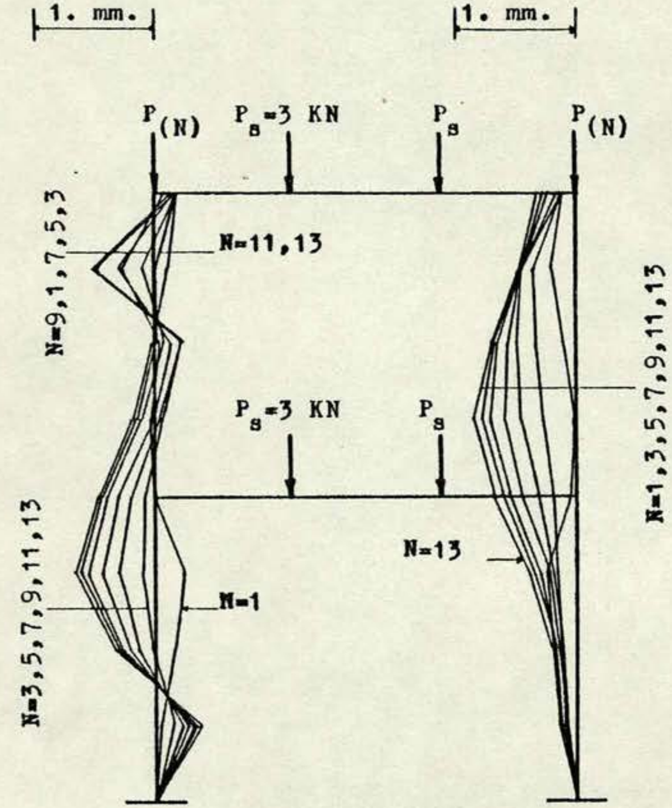
N.B. For Test Series(G), Loading Lower Slab Prior to Application of PRECOMPRESSION.

For Test Series(H), Loading Lower & Upper Slabs Prior to Application of PRECOMPRESSION.



Test Series (J)

Fig.(6.11)



Test Series (K)

Fig.(6.12)

N.B. For Test Series(K), N Represents the TOTAL Load Per One Storey..

6.3 FULL SCALE TEST STRUCTURE:

6.3.1 Determination of slab bending stiffness:

An estimate of the cracked floor slab flexural rigidity (EI) may be computed by loading the floor slab only, and obtaining the midspan deflection and end rotation corresponding to the magnitude of floor loads applied. Based on analysis of the continuous two span floor slab, with one span loaded only by point loads at third points, the following relations are obtained:

$$\Delta_c = 65\ PL^3/2592\ EI \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.12)$$

$$\theta_{end} = PL^2/12\ EI \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.13)$$

From which,

$$EI_\Delta = 65\ PL^3/2592\Delta_c \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.14)$$

$$EI_\theta = PL^2/12\ \theta_{end} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.15)$$

Where, Δ_c and θ_{end}, are the maximum midspan deflection and end rotation of the loaded floor slab respectively. With, P = 2 KN/point load, the following deformations were recorded:

$$\Delta_c = 0.48\ \text{mm}$$

$$\theta_{end} = 0.48 * 10^{-3}\ \text{Radians.}$$

Substituting these deformations in equations (6.14) and (6.15) gives:-

$$EI_\Delta = 3985.72\ \text{KN.m}^2$$

$$EI_\theta = 3785.83\ \text{KN.m}^2$$

The difference between (EI) obtained from equations (6.14) and (6.15) is 5.28%, and the average of the two equations sets the value of (EI) equal to 3885.77 KN.m^2, which will be used in all computations throughout.

6.3.2 Slab Deflections and Rotations:

The lower floor slab midspan displacement and end rotations are presented in Tables (B22) through (B33) for all test series, except (A), which is presented in Table (6.9).

In test series (A), only the floor slab was loaded in order to determine the joint rigidity and eccentricities without the application of wall precompression. In test series, B, C, D, E and F, the wall precompression was applied prior to loading the floor slab, simulating a cast-in situ type of construction, whereas, in test series G, H, J, K and L, floor loads were applied prior to precompressing the wall, which represents a precast floor construction analogy.

In test series M, N, and P, a constant wall precompression was applied in each test prior to loading the floor with variable loads to investigate the effect of applying variable live loads on joint eccentricities and the possible reduction in joint rigidity. Test series (R) is identical to test series M, N, and P, except that, the wall precompression was increased to 600 KN, and the floor loads were increased gradually in order to reach the "joint failure" limit. However, as the floor loads were increased, the joint never failed, but, severe tensile cracking of the reinforced concrete slab floor occurred at the bottom of the slab at the middle third of the span, and the concentration of these cracks was nearer to the precompressed wall, followed by cracking on the top side of the slab in the negative bending moment zone at the span ends. These cracks are illustrated in Plates (6.1) and (6.2) showing the bottom cracks, and Plates (6.3) and (6.4) showing the top cracks at each end of the floor slab.

Table (6.9)- Test Series (A)- Full Size Structure.

Slab Point Load (KN)	Slab End Rot $\times 10^{-3}$ (Rad.)	Slab Def. (mm.)	Moment Due to Floor Load (KN.M)		Average Moment Due to F.Load (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccent. (test)	Joint Eccentricity (Theory)	% Diff.
			M_θ	M_Δ						
3.00	- 0.58	+ 0.61	0.287	0.241	0.264	1.268	20.82	0.0431	0.0432	0.23
6.00	- 1.20	+ 1.24	0.480	0.426	0.453	2.537	17.85	0.0705	0.0745	5.67
9.00	- 1.80	+ 1.84	0.720	0.696	0.708	3.806	19.71	0.1050	0.0889	18.22
12.00	- 2.45	+ 2.49	0.842	0.823	0.832	5.075	16.40	0.1182	0.1137	3.95
15.00	- 3.10	+ 3.16	0.965	0.894	0.929	6.344	14.65	0.1264	0.1250	1.12

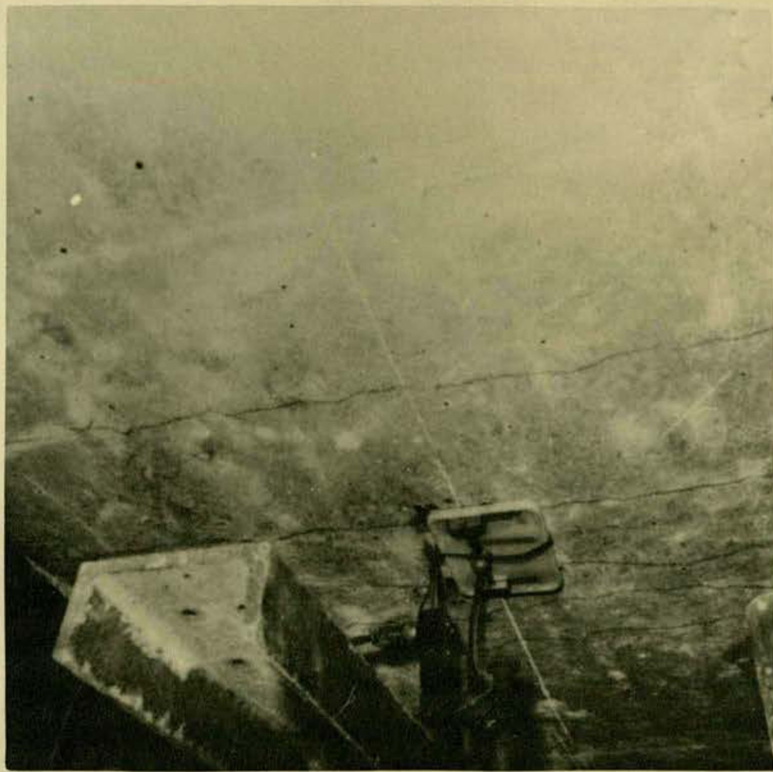


Plate (6.1)

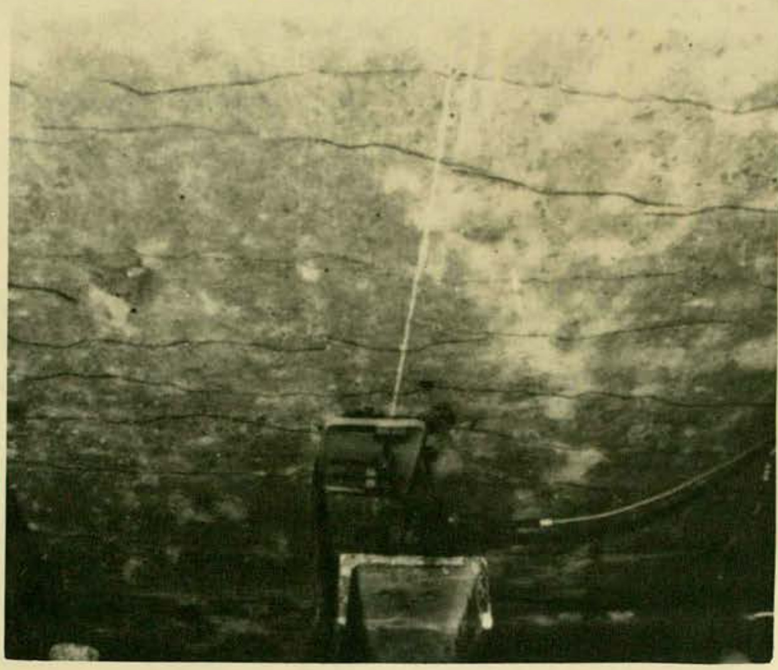


Plate (6.2)

Plate (6.3)

Plate (6.4)

6.3.3 Moments due to wall precompression:

As only one exterior wall was subjected to "Active precompression", the slab restraining moments due to wall precompression based on the analysis of a continuous two-span beam, may be given by:

1) Based on midspan deflection,

$$M = 16\ EI\ \Delta/L^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.16)$$

2) Based on slab end rotation,

$$M = 3\ EI\ \theta/L \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6.17)$$

Where, M, is the slab restraining moment, Δ and θ, are the deformations induced by the application of this end moment through precompressing the external wall only.

Equations (6.16) and (6.17) gives the slab restraining moments due to wall precompression only.

6.3.4 Moments due to floor loads:

The slab restraining moments due to floor loading, based on the analysis of a two span beam with one span loaded only by point loads at the third points of the span is given as:

1) Based on midspan deflection,

$$M = 0.2\ PL - 8\ EI\ \Delta/L^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.18)$$

2) Based on slab end rotation,

$$M = PL/6 - 2\ EI\ \theta/L \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.19)$$

Where the second part of equations (6.18) and (6.19) represents the end moments that will develop at the joint due to the mid-span deflection and end rotation caused by the application of the floor loads.

Using $E_{wall} = 11.7\ KN/mm^2$ from the brick prism test, and assuming uncracked wall, a moment distribution of the frame structure indicates that the rigid frame slab restraining moments are:

1) $\bar{M}_R = 1.268$ KN.m. (P = 3 KN/Point Load)

2) $\bar{M}_R = 2.537$ KN.m. (P = 6 " " ")

3) $\bar{M}_R = 3.806$ KN.m. (P = 9 " " ")

4) $\bar{M}_R = 5.075$ KN.m. (P = 12 " " ")

5) $\bar{M}_R = 6.344$ KN.m. (P = 15 " " ")

Having computed the rigid frame moment for one case of loading of the above, the remaining rigid frame moments were obtained as a multiple of the ratio of loading between these cases.

6.3.5 Total Moments:

Total slab restraining moments are computed once again using both slab deflection and rotation data in a similar fashion as discussed in Section (6.2.5) by using equations (6.16), (6.18) and (6.17), (6.19) based on deflection and rotation data respectively.

6.3.6 Joint Rigidity:

The joint fixity, which is a measure of the joint rigidity, is obtained by dividing the slab restraining moments due to floor loading by the corresponding rigid frame moment obtained by structural analysis.

The joint fixities were computed together with the joint eccentricities(as defined by equation (4.26))and the slab restraining moments and are presented in Tables (6.9) through (6.23). The percentage of joint fixity obtained by tests is shown in Figure (6.13). Theoretical and experimental comparisons of the joint eccentricities are shown in Figure (6.14).

Table (6.10)- Test Series (B)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.353	0.734	0.684	7.30	0.310	0.327	5.48	0.318	1.044	1.011	3.26	1.027
0.514	0.946	0.855	10.46	0.334	0.355	6.28	0.344	1.280	1.210	5.78	1.245
0.675	1.440	1.254	14.83	0.357	0.412	15.40	0.384	1.797	1.666	7.86	1.731
0.836	1.793	1.539	16.50	0.381	0.441	15.74	0.411	2.174	1.980	9.79	2.077
0.997	1.863	1.596	11.72	0.428	0.441	3.03	0.434	2.291	2.037	12.46	2.164
1.158	2.075	1.824	13.76	0.428	0.469	9.57	0.448	2.503	2.293	9.15	2.398
1.318	2.146	1.881	14.08	0.475	0.498	4.84	0.486	2.621	2.379	10.17	2.500
1.479	2.146	1.938	10.73	0.522	0.469	11.30	0.495	2.668	2.407	10.84	2.537
1.640	2.322	1.995	16.39	0.522	0.469	11.30	0.495	2.844	2.464	15.42	2.654
1.801	2.322	1.995	16.39	0.522	0.469	11.30	0.495	2.844	2.464	15.42	2.654
1.962	2.287	1.995	14.63	0.522	0.469	11.30	0.495	2.809	2.464	14.00	2.636
2.123	2.322	2.052	13.15	0.475	0.469	1.27	0.472	2.797	2.521	10.94	2.659

Table (6.10)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
0.353	44.00	56.00	0.785	0.318	1.268	25.11	0.0311	0.0320	2.89
0.514	64.00	76.00	0.842	0.344	=	27.16	0.0240	0.0250	4.16
0.675	84.00	96.00	0.875	0.384	=	30.32	0.0209	0.0204	2.45
0.836	104.00	116.00	0.896	0.411	=	32.41	0.0183	0.0173	5.78
0.997	124.00	136.00	0.911	0.434	=	34.26	0.0163	0.0149	9.39
1.158	144.00	156.00	0.923	0.448	=	35.37	0.0146	0.0132	10.60
1.318	164.00	176.00	0.931	0.486	=	38.36	0.0140	0.0118	18.64
1.479	184.00	196.00	0.938	0.495	=	39.07	0.0127	0.0106	19.81
1.640	204.00	216.00	0.944	0.495	=	39.07	0.0115	0.0097	18.55
1.801	224.00	236.00	0.949	0.495	=	39.07	0.0105	0.0088	19.31
1.962	244.00	256.00	0.953	0.495	=	39.07	0.0097	0.0083	16.86
2.123	264.00	276.00	0.956	0.472	=	37.22	0.0085	0.0077	11.16

Table (6.11)- Test Series (C)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.353	0.617	0.627	1.62	0.621	0.625	0.64	0.623	1.238	1.252	1.13	1.245
0.514	0.960	0.912	5.26	0.604	0.682	12.91	0.643	1.564	1.594	1.91	1.579
0.675	1.235	1.254	1.53	0.679	0.711	4.71	0.695	1.914	1.965	2.66	1.939
0.836	1.464	1.539	5.12	0.691	0.711	2.89	0.701	2.155	2.250	4.40	2.202
0.997	1.553	1.653	6.43	0.656	0.739	12.65	0.697	2.209	2.392	8.28	2.300
1.158	1.599	1.824	14.07	0.696	0.739	6.17	0.717	2.295	2.563	11.67	2.429
1.318	1.666	1.881	12.90	0.675	0.739	9.48	0.707	2.341	2.620	11.91	2.480
1.479	1.712	1.938	13.20	0.715	0,711	0.56	0.713	2.427	2.649	9.14	2.538
1.640	1.761	1.938	10.05	0.764	0.739	3.38	0.751	2.525	2.677	6.00	2.601
1.801	1.846	1.995	8.07	0.755	0.739	2.16	0.747	2.601	2.734	5.11	2.667
1.962	1.870	1.995	6.68	0.762	0.739	3.11	0.750	2.632	2.734	3.87	2.683
2.123	1.941	1.995	2.78	0.820	0.739	10.96	0.779	2.761	2.734	0.98	2.747

Table (6.11)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
0.353	44.00	59.00	0.748	0.623	2.537	24.55	0.0593	0.0581	2.06
0.514	64.00	79.00	0.810	0.643	=	25.34	0.0440	0.0466	5.90
0.675	84.00	99.00	0.848	0.695	=	27.39	0.0372	0.0387	4.03
0.836	104.00	119.00	0.873	0.701	=	27.63	0.0308	0.0329	6.81
0.997	124.00	139.00	0.892	0.697	=	27.49	0.0259	0.0287	10.81
1.158	144.00	159.00	0.905	0.717	=	28.28	0.0232	0.0254	9.48
1.318	164.00	179.00	0.916	0.707	=	27.86	0.0202	0.0228	12.87
1.479	184.00	199.00	0.924	0.713	=	28.10	0.0182	0.0207	13.73
1.640	204.00	219.00	0.931	0.751	=	29.62	0.0174	0.0189	8.62
1.801	224.00	239.00	0.937	0.747	=	29.44	0.0158	0.0174	10.12
1.962	244.00	259.00	0.942	0.750	=	29.58	0.0146	0.0161	10.27
2.123	264.00	279.00	0.946	0.779	=	30.72	0.0140	0.0150	7.14

Table (6.12)- Test Series (D)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.353	0.536	0.684	27.61	1.037	0.924	12.22	0.980	1.573	1.608	2.22	1.572
0.514	0.871	0.855	1.87	0.931	0.981	5.37	0.956	1.802	1.836	1.88	1.819
0.675	1.101	1.311	19.07	1.013	1.009	0.39	1.011	2.114	2.320	9.74	2.217
0.836	1.242	1.539	23.91	1.013	1.066	5.23	1.039	2.255	2.605	15.52	2.430
0.979	1.436	1.596	11.14	1.025	1.066	4.00	1.045	2.461	2.662	8.16	2.561
1.158	1.525	1.881	23.34	0.966	1.038	7.45	1.002	2.491	2.919	17.18	2.705
1.318	1.585	1.938	22.27	0.926	1.066	15.11	0.996	2.511	3.004	19.63	2.757
1.479	1.719	1.938	12.73	0.955	1.095	14.65	1.025	2.674	3.033	13.42	2.853
1.640	1.736	1.995	14.91	1.060	1.152	8.67	1.106	2.796	3.147	12.55	2.971
1.801	1.754	1.995	13.74	1.049	1.180	12.48	1.114	2.803	3.175	13.27	2.989
1.962	1.772	1.995	12.58	1.154	1.180	2.25	1.167	2.926	3.175	8.50	3.050
2.123	1.810	2.052	13.37	1.129	1.152	2.03	1.140	2.939	3.204	9.01	3.071

Table (6.12)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
0.353	44.00	62.00	0.709	0.980	3.806	25.76	0.0906	0.0795	13.96
0.514	64.00	82.00	0.780	0.956	=	25.11	0.0642	0.0653	1.71
0.675	84.00	102.00	0.823	1.011	=	26.56	0.0532	0.0549	3.19
0.836	104.00	122.00	0.852	1.039	=	27.31	0.0450	0.0473	5.11
0.997	124.00	142.00	0.873	1.045	=	27.46	0.0385	0.0415	7.79
1.158	144.00	162.00	0.888	1.002	=	26.32	0.0321	0.0368	3.42
1.318	164.00	182.00	0.901	0.996	=	26.16	0.0282	0.0332	17.73
1.479	184.00	202.00	0.910	1.025	=	26.93	0.0260	0.0302	16.15
1.640	204.00	222.00	0.918	1.106	=	29.05	0.0254	0.0276	8.66
1.801	224.00	242.00	0.925	1.114	=	29.28	0.0234	0.0255	8.97
1.962	244.00	262.00	0.931	1.167	=	30.66	0.0226	0.0236	4.42
2.123	264.00	282.00	0.936	1.140	=	29.96	0.0204	0.0221	8.33

Table (6.13)- Test Series (E)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.353	1.274	1.368	7.37	1.230	1.108	11.01	1.169	2.504	2.476	1.13	2.490
0.514	1.532	1.596	4.17	1.176	1.251	6.37	1.213	2.708	2.847	5.13	2.777
0.675	1.821	1.824	0.16	1.336	1.336	0.00	1.336	3.157	3.160	0.09	3.158
0.836	1.997	1.938	3.04	1.453	1.365	6.44	1.409	3.450	3.303	4.45	3.376
0.997	2.174	2.109	3.08	1.571	1.393	12.77	1.482	3.745	3.502	6.93	3.623
1.158	2.192	2.280	4.01	1.559	1.393	11.91	1.476	3.751	3.673	2.12	3.712
1.318	2.209	2.394	8.37	1.430	1.365	4.76	1.397	3.639	3.759	3.29	3.699
1.479	2.298	2.451	6.55	1.606	1.450	10.74	1.528	3.904	3.901	0.07	3.902
1.640	2.368	2.508	5.91	1.676	1.507	11.21	1.591	4.044	4.015	0.72	4.029
1.801	2.386	2.508	5.11	1.665	1.536	8.39	1.600	4.051	4.044	0.17	4.047
1.962	2.421	2.508	3.59	1.641	1.564	4.92	1.602	4.062	4.072	0.24	4.067
2.123	2.471	2.508	1.49	1.726	1.564	10.35	1.645	4.197	4.072	3.07	4.134

Table (6.13)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
0.353	44.00	65.00	0.677	1.169	5.075	23.03	0.1050	0.0972	8.02
0.514	64.00	85.00	0.753	1.213	=	23.91	0.0798	0.0815	2.13
0.675	84.00	105.00	0.800	1.336	=	26.32	0.0693	0.0695	0.28
0.836	104.00	125.00	0.832	1.409	=	27.76	0.0603	0.0604	0.16
0.997	124.00	145.00	0.855	1.482	=	29.20	0.0540	0.0532	1.50
1.158	144.00	165.00	0.873	1.476	=	29.08	0.0468	0.0476	1.70
1.318	164.00	185.00	0.886	1.397	=	27.53	0.0392	0.0430	9.69
1.479	184.00	205.00	0.897	1.528	=	30.10	0.0385	0.0392	1.81
1.640	204.00	225.00	0.906	1.591	=	31.35	0.0363	0.0360	0.83
1.801	224.00	245.00	0.914	1.600	=	31.53	0.0334	0.0333	0.30
1.962	244.00	265.00	0.920	1.602	=	31.57	0.0308	0.0309	0.32
2.123	264.00	285.00	0.926	1.645	=	32.41	0.0293	0.0289	1.38

Table (6.14)- Test Series (F)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.353	1.165	1.197	2.74	1.529	1.378	10.95	1.453	2.694	2.575	4.62	2.634
0.514	1.489	1.425	4.49	1.665	1.578	5.51	1.621	3.154	3.003	5.02	3.078
0.675	1.659	1.653	0.36	1.670	1.220	2.99	1.695	3.329	3.373	1.32	3.351
0.836	1.849	1.767	4.64	1.778	1.834	3.14	1.806	3.627	3.601	0.72	3.614
0.997	1.983	1.881	5.42	1.923	1.920	0.15	1.921	3.906	3.801	2.76	3.053
1.158	2.082	2.052	1.46	1.975	1.920	2.86	1.947	4.057	3.972	2.14	4.014
1.318	2.118	2.166	2.26	1.952	1.948	0.20	1.950	4.070	4.114	1.08	4.092
1.479	2.195	2.223	1.27	1.900	1.948	2.52	1.924	4.095	4.171	1.85	4.133
1.640	2.259	2.280	0.93	1.975	1.920	2.86	1.947	4.234	4.200	0.81	4.217
1.801	2.276	2.280	0.17	2.081	1.920	8.38	2.000	4.357	4.200	3.73	4.278
1.962	2.312	2.280	1.40	2.057	1.948	5.59	2.002	4.369	4.228	3.33	4.298
2.123	2.375	2.337	1.62	2.015	1.977	1.92	1.996	4.390	4.314	1.76	4.352

Table (6.14)- Contiued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
0.353	44.00	68.00	0.647	1.453	6.344	22.91	0.1271	0.1116	13.96
0.514	64.00	88.00	0.727	1.621	=	25.55	0.1045	0.0955	9.48
0.675	84.00	108.00	0.777	1.695	=	26.71	0.0865	0.0824	5.03
0.836	104.00	128.00	0.812	1.806	=	28.46	0.0763	0.0722	5.70
0.997	124.00	148.00	0.837	1.921	=	30.28	0.0692	0.0641	8.01
1.158	144.00	168.00	0.857	1.947	=	30.69	0.0611	0.0576	6.21
1.318	164.00	188.00	0.872	1.950	=	30.73	0.0543	0.0521	4.24
1.479	184.00	208.00	0.884	1.924	=	30.32	0.0481	0.0477	0.87
1.640	204.00	228.00	0.894	1.947	=	30.69	0.0441	0.0439	0.65
1.801	224.00	248.00	0.903	2.000	=	31.53	0.0415	0.0407	2.60
1.962	244.00	268.00	0.910	2.002	=	31.56	0.0383	0.0379	1.14
2.123	264.00	288.00	0.916	1.996	=	31.46	0.0354	0.0355	0.14

Table (6.15)- Test Series (G)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.193	----	----	----	0.223	0.213	4.69	0.218	0.223	0.213	4.69	0.218
0.353	0.645	0.741	14.88	0.312*	0.327*	5.48	0.318	0.955	1.068	11.83	1.011
0.514	1.408	1.368	2.92	0.334	0.355	6.28	0.344	1.742	1.723	1.10	1.732
0.675	1.630	1.596	2.13	0.357	0.412	15.40	0.384	1.987	2.008	1.05	1.997
0.836	1.730	1.710	1.11	0.381	0.441	15.74	0.411	2.110	2.151	1.94	2.130
0.997	1.881	1.824	3.12	0.428	0.441	3.03	0.434	2.309	2.265	1.94	2.287
1.158	1.987	1.938	2.52	0.428	0.469	9.57	0.448	2.415	2.407	0.33	2.411
1.318	2.047	2.052	0.24	0.475	0.498	4.84	0.486	2.522	2.550	1.11	2.536
1.479	2.100	2.109	0.42	0.522	0.469	11.30	0.495	2.622	2.578	1.70	2.600
1.640	2.146	2.109	1.75	0.522	0.469	11.30	0.495	2.668	2.578	3.49	2.623
1.801	2.206	2.166	1.84	0.522	0.469	11.30	0.495	2.728	2.635	3.52	2.681
1.962	2.223	2.166	2.63	0.522	0.469	11.30	0.495	2.745	2.635	4.17	2.690
2.123	2.248	2.223	1.12	0.475	0.469	1.27	0.472	2.723	2.692	1.15	2.707

* From Test Series (B).

Table (6.15)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (G)	Total Moments Test Series (B)	% Diff.
0.193	24.00	36.00	0.666	0.218	1.268	17.19	0.218	-----	-----
0.353	44.00	56.00	0.785	0.318	=	25.11	1.011	1.027	1.58
0.514	64.00	76.00	0.842	0.344	=	27.16	1.732	1.245	39.11
0.675	84.00	96.00	0.875	0.384	=	30.32	1.997	1.731	15.36
0.836	104.00	116.00	0.896	0.411	=	32.41	2.130	2.077	2.55
0.997	124.00	136.00	0.911	0.434	=	34.26	2.287	2.164	5.68
1.158	144.00	156.00	0.923	0.448	=	35.37	2.411	2.398	0.54
1.318	164.00	176.00	0.931	0.486	=	38.36	2.536	2.500	1.44
1.479	184.00	196.00	0.938	0.495	=	39.07	2.600	2.537	2.48
1.640	204.00	216.00	0.944	0.495	=	39.07	2.623	2.654	1.18
1.801	224.00	236.00	0.949	0.495	=	39.07	2.681	2.654	1.01
1.962	244.00	256.00	0.953	0.495	=	39.07	2.690	2.636	2.04
2.123	264.00	276.00	0.956	0.472	=	37.22	2.707	2.659	1.80

Table (6.16)- Test Series (H)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.193	----	----	----	0.585	0.426	37.32	0.505	0.585	0.426	37.32	0.505
0.353	0.582	0.684	17.52	0.621*	0.625*	0.64	0.623	1.203	1.309	8.81	1.256
0.514	1.412	1.425	0.92	0.604	0.682	12.91	0.693	2.016	2.107	4.51	2.061
0.675	1.659	1.539	7.79	0.679	0.711	4.71	0.695	2.338	2.250	3.91	2.294
0.836	1.923	1.824	5.42	0.691	0.711	2.89	0.701	2.614	2.535	3.11	2.574
0.997	2.047	1.938	5.62	0.656	0.739	12.65	0.697	2.703	2.677	0.97	2.690
1.158	2.142	2.052	4.38	0.696	0.739	6.17	0.717	2.838	2.791	1.68	2.814
1.318	2.234	2.166	3.13	0.675	0.739	9.48	0.707	2.909	2.905	0.13	2.907
1.479	2.305	2.280	1.09	0.715	0.711	0.56	0.713	3.020	2.991	0.96	3.005
1.640	2.358	2.280	3.42	0.764	0.739	3.38	0.751	3.122	3.019	3.41	3.070
1.801	2.382	2.337	1.92	0.755	0.739	2.16	0.747	3.137	3.076	1.98	3.106
1.962	2.453	2.337	4.96	0.762	0.739	3.11	0.750	3.215	3.076	4.51	3.145
2.123	2.488	2.451	1.50	0.820	0.739	10.96	0.779	3.308	3.190	3.69	3.249

* From Test Series (C).

Table (6.16)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (H)	Total Moments Test Series (C)	% Diff.
0.193	24.00	39.00	0.615	0.505	2.537	19.90	0.505	------	----
0.353	44.00	59.00	0.745	0.623	=	24.55	1.256	1.245	0.88
0.514	64.00	79.00	0.810	0.643	=	25.34	2.061	1.579	30.52
0.675	84.00	99.00	0.848	0.695	=	27.39	2.294	1.939	18.30
0.836	104.00	119.00	0.873	0.701	=	27.63	2.574	2.202	16.89
0.997	124.00	139.00	0.892	0.697	=	27.49	2.690	2.300	16.95
1.158	144.00	159.00	0.905	0.717	=	28.28	2.814	2.429	15.85
1.318	164.00	179.00	0.916	0.707	=	27.86	2.907	2.480	17.21
1.479	184.00	199.00	0.924	0.713	=	28.10	3.005	2.538	18.40
1.640	204.00	219.00	0.931	0.751	=	29.62	3.070	2.601	18.03
1.801	224.00	239.00	0.937	0.747	=	29.44	3.106	2.667	16.46
1.962	244.00	259.00	0.942	0.750	=	29.58	3.145	2.683	17.21
2.123	264.00	279.00	0.946	0.779	=	30.72	3.249	2.747	18.27

Table (6.17)- Test Series (J)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.193	----	----	----	0.790	0.753	4.91	0.771	0.790	0.753	4.91	0.771
0.353	1.129	1.254	11.07	1.037*	0.924*	12.22	0.980	2.166	2.178	0.55	2.172
0.514	1.581	1.596	0.94	0.931	0.981	5.37	0.956	2.512	2.557	1.79	2.534
0.675	1.906	1.824	4.49	1.013	1.009	0.39	1.011	2.919	2.833	3.03	2.876
0.836	2.276	2.223	2.38	1.013	1.066	5.23	1.039	3.289	3.289	0.00	3.261
0.997	2.393	2.337	2.39	1.025	1.066	4.00	1.045	3.418	3.403	0.44	3.410
1.158	2.569	2.451	4.81	0,966	1.038	7.45	1.002	3.535	3.489	1.31	3.512
1.318	2.629	2.508	4.82	0.926	1.066	15.11	0.996	3.555	3.574	0.53	3.564
1.479	2.647	2.565	3.19	0.955	1.095	14.65	1.025	3.602	3.660	1.61	3.631
1.640	2.753	2.736	0.62	1.060	1.152	0.67	1.106	3.813	3.888	1.96	3.850
1.801	2.831	2.793	1.36	1.049	1.180	12.48	1.114	3.880	3.973	2.39	3.926
1.962	2.859	2.793	2.36	1.154	1.180	2.25	1.167	4.013	3.973	1.00	3.993
2.123	2.894	2.793	3.61	1.129	1.152	2.03	1.140	4.023	3.945	1.97	3.984

* From Test Series (D).

Table (6.17)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (J)	Total Moments Test Series (D)	% Diff.
0.193	24.00	42.00	0.571	0.771	3.806	20.27	0.771	------	----
0.353	44.00	62.00	0.709	0.980	=	25.76	2.172	1.572	38.16
0.514	64.00	82.00	0.780	0.956	=	25.11	2.534	1.819	39.33
0.675	84.00	102.00	0.823	1.011	=	26.56	2.876	2.217	29.72
0.836	104.00	122.00	0.852	1.039	=	27.31	3.261	2.430	34.19
0.997	124.00	142.00	0.873	1.045	=	27.46	3.410	2.561	33.17
1.158	144.00	162.00	0.888	1.002	=	26,32	3.512	3.705	29.83
1.318	164.00	182.00	0.901	0.996	=	26.16	3.564	2.757	29.28
1.479	184.00	202.00	0.910	1.025	=	26.93	3.631	2.853	27.26
1.640	204.00	222.00	0.918	1.106	=	29.05	3.850	2.971	29.60
1.801	224.00	242.00	0.925	1.114	=	29.28	3.926	2.989	31.36
1.962	244.00	262.00	0.931	1.167	=	30.66	3.993	3.050	30.91
2.123	264.00	282.00	0.936	1.140	=	29.96	3.984	3.071	29.72

Table (6.18)- Test Series (K)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.193	----	-----	----	1.077	0.909	18.48	0.993	1.077	0.909	14.48	0.993
0.353	1.200	1.311	9.25	1.230*	1.108*	11.01	1.169	2.430	2.419	0.45	2.424
0.514	1.906	1.995	4.66	1.176	1.251	6.37	1.213	3.082	3.246	5.32	3.164
0.675	2.294	2.166	5.90	1.336	1.336	0.00	1.336	3.630	3.502	3.65	3.566
0.836	2.647	2.394	10.56	1.453	1.365	6.44	1.409	4.100	3.759	9.07	3.929
0.997	2.771	2.679	3.43	1.571	1.393	12.77	1.482	4.342	4.072	6.63	4.207
1.158	3.000	2.850	5.26	1.559	1.393	11.91	1.476	4.559	4.243	7.44	4.401
1.318	3.088	2.907	6.22	1.430	1.365	4.76	1.397	4.518	4.272	5.75	4.395
1.479	3.318	3.021	9.83	1.606	1.450	10.75	1.528	4.924	4.471	10.13	4.697
1.640	3.353	3.135	6.95	1.676	1.507	11.21	1.591	5.029	4.642	8.33	4.835
1.801	3.406	3.192	6.70	1.665	1.536	8.39	1.600	5.071	4.728	7.25	4.899
1.962	3.441	3.249	5.90	1.641	1.564	4.92	1.602	5.082	4.813	5.58	4.947
2.123	3.459	3.249	6.46	1.726	1.564	10.35	1.645	5.185	4.813	7.73	4.999

* From Test Series (E).

Table (6.18)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (K)	Total Moments Test Series (E)	% Diff.
0.193	24.00	35.00	0.685	0.993	5.075	19.56	0.993	------	------
0.353	44.00	65.00	0.677	1.169	=	23.03	2.424	2.490	2.72
0.514	64.00	85.00	0.753	1.213	=	23.91	3.164	2.777	13.93
0.675	84.00	105.00	0.800	1.336	=	26.32	3.566	3.158	12.91
0.836	104.00	125.00	0.832	1.409	=	27.76	3.929	3.376	16.38
0.997	124.00	145.00	0.855	1.482	=	29.20	4.207	3.623	16.12
1.158	144.00	165.00	0.872	1.476	=	29.08	4.401	3.712	18.56
1.318	164.00	185.00	0.886	1.397	=	27.23	4.395	3.699	18.81
1.479	184.00	205.00	0.897	1.528	=	30.10	4.697	3.902	20.37
1.640	204.00	225.00	0.906	1.591	=	31.35	4.835	4.029	20.00
1.801	224.00	245.00	0.914	1.600	=	31.53	4.899	4.047	21.05
1.962	244.00	265.00	0.920	1.602	=	31.57	4.947	4.067	21.63
2.123	264.00	285.00	0.926	1.645	=	32.41	4.999	4.134	20.92

Table (6.19)- Test Series (L)- Full Size Structure.

Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.193	----	----	----	1.059	0.920	14.80	0.990	1.059	0.922	14.80	0.990
0.353	1.623	1.653	1.84	1.529*	1.378*	10.95	1.453	3.152	3.031	3.99	3.091
0.514	2.065	2.109	2.13	1.665	1.578	5.51	1.621	3.730	3.687	1.16	3.708
0.675	2.612	2.622	0.38	1.670	1.720	2.99	1.695	4.282	4.342	1.40	4.312
0.836	3.053	3.021	1.06	1.778	1.834	3.14	1.806	4.831	4.855	0.50	4.843
0.997	3.141	3.078	2.04	1.923	1.920	0.15	1.921	5.064	4.998	1.32	5.031
1.158	3.353	3.249	3.20	1.975	1.920	2.86	1.947	5.328	5.169	3.07	5.248
1.318	3.565	3.420	4.24	1.952	1.948	0.20	1.950	5.517	5.368	2.77	5.442
1.479	3.646	3.977	4.86	1.900	1.948	2.52	1.924	5.546	5.425	2.23	5.485
1.640	3.724	3.534	5.37	1.975	1.920	2.86	1.947	5.699	5.454	4.49	5.576
1.801	3.830	3.534	8.37	2.081	1.920	8.38	2.000	5.911	5.454	8.37	5.682
1.962	3.918	3.591	9.10	2.057	1.948	5.59	2.002	5.975	5.539	7.87	5.757
2.123	4.041	3.705	9.06	2.015	1.977	1.92	1.966	6.056	5.682	6.58	5.869

* From Test Series (F).

Table (6.19)- Continued.

Total Wall Precom. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F.Loads (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Total Moments Test Series (L)	Total Moments Test Series (F)	% Diff.
0.193	24.00	48.00	0.500	0.990	6.344	15.61	0.990	--------	-----
0.353	44.00	68.00	0.647	1.453	=	22.91	3.091	2.634	17.35
0.514	64.00	88.00	0.727	1.621	=	25.55	3.708	3.078	20.46
0.675	84.00	108.00	0.777	1.695	=	26.71	4.312	3.351	28.67
0.836	104.00	128.00	0.812	1.806	=	28.46	4.843	3.614	34.00
0.997	124.00	148.00	0.837	1.921	=	30.28	5.031	3.853	30.57
1.158	144.00	168.00	0.857	1.947	=	30.69	5.248	4.014	30.72
1.318	164.00	188.00	0.872	1.950	=	30.73	5.442	4.092	32.99
1.479	184.00	208.00	0.884	1.924	=	30.32	5.485	4.133	32.71
1.640	204.00	228.00	0.894	1.947	=	30.69	5.576	4.217	32.22
1.801	224.00	248.00	0.903	2.000	=	31.53	5.682	4.278	32.81
1.962	244.00	268.00	0.910	2.002	=	31.56	5.757	4.298	33.94
2.123	264.00	288.00	0.916	1.996	=	31.46	5.869	4.352	34.85

Table (6.20)- Test Series (M)- Full Size Structure.

Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
0.997	1.500	1.311	14.41	-----	-----	-----	-----	1.500	1.311	14.41	1.405
=	1.863	1.596	16.72	0.428	0.469	9.58	0.448	2.291	2.065	10.94	2.178
=	1.553	1.653	6.44	0.726	0.682	6.45	0.704	2.279	2.335	2.45	2.307
=	1.436	1.596	11.14	1.084	1.095	1.01	1.089	2.520	2.691	6.78	2.605
=	2.174	2.109	3.08	1.559	1.450	7.51	1.504	3.733	3.559	4.88	3.646
=	1.983	1.881	5.42	1.681	2.091	24.39	1.886	3.664	3.972	8.40	3.818

Table (6.20)- Continued.

Total Wall Prec. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
0.997	124.00	136.00	0.911	0.448	1.268	35.37	0.0169	0.0149	13.42
=	=	139.00	0.892	0.704	2.537	27.74	0.0262	0.0287	9.54
=	=	142.00	0.873	1.089	3.806	28.62	0.0401	0.0415	3.49
=	=	145.00	0.855	1.504	5.075	29.64	0.0548	0.0540	1.48
=	=	148.00	0.837	1.886	6.344	29.72	0.0679	0.0641	5.92

Table (6.21)- Test Series (N)- Full Size Structure.

Total Wall Prec. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F. Load (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
1.801	1.683	1.767	4.99	-----	-----	-----	-----	1.683	1.767	4.99	1.725
=	2.322	1.995	16.39	0.482	0.469	2.77	0.475	2.804	2.464	13.79	2.634
=	1.846	1.995	8.07	0.945	0.711	32.91	0.828	2.791	2.706	3.14	2.748
=	1.754	1.995	13.74	1.067	1.095	2.62	1.081	2.821	3.090	9.53	2.955
=	2.386	2.508	5.11	1.660	1.507	10.15	1.583	4.046	4.015	0.77	4.030
=	2.276	2.280	0.17	2.135	2.176	1.92	2.155	4.411	4.456	1.02	4.433

Table (6.21)- Continued.

Total Wall Prec. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
1.801	224.00	236.00	0.949	0.475	1.268	37.50	0.0101	0.0088	14.77
=	=	239.00	0.937	0.828	2.537	32.63	0.0175	0.0174	0.57
=	=	242.00	0.925	1.081	3.806	28.40	0.0227	0.0255	12.33
=	=	245.00	0.914	1.583	5.075	31.20	0.0330	0.0333	0.91
=	=	248.00	0.903	2.155	6.344	33.97	0.0447	0.0407	9.82

Table (6.22)- Test Series (P)- Full Size Structure.

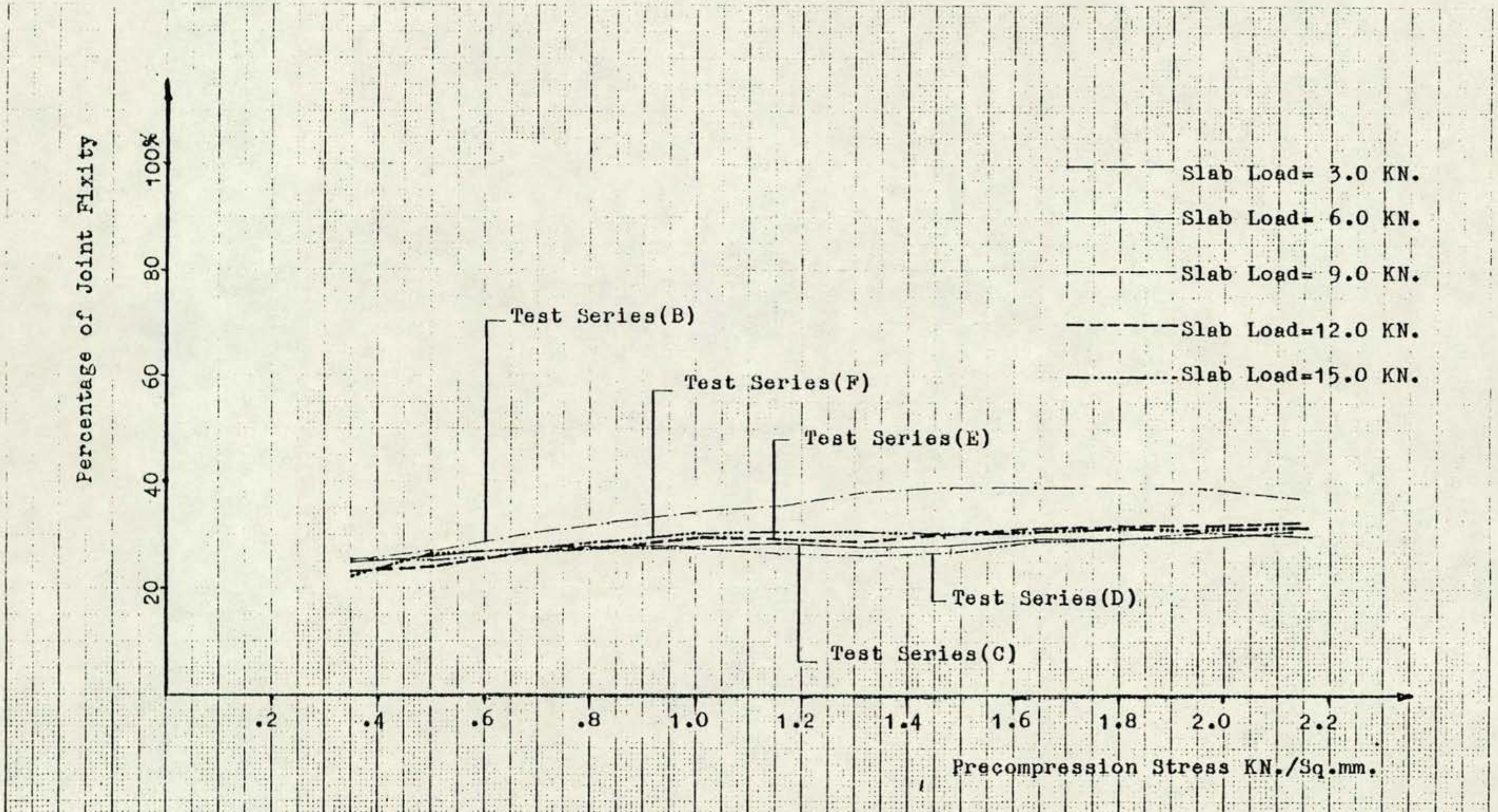
Total Wall Precom. N/mm^2	Moments Due to Wall Precomp. (KN.M)		% Diff.	Moments Due to Floor Loads (KN.M)		% Diff.	Average Moments Due to F.Loads (M_o)	Total Moments (KN.M)		% Diff.	Average Total Moments (M_T)
	M_θ	M_Δ		M_θ	M_Δ			M_θ	M_Δ		
2.600	1.923	1.938	0.78	----	----	----	------	1.923	1.938	0.78	1.930
=	1.969	2.052	4.21	0.580	0.555	4.50	0.567	2.549	2.607	2.27	2.578
=	2.065	2.052	0.63	1.149	0.825	39.27	0.987	3.214	2.877	11.71	3.045
=	2.206	2.109	4.60	1.601	1.180	35.37	1.390	3.807	3.289	15.75	3.548
=	2.464	2.622	6.41	1.958	2.134	8.98	2.046	4.422	4.756	7.55	4.589
=	2.594	2.508	8.53	2.433	2.290	6.24	2.361	5.027	4.798	4.77	4.912

Table (6.22)- Continued.

Total Wall Prec. N/mm^2	P_u (KN)	P_1 (KN)	P_u/P_1 (ψ)	Average Moment Due to F. Load (Tests) (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity	Joint Eccentricity (Tests)	Joint Eccentricity (Theory)	% Diff.
2.600	324.00	336.00	0.964	0.567	1.268	44.75	0.0084	0.0063	33.33
=	=	339.00	0.955	0.987	2.537	38.90	0.0145	0.0125	16.00
=	=	342.00	0.947	1.390	3.806	36.53	0.0204	0.0184	10.86
=	=	345.00	0.939	2.046	5.075	40.31	0.0299	0.0242	23.55
=	=	348.00	0.931	2.361	6.344	37.22	0.0344	0.0298	15.43

Table (6.23)- Test Series (R)- Full Size Structure.

Total Wall Precom. N/mm^2	Slab Load (KN)	Moment Due to Floor Loads		% Diff.	Average Moment Due to F.Load (M_o)	Rigid Frame Moment ($\bar{M}_R$)	% of Joint Fixity
		M_θ	M_Δ				
5.018	3.00	0.801	0.783	2.30	0.792	1.268	62.46
=	6.00	1.443	1.366	5.63	1.404	2.537	55.36
=	9.00	1.942	1.893	2.58	1.917	3.806	50.38
=	12.00	2.064	2.533	22.72	2.298	5.075	45.29
=	15.00	2.892	3.031	4.80	2.961	6.344	46.68
=	18.00	3.367	3.615	7.36	3.491	7.612	45.86
=	21.00	4.077	4.455	9.27	4.266	8.880	48.04
=	24.00	4.669	5.095	9.12	4.882	10.150	48.09
=	25.00	4.984	5.499	10.33	5.241	10.573	49.57
=	27.50	5.537	6.151	11.08	5.844	11.630	50.25
=	30.00	6.207	6.462	4.10	6.334	12.687	49.92
=	32.50	6.172	6.915	12.03	6.543	13.745	47.60
=	35.00	1.907	1.953	2.41	1.930	14.00	13.04
=	37.50	-----	-----	----	----	-----	-----



Percentage of Joint Fixity
100%
80
60
40
20
Test Series(B)
Test Series(F)
Test Series(E)
Test Series(D)
Test Series(C)
Slab Load= 3.0 KN.
Slab Load= 6.0 KN.
Slab Load= 9.0 KN.
Slab Load=12.0 KN.
Slab Load=15.0 KN.
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
Precompression Stress KN./Sq.mm.

Fig.(6.13)

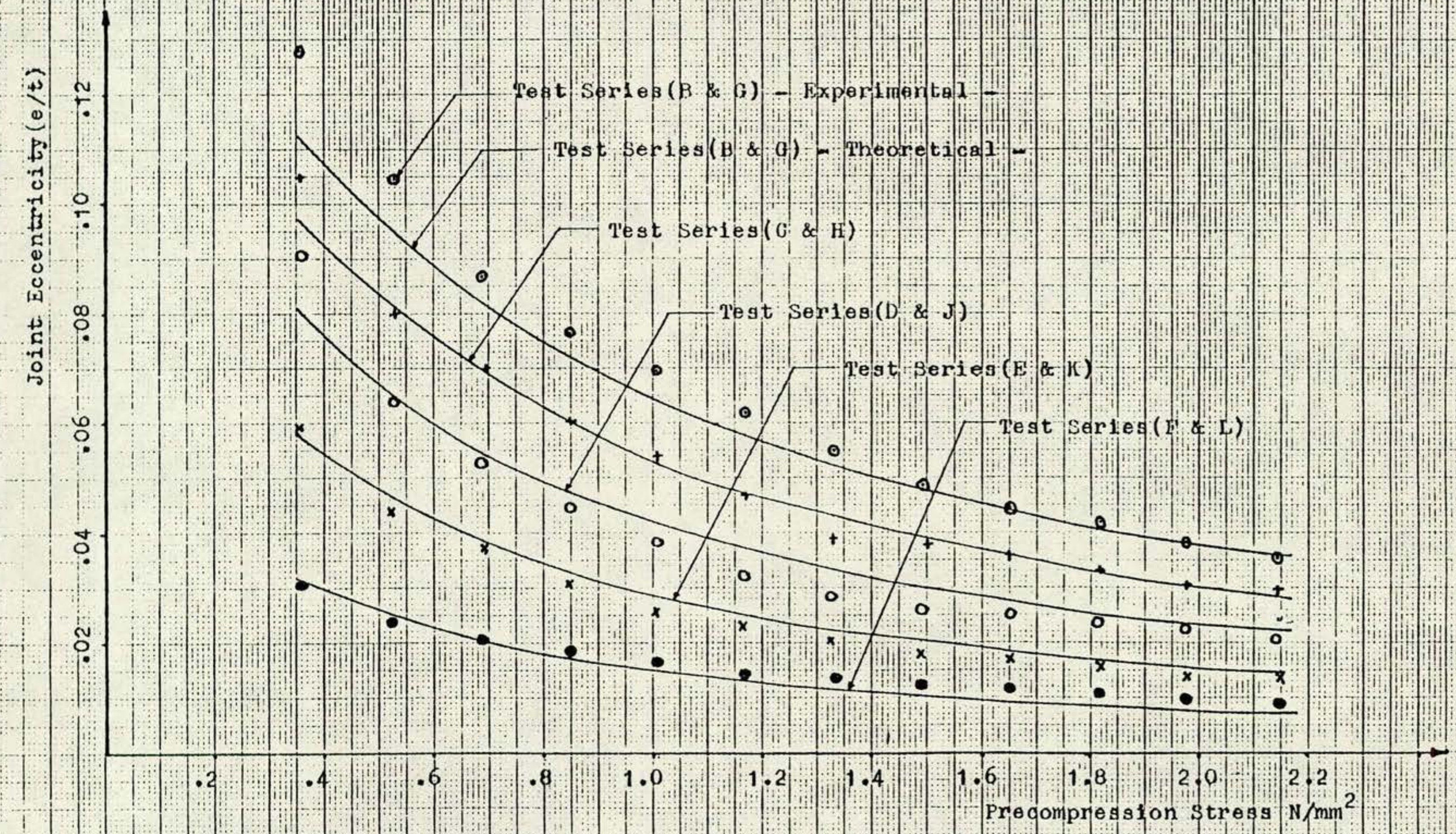


Fig.(6.14)– Joint Eccentricities Vs. Wall Precompression – Full Scale Test Structure –

Computation of joint eccentricities from test data are obtained by dividing the test slab restraining moments due to floor loading by the sum of P_u and P_L. It should be pointed out that, in computing the joint eccentricities, ε, from equation (4.26), a value of $\alpha = 3.37$ was used, which is based on the rigidity of the uncracked slab to the uncracked wall as defined by that equation.

6.3.7 <u>Wall deflections and rotations:</u>

Deflection of the exterior brick wall under various loading conditions up to the second floor level is given in Tables (B34) through (B45) and Figures (6.15) through (6.28) for test series (A) through (P) respectively. As may be seen from these figures, wall deflections are relatively small, and the floor slab sidesway increases with the increase of the floor slab loads.

Rotation data of the upper and lower brick walls are presented in Tables (B46) through (B57) for test series (A) through (P) respectively.

Based on the data presented in the tables of the slab restraining moments, joint eccentricities and percentage of joint fixity, several observations may be made on the results of both the half scale and full size test structures.

1) The magnitude of the floor live load does not have a significant effect on the degree of fixity obtained at the wall/floor joint.

2) Increasing the joint precompression rapidly increases the joint rigidity up to a maximum percentage of fixity, after which,

even with increased precompression stresses (see Figures (6.1) and (6.13)), the degree of fixity remains almost constant (may decrease slightly due to downward slab deflection, depending on the magnitude of the structure sidesway). Thus, the maximum percentage of the joint fixity to be achieved at any joint does not depend only on the magnitude of the pre-compression stress alone, but, on the type of joint stiffness ratio between the floor slab and the supporting wall, this is particularly true where the ratio of the floor/wall stiffness is small, when up to 94% fixity was obtained for the half scale structure. For the full size structure, where this ratio equals 3.37, the maximum joint fixity achieved was about 31% irrespective of the wall precompression, this indicates that the joint is nearer to the "hinged" support type due to the combination of a relatively stiff floor slab and a slender wall joint.

3) A significant restraining moment can be developed at the slab ends provided the ratio of floor/wall flexural rigidity is less than a unity. Thus, the floor slab design should account for such negative moment. On the other hand, where this ratio is large, the slab restraining moment which could develop at the slab ends is generally low, even at very high precompression, and in such cases, the floor slabs should be designed for a simple support positive moments and allow for normal negative moment reinforcement.

4) The development of joint rigidity is not linearly related to increase in wall precompression. A significant increase may occur due to initial application of the precompression and this increase becomes smaller as the wall precompression is increased.

5) For the half scale structure, the maximum difference in the average total moments between test series (D) where the floor load is 1.75 KN/m is around 13% less than those obtained in test series (F), this difference is about 15% for test series (E) and (G) where the floor live load is 3.5 KN/m.

6) For the full size structure, the maximum difference in the average total moments between test series (C) where the floor live load is 4.84 KN/m is around 30% smaller than those of test series (H) for a precompression value of 0.514 N/mm^2, this difference is less than 18% for all other precompression stresses and this particular high difference is not representative of the true average difference and can be neglected.

For a live load of 12.11 KN/m, the maximum difference in the average total moments between test series (F) and (L) is around 30% indicating that a significant difference in the restraining slab moments due to wall precompression (slab restraining moments due to floor loading are the same for both tests) may arise when the floor slab live loads are comparatively larger than the floor design loads. Since the floor slab in this particular structure has been loaded and unloaded repeatedly with relatively high loads, a permanent downward deflection and end rotation of the floor slab developed, thus, when applying the wall precompression rapidly, the magnitude of the floor upward midspan deflection and end rotation are greater than those of an undeflected one and hence, the restraining moments developed due to the wall precompression is larger than would normally be. This condition is not critical in actual structures since in practice, when a precast floor slab is placed in position, firstly, the design loads are normally not exceeded, and secondly, as the additional upper walls are constructed gradually, the tendency of a permanent large slab deflection will not occur under normal working loads.

As a result of these observations, it may be stated that the order of application of the floor loads does not have a significant effect on the restraining moments developed at the wall/floor joint as to be catered for. Thus, for practical design of masonry brick walls, the history of loading is not significant. This means that the method of construction, either cast-in-situ, or precast floor construction, will not significantly affect the moments that will eventually be developed in the supporting walls. The method of floor construction must be considered in the design of the floor slab, since the midspan deflection and end rotation (and hence the positive bending moment) will be greater initially for the precast floor systems.

6.4 CONCLUSIONS

From the previous remarks, the following is concluded:

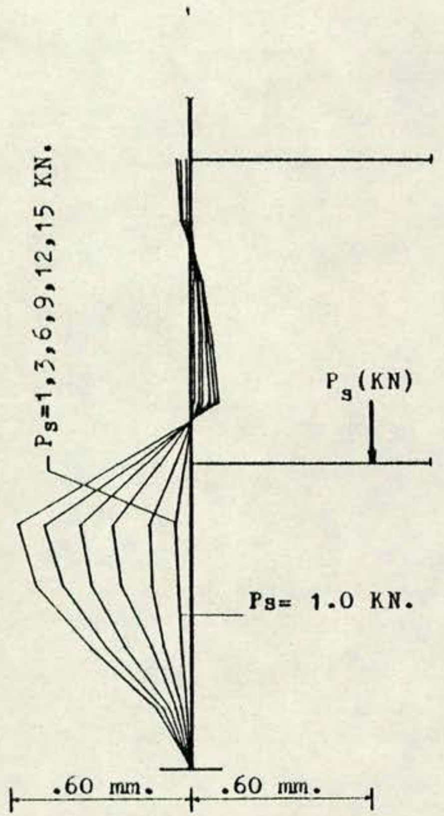
A) A significant restraining moment can be developed at a floor/wall joint if the ratio of the floor/wall stiffness is small (i.e. rigid wall, flexible slab joint) even at relatively low precompression stresses.

B) Increasing the precompression stress increases the joint rigidity (although this relationship is not linear) up to a maximum value depending on the ratio of floor/wall stiffness, but where this ratio is large (stiff slab / flexible wall joint), increasing the pre-compression stress increases the joint fixity by a marginal value, this means that in such cases, the joint is more likely to be a "hinged" joint rather than a 'fixed' one, and hence, the wall moments (and slab restraining moments) are generally smaller and will result in a small joint eccentricity.

C) The magnitude of the floor live load (provided it will not exceed the design loads limit) does not have a significant effect on the degree of fixity obtained at a floor/wall joint, particularly at higher precompression stresses.

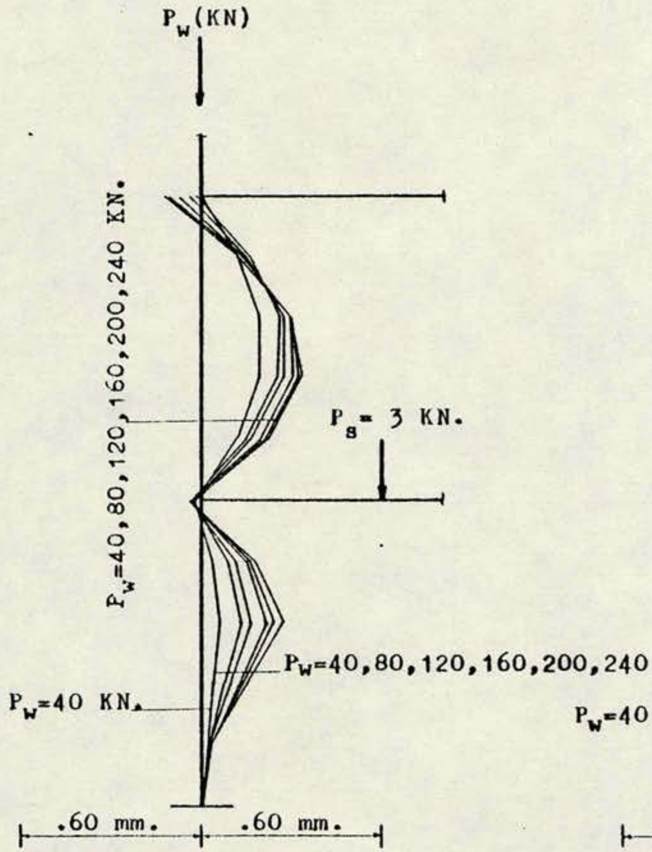
D) Complete joint fixity was not obtained in tests (even at such high precompression stresses) for the half scale model although the ratio of the floor/wall stiffness was quite small and may require a smaller ratio of stiffnesses (stiffer wall, flexible slab) which would be uneconomical for practical cases.

E) The sequence of loading does not significantly influence the slab restraining moments that can be developed (depending on the ratio of floor/wall stiffness) at the joints of a load bearing masonry structure. Thus, in design, it is not necessary to distinguish between different methods of floor construction, but, special consideration should be made in the design of the slabs for positive and/or negative bending moments that will develop depending on the ratio of the floor/wall stiffness.



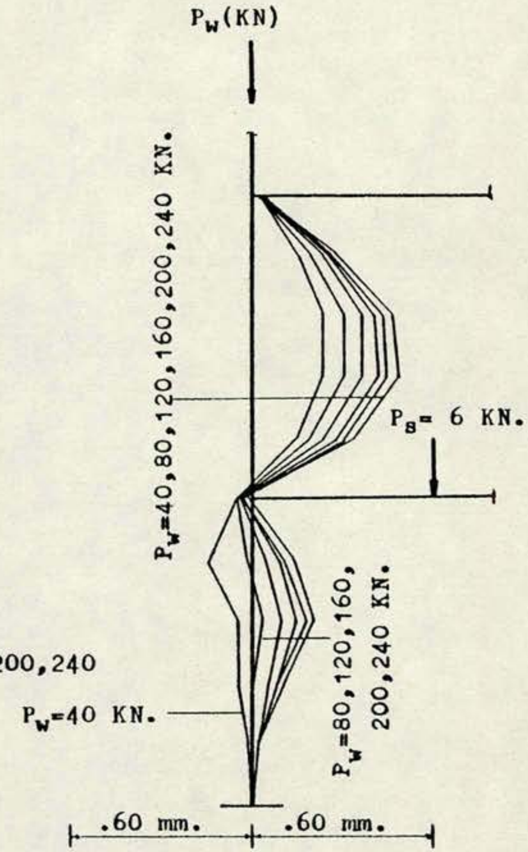
Test Series (A)

Fig.(6.15)



Test Series (B)

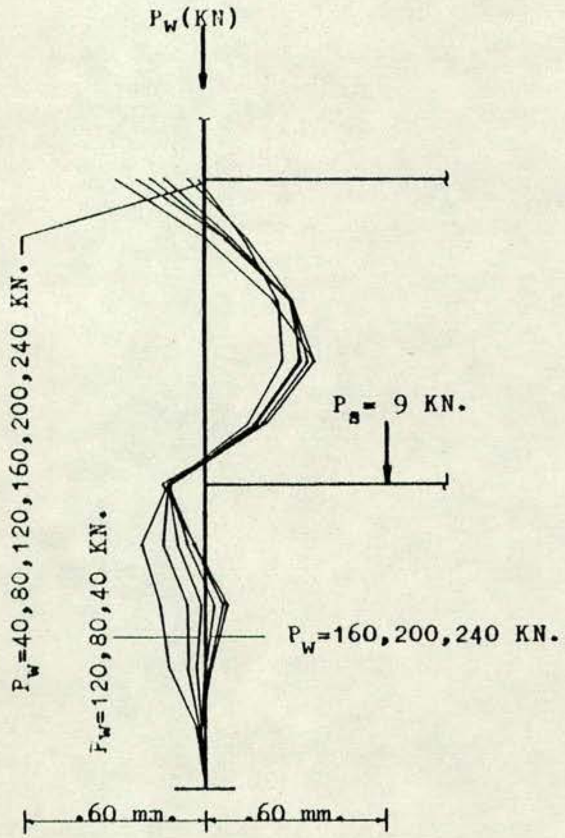
Fig.(6.16)



Test Series (C)

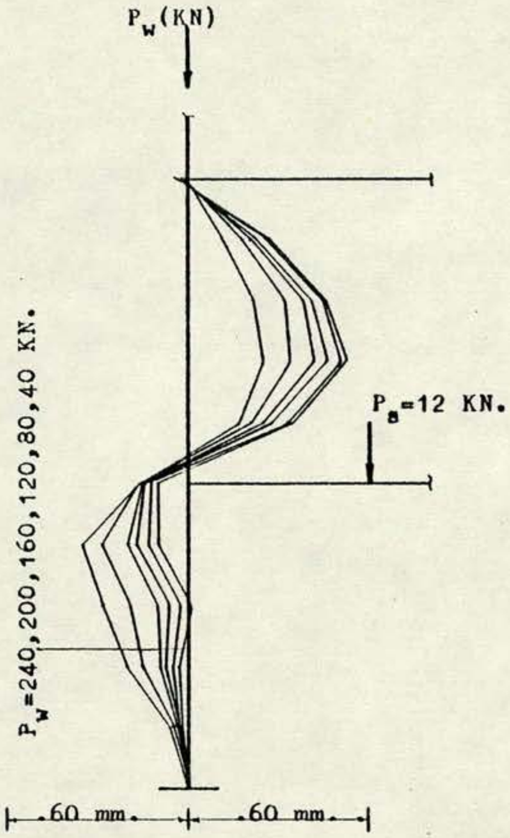
Fig.(6.17)

N.B. For Test Series(B)&(C),PRECOMPRESSION Applied Prior to Loading Lower Slab.



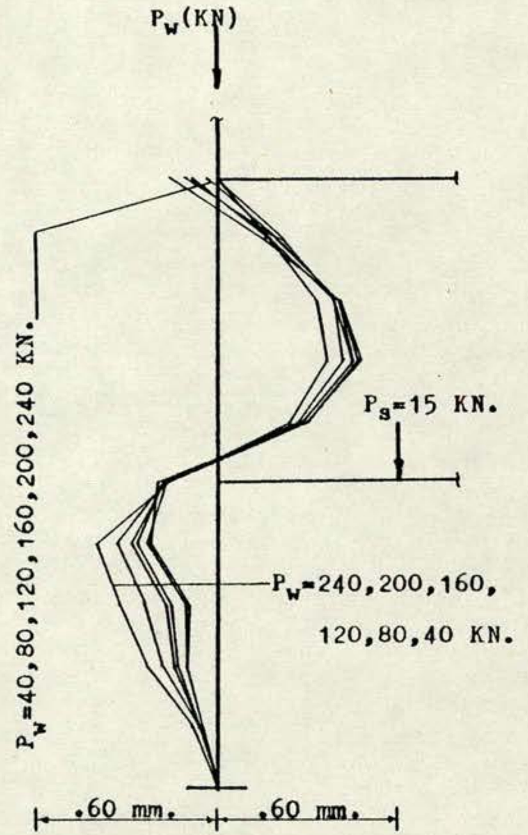
Test Series (D)

Fig.(6.18)



Test Series (E)

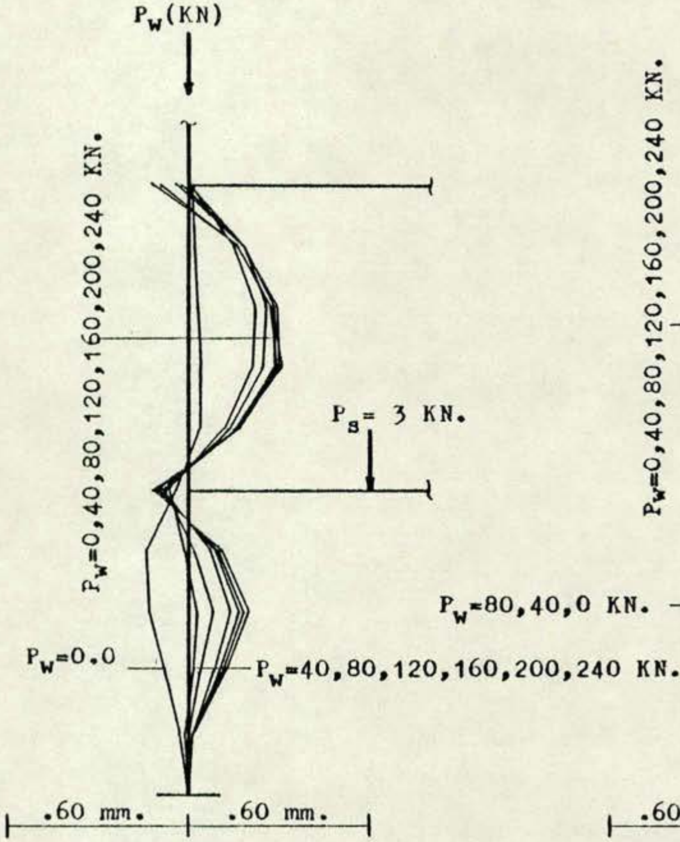
Fig.(6.19)



Test Series (F)

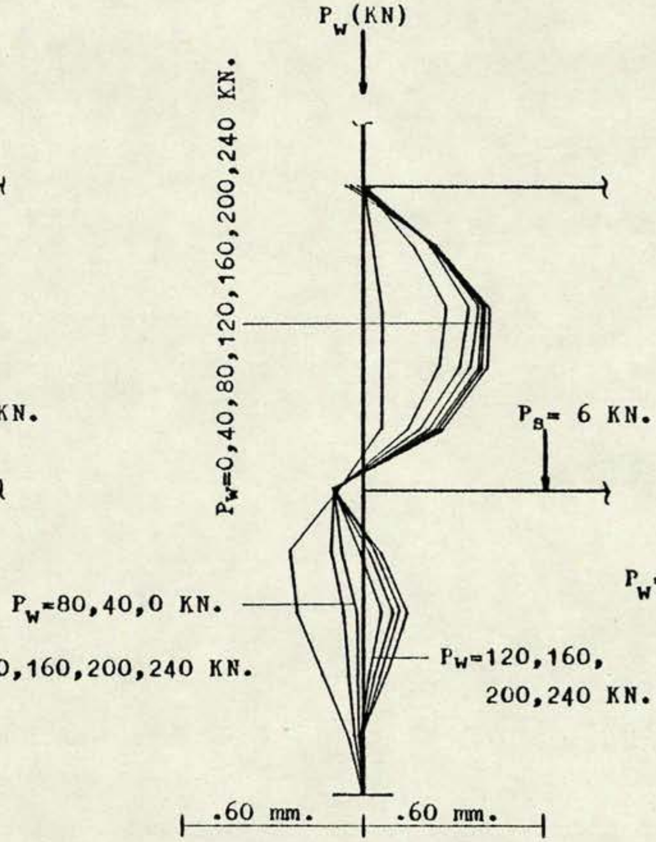
Fig.(6.20)

N.B. PRECOMPRESSION Applied Prior to Loading Lower Slab.



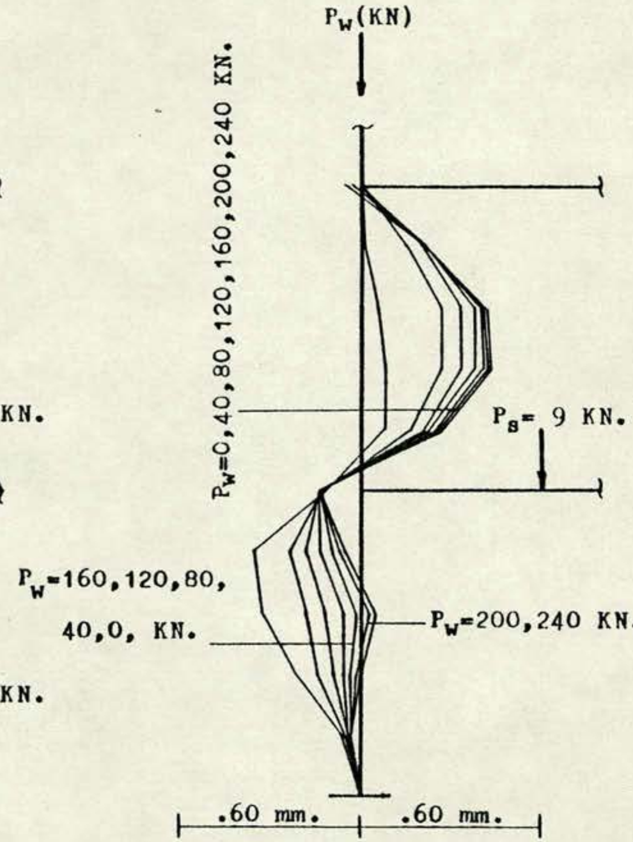
Test Series (G)

Fig.(6.21)



Test Series (H)

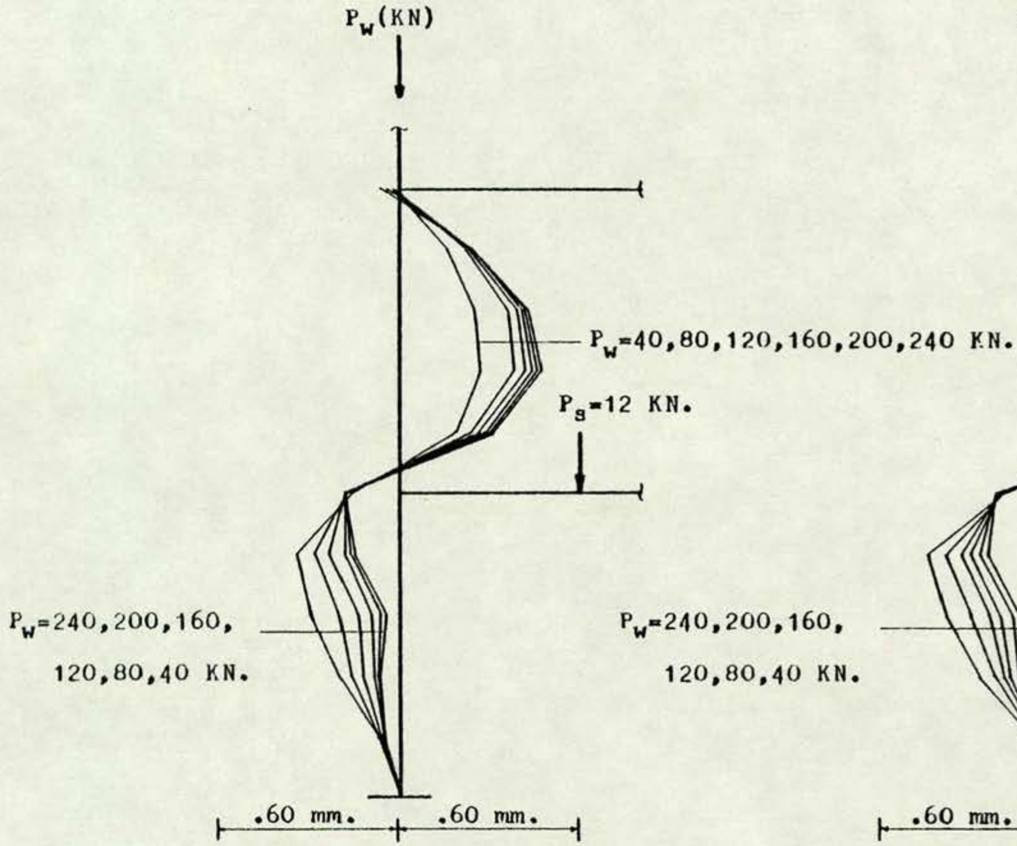
Fig.(6.22)



Test Series (J)

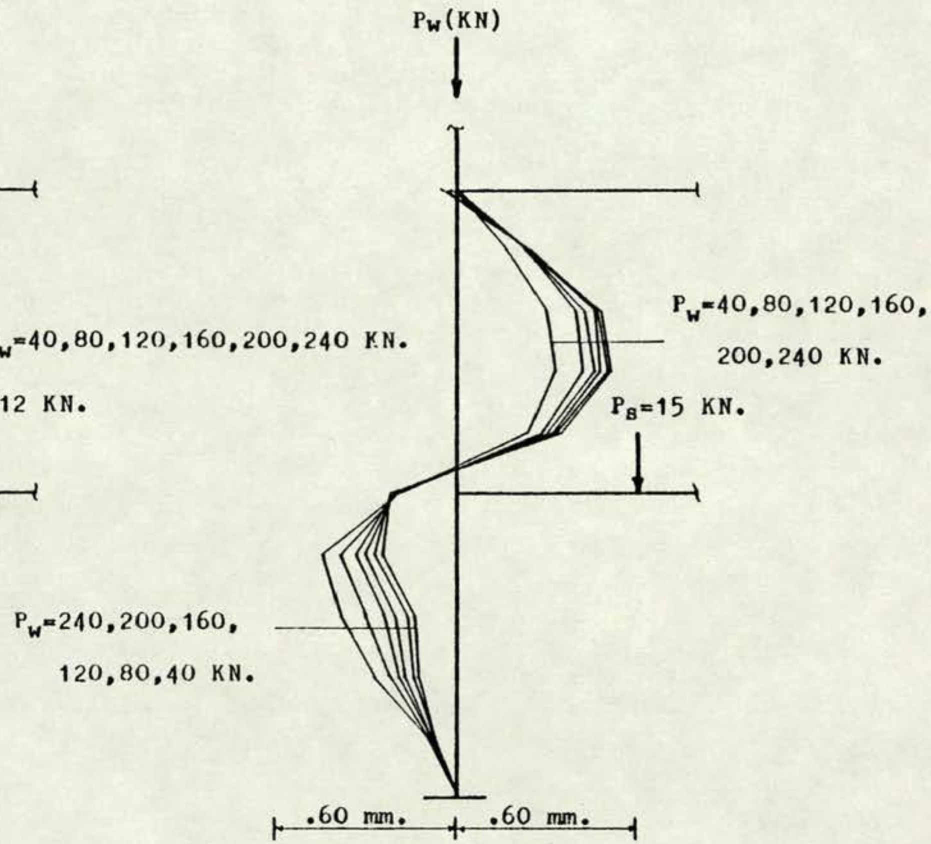
Fig.(6.23)

N.B. Loading Lower Slab Prior to Application of PRECOMPRESSION.



Test Series (K)

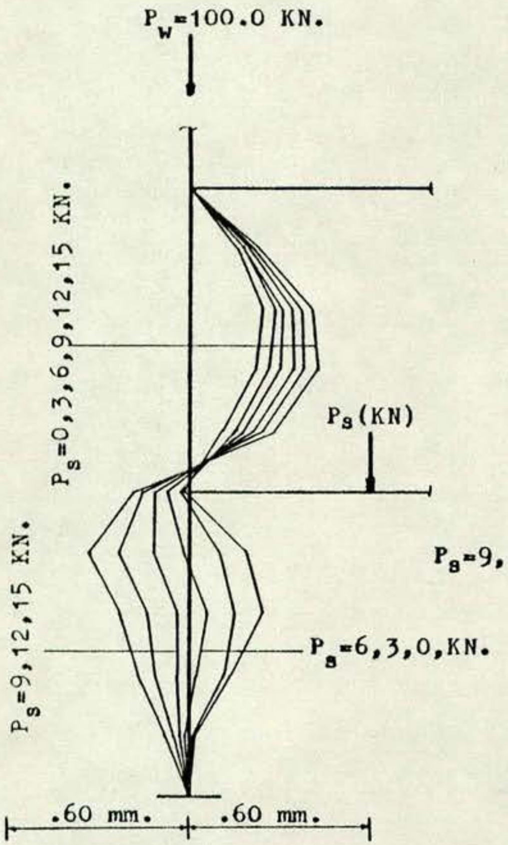
Fig.(6.24)



Test Series (L)

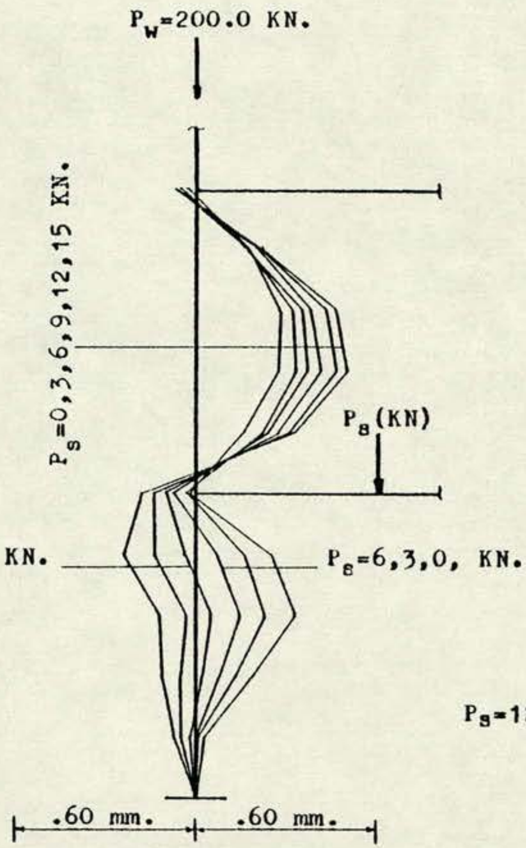
Fig.(6.25)

N.B. Loading Lower Slab Prior to Application of PRECOMPRESSION.



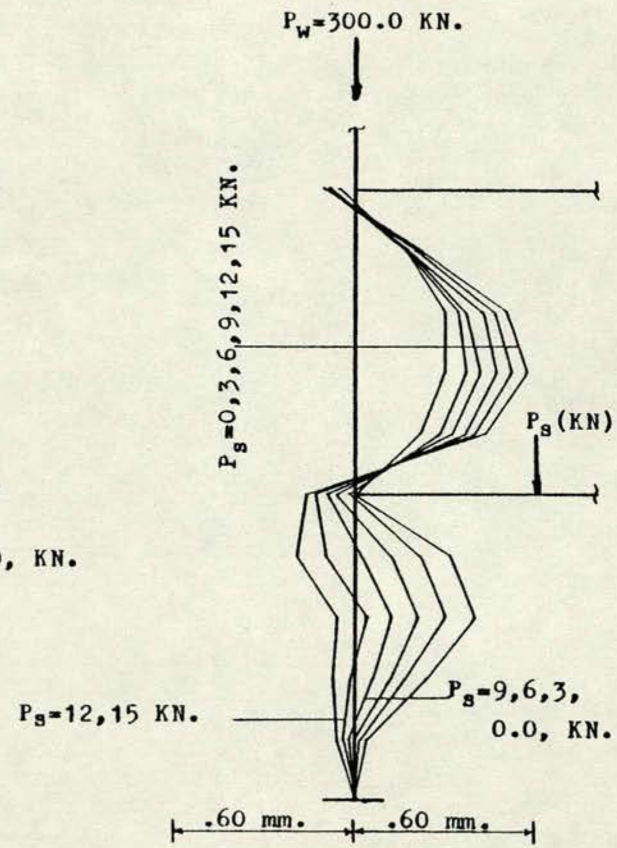
Test Series (M)

Fig.(6.26)



Test Series (N)

Fig.(6.27)



Test Series (P)

Fig.(6.28)

N.B. The Constant PRECOMPRESSION is Applied Prior to Loading Lower Slab.

CHAPTER 7 - APPLICATION OF THEORY:-

7.1 INTRODUCTION:

Conventional methods for the estimation of the bearing capacity of masonry walls subjected to eccentric loading are based on a theoretical model of a hinged ended, brittle column. Real walls in multi-storey buildings are, however, compressed between reinforced concrete slabs through joints which are capable of transmitting bending moments. The transmitted moments on the one hand influence the deflected form of the wall and, on the other, control the end rotation of the wall. It follows, therefore, that a satisfactory method for the design of masonry walls in compression must allow for interactive effects between walls and floor slabs.

The major problems that arise in the design of masonry walls to resist compressive loading are, firstly, the determination of the structural eccentricity at wall/floor slab joints, and secondly, the reduction in the load bearing capacity resulting from structural eccentricity and wall slenderness. Existing design codes treat these problems independently but in an actual structure, they are not independent because of the interaction between walls and floor slabs. Thus, eccentricity depends on the ratio of stiffnesses of walls and floors and on the characteristics of the joint between them. The load bearing capacity of a wall panel is influenced by the same factors so that both problems have to be dealt with on the basis of assumption and analytical methods which are mutually consistent. The present chapter outlines the theory developed earlier and its applications in practical structures. Comparisons are made between the results of calculations based on these theories and on the existing Code of Practice BS 5628 to examine the design procedures and its applicability in practical structures.

7.2 COMPARISON OF THEORY WITH BS 5628:1978

There is little point in comparing the capacity reduction factors given in BS 5628(25) with those presented in Chapters (2) and (3) since the former must be related to the effective heights and eccentricities prescribed in the Code. In general, it will be found that the eccentricities calculated by the method given in Chapter (4) which, as demonstrated in Figures (7.1) and (7.2), agree well with measured values. The discrepancy between the experimental results and the Code eccentricities for different levels of precompression stress on a particular wall/floor slab joint is also indicated in these figures.

In order to explore the practical implications of this situation, calculations have been carried out relating to an outer and inner wall of a hypothetical multi-storey building as shown in Figure (7.3). Eccentricities, capacity reduction factors and load bearing capacities for a practical masonry strength are shown in Tables (7.1 - 7.6) for two masonry thicknesses.

Considering first the outer wall case, Tables (7.1) and (7.2) it will be seen that for the 215 mm thick wall the reduction factors, and thus the bearing capacities, according to the Code and the theory are approximately the same at and below the seventh floor from the top of the building. At the first six levels, the Code bearing capacities are considerably higher than the theoretical values. With clay brickwork of the strength selected for purposes of illustration, this is not of practical importance since the bearing capacity is everywhere far in excess of the design load. However, in the case of the cavity wall (Table 7.2) the theoretical eccentricity at the third level exceeds the failure eccentricity $\varepsilon_f = 0.45$ as defined by equation (4.63) and therefore the wall section is unsatisfactory for the span and

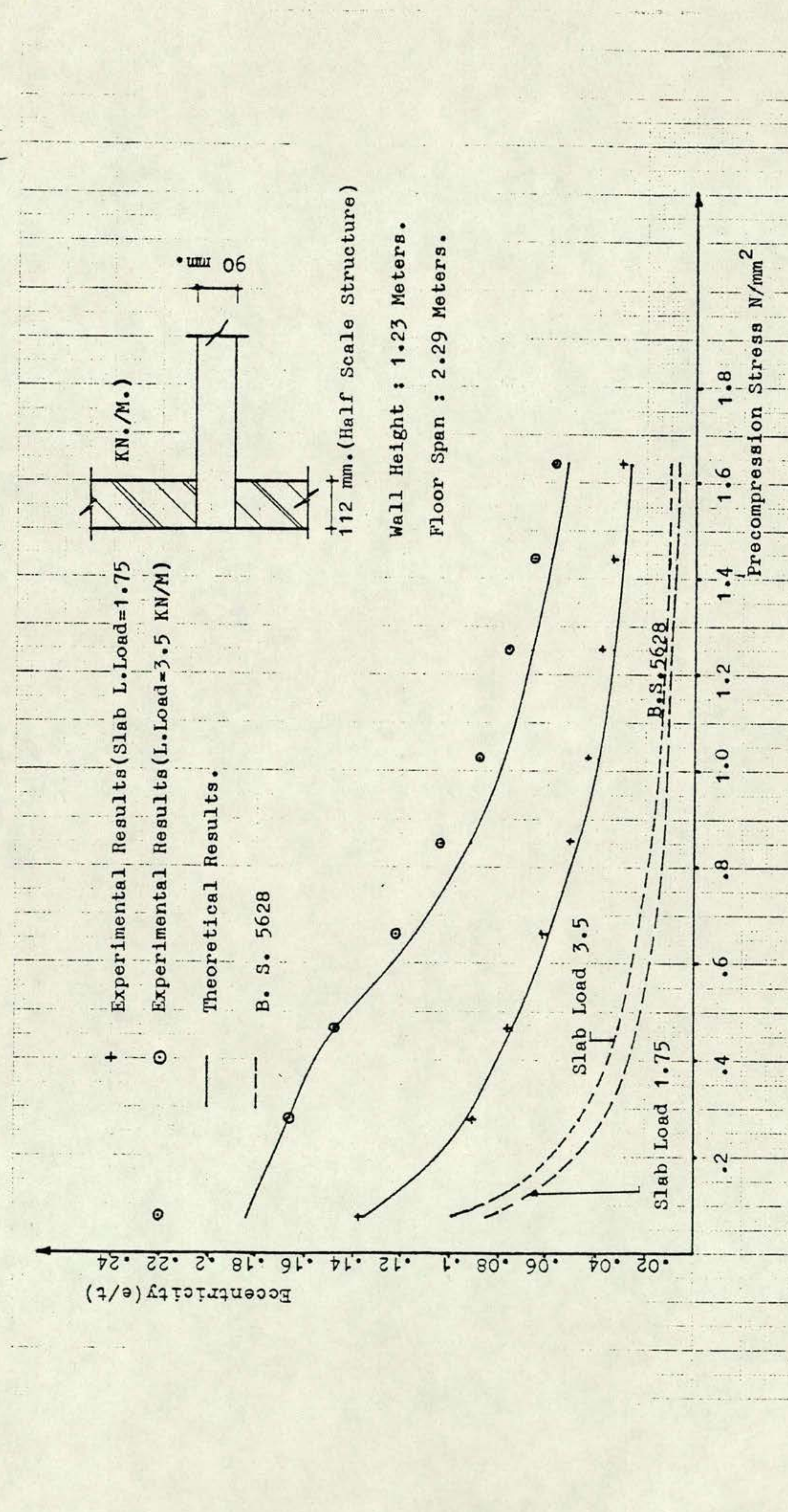

+ Experimental Results(Slab L.Load=1.75
⊙ Experimental Results(L.Load=3.5 KN/M)
Theoretical Results.
B. S. 5628
KN./M.)
90 mm.
112 mm.(Half Scale Structure)
Wall Height : 1.23 Meters.
Floor Span : 2.29 Meters.
Slab Load 3.5
Slab Load 1.75
B.S.5628
Eccentricity(e/t)
.02 .04 .06 .08 .1 .12 .14 .16 .18 .2 .22 .24
.2 .4 .6 .8 1.0 1.2 1.4 1.6 1.8
Precompression Stress N/mm²

Fig.(7.1)

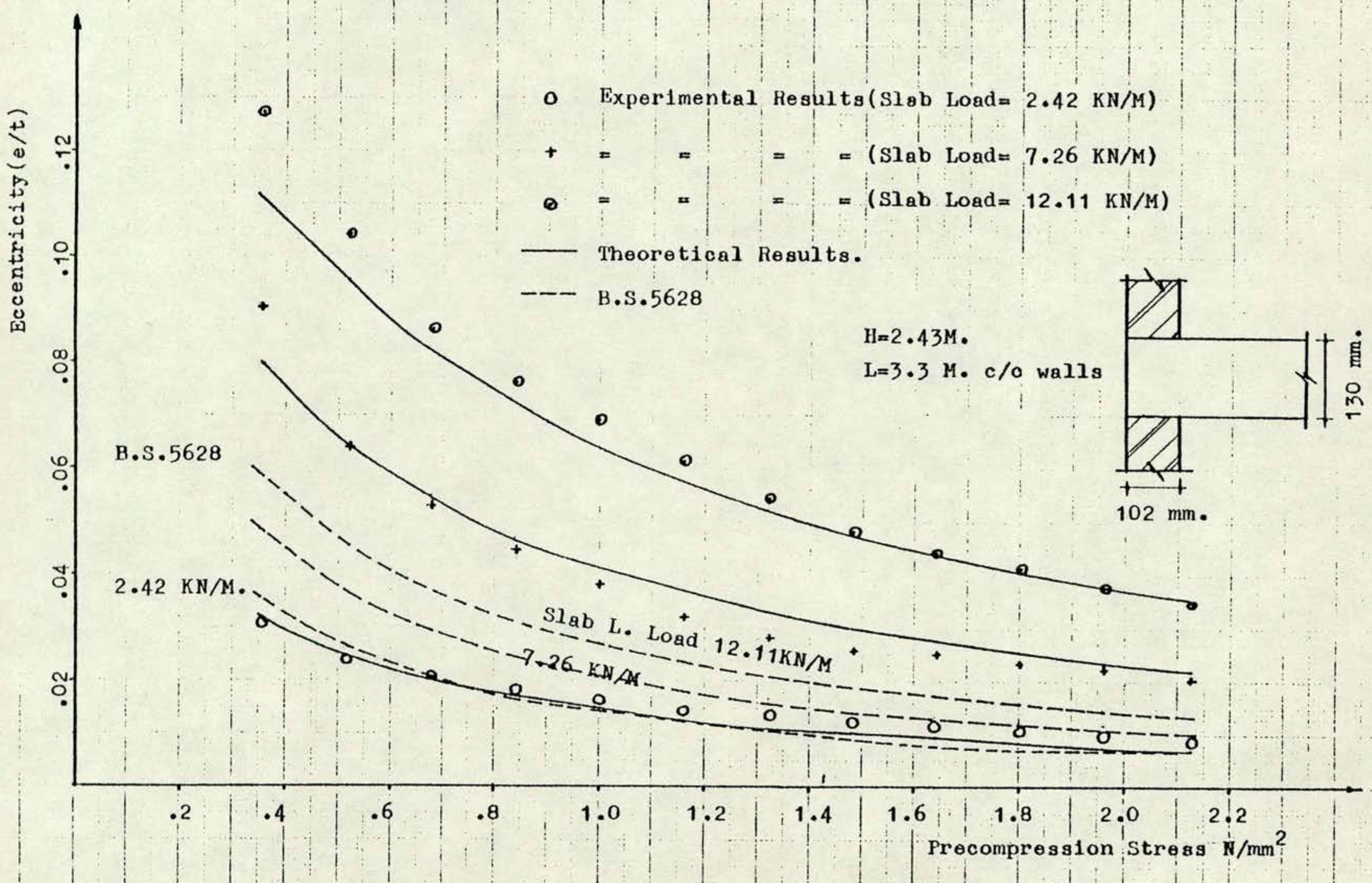
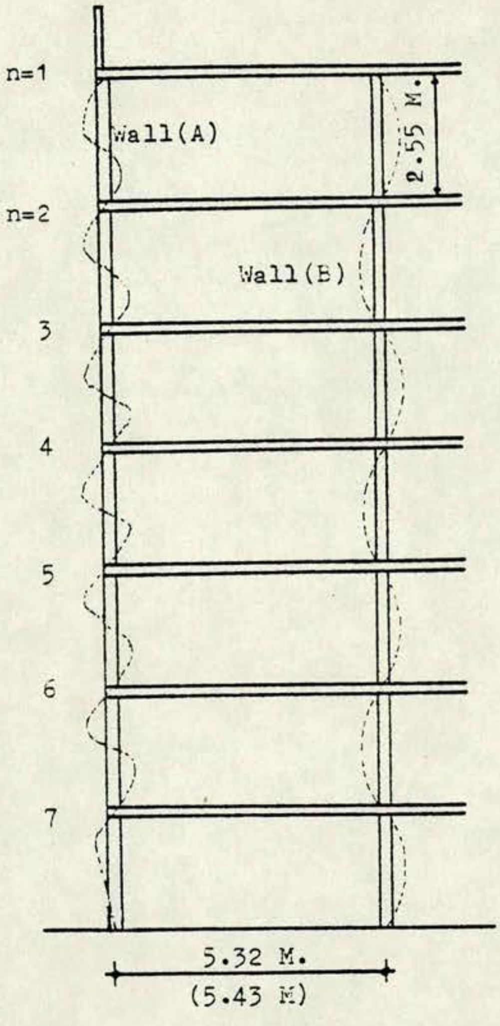
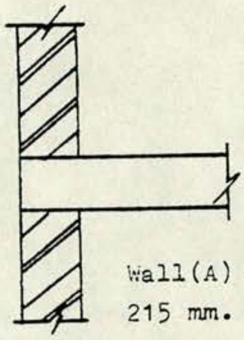

Experimental Results(Slab Load= 2.42 KN/M)
= = = = (Slab Load= 7.26 KN/M)
= = = = (Slab Load= 12.11 KN/M)
Theoretical Results.
B.S.5628
H=2.43M.
L=3.3 M. c/c walls
130 mm.
102 mm.
Eccentricity(e/t)
.02
.04
.06
.08
.10
.12
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
Precompression Stress N/mm2
B.S.5628
2.42 KN/M.
Slab L. Load 12.11KN/M
7.26 KN/M

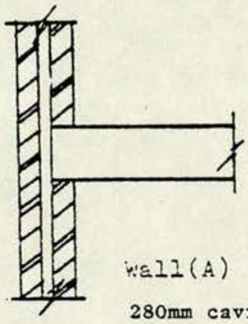
Fig.(7.2)



Clear spans



Wall(A) - Case(1)
215 mm. thick.



Wall(A) - Case(2)
280mm cavity wall

WALL(B) :

215 mm. Solid Internal Wall Case(1).
102.5 mm. Solid Int.Wall(Case2).

Floors. (One-Way Slabs) :

Thickness = 150 mm.
Dead Load = 4 KN/M^2
Live Load = 2 KN/M^2
E = 21 KN/mm^2
$I = 2.8125 * 10^{-4}\ M^4$

WALLS:

Storey Height = 2.55 M.
E = 11 KN/mm^2.
Wall Weight = 14 KN/M^3.
Characteristic Strength of Brickwork, $f_k = 7.9\ N/mm^2$.
Partial Safety Factor, $\gamma_m = 2.5$
$I_1 = 8.28 * 10^{-4}\ M^4$ (Case 1), $I_2 = 0.965 * 10^{-4}\ M^4$ (Case 2)

Fig.(7.3)

Table (7.1)- Wall (A)- Case (1)- Wall Thickness 215 mm.(Double Curvature, $\mathcal{E}_1/\mathcal{E}_2 = -1.0$)

Level	Design Load/Stress		Eccentricity ($\mathcal{E}$)		Reduction Factor (β)		Load Bearing Capacity $\beta \times f_k/\gamma_m$ N/mm^2	
(n)	P_1 (KN)	N/mm^2	Code	Theory	Code *	Theory **	Code	Theory
1	28.41	0.13	0.137	0.355	0.79	0.43	2.50	1.36
2	62.57	0.29	0.063	0.317	0.95	0.56	3.00	1.77
3	96.73	0.45	0.040	0.295	0.98	0.62	3.10	1.96
4	130.89	0.61	0.029	0.292	0.98	0.64	3.10	2.02
5	165.05	0.77	0.024	0.237	0.98	0.79	3.10	2.49
6	199.21	0.93	0.020	0.199	0.98	0.91	3.10	2.87
7	233.37	1.09	0.017	0.178	0.98	1.00	3.10	3.16

* S.R. = 0.75 x 12 = 9 (BS.5628)

** S.R. = 12

Table (7.2)- Wall (A)- Case (2)- Wall Thickness 280.5 mm (Cavity Wall)- (Double Curvature, $\mathcal{E}_1/\mathcal{E}_2=-1.0$)

Level	Design Load/Stress		Eccentricity ($\mathcal{E}$)		Reduction Factor (β)		Load Bearing Capacity $\beta \times f_k/\gamma_m$ N/mm^2	
(n)	P_1 (KN)	N/mm^2	Code	Theory	Code*	Theory**	Code	Theory
1	26.38	0.26	0.150	0.25	0.75	0.68	2.37	2.14
2	55.38	0.54	0.071	0.38	0.87	0.33	2.74	1.04
3	84.38	0.82	0.047	0.50	0.90	0.00	2.84	0.00
4	113.38	1.11	0.035	0.40	0.90	0.27	2.84	0.85
5	142.38	1.39	0.028	0.34	0.90	0.45	2.84	1.42
6	171.38	1.67	0.023	0.29	0.90	0.57	2.84	1.80
7	200.38	1.95	0.019	0.26	0.90	0.65	2.84	2.05

* S.R. = 2.55/(2 x 0.1025 x 2/3) x 0.75 = 14.0

** S.R. = 2.55/(2 x 0.1025 x 2/3) = 18.75

Table (7.3)- Wall (B)-Case (1)- Loading type (i)*- Wall Thickness 215 mm.(Single Curv., $\mathcal{E}_1/\mathcal{E}_2=+1.0$)

Level	Design Load/Stress		Eccentricity ($\mathcal{E}$)		Reduction Factor (β)		Load Bearing Capacity $\beta \times f_k/\gamma_m$ N/mm^2	
(n)	P_1 (KN)	N/mm^2	Code	Theory	Code	Theory	Code	Theory
2	87.35	0.40	0.0	0.083	0.98	0.90	3.10	2.84
3	145.01	0.67	0.0	0.077	0.98	0.94	3.10	2.97
4	194.11	0.90	0.0	0.060	0.98	1.00	3.10	3.16
5	243.21	1.13	0.0	0.049	0.98	1.00	3.10	3.16
6	292.31	1.36	0.0	0.041	0.98	1.00	3.10	3.16
7	341.41	1.58	0.0	0.036	0.98	1.00	3.10	3.16

* Loading type (i) : All Floors have been assumed to be loaded to one side of the wall.

Table (7.4)- Wall (B)-Case (2)-Loading type (i)-Wall Thickness 102.5 mm.(Single Curv., $\mathcal{E}_1/\mathcal{E}_2=+1.0$)

Level	Design Load/Stress		Eccentricity ($\mathcal{E}$)		Reduction Factor (β)		Load Bearing Capacity $\beta \times f_k/\gamma_m$ N/mm^2	
(n)	P_1 (KN)	N/mm^2	Code	Theory	Code	Theory	Code	Theory
2	83.29	0.81	0.0	0.053	0.77	0.61	2.43	1.92
3	136.43	1.33	0.0	0.043	0.77	0.70	2.43	2.21
4	180.77	1.76	0.0	0.035	0.77	0.71	2.43	2.24
5	225.10	2.19	0.0	0.029	0.77	0.75	2.43	2.37
6	269.44	2.62	0.0	0.025	0.77	0.77	2.43	2.43
7	313.77	3.06	0.0	0.022	0.77	0.78	2.43	2.46

Table (7.5)-Wall (B)-Case (1)-Loading type (ii)*-Wall thickness 215 mm.(Single Curv., $\mathcal{E}_1/\mathcal{E}_2=0.0$)

Level	Design Load/Stress		Eccentricity ($\mathcal{E}$)		Reduction Factor (β)		Load Bearing Capacity $\beta \times f_k/\gamma_m$ N/mm^2	
(n)	P_1 (KN)	N/mm^2	Code	Theory	Code	Theory	Code	Theory
2	95.91	0.44	0.0	0.083	0.98	1.0	3.10	3.16
3	153.47	0.71	0.0	0.074	0.98	1.0	3.10	3.16
4	202.67	0.94	0.0	0.059	0.98	1.0	3.10	3.16
5	251.77	1.17	0.0	0.048	0.98	1.0	3.10	3.16
6	300.87	1.40	0.0	0.041	0.98	1.0	3.10	3.16
7	349.97	1.63	0.0	0.036	0.98	1.0	3.10	3.16

* Loading type (ii): All floors above the level under consideration have been assumed loaded and the floor at this level loaded to one side of the wall only.

Table (7.6)-Wall (B)-Case (2)-Loading type (ii)-Wall thickness 102.5 mm.(Single Curv., $\mathcal{E}_1/\mathcal{E}_2=0.0$)

Level	Design Load/Stress		Eccentricity ($\mathcal{E}$)		Reduction Factor (β)		Load Bearing Capacity $\beta \times f_k/\gamma_m$ N/mm^2	
(n)	P_1 (KN)	N/mm^2	Code	Theory	Code	Theory	Code	Theory
2	91.97	0.89	0.0	0.053	0.77	0.82	2.43	2.59
3	144.86	1.41	0.0	0.042	0.77	0.84	2.43	2.65
4	189.00	1.84	0.0	0.034	0.77	0.86	2.43	2.71
5	233.78	2.28	0.0	0.029	0.77	0.89	2.43	2.81
6	278.12	2.71	0.0	0.025	0.77	0.90	2.43	2.84
7	322.45	3.14	0.0	0.022	0.77	0.91	2.43	2.87

loading conditions specified. The Code bearing capacity is, however, much higher than the design load throughout.

Two loading cases have been calculated for the internal walls. In the first, all floors have been assumed to be loaded to one side of the wall and in the second, all floors above the level under consideration have been assumed loaded and the floor at this level loaded to one side of the wall only. The first case produces equal eccentricities of the same sign at the ends of the walls, whilst the second gives zero eccentricity at one end but with a slightly higher axial load. The results of these calculations are shown in Tables (7.3 - 7.6). The eccentricities in all cases are quite small, being taken as zero in the Code method.

Both methods give the same result for the 215 mm wall but there are differences which could be significant in considering 102.5 mm walls. Thus, in Table (7.6) the theoretical method would indicate that the selected masonry type would be suitable for up to six storeys whereas the Code method would be suitable only to five storeys.

In general, it may be concluded from these examples that for external walls the Code method under-estimates eccentricities and thus tends to over-estimate the bearing capacity of walls of given masonry strength in buildings of up to six storeys. The differences are less pronounced for internal walls and no significant differences are to be expected for 215 mm walls. For 102.5 mm walls, the Code method sometimes over-estimates and sometimes under-estimates the bearing capacity as compared with the theoretical method. These differences would again be more pronounced in buildings of up to six storeys. In other words, the accurate estimation of eccentricity and related bearing capacity is of greatest importance in relatively low rise buildings with

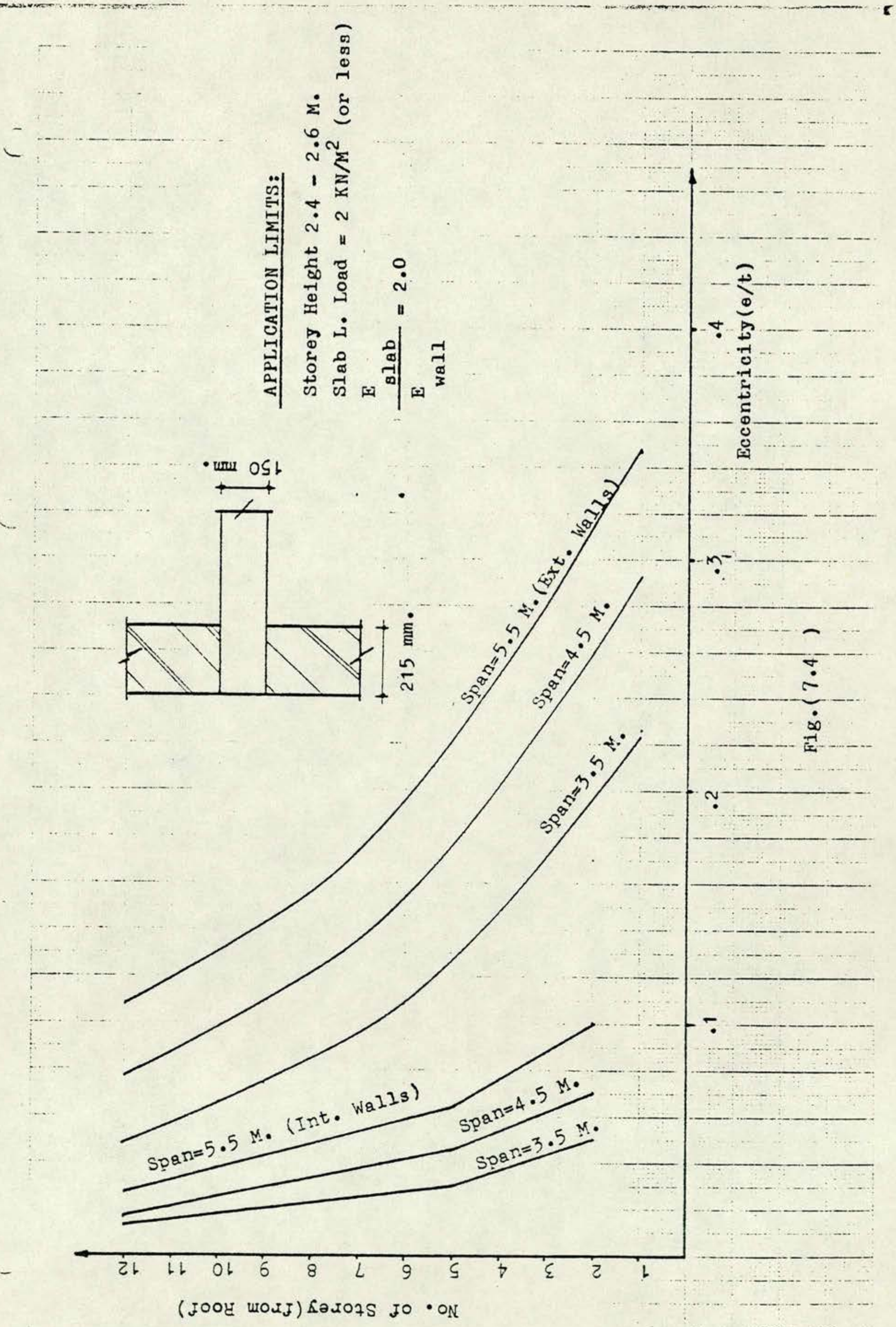
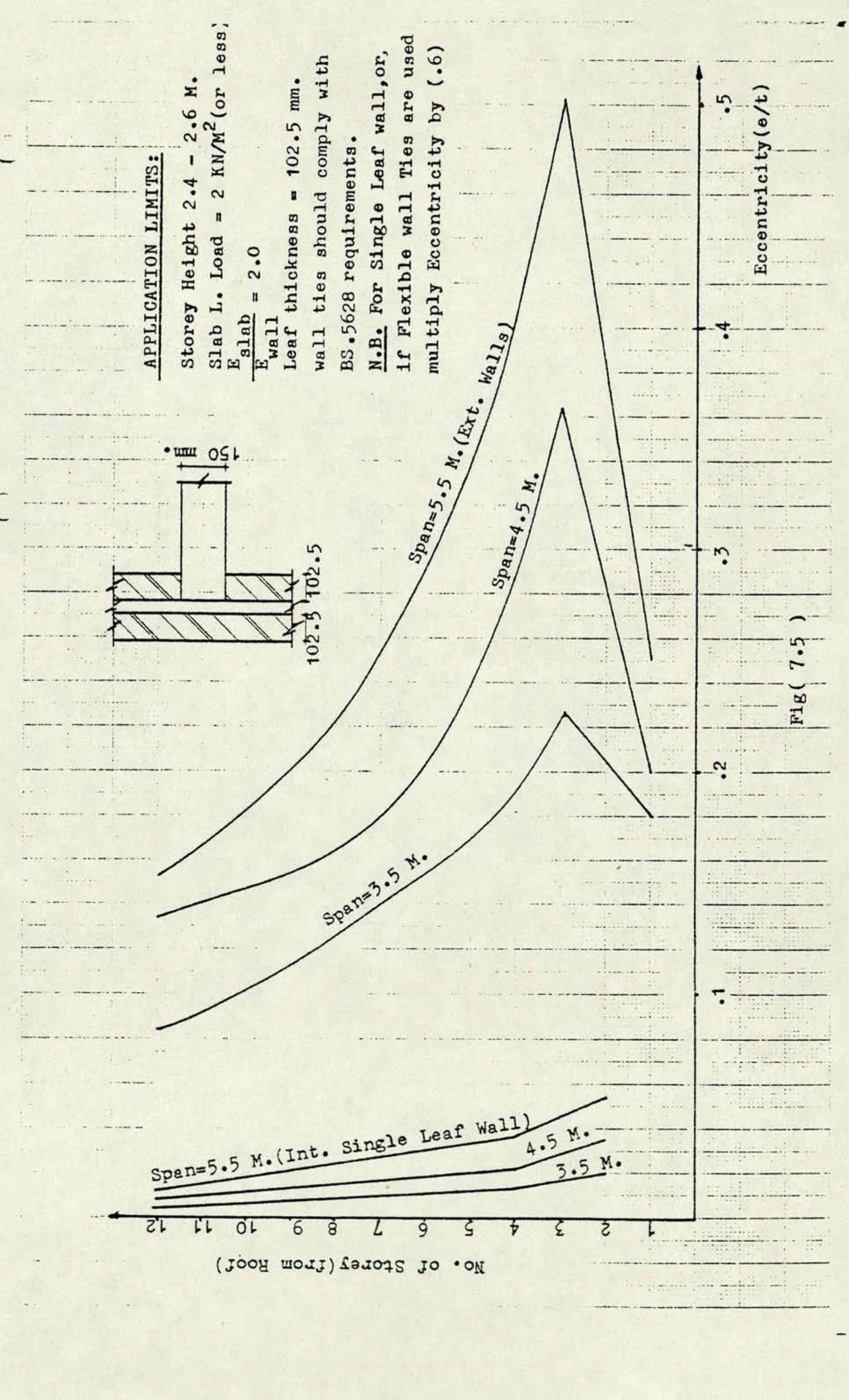

APPLICATION LIMITS:
Storey Height 2.4 - 2.6 M.
Slab L. Load = 2 KN/M^2 (or less)
$\frac{E_{slab}}{E_{wall}} = 2.0$
150 mm.
215 mm.
Span=5.5 M.(Ext. Walls)
Span=4.5 M.
Span=3.5 M.
Span=5.5 M. (Int. Walls)
Span=4.5 M.
Span=3.5 M.
Eccentricity(e/t)
.1
.2
.3
.4
No. of Storey(from Roof)
1
2
3
4
5
6
7
8
9
10
11
12

Fig.(7.4)


APPLICATION LIMITS:
Storey Height 2.4 - 2.6 M.
Slab L. Load = 2 KN/M^2(or less)
$\frac{E_{slab}}{E_{wall}} = 2.0$
Leaf thickness = 102.5 mm.
wall ties should comply with
BS.5628 requirements.
N.B. For Single Leaf wall,or,
if Flexible wall Ties are used
multiply Eccentricity by (.6)
150 mm.
102.5
102.5
Span=5.5 M.(Ext. Walls)
Span=4.5 M.
Span=3.5 M.
Span=5.5 M.(Int. Single Leaf Wall)
4.5 M.
3.5 M.
No. of Storey(from Roof)
Eccentricity(e/t)
.1
.2
.3
.4
.5
1
2
3
4
5
6
7
8
9
10
11
12

Fig(7.5)

slender walls. In such cases the load capacities indicated by the Code method may be significantly in error and further examination of this case would seem to be necessary.

As a quick aid in practical design procedure, the eccentricities calculated for internal and external walls for various floor spans and for two wall thicknesses are shown in Figures (7.4) and (7.5). Eccentricities decrease down the height of the building from roof level and are quite small for internal walls for both wall thicknesses. For external walls, the eccentricities are much larger and decrease considerably with the number of floors. The eccentricities given by these figures cover a range of practical cases and could be used in design within the limits indicated. It should be noted, however, that to obtain accurate results, these eccentricities should be applied in conjunction with the reduction factors given in Chapters Two and Three for walls bent in double and single curvature respectively.

7.3 DESIGN CALCULATIONS FOR A TYPICAL WALL:

Consider design of a 280 mm outer cavity wall for a six storey building assuming that the slab loads are entirely transmitted by the inner leaf (see Figure 7.6).

Floor span: 3.5 Metres

Storey height: 2.55 m., walls bent in double curvature at all storeys above ground floor. Ground floor wall bent in single curvature, $\varepsilon_1/\varepsilon_2 = 0.0$

Design Loads:	Dead Load		4 KN/m^2	γ_f = 1.4
	Superimposed Load		2 KN/m^2	γ_f = 1.6
	Wall self weight		14 KN/m^3	γ_f = 1.4
	Slab Loads:	D.L.	3.5 x 4 x 1.4	= 19.6
		S.L.L.	3.5 x 2 x 1.6	= 11.2
			Design Load	= 30.8 KN/m
	Wall Loads:		2.55 x 0.103 x 14 x 1.4	= 5.12 KN/m

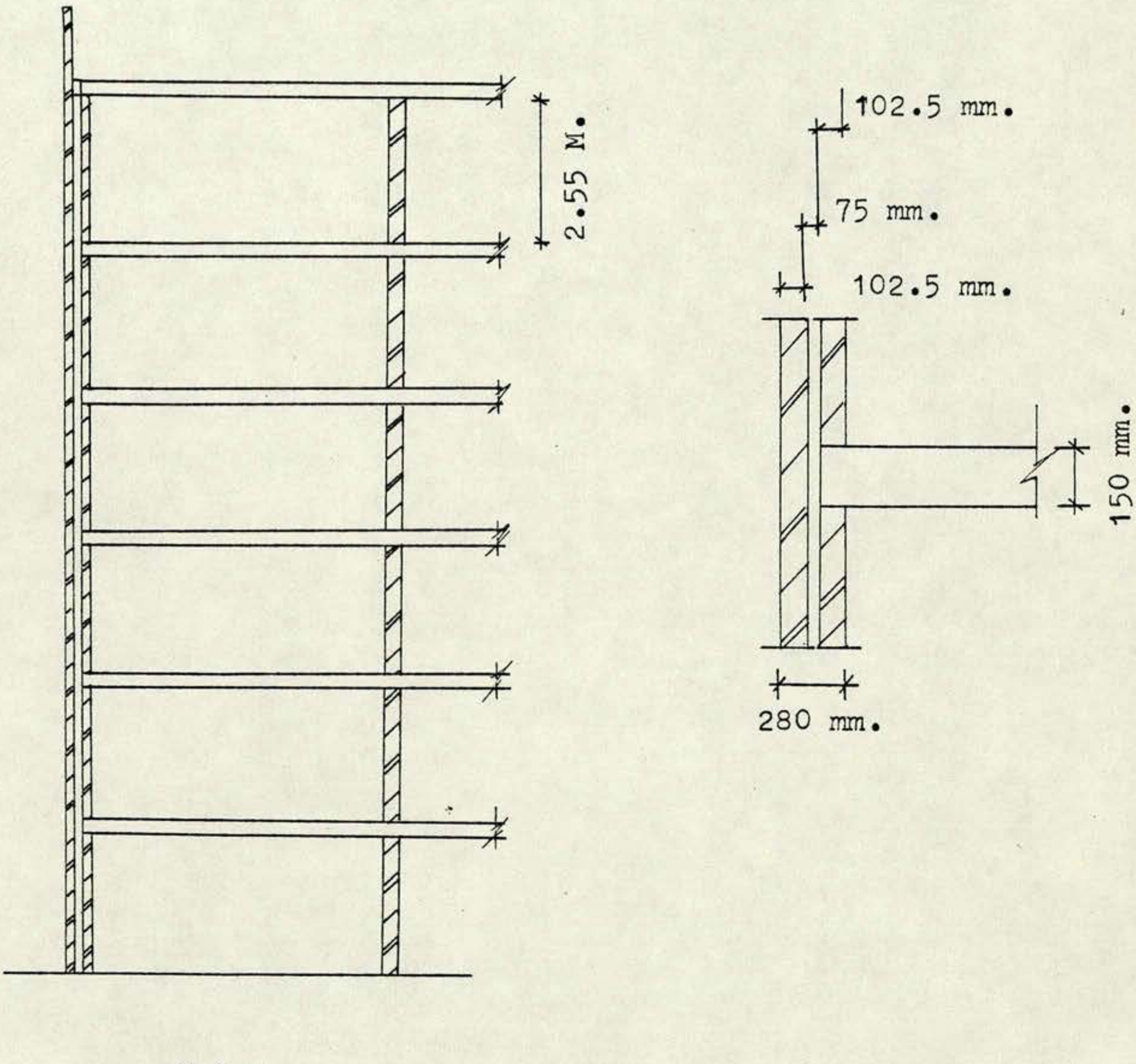


Figure (7.6)

<u>Stiffness Parameters:</u>

Walls: $E = 11\ KN/mm^2$

$I = 2 \times 1/12 \times (0.1025)^3 \qquad = 1.79 \times 10^{-4}\ m^4$

Slabs: $E = 21\ KN/mm^2$

$I = 1/12 \times (0.15)^3 \qquad = 2.81 \times 10^{-4}\ m^4$

$$\bar{K} = \frac{2(EI)_s}{(EI)_w} \cdot \frac{H}{L}$$

$$= \frac{2 \times 21 \times 2.81 \times 10^{-4}}{11 \times 1.79 \times 10^{-4}} \times \frac{2.55/2}{3.5} = 2.18$$

(Double curvature)

$$\alpha = \frac{\bar{K}}{2} = 1.09 \text{ (Double curvature)}$$

$\bar{K} = 4.36$ (Single curvature)

$\alpha = 2.18$ (Single curvature)

<u>Rigid Frame Moments:</u>

$$\bar{M}_R = \frac{WL^2}{12} \left(\frac{3}{3 + \alpha}\right)$$

$$= \frac{30.8 \times 3.5}{12} \times \frac{3}{4.09}$$

$= 6.59$ KN.m/m (For all Floors except First Floor)

$$\bar{M}_R = \frac{30.8 \times 3.5}{12} \times \frac{3}{5.18} = 5.20 \text{ KN.m/m (First Floor)}$$

<u>Slenderness Ratio:</u>

$$S.R. = \frac{2.55}{2 \times 0.103 \times 0.67} = 18.5$$

<u>Eccentricity:</u>

$$\varepsilon = \frac{M_R . R . \psi}{P_L . t . (\acute{\psi} R + \bar{K})} \times [1/\acute{\psi}]$$

(The term with the square brackets is used when the precompression is less than 0.3 N/mm^2, the average with and without this term adopted is in the vicinity of this value.)

Level (n)	Precomp. on joint (N/mm^2)	P_u (KN)	P_L (KN)	ψ	$\acute{\psi}$	R	ε
1	0.025	2.56*	17.96	0.14	1.14	1.275/2.345	0.16/0.21**
2	0.225	23.06	38.46	0.60	1.60	1.275	0.19
3	0.425	43.58	58.98	0.74	1.74	1.275	0.23
4	0.625	64.12	78.52	0.82	1.82	1.275	0.19
5	0.815	83.64	99.04	0.85	1.85	1.275/2.345	0.15/0.19**
6	1.015	104.16	119.56	0.87	1.87	1.85	0.087

* Assumed parapet height = 1.2 metre

** Transition from cracked to uncracked section - mean value adopted.

Adopting masonry characteristic strength, f_K = 5.8 N/mm^2, and γ_m = 3.5:

Level (n)	ε	ε_f	β	$\beta . f_K/\gamma_m$ N/mm^2	Design Stress N/mm^2
1	0.18	0.48	0.87	1.44	0.18
2	0.19	0.46	0.83	1.38	0.38
3	0.23	0.44	0.72	1.19	0.58
4	0.19	0.43	0.83	1.38	0.77
5	0.17	0.41	0.89	1.47	0.97
6	0.087	0.39	0.91	1.50	1.16

The calculated eccentricity at each level is less than the failure eccentricity ε_f. Comparison of the last two columns indicates that the selected masonry strength is adequate at all levels.

CHAPTER 8 - GENERAL CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK:-

8.1 CONCLUSIONS:

The following conclusions have been reached as a result of the investigations presented in this thesis:

1. The capacity reduction factors equations and consequently the wall bearing capacity equations presented in Chapters (2) and (3) were compared with test results and these comparisons showed good agreement between test and theory.

2. A method is set out for calculating the eccentricity at wall/floor slab joints in brickwork masonry structures, taking into account the interaction between walls and floor slabs is shown to give results in accordance with those obtained experimentally and could be applied in design.

3. The solution presented in this thesis permits the calculation of the load bearing capacity of walls compressed between floor slabs. For convenience in practical design calculations, the results are presented in terms of capacity reduction factors which allow for slenderness and eccentricity and for the deflected shape of the wall. The latter is usually apparent from the loading pattern and geometry of the structure but may if necessary be found by structural analysis.

4. Comparison with results for eccentricities and load bearing capacities calculated by the methods presented in this thesis and by the procedures given in BS 5628 show considerable difference in particular cases. For example it would appear that the eccentricity in a cavity wall can, under certain circumstances, be very much higher, and the load bearing capacity correspondingly lower, than it would appear from the Code. These cases can arise in low rise structures or

in the upper floors of high rise buildings. In walls more than about eight floors from the roof of a building, eccentricity is unlikely to be significant.

5. It is considered that the solution presented in this thesis together with the determination of eccentricity allowing for wall/floor slab interaction results in a more accurate assessment of wall bearing capacity than conventional empirical procedures.

6. The results of the test programme conducted on a half scale two-storey, single bay frame and on a full scale three-storey, two bay structure are given and load eccentricities computed from the test data are compared with values obtained from the theory developed herein. As a result of the experimental work presented, the following conclusions are given:

A) A significant restraining moment can be developed at a floor/wall joint if the ratio of the floor/wall stiffness is small even at relatively low precompression stresses.

B) Increasing the precompression stress increases the joint rigidity (although this relationship is not linear) up to a maximum value depending on the ratio of floor/wall stiffness, but where this ratio is large, increasing the precompression stress increases the joint fixity by a marginal value.

C) The magnitude of the floor live load does not have a significant effect on the degree of fixity obtained at a floor/wall joint, particularly at higher precompression stresses.

(D) Complete joint fixity was not obtained in tests even at high precompression stresses for the half scale model although the ratio of the floor/wall stiffness is quite small and may require a smaller ratio of stiffnesses which would be uneconomical for practical cases.

(E) The sequence of loading does not significantly influence the slab restraining moments that can be developed at the joints of a load bearing masonry structure. Thus, in design, it is not necessary to distinguish between different methods of floor construction, but, special consideration should be made in the design of the slabs for positive and/or negative bending moments that will develop depending on the ratio of the floor/wall stiffness.

8.2 RECOMMENDATIONS FOR FUTURE WORK:

The work covered in this thesis provides comprehensive theoretical and experimental study on the compressive strength of brick masonry walls supporting one way or two way span systems and the joint eccentricities developed due to gravity loadings.

It is recommended that, for future work, the following may be considered:

1) The joint eccentricities due to gravity loading developed at wall/floor slab joints where the reinforced concrete slabs are stiffened by end beams.

2) Confirmation of calculated joint eccentricities due to gravity loading at joints where reinforced concrete floor slabs are supported by cavity walls.

3) Experimental confirmation of calculated eccentricities where floor slabs are supported on three or four sides.

4) Eccentricities at wall/floor slab joints differing from those in the experimental structures tested in this programme, e.g. partially supported floor slabs.

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APPENDIX (A)

A.1. Development of Basic Equations (Section 2.3)

Equations (2.3 and 2.4) from Section(2.3.1) are given below

$$e = \frac{2U_oS}{H_s}(1 - S/2H_s) \quad \text{(A1)}$$

$$\omega = \frac{4U_o}{H_s}(1 - S/H_s) \quad \text{(A2)}$$

Introducing the notation:

$$\varepsilon = e/t \quad \text{(A3)}$$

$$\Omega = (H/2/t).\omega \quad \text{(A4)}$$

$$\Phi = (H/2/t).\emptyset \quad \text{(A5)}$$

Substituting Equation (A2) into Equation (A4) gives:

$$\Omega = (H/2t).(4U_o/H_s).(1 - S/H_s) \quad \text{(A6)}$$

Substituting $H_s = H/2 + S/2$ we obtain:

$$\Omega = \frac{4U_o}{t}.\frac{(1 - S/H)}{(1 + S/H)^2} \quad \text{(A7)}$$

Equation (A7) corresponds to Equation (2. 8).

Also substituting Equation (A1) into Equation (A3) gives:

$$\varepsilon = \frac{2\,U_oS}{H_s.t}.(1 - S/2H_s) \quad \text{(A8)}$$

Substituting, $H_s = H/2 + S/2$ into Equation (A8) we obtain:

$$\varepsilon = \frac{4\,U_o}{t}.\frac{S}{H}.\frac{1}{(1 + S/H)^2} \quad \text{(A9)}$$

Equation (A9) corresponds to Equation (2.9).

Also, since:

$$\Phi = \Omega + \varepsilon \quad \text{(A10)}$$

Substituting Equations (A7) and (A9) into Equation (A10) gives:

$$\Phi = \frac{4U_o}{t} \cdot \frac{(1 - S/H)}{(1 + S/H)^2} + \frac{4U_o}{t} \cdot \frac{S}{H} \cdot \frac{1}{(1 + S/H)^2}$$

$$= \frac{4U_o}{t(1 + S/H)^2} \quad \text{(A11)}$$

From which:

$$\frac{U_o}{t} = \frac{\Phi}{4} \cdot (1 + S/H)^2 \quad \text{(A12)}$$

Dividing Equation (A9) by Equation (A11) gives:

$$\frac{\varepsilon}{\Phi} = \frac{\frac{4U_o}{t} \cdot \frac{S}{H} \cdot \frac{1}{(1 + S/H)^2}}{\frac{4U_o}{t(1 + S/H)^2}} \quad \text{(A13)}$$

Thus, $\frac{\varepsilon}{\Phi} = \frac{S}{H}$ .... (A14)

Equation (A14) corresponds to Equation (2.13)

By following the same procedure outlined earlier, the basic equations for the case of walls bent in single curvature with zero eccentricity at one end can be developed, noting that:-

$H_s = H + S/2$ .... (A15)

$\varepsilon = e/t$ .... (A3)

$\Omega = (H/t).\omega$ .... (A16)

And, $\Phi = (H/t).\emptyset$ .... (A17).

A.2. <u>Determination of the equivalent uniformly distributed loading</u>:-

A.2.1 <u>Half scale model frame</u>:

The midspan bending moment due to point loads w_1 is (22)

$$M_1 = \frac{W_1 L}{3} \quad \text{(A18)}$$

The equivalent bending moment due to W_2;

$$M_2 = \frac{W_2 L}{4} \quad \text{......................} \quad (A19)$$

Equating M_1 and M_2 yields:-

$$W_2 = \frac{4}{3} W_1 \quad \text{......................} \quad (A20)$$

The equivalent bending moment due to U.D. load, q:

$$M_3 = \frac{qL^2}{8} \quad \text{......................} \quad (A21)$$

Equating M_2 and M_3 yields:-

$$\frac{qL^2}{8} = \frac{W_2 L}{4}$$

From which:

$$q = \frac{8}{3} \cdot \frac{W_1}{L} \quad \text{...................} \quad (A22)$$

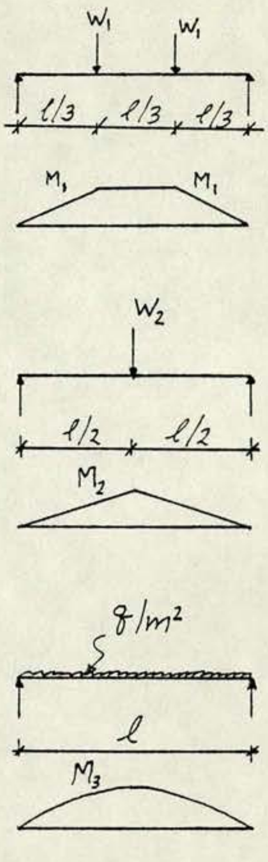

Fig.(A1)

Thus, the equivalent U.D. Load is obtained from Equation (A22) for the half scale model frame.

A.2.2. Full size structure:

The midspan bending moment due to W_1, with only one span loaded (23)

$$M_1 = W_1 L/6 \quad \text{..................} \quad (A23)$$

The midspan bending moment due to W_2 is:

$$M_2 = \frac{5W_2 L}{32} \quad \text{..................} \quad (A24)$$

Equating equations (A23) and (A24) yields:

$$W_2 = \frac{16W_1}{15} \quad \text{..................} \quad (A25)$$

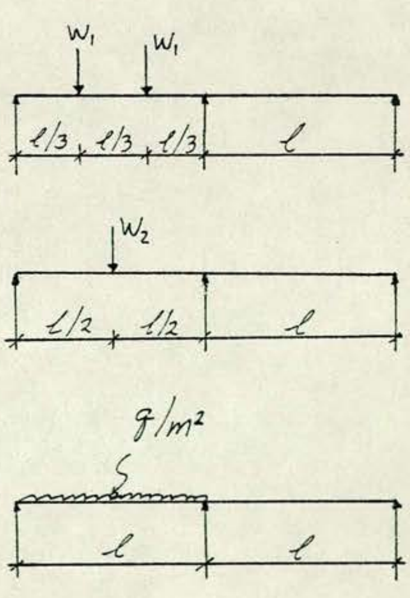

Fig.(A2)

The midspan bending moment due to the U.D.L. is:

$$M_3 = \frac{qL^2}{16} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A26)}$$

Equating Equations (A24) and (A26) and applying Equation (A25) gives:

$$q = \frac{8}{3} \cdot \frac{W_1}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A27)}$$

Equation (A27) gives the equivalent U.D.L. for the full size structure due to point loads applied at third points of the span (one span loaded only).

A.3 Determination of Experimental slab restraining moments:

A.3.1 Half scale model frame:

A.3.1.1 Based on midspan deflection:-

For the beam shown in Figures (A.3), the midspan deflection of the beam (Figure a) due to floor loads application, P;

$$\Delta_{c_1} = 23PL^3/648EI \ \ldots\ldots\ldots\ldots\ldots\ldots \text{(A28)}$$

The equivalent bending moment, M_o, that would induce the same midspan deflection Δ_c (Fig. b)

$$M_o = 8E\Delta_c/L^2 \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A29)}$$

From which;

$$\Delta_{c_1} = M_oL^2/8EI \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A30)}$$

Equating Equations (A28) and (A30) gives:

$$M_o = 23PL/81 = 0.284\ PL \ \ldots\ldots\ldots \text{(A31)}$$

The net slab restraining moment, M_Δ, developed due to midspan deflection, may be written as (see Figures b and c)

$$M_\Delta = M_o - M_R \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(A32)}$$

Where, M_R is the joint restraining moment developed due to the slab end restraint, whence;

$$M_\Delta = 0.284PL - 8EI\Delta/L^2 \ \ldots\ldots\ldots\ldots \text{(A33)}$$

Equation (A33) corresponds to Equation (6.10).

The slab restraining moments induced due to the application of wall precompression at both ends is:

$M_{\Delta} = 8EI\Delta/L^2$ ....(A34)

Equation (A34) corresponds to Equation (6.8).

A.3.1.2 Based on end rotation:

The floor slab end rotation may be given (20) as:

Due to floor loads application, P:

$\theta_{end} = PL^2/9EI$ ....(A35)

And due to the application of the equivalent moment, M_o,

$\theta_{end} = M_oL/2EI$ .... (A36)

Equating Equations (A35) and (A36) we obtain:

$M_o = 2PL/9$ .... (A37)

Thus, the net slab restraining moment, M_θ, based on end rotation of the floor slab becomes:

$M_\theta = 2PL/9 - 2EI\ \theta/L$ .... (A38)

Equation (A38) corresponds to Equation (6.11).

The slab restraining moment due to the application of wall precompression at both ends is equal to:

$M_\theta = 2EI\ \theta/L$ .... (A39)

Equation (A39) corresponds to Equation (6.9).

A.3.2 Full size test structure

A.3.2.1 Based on midspan deflection:-

Consider Figure (A4), from which Part (AB) of the continuous beam is assumed equivalent to the beam shown in (b). Thus, the midspan deflection due to floor loads equals:

$\Delta c_1 = 23PL^3/648EI - \dfrac{(PL/6).L^2}{16EL}$ ..... (A40)

Thus:

$\Delta c_1 = 65PL^3/2592EI$ .... (A41)

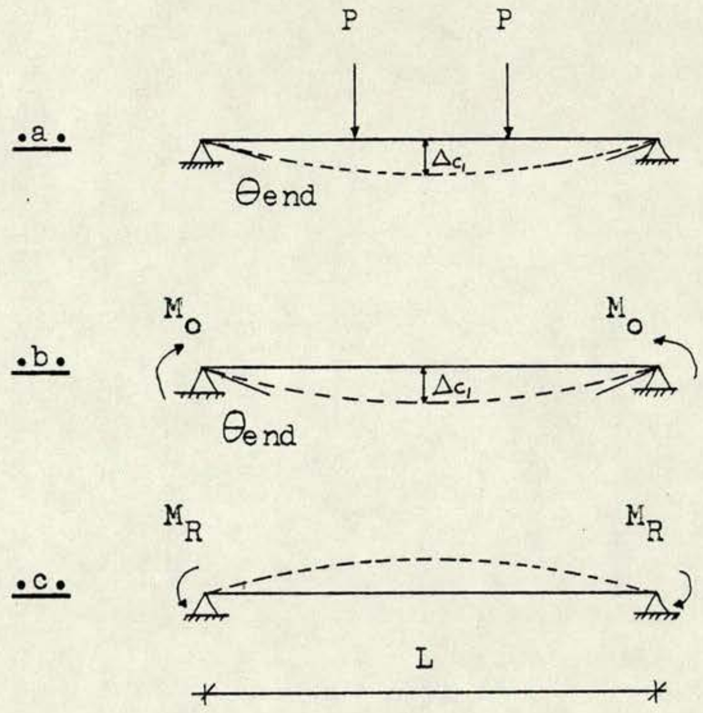
P
P
.a.
Δ_{c_1}
θ_{end}
M_o
M_o
.b.
Δ_{c_1}
θ_{end}
M_R
M_R
.c.
L

Fig.(A.3)

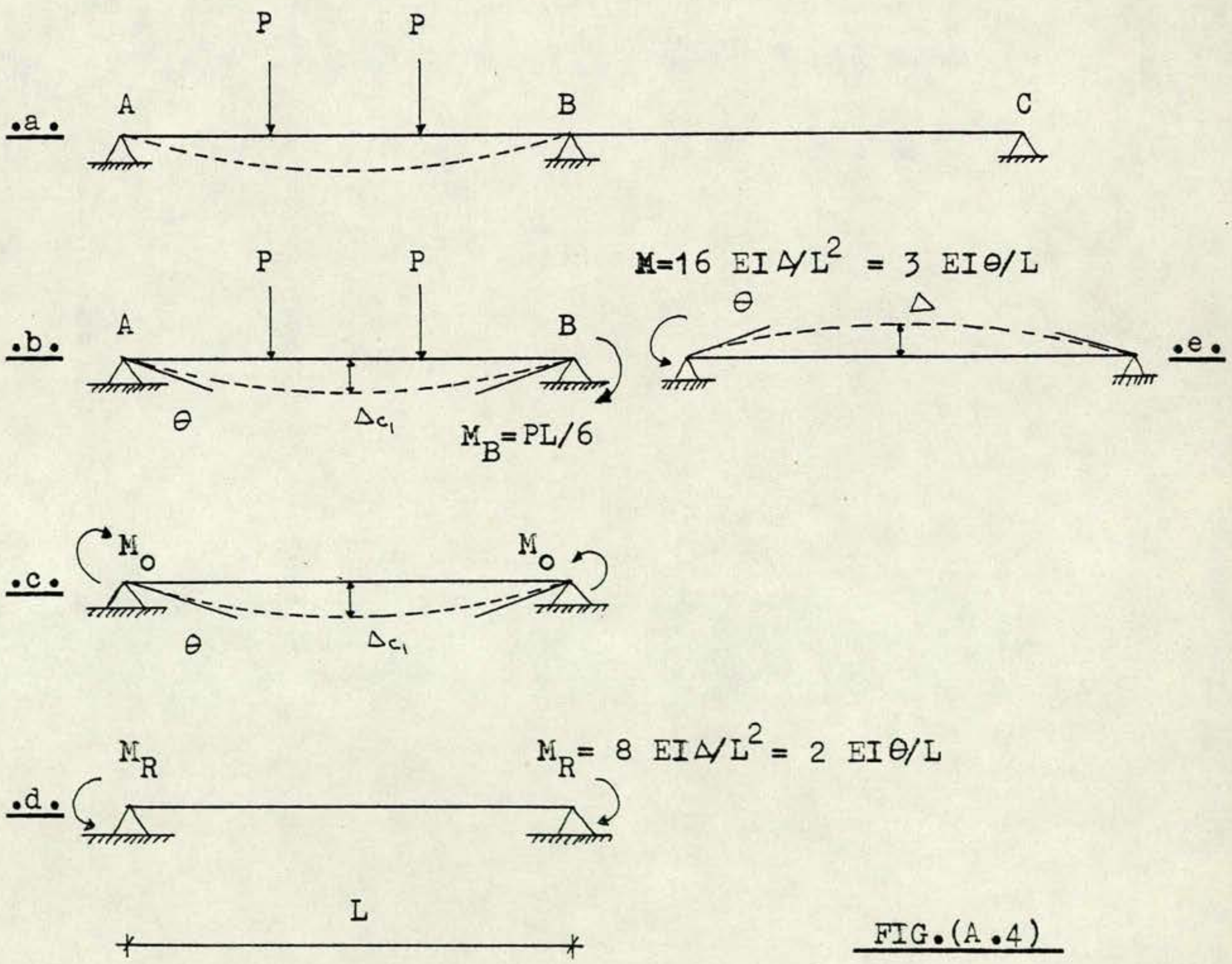
P
P
.a.
A
B
C
P
P
$M=16\ EI\Delta/L^2 = 3\ EI\theta/L$
θ
Δ
.b.
A
B
.e.
θ
Δ_{c_1}
$M_B=PL/6$
M_o
M_o
.c.
θ
Δ_{c_1}
M_R
$M_R= 8\ EI\Delta/L^2= 2\ EI\theta/L$
.d.
L

FIG.(A.4)

The equal midspan deflection that would be induced due to the end moments, M_o, is (Fig c)

$\Delta c_1 = M_o L^2/8EI$ .... (A42)

Equating Equations (A41) and (A42) gives:

$M_o = 0.20\ PL$ .... (A43)

Thus, the net slab restraining moment, M_Δ, is (Figs c and d):

$M\Delta = 0.20\ PL - 8EI\Delta/L^2$ .... (A44)

Equation (A44) corresponds to Equation (6.18).

The slab restraining moment, M_Δ, due to the application of the wall precompression (one wall precompressed only):

$M_\Delta = 16\ EI\Delta/L^2$ .... (A45)

Equation (A45) corresponds to Equation (6.16).

A.3.2.2 Based on end rotation:

The slab end rotation is given as:

$\theta_{end} = PL^2/9EI - (PL/6).L/6EI$ .... (A46)

Thus;

$\theta_{end} = PL^2/12EI$ .... (A47)

The equivalent bending moment, M_o, that would induce the equal end rotation is:

$M_o = 2EI\ \theta/L$ .... (A48)

From which;

$\theta_{end} = M_o L/2EI$ .... (A49)

Equating equations (A47) and (A49) gives:

$M_o = PL/6$ .... (A50)

And the net slab restraining moment, M_θ, becomes:

$M_\theta \; = \; PL/6 - 2EI\theta/L$ .... (A51)

Equation (A51) corresponds to Equation (6.19).

The slab restraining moment due to the application of wall precompression to the outer wall only is:

$M_\theta \; = \; 3EI\theta/L$ .... (A52)

Equation (A52) corresponds to Equation (6.17).

APPENDIX (B) - TEST RESULTS

Table (B1)- Test Series (C)- Half Scale Model.

Slab Load (KN)	Lower Slab Deflection (mm)*	Lower Slab Rotation x 10^{-3} (Radians)**
1.0	+ 1.03	- 1.35
1.5	+ 1.51	- 1.90
2.0	+ 2.03	- 2.70
2.5	+ 2.60	- 3.50
3.0	+ 3.19	- 4.15
3.5	+ 3.34	- 4.35
4.0	+ 3.67	- 4.65

* Positive Sign " Downward Deflection "

** Negative Sign " Clockwise Rotation "

Table (B2)- Test Series (D)- Half Scale Model.

Slab Load (KN)	Active Wall Prec. (KN)	Total Wall Prec. (N/mm^2)	Lower Slab Rot. x 10^{-3} (Radians)	Lower Slab Deflection (mm)	Due to Precomp.		Due to F.Load	
					θ	Δ	θ	Δ
0.00	10.00	0.267	+ 0.52	- 0.37	+ 0.52	- 0.37	----	----
1.50	10.00	0.267	- 0.94	+ 0.92	----	----	- 1.46	+ 1.29
0.00	20.00	0.462	+ 0.66	- 0.49	+ 0.66	- 0.49	----	----
1.50	=	=	- 0.35	+ 0.57	----	----	- 1.01	+ 1.06
0.00	30.00	0.656	+ 0.74	- 0.53	+ 0.74	- 0.53	----	----
1.50	=	=	- 0.19	+ 0.38	----	----	- 0.93	+ 0.91
0.00	40.00	0.854	+ 0.79	- 0.52	+ 0.79	- 0.52	----	----
1.50	=	=	- 0.05	+ 0.33	----	----	- 0.84	+ 0.85
0.00	50.00	1.050	+ 0.86	- 0.50	+ 0.86	- 0.50	----	----
1.50	=	=	+ 0.10	+ 0.31	----	----	- 0.76	+ 0.81
0.00	60.00	1.246	+ 0.90	- 0.46	+ 0.90	- 0.46	----	----
1.50	=	=	+ 0.20	+ 0.32	----	----	- 0.70	+ 0.78
0.00	70.00	1.442	+ 0.90	- 0.42	+ 0.90	- 0.42	----	----
1.50	=	=	+ 0.26	+ 0.33	----	----	- 0.64	+ 0.75
0.00	80.00	1.638	+ 0.87	- 0.39	+ 0.87	- 0.39	----	----
1.50	=	=	+ 0.25	+ 0.34	----	----	- 0.62	+ 0.73

Table (B3)- Test Series (E)- Half Scale Model.

Slab Load (KN)	Active Wall Prec. (KN)	Total Wall Prec. (N/mm^2)	Lower Slab Rot.x 10^{-3} (Radians)	Lower Slab Deflection (mm)	Due to Prec.		Due to F.Load	
					θ	Δ	θ	Δ
0.00	10.00	0.267	+ 0.52	- 0.37	+ 0.52	- 0.37	----	----
3.00	10.00	0.267	- 2.55	+ 2.27	----	----	- 3.07	+ 2.64
0.00	20.00	0.462	+ 0.66	- 0.49	+ 0.66	- 0.49	----	----
3.00	=	=	- 1.65	+ 1.54	----	----	- 2.31	+ 2.03
0.00	30.00	0.658	+ 0.74	- 0.53	+ 0.74	- 0.53	----	----
3.00	=	=	- 1.15	+ 1.28	----	----	- 1.89	+ 1.81
0.00	40.00	0.854	+ 0.79	- 0.52	+ 0.79	- 0.52	----	----
3.00	=	=	- 0.75	+ 1.15	----	----	- 1.54	+ 1.67
0.00	50.00	1.050	+ 0.86	- 0.50	+ 0.86	- 0.50	----	----
3.00	=	=	- 0.60	+ 1.10	----	----	- 1.46	+ 1.60
0.00	60.00	1.246	+ 0.90	- 0.46	+ 0.90	- 0.46	----	----
3.00	=	=	- 0.49	+ 1.07	----	----	- 1.39	+ 1.53
0.00	70.00	1.442	+ 0.90	- 0.42	+ 0.90	- 0.42	----	----
3.00	=	=	- 0.44	+ 1.17	----	----	- 1.34	+ 1.59
0.00	80.00	1.638	+ 0.87	- 0.39	+ 0.87	- 0.39	----	----
3.00	=	=	- 0.42	+ 1.26	----	----	- 1.29	+ 1.65

Table (B4)- Test Series (F)- Half Scale Model.

Slab Load (KN)	Active Wall Prec. (KN)	Lower Slab Rotat. x 10^{-3}	Lower Slab Deflec (mm)	Due to wall Precompression		Due to Floor Load	
				θ	△	θ	△
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.5	0.0	- 2.05	+ 1.56	0.0	0.0	- 2.05	+ 1.56
1.5	10.0	- 1.40	+ 1.28	+ 0.65	- 0.28	- 1.46*	+ 1.29*
1.5	20.0	- 1.05	+ 1.12	+ 1.00	- 0.44	- 1.01	+ 1.06
1.5	30.0	- 0.92	+ 1.01	+ 1.13	- 0.55	- 0.93	+ 0.91
1.5	40.0	- 0.84	+ 0.96	+ 1.21	- 0.60	- 0.84	+ 0.85
1.5	50.0	- 0.82	+ 0.92	+ 1.23	- 0.64	- 0.76	+ 0.81
1.5	60.0	- 0.78	+ 0.90	+ 1.27	- 0.66	- 0.70	+ 0.78
1.5	70.0	- 0.78	+ 0.92	+ 1.27	- 0.64	- 0.64	+ 0.75
1.5	80.0	- 0.84	+ 0.92	+ 1.21	- 0.64	- 0.62	+ 0.73

* From test series (D).

Table (B5)- Test Series (G)- Half Scale Model.

Slab Load (KN)	Active Wall Prec. (KN)	Lower Slab Rotat. x 10^{-3}	lower Slab Deflec. (mm)	Due to wall Precompression		Due to Floor Load	
				θ	△	θ	△
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.0	0.0	- 4.10	+ 3.24	0.0	0.0	- 4.10	+ 3.24
3.0	10.0	- 3.30	+ 2.81	+ 0.80	- 0.43	- 3.07*	+ 2.64*
3.0	20.0	- 2.95	+ 2.62	+ 1.15	- 0.62	- 2.31	+ 2.03
3.0	30.0	- 2.70	+ 2.48	+ 1.40	- 0.76	- 1.89	+ 1.81
3.0	40.0	- 2.60	+ 2.42	+ 1.50	- 0.82	- 1.54	+ 1.67
3.0	50.0	- 2.60	+ 2.41	+ 1.50	- 0.83	- 1.46	+ 1.60
3.0	60.0	- 2.50	+ 2.39	+ 1.60	- 0.85	- 1.39	+ 1.53
3.0	70.0	- 2.50	+ 2.38	+ 1.60	- 0.86	- 1.34	+ 1.59
3.0	80.0	- 2.60	+ 2.39	+ 1.50	- 0.85	- 1.29	+ 1.65

* From test series (E)

Table (B6)- Test Series (K)- Half Scale Model.

No. of Floors Above Joint	Slab Load (KN)	Active Wall Prec. (KN)	Total Wall Prec. (N/mm^2)	Lower Slab Rot. x 10^{-3} (Radians)	Lower Slab Deflection (mm)	Due to Precomp.		Due to F.Load	
						θ	Δ	θ	Δ
---	0.0	0.0	0.0715	0.0	0.0	0.0	0.0	0.0	0.0
1	3.0	6.72	0.1300	- 3.95	+ 3.24	0.0	0.0	- 3.95	+ 3.24
2	=	13.44	0.2630	- 3.41	+ 2.82	+ 0.54	- 0.42	- 3.05	+ 2.67
3	=	20.16	0.3950	- 3.15	+ 2.66	+ 0.80	- 0.58	- 2.61	+ 2.24
4	=	26.88	0.5260	- 2.97	+ 2.57	+ 0.98	- 0.67	- 2.22	+ 2.00
5	=	33.60	0.6580	- 2.75	+ 2.49	+ 1.20	- 0.75	- 1.90	+ 1.81
6	=	40.32	0.7890	- 2.55	+ 2.43	+ 1.40	- 0.81	- 1.65	+ 1.72
7	=	47.04	0.9210	- 2.35	+ 2.41	+ 1.60	- 0.83	- 1.50	+ 1.66
8	=	53.76	1.0530	- 2.40	+ 2.40	+ 1.55	- 0.84	- 1.45	+ 1.61
9	=	60.48	1.1850	- 2.45	+ 2.39	+ 1.50	- 0.85	- 1.42	+ 1.57
10	=	67.20	1.3160	- 2.35	+ 2.39	+ 1.60	- 0.85	- 1.38	+ 1.55
11	=	73.92	1.4480	- 2.35	+ 2.38	+ 1.60	- 0.86	- 1.34	+ 1.55
12	=	80.64	1.5800	- 2.30	+ 2.39	+ 1.65	- 0.85	- 1.30	+ 1.57
13	=	87.36	1.7110	- 2.30	+ 2.39	+ 1.65	- 0.85	- 1.30	+ 1.56
14	=	94.08	1.8430	- 2.30	+ 2.39	+ 1.65	- 0.85	- 1.32	+ 1.56

Table (B7) - Wall Deflections* in (mm) - Test Series (A) - Half Scale Model

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.0	0.00	0.00	0.00	0.00	- .01	- .01	+ .06	- .02	- .01	- .01	0.00	0.00	- .02	- .03	- .05	- .04
1.0	0.0	0.00	0.00	0.00	0.00	- .03	- .05	+ .13	- .05	- .01	- .02	- .01	0.00	- .05	- .07	- .10	- .09
1.5	0.0	0.00	0.00	0.00	0.00	- .08	- .10	+ .22	- .09	- .02	- .03	- .01	0.00	- .08	- .11	- .16	- .14
2.0	0.0	0.00	0.00	+ .02	- .02	- .20	- .19	+ .36	- .14	- .02	- .03	- .01	- .01	- .12	- .16	- .21	- .19
2.5	0.0	0.00	0.00	+ .04	- .03	- .26	- .24	+ .44	- .17	- .02	- .03	- .01	- .01	- .16	- .20	- .27	- .23
3.0	0.0	0.00	0.00	+ .05	- .03	- .30	- .29	+ .52	- .19	- .03	- .04	- .01	- .01	- .19	- .24	- .32	- .28
3.5	0.0	0.00	0.00	+ .06	- .04	- .34	- .33	+ .60	- .22	- .03	- .04	- .01	0.00	- .21	- .29	- .38	- .34
4.0	0.0	0.00	+ .01	+ .07	- .05	- .38	- .38	+ .68	- .24	- .03	- .04	0.00	0.00	- .24	- .33	- .45	- .40

* Positive sign indicates "outward" wall deflection.

Table (B8) - Wall Deflections* in (mm) - Test Series (B) - Half Scale Model

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0.0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	10	- .10	+ .15	+ .28	+ .13	- .04	- .13	+ .21	- .02	+ .02	+ .04	+ .04	- .16	- .25	- .18	- .08	+ .09
0.0	20	- .16	+ .24	+ .44	+ .25	- .01	- .11	+ .13	- .04	+ .01	+ .03	0.00	- .29	- .40	- .29	- .08	+ .14
0.0	30	- .19	+ .28	+ .52	+ .32	+ .03	- .07	0.00	- .06	0.00	+ .01	- .03	- .37	- .47	- .31	- .05	+ .12
0.0	40	- .20	+ .30	+ .56	+ .36	+ .08	- .02	- .11	- .07	- .02	- .01	- .07	- .42	- .52	- .33	- .04	+ .05
0.0	50	- .20	+ .31	+ .60	+ .40	+ .12	+ .03	- .22	- .08	- .04	- .04	- .12	- .47	- .56	- .36	- .05	- .01
0.0	60	- .21	+ .32	+ .63	+ .44	+ .16	+ .07	- .31	- .09	- .06	- .06	-0.16	-0.51	- .60	- .39	- .07	- .07
0.0	70	- .21	+ .33	+ .65	+ .46	+ .19	+ .10	- .36	- .10	- .07	- .09	- .19	- .54	- .63	- .41	- .09	- .11
0.0	80	- .23	+ .35	+ .70	+ .52	+ .25	+ .16	- .47	- .11	- .09	- .12	- .25	- .61	- .69	- .46	- .14	- .20
0.0	90	- .24	+ .37	+ .72	+ .53	+ .25	+ .16	- .47	- .13	- .09	- .12	- .25	- .62	- .71	- .47	- .14	- .20
0.0	100	- .26	+ .41	+ .80	+ .61	+ .32	+ .23	- .60	- .16	- .11	- .16	- .32	- .70	- .79	- .53	- .16	- .23

* Positive sign indicates "outward" wall deflection.

Table (B9) - Wall Deflections* in (mm) - Test Series (C) - Half Scale Model

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.0	0.0	+ .02	- .03	- .03	0.00	+ .05	+ .03	- .05	+ .01	- .02	- .03	- .05	- .01	+ .05	+ .02	0.00	+ .02
1.5	0.0	+ .02	- .04	- .05	0.00	+ .07	+ .05	- .06	+ .02	- .03	- .05	- .07	- .01	+ .07	+ .03	+ .01	+ .03
2.0	0.0	+ .03	- .05	- .07	0.00	+ .09	+ .06	- .07	+ .03	- .04	- .06	- .10	- .02	+ .09	+ .05	+ .01	+ .04
2.5	0.0	+ .03	- .06	- .09	- .01	+ .12	+ .08	- .09	+ .04	- .05	- .08	- .13	- .02	+ .12	+ .06	+ .02	+ .05
3.0	0.0	+ .04	- .07	- .11	- .01	+ .16	+ .11	- .13	+ .06	- .06	- .10	- .16	- .03	+ .15	+ .07	+ .01	+ .06
3.5	0.0	+ .04	- .07	- .11	- .01	+ .16	+ .11	- .13	+ .07	- .06	- .10	- .17	- .03	+ .15	+ .08	+ .01	+ .06
4.0	0.0	+ .05	- .08	- .12	- .01	+ .18	+ .13	- .15	+ .08	- .07	- .11	- .19	- .03	+ .17	+ .08	+ .01	+ .06

* Positive sign indicates "outward" wall deflection.

Table (B10) - Wall Deflections* in (mm) - Test Series (D) - Half Scale Model

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	10	- .09	+ .15	+ .26	+ .08	- .17	- .24	+ .34	- .05	+ .01	+ .02	+ .06	- .11	- .26	- .18	- .06	+ .07
1.5	10	- .05	+ .09	+ .17	+ .07	- .13	- .19	+ .29	- .05	- .03	- .04	- .02	- .10	- .20	- .13	- .04	+ .07
0	20	- .15	+ .23	+ .40	+ .18	- .15	- .24	+ .32	- .05	0.00	+ .02	- .03	- .23	- .39	- .27	- .08	+ .11
1.5	20	- .12	+ .19	+ .35	+ .18	- .10	- .19	+ .25	- .05	- .02	- .03	- .03	- .23	- .36	- .24	- .06	+ .11
0	30	- .19	+ .29	+ .52	+ .27	- .09	- .18	+ .17	- .08	- .01	0.00	- .01	- .33	- .49	- .32	- .06	+ .08
1.5	30	- .16	+ .26	+ .47	+ .27	- .06	- .14	+ .11	- .08	- .03	- .04	- .06	- .33	- .46	- .28	- .04	+ .08
0	40	- .20	+ .31	+ .56	+ .32	- .05	- .14	+ .07	- .10	- .02	- .01	- .03	- .36	- .53	- .33	- .04	+ .03
1.5	40	- .17	+ .28	+ .52	+ .31	- .03	- .11	+ .03	- .10	- .04	- .05	- .08	- .37	- .50	- .30	- .03	+ .02
0	50	- .20	+ .32	+ .59	+ .35	- .02	- .10	- .01	- .11	- .03	- .03	- .07	- .40	- .56	- .34	- .05	- .02
1.5	50	- .18	+ .29	+ .55	+ .35	0.00	- .07	- .05	- .11	- .05	- .07	- .11	- .41	- .54	- .32	- .04	- .03
0	60	- .21	+ .33	+ .62	+ .39	+ .02	- .06	- .09	- .13	- .05	- .05	- .10	- .43	- .59	- .36	- .06	- .08
1.5	60	- .19	+ .30	+ .58	+ .39	+ .04	- .03	- .13	- .13	- .07	- .09	- .14	- .44	- .57	- .34	- .05	- .08
0	70	- .21	+ .34	+ .65	+ .43	+ .05	- .02	- .16	- .14	- .06	- .08	- .13	- .47	- .62	- .39	- .08	- .13
1.5	70	- .20	+ .32	+ .62	+ .42	+ .08	0.00	- .21	- .14	- .08	- .11	- .17	- .48	- .60	- .36	- .07	- .14
0	80	- .22	+ .35	+ .68	+ .46	+ .09	+ .01	- .24	- .14	- .07	- .09	- .17	- .50	- .65	- .41	- .10	- .19
1.5	80	- .21	+ .33	+ .65	+ .46	+ .11	+ .04	- .28	- .14	- .09	- .13	- .21	- .51	- .64	- .39	- .09	- .19

* Positive sign indicates "outward" wall deflection.

Table (B11) - Wall Deflections* in (mm) - Test Series (E) - Half Scale Model

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	10	- .10	+ .15	+ .27	+ .04	- .24	- .30	+ .43	- .05	+ .01	+ .04	+ .09	- .09	- .26	- .18	- .07	+ .07
3	10	- .01	+ .03	+ .11	+ .01	- .15	- .20	+ .31	- .05	- .07	- .08	- .07	- .09	- .17	- .11	- .03	+ .07
0	20	- .17	+ .26	+ .46	+ .17	- .20	- .29	+ .36	- .07	0.00	+ .02	+ .04	- .25	- .44	- .30	- .08	+ .11
3	20	- .01	+ .16	+ .33	+ .14	- .14	- .21	+ .26	- .07	- .06	- .07	- .07	- .24	- .36	- .22	- .03	+ .10
0	30	- .20	+ .30	+ .54	+ .24	- .15	- .24	+ .24	- .10	- .01	+ .01	0.00	- .32	- .51	- .33	- .06	+ .07
3	30	- .14	+ .22	+ .43	+ .22	- .10	- .18	+ .16	- .10	- .06	- .07	- .09	- .32	- .44	- .26	- .02	+ .07
0	40	- .20	+ .31	+ .57	+ .27	- .12	- .21	+ .15	- .12	- .02	- .01	- .02	- .36	- .55	- .34	- .05	+ .01
3	40	- .15	+ .24	+ .48	+ .26	- .07	- .14	+ .06	- .12	- .08	- .08	- .12	- .38	- .49	- .28	- .01	0.00
0	50	- .21	+ .32	+ .60	+ .31	- .08	- .16	+ .06	- .14	- .04	- .03	- .06	- .40	- .59	- .37	- .06	- .04
3	50	- .16	+ .26	+ .53	+ .29	- .04	- .11	0.00	- .14	- .09	- .10	- .15	- .42	- .54	- .31	- .04	- .05
0	60	- .21	+ .33	+ .63	+ .35	- .05	- .12	- .01	- .14	- .06	- .06	- .10	- .44	- .63	- .40	- .09	- .11
3	60	- .18	+ .28	+ .56	+ .33	0.00	- .07	- .08	- .14	- .10	- .12	- .19	- .47	- .59	- .35	- .06	- .12
0	70	- .22	+ .34	+ .65	+ .37	- .02	- .10	- .06	- .16	- .07	- .07	- .12	- .47	- .65	- .42	- .11	- .15
3	70	- .18	+ .28	+ .57	+ .34	0.00	- .06	- .09	- .16	- .11	- .13	- .20	- .48	- .60	- .36	- .08	- .15
0	80	- .23	+ .35	+ .67	+ .39	0.00	- .07	- .11	- .16	- .08	- .09	- .15	- .50	- .69	- .45	- .14	- .19
3	80	- .19	+ .30	+ .60	+ .37	+ .02	- .03	- .14	- .16	- .12	- .14	- .23	- .52	- .64	- .40	- .11	- .19

* Positive sign indicates "outward" wall deflection.

Table (B12) - Wall Deflections* in (mm) - Test Series (F) - Half Scale Model.

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.5	0	+ .01	- .03	- .05	0.00	+ .09	+ .06	- .05	+ .03	- .02	- .04	- .07	- .01	+ .08	+ .05	+ .02	+ .04
1.5	10	+ .10	- .12	- .09	- .05	- .29	- .30	+ .50	- .15	- .15	- .20	- .20	- .02	- .10	- .03	+ .04	+ .11
1.5	20	+ .03	- .02	+ .11	0.00	- .39	- .40	+ .56	- .19	- .14	- .17	- .15	- .13	- .27	- .12	+ .05	+ .15
1.5	30	- .03	+ .07	+ .27	+ .08	- .37	- .37	+ .45	- .22	- .13	- .16	- .14	- .24	- .39	- .17	+ .07	+ .12
1.5	40	- .06	+ .12	+ .36	+ .14	- .34	- .35	+ .38	- .24	- .13	- .15	- .15	- .30	- .46	- .20	+ .08	+ .07
1.5	50	- .08	+ .14	+ .39	+ .17	- .32	- .32	+ .31	- .26	- .14	- .17	- .17	- .34	- .49	- .23	+ .06	+ .02
1.5	60	- .11	+ .19	+ .48	+ .23	- .29	- .29	+ .25	- .28	- .15	- .18	- .20	- .41	- .58	- .28	+ .03	- .06
1.5	70	- .12	+ .20	+ .50	+ .25	- .26	- .27	+ .20	- .29	- .15	- .19	- .23	- .44	- .61	- .31	0.00	- .09
1.5	80	- .14	+ .22	+ .54	+ .29	- .24	- .25	+ .15	- .30	- .16	- .20	- .25	- .47	- .65	- .35	- .02	- .15

* Positive sign indicates "outward" wall deflection.

Table (B13) - Wall Deflections* in (mm) - Test Series (G) - Half Scale Model

Slab Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	0	- 0.3	- .06	- .11	0.00	+ .16	+ .11	- .12	+ .07	- .05	- .09	- .16	- .02	+ .15	+ .09	+ .03	+ .07
3	10	+ .15	- .20	- .21	+ .03	0.00	- .08	+ .27	- .07	- .18	- .27	- .33	- .05	0.00	+ .03	+ .07	+ .17
3	20	+ .15	- .17	- .09	- .02	- .28	- .29	+ .43	- .18	- .23	- .32	- .35	- .13	- .17	- .04	+ .10	+ .17
3	30	+ .07	- .06	+ .09	+ .04	- .30	- .28	+ .34	- .22	- .21	- .28	- .30	- .24	- .30	- .08	+ .13	+ .13
3	40	+ .02	+ .01	+ .22	+ .11	- .28	- .26	+ .26	- .25	- .21	- .27	- .30	- .32	- .39	- .12	+ .13	+ .06
3	50	- .01	+ .06	+ .30	+ .16	- .26	- .24	+ .20	- .27	- .20	- .27	- .31	- .38	- .46	- .17	+ .11	0.00
3	60	- .05	+ .10	+ .37	+ .21	- .23	- .21	+ .13	- .28	- .21	- .27	- .32	- .43	- .52	- .21	+ 0.9	- .05
3	70	- .08	+ .14	+ .44	+ .26	- .20	- .18	+ .07	- .29	- .21	- .28	- .35	- .49	- .59	- .26	+ .05	- .11
3	80	- .10	+ .17	+ .49	+ .30	- .17	- .15	+ .01	- .30	- .22	- .29	- .38	- .53	- .64	- .30	+ .02	- .16

* Positive sign indicates "outward" wall deflection.

Table (B14) - Wall Deflections* in (mm) - Test Series (H) - Half Scale Model

Slabs Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.5	0	+ .05	- .11	- .17	+ .01	+ .16	+ .09	- .01	+ .01	- .05	- .08	- .12	0.00	+ .07	- .01	- .12	- .10
1.5	10	+ .03	- .04	- .02	+ .04	- .04	- .14	+ .37	- .11	- .09	- .11	- .13	- .09	- .13	- .16	- .19	- .08
1.5	20	- .04	+ .09	+ .20	+ .12	- .10	- .26	+ .55	- .12	- .06	- .07	- .08	- .20	- .31	- .32	- .31	- .07
1.5	30	- .09	+ .12	+ .28	+ .17	- .09	- .26	+ .49	- .13	- .06	- .07	- .09	- .28	- .40	- .40	- .34	- .06
1.5	40	- .13	+ .23	+ .47	+ .31	- .02	- .21	+ .37	- .14	- .07	- .09	- .17	- .45	- .60	- .54	- .38	- .08
1.5	50	- .16	+ .27	+ .54	+ .36	0.00	- .16	+ .22	- .15	- .08	- .11	- .20	- .51	- .66	- .56	- .36	- .14
1.5	60	- .18	+ .29	+ .57	+ .39	+ .01	- .16	+ .22	- .15	- .08	- .11	- .20	- .50	- .64	- .55	- .35	- .13
1.5	70	- .20	+ .28	+ .57	+ .39	+ .04	- .12	+ .12	- .16	- .10	- .13	- .23	- .55	- .69	- .57	- .35	- .18
1.5	80	- .21	+ .32	+ .63	+ .45	+ .09	- .06	- .03	- .17	- .11	- .15	- .28	- .61	- .75	- .60	- .34	- .28

* Positive sign indicates "outward" wall deflection.

Table (B15) - Wall Deflections* in (mm) - Test Series (J) - Half Scale Model

Slabs Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0	+ .11	- .13	- .20	+ .01	+ .19	+ .09	+ .01	0.00	- .07	- .12	- .19	- .02	+ .07	- .04	- .18	- .15
3.0	10	+ .12	- .09	- .07	+ .03	0.00	- .12	+ .35	- .10	- .11	- .16	- .21	- .10	- .12	- .18	- .25	- .13
3.0	20	+ .03	+ .03	+ .13	+ .11	- .06	- .24	+ .53	- .13	- .08	- .11	- .14	- .21	- .30	- .34	- .37	- .13
3.0	30	- .03	+ .12	+ .28	+ .18	- .08	- .27	+ .56	- .13	- .07	- .09	- .15	- .31	- .44	- .47	- .45	- .13
3.0	40	- .07	+ .17	+ .37	+ .24	- .05	- .25	+ .46	- .14	- .07	- .10	- .18	- .40	- .53	- .54	- .45	- .13
3.0	50	- .11	+ .22	+ .45	+ .30	- .02	- .21	+ .34	- .14	- .08	- .11	- .21	- .47	- .60	- .57	- .44	- .17
3.0	60	- .13	+ .28	+ .57	+ .43	+ .11	- .05	+ .04	- .15	- .11	- .17	- .32	- .61	- .74	- .67	- .47	- .23
3.0	70	- .15	+ .30	+ .62	+ .47	+ .15	- .01	- .06	- .16	- .13	- .19	- .36	- .66	- .79	- .69	- .48	- .28
3.0	80	- .17	+ .32	+ .65	+ .50	+ .18	+ .02	- .13	- .16	- .13	- .21	- .39	- .70	- .83	- .72	- .48	- .33

* Positive sign indicates "outward" wall deflection.

Table (B16) - Wall Deflections* in (mm) - Test Series (K) - Half Scale Model

Slabs Load (KN)	Active Prec. (KN)	"Wall Deflection" - Displacement of Transducer Number :															
		1	2	3	4	5	6	7	8	11	12	13	14	15	16	17	18
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0	- .08	- .15	- .22	0.00	+ .18	+ .09	+ .05	- .01	- .08	- .13	- .18	0.00	+ .10	- .03	- .17	- .13
3.0	6.72	- .05	- .17	- .19	0.00	+ .04	- .05	+ .27	- .08	- .14	- .20	- .25	- .04	- .02	- .11	- .21	- .12
3.0	13.44	- .10	- .07	- .01	+ .05	- .04	- .16	+ .44	- .11	- .11	- .16	- .19	- .13	- .17	- .24	- .29	- .12
3.0	20.16	- .16	+ .01	+ .11	+ .10	- .08	- .23	+ .53	- .13	- .09	- .12	- .15	- .20	- .28	- .34	- .36	- .12
3.0	26.88	- .21	+ .07	+ .21	+ .14	- .09	- .25	+ .56	- .13	- .08	- .11	- .15	- .26	- .37	- .42	- .41	- .13
3.0	33.60	- .24	+ .12	+ .30	+ .20	- .06	- .23	+ .49	- .13	- .08	- .11	- .17	- .34	- .46	- .49	- .45	- .13
3.0	40.32	- .27	+ .16	+ .37	+ .26	- .03	- .19	+ .39	- .14	- .09	- .12	- .20	- .40	- .53	- .54	- .45	- .13
3.0	47.04	- .29	+ .20	+ .44	+ .31	0.00	- .15	+ .29	- .15	- .10	- .14	- .24	- .47	- .60	- .58	- .45	- .15
3.0	53.76	- .31	+ .23	+ .49	+ .36	+ .04	- .11	+ .21	- .16	- .10	- .15	- .27	- .52	- .65	- .61	- .45	- .17
3.0	60.48	- .34	+ .26	+ .55	+ .40	+ .07	- .07	+ .11	- .16	- .11	- .16	- .30	- .57	- .70	- .64	- .45	- .21
3.0	67.20	- .35	+ .27	+ .57	+ .42	+ .09	- .05	+ .05	- .16	- .12	- .17	- .31	- .59	- .73	- .65	- .45	- .23
3.0	73.92	- .36	+ .29	+ .60	+ .45	+ .12	- .02	- .01	- .17	- .12	- .18	- .33	- .62	- .76	- .67	- .45	- .26
3.0	80.64	- .37	+ .30	+ .63	+ .47	+ .14	0.00	-. 06	- .17	- .13	- .19	- .35	- .65	- .78	- .68	- .45	- .30
3.0	87.36	- .38	+ .32	+ .66	+ .47	+ .16	+ .03	- .12	- .18	- .13	- .20	- .37	- .68	- .81	- .70	- .46	- .33
3.0	94.08	- .39	+ .33	+ .69	+ .48	+ .19	+ .06	- .19	- .18	- .14	- .22	- .39	- .71	- .84	- .72	- .47	- .36

* Positive sign indicates "outward" wall deflection.

Table (B17)- Wall Rotations- Test Series (A), (B) & (C).

Half Scale Model

Test Series (A)		Test Series (B)		Test Series (C)		
Upper Slab Load (KN)	Rotation of Upper Wall $\theta_4 x10^{-3}$	Active Wall Precom. (KN)	Rot. of Upper Wall $\theta_4 x10^{-3}$	Lower Slab Load (KN)	Rot.of Lower Wall $\theta_6 x10^{-3}$	Rot.of Upper Wall $\theta_5 x10^{-3}$
0.0	0.0	10.0	- 0.68	0.0	0.0	0.0
1.0	- 0.28	20.0	- 0.39	1.0	- 0.12	+ 0.28
1.5	- 0.48	030.0	- 0.005	1.5	- 0.185	+ 0.365
2.0	- 0.78	40.0	+ 0.35	2.0	- 0.25	+ 0.445
2.5	- 1.00	50.0	+ 0.72	2.5	- 0.295	+ 0.580
3.0	- 1.20	60.0	+ 0.98	3.0	- 0.46	+ 0.65
3.5	- 1.42	70.0	+ 1.24	3.5	- 0.535	+ 0.72
4.0	- 1.64	80.0	+ 1.55	4.0	- 0.59	+ 0.81

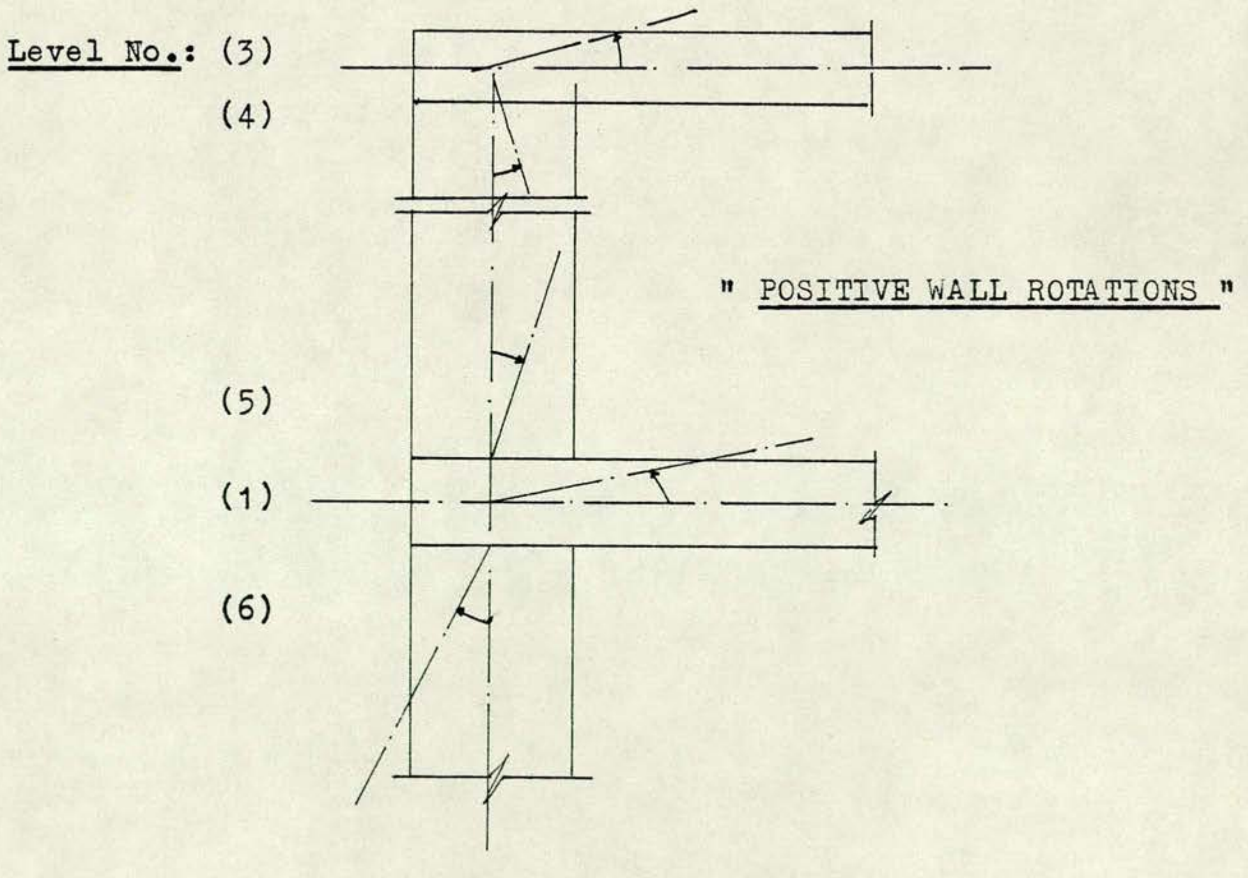

Table (B18)- Wall Rotations- Test Series (D) & (E).

Half Scale Model

Slab Load (KN)	Active Wall Prec. (KN)	Rot.of Upper Wall $\theta_5 x10^{-3}$	Rot.of Lower Wall $\theta_6 x10^{-3}$	Due to Precomp. θ_{upper}	Due to Precomp. θ_{lower}	Due to F.Load θ_{upper}	Due to F.Load θ_{lower}
Test Series (D)							
0.0	10.0	- 0.88	+ 1.20	- 0.88	+ 1.20	----	----
1.5	=	- 0.53	+ 0.72	----	----	+ 0.35	- 0.48
0.0	20.0	- 1.21	+ 1.50	- 1.21	+ 1.50	----	----
1.5	=	- 0.98	+ 1.15	----	----	+ 0.23	- 0.35
0.0	30.0	- 1.42	+ 1.60	- 1.42	+ 1.60	----	----
1.5	=	- 1.20	+ 1.32	----	----	+ 0.22	- 0.28
0.0	40.0	- 1.48	+ 1.60	- 1.48	+ 1.60	----	----
1.5	=	- 1.28	+ 1.39	----	----	+ 0.20	- 0.21
0.0	50.0	- 1.45	+ 1.53	- 1.45	+ 1.53	----	----
1.5	=	- 1.30	+ 1.33	----	----	+ 0.15	- 0.20
0.0	60.0	- 1.46	+ 1.48	- 1.46	+ 1.48	----	----
1.5	=	- 1.30	+ 1.28	----	----	+ 0.16	- 0.20
0.0	70.0	- 1.45	+ 1.40	- 1.45	+ 1.40	----	----
1.5	=	- 1.28	+ 1.24	----	----	+ 0.17	- 0.16
0.0	80.0	- 1.47	+ 1.39	- 1.47	+ 1.39	----	----
1.5	=	- 1.32	+ 1.20	----	----	+ 0.15	- 0.19
Test Series (E)							
0.0	10.0	- 0.85	+ 1.39	- 0.85	+ 1.39	----	----
3.0	=	- 0.29	+ 0.44	----	----	+ 0.56	- 0.95
0.0	20.0	- 1.23	+ 1.75	- 1.23	+ 1.75	----	----
3.0	=	- 0.76	+ 1.12	----	----	+ 0.47	- 0.63
0.0	30.0	- 1.39	+ 1.80	- 1.39	+ 1.80	----	----
3.0	=	- 1.00	+ 1.32	----	----	+ 0.39	- 0.48
0.0	40.0	- 1.42	+ 1.80	- 1.42	+ 1.80	----	----
3.0	=	- 1.08	+ 1.36	----	----	+ 0.34	- 0.44
0.0	50.0	- 1.41	+ 1.75	- 1.41	+ 1.75	----	----
3.0	=	- 1.10	+ 1.36	----	----	+ 0.31	- 0.39
0.0	60.0	- 1.41	+ 1.72	- 1.41	+ 1.72	----	----
3.0	=	- 1.12	+ 1.33	----	----	+ 0.29	- 0.39
0.0	70.0	- 1.39	+ 1.70	- 1.39	+ 1.70	----	----
3.0	=	- 1.10	+ 1.37	----	----	+ 0.29	- 0.33
0.0	80.0	- 1.39	+ 1.68	- 1.39	+ 1.68	----	----
3.0	=	- 1.10	+ 1.33	----	----	+ 0.29	- 0.35

Table (B19)- Wall Rotations- Test Series (F) & (G).

Half Scale Model

Slab Load (KN)	Active Wall Prec. (KN)	Test Series (F)					
		Rot.of Upper Wall $\theta_5 x10^{-3}$	Rot.of Lower Wall $\theta_6 x10^{-3}$	Due to Prec.		Due to F.Load	
				θ_{upper}	θ_{lower}	θ_{upper}	θ_{lower}
1.5	0.0	+ 0.38	- 0.53	---	---	+ 0.38	- 0.53
=	10.0	- 0.90	+ 0.82	- 1.28	+ 1.35	+ 0.35	- 0.48
=	20.0	- 1.23	+ 1.02	- 1.61	+ 1.55	+ 0.23	- 0.35
=	30.0	- 1.44	+ 0.87	- 1.82	+ 1.40	+ 0.22	- 0.28
=	40.0	- 1.50	+ 0.97	- 1.88	+ 1.50	+ 0.20	- 0.21
=	50.0	- 1.47	+ 1.00	- 1.85	+ 1.53	+ 0.15	- 0.20
=	60.0	- 1.48	+ 1.03	- 1.86	+ 1.56	+ 0.16	- 0.20
=	70.0	- 1.47	+ 0.95	- 1.85	+ 1.48	+ 0.17	- 0.16
=	80.0	- 1.47	+ 0.87	- 1.85	+ 1.40	+ 0.15	- 0.19
Test Series (G)							
3.0	0.0	+ 0.70	- 1.10	---	---	+ 0.70	- 1.10
=	10.0	- 0.66	+ 0.12	- 1.36	+ 1.22	+ 0.56	- 0.95
=	20.0	- 0.82	+ 0.36	- 1.52	+ 1.46	+ 0.47	- 0.63
=	30.0	- 1.00	+ 0.25	- 1.70	+ 1.35	+ 0.39	- 0.48
=	40.0	- 1.04	+ 0.25	- 1.74	+ 1.35	+ 0.34	- 0.44
=	50.0	- 1.03	+ 0.28	- 1.73	+ 1.38	+ 0.31	- 0.39
=	60.0	- 1.08	+ 0.30	- 1.78	+ 1.40	+ 0.29	- 0.39
=	70.0	- 1.08	+ 0.34	- 1.78	+ 1.44	+ 0.29	- 0.33
=	80.0	- 1.05	+ 0.40	- 1.75	+ 1.50	+ 0.29	- 0.35

Table (B20)- Wall Rotations - Test Series (H) & (J)- Half Scale Model.

Test Series (H)					Test Series (J)				
Active Prec. (KN)	Slab Loads (KN)	Rotation $\theta_4 x10^{-3}$ (Radians)	Rotation $\theta_5 x10^{-3}$ (Radians)	Rotation $\theta_6 x10^{-3}$ (Radians)	Active Prec. (KN)	Slab Loads (KN)	Rotation $\theta_4 x10^{-3}$ (Radians)	Rotation $\theta_5 x10^{-3}$ (Radians)	Rotation $\theta_6 x10^{-3}$ (Radians)
0.0	1.5	+ 0.075	+ 0.025	- 0.220	0.0	3.0	- 0.02	+ 0.12	- 0.49
10.0	1.5	- 1.81	- 0.23	- 0.025	10.0	3.0	- 1.78	- 0.07	- 0.185
20.0	1.5	- 2.25	- 0.73	+ 0.60	20.0	3.0	- 2.15	- 0.55	+ 0.35
30.0	1.5	- 2.25	- 1.00	+ 0.88	30.0	3.0	- 2.45	- 0.93	+ 0.79
40.0	1.5	- 2.00	- 1.38	+ 1.16	40.0	3.0	- 2.65	- 1.08	+ 0.98
50.0	1.5	- 1.10	- 1.46	+ 1.21	50.0	3.0	- 1.21	- 1.20	+ 1.08
60.0	1.5	- 1.10	- 1.38	+ 1.15	60.0	3.0	- 0.47	- 1.10	+ 0.99
70.0	1.5	- 0.63	- 1.43	+ 1.11	70.0	3.0	+ 0.24	- 1.20	+ 1.04
80.0	1.5	- 0.03	- 1.48	+ 1.11	80.0	3.0	+ 0.48	- 1.18	+ 1.00

Table (B21)- Wall Rotations- Test Series (K).

Half Scale Model

No. of Floors Above Joint	Equivalent Active Wall Prec. (KN)	Slab Loads (KN)	Wall Rotation θ_4 x10^{-3} (Radian)	Wall Rotation θ_5 x10^{-3} (Radian)	Wall Rotation θ_6 x10^{-3} (Radian)
---	0.0	3.0	- 0.08	+ 0.035	- 0.43
1	6.72	3.0	- 1.28	+ 0.035	- 0.29
2	13.44	3.0	- 1.80	- 0.26	- 0.036
3	20.16	3.0	- 2.1	- 0.50	+ 0.16
4	26.88	3.0	- 2.25	- 0.74	+ 0.50
5	33.60	3.0	- 2.65	- 0.91	+ 0.70
6	40.32	3.0	- 2.65	- 1.00	+ 0.80
7	47.04	3.0	- 1.44	- 1.08	+ 0.88
8	53.76	3.0	- 0.72	- 1.12	+ 0.88
9	60.48	3.0	- 0.15	- 1.16	+ 0.90
10	67.20	3.0	+ 0.12	- 1.18	+ 0.94
11	73.92	3.0	+ 0.40	- 1.18	+ 0.92
12	80.64	3.0	+ 0.60	- 1.20	+ 0.96
13	87.36	3.0	+ 0.80	- 1.21	+ 0.95
14	94.08	3.0	+ 1.05	- 1.22	+ 0.95

Table (B 22) - Slab Deflections and Rotations - Test Series (B)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	- .108	+ .67	---	---	---	---
0	20	+ .10	+ .55	+ .208	---	- .12	---
3	20	- .47	+1.13	---	- .57	---	+ .58
0	40	+ .16	+ .52	+ .268	---	- .15	---
3	40	- .40	+1.09	---	- .56	---	+ .57
0	60	+ .30	+ .45	+ .408	---	- .22	---
3	60	- .25	+1.00	---	- .55	---	+ .55
0	80	+ .40	+ .40	+ .508	---	- .27	---
3	80	- .14	+ .94	---	- .54	---	+ .54
0	100	+ .42	+ .39	+ .528	---	- .28	---
3	100	- .10	+ .93	---	- .52	---	+ .54
0	120	+ .48	+ .35	+ .588	---	- .32	---
3	120	- .04	+ .88	---	- .52	---	+ .53
0	140	+ .50	+ .34	+ .608	---	- .33	---
3	140	- .00	+ .86	---	- .50	---	+ .52
0	160	+ .50	+ .33	+ .608	---	- .34	---
3	160	+ .02	+ .86	---	- .48	---	+ .53
0	180	+ .55	+ .32	+ .658	---	- .35	---
3	180	+ .07	+ .85	---	- .48	---	+ .53
0	200	+ .55	+ .32	+ .658	---	- .35	---
3	200	+ .07	+ .85	---	- .48	---	+ .53
0	220	+ .54	+ .32	+ .648	---	- .35	---
3	220	+ .06	+ .85	---	- .48	---	+ .53
0	240	+ .55	+ .31	+ .658	---	- .36	---
3	240	+ .05	+ .84	---	- .50	---	+ .53

Table (B23) - Slab Deflections and Rotations - Test Series (C)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	- .095	+ .63	---	---	---	---
0	20	+ .080	+ .52	+ .175	---	- .11	---
6	20	-1.06	+1.69	---	-1.140	---	+1.17
0	40	+ .177	+ .47	+ .272	---	- .16	---
6	40	- .97	+1.62	---	-1.147	---	+1.15
0	60	+ .255	+ .41	+ .35	---	- .22	---
6	60	- .860	+1.55	---	-1.115	---	+1.14
0	80	+ .320	+ .36	+ .415	---	- .27	---
6	80	- .790	+1.50	---	-1.110	---	+1.14
0	100	+ .345	+ .34	+ .44	---	- .29	---
6	100	- .780	+1.47	---	-1.125	---	+1.13
0	120	+ .358	+ .31	+ .453	---	- .32	---
6	120	- .750	+1.44	---	-1.108	---	+1.13
0	140	+ .377	+ .30	+ .472	---	- .33	---
6	140	- .74	+1.43	---	-1.117	---	+1.13
0	160	+ .390	+ .29	+ .485	---	- .34	---
6	160	- .710	+1.43	---	-1.10	---	+1.14
0	180	+ .404	+ .29	+ .499	---	- .34	---
6	180	- .675	+1.42	---	-1.079	---	+1.13
0	200	+ .428	+ .28	+ .523	---	- .35	---
6	200	- .655	+1.41	---	-1.083	---	+1.13
0	220	+ .435	+ .28	+ .530	---	- .35	---
6	220	- .645	+1.41	---	-1.080	---	+1.13
0	240	+ .455	+ .28	+ .55	---	- .35	---
6	240	- .600	+1.41	---	-1.055	---	+1.13

Table (B24) - Slab Deflections and Rotations - Test Series (D)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	- .087	+ .63	---	---	---	---
0	20	+ .065	+ .51	+ .152	---	- .12	---
9	20	-1.60	+2.27	---	-1.665	---	+1.76
0	40	+ .160	+ .48	+ .247	---	- .15	---
9	40	-1.55	+2.22	---	-1.710	---	+1.74
0	60	+ .225	+ .40	+ .312	---	- .23	---
9	60	-1.45	+2.13	---	-1.675	---	+1.73
0	80	+ .265	+ .36	+ .352	---	- .27	---
9	80	-1.41	+2.07	---	-1.675	---	+1.71
0	100	+ .320	+ .35	+ .407	---	- .28	---
9	100	-1.35	+2.06	---	-1.670	---	+1.71
0	120	+ .345	+ .30	+ .432	---	- .33	---
9	120	-1.35	+2.02	---	-1.695	---	+1.72
0	140	+ .362	+ .29	+ .449	---	- .34	---
9	140	-1.35	+2.00	---	-1.712	---	+1.71
0	160	+ .400	+ .29	+ .487	---	- .34	---
9	160	- 1.30	+1.99	---	-1.700	---	+1.70
0	180	+ .405	+ .28	+ .492	---	- .35	---
9	180	-1.25	+1.96	---	-1.655	---	+1.68
0	200	+ .410	+ .28	+ .497	---	- .35	---
9	200	- 1.25	+ 1.95	---	-1.660	---	+1.67
0	220	+ .415	+ .28	+ .502	---	- .35	---
9	220	-1.20	+1.95	---	-1.615	---	+1.67
0	240	+ .426	+ .27	+ .513	---	- .36	---
9	240	-1.20	+1.95	---	-1.626	---	+1.68

Table (B25) - Slab Deflections and Rotations - Test Series (E)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	- .276	+ .73	---	---	---	---
0	20	+ .085	+ .49	+ .361	---	- .24	---
12	20	-2.20	+2.88	---	-2.285	---	+2.39
0	40	+ .158	+ .45	+ .434	---	- .28	---
12	40	-2.15	+2.79	---	-2.308	---	+2.34
0	60	+ .240	+ .41	+ .516	---	- .32	---
12	60	-2.00	+2.72	---	-2.240	---	+2.31
0	80	+ .290	+ .39	+ .566	---	- .34	---
12	80	-1.90	+2.69	---	-2.190	---	+2.30
0	100	+ .34	+ .36	+ .616	---	- .37	---
12	100	-1.80	+2.65	---	-2.140	---	+2.29
0	120	+ .345	+ .33	+ .621	---	- .40	---
12	120	-1.80	+2.62	---	-2.145	---	+2.29
0	140	+ .350	+ .31	+ .626	---	- .42	---
12	140	-1.85	+2.61	---	-2.200	---	+2.30
0	160	+ .375	+ .30	+ .651	---	- .43	---
12	160	-1.75	+2.57	---	-2.125	---	+2.27
0	180	+ .395	+ .29	+ .671	---	- .44	---
12	180	-1.70	+2.54	---	-2.095	---	+2.25
0	200	+ .400	+ .29	+ .676	---	- .44	---
12	200	-1.70	+2.53	---	-2.100	---	+2.24
0	220	+ .410	+ .29	+ .686	---	- .44	---
12	220	-1.70	+2.52	---	-2.110	---	+2.23
0	240	+ .424	+ .29	+ .70	---	- .44	---
12	240	-1.65	+2.52	---	-2.074	---	+2.23

Table (B26) - Slab Deflections and Rotations - Test Series (F)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	- .22	+ .70	---	---	---	---
0	20	+ .11	+ .49	+ .330	---	- .21	---
15	20	-2.75	+3.48	---	-2.860	---	+2.99
0	40	+ .202	+ .45	+ .422	---	- .25	---
15	40	-2.60	+3.37	---	-2.802	---	+2.92
0	60	+ .250	+ .41	+ .470	---	- .29	---
15	60	-2.55	+3.28	---	-2.800	---	+2.87
0	80	+ .304	+ .39	+ .524	---	- .31	---
15	80	-2.45	+3.22	---	-2.754	---	+2.83
0	100	+ .342	+ .37	+ .562	---	- .33	---
15	100	-2.35	+3.17	---	-2.692	---	+2.80
0	120	+ .370	+ .34	+ .590	---	- .36	---
15	120	-2.30	+3.14	---	-2.670	---	+2.80
0	140	+ .38	+ .32	+ .600	---	- .38	---
15	140	-2.30	+3.11	---	-2.680	---	+2.79
0	160	+ .402	+ .31	+ .622	---	- .39	---
15	160	-2.30	+3.10	---	-2.702	---	+2.79
0	180	+ .420	+ .30	+ .640	---	- .40	---
15	180	-2.25	+3.10	---	-2.670	---	+2.80
0	200	+ .425	+ .30	+ .645	---	- .40	---
15	200	-2.20	+3.10	---	-2.625	---	+2.80
0	220	+ .435	+ .30	+ .655	---	- .40	---
15	220	-2.20	+3.09	---	-2.635	---	+2.79
0	240	+ .453	+ .29	+ .673	---	- .41	---
15	240	-2.20	+3.07	---	-2.653	---	+2.78

Table (B27) - Slab Deflections and Rotations - Test Series (G)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp. θp	F.Load θF	Precomp. Δp	F.Load ΔF
0	0	+ .152	+ .57	---	---	---	---
3	0	- .455	+1.19	---	- .607	---	+ .62
3	20	- .272	+1.06	+ .183	- .570	- .13	+ .58
3	40	- .056	+ .95	+ .399	- .56	- .24	+ .57
3	60	+ .007	+ .91	+ .462	- .55	- .28	+ .55
3	80	+ .035	+ .89	+ .490	- .54	- .30	+ .54
3	100	+ .078	+ .87	+ .533	- .52	- .32	+ .54
3	120	+ .108	+ .85	+ .563	- .52	- .34	+ .53
3	140	+ .125	+ .83	+ .580	- .50	- .36	+ .52
3	160	+ .140	+ .82	+ .595	- .48	- .37	+ .53
3	180	+ .153	+ .82	+ .608	- .48	- .37	+ .53
3	200	+ .170	+ .81	+ .625	- .48	- .38	+ .53
3	220	+ .175	+ .81	+ .630	- .48	- .38	+ .53
3	240	+ .182	+ .80	+ .637	- .50	- .39	+ .53

Table (B28) - Slab Deflections and Rotations - Test Series (H)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	+ 51	+ .59	---	---	---	---
6	0	- .645	+1.83	---	-1.155	---	+1.24
6	20	- .48	+1.71	+ .165	-1.140	- .12	+1.17
6	40	- .245	+1.58	+ .40	-1.147	- .25	+1.15
6	60	- .175	+1.56	+ .470	-1.115	- .27	+1.14
6	80	- .100	+1.51	+ .545	-1.110	- .32	+1.14
6	100	- .065	+1.49	+ .580	-1.125	- .34	+1.13
6	120	- .038	+1.47	+ .607	-1.108	- .36	+1.13
6	140	- .012	+1.45	+ .633	-1.117	- .38	+1.13
6	160	+ .008	+1.43	+ .653	-1.10	- .40	+1.14
6	180	+ .023	+1.43	+ .668	-1.079	- .40	+1.13
6	200	+ .030	+1.42	+ .675	-1.083	- .41	+1.13
6	220	+ .050	+1.42	+ .695	-1.080	- .41	+1.13
6	240	+ .060	+1.40	+ .705	-1.055	- .43	+1.13

Table (B29) - Slab Deflections and Rotations - Test Series (J)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp. θp	F.Load θF	Precomp. Δp	F.Load ΔF
0	0	+ .86	+ .52	---	---	---	---
9	0	- .91	+2.34	---	-1.770	---	+1.82
9	20	- .59	+2.12	+ .32	-1.665	- .22	+1.76
9	40	- .462	+2.06	+ .448	-1.710	- .28	+1.74
9	60	- .37	+2.02	+ .540	-1.675	- .32	+1.73
9	80	- .265	+1.95	+ .645	-1.675	- .39	+1.71
9	100	- .232	+1.93	+ .678	-1.670	- .41	+1.71
9	120	- .182	+1.91	+ .728	-1.695	- .43	+1.72
9	140	- .165	+1.90	+ .745	-1.712	- .44	+1.71
9	160	- .160	+1.89	+ .750	-1.700	- .45	+1.70
9	180	- .130	+1.86	+ .780	-1.655	- .48	+1.68
9	200	- .108	+1.85	+ .802	-1.660	- .49	+1.67
9	220	- .10	+1.85	+ .810	-1.615	- .49	+1.67
9	240	- .09	+1.85	+ .82	-1.626	- .49	+1.68

Table (B30) - Slab Deflections and Rotations - Test Series (K)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp.	F.Load	Precomp.	F.Load
0	0	+1.15	+ .43	---	---	---	---
12	0	-1.20	+2.89	---	-2.35	---	+2.46
12	20	- .86	+2.66	+ .340	-2.285	- .23	+2.39
12	40	- .66	+2.54	+ .540	-2.308	- .35	+2.34
12	60	- .55	+2.51	+ .650	-2.240	- .38	+2.31
12	80	- .45	+2.47	+ .750	-2.190	- .42	+2.30
12	100	- .415	+2.42	+ .785	-2.140	- .47	+2.29
12	120	- .350	+2.39	+ .850	-2.145	- .50	+2.29
12	140	- .325	+2.38	+ .875	-2.200	- .51	+2.30
12	160	- .260	+2.36	+ .940	-2.125	- .53	+2.27
12	180	- .250	+2.34	+ .950	-2.095	- .55	+2.25
12	200	- .235	+2.33	+ .965	-2.100	- .56	+2.24
12	220	- .225	+2.32	+ .975	-2.110	- .57	+2.23
12	240	- .220	+2.32	+ .980	-2.074	- .57	+2.23

Table (B31) - Slab Deflections and Rotations - Test Series (L)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	Slab Rot. $*10^{-3}$ (Radians)	Slab Deflection (mm)	Slab Rotation due to:-		Slab Deflection due to:	
				Precomp. θp	F.Load θF	Precomp. Δp	F.Load ΔF
0	0	+1.55	+ .27	---	---	---	---
15	0	-1.51	+3.42	---	-3.06	---	+3.15
15	20	-1.05	+3.13	+ .460	-2.86	- .29	+2.99
15	40	- .925	+3.05	+ .585	-2.802	- .37	+2.92
15	60	- .770	+2.96	+ .740	-2.800	- .46	+2.87
15	80	- .645	+2.89	+ .865	-2.754	- .53	+2.83
15	100	- .620	+2.88	+ .890	-2.692	- .54	+2.80
15	120	- .560	+2.85	+ .950	-2.670	- .57	+2.80
15	140	- .500	+2.82	+1.01	-2.680	- .60	+2.79
15	160	- .477	+2.81	+1.033	-2.702	- .61	+2.79
15	180	- .455	+2.80	+1.055	-2.670	- .62	+2.80
15	200	- .425	+2.80	+1.085	-2.625	- .62	+2.80
15	220	- .400	+2.79	+1.110	-2.635	- .63	+2.79
15	240	- .365	+2.77	+1.145	-2.653	- .65	+2.78

Table (B32) - Slab Deflections and Rotations - Test Series (M N, P)

Full Size Structure

Slab load (KN)	Active Precomp (KN)	Slab Rotation $*10^{-3}$	Slab Deflection (mm)	Slab Rotation due to:		Slab Deflection due to:	
				Prec.	F.Load	Prec.	F.Load
TEST SERIES (M)							
0.0	0.0	- .130	+ .55	0.0	0.0	---	---
0.0	100.0	+ .295	+ .32	+ .425	0.0	- .23	---
3.0	100.0	- .225	+ .85	+ .528	- .520	- .28	+ .53
6.0	100.0	- .800	+1.47	+ .440	-1.095	- .29	+1.15
9.0	100.0	-1.350	+2.02	+ .407	-1.645	- .28	+1.70
12.0	100.0	-1.850	+2.59	+ .616	-2.145	- .37	+2.27
15.0	100.0	-2.500	+3.06	+ .562	-2.795	- .33	+2.74
TEST SERIES (N)							
0.0	0.0	- .125	+ .55	---	---	---	---
0.0	200.0	+ .352	+ .24	+ .477	---	- .31	---
3.0	200.0	- .145	+ .77	+ .658	- .497	- .35	+ .53
6.0	200.0	- .650	+1.38	+ .523	-1.002	- .35	+1.14
9.0	200.0	-1.300	+1.94	+ .497	-1.652	- .35	+1.70
12.0	200.0	-1.750	+2.49	+ .676	-2.102	- .44	+2.25
15.0	200.0	-2.250	+2.95	+ .645	-2.602	- .40	+2.71
TEST SERIES (P)							
0.0	0.0	- .120	+ .55	---	---	---	---
0.0	300.0	+ .425	+ .21	+ .545	---	- .34	---
3.0	300.0	- .030	+ .71	+ .558	- .445	- .36	+ .50
6.0	300.0	- .490	+1.31	+ .585	- .915	- .36	+1.10
9.0	300.0	-1.00	+1.88	+ .625	-1.425	- .37	+1.67
12.0	300.0	-1.55	+2.24	+ .698	-1.975	- .46	+2.03
15.0	300.0	-2.05	+2.88	+ .735	-2.475	- .44	+2.67

Table (B33) - Slab Deflections and Rotations - Test Series (R)

Full Size Structure

Active Wall Prec. (KN)	Slab Load (KN)	Slab Def. (mm)	Slab Rotation * 10^{-3} (Radians)	Due to F.Load (net)	
				Deflection (Δ)	Rotation (θ)
0	0	+ .53	- .243	0	0
600	0	+ .05	+ .480	- .48	+ .723
600	3	+ .47	+ .120	+ .42	- .360
600	6	+ .96	- .31	+ .91	- .790
600	9	+ 1.47	- .80	+1.42	-1.28
600	12	+ 1.94	-1.45	+1.89	-1.93
600	15	+ 2.46	-1.80	+2.41	-2.28
600	18	+ 2.95	-2.30	+2.90	-2.78
600	21	+ 3.35	-2.70	+3.30	-3.18
600	24	+ 3.82	-3.15	+3.77	-3.63
600	25	+ 3.91	-3.25	+3.86	-3.73
600	27.5	+ 4.26	-3.60	+4.21	-4.08
600	30.0	+ 4.73	-3.90	+4.68	-4.38
600	32.5	+ 5.15	-4.50	+5.10	-4.98
600	35.0	+ 7.47	-6.90	+7.42	-7.38
600	37.5	+11.04	>10.0	+10.99	---
	> 37.5	*	*		

* Continuous increase "yielding" in deformations

Table (B 34) - Wall Deflections* in (mm) - Test Series (A) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	1.0	- .01	- .03	- .04	- .05	- .02	+ .01	+ .01	+ .01	0.00	0.00
0.0	2.0	- .02	- .06	- .09	- .10	- .03	+ .02	+ .02	+ .02	0.00	0.00
0.0	3.0	- .03	- .09	- .13	- .14	- .05	+ .03	+ .03	+ .02	0.00	0.00
0.0	4.0	- .04	- .10	- .16	- .18	- .06	+ .04	+ .04	+ .02	0.00	0.00
0.0	5.0	- .04	- .13	- .20	- .22	- .08	+ .04	+ .04	+ .03	0.00	0.00
0.0	6.0	- .05	- .15	- .23	- .26	- .10	+ .04	+ .04	+ .03	- .01	- .01
0.0	7.0	- .06	- .17	- .27	- .30	- .11	+ .05	+ .04	+ .03	- .01	- .01
0.0	8.0	- .06	- .20	- .30	- .34	- .13	+ .05	+ .05	+ .03	- .01	- .01
0.0	9.0	- .07	- .22	- .33	- .37	- .15	+ .06	+ .05	+ .04	- .01	- .01
0.0	10.0	- .08	- .24	- .36	- .41	- .17	+ .06	+ .05	+ .04	- .02	- .03
0.0	11.0	- .08	- .26	- .39	- .44	- .18	+ .07	+ .06	+ .04	- .02	- .03
0.0	12.0	- .09	- .28	- .42	- .48	- .19	+ .07	+ .06	+ .04	- .02	- .03
0.0	13.0	- .09	- .29	- .45	- .51	- .21	+ .08	+ .07	+ .04	- .02	- .03
0.0	14.0	- .10	- .31	- .48	- .54	- .22	+ .08	+ .07	+ .04	- .02	- .03
0.0	15.0	- .11	- .33	- .51	- .57	- .24	+ .09	+ .07	+ .04	- .03	- .04

* 'Negative' sign indicates outward wall deflection.

Table (B 35) - Wall Deflections* in (mm) - Test Series (B) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20.0	0.0	+ .01	+ .03	+ .10	+ .08	0.00	+ .06	+ .11	+ :12	+ .08	0.00
20.0	3.0	- .01	- .02	- .01	- .03	- .03	+ .09	+ .14	+ .14	+ .08	- .01
40.0	0.0	+ .02	+ .08	+ .14	+ .12	- .01	+ .08	+ .15	+ .16	+ .11	- .01
40.0	3.0	+ .01	+ .03	+ .06	+ .03	- .03	+ .12	+ .19	+ .19	+ .12	- .01
60.0	0.0	+ .02	+ .10	+ .19	+ .15	- .01	+ .09	+ .18	+ .19	+ .13	- .01
60.0	3.0	+ .01	+ .07	+ .10	+ .06	- .03	+ .14	+ .23	+ .23	+ .14	- .01
80.0	0.0	+ .03	+ .12	+ .22	+ .17	- .02	+ .10	+ .20	+ .21	+ .14	- .01
80.0	3.0	+ .01	+ .07	+ .13	+ .08	- .04	+ .15	+ .25	+ .26	+ .16	- .01
100.0	0.0	+ .03	+ .14	+ .25	+ .20	- .02	+ .11	+ .21	+ .23	+ .16	- .01
100.0	3.0	+ .01	+ .09	+ .16	+ .11	- .04	+ .16	+ .27	+ .27	+ .16	- .02
120.0	0.0	+ .03	+ .15	+ .27	+ .21	- .03	+ .11	+ .22	+ .24	+ .16	- .02
120.0	3.0	+ .01	+ .09	+ .16	+ .11	- .04	+ .16	+ .27	+ .27	+ .16	- .04
140.0	0.0	+ .03	+ .16	+ .29	+ .23	- .03	+ .12	+ .23	+ .24	+ .15	- .04
140.0	3.0	+ .02	+ .11	+ .20	+ .13	- .04	+ .18	+ .29	+ .28	+ .15	- .06
160.0	0.0	+ .04	+ .18	+ .32	+ .24	- .03	+ .12	+ .23	+ .24	+ .15	- .06
160.0	3.0	+ .02	+ .12	+ .21	+ .15	- .04	+ .19	+ .30	+ .29	+ .15	- .08
180.0	0.0	+ .04	+ .19	+ .33	+ .26	- .04	+ .12	+ .24	+ .24	+ .24	+ .08
180.0	3.0	+ .02	+ .12	+ .22	+ .15	- .04	+ .19	+ .30	+ .28	+ .15	- .10
200.0	0.0	+ .04	+ .20	+ .35	+ .27	- .04	+ .12	+ .23	+ .24	+ .14	- .10
200.0	3.0	+ .02	+ .13	+ .24	+ .16	- .04	+ .20	+ .32	+ .30	+ .14	- .11
220.0	0.0	+ .04	+ .21	+ .36	+ .28	- .04	+ .13	+ .24	+ .24	+ .14	- .11
220.0	3.0	+ .02	+ .14	+ .25	+ .16	- .05	+ .20	+ .32	+ .29	+ .13	- .13
240.0	0.0	+ .04	+ .21	+ .37	+ .29	- .04	+ .13	+ .25	+ .24	+ .13	- .13
240.0	3.0	+ .02	+ .15	+ .27	+ .18	- .04	+ .21	+ .33	+ .30	+ .13	- .13

* "Negative" sign indicates outward wall deflection.

Table (B 36) - Wall Deflections* in (mm) - Test Series (C) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20.0	0.0	+ .01	+ .05	+ .10	+ .09	0.00	+ .06	+ .12	+ .13	+ .09	0.00
20.0	6.0	- .02	- .07	- .10	- .13	- .07	+ .12	+ .19	+ .19	+ .12	0.00
40.0	0.0	+ .02	+ .08	+ .14	+ .12	- .01	+ .08	+ .16	+ .18	+ .14	+ .02
40.0	6.0	- .02	- .05	- .05	- .15	- .06	+ .15	+ .23	+ .23	+ .15	+ .02
60.0	0.0	+ .03	+ .11	+ .21	+ .17	0.00	+ .12	+ .21	+ .23	+ .18	+ .02
60.0	6.0	- .01	0.00	+ .02	- .03	- .06	+ .19	+ .30	+ .30	+ .19	+ .02
80.0	0.0	+ .03	+ .13	+ .23	+ .18	- .01	+ .12	+ .22	+ .24	+ .19	+ .02
80.0	6.0	0.00	0.00	+ .03	- .02	- .06	+ .19	+ .30	+ .30	+ .19	+ .02
100.0	0.0	+ .03	+ .15	+ .27	+ .22	0.00	+ .14	+ .25	+ .27	+ .19	+ .02
100.0	6.0	0.00	+ .04	+ .08	+ .02	- .05	+ .22	+ .35	+ .35	+ .23	+ .02
120.0	0.0	+ .04	+ .17	+ .29	+ .24	0.00	+ .15	+ .26	+ .28	+ .21	+ .02
120.0	6.0	+ .01	+ .05	+ .09	+ .04	- .05	+ .23	+ .36	+ .36	+ .23	+ .02
140.0	0.0	+ .04	+ .18	+ .32	+ .26	0.00	+ .16	+ .28	+ .30	+ .21	+ .02
140.0	6.0	+0.01	+ .05	+ .11	+ .05	- .05	+ .25	+ .38	+ .38	+ .24	+ .02
160.0	0.0	+ .04	+ .19	+ .34	+ .27	0.00	+ .17	+ .29	+ .31	+ .22	+ .02
160.0	6.0	+ .01	+ .08	+ .14	+ .08	- .04	+ .27	+ .41	+ .40	+ .25	+ .02
180.0	0.0	+ .05	+ .20	+ .36	+ .29	0.00	+ .18	+ .30	+ .32	+ .23	+ .02
180.0	6.0	+ .01	+ .08	+ .16	+ .09	- .04	+ .28	+ .43	+ .41	+ .26	+ .02
200.0	0.0	+ .05	+ .22	+ .38	+ .31	0.00	+ .18	+ .31	+ .32	+ .24	+ .02
200.0	6.0	+ .01	+ .08	+ .17	+ .10	- .04	+ .30	+ .44	+ .43	+ .26	+ .02
220.0	0.0	+ .05	+ .23	+ .40	+ .32	+ .01	+ .19	+ .32	+ .33	+ .24	+ .02
220.0	6.0	+ .01	+ .09	+ .19	+ .11	- .03	+ .31	+ .46	+ .44	+ .27	+ .02
240.0	0.0	+ .05	+ .24	+ .41	+ .34	+ .01	+ .20	+ .32	+ .34	+ .24	+ .02
240.0	6.0	+ .01	+ .10	+ .20	+ .13	- .03	+ .33	+ .48	+ .46	+ .28	+ .02

* "Negative" sign indicates outward wall deflection.

Table (B 37) - Wall Deflections* in (mm) - Test Series (D) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20.0	0.0	+ .02	+ .05	+ .12	+ .09	- .01	+ .03	+ .07	+ .08	+ .07	0.00
20.0	9.0	- .04	- .17	- .22	- .27	- .11	+ .12	+ .20	+ .18	+ .09	0.00
40.0	0.0	+ .02	+ .06	+ .15	+ .11	- .02	+ .06	+ .14	+ .16	+ .10	0.00
40.0	9.0	- .03	- .12	- .15	- .21	- .12	+ .14	+ .25	+ .24	+ .13	- .01
60.0	0.0	+ .03	+ .08	+ .18	+ .14	- .02	+ .07	+ .17	+ .18	+ .12	- .01
60.0	9.0	- .02	- .08	- .10	- .16	- .11	+ .16	+ .29	+ .28	+ .15	- .02
80.0	0.0	+ .03	+ .10	+ .21	+ .16	- .03	+ .07	+ .18	+ .19	+ .13	- .02
80.0	9.0	- .02	- .06	- .06	- .14	- .12	+ .16	+ .31	+ .29	+ .14	- .05
100.0	0.0	+ .04	+ .12	+ .25	+ .19	- .03	+ .07	+ .18	+ .19	+ .12	- .05
100.0	9.0	- .02	- .05	- .04	- .12	- .12	+ .17	+ .32	+ .28	+ .12	- .10
120.0	0.0	+ .04	+ .14	+ .27	+ .21	- .03	+ .07	+ .18	+ .18	+ .10	- .10
120.0	9.0	- .01	- .03	- .01	- .09	- .12	+ .17	+ .32	+ .28	+ .10	- .13
140.0	0.0	+ .04	+ .16	+ .30	+ .23	- .04	+ .06	+ .17	+ .16	+ .07	- .13
140.0	9.0	- .01	- .02	0.00	- .08	- .12	+ .17	+ .32	+ .27	+ .09	- .15
160.0	0.0	+ .05	+ .17	+ .32	+ .24	- .04	+ .06	+ .18	+ .16	+ .06	- .15
160.0	9.0	- .01	0.00	+ .03	- .06	- .12	+ .19	+ .34	+ .28	+ .08	- .18
180.0	0.0	+ .05	+ .18	+ .34	+ .26	- .04	+ .06	+ .18	+ .15	+ .05	- .18
180.0	9.0	- .01	0.00	+ .04	- .05	- .12	+ .19	+ .35	+ .28	+ .07	- .19
200.0	0.0	+ .05	+ .19	+ .35	+ .27	- .04	+ .06	+ .18	+ .15	+ .05	- .20
200.0	9.0	- .01	+ .01	+ .06	- .04	- .12	+ .20	+ .36	+ .28	+ .07	- .22
220.0	0.0	+ .05	+ .20	+ .37	+ .28	- .04	+ .06	+ .18	+ .14	+ .04	- .22
220.0	9.0	- .01	+0.01	+ .04	- .05	- .13	+ .20	+ .35	+ .26	+ .05	- .24
240.0	0.0	+ .05	+0.20	+ .38	+ .28	- .06	+ .03	+ .17	+ .12	+ .02	- .25
240.0	9.0	- .01	+ .02	+ .07	- .04	- .14	+ .20	+ .36	+ .23	+ .01	- .29

* "Negative" sign indicates outward wall deflection.

Table (B 38) - Wall Deflections* in (mm) - Test Series (E) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20.0	0.0	+ .01	+ .03	+ .05	+ .04	0.00	+ .04	+ .06	+ .06	+ .05	0.00
20.0	12.0	- .07	- .24	- .36	- .42	- .15	+ .14	+ .16	+ .13	+ .06	- .01
40.0	0.0	+ .02	+ .06	+ .11	+ .07	- .03	+ .06	+ .12	+ .13	+ .09	- .01
40.0	12.0	- .06	- .20	- .28	- .35	- .16	+ .17	+ .25	+ .22	+ .12	- .01
60.0	0.0	+ .02	+ .09	+ .19	+ .13	- .04	+ .08	+ .17	+ .19	+ .12	- .01
60.0	12.0	- .05	- .15	- .20	- .28	- .16	+ .21	+ .34	+ .32	+ .18	- .01
80.0	0.0	+ .03	+ .11	+ .22	+ .16	- .04	+ .08	+ .19	+ .21	+ .15	- .01
80.0	12.0	- .05	- .15	- .19	- .28	- .16	+ .21	+ .34	+ .32	+ .18	- .01
100.0	0.0	+ .03	+ .12	+ .25	+ .18	- .04	+ .09	+ .21	+ .22	+ .15	- .02
100.0	12.0	- .05	- .13	- .15	- .24	- .16	+ .22	+ .37	+ .34	+ .19	- .02
120.0	0.0	+ .03	+ .14	+ .27	+ .20	- .04	+ .10	+ .22	+ .23	+ .16	- .02
120.0	12.0	- .04	- .09	- .10	- .20	- .15	+ .25	+ .42	+ .38	+ .21	- .02
140.0	0.0	+ .04	+ .15	+ .30	+ .22	- .03	+ .12	+ .24	+ .24	+ .18	- .02
140.0	12.0	- .04	- .08	- .08	- .19	- .15	+ .27	+ .44	+ .40	+ .22	- .02
160.0	0.0	+ .04	+ .17	+ .32	+ .24	- .03	+ .13	+ .24	+ .24	+ .18	- .03
160.0	12.0	- .03	- .07	- .06	- .17	- .14	+ .28	+ .46	+ .41	+ .23	- .03
180.0	0.0	+ .04	+ .18	+ .34	+ .25	- .02	+ .13	+ .26	+ .25	+ .19	- .03
180.0	12.0	- .03	- .05	- .04	- .15	- .13	+ .31	+ .49	+ .44	+ .25	- .03
200.0	0.0	+ .05	+ .19	+ .36	+ .27	- .02	+ .15	+ .28	+ .27	+ .21	- .03
200.0	12.0	- 0.3	- .05	- .02	- .13	- .12	+ .33	+ .51	+ .45	+ .26	- .03
220.0	0.0	+ .05	+ .20	+ .38	+ .29	- .01	+ .16	+ .29	+ .29	+ .22	- .03
220.0	12.0	- .03	- .04	- .01	- .12	- .11	+ .34	+ .51	+ .46	+ .26	- .03
240.0	0.0	+ .05	+ .21	+ .40	+ .31	0.00	+ .17	+ .30	+ .29	+ .22	- .03
240.0	12.0	- .02	- .03	+ .01	- .10	- .10	+ .35	+ .53	+ .46	+ .26	- .03

* "Negative" sign indicates outward wall deflection.

Table (B39) - Wall Deflections* in (mm) - Test Series (F) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
20.0	0.0	+ .01	+ .04	+ .10	+ .08	0.00	+ .06	+ .11	+ .12	+ .09	0.00
20.0	15.0	- .08	- .26	- .37	- .45	- .16	+ .21	+ .29	+ .25	+ .14	0.00
40.0	0.0	+ .02	+ .07	+ .15	+ .12	- .01	+ .08	+ .15	+ .17	+ .12	0.00
40.0	15.0	- .08	- .23	- .32	- .40	- .16	+ .24	+ .36	+ .33	+ .18	0.00
60.0	0.0	+ .02	+ .09	+ .20	+ .15	- .01	+ .09	+ .18	+ .20	+ .15	0.00
60.0	15.0	- .07	- .21	- .28	- .36	- .16	+ .26	+ .40	+ .37	+ .21	0.00
80.0	0.0	+ .03	+ .11	+ .24	+ .18	- .01	+ .10	+ .20	+ .22	+ .17	0.00
80.0	15.0	- .06	- .18	- .24	- .33	- .17	+ .27	+ .42	+ .39	+ .22	0.00
100.0	0.0	+ .03	+ .13	+ .27	+ .21	- .01	+ .11	+ .21	+ .23	+ .17	0.00
100.0	15.0	- .06	- .16	- .21	- .30	- .17	+ .28	+ .44	+ .40	+ .22	- .01
120.00	0.0	+ .03	+ .14	+ .29	+ .22	- .02	+ .12	+ .22	+ .23	+ .16	- .01
120.00	15.0	- .06	- .14	- .17	- .28	- .17	+ .28	+ .45	+ .40	+ .21	- .03
140.00	0.0	+ .04	+ .16	+ .31	+ .24	- .03	+ .11	+ .22	+ .22	+ .16	- .03
140.00	15.0	- .06	- .13	- .15	- .26	- .17	+ .29	+ .45	+ .40	+ .20	- .06
160.00	0.0	+ .04	+ .17	+ .33	+ .25	- .03	+ .11	+ .22	+ .22	+ .15	- .06
160.00	15.0	- .06	- .13	- .15	- .26	- .17	+ .28	+ .45	+ .39	+ .18	- .08
180.00	0.0	+ .04	+ .18	+ .34	+ .26	- .03	+ .11	+ .21	+ .20	+ .13	- .08
180.00	15.0	- .06	- .12	- .13	- .25	- .17	+ .29	+ .46	+ .39	+ .19	- .09
200.00	0.0	+ .04	+ .19	+ .36	+ .27	- .04	+ .11	+ .22	+ .20	+ .13	- .09
200.00	15.0	- .05	- .10	- .10	- .23	- .17	+ .30	+ .47	+ .41	+ .20	- .10
220.00	0.0	+ .04	+ .19	+ .37	+ .28	- .04	+ .11	+ .22	+ .20	+ .13	- .10
220.00	15.0	- .05	- .09	- .09	- .22	- .18	+ .31	+ .48	+ .41	+ .18	- .11
240.00	0.0	+ .04	+ .20	+ .38	+ .28	- .04	+ .10	+ .21	+ .19	+ .12	- .11
240.00	15.0	- .06	- .10	- .09	- .22	- .19	+ .30	+ .48	+ .39	+ .16	- .15

* "Negative" sign indicates outward wall deflection.

Table (B40) - Wall Deflections* in (mm) - Test Series (G) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	3.0	- .03	- .08	- .13	- .14	- .04	+ .04	+ .04	+ .03	+ .02	+ .01
20.0	3.0	- .02	- .04	- .07	- .09	- .06	+ .07	+ .10	+ .10	+ .08	+ .01
40.0	3.0	- .01	+ .01	+ .03	- .01	- .06	+ .12	+ .21	+ .22	+ .15	+ .01
60.0	3.0	0.00	+ .04	+ .07	+ .02	- .07	+ .13	+ .24	+ .25	+ .17	+ .01
80.0	3.0	0.00	+ .04	+ .08	+ .02	- .07	+ .13	+ .24	+ .25	+ .18	+ .01
100.0	3.0	0.00	+ .07	+ .12	+ .05	- .08	+ .14	+ .27	+ .27	+ .18	0.00
120.00	3.0	+ .01	+ .08	+ .14	+ .07	- .08	+ .15	+ .28	+ .28	+ .18	- .02
140.00	3.0	+ .01	+ .09	+ .15	+ .08	- .09	+ .16	+ .29	+ .29	+ .18	- .03
160.00	3.0	+ .01	+ .10	+ .17	+ .09	- .09	+ .16	+ .30	+ .29	+ .18	- .04
180.00	3.0	+ .01	+ .10	+ .17	+ .09	- .10	+ .16	+ .30	+ .28	+ .18	- .06
200.00	3.0	+ .01	+ .11	+ .18	+ .10	- .10	+ .15	+ .29	+ .27	+ .16	- .09
220.00	3.0	+ .01	+ .11	+ .19	+ .10	- .11	+ .17	+ .30	+ .27	+ .16	- .10
240.00	3.0	+ .01	+ .12	+ .20	+ .11	- .12	+ .17	+ .30	+ .27	+ .16	- .11

* "Negative" sign indicates outward wall deflection.

Table (B41) - Wall Deflections* in (mm) - Test Series (H) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	6.0	- .05	- .14	- .22	- .24	- .08	+ .06	+ .06	+ .06	+ .03	0.00
20.0	6.0	- .04	- .11	- .17	- .20	- .09	+ .09	+ .13	+ .13	+ .08	0.00
40.0	6.0	- .03	- .05	- .07	- .11	- .09	+ .15	+ .25	+ .27	+ .17	0.00
60.0	6.0	- .02	- .03	- .02	- .07	- .09	+ .17	+ .29	+ .32	+ .20	0.00
80.0	6.0	- .02	- .02	- .02	- .07	- .09	+ .19	+ .32	+ .35	+ .22	0.00
100.0	6.0	- .02	+ .01	+ .03	- .03	- .09	+ .20	+ .34	+ .37	+ .23	0.00
120.0	6.0	- .01	+ .02	+ .06	- .01	- .09	+ .21	+ .35	+ .38	+ .23	0.00
140.0	6.0	- .01	+ .03	+ .07	- .01	- .09	+ .21	+ .36	+ .38	+ .23	- .01
160.0	6.0	- .01	+ .04	+ .10	+ .02	- .09	+ .23	+ .38	+ .39	+ .23	- .02
180.0	6.0	- .01	+ .06	+ .11	+ .03	- .09	+ .23	+ .39	+ .40	+ .24	- .03
200.0	6.0	- .01	+ .06	+ .12	+ .04	- .09	+ .24	+ .39	+ .40	+ .24	- .04
220.0	6.0	- .01	+ .06	+ .13	+ .05	- .09	+ .25	+ .40	+ .40	+ .24	- .05
240.0	6.0	- .01	+ .08	+ .15	+ .06	- .09	+ .25	+ .41	+ .41	+ .24	- .05

*"Negative" sign indicates outward wall deflection

Table (B42) - Wall Deflections* in (mm) - Test Series (J) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	9.0	- .06	- .21	- .32	- .35	- .11	+ .08	+ .08	+ .06	+ .01	0.00
20.0	9.0	- .06	- .17	- .23	- .28	- .12	+ .14	+ .20	+ .20	+ .11	0.00
40.0	9.0	- .05	- .14	- .18	- .23	- .12	+ .17	+ .27	+ .27	+ .15	0.00
60.0	9.0	- .05	- .11	- .14	- .20	- .12	+ .19	+ .31	+ .31	+ .18	0.00
80.0	9.0	- .04	- .09	- .12	- .18	- .13	+ .21	+ .33	+ .33	+ .20	0.00
100.0	9.0	- .04	- .07	- .07	- .14	- .13	+ .23	+ .37	+ .37	+ .21	0.00
120.0	9.0	- .04	- .06	- .05	- .13	- .13	+ .23	+ .38	+ .38	+ .22	0.00
140.0	9.0	- .03	- .05	- .03	- .11	- .13	+ .24	+ .39	+ .39	+ .22	0.00
160.0	9.0	- .03	- .03	- .01	- .09	- .13	+ .25	+ .41	+ .40	+ .22	0.00
180.0	9.0	- .03	- .02	+ .01	- .08	- .13	+ .26	+ .42	+ .41	+ .22	- .01
200.0	9.0	- .03	- .01	+ .03	- .07	- .13	+ .26	+ .42	+ .41	+ .22	- .02
220.0	9.0	- .03	0.00	+ .04	- .06	- .13	+ .27	+ .43	+ .41	+ .22	- .03
240.0	9.0	- .03	0.00	+ .05	- .05	- .13	+ .27	+ .44	+ .42	+ .22	- .03

* "Negative" sign indicates outward wall deflection.

Table (B43) - Wall Deflections* in (mm) - Test Series (K) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	12.0	- .08	- .25	- .39	- .43	- .14	+ .11	+ .10	+ .06	+ .02	- .01
20.0	12.0	- .08	- .23	- .34	- .39	- .16	+ .16	+ .20	+ .18	+ .11	- .01
40.0	12.0	- .07	- .20	- .29	- .34	- .16	+ .19	+ .27	+ .25	+ .16	- .01
60.0	12.0	- .07	- .17	- .23	- .30	- .16	+ .23	+ .35	+ .33	+ .20	- .01
80.0	12.0	- .07	- .16	- .20	- .28	- .17	+ .24	+ .38	+ .36	+ .22	- .01
100.0	12.0	- .06	- .14	- .17	- .25	- .17	+ .25	+ .40	+ .38	+ .22	- .01
120.0	12.0	- .06	- .12	- .14	- .23	- .17	+ .26	+ .42	+ .39	+ .23	- .01
140.0	12.0	- .06	- .11	- .13	- .22	- .17	+ .27	+ .43	+ .40	+ .24	- .01
160.0	12.0	- .05	- .08	- .09	- .18	- .17	+ .28	+ .44	+ .41	+ .24	- .02
180.0	12.0	- .05	- .08	- .07	- .18	- .17	+ .28	+ .45	+ .42	+ .24	- .03
200.0	12.0	- .05	- .07	- .06	- .16	- .17	+ .29	+ .46	+ .42	+ .24	- .04
220.0	12.0	- .05	- .06	- .05	- .16	- .17	+ .29	+ .46	+ .42	+ .24	- .05
240.0	12.0	- .05	- .06	- .04	- .15	- .17	+ .30	+ .47	+ .43	+ .24	- .06

* "Negative" sign indicates outward wall deflection.

Table (B 44) - Wall Deflections* in (mm) - Test Series (L) - Full Size Structure

Active Precomp (KN)	Slab Load (KN)	"Wall Deflection" Displacement of Transducer Number:									
		1	2	3	4	5	6	7	8	9	10
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.0	15.0	- .10	- .31	- .47	- .52	- .17	+ .13	+ .12	+ .07	+ .01	0.00
20.0	15.0	- .10	- .29	- .42	- .47	- .18	+ .20	+ .25	+ .23	+ .11	0.00
40.0	15.0	- .09	- .27	- .38	- .44	- .19	+ .24	+ .33	+ .30	+ .17	0.00
60.0	15.0	- .09	- .25	- .34	- .41	- .19	+ .27	+ .39	+ .37	+ .21	0.00
80.0	15.0	- .09	- .22	- .30	- .38	- .19	+ .28	+ .42	+ .40	+ .23	0.00
100.0	15.0	- .09	- .21	- .27	- .36	- .20	+ .29	+ .44	+ .42	+ .23	0.00
120.0	15.0	- .08	- .19	- .24	- .33	- .20	+ .30	+ .46	+ .43	+ .24	0.00
140.0	15.0	- .08	- .18	- .22	- .32	- .20	+ .31	+ .47	+ .45	+ .24	- .01
160.0	15.0	- .08	- .17	- .20	- .30	- .20	+ .31	+ .48	+ .46	+ .24	- .01
180.0	15.0	- .07	- .15	- .18	- .28	- .21	+ .32	+ .50	+ .47	+ .25	- .02
200.0	15.0	- .07	- .14	- .16	- .27	- .21	+ .32	+ .50	+ .47	+ .25	- .02
220.0	15.0	- .07	- .13	- .15	- .26	- .21	+ .33	+ .51	+ .47	+ .25	- .03
240.0	15.0	- .07	- .12	- .13	- .24	- .21	+ .34	+ .51	+ .48	+ .25	- .04

* "Negative" sign indicates outward wall deflection

Full Size Structure

Active precomp. (KN)	Slab Load (KN)	"Wall Deflection" - Displacement of Transducer number:									
		1	2	3	4	5	6	7	8	9	10
						Test Series (M)					
100.0	0.0	+ .02	+ .14	+ .24	+ .19	+ .01	+ .11	+ .22	+ .24	+ .16	0.00
100.0	3.0	+ .01	+ .12	+ .15	+ .10	- .03	+ .15	+ .26	+ .28	+ .18	0.00
100.0	6.0	- .01	+ .03	+ .06	- .01	- .07	+ .18	+ .30	+ .31	+ .18	0.00
100.0	9.0	- .03	- .04	- .04	- .12	- .11	+ .21	+ .34	+ .34	+ .20	0.00
100.0	12.0	- .05	- .11	- .14	- .23	- .15	+ .24	+ .38	+ .37	+ .21	0.00
100.0	15.0	- .08	- .17	- .23	- .33	- .18	+ .28	+ .43	+ .41	+ .23	0.00
						Test Series (N)					
200.0	0.0	+ .03	+ .19	+ .33	+ .25	- .01	+ .16	+ .27	+ .28	+ .18	- .03
200.0	3.0	+ .01	+ .13	+ .23	+ .16	- .03	+ .22	+ .33	+ .32	+ .18	- .05
200.0	6.0	0.00	+ .08	+ .15	+ .07	- .07	+ .24	+ .37	+ .35	+ .18	- .07
200.0	9.0	- .02	+ .02	+ .05	- .04	- .10	+ .27	+ .41	+ .38	+ .20	- .07
200.0	12.0	- .05	- .05	- .03	- .14	- .14	+ .31	+ .46	+ .43	+ .21	- .07
200.0	15.0	- .07	- .11	- .12	- .24	- .18	+ .33	+ .50	+ .46	+ .22	- .07
						Test Series (P)					
300.0	0.0	+ .04	+ .24	+ .40	+ .32	0.00	+ .22	+ .30	+ .30	+ .17	- .05
300.0	3.0	+ .02	+ .17	+ .30	+ .22	- .02	+ .28	+ .37	+ .34	+ .17	- .08
300.0	6.0	0.00	+ .12	+ .21	+ .12	- .06	+ .32	+ .41	+ .37	+ .18	- .10
300.0	9.0	- .02	+ .06	+ .12	+ .02	- .09	+ .36	+ .47	+ .42	+ .18	- .10
300.0	12.0	- .04	- .01	+ .04	- .10	- .13	+ .39	+ .52	+ .46	+ .20	- .10
300.0	15.0	- .06	- .07	- .06	- .19	- .16	+ .43	+ .57	+ .50	+ .22	- .10

Table (B46) - Wall Rotations - Test Series (A)

Full Size Structure

Slab Load (KN)	(θ_3)* Lower wall Rotation* 10^{-3} (Radians)	(θ_5)** Upper wall Rotation* 10^{-3} (Radians)
0.0	0.0	0.0
1.0	- 0.01	+ 0.057
2.0	- 0.02	+ 0.139
3.0	- 0.024	+ 0.212
4.0	- 0.030	+ 0.295
5.0	- 0.035	+ 0.417
6.0	- 0.042	+ 0.424
7.0	- 0.048	+ 0.550
8.0	- 0.053	+ 0.610
9.0	- 0.058	+ 0.630
10.0	- 0.060	+ 0.710
11.0	- 0.066	+ 0.760
12.0	- 0.066	+ 0.830
13.0	- 0.070	+ 0.850
14.0	- 0.073	+ 0.945
15.0	- 0.078	+ 1.010

* Clockwise Rotation "Positive"

** Counter clockwise Rotation "Positive"

Table (B47) - Wall Rotations - Test Series (B)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	20.0	+ .026	- .120	+ .026	- .120	---	---
3.0	20.0	+ .008	+ .015	---	---	- .018	+ .135
0.0	40.0	+ .038	- .225	+ .038	- .225	---	---
3.0	40.0	+ .016	- .085	---	---	- .022	+ .140
0.0	60.0	+ .046	- .230	+ .046	- .23	---	---
3.0	60.0	+ .026	- .083	---	---	- .020	+ .147
0.0	80.0	+ .058	- .280	+ .058	- .28	---	---
3.0	80.0	+ .038	- .130	---	---	- .020	+ .150
0.0	100.0	+ .063	- .287	+ .063	- .287	---	---
3.0	100.0	+ .044	- .130	---	---	- .019	+ .157
0.0	120.0	+ .066	- .30	+ .066	- .30	---	---
3.0	120.0	+ .046	- .145	---	---	- .020	+ .155
0.0	140.0	+ .068	- .272	+ .068	- .272	---	---
3.0	140.0	+ .050	- .128	---	---	- .018	+ .144
0.0	160.0	+ .078	- .263	+ .078	- .263	---	---
3.0	160.0	+ .053	- .115	---	---	- .025	+ .148
0.0	180.0	+ .083	- .245	+ .083	- .245	---	---
3.0	180.0	+ .062	- .095	---	---	- .021	+ .150
0.0	200.0	+ .088	- .233	+ .088	- .233	---	---
3.0	200.0	+ .063	- .105	---	---	- .025	+ .128
0.0	220.0	+ .090	- .265	+ .090	- .265	---	---
3.0	220.0	+ .066	- .093	---	---	- .024	+ .172
0.0	240.0	+ .094	- .270	+ .094	- .270	------	
3.0	240.0	+ .070	- .085	---	---	- .024	+ .185

Table (B48) - Wall Rotations - Test Series (C)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.0	20.0	+ .023	- .152	+ .023	- .152	---	---
6.0	20.0	- .012	+ .158	---	---	- .035	+ .31
0.0	40.0	+ .030	- .217	+ .030	- .217	---	---
6.0	40.0	- .002	+ .090	---	---	- .032	+ .307
0.0	60.0	+ .043	- .250	+ .043	- .250	---	---
6.0	60.0	+ .002	+ .046	---	---	- .041	+ .296
0.0	80.0	+ .052	- .282	+ .052	- .282	---	---
6.0	80.0	+ .015	+ .016	---	---	- .037	+ .298
0.0	100.0	+ .058	- .284	+ .058	- .284	---	---
6.0	100.0	+ .023	+ .004	---	---	- .035	+ .288
0.0	120.0	+ .052	- .262	+ .052	- .262	---	---
6.0	120.0	+ .028	+ .004	---	---	- .024	+ .266
0.0	140.0	+ .068	- .267	+ .068	- .267	---	---
6.0	140.0	+ .036	+ .008	---	---	- .032	+ .275
0.0	160.0	+ .076	- .265	+ .076	- .265	---	---
6.0	160.0	+ .040	+ .028	---	---	- .036	+ .293
0.0	180.0	+ .078	- .249	+ .078	- .249	---	---
6.0	180.0	+ .043	+ .034	---	---	- .035	+ .283
0.0	200.0	+ .083	- .260	+ .083	- .260	---	---
6.0	200.0	+ .048	- .048	---	---	- .035	+ .308
0.0	220.0	+ .081	- .222	+ .081	- .222	---	---
6.0	220.0	+ .048	+ .072	---	---	- .033	+ .294
0.0	240.0	+ .087	- .247	+ .087	- .247	---	---
6.0	240.0	+ .052	+ .077	---	---	- .035	+ .324

Table (B49) - Wall Rotations - Test Series (D)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.0	20.0	+ .018	- .147	+ .018	- .147	---	---
9.0	20.0	- .022	+ .410	---	---	- .040	+ .557
0.0	40.0	+ .033	- .212	+ .033	- .212	---	---
9.0	40.0	- .009	+ .268	---	---	- .042	+ .48
0.0	60.0	+ .040	- .237	+ .040	- .237	---	---
9.0	60.0	0.00	+ .186	---	---	- .040	+ .423
0.0	80.0	+ .043	- .262	+ .043	- .262	---	---
9.0	80.0	+ .004	+ .153	---	---	- .039	+ .415
0.0	100.0	+ .053	- .282	+ .053	- .282	---	---
9.0	100.0	+ .010	+ .140	---	---	- .043	+ .422
0.0	120.0	+ .054	- .287	+ .054	- .287	---	---
9.0	120.0	+ .014	+ .130	---	---	- .040	+ .417
0.0	140.0	+ .058	- .292	+ .058	- .292	---	---
9.0	140.0	+ .018	+ .130	---	---	- .040	+ .422
0.0	160.0	+ .064	- .287	+ .064	- .287	---	---
9.0	160.0	+ .023	+ .150	---	---	- .041	+ .437
0.0	180.0	+ .064	- .287	+ .064	- .287	---	---
9.0	180.0	+ .026	+ .158	---	---	- .038	+ .445
0.0	200.0	+ .072	- .287	+ .072	- .287	---	---
9.0	200.0	+ .028	+ .170	---	---	-. 044	+ .457
0.0	220.0	+ .068	- .254	+ .068	- .254	---	---
9.0	220.0	+ .028	+ .168	---	---	- .040	+ .422
0.0	240.0	+ .078	- .217	+ .078	- .217	---	---
9.0	240.0	+ .033	+ .170	---	---	- .045	+ .387

Table (B50) - Wall Rotations - Test Series (^{E})

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.0	20.0	+ .014	- .062	+ .014	- .062	---	---
12.0	20.0	- .050	+ .753	---	---	- .064	+ .815
0.0	40.0	+ .028	- .182	+ .028	- .182	---	---
12.0	40.0	- .035	+ .518	---	---	- .063	+ .70
0.0	60.0	+ .034	- .212	+ .034	- .212	---	---
12.0	60.0	- .022	+ .343	---	---	- .056	+ .555
0.0	80.0	+ .036	- .232	+ .036	- .232	---	---
12.0	80.0	- .016	+ .303	---	---	- .052	+ .535
0.0	100.0	+ .048	- .237	+ .048	- .237	---	---
12.0	100.0	- .011	+ .306	---	---	- .059	+ .543
0.0	120.0	+ .050	- .249	+ .050	- .249	---	---
12.0	120.0	- .001	+ .303	---	---	- .051	+ .552
0.0	140.0	+ .054	- .230	+ .054	- .23	---	---
12.0	140.0	- .001	+ .317	---	---	- .055	+ .547
0.0	160.0	+ .059	- .222	+ .059	- .222	---	---
12.0	160.0	+ .001	+ .317	---	---	- .058	+ .539
0.0	180.0	+ .064	- .239	+ .064	- .239	---	---
12.0	180.0	+ .004	+ .313	---	---	- .060	+ .552
0.0	200.0	+ .066	- .197	+ .066	- .197	---	---
12.0	200.0	+ .011	+ .338	---	---	- .055	+ .535
0.0	220.0	+ .069	- .232	+ .069	- .232	---	---
12.0	220.0	+ .014	+ .256	---	---	- .055	+ .488
0.0	240.0	+ .076	- .232	+ .076	- .232	---	---
12.0	240.0	+ .016	+ .335	---	---	- .060	+ .567

Table (B51) - Wall Rotations - Test Series (F)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
0.0	0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.0	20.0	+ .024	- .144	+ .024	- .144	---	---
15.0	20.0	- .051	+ .674	---	---	- .075	+ .818
0.0	40.0	+ .036	- .224	+ .036	- .224	---	---
15.0	40.0	- .043	+ .634	---	---	- .079	+ .858
0.0	60.0	+ .044	- .234	+ .044	- .234	---	---
15.0	60.0	- .032	+ .549	---	---	- .076	+ .783
0.0	80.0	+ .050	- .226	+ .050	- .226	---	---
15.0	80.0	- .020	+ .449	---	---	- .070	+ .675
0.0	100.0	+ .060	- .238	+ .060	- .238	---	---
15.0	100.0	- .013	+ .479	---	---	- .073	+ .717
0.0	120.0	+ .060	- .238	+ .060	- .238	---	---
15.0	120.0	- .013	+ .452	---	---	- .073	+ .690
0.0	140.0	+ .065	- .276	+ .065	- .276	---	---
15.0	140.0	- .002	+ .459	---	---	- .067	+ .735
0.0	160.0	+ .074	- .268	+ .074	- .268	---	---
15.0	160.0	- .001	+ .436	---	---	- .075	+ .704
0.0	180.0	+ .074	- .281	+ .074	- .281	---	---
15.0	180.0	- .001	+ .434	---	---	- .075	+ .715
0.0	200.0	+ .074	- .256	+ .074	- .256	---	---
15.0	200.0	0.00	+ .398	---	---	- .074	+ .654
0.0	220.0	+ .076	- .276	+ .076	- .276	---	---
15.0	220.0	-0.00	+ .486	---	---	+ .076	+ .762
0.0	240.0	+ .076	- .244	+ .076	- .244	---	---
15.0	240.0	+ .004	+ .394	---	---	- .080	+ .638

Table (B52) - Wall Rotations - Test Series (G)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3.0	0.0	- .018	+ .243	---	---	- .018	+ .243
3.0	20.0	- .003	+ .086	+ .015	- .157	- .018	+ .135
3.0	40.0	+ .014	- .077	+ .032	- .32	- .022	+ .140
3.0	60.0	+ .025	- .087	+ .043	- .33	- .020	+ .147
3.0	80.0	+ .030	- .107	+ .048	- .35	- .020	+ .150
3.0	100.0	+ .030	- .137	+ .048	- .38	- .019	+ .157
3.0	120.0	+ .037	- .137	+ .055	- .38	- .020	+ .155
3.0	140.0	+ .047	- .125	+ .065	- .368	- .018	+ .144
3.0	160.0	+ .047	- .105	+ .065	- .348	- .025	+ .148
3.0	180.0	+ .047	- .105	+ .065	- .348	- .021	+ .150
3.0	200.0	+ .050	- .105	+ .068	- .348	- .025	+ .128
3.0	220.0	+ .052	- .094	+ .070	- .337	- .024	+ .172
3.0	240.0	+ .057	- .087	+ .075	- .330	- .024	+ .185

Table (B 53) - Wall Rotations - Test Series (H)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
6.0	0.0	- .035	+ .466	---	---	- .035	+ .466
6.0	20.0	- .025	+ .33	+ .01	- .136	- .035	+ .31
6.0	40.0	+ .003	+ .083	+ .038	- .383	- .032	+ .307
6.0	60.0	+ .010	+ .053	+ .045	- .413	- .041	+ .296
6.0	80.0	+ .012	+ .018	+ .047	- .448	- .037	+ .298
6.0	100.0	+ .025	+ .013	+ .060	- .453	- .035	+ .288
6.0	120.0	+ .027	- .007	+ .062	- .473	- .024	+ .266
6.0	140.0	+ .030	- .012	+ .065	- .478	- .032	+ .275
6.0	160.0	+ .040	- .012	+ .075	- .478	- .036	+ .293
6.0	180.0	+ .042	- .012	+ .077	- .478	- .035	+ .283
6.0	200.0	+ .045	- .007	+ .080	- .473	- .035	+ .308
6.0	220.0	+ .050	+ .013	+ .085	- .453	- .033	+ .294
6.0	240.0	+ .052	+ .013	+ .087	- .453	- .035	+ .324

Table (B54) - Wall Rotations - Test Series (J)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
9.0	0.0	- .049	+ .668	---	---	- .049	+ .668
9.0	20.0	- .027	+ .446	+ .022	- .222	- .040	+ .557
9.0	40.0	- .022	+ .293	+ .027	- .375	- .042	+ .480
9.0	60.0	- .006	+ .173	+ .043	- .495	- .040	+ .423
9.0	80.0	- .002	+ .153	+ .047	- .515	- .039	+ .415
9.0	100.0	+ .006	+ .143	+ .055	- .525	- .043	+ .422
9.0	120.0	+ .016	+ .138	+ .065	- .530	- .040	+ .417
9.0	140.0	+ .023	+ .133	+ .072	- .535	- .040	+ .422
9.0	160.0	+ .028	+ .143	+ .077	- .525	- .041	+ .437
9.0	180.0	+ .033	+ .143	+ .082	- .525	- .038	+ .445
9.0	200.0	+ .033	+ .146	+ .082	- .522	- .044	+ .457
9.0	220.0	+ .033	+ .158	+ .082	- .510	- .040	+ .422
9.0	240.0	+ .038	+ .163	+ .087	- .505	- .045	+ .387

Table (B55) - Wall Rotations - Test Series (K)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
12.0	0.0	- .066	+ .85	---	---	- .066	+ .85
12.0	20.0	- .055	+ .71	+ .011	- .140	- .064	+ .815
12.0	40.0	- .048	+ .51	+ .018	- .34	- .063	+ .700
12.0	60.0	- .038	+ .35	+ .028	- .50	- .056	+ .555
12.0	80.0	- .033	+ .305	+ .033	- .545	- .052	+ .535
12.0	100.0	- .023	+ .305	+ .043	- .545	- .059	+ .543
12.0	120.0	- .013	+ .280	+ .053	- .570	- .051	+ .552
12.0	140.0	- .008	+ .265	+ .058	- .585	- .055	+ .547
12.0	160.0	- .005	+ .255	+ .061	- .595	- .058	+ .539
12.0	180.0	- .005	+ .285	+ .061	- .565	- .060	+ .552
12.0	200.0	+ .002	+ .305	+ .068	- .545	-.055	+ .535
12.0	220.0	+ .002	+ .310	+ .068	- .540	- .055	+ .488
12.0	240.0	+ .007	+ .315	+ .073	- .535	- .060	+ .567

Table (B56) - Wall Rotations - Test Series (L)

Full Size Structure

Slab load (KN)	Active Precomp, (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to precomp.		Due to F.Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
15.0	0.0	- .080	+ .97	---	---	- .080	+ .97
15.0	20.0	- .069	+ .80	+ .011	- .17	- .075	+ .818
15.0	40.0	- .068	+ .72	+ .012	- .25	- .079	+ .858
15.0	60.0	- .046	+ .61	+ .034	- .36	- .076	+ .783
15.0	80.0	- .046	+ .55	+ .034	- .42	- .070	+ .675
15.0	100.0	- .033	+ .55	+ .047	- .42	- .073	+ .717
15.0	120.0	- .027	+ .54	+ .053	- .43	- .073	+ .690
15.0	140.0	- .020	+ .492	+ .060	- .478	- .067	+ .735
15.0	160.0	- .015	+ .515	+ .065	- .455	- .075	+ .704
15.0	180.0	- .013	+ .525	+ .067	- .445	- .075	+ .715
15.0	200.0	- .010	+ .515	+ .070	- .455	- .074	+ .654
15.0	220.0	- .008	+ .500	+ .072	- .470	- .076	+ .762
15.0	240.0	- .006	+ .465	+ .074	- .505	- .080	+ .638

Table (B57) - Wall Rotations - Test Series (M, N, P)

Full Size Structure

Slab Load (KN)	Active Precomp. (KN)	$\theta_3 * 10^{-3}$ lower wall	$\theta_5 * 10^{-3}$ upper wall	Due to Precomp.		Due to F. Load	
				θ_{lower}	θ_{upper}	θ_{lower}	θ_{upper}
TEST SERIES (M)							
0.0	100.0	+ .052	- .325	+ .052	- .325	---	---
3.0	100.0	+ .035	- .147	+ .048	- .38	- .017	+ .178
6.0	100.0	+ .020	- .035	+ .060	- .453	- .032	+ .290
9.0	100.0	+ .005	+ .085	+ .055	- .525	- .047	+ .410
12.0	100.0	- .012	+ .235	+ .043	- .545	- .064	+ .560
15.0	100.0	- .010	+ .425	+ .047	- .42	- .062	+ .750
TEST SERIES (N)							
0.0	200.0	+ .073	- .240	+ .073	- .240	---	---
3.0	200.0	+ .063	- .105	+ .068	- .348	- .010	+ .135
6.0	200.0	+ .043	- .003	+ .080	- .473	- .03	+ .237
9.0	200.0	+ .033	+ .125	+ .082	- .522	- .04	+ .365
12.0	200.0	+ .015	+ .270	+ .068	- .545	- .058	+ .510
15.0	200.0	0.00	+ .465	+ .070	- .455	- .073	+ .705
TEST SERIES (P)							
0.0	300.0	+ .075	- .174	+ .075	- .174	---	---
3.0	300.0	+ .060	- .044	+ .071	- .360	- .015	+ .130
6.0	300.0	+ .050	+ .046	+ .079	- .486	- .025	+ .220
9.0	300.0	+ .030	+ .186	+ .082	- .540	- .045	+ .360
12.0	300.0	+ .020	+ .351	+ .075	- .545	- .055	+ .525
15.0	300.0	+ .005	+ .576	+ .075	- .535	- .070	+ .750

APPENDIX (C) - COMPUTER PROGRAMMES

C1 - MOMENT-ROTATION EQUATIONS (DOUBLE CURVATURE).

C2 - MOMENT-ROTATION EQUATIONS (SINGLE CURVATURE).

C3 - WALL BUCKLING LOADS (DOUBLE & SINGLE CURVATURE).

C4 - CAPACITY REDUCTION FACTORS(DOUBLE CURVATURE-UNCRACKED).

C5 - CAPACITY REDUCTION FACTORS(DOUBLE CURVATURE-CRACKED).

C6 - CAPACITY REDUCTION FACTORS, SINGLE CURVATURE BENDING, $\mathcal{E}_1/\mathcal{E}_2$ = 0.0 (UNCRACKED WALLS).

C7 - CAPACITY REDUCTION FACTORS, SINGLE CURVATURE BENDING, $\mathcal{E}_1/\mathcal{E}_2$ = 0.0 (CRACKED WALLS).

C8 - CAPACITY REDUCTION FACTORS, SINGLE CURVATURE BENDING, $\mathcal{E}_1/\mathcal{E}_2$ = 1.0(UNCRACKED &CRACKED WALLS).

C9 - SOLUTION OF CUBIC EQUATIONS PROGRAMME.

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
C       MOMENT ROTATION RELATIONS
C       WALLS IN DOUBLE CURVITURE
C       PROGRAMER : A. AWNI
C       UE CORRESPONDS TO UNCRACKED ECCENTRICITY
C        CE CORRESPONDS TO CRACKED ECCENTICITY
        REAL UE(9)
        REAL PHY(20)
        REAL CE(18)
      1 FORMAT(9F6.2)
      2 FORMAT(20F4.1)
      3 FORMAT('1',///T21,'FINAL     COMPUTED     RESULTS'//)
      4 FORMAT(    22X,'MOMENT ROTATION RELATIONS'/)
      5 FORMAT(    22X,'WALLS IN DOUBLE CURVITURE'////)
      6 FORMAT(    13X,'ECC/WALL T.',10X,'PHY',15X,'Z'///)
      7 FORMAT(    12X,F10.6,5X,F10.3,10X,F10.6)
        WRITE(6,3)
        WRITE(6,4)
        WRITE(6,5)
        WRITE(6,6)
        READ(5,1) (UE(I),I=1,9)
        READ(5,2) (PHY(J),J=1,20)
        BKERN=(1.0/6.0)
        IF(UE(I)-BKERN)100,300,300
    100 DO 200 I=1,9
        DO 200 J=1,20
        Z=(9.869*(PHY(J))**2)/(UE(I)+PHY(J))**2
        ZMAX=9.869
        IF(Z-ZMAX)30,30,20
     20 Z=ZMAX
     30 WRITE(6,7) (UE(I),PHY(J),Z)
    200 CONTINUE
        READ(5,1) (CE(I),I=1,18)
        READ(5,2) (PHY(J),J=1,20)
    300 DO 400 I=1,18
        DO 400 J=1,20
        CHECK1=(CE(I)-PHY(J))
        IF(CHECK1)10,11,11
     10 Z=(33.31*PHY(J))*(1.0-((CE(I)+PHY(J))**2/
       1(2.0*PHY(J))))**2
        ZMAX=9.869
        IF(Z-ZMAX)60,60,59
     59 Z=ZMAX
     60 GOTO 50
     11 Z=(33.31*PHY(J))*(1.0-2.0*CE(I))**2
        ZMAX=9.869
        IF(Z-ZMAX)50,50,40
     40 Z=ZMAX
     50 WRITE(6,7) (CE(I),PHY(J),Z)
    400 CONTINUE
        STOP
        END
```

CODE+GLA+SYMTABS+ARRAYS = 1512+ 752+ 88+ 192= 2544 BYTES

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
      C     MOMENT ROTATION RELATIONS
      C     WALLS IN SINGLE CURVITURE
      C     PROGRAMER : A. AWNI
      C     UE CORRESPONDS TO UNCRACKED ECCENTRICITY
      C      CE CORRESPONDS TO CRACKED ECCENTRICITY
            REAL UE(9)
            REAL PHY(20)
            REAL CE(18)
          1 FORMAT(9F6.2)
          2 FORMAT(20F4.1)
          3 FORMAT('1',///T21,'FINAL     COMPUTED     RESULTS'//)
          4 FORMAT(    22X,'MOMENT ROTATION RELATIONS'/)
          5 FORMAT(    22X,'WALLS IN SINGLE CURVITURE'////)
          6 FORMAT(    13X,'ECC/WALL T.',10X,'PHY',15X,'Z'///)
          7 FORMAT(    12X,F10.6,5X,F10.3,10X,F10.6)
            WRITE(6,3)
            WRITE(6,4)
            WRITE(6,5)
            WRITE(6,6)
            READ(5,1) (UE(I),I=1,9)
            READ(5,2) (PHY(J),J=1,20)
            BKERN=(1.0/6.0)
            IF(UE(I)-BKERN)100,300,300
        100 DO 200 I=1,9
            DO 200 J=1,20
            Z=(9.869*PHY(J))/(4.0*UE(I)+PHY(J))
            ZMAX=9.869
            IF(Z-ZMAX)30,30,20
         20 Z=ZMAX
         30 WRITE(6,7) (UE(I),PHY(J),Z)
        200 CONTINUE
            READ(5,1) (CE(I),I=1,18)
            READ(5,2) (PHY(J),J=1,20)
        300 DO 400 I=1,18
            DO 400 J=1,20
            Z=(33.31*PHY(J))*(1.0-2.0*CE(I)-(PHY(J)/2.0))**2
            ZMAX=9.869
            IF(Z-ZMAX)50,50,40
         40 Z=ZMAX
         50 WRITE(6,7) (CE(I),PHY(J),Z)
        400 CONTINUE
            STOP
            END
```

CODE+GLA+SYMTABS+ARRAYS = 1120+ 752+ 80+ 192= 2144 BYTES

*COMPILATION SUCCESSFUL

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
C       THIS PROGRAM CALCULATES THE WALL BUCKLING LOADS
C       FOR WALLS IN DOUBLE & SINGLE CURVATURE
C       PROGRAMMER : ADNAN A. AWNI
C
C
C
C
    001 REAL E(41)
    002 FORMAT(  6F10.4)
    003 FORMAT('1',// T21,'FINAL    COMPUTED    RESULTS'//)
    004 FORMAT(   22X,'WALLS IN DOUBLE CURVATURE'////)
    005 FORMAT(   11X,'E=ECC/WALL T.',13X,'PHY',14X,
       1'ZMAX'///)
    006 FORMAT(   12X,F10.6,10X,F10.6,9X,F10.6)
    007 FORMAT(   22X,'WALLS IN SINGLE CURVATURE'////)
     15 FORMAT(   25X,'WALL BUCKLING LOADS'//)
    008 WRITE(6,3)
        WRITE(6,15)
    009 WRITE(6,4)
    010 WRITE(6,5)
     11 READ(5,2) (E(I),I=1,41)
     12 DO 800 I=1,41
    013 XROOT=((4.*E(I)**2)-(2.*E(I))+1.0)
    014 IF(XROOT)901,100,100
    100 ROOT=((4.*E(I)**2)-(2.*E(I))+1.0)**0.5
    101 PHY1=(1.0-E(I))/(3.0)+((ROOT)/(3.0))
    102 PHYMAX=(2.0/3.0)
C       WHEN THE ECCENTRICITY=0, THE VALUE OF THE
C       BUCKLING ROTATION IS MAXIMUM .
    103 CHECK1=(PHYMAX-PHY1)
    104 IF(CHECK1)200,300,300
    200 PHY=PHYMAX
    201 GOTO 301
    300 PHY=PHY1
    301 BKERN=((1.0)/(6.0))
    302 CHECK2=(E(I)-BKERN)
    303 IF(CHECK2)400,500,500
    400 ZMAX=(9.86962*PHY**2)/((E(I)+PHY))**2
C       STAT. 400 CORRESPONDS TO UNCRACKED  WALL
C       AND EQUATION (2.32) IS USED.
        ZULT=9.869
        IF(ZMAX-ZULT)388,388,389
    388 ZMAX=ZMAX
        GOTO 701
    389 ZMAX=ZULT
    401 GOTO 701
    500 CHECK3=(PHY-E(I))
    501 IF(CHECK3)600,700,700
C
C
C
C
C               PROGRAM  CONTINUED
C
C
C
C
```

```
C
C
C
C
C
C
C
C
      600 ZMAX=33.31*PHY*(1.0-2.0*E(I))**2
C         STAT. 600 CORRESPONDS TO CRACKED WALLS WITH
C         'PHY' LESS THAN THE RELATIVE END ECCENTRICITY
C         AND EQUATION (2.36) IS USED .
          ZULT=9.869
          IF(ZMAX-ZULT)488,488,489
      488 ZMAX=ZMAX
          GOTO 701
      489 ZMAX=ZULT
      601 GOTO 701
      700 ZMAX=33.31*PHY*(1.0-((E(I)+PHY)**2)/(2.*PHY))**2
C         STAT. 700 CORRESPONDS TO CRACKED WALLS WITH
C         WALL END ROTATION GREATER OR EQUAL TO  THE
C         RELATIVE END ECCENTRICITY .
          ZULT=9.869
          IF(ZMAX-ZULT)688,688,689
      688 ZMAX=ZMAX
          GOTO 701
      689 ZMAX=ZULT
      701 WRITE(6,6) (E(I),PHY,ZMAX)
      800 CONTINUE
C         CALCULATION OF 'PHY' & 'ZMAX'  FOR WALLS IN
C         SINGLE CURVATURE  BEGINS
      801 WRITE(6,3)
          WRITE(6,15)
      802 WRITE(6,7)
      803 WRITE(6,5)
      804 READ(5,2) (E(J),J=1,41)
      805 DO 900 J=1,41
      806 PHY=((2.)/(3.))*(1.-2.*E(J))
      807 ZMAX=9.86962*(1.-2.*E(J))**3
          ZULT=9.869
          IF(ZMAX-ZULT)588,588,589
      588 ZMAX=ZMAX
          GOTO 808
      589 ZMAX=ZULT
      808 WRITE(6,6) (E(J),PHY,ZMAX)
      900 CONTINUE
      901 STOP
      902 END
```

CODE+GLA+SYMTABS+ARRAYS = 2272+ 744+ 176+ 168= 3360 BYTES

*COMPILATION SUCCESSFUL

FINAL COMPUTED RESULTS

WALL BUCKLING LOADS

WALLS IN DOUBLE CURVATURE

E=ECC/WALL T.	PHY	ZMAX
0.000000	0.666667	9.868999
0.010000	0.660050	9.577223
0.020000	0.653537	9.292184
0.030000	0.647130	9.014451
0.040000	0.640832	8.743974
0.050000	0.634646	8.480693
0.060000	0.628576	8.224551
0.070000	0.622623	7.975481
0.080000	0.616792	7.733418
0.090000	0.611085	7.498290
0.100000	0.605505	7.270021
0.110000	0.600055	7.048530
0.120000	0.594737	6.833732
0.130000	0.589555	6.625537
0.140000	0.584511	6.423853
0.150000	0.579606	6.228585
0.160000	0.574845	6.039629
0.170000	0.570227	5.127078
0.180000	0.565756	4.872638
0.190000	0.561433	4.621873
0.200000	0.557260	4.374967
0.210000	0.553237	4.132108
0.220000	0.549367	3.893493
0.230000	0.545650	3.659335
0.240000	0.542085	3.429843
0.250000	0.538675	3.205248
0.260000	0.535419	2.985789
0.270000	0.532316	2.771694
0.280000	0.529367	2.563225
0.290000	0.526571	2.360630
0.300000	0.523927	2.164170
0.320000	0.519089	1.790791
0.340000	0.514844	1.445362
0.360000	0.511177	1.130287
0.380000	0.508071	0.848095
0.400000	0.505505	0.601416
0.420000	0.503459	0.392996
0.440000	0.501909	0.225670
0.460000	0.500832	0.102372
0.480000	0.500204	0.026117
0.500000	0.500000	0.000000

FINAL COMPUTED RESULTS

WALL BUCKLING LOADS

WALLS IN SINGLE CURVATURE

E=ECC/WALL T.	PHY	ZMAX
0.000000	0.666667	9.868999
0.010000	0.653333	9.289207
0.020000	0.640000	8.732008
0.030000	0.626667	8.197549
0.040000	0.613333	7.685354
0.050000	0.600000	7.194952
0.060000	0.586667	6.725870
0.070000	0.573333	6.277630
0.080000	0.560000	5.849763
0.090000	0.546667	5.441792
0.100000	0.533333	5.053246
0.110000	0.520000	4.683649
0.120000	0.506667	4.332527
0.130000	0.493333	3.999406
0.140000	0.480000	3.683816
0.150000	0.466667	3.385280
0.160000	0.453333	3.103325
0.170000	0.440000	2.837477
0.180000	0.426667	2.587262
0.190000	0.413333	2.352206
0.200000	0.400000	2.131838
0.210000	0.386667	1.925680
0.220000	0.373333	1.733263
0.230000	0.360000	1.554110
0.240000	0.346667	1.387748
0.250000	0.333333	1.233702
0.260000	0.320000	1.091500
0.270000	0.306667	0.960669
0.280000	0.293333	0.840733
0.290000	0.280000	0.731221
0.300000	0.266667	0.631656
0.320000	0.240000	0.460477
0.340000	0.213333	0.323406
0.360000	0.186667	0.216658
0.380000	0.160000	0.136438
0.400000	0.133333	0.078957
0.420000	0.106667	0.040426
0.440000	0.080000	0.017055
0.460000	0.053333	0.005053
0.480000	0.026667	0.000632
0.500000	0.000000	0.000000

STOP

91 KBYTES USED JOB-STEP RETURN CODE = 0

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
C        STRESS REDUCTION FACTORS 'WALLS IN DOUBLE CURVATURE'
C       PROGRAMMER: ADNAN A. AWNI
        REAL PHY(799)
        REAL E(17)
        REAL SLEND(47)
    001 FORMAT(9F8.4)
    002 FORMAT(10F8.3)
    003 FORMAT('1',// T21,'FINAL    COMPUTED    RESULTS'/)
    004 FORMAT(   22X,'WALLS IN DOUBLE CURVATURE')
    005 FORMAT(   /13X,'E/T',12X,'H/T',10X,'PHY',13X,'V'/)
    006 FORMAT(   12X,F6.4,8X,F6.2,8X,F6.4,8X,F8.6)
    007 FORMAT(   23X,'STRESS REDUCTION FACTOR'/)
     08 FORMAT(13F6.4)
     09 READ(5,1) (E(I),I=1,17)
    010 READ(5,2) (SLEND(J),J=1,47)
    011 READ(5,8) (PHY(K),K=1,799)
    012 DO 140 I=1,17
    013 DO 140 J=1,47
    014 K=((I-1)*47)+J
    015 REALK=((SLEND(J)/(2.0))**2)/(55.5)
    016 PHYB=((1.0-E(I))+(4.0*E(I)**2-2.0*E(I)+1.0)**0.5)/(3.0)
    017 IF(PHY(K)-PHYB)20,18,18
    018 PHY(K)=PHYB
    019 GOTO 36
    020 CHECKB=(PHY(K)-E(I))
    021 IF(CHECKB)22,22,24
    022 V1=(1.5*SLEND(J)/2.0)/((1.+6.*E(I))*((SLEND(J)/2.0)-1.5))
    023 VMAX=1.0
    024 IF(V1-VMAX)25,25,27
    025 V=V1
    026 GOTO 80
    027 V=VMAX
    028 GOTO 80
    029 V2=(3.0)/((2.0)+((3.0/PHY(K))*(E(I)+PHY(K))**2))
    030 VMAX=1.0
    031 IF(V2-VMAX)32,32,34
    032 V=V2
    033 GOTO 80
    034 V=VMAX
    035 GOTO 80
    036 Z1=(9.869*(PHY(K)**2))/((E(I)+PHY(K))**2)
    037 V3=Z1/REALK
    038 VMAX=1.0
    039 IF(V3-VMAX)40,40,60
    040 V=V3
    050 GOTO 80
    060 V=VMAX
    080 IF(SLEND(J)-5.0)90,90,130
    090 WRITE(6,3)
    100 WRITE(6,7)
    110 WRITE(6,4)
    120 WRITE(6,5)
    130 WRITE(6,6) (E(I),SLEND(J),PHY(K),V)
    140 CONTINUE
    150 STOP
    160 END
```

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0000	5.00	0.0000	1.000000
0.0000	6.00	0.0000	1.000000
0.0000	7.00	0.0000	1.000000
0.0000	8.00	0.0000	1.000000
0.0000	9.00	0.0000	1.000000
0.0000	10.00	0.0000	1.000000
0.0000	11.00	0.0000	1.000000
0.0000	12.00	0.0000	1.000000
0.0000	13.00	0.0000	1.000000
0.0000	14.00	0.0000	1.000000
0.0000	15.00	0.0000	1.000000
0.0000	16.00	0.0000	1.000000
0.0000	17.00	0.0000	1.000000
0.0000	18.00	0.0000	1.000000
0.0000	19.00	0.0000	1.000000
0.0000	20.00	0.0000	1.000000
0.0000	21.00	0.0000	1.000000
0.0000	22.00	0.0000	1.000000
0.0000	23.00	0.0000	1.000000
0.0000	24.00	0.0000	1.000000
0.0000	25.00	0.0000	1.000000
0.0000	26.00	0.0000	1.000000
0.0000	27.00	0.0000	1.000000
0.0000	28.00	0.0000	1.000000
0.0000	29.00	0.0000	1.000000
0.0000	30.00	0.0000	1.000000
0.0000	31.00	0.0000	1.000000
0.0000	32.00	0.0000	1.000000
0.0000	33.00	0.0000	1.000000
0.0000	34.00	0.0000	1.000000
0.0000	35.00	0.0000	1.000000
0.0000	36.00	0.0000	1.000000
0.0000	37.00	0.0000	1.000000
0.0000	38.00	0.0000	1.000000
0.0000	39.00	0.0275	1.000000
0.0000	40.00	0.0636	1.000000
0.0000	42.00	0.1384	1.000000
0.0000	44.00	0.2169	1.000000
0.0000	46.00	0.2991	1.000000
0.0000	48.00	0.3849	1.000000
0.0000	50.00	0.4744	1.000000
0.0000	55.00	0.6667	0.724270
0.0000	60.00	0.6667	0.608589
0.0000	65.00	0.6667	0.518561
0.0000	70.00	0.6667	0.447126
0.0000	75.00	0.6667	0.389497
0.0000	80.00	0.6667	0.342331

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0100	5.00	0.0013	1.000000
0.0100	6.00	0.0016	1.000000
0.0100	7.00	0.0019	1.000000
0.0100	8.00	0.0022	1.000000
0.0100	9.00	0.0026	1.000000
0.0100	10.00	0.0029	1.000000
0.0100	11.00	0.0033	1.000000
0.0100	12.00	0.0037	1.000000
0.0100	13.00	0.0041	1.000000
0.0100	14.00	0.0046	1.000000
0.0100	15.00	0.0050	1.000000
0.0100	16.00	0.0055	1.000000
0.0100	17.00	0.0061	1.000000
0.0100	18.00	0.0066	1.000000
0.0100	19.00	0.0073	1.000000
0.0100	20.00	0.0079	1.000000
0.0100	21.00	0.0087	1.000000
0.0100	22.00	0.0095	1.000000
0.0100	23.00	0.0104	1.000000
0.0100	24.00	0.0113	1.000000
0.0100	25.00	0.0124	1.000000
0.0100	26.00	0.0137	1.000000
0.0100	27.00	0.0151	1.000000
0.0100	28.00	0.0167	1.000000
0.0100	29.00	0.0186	1.000000
0.0100	30.00	0.0208	1.000000
0.0100	31.00	0.0235	1.000000
0.0100	32.00	0.0267	1.000000
0.0100	33.00	0.0308	1.000000
0.0100	34.00	0.0359	1.000000
0.0100	35.00	0.0425	1.000000
0.0100	36.00	0.0511	1.000000
0.0100	37.00	0.0625	1.000000
0.0100	38.00	0.0773	1.000000
0.0100	39.00	0.0961	1.000000
0.0100	40.00	0.1191	1.000000
0.0100	42.00	0.1761	1.000000
0.0100	44.00	0.2442	1.000000
0.0100	46.00	0.3199	1.000000
0.0100	48.00	0.4015	1.000000
0.0100	50.00	0.4880	1.000000
0.0100	55.00	0.6601	0.702813
0.0100	60.00	0.6601	0.590559
0.0100	65.00	0.6601	0.503198
0.0100	70.00	0.6601	0.433880
0.0100	75.00	0.6601	0.377957
0.0100	80.00	0.6601	0.332189

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0200	5.00	0.0024	1.000000
0.0200	6.00	0.0030	1.000000
0.0200	7.00	0.0036	1.000000
0.0200	8.00	0.0043	1.000000
0.0200	9.00	0.0050	1.000000
0.0200	10.00	0.0057	1.000000
0.0200	11.00	0.0064	1.000000
0.0200	12.00	0.0072	1.000000
0.0200	13.00	0.0080	1.000000
0.0200	14.00	0.0089	1.000000
0.0200	15.00	0.0098	1.000000
0.0200	16.00	0.0107	1.000000
0.0200	17.00	0.0118	1.000000
0.0200	18.00	0.0129	1.000000
0.0200	19.00	0.0141	1.000000
0.0200	20.00	0.0153	1.000000
0.0200	21.00	0.0167	1.000000
0.0200	22.00	0.0182	1.000000
0.0200	23.00	0.0199	1.000000
0.0200	24.00	0.0217	1.000000
0.0200	25.00	0.0237	1.000000
0.0200	26.00	0.0260	1.000000
0.0200	27.00	0.0285	1.000000
0.0200	28.00	0.0314	1.000000
0.0200	29.00	0.0346	1.000000
0.0200	30.00	0.0384	1.000000
0.0200	31.00	0.0428	1.000000
0.0200	32.00	0.0480	1.000000
0.0200	33.00	0.0541	1.000000
0.0200	34.00	0.0615	1.000000
0.0200	35.00	0.0704	1.000000
0.0200	36.00	0.0812	1.000000
0.0200	37.00	0.0943	1.000000
0.0200	38.00	0.1101	1.000000
0.0200	39.00	0.1288	1.000000
0.0200	40.00	0.1507	1.000000
0.0200	42.00	0.2034	1.000000
0.0200	44.00	0.2667	1.000000
0.0200	46.00	0.3384	1.000000
0.0200	48.00	0.4168	1.000000
0.0200	50.00	0.5009	1.000000
0.0200	55.00	0.6535	0.681896
0.0200	60.00	0.6535	0.572982
0.0200	65.00	0.6535	0.488222
0.0200	70.00	0.6535	0.420966
0.0200	75.00	0.6535	0.366709
0.0200	80.00	0.6535	0.322302

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

L/T	H/T	PHY	V
0.0300	5.00	0.0034	1.000000
0.0300	6.00	0.0043	1.000000
0.0300	7.00	0.0052	1.000000
0.0300	8.00	0.0062	1.000000
0.0300	9.00	0.0072	1.000000
0.0300	10.00	0.0082	1.000000
0.0300	11.00	0.0093	1.000000
0.0300	12.00	0.0104	1.000000
0.0300	13.00	0.0116	1.000000
0.0300	14.00	0.0129	1.000000
0.0300	15.00	0.0142	1.000000
0.0300	16.00	0.0156	1.000000
0.0300	17.00	0.0171	1.000000
0.0300	18.00	0.0187	1.000000
0.0300	19.00	0.0204	1.000000
0.0300	20.00	0.0222	1.000000
0.0300	21.00	0.0242	1.000000
0.0300	22.00	0.0264	1.000000
0.0300	23.00	0.0287	1.000000
0.0300	24.00	0.0312	1.000000
0.0300	25.00	0.0340	1.000000
0.0300	26.00	0.0371	1.000000
0.0300	27.00	0.0406	1.000000
0.0300	28.00	0.0445	1.000000
0.0300	29.00	0.0488	1.000000
0.0300	30.00	0.0538	1.000000
0.0300	31.00	0.0595	1.000000
0.0300	32.00	0.0660	1.000000
0.0300	33.00	0.0735	1.000000
0.0300	34.00	0.0823	1.000000
0.0300	35.00	0.0925	1.000000
0.0300	36.00	0.1045	1.000000
0.0300	37.00	0.1186	1.000000
0.0300	38.00	0.1349	1.000000
0.0300	39.00	0.1536	1.000000
0.0300	40.00	0.1750	1.000000
0.0300	42.00	0.2257	1.000000
0.0300	44.00	0.2861	1.000000
0.0300	46.00	0.3550	1.000000
0.0300	48.00	0.4311	1.000000
0.0300	50.00	0.5132	1.000000
0.0300	55.00	0.6471	0.661515
0.0300	60.00	0.6471	0.555857
0.0300	65.00	0.6471	0.473629
0.0300	70.00	0.6471	0.408384
0.0300	75.00	0.6471	0.355748
0.0300	80.00	0.6471	0.312669

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0400	5.00	0.0043	1.000000
0.0400	6.00	0.0055	1.000000
0.0400	7.00	0.0066	1.000000
0.0400	8.00	0.0079	1.000000
0.0400	9.00	0.0092	1.000000
0.0400	10.00	0.0105	1.000000
0.0400	11.00	0.0120	1.000000
0.0400	12.00	0.0134	1.000000
0.0400	13.00	0.0150	1.000000
0.0400	14.00	0.0166	1.000000
0.0400	15.00	0.0183	1.000000
0.0400	16.00	0.0202	1.000000
0.0400	17.00	0.0221	1.000000
0.0400	18.00	0.0241	1.000000
0.0400	19.00	0.0263	1.000000
0.0400	20.00	0.0287	1.000000
0.0400	21.00	0.0312	1.000000
0.0400	22.00	0.0339	1.000000
0.0400	23.00	0.0368	1.000000
0.0400	24.00	0.0401	1.000000
0.0400	25.00	0.0436	1.000000
0.0400	26.00	0.0474	1.000000
0.0400	27.00	0.0517	1.000000
0.0400	28.00	0.0564	1.000000
0.0400	29.00	0.0616	1.000000
0.0400	30.00	0.0675	1.000000
0.0400	31.00	0.0742	1.000000
0.0400	32.00	0.0817	1.000000
0.0400	33.00	0.0903	1.000000
0.0400	34.00	0.1000	1.000000
0.0400	35.00	0.1112	1.000000
0.0400	36.00	0.1240	1.000000
0.0400	37.00	0.1387	1.000000
0.0400	38.00	0.1554	1.000000
0.0400	39.00	0.1742	1.000000
0.0400	40.00	0.1954	1.000000
0.0400	42.00	0.2449	1.000000
0.0400	44.00	0.3035	1.000000
0.0400	46.00	0.3703	1.000000
0.0400	48.00	0.4444	1.000000
0.0400	50.00	0.5249	1.000000
0.0400	55.00	0.6408	0.641666
0.0400	60.00	0.6408	0.539178
0.0400	65.00	0.6408	0.459418
0.0400	70.00	0.6408	0.396131
0.0400	75.00	0.6408	0.345074
0.0400	80.00	0.6408	0.303288

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0500	5.00	0.0051	1.000000
0.0500	6.00	0.0065	1.000000
0.0500	7.00	0.0079	1.000000
0.0500	8.00	0.0095	1.000000
0.0500	9.00	0.0111	1.000000
0.0500	10.00	0.0127	1.000000
0.0500	11.00	0.0144	1.000000
0.0500	12.00	0.0162	1.000000
0.0500	13.00	0.0181	1.000000
0.0500	14.00	0.0201	1.000000
0.0500	15.00	0.0222	1.000000
0.0500	16.00	0.0244	1.000000
0.0500	17.00	0.0268	1.000000
0.0500	18.00	0.0292	1.000000
0.0500	19.00	0.0319	1.000000
0.0500	20.00	0.0347	1.000000
0.0500	21.00	0.0377	1.000000
0.0500	22.00	0.0410	1.000000
0.0500	23.00	0.0445	1.000000
0.0500	24.00	0.0483	1.000000
0.0500	25.00	0.0524	1.000000
0.0500	26.00	0.0569	1.000000
0.0500	27.00	0.0618	1.000000
0.0500	28.00	0.0673	1.000000
0.0500	29.00	0.0733	1.000000
0.0500	30.00	0.0800	1.000000
0.0500	31.00	0.0874	1.000000
0.0500	32.00	0.0958	1.000000
0.0500	33.00	0.1051	1.000000
0.0500	34.00	0.1157	1.000000
0.0500	35.00	0.1276	1.000000
0.0500	36.00	0.1410	1.000000
0.0500	37.00	0.1562	1.000000
0.0500	38.00	0.1731	1.000000
0.0500	39.00	0.1921	1.000000
0.0500	40.00	0.2132	1.000000
0.0500	42.00	0.2619	1.000000
0.0500	44.00	0.3192	1.000000
0.0500	46.00	0.3845	1.000000
0.0500	48.00	0.4570	1.000000
0.0500	50.00	0.5360	1.000000
0.0500	55.00	0.6346	0.622346
0.0500	60.00	0.6346	0.522944
0.0500	65.00	0.6346	0.445585
0.0500	70.00	0.6346	0.384203
0.0500	75.00	0.6346	0.334684
0.0500	80.00	0.6346	0.294156

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0600	5.00	0.0057	1.000000
0.0600	6.00	0.0074	1.000000
0.0600	7.00	0.0091	1.000000
0.0600	8.00	0.0109	1.000000
0.0600	9.00	0.0128	1.000000
0.0600	10.00	0.0147	1.000000
0.0600	11.00	0.0167	1.000000
0.0600	12.00	0.0189	1.000000
0.0600	13.00	0.0211	1.000000
0.0600	14.00	0.0234	1.000000
0.0600	15.00	0.0259	1.000000
0.0600	16.00	0.0284	1.000000
0.0600	17.00	0.0312	1.000000
0.0600	18.00	0.0340	1.000000
0.0600	19.00	0.0371	1.000000
0.0600	20.00	0.0404	1.000000
0.0600	21.00	0.0438	1.000000
0.0600	22.00	0.0476	1.000000
0.0600	23.00	0.0516	1.000000
0.0600	24.00	0.0559	1.000000
0.0600	25.00	0.0606	1.000000
0.0600	26.00	0.0657	1.000000
0.0600	27.00	0.0712	1.000000
0.0600	28.00	0.0773	1.000000
0.0600	29.00	0.0840	1.000000
0.0600	30.00	0.0914	1.000000
0.0600	31.00	0.0995	1.000000
0.0600	32.00	0.1085	1.000000
0.0600	33.00	0.1186	1.000000
0.0600	34.00	0.1298	1.000000
0.0600	35.00	0.1423	1.000000
0.0600	36.00	0.1562	1.000000
0.0600	37.00	0.1717	1.000000
0.0600	38.00	0.1890	1.000000
0.0600	39.00	0.2081	1.000000
0.0600	40.00	0.2291	1.000000
0.0600	42.00	0.2773	1.000000
0.0600	44.00	0.3336	1.000000
0.0600	46.00	0.3977	1.000000
0.0600	48.00	0.4690	1.000000
0.0600	50.00	0.5467	1.000000
0.0600	55.00	0.6286	0.603549
0.0600	60.00	0.6286	0.507149
0.0600	65.00	0.6286	0.432127
0.0600	70.00	0.6286	0.372599
0.0600	75.00	0.6286	0.324575
0.0600	80.00	0.6286	0.285271

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0700	5.00	0.0063	1.000000
0.0700	6.00	0.0082	1.000000
0.0700	7.00	0.0102	1.000000
0.0700	8.00	0.0122	1.000000
0.0700	9.00	0.0143	1.000000
0.0700	10.00	0.0166	1.000000
0.0700	11.00	0.0189	1.000000
0.0700	12.00	0.0213	1.000000
0.0700	13.00	0.0238	1.000000
0.0700	14.00	0.0265	1.000000
0.0700	15.00	0.0293	1.000000
0.0700	16.00	0.0322	1.000000
0.0700	17.00	0.0353	1.000000
0.0700	18.00	0.0386	1.000000
0.0700	19.00	0.0420	1.000000
0.0700	20.00	0.0457	1.000000
0.0700	21.00	0.0496	1.000000
0.0700	22.00	0.0538	1.000000
0.0700	23.00	0.0583	1.000000
0.0700	24.00	0.0631	1.000000
0.0700	25.00	0.0683	1.000000
0.0700	26.00	0.0739	1.000000
0.0700	27.00	0.0800	1.000000
0.0700	28.00	0.0867	1.000000
0.0700	29.00	0.0939	1.000000
0.0700	30.00	0.1019	1.000000
0.0700	31.00	0.1106	1.000000
0.0700	32.00	0.1202	1.000000
0.0700	33.00	0.1308	1.000000
0.0700	34.00	0.1426	1.000000
0.0700	35.00	0.1556	1.000000
0.0700	36.00	0.1699	1.000000
0.0700	37.00	0.1858	1.000000
0.0700	38.00	0.2033	1.000000
0.0700	39.00	0.2225	1.000000
0.0700	40.00	0.2436	1.000000
0.0700	42.00	0.2914	1.000000
0.0700	44.00	0.3470	1.000000
0.0700	46.00	0.4101	1.000000
0.0700	48.00	0.4803	1.000000
0.0700	50.00	0.5570	1.000000
0.0700	55.00	0.6226	0.585271
0.0700	60.00	0.6226	0.491791
0.0700	65.00	0.6226	0.419040
0.0700	70.00	0.6226	0.361315
0.0700	75.00	0.6226	0.314746
0.0700	80.00	0.6226	0.276632

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0800	5.00	0.0068	1.000000
0.0800	6.00	0.0089	1.000000
0.0800	7.00	0.0111	1.000000
0.0800	8.00	0.0134	1.000000
0.0800	9.00	0.0158	1.000000
0.0800	10.00	0.0183	1.000000
0.0800	11.00	0.0209	1.000000
0.0800	12.00	0.0236	1.000000
0.0800	13.00	0.0264	1.000000
0.0800	14.00	0.0294	1.000000
0.0800	15.00	0.0325	1.000000
0.0800	16.00	0.0358	1.000000
0.0800	17.00	0.0392	1.000000
0.0800	18.00	0.0428	1.000000
0.0800	19.00	0.0467	1.000000
0.0800	20.00	0.0507	1.000000
0.0800	21.00	0.0550	1.000000
0.0800	22.00	0.0597	1.000000
0.0800	23.00	0.0646	1.000000
0.0800	24.00	0.0699	1.000000
0.0800	25.00	0.0755	1.000000
0.0800	26.00	0.0816	1.000000
0.0800	27.00	0.0882	1.000000
0.0800	28.00	0.0954	1.000000
0.0800	29.00	0.1032	1.000000
0.0800	30.00	0.1117	1.000000
0.0800	31.00	0.1209	1.000000
0.0800	32.00	0.1311	1.000000
0.0800	33.00	0.1422	1.000000
0.0800	34.00	0.1544	1.000000
0.0800	35.00	0.1678	1.000000
0.0800	36.00	0.1825	1.000000
0.0800	37.00	0.1987	1.000000
0.0800	38.00	0.2164	1.000000
0.0800	39.00	0.2358	1.000000
0.0800	40.00	0.2569	1.000000
0.0800	42.00	0.3045	1.000000
0.0800	44.00	0.3595	1.000000
0.0800	46.00	0.4218	1.000000
0.0800	48.00	0.4911	1.000000
0.0800	50.00	0.5668	1.000000
0.0800	55.00	0.6168	0.567508
0.0800	60.00	0.6168	0.476865
0.0800	65.00	0.6168	0.406322
0.0800	70.00	0.6168	0.350349
0.0800	75.00	0.6168	0.305193
0.0800	80.00	0.6168	0.268236

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.0900	5.00	0.0073	1.000000
0.0900	6.00	0.0096	1.000000
0.0900	7.00	0.0120	1.000000
0.0900	8.00	0.0145	1.000000
0.0900	9.00	0.0171	1.000000
0.0900	10.00	0.0199	1.000000
0.0900	11.00	0.0227	1.000000
0.0900	12.00	0.0257	1.000000
0.0900	13.00	0.0288	1.000000
0.0900	14.00	0.0321	1.000000
0.0900	15.00	0.0355	1.000000
0.0900	16.00	0.0391	1.000000
0.0900	17.00	0.0429	1.000000
0.0900	18.00	0.0468	1.000000
0.0900	19.00	0.0510	1.000000
0.0900	20.00	0.0555	1.000000
0.0900	21.00	0.0602	1.000000
0.0900	22.00	0.0652	1.000000
0.0900	23.00	0.0705	1.000000
0.0900	24.00	0.0762	1.000000
0.0900	25.00	0.0823	1.000000
0.0900	26.00	0.0889	1.000000
0.0900	27.00	0.0960	1.000000
0.0900	28.00	0.1036	1.000000
0.0900	29.00	0.1118	1.000000
0.0900	30.00	0.1208	1.000000
0.0900	31.00	0.1305	1.000000
0.0900	32.00	0.1411	1.000000
0.0900	33.00	0.1527	1.000000
0.0900	34.00	0.1653	1.000000
0.0900	35.00	0.1791	1.000000
0.0900	36.00	0.1942	1.000000
0.0900	37.00	0.2106	1.000000
0.0900	38.00	0.2286	1.000000
0.0900	39.00	0.2481	1.000000
0.0900	40.00	0.2692	1.000000
0.0900	42.00	0.3167	1.000000
0.0900	44.00	0.3712	1.000000
0.0900	46.00	0.4329	1.000000
0.0900	48.00	0.5014	1.000000
0.0900	50.00	0.5763	1.000000
0.0900	55.00	0.6111	0.550253
0.0900	60.00	0.6111	0.462366
0.0900	65.00	0.6111	0.393968
0.0900	70.00	0.6111	0.339697
0.0900	75.00	0.6111	0.295914
0.0900	80.00	0.6111	0.260081

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1000	5.00	0.0077	1.000000
0.1000	6.00	0.0101	1.000000
0.1000	7.00	0.0128	1.000000
0.1000	8.00	0.0155	1.000000
0.1000	9.00	0.0184	1.000000
0.1000	10.00	0.0214	1.000000
0.1000	11.00	0.0245	1.000000
0.1000	12.00	0.0277	1.000000
0.1000	13.00	0.0311	1.000000
0.1000	14.00	0.0347	1.000000
0.1000	15.00	0.0384	1.000000
0.1000	16.00	0.0423	1.000000
0.1000	17.00	0.0464	1.000000
0.1000	18.00	0.0507	1.000000
0.1000	19.00	0.0552	1.000000
0.1000	20.00	0.0600	1.000000
0.1000	21.00	0.0651	1.000000
0.1000	22.00	0.0704	1.000000
0.1000	23.00	0.0762	1.000000
0.1000	24.00	0.0823	1.000000
0.1000	25.00	0.0888	1.000000
0.1000	26.00	0.0957	1.000000
0.1000	27.00	0.1032	1.000000
0.1000	28.00	0.1113	1.000000
0.1000	29.00	0.1200	1.000000
0.1000	30.00	0.1294	1.000000
0.1000	31.00	0.1396	1.000000
0.1000	32.00	0.1506	1.000000
0.1000	33.00	0.1625	1.000000
0.1000	34.00	0.1756	1.000000
0.1000	35.00	0.1897	1.000000
0.1000	36.00	0.2051	1.000000
0.1000	37.00	0.2218	1.000000
0.1000	38.00	0.2399	1.000000
0.1000	39.00	0.2595	1.000000
0.1000	40.00	0.2807	1.000000
0.1000	42.00	0.3281	1.000000
0.1000	44.00	0.3823	1.000000
0.1000	46.00	0.4434	1.000000
0.1000	48.00	0.5112	1.000000
0.1000	50.00	0.5855	1.000000
0.1000	55.00	0.6055	0.533502
0.1000	60.00	0.6055	0.448290
0.1000	65.00	0.6055	0.381975
0.1000	70.00	0.6055	0.329356
0.1000	75.00	0.6055	0.286906
0.1000	80.00	0.6055	0.252163

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1100	5.00	0.0080	1.000000
0.1100	6.00	0.0107	1.000000
0.1100	7.00	0.0135	1.000000
0.1100	8.00	0.0164	1.000000
0.1100	9.00	0.0195	1.000000
0.1100	10.00	0.0227	1.000000
0.1100	11.00	0.0261	1.000000
0.1100	12.00	0.0296	1.000000
0.1100	13.00	0.0333	1.000000
0.1100	14.00	0.0371	1.000000
0.1100	15.00	0.0411	1.000000
0.1100	16.00	0.0453	1.000000
0.1100	17.00	0.0497	1.000000
0.1100	18.00	0.0543	1.000000
0.1100	19.00	0.0591	1.000000
0.1100	20.00	0.0643	1.000000
0.1100	21.00	0.0697	1.000000
0.1100	22.00	0.0754	1.000000
0.1100	23.00	0.0815	1.000000
0.1100	24.00	0.0880	1.000000
0.1100	25.00	0.0949	1.000000
0.1100	26.00	0.1022	1.000000
0.1100	27.00	0.1101	1.000000
0.1100	28.00	0.1186	1.000000
0.1100	29.00	0.1277	1.000000
0.1100	30.00	0.1375	1.000000
0.1100	31.00	0.1480	1.000000
0.1100	32.00	0.1594	1.000000
0.1100	33.00	0.1718	1.000000
0.1100	34.00	0.1851	1.000000
0.1100	35.00	0.1996	1.000000
0.1100	36.00	0.2152	1.000000
0.1100	37.00	0.2322	1.000000
0.1100	38.00	0.2505	1.000000
0.1100	39.00	0.2702	1.000000
0.1100	40.00	0.2915	1.000000
0.1100	42.00	0.3389	1.000000
0.1100	44.00	0.3928	1.000000
0.1100	46.00	0.4535	1.000000
0.1100	48.00	0.5207	1.000000
0.1100	50.00	0.5943	1.000000
0.1100	55.00	0.6001	0.517248
0.1100	60.00	0.6001	0.434632
0.1100	65.00	0.6001	0.370337
0.1100	70.00	0.6001	0.319321
0.1100	75.00	0.6001	0.278164
0.1100	80.00	0.6001	0.244480

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1200	5.00	0.0083	1.000000
0.1200	6.00	0.0111	1.000000
0.1200	7.00	0.0141	1.000000
0.1200	8.00	0.0173	1.000000
0.1200	9.00	0.0206	1.000000
0.1200	10.00	0.0240	1.000000
0.1200	11.00	0.0276	1.000000
0.1200	12.00	0.0314	1.000000
0.1200	13.00	0.0353	1.000000
0.1200	14.00	0.0394	1.000000
0.1200	15.00	0.0436	1.000000
0.1200	16.00	0.0481	1.000000
0.1200	17.00	0.0528	1.000000
0.1200	18.00	0.0574	1.000000
0.1200	19.00	0.0629	1.000000
0.1200	20.00	0.0683	1.000000
0.1200	21.00	0.0741	1.000000
0.1200	22.00	0.0801	1.000000
0.1200	23.00	0.0866	1.000000
0.1200	24.00	0.0934	0.996679
0.1200	25.00	0.1006	0.991016
0.1200	26.00	0.1084	0.985845
0.1200	27.00	0.1166	0.981105
0.1200	28.00	0.1255	0.981105
0.1200	29.00	0.1350	0.981105
0.1200	30.00	0.1451	0.981105
0.1200	31.00	0.1561	0.981105
0.1200	32.00	0.1678	0.981105
0.1200	33.00	0.1805	0.981105
0.1200	34.00	0.1942	0.981105
0.1200	35.00	0.2089	0.981105
0.1200	36.00	0.2248	0.981105
0.1200	37.00	0.2420	0.981105
0.1200	38.00	0.2605	0.981105
0.1200	39.00	0.2804	0.981105
0.1200	40.00	0.3017	0.981105
0.1200	42.00	0.3491	0.981105
0.1200	44.00	0.4028	0.981105
0.1200	46.00	0.4631	0.981105
0.1200	48.00	0.5298	0.981105
0.1200	50.00	0.5947	0.606797
0.1200	55.00	0.5947	0.501485
0.1200	60.00	0.5947	0.421387
0.1200	65.00	0.5947	0.359052
0.1200	70.00	0.5947	0.309590
0.1200	75.00	0.5947	0.269688
0.1200	80.00	0.5947	0.237030

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1300	5.00	0.0086	1.000000
0.1300	6.00	0.0115	1.000000
0.1300	7.00	0.0147	1.000000
0.1300	8.00	0.0180	1.000000
0.1300	9.00	0.0215	1.000000
0.1300	10.00	0.0252	1.000000
0.1300	11.00	0.0290	1.000000
0.1300	12.00	0.0330	1.000000
0.1300	13.00	0.0372	1.000000
0.1300	14.00	0.0415	1.000000
0.1300	15.00	0.0460	1.000000
0.1300	16.00	0.0508	1.000000
0.1300	17.00	0.0557	1.000000
0.1300	18.00	0.0609	1.000000
0.1300	19.00	0.0664	1.000000
0.1300	20.00	0.0722	0.991408
0.1300	21.00	0.0782	0.983147
0.1300	22.00	0.0846	0.975755
0.1300	23.00	0.0914	0.969102
0.1300	24.00	0.0985	0.963082
0.1300	25.00	0.1061	0.957610
0.1300	26.00	0.1142	0.952614
0.1300	27.00	0.1228	0.948034
0.1300	28.00	0.1320	0.948034
0.1300	29.00	0.1419	0.948034
0.1300	30.00	0.1524	0.948034
0.1300	31.00	0.1636	0.948034
0.1300	32.00	0.1757	0.948034
0.1300	33.00	0.1887	0.948034
0.1300	34.00	0.2027	0.948034
0.1300	35.00	0.2177	0.948034
0.1300	36.00	0.2339	0.948034
0.1300	37.00	0.2513	0.948034
0.1300	38.00	0.2699	0.948034
0.1300	39.00	0.2899	0.948034
0.1300	40.00	0.3114	0.948034
0.1300	42.00	0.3587	0.948034
0.1300	44.00	0.4123	0.948034
0.1300	46.00	0.4723	0.948034
0.1300	48.00	0.5386	0.948034
0.1300	50.00	0.5896	0.588311
0.1300	55.00	0.5896	0.486207
0.1300	60.00	0.5896	0.408549
0.1300	65.00	0.5896	0.348113
0.1300	70.00	0.5896	0.300159
0.1300	75.00	0.5896	0.261471
0.1300	80.00	0.5896	0.229809

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1400	5.00	0.0088	1.000000
0.1400	6.00	0.0119	1.000000
0.1400	7.00	0.0152	1.000000
0.1400	8.00	0.0187	1.000000
0.1400	9.00	0.0224	1.000000
0.1400	10.00	0.0263	1.000000
0.1400	11.00	0.0303	1.000000
0.1400	12.00	0.0345	1.000000
0.1400	13.00	0.0389	1.000000
0.1400	14.00	0.0435	1.000000
0.1400	15.00	0.0483	1.000000
0.1400	16.00	0.0533	1.000000
0.1400	17.00	0.0586	0.989907
0.1400	18.00	0.0640	0.978261
0.1400	19.00	0.0698	0.968071
0.1400	20.00	0.0758	0.959080
0.1400	21.00	0.0822	0.951088
0.1400	22.00	0.0889	0.943937
0.1400	23.00	0.0960	0.937501
0.1400	24.00	0.1035	0.931678
0.1400	25.00	0.1114	0.926384
0.1400	26.00	0.1198	0.921551
0.1400	27.00	0.1287	0.917120
0.1400	28.00	0.1383	0.913044
0.1400	29.00	0.1484	0.913044
0.1400	30.00	0.1593	0.913044
0.1400	31.00	0.1709	0.913044
0.1400	32.00	0.1833	0.913044
0.1400	33.00	0.1966	0.913044
0.1400	34.00	0.2108	0.913044
0.1400	35.00	0.2261	0.913044
0.1400	36.00	0.2425	0.913044
0.1400	37.00	0.2601	0.913044
0.1400	38.00	0.2789	0.913044
0.1400	39.00	0.2990	0.913044
0.1400	40.00	0.3206	0.913044
0.1400	42.00	0.3680	0.913044
0.1400	44.00	0.4214	0.913044
0.1400	46.00	0.4811	0.913044
0.1400	48.00	0.5470	0.913044
0.1400	50.00	0.5845	0.570402
0.1400	55.00	0.5845	0.471407
0.1400	60.00	0.5845	0.396113
0.1400	65.00	0.5845	0.337516
0.1400	70.00	0.5845	0.291022
0.1400	75.00	0.5845	0.253512
0.1400	80.00	0.5845	0.222813

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1500	5.00	0.0091	1.000000
0.1500	6.00	0.0123	1.000000
0.1500	7.00	0.0157	1.000000
0.1500	8.00	0.0194	1.000000
0.1500	9.00	0.0233	1.000000
0.1500	10.00	0.0273	1.000000
0.1500	11.00	0.0316	1.000000
0.1500	12.00	0.0360	1.000000
0.1500	13.00	0.0406	1.000000
0.1500	14.00	0.0454	1.000000
0.1500	15.00	0.0505	0.986842
0.1500	16.00	0.0557	0.971660
0.1500	17.00	0.0612	0.958647
0.1500	18.00	0.0670	0.947369
0.1500	19.00	0.0730	0.937500
0.1500	20.00	0.0793	0.928793
0.1500	21.00	0.0860	0.921053
0.1500	22.00	0.0930	0.914128
0.1500	23.00	0.1003	0.907895
0.1500	24.00	0.1081	0.902256
0.1500	25.00	0.1164	0.897129
0.1500	26.00	0.1251	0.892449
0.1500	27.00	0.1344	0.888158
0.1500	28.00	0.1442	0.884211
0.1500	29.00	0.1547	0.884211
0.1500	30.00	0.1658	0.884211
0.1500	31.00	0.1777	0.884211
0.1500	32.00	0.1904	0.884211
0.1500	33.00	0.2040	0.884211
0.1500	34.00	0.2185	0.884211
0.1500	35.00	0.2341	0.884211
0.1500	36.00	0.2507	0.884211
0.1500	37.00	0.2684	0.884211
0.1500	38.00	0.2874	0.884211
0.1500	39.00	0.3077	0.884211
0.1500	40.00	0.3293	0.884211
0.1500	42.00	0.3768	0.884211
0.1500	44.00	0.4302	0.884211
0.1500	46.00	0.4896	0.884211
0.1500	48.00	0.5552	0.884211
0.1500	50.00	0.5796	0.553064
0.1500	55.00	0.5796	0.457077
0.1500	60.00	0.5796	0.384072
0.1500	65.00	0.5796	0.327257
0.1500	70.00	0.5796	0.282175
0.1500	75.00	0.5796	0.245806
0.1500	80.00	0.5796	0.216040

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1600	5.00	0.0092	1.000000
0.1600	6.00	0.0126	1.000000
0.1600	7.00	0.0162	1.000000
0.1600	8.00	0.0200	1.000000
0.1600	9.00	0.0240	1.000000
0.1600	10.00	0.0283	1.000000
0.1600	11.00	0.0327	1.000000
0.1600	12.00	0.0374	1.000000
0.1600	13.00	0.0422	0.994898
0.1600	14.00	0.0473	0.974026
0.1600	15.00	0.0525	0.956633
0.1600	16.00	0.0580	0.941916
0.1600	17.00	0.0638	0.929301
0.1600	18.00	0.0698	0.918368
0.1600	19.00	0.0761	0.908801
0.1600	20.00	0.0827	0.900361
0.1600	21.00	0.0896	0.892858
0.1600	22.00	0.0969	0.886145
0.1600	23.00	0.1045	0.880102
0.1600	24.00	0.1126	0.874636
0.1600	25.00	0.1211	0.869667
0.1600	26.00	0.1302	0.865129
0.1600	27.00	0.1397	0.860970
0.1600	28.00	0.1499	0.857143
0.1600	29.00	0.1606	0.857143
0.1600	30.00	0.1721	0.857143
0.1600	31.00	0.1843	0.857143
0.1600	32.00	0.1973	0.857143
0.1600	33.00	0.2111	0.857143
0.1600	34.00	0.2259	0.857143
0.1600	35.00	0.2416	0.857143
0.1600	36.00	0.2684	0.857143
0.1600	37.00	0.2974	0.857143
0.1600	38.00	0.3177	0.857143
0.1600	39.00	0.3393	0.857143
0.1600	40.00	0.3868	0.857143
0.1600	42.00	0.4402	0.857143
0.1600	44.00	0.4996	0.857143
0.1600	46.00	0.5654	0.857143
0.1600	48.00	0.5748	0.581907
0.1600	50.00	0.5748	0.536285
0.1600	55.00	0.5748	0.443211
0.1600	60.00	0.5748	0.372421
0.1600	65.00	0.5748	0.317329
0.1600	70.00	0.5748	0.273615
0.1600	75.00	0.5748	0.238349
0.1600	80.00	0.5748	0.209486

TOP

93 KBYTES USED JOB-STEP RETURN CODE = 0

.END JOB 4532 12:34 20 JUN 79 A.AWNI,J.C.M.B.

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
C         STRESS REDUCTION FACTORS
C          WALLS IN DOUBLE CURVATURE
C         PROGRAMMER: ADNAN A. AWNI
C
          REAL E(18)
          REAL SLEND(47)
      001 FORMAT(9F8.4)
      002 FORMAT(10F8.3)
      003 FORMAT('1',// T21,'FINAL     COMPUTED     RESULTS'/)
      004 FORMAT(    22X,'WALLS IN DOUBLE CURVATURE')
      005 FORMAT(    /13X,'E/T',12X,'H/T',10X,'PHY',13X,'V'/)
      006 FORMAT(    12X,F6.4,8X,F6.2,8X,F6.4,8X,F8.6)
      007 FORMAT(    23X,'STRESS REDUCTION FACTOR'/)
      008 READ(5,1) (E(I),I=1,18)
      009 READ(5,2) (SLEND(J),J=1,47)
      010 DO 260 I=1,18
      011 DO 260 J=1,47
      012 REALK=((SLEND(J))/(2.0))**2*((1.0)/(55.5))
      013 IF(SLEND(J)-10)55,14,14
      014 A=16.655
      015 B=33.31*E(I)-(33.31)
      016 IF(B)17,19,19
      017 BPOST=-(B)
      018 GOTO 20
      019 BPOST=(B)
      020 C=(16.655*(E(I))**2)+((9.0)/(8.0))*REALK
      030 XROOT=(BPOST**2)-(4.0*(A)*(C))
      031 IF(XROOT)160,32,32
      032 PHY1=(-(B)+(BPOST**2-4.0*(A)*(C))**0.5)/(2.*(A))
      033 PHY2=(-(B)-(BPOST**2-4.0*(A)*(C))**0.5)/(2.*(A))
      034 PHYB=((1.-E(I))+(4.*E(I)**2-2.*E(I)+1.0)
         1**0.5)/(3.0)
      035 IF(PHY1)70,36,36
      036 IF(PHY2)37,37,39
      037 PHY2=0
      038 GOTO 42
      039 PHY2=PHY2
      040 PHYMIN=(PHY1-PHY2)
      041 IF(PHYMIN)42,42,51
      042 PHY1=PHY1
      050 GOTO 57
      051 PHY1=PHY2
      052 IF(PHY1)53,57,57
      053 PHY1=-PHY1
      054 GOTO 57
      055 PHY1=(9.*REALK*(SLEND(J)/2.))/((8.*((SLEND(J)/2.)-1.5))
         1  *33.31*(1.-2.*E(I)))
      056 PHYB=((1.0-E(I))+(4.0*E(I)**2-2.0*E(I)+1.0)**0.5)/(3.0)
      057 CHECK1=(PHY1-PHYB)
      058 IF(CHECK1)59,70,70
      059 PHY=PHY1
      060 GOTO 72
      070 PHY=PHYB
      071 GOTO 170
C
C
C
C
C
```

```
      C
      C
      C
      072 CHECK2=(PHY-E(I))
      073 IF(CHECK2)74,74,90
      074 V1=(((9.0)/(8.0))*(1.0-2.0*E(I))*((SLEND(J))/(2.0)))
         2/((SLEND(J)/(2.0))-(1.5))
      075 VMAX=1.0
      076 IF(V1-VMAX)77,77,79
      077 V=V1
      078 GOTO 200
      079 V=VMAX
      080 GOTO 200
      090 V2=((9.0)/(8.0))*((1.0-((E(I)+PHY)**2)/(2.0*PHY)))
      100 VMAX=1.0
      110 IF(V2-VMAX)120,120,140
      120 V=V2
      130 GOTO 200
      140 V=VMAX
      150 GOTO 200
      160 PHY=((1.0-E(I))+(4.*E(I)**2-2.*E(I)+1.0)**0.5)/(3.0)
      170 CHECK3=(PHY-E(I))
      180 IF(CHECK3)190,192,192
      190 Z=(33.31*PHY)*((1.-2.*E(I))**2)
      191 GOTO 193
      192 Z=33.31*PHY*(1.-((E(I)+PHY)**2)/(2.*PHY))**2
      193 V3=Z/REALK
      194 VMAX=1.0
      195 CHECK4=(V3-VMAX)
      196 IF(CHECK4)197,197,199
      197 V=V3
      198 GOTO 200
      199 V=VMAX
      200 IF(SLEND(J)-5.0)210,210,250
      210 WRITE(6,3)
      220 WRITE(6,7)
      230 WRITE(6,4)
      240 WRITE(6,5)
      250 WRITE(6,6) (E(I),SLEND(J),PHY,V)
      260 CONTINUE
      270 STOP
      280 END
```

```
      C      END OF PROGRAM
      C      INPUT DATA FOR ABOVE PROGRAM :
      C      ECCENTRICITY/WALL THICKNESS 'E/T' VALUES :
      C  .1666   .18    920     .22     .24     .26     .28     .30
      C  .34     .36    .38     .40     .42     .44     .46     .48
      C       SLENDERNESS RATIO 'H/T' VALUES :
      C  5.0    6.0    7.0    8.0    9.0    10.0   11.0   12.0   13.0   1
      C 15.0   16.0   17.0   18.0   19.0   20.0   21.0   22.0   23.0   2
      C 25.0   26.0   27.0   28.0   29.0   30.0   31.0   32.0   33.0   3
      C 35.0   36.0   37.0   38.0   39.0   40.0   42.0   44.0   46.0   4
      C 50.0   52.0   54.0   56.0   58.0   60.0   65.0   70.0   75.0   8
      C      END OF INPUT DATA.

CODE+GLA+SYMTABS+ARRAYS = 3664+ 680+ 296+ 264= 4904 BYTES

*COMPILATION SUCCESSFUL
```

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1666	5.00	0.0143	1.000000
0.1666	6.00	0.0164	1.000000
0.1666	7.00	0.0196	1.000000
0.1666	8.00	0.0234	1.000000
0.1666	9.00	0.0277	1.000000
0.1666	10.00	0.0357	1.000000
0.1666	11.00	0.0397	1.000000
0.1666	12.00	0.0441	1.000000
0.1666	13.00	0.0489	0.975195
0.1666	14.00	0.0542	0.954736
0.1666	15.00	0.0599	0.937687
0.1666	16.00	0.0660	0.923261
0.1666	17.00	0.0726	0.910896
0.1666	18.00	0.0796	0.900180
0.1666	19.00	0.0871	0.890803
0.1666	20.00	0.0951	0.882529
0.1666	21.00	0.1036	0.875175
0.1666	22.00	0.1126	0.868595
0.1666	23.00	0.1222	0.862673
0.1666	24.00	0.1323	0.857314
0.1666	25.00	0.1430	0.852443
0.1666	26.00	0.1543	0.847996
0.1666	27.00	0.1663	0.843919
0.1666	28.00	0.1790	0.749668
0.1666	29.00	0.1924	0.748207
0.1666	30.00	0.2065	0.745807
0.1666	31.00	0.2215	0.742491
0.1666	32.00	0.2374	0.738276
0.1666	33.00	0.2542	0.733164
0.1666	34.00	0.2721	0.727144
0.1666	35.00	0.2911	0.720193
0.1666	36.00	0.3114	0.712271
0.1666	37.00	0.3331	0.703318
0.1666	38.00	0.3565	0.693252
0.1666	39.00	0.3817	0.681955
0.1666	40.00	0.4092	0.669261
0.1666	42.00	0.4727	0.638631
0.1666	44.00	0.5546	0.597470
0.1666	46.00	0.5718	0.547067
0.1666	48.00	0.5718	0.502428
0.1666	50.00	0.5718	0.463037
0.1666	55.00	0.5718	0.382676
0.1666	60.00	0.5718	0.321554
0.1666	65.00	0.5718	0.273987
0.1666	70.00	0.5718	0.236244
0.1666	75.00	0.5718	0.205794
0.1666	80.00	0.5718	0.180874

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.1800	5.00	0.0149	1.000000
0.1800	6.00	0.0171	1.000000
0.1800	7.00	0.0204	1.000000
0.1800	8.00	0.0243	1.000000
0.1800	9.00	0.0289	1.000000
0.1800	10.00	0.0392	1.000000
0.1800	11.00	0.0434	0.990000
0.1800	12.00	0.0479	0.960000
0.1800	13.00	0.0528	0.936000
0.1800	14.00	0.0582	0.916364
0.1800	15.00	0.0640	0.900000
0.1800	16.00	0.0703	0.886154
0.1800	17.00	0.0770	0.874286
0.1800	18.00	0.0842	0.864000
0.1800	19.00	0.0919	0.855000
0.1800	20.00	0.1001	0.847059
0.1800	21.00	0.1088	0.840000
0.1800	22.00	0.1180	0.833684
0.1800	23.00	0.1279	0.828000
0.1800	24.00	0.1383	0.822857
0.1800	25.00	0.1493	0.818182
0.1800	26.00	0.1610	0.813913
0.1800	27.00	0.1733	0.810000
0.1800	28.00	0.1864	0.719877
0.1800	29.00	0.2002	0.718850
0.1800	30.00	0.2149	0.716814
0.1800	31.00	0.2304	0.713793
0.1800	32.00	0.2469	0.709801
0.1800	33.00	0.2644	0.704835
0.1800	34.00	0.2831	0.698882
0.1800	35.00	0.3030	0.691903
0.1800	36.00	0.3243	0.683864
0.1800	37.00	0.3473	0.674673
0.1800	38.00	0.3721	0.664227
0.1800	39.00	0.3990	0.652369
0.1800	40.00	0.4286	0.638878
0.1800	42.00	0.4986	0.605466
0.1800	44.00	0.5658	0.558743
0.1800	46.00	0.5658	0.511213
0.1800	48.00	0.5658	0.469499
0.1800	50.00	0.5658	0.432690
0.1800	55.00	0.5658	0.357595
0.1800	60.00	0.5658	0.300480
0.1800	65.00	0.5658	0.256030
0.1800	70.00	0.5658	0.220760
0.1800	75.00	0.5658	0.192307
0.1800	80.00	0.5658	0.169020

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.2000	5.00	0.0158	1.000000
0.2000	6.00	0.0183	1.000000
0.2000	7.00	0.0217	1.000000
0.2000	8.00	0.0260	1.000000
0.2000	9.00	0.0308	1.000000
0.2000	10.00	0.0453	0.964286
0.2000	11.00	0.0495	0.928125
0.2000	12.00	0.0542	0.900000
0.2000	13.00	0.0593	0.877500
0.2000	14.00	0.0649	0.859091
0.2000	15.00	0.0709	0.843750
0.2000	16.00	0.0774	0.830769
0.2000	17.00	0.0844	0.819643
0.2000	18.00	0.0919	0.810000
0.2000	19.00	0.0999	0.801562
0.2000	20.00	0.1084	0.794118
0.2000	21.00	0.1175	0.787500
0.2000	22.00	0.1271	0.781579
0.2000	23.00	0.1374	0.776250
0.2000	24.00	0.1483	0.771428
0.2000	25.00	0.1598	0.767045
0.2000	26.00	0.1721	0.763043
0.2000	27.00	0.1850	0.759375
0.2000	28.00	0.1988	0.756000
0.2000	29.00	0.2134	0.674527
0.2000	30.00	0.2289	0.672948
0.2000	31.00	0.2454	0.670279
0.2000	32.00	0.2629	0.666525
0.2000	33.00	0.2817	0.661677
0.2000	34.00	0.3017	0.655705
0.2000	35.00	0.3233	0.648560
0.2000	36.00	0.3465	0.640162
0.2000	37.00	0.3717	0.630394
0.2000	38.00	0.3992	0.619086
0.2000	39.00	0.4296	0.605984
0.2000	40.00	0.4636	0.590697
0.2000	42.00	0.5485	0.550468
0.2000	44.00	0.5573	0.501675
0.2000	46.00	0.5573	0.458999
0.2000	48.00	0.5573	0.421546
0.2000	50.00	0.5573	0.388497
0.2000	55.00	0.5573	0.321072
0.2000	60.00	0.5573	0.269790
0.2000	65.00	0.5573	0.229880
0.2000	70.00	0.5573	0.198213
0.2000	75.00	0.5573	0.172665
0.2000	80.00	0.5573	0.151757

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.2200	5.00	0.0170	1.000000
0.2200	6.00	0.0196	1.000000
0.2200	7.00	0.0233	1.000000
0.2200	8.00	0.0278	1.000000
0.2200	9.00	0.0330	0.945000
0.2200	10.00	0.0523	0.900000
0.2200	11.00	0.0567	0.866250
0.2200	12.00	0.0615	0.840000
0.2200	13.00	0.0669	0.819000
0.2200	14.00	0.0726	0.801818
0.2200	15.00	0.0789	0.787500
0.2200	16.00	0.0857	0.775385
0.2200	17.00	0.0929	0.765000
0.2200	18.00	0.1007	0.756000
0.2200	19.00	0.1091	0.748125
0.2200	20.00	0.1180	0.741176
0.2200	21.00	0.1275	0.735000
0.2200	22.00	0.1376	0.729474
0.2200	23.00	0.1483	0.724500
0.2200	24.00	0.1597	0.720000
0.2200	25.00	0.1719	0.715909
0.2200	26.00	0.1848	0.712174
0.2200	27.00	0.1985	0.708750
0.2200	28.00	0.2130	0.705600
0.2200	29.00	0.2285	0.629821
0.2200	30.00	0.2451	0.628559
0.2200	31.00	0.2627	0.626096
0.2200	32.00	0.2816	0.622427
0.2200	33.00	0.3018	0.617523
0.2200	34.00	0.3236	0.611332
0.2200	35.00	0.3472	0.603770
0.2200	36.00	0.3730	0.594708
0.2200	37.00	0.4012	0.583950
0.2200	38.00	0.4327	0.571200
0.2200	39.00	0.4682	0.555983
0.2200	40.00	0.5095	0.537474
0.2200	42.00	0.5494	0.489998
0.2200	44.00	0.5494	0.446465
0.2200	46.00	0.5494	0.408486
0.2200	48.00	0.5494	0.375155
0.2200	50.00	0.5494	0.345742
0.2200	55.00	0.5494	0.285738
0.2200	60.00	0.5494	0.240099
0.2200	65.00	0.5494	0.204581
0.2200	70.00	0.5494	0.176399
0.2200	75.00	0.5494	0.153663
0.2200	80.00	0.5494	0.135056

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.2400	5.00	0.0183	1.000000
0.2400	6.00	0.0211	1.000000
0.2400	7.00	0.0251	1.000000
0.2400	8.00	0.0300	0.936000
0.2400	9.00	0.0355	0.877500
0.2400	10.00	0.0603	0.835714
0.2400	11.00	0.0649	0.804375
0.2400	12.00	0.0699	0.780000
0.2400	13.00	0.0755	0.760500
0.2400	14.00	0.0815	0.744545
0.2400	15.00	0.0880	0.731250
0.2400	16.00	0.0951	0.720000
0.2400	17.00	0.1027	0.710357
0.2400	18.00	0.1108	0.702000
0.2400	19.00	0.1196	0.694688
0.2400	20.00	0.1289	0.688235
0.2400	21.00	0.1389	0.682500
0.2400	22.00	0.1495	0.677368
0.2400	23.00	0.1608	0.672750
0.2400	24.00	0.1729	0.668571
0.2400	25.00	0.1857	0.664773
0.2400	26.00	0.1994	0.661304
0.2400	27.00	0.2139	0.658125
0.2400	28.00	0.2295	0.655200
0.2400	29.00	0.2461	0.584916
0.2400	30.00	0.2639	0.583787
0.2400	31.00	0.2829	0.581337
0.2400	32.00	0.3035	0.577536
0.2400	33.00	0.3257	0.572327
0.2400	34.00	0.3498	0.565615
0.2400	35.00	0.3762	0.557250
0.2400	36.00	0.4055	0.547005
0.2400	37.00	0.4384	0.534515
0.2400	38.00	0.4760	0.519170
0.2400	39.00	0.5208	0.499832
0.2400	40.00	0.5421	0.475891
0.2400	42.00	0.5421	0.431647
0.2400	44.00	0.5421	0.393298
0.2400	46.00	0.5421	0.359842
0.2400	48.00	0.5421	0.330480
0.2400	50.00	0.5421	0.304570
0.2400	55.00	0.5421	0.251711
0.2400	60.00	0.5421	0.211507
0.2400	65.00	0.5421	0.180219
0.2400	70.00	0.5421	0.155393
0.2400	75.00	0.5421	0.135365
0.2400	80.00	0.5421	0.118973

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.2600	5.00	0.0198	1.000000
0.2600	6.00	0.0228	1.000000
0.2600	7.00	0.0272	0.945000
0.2600	8.00	0.0325	0.864000
0.2600	9.00	0.0385	0.810000
0.2600	10.00	0.0695	0.771428
0.2600	11.00	0.0743	0.742500
0.2600	12.00	0.0796	0.720000
0.2600	13.00	0.0853	0.702000
0.2600	14.00	0.0916	0.687273
0.2600	15.00	0.0985	0.675000
0.2600	16.00	0.1059	0.664615
0.2600	17.00	0.1138	0.655714
0.2600	18.00	0.1224	0.648000
0.2600	19.00	0.1316	0.641250
0.2600	20.00	0.1414	0.635294
0.2600	21.00	0.1519	0.630000
0.2600	22.00	0.1632	0.625263
0.2600	23.00	0.1752	0.621000
0.2600	24.00	0.1880	0.617143
0.2600	25.00	0.2016	0.613636
0.2600	26.00	0.2162	0.610435
0.2600	27.00	0.2319	0.607500
0.2600	28.00	0.2486	0.604800
0.2600	29.00	0.2666	0.539908
0.2600	30.00	0.2860	0.538675
0.2600	31.00	0.3069	0.535973
0.2600	32.00	0.3296	0.531734
0.2600	33.00	0.3544	0.525845
0.2600	34.00	0.3819	0.518127
0.2600	35.00	0.4125	0.508296
0.2600	36.00	0.4473	0.495882
0.2600	37.00	0.4881	0.480044
0.2600	38.00	0.5354	0.459034
0.2600	39.00	0.5354	0.435796
0.2600	40.00	0.5354	0.414279
0.2600	42.00	0.5354	0.375763
0.2600	44.00	0.5354	0.342379
0.2600	46.00	0.5354	0.313254
0.2600	48.00	0.5354	0.287693
0.2600	50.00	0.5354	0.265138
0.2600	55.00	0.5354	0.219123
0.2600	60.00	0.5354	0.184124
0.2600	65.00	0.5354	0.156887
0.2600	70.00	0.5354	0.135275
0.2600	75.00	0.5354	0.117839
0.2600	80.00	0.5354	0.103570

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.2800	5.00	0.0216	1.000000
0.2800	6.00	0.0249	0.990000
0.2800	7.00	0.0296	0.866250
0.2800	8.00	0.0354	0.792000
0.2800	9.00	0.0420	0.742500
0.2800	10.00	0.0800	0.707143
0.2800	11.00	0.0850	0.680625
0.2800	12.00	0.0906	0.660000
0.2800	13.00	0.0966	0.643500
0.2800	14.00	0.1033	0.630000
0.2800	15.00	0.1105	0.618750
0.2800	16.00	0.1182	0.609231
0.2800	17.00	0.1266	0.601071
0.2800	18.00	0.1357	0.594000
0.2800	19.00	0.1454	0.587813
0.2800	20.00	0.1558	0.582353
0.2800	21.00	0.1670	0.577500
0.2800	22.00	0.1790	0.573158
0.2800	23.00	0.1918	0.569250
0.2800	24.00	0.2055	0.565714
0.2800	25.00	0.2202	0.562500
0.2800	26.00	0.2359	0.559565
0.2800	27.00	0.2529	0.556875
0.2800	28.00	0.2712	0.554400
0.2800	29.00	0.2909	0.494770
0.2800	30.00	0.3124	0.493113
0.2800	31.00	0.3358	0.489782
0.2800	32.00	0.3616	0.484636
0.2800	33.00	0.3904	0.477446
0.2800	34.00	0.4229	0.467839
0.2800	35.00	0.4606	0.455158
0.2800	36.00	0.5063	0.438106
0.2800	37.00	0.5294	0.415658
0.2800	38.00	0.5294	0.394069
0.2800	39.00	0.5294	0.374119
0.2800	40.00	0.5294	0.355647
0.2800	42.00	0.5294	0.322582
0.2800	44.00	0.5294	0.293923
0.2800	46.00	0.5294	0.268920
0.2800	48.00	0.5294	0.246977
0.2800	50.00	0.5294	0.227614
0.2800	55.00	0.5294	0.188111
0.2800	60.00	0.5294	0.158065
0.2800	65.00	0.5294	0.134683
0.2800	70.00	0.5294	0.116130
0.2800	75.00	0.5294	0.101162
0.2800	80.00	0.5294	0.088912

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.3000	5.00	0.0238	1.000000
0.3000	6.00	0.0274	0.900000
0.3000	7.00	0.0326	0.787500
0.3000	8.00	0.0389	0.720000
0.3000	9.00	0.0462	0.675000
0.3000	10.00	0.0921	0.642857
0.3000	11.00	0.0974	0.618750
0.3000	12.00	0.1032	0.600000
0.3000	13.00	0.1096	0.585000
0.3000	14.00	0.1166	0.572727
0.3000	15.00	0.1242	0.562500
0.3000	16.00	0.1325	0.553846
0.3000	17.00	0.1414	0.546429
0.3000	18.00	0.1510	0.540000
0.3000	19.00	0.1613	0.534375
0.3000	20.00	0.1725	0.529412
0.3000	21.00	0.1844	0.525000
0.3000	22.00	0.1973	0.521053
0.3000	23.00	0.2111	0.517500
0.3000	24.00	0.2259	0.514286
0.3000	25.00	0.2419	0.511364
0.3000	26.00	0.2592	0.508696
0.3000	27.00	0.2779	0.506250
0.3000	28.00	0.2982	0.504000
0.3000	29.00	0.3204	0.449271
0.3000	30.00	0.3448	0.446724
0.3000	31.00	0.3720	0.442166
0.3000	32.00	0.4026	0.435286
0.3000	33.00	0.4380	0.425548
0.3000	34.00	0.4803	0.411928
0.3000	35.00	0.5239	0.392201
0.3000	36.00	0.5239	0.370715
0.3000	37.00	0.5239	0.350947
0.3000	38.00	0.5239	0.332719
0.3000	39.00	0.5239	0.315876
0.3000	40.00	0.5239	0.300279
0.3000	42.00	0.5239	0.272362
0.3000	44.00	0.5239	0.248165
0.3000	46.00	0.5239	0.227054
0.3000	48.00	0.5239	0.208527
0.3000	50.00	0.5239	0.192179
0.3000	55.00	0.5239	0.158825
0.3000	60.00	0.5239	0.133457
0.3000	65.00	0.5239	0.113715
0.3000	70.00	0.5239	0.098050
0.3000	75.00	0.5239	0.085413
0.3000	80.00	0.5239	0.075070

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.3200	5.00	0.0264	1.000000
0.3200	6.00	0.0304	0.809999
0.3200	7.00	0.0362	0.708750
0.3200	8.00	0.0433	0.648000
0.3200	9.00	0.0513	0.607500
0.3200	10.00	0.1059	0.578571
0.3200	11.00	0.1115	0.556875
0.3200	12.00	0.1177	0.540000
0.3200	13.00	0.1245	0.526500
0.3200	14.00	0.1319	0.515454
0.3200	15.00	0.1401	0.506250
0.3200	16.00	0.1489	0.498461
0.3200	17.00	0.1584	0.491786
0.3200	18.00	0.1687	0.486000
0.3200	19.00	0.1798	0.480937
0.3200	20.00	0.1918	0.476470
0.3200	21.00	0.2048	0.472500
0.3200	22.00	0.2188	0.468947
0.3200	23.00	0.2339	0.465750
0.3200	24.00	0.2502	0.462857
0.3200	25.00	0.2679	0.460227
0.3200	26.00	0.2872	0.457826
0.3200	27.00	0.3083	0.455625
0.3200	28.00	0.3315	0.404776
0.3200	29.00	0.3573	0.402806
0.3200	30.00	0.3865	0.398569
0.3200	31.00	0.4200	0.391607
0.3200	32.00	0.4599	0.381053
0.3200	33.00	0.5107	0.364935
0.3200	34.00	0.5191	0.343907
0.3200	35.00	0.5191	0.324535
0.3200	36.00	0.5191	0.306756
0.3200	37.00	0.5191	0.290399
0.3200	38.00	0.5191	0.275316
0.3200	39.00	0.5191	0.261378
0.3200	40.00	0.5191	0.248472
0.3200	42.00	0.5191	0.225372
0.3200	44.00	0.5191	0.205349
0.3200	46.00	0.5191	0.187881
0.3200	48.00	0.5191	0.172550
0.3200	50.00	0.5191	0.159022
0.3200	55.00	0.5191	0.131423
0.3200	60.00	0.5191	0.110432
0.3200	65.00	0.5191	0.094096
0.3200	70.00	0.5191	0.081134
0.3200	75.00	0.5191	0.070677
0.3200	80.00	0.5191	0.062118

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.3400	5.00	0.0297	0.900000
0.3400	6.00	0.0342	0.720000
0.3400	7.00	0.0408	0.630000
0.3400	8.00	0.0487	0.576000
0.3400	9.00	0.0578	0.540000
0.3400	10.00	0.1219	0.514285
0.3400	11.00	0.1279	0.495000
0.3400	12.00	0.1345	0.480000
0.3400	13.00	0.1418	0.468000
0.3400	14.00	0.1497	0.458182
0.3400	15.00	0.1585	0.450000
0.3400	16.00	0.1680	0.443077
0.3400	17.00	0.1783	0.437143
0.3400	18.00	0.1894	0.432000
0.3400	19.00	0.2016	0.427500
0.3400	20.00	0.2147	0.423529
0.3400	21.00	0.2289	0.420000
0.3400	22.00	0.2444	0.416842
0.3400	23.00	0.2612	0.414000
0.3400	24.00	0.2795	0.411429
0.3400	25.00	0.2997	0.409091
0.3400	26.00	0.3219	0.406956
0.3400	27.00	0.3466	0.359928
0.3400	28.00	0.3746	0.358203
0.3400	29.00	0.4068	0.353830
0.3400	30.00	0.4452	0.346028
0.3400	31.00	0.4939	0.333034
0.3400	32.00	0.5148	0.313350
0.3400	33.00	0.5148	0.294647
0.3400	34.00	0.5148	0.277569
0.3400	35.00	0.5148	0.261935
0.3400	36.00	0.5148	0.247585
0.3400	37.00	0.5148	0.234383
0.3400	38.00	0.5148	0.222209
0.3400	39.00	0.5148	0.210960
0.3400	40.00	0.5148	0.200544
0.3400	42.00	0.5148	0.181899
0.3400	44.00	0.5148	0.165739
0.3400	46.00	0.5148	0.151640
0.3400	48.00	0.5148	0.139266
0.3400	50.00	0.5148	0.128348
0.3400	55.00	0.5148	0.106073
0.3400	60.00	0.5148	0.089131
0.3400	65.00	0.5148	0.075946
0.3400	70.00	0.5148	0.065484
0.3400	75.00	0.5148	0.057044
0.3400	80.00	0.5148	0.050136

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.3600	5.00	0.0340	0.787500
0.3600	6.00	0.0391	0.630000
0.3600	7.00	0.0466	0.551250
0.3600	8.00	0.0556	0.504000
0.3600	9.00	0.0660	0.472500
0.3600	10.00	0.1404	0.450000
0.3600	11.00	0.1469	0.433125
0.3600	12.00	0.1540	0.420000
0.3600	13.00	0.1619	0.409500
0.3600	14.00	0.1706	0.400909
0.3600	15.00	0.1801	0.393750
0.3600	16.00	0.1904	0.387692
0.3600	17.00	0.2017	0.382500
0.3600	18.00	0.2141	0.378000
0.3600	19.00	0.2275	0.374063
0.3600	20.00	0.2421	0.370588
0.3600	21.00	0.2581	0.367500
0.3600	22.00	0.2757	0.364737
0.3600	23.00	0.2950	0.362250
0.3600	24.00	0.3164	0.360000
0.3600	25.00	0.3403	0.357955
0.3600	26.00	0.3674	0.314916
0.3600	27.00	0.3988	0.312879
0.3600	28.00	0.4364	0.307477
0.3600	29.00	0.4847	0.296949
0.3600	30.00	0.5112	0.278804
0.3600	31.00	0.5112	0.261107
0.3600	32.00	0.5112	0.245043
0.3600	33.00	0.5112	0.230417
0.3600	34.00	0.5112	0.217062
0.3600	35.00	0.5112	0.204836
0.3600	36.00	0.5112	0.193614
0.3600	37.00	0.5112	0.183290
0.3600	38.00	0.5112	0.173770
0.3600	39.00	0.5112	0.164973
0.3600	40.00	0.5112	0.156827
0.3600	42.00	0.5112	0.142247
0.3600	44.00	0.5112	0.129609
0.3600	46.00	0.5112	0.118584
0.3600	48.00	0.5112	0.108908
0.3600	50.00	0.5112	0.100370
0.3600	55.00	0.5112	0.082950
0.3600	60.00	0.5112	0.069701
0.3600	65.00	0.5112	0.059390
0.3600	70.00	0.5112	0.051209
0.3600	75.00	0.5112	0.044609
0.3600	80.00	0.5112	0.039207

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.3800	5.00	0.0396	0.675000
0.3800	6.00	0.0456	0.540000
0.3800	7.00	0.0544	0.472500
0.3800	8.00	0.0649	0.432000
0.3800	9.00	0.0770	0.405000
0.3800	10.00	0.1622	0.385714
0.3800	11.00	0.1692	0.371250
0.3800	12.00	0.1771	0.360000
0.3800	13.00	0.1857	0.351000
0.3800	14.00	0.1953	0.343636
0.3800	15.00	0.2058	0.337500
0.3800	16.00	0.2174	0.332308
0.3800	17.00	0.2300	0.327857
0.3800	18.00	0.2439	0.324000
0.3800	19.00	0.2592	0.320625
0.3800	20.00	0.2761	0.317647
0.3800	21.00	0.2947	0.315000
0.3800	22.00	0.3155	0.312631
0.3800	23.00	0.3389	0.310500
0.3800	24.00	0.3656	0.308571
0.3800	25.00	0.3966	0.269602
0.3800	26.00	0.4348	0.266121
0.3800	27.00	0.4851	0.257185
0.3800	28.00	0.5081	0.240150
0.3800	29.00	0.5081	0.223873
0.3800	30.00	0.5081	0.209197
0.3800	31.00	0.5081	0.195918
0.3800	32.00	0.5081	0.183865
0.3800	33.00	0.5081	0.172890
0.3800	34.00	0.5081	0.162870
0.3800	35.00	0.5081	0.153696
0.3800	36.00	0.5081	0.145276
0.3800	37.00	0.5081	0.137529
0.3800	38.00	0.5081	0.130386
0.3800	39.00	0.5081	0.123785
0.3800	40.00	0.5081	0.117673
0.3800	42.00	0.5081	0.106733
0.3800	44.00	0.5081	0.097251
0.3800	46.00	0.5081	0.088978
0.3800	48.00	0.5081	0.081718
0.3800	50.00	0.5081	0.075311
0.3800	55.00	0.5081	0.062240
0.3800	60.00	0.5081	0.052299
0.3800	65.00	0.5081	0.044563
0.3800	70.00	0.5081	0.038424
0.3800	75.00	0.5081	0.033472
0.3800	80.00	0.5081	0.029418

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.4000	5.00	0.0475	0.562500
0.4000	6.00	0.0548	0.450000
0.4000	7.00	0.0652	0.393750
0.4000	8.00	0.0779	0.360000
0.4000	9.00	0.0924	0.337500
0.4000	10.00	0.1882	0.321429
0.4000	11.00	0.1960	0.309375
0.4000	12.00	0.2048	0.300000
0.4000	13.00	0.2145	0.292500
0.4000	14.00	0.2253	0.286364
0.4000	15.00	0.2373	0.281250
0.4000	16.00	0.2506	0.276923
0.4000	17.00	0.2652	0.273214
0.4000	18.00	0.2815	0.270000
0.4000	19.00	0.2997	0.267187
0.4000	20.00	0.3202	0.264706
0.4000	21.00	0.3434	0.262500
0.4000	22.00	0.3704	0.260526
0.4000	23.00	0.4024	0.224992
0.4000	24.00	0.4427	0.222683
0.4000	25.00	0.5008	0.213579
0.4000	26.00	0.5055	0.197507
0.4000	27.00	0.5055	0.183147
0.4000	28.00	0.5055	0.170299
0.4000	29.00	0.5055	0.158757
0.4000	30.00	0.5055	0.148349
0.4000	31.00	0.5055	0.138933
0.4000	32.00	0.5055	0.130385
0.4000	33.00	0.5055	0.122603
0.4000	34.00	0.5055	0.115497
0.4000	35.00	0.5055	0.108991
0.4000	36.00	0.5055	0.103020
0.4000	37.00	0.5055	0.097527
0.4000	38.00	0.5055	0.092462
0.4000	39.00	0.5055	0.087781
0.4000	40.00	0.5055	0.083447
0.4000	42.00	0.5055	0.075688
0.4000	44.00	0.5055	0.068964
0.4000	46.00	0.5055	0.063098
0.4000	48.00	0.5055	0.057949
0.4000	50.00	0.5055	0.053406
0.4000	55.00	0.5055	0.044137
0.4000	60.00	0.5055	0.037087
0.4000	65.00	0.5055	0.031601
0.4000	70.00	0.5055	0.027248
0.4000	75.00	0.5055	0.023736
0.4000	80.00	0.5055	0.020862

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.4200	5.00	0.0594	0.450000
0.4200	6.00	0.0685	0.360000
0.4200	7.00	0.0815	0.315000
0.4200	8.00	0.0974	0.288000
0.4200	9.00	0.1155	0.270000
0.4200	10.00	0.2200	0.257143
0.4200	11.00	0.2290	0.247500
0.4200	12.00	0.2391	0.240000
0.4200	13.00	0.2505	0.234000
0.4200	14.00	0.2632	0.229091
0.4200	15.00	0.2774	0.225000
0.4200	16.00	0.2935	0.221539
0.4200	17.00	0.3115	0.218571
0.4200	18.00	0.3322	0.216000
0.4200	19.00	0.3560	0.213750
0.4200	20.00	0.3843	0.211765
0.4200	21.00	0.4193	0.210000
0.4200	22.00	0.4672	0.177323
0.4200	23.00	0.5035	0.164925
0.4200	24.00	0.5035	0.151468
0.4200	25.00	0.5035	0.139593
0.4200	26.00	0.5035	0.129061
0.4200	27.00	0.5035	0.119678
0.4200	28.00	0.5035	0.111282
0.4200	29.00	0.5035	0.103740
0.4200	30.00	0.5035	0.096939
0.4200	31.00	0.5035	0.090786
0.4200	32.00	0.5035	0.085200
0.4200	33.00	0.5035	0.080115
0.4200	34.00	0.5035	0.075472
0.4200	35.00	0.5035	0.071221
0.4200	36.00	0.5035	0.067319
0.4200	37.00	0.5035	0.063729
0.4200	38.00	0.5035	0.060419
0.4200	39.00	0.5035	0.057361
0.4200	40.00	0.5035	0.054528
0.4200	42.00	0.5035	0.049459
0.4200	44.00	0.5035	0.045065
0.4200	46.00	0.5035	0.041231
0.4200	48.00	0.5035	0.037867
0.4200	50.00	0.5035	0.034898
0.4200	55.00	0.5035	0.028841
0.4200	60.00	0.5035	0.024235
0.4200	65.00	0.5035	0.020650
0.4200	70.00	0.5035	0.017805
0.4200	75.00	0.5035	0.015510
0.4200	80.00	0.5035	0.013632

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.4400	5.00	0.0792	0.337500
0.4400	6.00	0.0913	0.270000
0.4400	7.00	0.1087	0.236250
0.4400	8.00	0.1298	0.216000
0.4400	9.00	0.1540	0.202500
0.4400	10.00	0.2607	0.192857
0.4400	11.00	0.2716	0.185625
0.4400	12.00	0.2840	0.180000
0.4400	13.00	0.2981	0.175500
0.4400	14.00	0.3143	0.171818
0.4400	15.00	0.3330	0.168750
0.4400	16.00	0.3548	0.166154
0.4400	17.00	0.3809	0.163928
0.4400	18.00	0.4137	0.162000
0.4400	19.00	0.4592	0.134548
0.4400	20.00	0.5019	0.125247
0.4400	21.00	0.5019	0.113603
0.4400	22.00	0.5019	0.103510
0.4400	23.00	0.5019	0.094705
0.4400	24.00	0.5019	0.086977
0.4400	25.00	0.5019	0.080158
0.4400	26.00	0.5019	0.074111
0.4400	27.00	0.5019	0.068723
0.4400	28.00	0.5019	0.063902
0.4400	29.00	0.5019	0.059571
0.4400	30.00	0.5019	0.055665
0.4400	31.00	0.5019	0.052132
0.4400	32.00	0.5019	0.048925
0.4400	33.00	0.5019	0.046004
0.4400	34.00	0.5019	0.043338
0.4400	35.00	0.5019	0.040897
0.4400	36.00	0.5019	0.038657
0.4400	37.00	0.5019	0.036595
0.4400	38.00	0.5019	0.034695
0.4400	39.00	0.5019	0.032938
0.4400	40.00	0.5019	0.031312
0.4400	42.00	0.5019	0.028401
0.4400	44.00	0.5019	0.025878
0.4400	46.00	0.5019	0.023676
0.4400	48.00	0.5019	0.021744
0.4400	50.00	0.5019	0.020040
0.4400	55.00	0.5019	0.016562
0.4400	60.00	0.5019	0.013916
0.4400	65.00	0.5019	0.011858
0.4400	70.00	0.5019	0.010224
0.4400	75.00	0.5019	0.008906
0.4400	80.00	0.5019	0.007828

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.4600	5.00	0.1189	0.225000
0.4600	6.00	0.1369	0.180000
0.4600	7.00	0.1631	0.157500
0.4600	8.00	0.1947	0.144000
0.4600	9.00	0.2311	0.135000
0.4600	10.00	0.3173	0.128571
0.4600	11.00	0.3322	0.123750
0.4600	12.00	0.3498	0.120000
0.4600	13.00	0.3709	0.117000
0.4600	14.00	0.3973	0.114545
0.4600	15.00	0.4326	0.112500
0.4600	16.00	0.4941	0.088677
0.4600	17.00	0.5008	0.078638
0.4600	18.00	0.5008	0.070143
0.4600	19.00	0.5008	0.062954
0.4600	20.00	0.5008	0.056816
0.4600	21.00	0.5008	0.051534
0.4600	22.00	0.5008	0.046956
0.4600	23.00	0.5008	0.042961
0.4600	24.00	0.5008	0.039456
0.4600	25.00	0.5008	0.036362
0.4600	26.00	0.5008	0.033619
0.4600	27.00	0.5008	0.031175
0.4600	28.00	0.5008	0.028988
0.4600	29.00	0.5008	0.027023
0.4600	30.00	0.5008	0.025252
0.4600	31.00	0.5008	0.023649
0.4600	32.00	0.5008	0.022194
0.4600	33.00	0.5008	0.020869
0.4600	34.00	0.5008	0.019660
0.4600	35.00	0.5008	0.018552
0.4600	36.00	0.5008	0.017536
0.4600	37.00	0.5008	0.016601
0.4600	38.00	0.5008	0.015739
0.4600	39.00	0.5008	0.014942
0.4600	40.00	0.5008	0.014204
0.4600	42.00	0.5008	0.012883
0.4600	44.00	0.5008	0.011739
0.4600	46.00	0.5008	0.010740
0.4600	48.00	0.5008	0.009864
0.4600	50.00	0.5008	0.009091
0.4600	55.00	0.5008	0.007513
0.4600	60.00	0.5008	0.006313
0.4600	65.00	0.5008	0.005379
0.4600	70.00	0.5008	0.004638
0.4600	75.00	0.5008	0.004040
0.4600	80.00	0.5008	0.003551

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.4800	5.00	0.2377	0.112500
0.4800	6.00	0.2738	0.090000
0.4800	7.00	0.3261	0.078750
0.4800	8.00	0.3895	0.072000
0.4800	9.00	0.4621	0.067500
0.4800	10.00	0.4222	0.064286
0.4800	11.00	0.4636	0.061875
0.4800	12.00	0.5002	0.040265
0.4800	13.00	0.5002	0.034308
0.4800	14.00	0.5002	0.029582
0.4800	15.00	0.5002	0.025769
0.4800	16.00	0.5002	0.022649
0.4800	17.00	0.5002	0.020063
0.4800	18.00	0.5002	0.017895
0.4800	19.00	0.5002	0.016061
0.4800	20.00	0.5002	0.014495
0.4800	21.00	0.5002	0.013148
0.4800	22.00	0.5002	0.011980
0.4800	23.00	0.5002	0.010961
0.4800	24.00	0.5002	0.010066
0.4800	25.00	0.5002	0.009277
0.4800	26.00	0.5002	0.008577
0.4800	27.00	0.5002	0.007954
0.4800	28.00	0.5002	0.007396
0.4800	29.00	0.5002	0.006894
0.4800	30.00	0.5002	0.006442
0.4800	31.00	0.5002	0.006033
0.4800	32.00	0.5002	0.005662
0.4800	33.00	0.5002	0.005324
0.4800	34.00	0.5002	0.005016
0.4800	35.00	0.5002	0.004733
0.4800	36.00	0.5002	0.004474
0.4800	37.00	0.5002	0.004235
0.4800	38.00	0.5002	0.004015
0.4800	39.00	0.5002	0.003812
0.4800	40.00	0.5002	0.003624
0.4800	42.00	0.5002	0.003287
0.4800	44.00	0.5002	0.002995
0.4800	46.00	0.5002	0.002740
0.4800	48.00	0.5002	0.002517
0.4800	50.00	0.5002	0.002319
0.4800	55.00	0.5002	0.001917
0.4800	60.00	0.5002	0.001611
0.4800	65.00	0.5002	0.001372
0.4800	70.00	0.5002	0.001183
0.4800	75.00	0.5002	0.001031
0.4800	80.00	0.5002	0.000906

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN DOUBLE CURVATURE

E/T	H/T	PHY	V
0.4999	5.00	0.5000	0.000006
0.4999	6.00	0.5000	0.000004
0.4999	7.00	0.5000	0.000003
0.4999	8.00	0.5000	0.000002
0.4999	9.00	0.5000	0.000002
0.4999	10.00	0.5000	0.000001
0.4999	11.00	0.5000	0.000001
0.4999	12.00	0.5000	0.000001
0.4999	13.00	0.5000	0.000001
0.4999	14.00	0.5000	0.000001
0.4999	15.00	0.5000	0.000001
0.4999	16.00	0.5000	0.000001
0.4999	17.00	0.5000	0.000001
0.4999	18.00	0.5000	0.000000
0.4999	19.00	0.5000	0.000000
0.4999	20.00	0.5000	0.000000
0.4999	21.00	0.5000	0.000000
0.4999	22.00	0.5000	0.000000
0.4999	23.00	0.5000	0.000000
0.4999	24.00	0.5000	0.000000
0.4999	25.00	0.5000	0.000000
0.4999	26.00	0.5000	0.000000
0.4999	27.00	0.5000	0.000000
0.4999	28.00	0.5000	0.000000
0.4999	29.00	0.5000	0.000000
0.4999	30.00	0.5000	0.000000
0.4999	31.00	0.5000	0.000000
0.4999	32.00	0.5000	0.000000
0.4999	33.00	0.5000	0.000000
0.4999	34.00	0.5000	0.000000
0.4999	35.00	0.5000	0.000000
0.4999	36.00	0.5000	0.000000
0.4999	37.00	0.5000	0.000000
0.4999	38.00	0.5000	0.000000
0.4999	39.00	0.5000	0.000000
0.4999	40.00	0.5000	0.000000
0.4999	42.00	0.5000	0.000000
0.4999	44.00	0.5000	0.000000
0.4999	46.00	0.5000	0.000000
0.4999	48.00	0.5000	0.000000
0.4999	50.00	0.5000	0.000000
0.4999	55.00	0.5000	0.000000
0.4999	60.00	0.5000	0.000000
0.4999	65.00	0.5000	0.000000
0.4999	70.00	0.5000	0.000000
0.4999	75.00	0.5000	0.000000
0.4999	80.00	0.5000	0.000000

OP

93 KBYTES USED JOB-STEP RETURN CODE = 0

END JOB 1295 21:05 12 JUN 79 A.AWNT.J.C.M.B.

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

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C        STRESS REDUCTION FACTORS 'WALLS IN SINGLE CURVATURE'
C       PROGRAMMER: ADNAN A. AWNI
        REAL PHY(799)
        REAL E(17)
        REAL SLEND(47)
    001 FORMAT(9F8.4)
    002 FORMAT(10F8.3)
    003 FORMAT('1',// T21,'FINAL    COMPUTED    RESULTS'/)
    004 FORMAT(    22X,'WALLS IN SINGLE CURVATURE')
    005 FORMAT(    /13X,'E/T',12X,'H/T',10X,'PHY',13X,'V'/)
    006 FORMAT(    12X,F6.4,8X,F6.2,8X,F6.4,8X,F8.6)
    007 FORMAT(    23X,'STRESS REDUCTION FACTOR'/)
    070 FORMAT(    22X,'ECC. AT ONE END EQUAL ZERO'/)
    008 FORMAT(13F6.4)
    009 READ(5,1) (E(I),I=1,17)
    010 READ(5,2) (SLEND(J),J=1,47)
    011 READ(5,8) (PHY(K),K=1,799)
    012 DO 140 I=1,17
    013 DO 140 J=1,47
    014 K=((I-1)*47)+J
    015 REALK=((SLEND(J))**2)/(55.5)
    016 PHYB=((1.0-E(I))+(4.0*E(I)**2-2.0*E(I)+1.0)**0.5)/(3.0)
    017 IF(PHY(K)-PHYB)20,18,18
    018 PHY(K)=PHYB
    019 GOTO 36
    020 CHECKB=(PHY(K)-E(I))
    021 IF(CHECKB)22,22,24
    022 V1=(1.5*SLEND(J)/2.0)/((1.+6.*E(I))*((SLEND(J)/2.0)-1.5))
    023 VMAX=1.0
    024 IF(V1-VMAX)25,25,27
    025 V=V1
    026 GOTO 80
    027 V=VMAX
    028 GOTO 80
    029 V2=(3.0)/((2.0)+((3.0/PHY(K))*(E(I)+PHY(K))**2))
    030 VMAX=1.0
    031 IF(V2-VMAX)32,32,34
    032 V=V2
    033 GOTO 80
    034 V=VMAX
    035 GOTO 80
    036 Z1=(9.869*(PHY(K)**2))/((E(I)+PHY(K))**2)
    037 V3=Z1/REALK
    038 VMAX=1.0
    039 IF(V3-VMAX)40,40,60
    040 V=V3
    050 GOTO 80
    060 V=VMAX
    080 IF(SLEND(J)-5.0)90,90,130
    090 WRITE(6,3)
    100 WRITE(6,7)
    110 WRITE(6,4)
    111 WRITE(6,70)
    120 WRITE(6,5)
    130 WRITE(6,6) (E(I),SLEND(J),PHY(K),V)
    140 CONTINUE
    150 STOP
    160 END
```

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0000	5.00	0.0000	1.000000
0.0000	6.00	0.0000	1.000000
0.0000	7.00	0.0000	1.000000
0.0000	8.00	0.0000	1.000000
0.0000	9.00	0.0000	1.000000
0.0000	10.00	0.0000	1.000000
0.0000	11.00	0.0000	1.000000
0.0000	12.00	0.0000	1.000000
0.0000	13.00	0.0000	1.000000
0.0000	14.00	0.0000	1.000000
0.0000	15.00	0.0000	1.000000
0.0000	16.00	0.0000	1.000000
0.0000	17.00	0.0000	1.000000
0.0000	18.00	0.0000	1.000000
0.0000	19.00	0.0000	1.000000
0.0000	20.00	0.0636	1.000000
0.0000	21.00	0.1384	1.000000
0.0000	22.00	0.2169	1.000000
0.0000	23.00	0.2991	1.000000
0.0000	24.00	0.3849	1.000000
0.0000	25.00	0.4744	1.000000
0.0000	26.00	0.5675	1.000000
0.0000	27.00	0.6642	1.000000
0.0000	28.00	0.6667	0.698634
0.0000	29.00	0.6667	0.651284
0.0000	30.00	0.6667	0.608589
0.0000	31.00	0.6667	0.569958
0.0000	32.00	0.6667	0.534892
0.0000	33.00	0.6667	0.502966
0.0000	34.00	0.6667	0.473814
0.0000	35.00	0.6667	0.447126
0.0000	36.00	0.6667	0.422631
0.0000	37.00	0.6667	0.400095
0.0000	38.00	0.6667	0.379314
0.0000	39.00	0.6667	0.360111
0.0000	40.00	0.6667	0.342331
0.0000	42.00	0.6667	0.310504
0.0000	44.00	0.6667	0.282918
0.0000	46.00	0.6667	0.258851
0.0000	48.00	0.6667	0.237730
0.0000	50.00	0.6667	0.219092
0.0000	55.00	0.6667	0.181068
0.0000	60.00	0.6667	0.152147
0.0000	65.00	0.6667	0.129640
0.0000	70.00	0.6667	0.111781
0.0000	75.00	0.6667	0.097374
0.0000	80.00	0.6667	0.085583

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0100	5.00	0.0029	1.000000
0.0100	6.00	0.0037	1.000000
0.0100	7.00	0.0046	1.000000
0.0100	8.00	0.0055	1.000000
0.0100	9.00	0.0066	1.000000
0.0100	10.00	0.0079	1.000000
0.0100	11.00	0.0095	1.000000
0.0100	12.00	0.0113	1.000000
0.0100	13.00	0.0137	1.000000
0.0100	14.00	0.0167	1.000000
0.0100	15.00	0.0208	1.000000
0.0100	16.00	0.0267	1.000000
0.0100	17.00	0.0359	1.000000
0.0100	18.00	0.0511	1.000000
0.0100	19.00	0.0773	1.000000
0.0100	20.00	0.1191	1.000000
0.0100	21.00	0.1761	1.000000
0.0100	22.00	0.2442	1.000000
0.0100	23.00	0.3199	1.000000
0.0100	24.00	0.4015	1.000000
0.0100	25.00	0.4880	1.000000
0.0100	26.00	0.5790	1.000000
0.0100	27.00	0.6601	0.729084
0.0100	28.00	0.6601	0.677937
0.0100	29.00	0.6601	0.631989
0.0100	30.00	0.6601	0.590559
0.0100	31.00	0.6601	0.553073
0.0100	32.00	0.6601	0.519046
0.0100	33.00	0.6601	0.488065
0.0100	34.00	0.6601	0.459777
0.0100	35.00	0.6601	0.433880
0.0100	36.00	0.6601	0.410110
0.0100	37.00	0.6601	0.388242
0.0100	38.00	0.6601	0.368077
0.0100	39.00	0.6601	0.349443
0.0100	40.00	0.6601	0.332189
0.0100	42.00	0.6601	0.301305
0.0100	44.00	0.6601	0.274536
0.0100	46.00	0.6601	0.251183
0.0100	48.00	0.6601	0.230687
0.0100	50.00	0.6601	0.212601
0.0100	55.00	0.6601	0.175703
0.0100	60.00	0.6601	0.147640
0.0100	65.00	0.6601	0.125799
0.0100	70.00	0.6601	0.108470
0.0100	75.00	0.6601	0.094489
0.0100	80.00	0.6601	0.083047

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0200	5.00	0.0057	1.000000
0.0200	6.00	0.0072	1.000000
0.0200	7.00	0.0089	1.000000
0.0200	8.00	0.0107	1.000000
0.0200	9.00	0.0129	1.000000
0.0200	10.00	0.0153	1.000000
0.0200	11.00	0.0182	1.000000
0.0200	12.00	0.0217	1.000000
0.0200	13.00	0.0260	1.000000
0.0200	14.00	0.0314	1.000000
0.0200	15.00	0.0384	1.000000
0.0200	16.00	0.0480	1.000000
0.0200	17.00	0.0615	1.000000
0.0200	18.00	0.0812	1.000000
0.0200	19.00	0.1106	1.000000
0.0200	20.00	0.1507	1.000000
0.0200	21.00	0.2034	1.000000
0.0200	22.00	0.2667	1.000000
0.0200	23.00	0.3384	1.000000
0.0200	24.00	0.4168	1.000000
0.0200	25.00	0.5009	1.000000
0.0200	26.00	0.5900	1.000000
0.0200	27.00	0.6535	0.707385
0.0200	28.00	0.6535	0.657760
0.0200	29.00	0.6535	0.613179
0.0200	30.00	0.6535	0.572982
0.0200	31.00	0.6535	0.536612
0.0200	32.00	0.6535	0.503598
0.0200	33.00	0.6535	0.473539
0.0200	34.00	0.6535	0.446093
0.0200	35.00	0.6535	0.420966
0.0200	36.00	0.6535	0.397904
0.0200	37.00	0.6535	0.376687
0.0200	38.00	0.6535	0.357122
0.0200	39.00	0.6535	0.339043
0.0200	40.00	0.6535	0.322302
0.0200	42.00	0.6535	0.292338
0.0200	44.00	0.6535	0.266366
0.0200	46.00	0.6535	0.243707
0.0200	48.00	0.6535	0.223821
0.0200	50.00	0.6535	0.206273
0.0200	55.00	0.6535	0.170474
0.0200	60.00	0.6535	0.143246
0.0200	65.00	0.6535	0.122055
0.0200	70.00	0.6535	0.105242
0.0200	75.00	0.6535	0.091677
0.0200	80.00	0.6535	0.080576

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0300	5.00	0.0082	1.000000
0.0300	6.00	0.0104	1.000000
0.0300	7.00	0.0129	1.000000
0.0300	8.00	0.0156	1.000000
0.0300	9.00	0.0187	1.000000
0.0300	10.00	0.0222	1.000000
0.0300	11.00	0.0264	1.000000
0.0300	12.00	0.0312	1.000000
0.0300	13.00	0.0371	1.000000
0.0300	14.00	0.0445	1.000000
0.0300	15.00	0.0538	1.000000
0.0300	16.00	0.0660	1.000000
0.0300	17.00	0.0823	1.000000
0.0300	18.00	0.0104	1.000000
0.0300	19.00	0.1349	1.000000
0.0300	20.00	0.1750	1.000000
0.0300	21.00	0.2257	1.000000
0.0300	22.00	0.2861	1.000000
0.0300	23.00	0.3550	1.000000
0.0300	24.00	0.4311	1.000000
0.0300	25.00	0.5132	1.000000
0.0300	26.00	0.6007	1.000000
0.0300	27.00	0.6471	0.686242
0.0300	28.00	0.6471	0.638100
0.0300	29.00	0.6471	0.594852
0.0300	30.00	0.6471	0.555857
0.0300	31.00	0.6471	0.520573
0.0300	32.00	0.6471	0.488546
0.0300	33.00	0.6471	0.459386
0.0300	34.00	0.6471	0.432760
0.0300	35.00	0.6471	0.408384
0.0300	36.00	0.6471	0.386011
0.0300	37.00	0.6471	0.365428
0.0300	38.00	0.6471	0.346448
0.0300	39.00	0.6471	0.328909
0.0300	40.00	0.6471	0.312669
0.0300	42.00	0.6471	0.283600
0.0300	44.00	0.6471	0.258404
0.0300	46.00	0.6471	0.236423
0.0300	48.00	0.6471	0.217131
0.0300	50.00	0.6471	0.200108
0.0300	55.00	0.6471	0.165379
0.0300	60.00	0.6471	0.138964
0.0300	65.00	0.6471	0.118407
0.0300	70.00	0.6471	0.102096
0.0300	75.00	0.6471	0.088937
0.0300	80.00	0.6471	0.078167

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0400	5.00	0.0105	1.000000
0.0400	6.00	0.0134	1.000000
0.0400	7.00	0.0166	1.000000
0.0400	8.00	0.0202	1.000000
0.0400	9.00	0.0241	1.000000
0.0400	10.00	0.0287	1.000000
0.0400	11.00	0.0339	1.000000
0.0400	12.00	0.0401	1.000000
0.0400	13.00	0.0474	1.000000
0.0400	14.00	0.0564	1.000000
0.0400	15.00	0.0675	1.000000
0.0400	16.00	0.0817	1.000000
0.0400	17.00	0.1000	1.000000
0.0400	18.00	0.1240	1.000000
0.0400	19.00	0.1554	1.000000
0.0400	20.00	0.1954	1.000000
0.0400	21.00	0.2449	1.000000
0.0400	22.00	0.3035	1.000000
0.0400	23.00	0.3703	1.000000
0.0400	24.00	0.4444	1.000000
0.0400	25.00	0.5249	1.000000
0.0400	26.00	0.6109	1.000000
0.0400	27.00	0.6408	0.665652
0.0400	28.00	0.6408	0.618954
0.0400	29.00	0.6408	0.577004
0.0400	30.00	0.6408	0.539178
0.0400	31.00	0.6408	0.504953
0.0400	32.00	0.6408	0.473887
0.0400	33.00	0.6408	0.445602
0.0400	34.00	0.6408	0.419775
0.0400	35.00	0.6408	0.396131
0.0400	36.00	0.6408	0.374429
0.0400	37.00	0.6408	0.354463
0.0400	38.00	0.6408	0.336053
0.0400	39.00	0.6408	0.319040
0.0400	40.00	0.6408	0.303288
0.0400	42.00	0.6408	0.275091
0.0400	44.00	0.6408	0.250651
0.0400	46.00	0.6408	0.229329
0.0400	48.00	0.6408	0.210616
0.0400	50.00	0.6408	0.194104
0.0400	55.00	0.6408	0.160417
0.0400	60.00	0.6408	0.134794
0.0400	65.00	0.6408	0.114854
0.0400	70.00	0.6408	0.099033
0.0400	75.00	0.6408	0.086268
0.0400	80.00	0.6408	0.075822

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0500	5.00	0.0127	1.000000
0.0500	6.00	0.0162	1.000000
0.0500	7.00	0.0201	1.000000
0.0500	8.00	0.0244	1.000000
0.0500	9.00	0.0292	1.000000
0.0500	10.00	0.0347	1.000000
0.0500	11.00	0.0410	1.000000
0.0500	12.00	0.0483	1.000000
0.0500	13.00	0.0569	1.000000
0.0500	14.00	0.0673	1.000000
0.0500	15.00	0.0800	1.000000
0.0500	16.00	0.0958	1.000000
0.0500	17.00	0.1157	1.000000
0.0500	18.00	0.1410	1.000000
0.0500	19.00	0.1731	1.000000
0.0500	20.00	0.2132	1.000000
0.0500	21.00	0.2619	1.000000
0.0500	22.00	0.3192	1.000000
0.0500	23.00	0.3845	1.000000
0.0500	24.00	0.4570	1.000000
0.0500	25.00	0.5360	1.000000
0.0500	26.00	0.6208	1.000000
0.0500	27.00	0.6346	0.645609
0.0500	28.00	0.6346	0.600318
0.0500	29.00	0.6346	0.559630
0.0500	30.00	0.6346	0.522944
0.0500	31.00	0.6346	0.489749
0.0500	32.00	0.6346	0.459618
0.0500	33.00	0.6346	0.432185
0.0500	34.00	0.6346	0.407136
0.0500	35.00	0.6346	0.384203
0.0500	36.00	0.6346	0.363155
0.0500	37.00	0.6346	0.343790
0.0500	38.00	0.6346	0.325934
0.0500	39.00	0.6346	0.309434
0.0500	40.00	0.6346	0.294156
0.0500	42.00	0.6346	0.266808
0.0500	44.00	0.6346	0.243104
0.0500	46.00	0.6346	0.222424
0.0500	48.00	0.6346	0.204275
0.0500	50.00	0.6346	0.188260
0.0500	55.00	0.6346	0.155586
0.0500	60.00	0.6346	0.130736
0.0500	65.00	0.6346	0.111396
0.0500	70.00	0.6346	0.096051
0.0500	75.00	0.6346	0.083671
0.0500	80.00	0.6346	0.073539

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0600	5.00	0.0147	1.000000
0.0600	6.00	0.0189	1.000000
0.0600	7.00	0.0234	1.000000
0.0600	8.00	0.0284	1.000000
0.0600	9.00	0.0340	1.000000
0.0600	10.00	0.0404	1.000000
0.0600	11.00	0.0476	1.000000
0.0600	12.00	0.0559	1.000000
0.0600	13.00	0.0657	1.000000
0.0600	14.00	0.0773	1.000000
0.0600	15.00	0.0914	1.000000
0.0600	16.00	0.1085	1.000000
0.0600	17.00	0.1298	1.000000
0.0600	18.00	0.1562	1.000000
0.0600	19.00	0.1890	1.000000
0.0600	20.00	0.2291	1.000000
0.0600	21.00	0.2773	1.000000
0.0600	22.00	0.3336	1.000000
0.0600	23.00	0.3977	1.000000
0.0600	24.00	0.4690	1.000000
0.0600	25.00	0.5467	1.000000
0.0600	26.00	0.6286	0.675198
0.0600	27.00	0.6286	0.626110
0.0600	28.00	0.6286	0.582186
0.0600	29.00	0.6286	0.542728
0.0600	30.00	0.6286	0.507149
0.0600	31.00	0.6286	0.474957
0.0600	32.00	0.6286	0.445736
0.0600	33.00	0.6286	0.419131
0.0600	34.00	0.6286	0.394839
0.0600	35.00	0.6286	0.372599
0.0600	36.00	0.6286	0.352187
0.0600	37.00	0.6286	0.333407
0.0600	38.00	0.6286	0.316090
0.0600	39.00	0.6286	0.300088
0.0600	40.00	0.6286	0.285271
0.0600	42.00	0.6286	0.258749
0.0600	44.00	0.6286	0.235761
0.0600	46.00	0.6286	0.215706
0.0600	48.00	0.6286	0.198105
0.0600	50.00	0.6286	0.182573
0.0600	55.00	0.6286	0.150887
0.0600	60.00	0.6286	0.126787
0.0600	65.00	0.6286	0.108032
0.0600	70.00	0.6286	0.093150
0.0600	75.00	0.6286	0.081144
0.0600	80.00	0.6286	0.071318

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0700	5.00	0.0166	1.000000
0.0700	6.00	0.0213	1.000000
0.0700	7.00	0.0265	1.000000
0.0700	8.00	0.0322	1.000000
0.0700	9.00	0.0386	1.000000
0.0700	10.00	0.0457	1.000000
0.0700	11.00	0.0538	1.000000
0.0700	12.00	0.0631	1.000000
0.0700	13.00	0.0739	1.000000
0.0700	14.00	0.0867	1.000000
0.0700	15.00	0.1019	1.000000
0.0700	16.00	0.1202	1.000000
0.0700	17.00	0.1426	1.000000
0.0700	18.00	0.1699	1.000000
0.0700	19.00	0.2033	1.000000
0.0700	20.00	0.2436	1.000000
0.0700	21.00	0.2914	1.000000
0.0700	22.00	0.3470	1.000000
0.0700	23.00	0.4101	1.000000
0.0700	24.00	0.4803	1.000000
0.0700	25.00	0.5570	1.000000
0.0700	26.00	0.6226	0.654751
0.0700	27.00	0.6226	0.607149
0.0700	28.00	0.6226	0.564555
0.0700	29.00	0.6226	0.526292
0.0700	30.00	0.6226	0.491791
0.0700	31.00	0.6226	0.460574
0.0700	32.00	0.6226	0.432238
0.0700	33.00	0.6226	0.406439
0.0700	34.00	0.6226	0.382882
0.0700	35.00	0.6226	0.361315
0.0700	36.00	0.6226	0.341521
0.0700	37.00	0.6226	0.323310
0.0700	38.00	0.6226	0.306518
0.0700	39.00	0.6226	0.291000
0.0700	40.00	0.6226	0.276632
0.0700	42.00	0.6226	0.250913
0.0700	44.00	0.6226	0.228622
0.0700	46.00	0.6226	0.209174
0.0700	48.00	0.6226	0.192106
0.0700	50.00	0.6226	0.177045
0.0700	55.00	0.6226	0.146318
0.0700	60.00	0.6226	0.122948
0.0700	65.00	0.6226	0.104760
0.0700	70.00	0.6226	0.090329
0.0700	75.00	0.6226	0.078686
0.0700	80.00	0.6226	0.069158

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0800	5.00	0.0183	1.000000
0.0800	6.00	0.0236	1.000000
0.0800	7.00	0.0294	1.000000
0.0800	8.00	0.0358	1.000000
0.0800	9.00	0.0428	1.000000
0.0800	10.00	0.0507	1.000000
0.0800	11.00	0.0597	1.000000
0.0800	12.00	0.0699	1.000000
0.0800	13.00	0.0816	1.000000
0.0800	14.00	0.0954	1.000000
0.0800	15.00	0.1117	1.000000
0.0800	16.00	0.1311	1.000000
0.0800	17.00	0.1574	1.000000
0.0800	18.00	0.1899	1.000000
0.0800	19.00	0.2164	1.000000
0.0800	20.00	0.2569	1.000000
0.0800	21.00	0.3045	1.000000
0.0800	22.00	0.3595	1.000000
0.0800	23.00	0.4218	1.000000
0.0800	24.00	0.4911	1.000000
0.0800	25.00	0.5668	1.000000
0.0800	26.00	0.6168	0.634878
0.0800	27.00	0.6168	0.588721
0.0800	28.00	0.6168	0.547421
0.0800	29.00	0.6168	0.510318
0.0800	30.00	0.6168	0.476865
0.0800	31.00	0.6168	0.446595
0.0800	32.00	0.6168	0.419119
0.0800	33.00	0.6168	0.394103
0.0800	34.00	0.6168	0.371261
0.0800	35.00	0.6168	0.350349
0.0800	36.00	0.6168	0.331156
0.0800	37.00	0.6168	0.313497
0.0800	38.00	0.6168	0.297215
0.0800	39.00	0.6168	0.282168
0.0800	40.00	0.6168	0.268236
0.0800	42.00	0.6168	0.243298
0.0800	44.00	0.6168	0.221683
0.0800	46.00	0.6168	0.202825
0.0800	48.00	0.6168	0.186275
0.0800	50.00	0.6168	0.171671
0.0800	55.00	0.6168	0.141877
0.0800	60.00	0.6168	0.119216
0.0800	65.00	0.6168	0.101581
0.0800	70.00	0.6168	0.087587
0.0800	75.00	0.6168	0.076298
0.0800	80.00	0.6168	0.067059

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.0900	5.00	0.0199	1.000000
0.0900	6.00	0.0257	1.000000
0.0900	7.00	0.0321	1.000000
0.0900	8.00	0.0391	1.000000
0.0900	9.00	0.0468	1.000000
0.0900	10.00	0.0555	1.000000
0.0900	11.00	0.0652	1.000000
0.0900	12.00	0.0762	1.000000
0.0900	13.00	0.0889	1.000000
0.0900	14.00	0.1036	1.000000
0.0900	15.00	0.1208	1.000000
0.0900	16.00	0.1411	1.000000
0.0900	17.00	0.1653	1.000000
0.0900	18.00	0.1942	1.000000
0.0900	19.00	0.2286	1.000000
0.0900	20.00	0.2692	1.000000
0.0900	21.00	0.3167	1.000000
0.0900	22.00	0.3712	1.000000
0.0900	23.00	0.4329	1.000000
0.0900	24.00	0.5014	1.000000
0.0900	25.00	0.5763	1.000000
0.0900	26.00	0.6111	0.615575
0.0900	27.00	0.6111	0.570822
0.0900	28.00	0.6111	0.530777
0.0900	29.00	0.6111	0.494803
0.0900	30.00	0.6111	0.462366
0.0900	31.00	0.6111	0.433017
0.0900	32.00	0.6111	0.406376
0.0900	33.00	0.6111	0.382120
0.0900	34.00	0.6111	0.359973
0.0900	35.00	0.6111	0.339697
0.0900	36.00	0.6111	0.321087
0.0900	37.00	0.6111	0.303966
0.0900	38.00	0.6111	0.288178
0.0900	39.00	0.6111	0.273589
0.0900	40.00	0.6111	0.260081
0.0900	42.00	0.6111	0.235901
0.0900	44.00	0.6111	0.214943
0.0900	46.00	0.6111	0.196658
0.0900	48.00	0.6111	0.180612
0.0900	50.00	0.6111	0.166452
0.0900	55.00	0.6111	0.137563
0.0900	60.00	0.6111	0.115591
0.0900	65.00	0.6111	0.098492
0.0900	70.00	0.6111	0.084924
0.0900	75.00	0.6111	0.073978
0.0900	80.00	0.6111	0.065020

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1000	5.00	0.0214	1.000000
0.1000	6.00	0.0277	1.000000
0.1000	7.00	0.0347	1.000000
0.1000	8.00	0.0423	1.000000
0.1000	9.00	0.0507	1.000000
0.1000	10.00	0.0600	1.000000
0.1000	11.00	0.0704	1.000000
0.1000	12.00	0.0823	1.000000
0.1000	13.00	0.0957	1.000000
0.1000	14.00	0.1113	1.000000
0.1000	15.00	0.1294	1.000000
0.1000	16.00	0.1506	1.000000
0.1000	17.00	0.1756	1.000000
0.1000	18.00	0.2051	1.000000
0.1000	19.00	0.2399	1.000000
0.1000	20.00	0.2807	1.000000
0.1000	21.00	0.3281	1.000000
0.1000	22.00	0.3823	1.000000
0.1000	23.00	0.4434	1.000000
0.1000	24.00	0.5112	1.000000
0.1000	25.00	0.5855	1.000000
0.1000	26.00	0.6055	0.596835
0.1000	27.00	0.6055	0.553444
0.1000	28.00	0.6055	0.514618
0.1000	29.00	0.6055	0.479739
0.1000	30.00	0.6055	0.448290
0.1000	31.00	0.6055	0.419834
0.1000	32.00	0.6055	0.394005
0.1000	33.00	0.6055	0.370488
0.1000	34.00	0.6055	0.349015
0.1000	35.00	0.6055	0.329356
0.1000	36.00	0.6055	0.311312
0.1000	37.00	0.6055	0.294712
0.1000	38.00	0.6055	0.279405
0.1000	39.00	0.6055	0.265260
0.1000	40.00	0.6055	0.252163
0.1000	42.00	0.6055	0.228719
0.1000	44.00	0.6055	0.208399
0.1000	46.00	0.6055	0.190672
0.1000	48.00	0.6055	0.175113
0.1000	50.00	0.6055	0.161384
0.1000	55.00	0.6055	0.133375
0.1000	60.00	0.6055	0.112072
0.1000	65.00	0.6055	0.095494
0.1000	70.00	0.6055	0.082339
0.1000	75.00	0.6055	0.071726
0.1000	80.00	0.6055	0.063041

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1100	5.00	0.0227	1.000000
0.1100	6.00	0.0296	1.000000
0.1100	7.00	0.0371	1.000000
0.1100	8.00	0.0453	1.000000
0.1100	9.00	0.0543	1.000000
0.1100	10.00	0.0643	1.000000
0.1100	11.00	0.0754	1.000000
0.1100	12.00	0.0880	1.000000
0.1100	13.00	0.1022	1.000000
0.1100	14.00	0.1186	1.000000
0.1100	15.00	0.1375	1.000000
0.1100	16.00	0.1594	1.000000
0.1100	17.00	0.1851	1.000000
0.1100	18.00	0.2152	1.000000
0.1100	19.00	0.2505	1.000000
0.1100	20.00	0.2915	1.000000
0.1100	21.00	0.3389	1.000000
0.1100	22.00	0.3928	1.000000
0.1100	23.00	0.4535	1.000000
0.1100	24.00	0.5207	1.000000
0.1100	25.00	0.5943	1.000000
0.1100	26.00	0.6001	0.578652
0.1100	27.00	0.6001	0.536583
0.1100	28.00	0.6001	0.498940
0.1100	29.00	0.6001	0.465123
0.1100	30.00	0.6001	0.434632
0.1100	31.00	0.6001	0.407044
0.1100	32.00	0.6001	0.382001
0.1100	33.00	0.6001	0.359200
0.1100	34.00	0.6001	0.338381
0.1100	35.00	0.6001	0.319321
0.1100	36.00	0.6001	0.301828
0.1100	37.00	0.6001	0.285733
0.1100	38.00	0.6001	0.270893
0.1100	39.00	0.6001	0.257179
0.1100	40.00	0.6001	0.244480
0.1100	42.00	0.6001	0.221751
0.1100	44.00	0.6001	0.202050
0.1100	46.00	0.6001	0.184862
0.1100	48.00	0.6001	0.169778
0.1100	50.00	0.6001	0.156467
0.1100	55.00	0.6001	0.129312
0.1100	60.00	0.6001	0.108658
0.1100	65.00	0.6001	0.092584
0.1100	70.00	0.6001	0.079830
0.1100	75.00	0.6001	0.069541
0.1100	80.00	0.6001	0.061120

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1200	5.00	0.0240	1.000000
0.1200	6.00	0.0314	1.000000
0.1200	7.00	0.0394	1.000000
0.1200	8.00	0.0481	1.000000
0.1200	9.00	0.0577	1.000000
0.1200	10.00	0.0683	1.000000
0.1200	11.00	0.0801	1.000000
0.1200	12.00	0.0934	1.000000
0.1200	13.00	0.1084	1.000000
0.1200	14.00	0.1255	1.000000
0.1200	15.00	0.1451	1.000000
0.1200	16.00	0.1678	1.000000
0.1200	17.00	0.1942	1.000000
0.1200	18.00	0.2248	1.000000
0.1200	19.00	0.2605	1.000000
0.1200	20.00	0.3017	1.000000
0.1200	21.00	0.3491	1.000000
0.1200	22.00	0.4028	1.000000
0.1200	23.00	0.4631	1.000000
0.1200	24.00	0.5298	1.000000
0.1200	25.00	0.5947	0.606797
0.1200	26.00	0.5947	0.561018
0.1200	27.00	0.5947	0.520231
0.1200	28.00	0.5947	0.483735
0.1200	29.00	0.5947	0.450949
0.1200	30.00	0.5947	0.421387
0.1200	31.00	0.5947	0.394639
0.1200	32.00	0.5947	0.370360
0.1200	33.00	0.5947	0.348254
0.1200	34.00	0.5947	0.328069
0.1200	35.00	0.5947	0.309590
0.1200	36.00	0.5947	0.292630
0.1200	37.00	0.5947	0.277026
0.1200	38.00	0.5947	0.262637
0.1200	39.00	0.5947	0.249341
0.1200	40.00	0.5947	0.237030
0.1200	42.00	0.5947	0.214993
0.1200	44.00	0.5947	0.195893
0.1200	46.00	0.5947	0.179229
0.1200	48.00	0.5947	0.164604
0.1200	50.00	0.5947	0.151699
0.1200	55.00	0.5947	0.125371
0.1200	60.00	0.5947	0.105347
0.1200	65.00	0.5947	0.089763
0.1200	70.00	0.5947	0.077398
0.1200	75.00	0.5947	0.067422
0.1200	80.00	0.5947	0.059258

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1300	5.00	0.0252	1.000000
0.1300	6.00	0.0330	1.000000
0.1300	7.00	0.0415	1.000000
0.1300	8.00	0.0508	1.000000
0.1300	9.00	0.0609	1.000000
0.1300	10.00	0.0722	1.000000
0.1300	11.00	0.0846	1.000000
0.1300	12.00	0.0985	1.000000
0.1300	13.00	0.1142	1.000000
0.1300	14.00	0.1320	1.000000
0.1300	15.00	0.1524	1.000000
0.1300	16.00	0.1757	1.000000
0.1300	17.00	0.2027	1.000000
0.1300	18.00	0.2339	1.000000
0.1300	19.00	0.2699	1.000000
0.1300	20.00	0.3114	1.000000
0.1300	21.00	0.3587	1.000000
0.1300	22.00	0.4123	1.000000
0.1300	23.00	0.4723	1.000000
0.1300	24.00	0.5386	1.000000
0.1300	25.00	0.5896	0.588311
0.1300	26.00	0.5896	0.543926
0.1300	27.00	0.5896	0.504381
0.1300	28.00	0.5896	0.468998
0.1300	29.00	0.5896	0.437211
0.1300	30.00	0.5896	0.408549
0.1300	31.00	0.5896	0.382616
0.1300	32.00	0.5896	0.359076
0.1300	33.00	0.5896	0.337644
0.1300	34.00	0.5896	0.318074
0.1300	35.00	0.5896	0.300159
0.1300	36.00	0.5896	0.283715
0.1300	37.00	0.5896	0.268586
0.1300	38.00	0.5896	0.254636
0.1300	39.00	0.5896	0.241745
0.1300	40.00	0.5896	0.229809
0.1300	42.00	0.5896	0.208443
0.1300	44.00	0.5896	0.189925
0.1300	46.00	0.5896	0.173769
0.1300	48.00	0.5896	0.159589
0.1300	50.00	0.5896	0.147078
0.1300	55.00	0.5896	0.121552
0.1300	60.00	0.5896	0.102137
0.1300	65.00	0.5896	0.087028
0.1300	70.00	0.5896	0.075040
0.1300	75.00	0.5896	0.065368
0.1300	80.00	0.5896	0.057452

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1400	5.00	0.0263	1.000000
0.1400	6.00	0.0345	1.000000
0.1400	7.00	0.0435	1.000000
0.1400	8.00	0.0533	1.000000
0.1400	9.00	0.0640	1.000000
0.1400	10.00	0.0758	1.000000
0.1400	11.00	0.0889	1.000000
0.1400	12.00	0.1035	1.000000
0.1400	13.00	0.1198	1.000000
0.1400	14.00	0.1383	1.000000
0.1400	15.00	0.1593	1.000000
0.1400	16.00	0.1833	1.000000
0.1400	17.00	0.2108	1.000000
0.1400	18.00	0.2425	1.000000
0.1400	19.00	0.2789	1.000000
0.1400	20.00	0.3206	1.000000
0.1400	21.00	0.3680	1.000000
0.1400	22.00	0.4214	1.000000
0.1400	23.00	0.4811	1.000000
0.1400	24.00	0.5470	1.000000
0.1400	25.00	0.5845	0.570402
0.1400	26.00	0.5845	0.527369
0.1400	27.00	0.5845	0.489028
0.1400	28.00	0.5845	0.454721
0.1400	29.00	0.5845	0.423902
0.1400	30.00	0.5845	0.396113
0.1400	31.00	0.5845	0.370969
0.1400	32.00	0.5845	0.348146
0.1400	33.00	0.5845	0.327366
0.1400	34.00	0.5845	0.308392
0.1400	35.00	0.5845	0.291022
0.1400	36.00	0.5845	0.275078
0.1400	37.00	0.5845	0.260410
0.1400	38.00	0.5845	0.246885
0.1400	39.00	0.5845	0.234386
0.1400	40.00	0.5845	0.222813
0.1400	42.00	0.5845	0.202098
0.1400	44.00	0.5845	0.184143
0.1400	46.00	0.5845	0.168479
0.1400	48.00	0.5845	0.154732
0.1400	50.00	0.5845	0.142601
0.1400	55.00	0.5845	0.117852
0.1400	60.00	0.5845	0.099028
0.1400	65.00	0.5845	0.084379
0.1400	70.00	0.5845	0.072755
0.1400	75.00	0.5845	0.063378
0.1400	80.00	0.5845	0.055703

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1500	5.00	0.0273	1.000000
0.1500	6.00	0.0360	1.000000
0.1500	7.00	0.0454	1.000000
0.1500	8.00	0.0557	1.000000
0.1500	9.00	0.0670	1.000000
0.1500	10.00	0.0793	1.000000
0.1500	11.00	0.0930	1.000000
0.1500	12.00	0.1081	1.000000
0.1500	13.00	0.1251	1.000000
0.1500	14.00	0.1442	1.000000
0.1500	15.00	0.1658	1.000000
0.1500	16.00	0.1904	1.000000
0.1500	17.00	0.2185	1.000000
0.1500	18.00	0.2507	1.000000
0.1500	19.00	0.2874	1.000000
0.1500	20.00	0.3293	1.000000
0.1500	21.00	0.3768	1.000000
0.1500	22.00	0.4302	1.000000
0.1500	23.00	0.4896	1.000000
0.1500	24.00	0.5552	1.000000
0.1500	25.00	0.5796	0.553064
0.1500	26.00	0.5796	0.511338
0.1500	27.00	0.5796	0.474163
0.1500	28.00	0.5796	0.440899
0.1500	29.00	0.5796	0.411016
0.1500	30.00	0.5796	0.384072
0.1500	31.00	0.5796	0.359693
0.1500	32.00	0.5796	0.337563
0.1500	33.00	0.5796	0.317415
0.1500	34.00	0.5796	0.299018
0.1500	35.00	0.5796	0.282175
0.1500	36.00	0.5796	0.266717
0.1500	37.00	0.5796	0.252494
0.1500	38.00	0.5796	0.239380
0.1500	39.00	0.5796	0.227262
0.1500	40.00	0.5796	0.216040
0.1500	42.00	0.5796	0.195955
0.1500	44.00	0.5796	0.178546
0.1500	46.00	0.5796	0.163358
0.1500	48.00	0.5796	0.150028
0.1500	50.00	0.5796	0.138266
0.1500	55.00	0.5796	0.114269
0.1500	60.00	0.5796	0.096018
0.1500	65.00	0.5796	0.081814
0.1500	70.00	0.5796	0.070544
0.1500	75.00	0.5796	0.061452
0.1500	80.00	0.5796	0.054010

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1600	5.00	0.0283	1.000000
0.1600	6.00	0.0374	1.000000
0.1600	7.00	0.0473	1.000000
0.1600	8.00	0.0580	1.000000
0.1600	9.00	0.0698	1.000000
0.1600	10.00	0.0827	1.000000
0.1600	11.00	0.0969	1.000000
0.1600	12.00	0.1126	1.000000
0.1600	13.00	0.1302	0.994898
0.1600	14.00	0.1499	0.974026
0.1600	15.00	0.1721	0.974026
0.1600	16.00	0.1973	0.974026
0.1600	17.00	0.2259	0.974026
0.1600	18.00	0.2585	0.974026
0.1600	19.00	0.2956	0.974026
0.1600	20.00	0.3377	0.974026
0.1600	21.00	0.3852	0.974026
0.1600	22.00	0.4385	0.974026
0.1600	23.00	0.4978	0.974026
0.1600	24.00	0.5631	0.974026
0.1600	25.00	0.5748	0.536285
0.1600	26.00	0.5748	0.495826
0.1600	27.00	0.5748	0.459778
0.1600	28.00	0.5748	0.427523
0.1600	29.00	0.5748	0.398547
0.1600	30.00	0.5748	0.372421
0.1600	31.00	0.5748	0.348781
0.1600	32.00	0.5748	0.327323
0.1600	33.00	0.5748	0.307786
0.1600	34.00	0.5748	0.289947
0.1600	35.00	0.5748	0.273615
0.1600	36.00	0.5748	0.258625
0.1600	37.00	0.5748	0.244834
0.1600	38.00	0.5748	0.232118
0.1600	39.00	0.5748	0.220367
0.1600	40.00	0.5748	0.209486
0.1600	42.00	0.5748	0.190010
0.1600	44.00	0.5748	0.173129
0.1600	46.00	0.5748	0.158402
0.1600	48.00	0.5748	0.145477
0.1600	50.00	0.5748	0.134071
0.1600	55.00	0.5748	0.110803
0.1600	60.00	0.5748	0.093105
0.1600	65.00	0.5748	0.079332
0.1600	70.00	0.5748	0.068404
0.1600	75.00	0.5748	0.059587
0.1600	80.00	0.5748	0.052372

3 KBYTES USED JOB-STEP RETURN CODE = 0

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

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C       STRESS REDUCTION FACTORS
C        WALLS IN SINGLE CURVATURE
C       PROGRAMMER: ADNAN A. AWNI
C
        REAL E(18)
        REAL SLEND(47)
    001 FORMAT(9F8.4)
    002 FORMAT(10F8.3)
    003 FORMAT('1',// T21,'FINAL    COMPUTED    RESULTS'/)
    004 FORMAT(   22X,'WALLS IN SINGLE CURVATURE')
    005 FORMAT(   /13X,'E/T',12X,'H/T',10X,'PHY',13X,'V'/)
    006 FORMAT(   12X,F6.4,8X,F6.2,8X,F6.4,8X,F8.6)
    007 FORMAT(   23X,'STRESS REDUCTION FACTOR'/)
    021 FORMAT(   22X,'ECC. AT ONE END EQUAL ZERO'/)
    008 READ(5,1) (E(I),I=1,18)
    009 READ(5,2) (SLEND(J),J=1,47)
    010 DO 260 I=1,18
    011 DO 260 J=1,47
    012 REALK=((SLEND(J))**2)/(55.5)
    013 IF(SLEND(J)-10)55,14,14
    014 A=16.655
    015 B=33.31*E(I)-(33.31)
    016 IF(B)17,19,19
    017 BPOST=-(B)
    018 GOTO 20
    019 BPOST=(B)
    020 C=(16.655*(E(I))**2)+((9.0)/(8.0))*REALK
    030 XROOT=(BPOST**2)-(4.0*(A)*(C))
    031 IF(XROOT)160,32,32
    032 PHY1=(-(B)+(BPOST**2-4.0*(A)*(C))**0.5)/(2.*(A))
    033 PHY2=(-(B)-(BPOST**2-4.0*(A)*(C))**0.5)/(2.*(A))
    034 PHYB=((1.-E(I))+(4.*E(I)**2-2.*E(I)+1.0)
       1**0.5)/(3.0)
    035 IF(PHY1)70,36,36
    036 IF(PHY2)37,37,39
    037 PHY2=0
    038 GOTO 42
    039 PHY2=PHY2
    040 PHYMIN=(PHY1-PHY2)
    041 IF(PHYMIN)42,42,51
    042 PHY1=PHY1
    050 GOTO 57
    051 PHY1=PHY2
    052 IF(PHY1)53,57,57
    053 PHY1=-PHY1
    054 GOTO 57
    055 PHY1=(9.*REALK*(SLEND(J)/2.))/((8.*((SLEND(J)/2.)-1.5))
       1  *33.31*(1.-2.*E(I)))
    056 PHYB=((1.0-E(I))+(4.0*E(I)**2-2.0*E(I)+1.0)**0.5)/(3.0)
    057 CHECK1=(PHY1-PHYB)
    058 IF(CHECK1)59,70,70
    059 PHY=PHY1
    060 GOTO 72
    070 PHY=PHYB
    071 GOTO 170
C
C
C
C
```

```
      C
      C
      C
      C
        072 CHECK2=(PHY-E(I))
        073 IF(CHECK2)74,74,90
        074 V1=(((9.0)/(8.0))*(1.0-2.0*E(I))*((SLEND(J))/(2.0)))
           2/((SLEND(J)/(2.0))-(1.5))
        075 VMAX=1.0
        076 IF(V1-VMAX)77,77,79
        077 V=V1
        078 GOTO 200
        079 V=VMAX
        080 GOTO 200
        090 V2=((9.0)/(8.0))*((1.0-((E(I)+PHY)**2)/(2.0*PHY)))
        100 VMAX=1.0
        110 IF(V2-VMAX)120,120,140
        120 V=V2
        130 GOTO 200
        140 V=VMAX
        150 GOTO 200
        160 PHY=((1.0-E(I))+(4.*E(I)**2-2.*E(I)+1.0)**0.5)/(3.0)
        170 CHECK3=(PHY-E(I))
        180 IF(CHECK3)190,192,192
        190 Z=(33.31*PHY)*((1.-2.*E(I))**2)
        191 GOTO 193
        192 Z=33.31*PHY*(1.-((E(I)+PHY)**2)/(2.*PHY))**2
        193 V3=Z/REALK
        194 VMAX=1.0
        195 CHECK4=(V3-VMAX)
        196 IF(CHECK4)197,197,199
        197 V=V3
        198 GOTO 200
        199 V=VMAX
        200 IF(SLEND(J)-5.0)210,210,250
        210 WRITE(6,3)
        220 WRITE(6,7)
        230 WRITE(6,4)
        231 WRITE(6,21)
        240 WRITE(6,5)
        250 WRITE(6,6) (E(I),SLEND(J),PHY,V)
        260 CONTINUE
        270 STOP
        280 END

      C      END OF PROGRAM
      C      INPUT DATA FOR ABOVE PROGRAM :
      C      ECCENTRICITY/WALL THICKNESS 'E/T' VALUES :
      C  .1666    .18      920      .22      .24      .26      .28      .30
      C  .34      .36      .38      .40      .42      .44      .46      .48
      C       SLENDERNESS RATIO 'H/T' VALUES :
      C  5.0     6.0     7.0     8.0     9.0     10.0    11.0    12.0    13.0
      C 15.0    16.0    17.0    18.0    19.0    20.0    21.0    22.0    23.0
      C 25.0    26.0    27.0    28.0    29.0    30.0    31.0    32.0    33.0
      C 35.0    36.0    37.0    38.0    39.0    40.0    42.0    44.0    46.0
      C 50.0    52.0    54.0    56.0    58.0    60.0    65.0    70.0    75.0
      C      END OF INPUT DATA.

CODE+GLA+SYMTABS+ARRAYS = 3680+ 736+ 296+ 264= 4976 BYTES

*COMPILATION SUCCESSFUL
```

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1666	5.00	0.0570	1.000000
0.1666	6.00	0.0657	1.000000
0.1666	7.00	0.0783	1.000000
0.1666	8.00	0.0935	1.000000
0.1666	9.00	0.1109	1.000000
0.1666	10.00	0.0951	1.000000
0.1666	11.00	0.1126	1.000000
0.1666	12.00	0.1323	1.000000
0.1666	13.00	0.1543	0.975195
0.1666	14.00	0.1790	0.749668
0.1666	15.00	0.2065	0.745807
0.1666	16.00	0.2374	0.738276
0.1666	17.00	0.2721	0.727144
0.1666	18.00	0.3114	0.712271
0.1666	19.00	0.3565	0.693252
0.1666	20.00	0.4092	0.669261
0.1666	21.00	0.4727	0.638631
0.1666	22.00	0.5546	0.597470
0.1666	23.00	0.5718	0.547067
0.1666	24.00	0.5718	0.502428
0.1666	25.00	0.5718	0.463037
0.1666	26.00	0.5718	0.428104
0.1666	27.00	0.5718	0.396980
0.1666	28.00	0.5718	0.369131
0.1666	29.00	0.5718	0.344112
0.1666	30.00	0.5718	0.321554
0.1666	31.00	0.5718	0.301143
0.1666	32.00	0.5718	0.282616
0.1666	33.00	0.5718	0.265747
0.1666	34.00	0.5718	0.250345
0.1666	35.00	0.5718	0.236244
0.1666	36.00	0.5718	0.223301
0.1666	37.00	0.5718	0.211394
0.1666	38.00	0.5718	0.200414
0.1666	39.00	0.5718	0.190269
0.1666	40.00	0.5718	0.180874
0.1666	42.00	0.5718	0.164058
0.1666	44.00	0.5718	0.149483
0.1666	46.00	0.5718	0.136767
0.1666	48.00	0.5718	0.125607
0.1666	50.00	0.5718	0.115759
0.1666	55.00	0.5718	0.095669
0.1666	60.00	0.5718	0.080388
0.1666	65.00	0.5718	0.068497
0.1666	70.00	0.5718	0.059061
0.1666	75.00	0.5718	0.051449
0.1666	80.00	0.5718	0.045218

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.1800	5.00	0.0594	1.000000
0.1800	6.00	0.0685	1.000000
0.1800	7.00	0.0815	1.000000
0.1800	8.00	0.0974	1.000000
0.1800	9.00	0.1155	1.000000
0.1800	10.00	0.1001	1.000000
0.1800	11.00	0.1180	0.990000
0.1800	12.00	0.1383	0.960000
0.1800	13.00	0.1610	0.936000
0.1800	14.00	0.1864	0.719377
0.1800	15.00	0.2149	0.716814
0.1800	16.00	0.2469	0.709801
0.1800	17.00	0.2831	0.698881
0.1800	18.00	0.3243	0.683864
0.1800	19.00	0.3721	0.664227
0.1800	20.00	0.4286	0.638878
0.1800	21.00	0.4986	0.605466
0.1800	22.00	0.5658	0.558743
0.1800	23.00	0.5658	0.511212
0.1800	24.00	0.5658	0.469499
0.1800	25.00	0.5658	0.432690
0.1800	26.00	0.5658	0.400046
0.1800	27.00	0.5658	0.370962
0.1800	28.00	0.5658	0.344938
0.1800	29.00	0.5658	0.321559
0.1800	30.00	0.5658	0.300480
0.1800	31.00	0.5658	0.281406
0.1800	32.00	0.5658	0.264093
0.1800	33.00	0.5658	0.248330
0.1800	34.00	0.5658	0.233937
0.1800	35.00	0.5658	0.220760
0.1800	36.00	0.5658	0.208666
0.1800	37.00	0.5658	0.197539
0.1800	38.00	0.5658	0.187279
0.1800	39.00	0.5658	0.177798
0.1800	40.00	0.5658	0.169020
0.1800	42.00	0.5658	0.153306
0.1800	44.00	0.5658	0.139686
0.1800	46.00	0.5658	0.127803
0.1800	48.00	0.5658	0.117375
0.1800	50.00	0.5658	0.108173
0.1800	55.00	0.5658	0.089399
0.1800	60.00	0.5658	0.075120
0.1800	65.00	0.5658	0.064007
0.1800	70.00	0.5658	0.055190
0.1800	75.00	0.5658	0.048077
0.1800	80.00	0.5658	0.042255

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.2000	5.00	0.0634	1.000000
0.2000	6.00	0.0730	1.000000
0.2000	7.00	0.0870	1.000000
0.2000	8.00	0.1039	1.000000
0.2000	9.00	0.1232	1.000000
0.2000	10.00	0.1084	0.964286
0.2000	11.00	0.1271	0.928125
0.2000	12.00	0.1483	0.900000
0.2000	13.00	0.1721	0.877500
0.2000	14.00	0.1988	0.859091
0.2000	15.00	0.2289	0.672948
0.2000	16.00	0.2629	0.666525
0.2000	17.00	0.3017	0.655705
0.2000	18.00	0.3465	0.640162
0.2000	19.00	0.3992	0.619086
0.2000	20.00	0.4636	0.590697
0.2000	21.00	0.5485	0.550468
0.2000	22.00	0.5573	0.501675
0.2000	23.00	0.5573	0.458999
0.2000	24.00	0.5573	0.421546
0.2000	25.00	0.5573	0.388497
0.2000	26.00	0.5573	0.359187
0.2000	27.00	0.5573	0.333074
0.2000	28.00	0.5573	0.309707
0.2000	29.00	0.5573	0.288717
0.2000	30.00	0.5573	0.269790
0.2000	31.00	0.5573	0.252665
0.2000	32.00	0.5573	0.237120
0.2000	33.00	0.5573	0.222967
0.2000	34.00	0.5573	0.210044
0.2000	35.00	0.5573	0.198213
0.2000	36.00	0.5573	0.187354
0.2000	37.00	0.5573	0.177364
0.2000	38.00	0.5573	0.168151
0.2000	39.00	0.5573	0.159639
0.2000	40.00	0.5573	0.151757
0.2000	42.00	0.5573	0.137648
0.2000	44.00	0.5573	0.125419
0.2000	46.00	0.5573	0.114750
0.2000	48.00	0.5573	0.105387
0.2000	50.00	0.5573	0.097124
0.2000	55.00	0.5573	0.080268
0.2000	60.00	0.5573	0.067447
0.2000	65.00	0.5573	0.057470
0.2000	70.00	0.5573	0.049553
0.2000	75.00	0.5573	0.043166
0.2000	80.00	0.5573	0.037939

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.2200	5.00	0.0679	1.000000
0.2200	6.00	0.0782	1.000000
0.2200	7.00	0.0932	1.000000
0.2200	8.00	0.1113	1.000000
0.2200	9.00	0.1320	0.945000
0.2200	10.00	0.1180	0.900000
0.2200	11.00	0.1376	0.866250
0.2200	12.00	0.1597	0.840000
0.2200	13.00	0.1848	0.619000
0.2200	14.00	0.2130	0.801618
0.2200	15.00	0.2451	0.628559
0.2200	16.00	0.2816	0.622427
0.2200	17.00	0.3236	0.611332
0.2200	18.00	0.3730	0.594708
0.2200	19.00	0.4327	0.571200
0.2200	20.00	0.5095	0.557473
0.2200	21.00	0.5494	0.489998
0.2200	22.00	0.5494	0.446465
0.2200	23.00	0.5494	0.408486
0.2200	24.00	0.5494	0.375154
0.2200	25.00	0.5494	0.345742
0.2200	26.00	0.5494	0.319658
0.2200	27.00	0.5494	0.296418
0.2200	28.00	0.5494	0.275624
0.2200	29.00	0.5494	0.256943
0.2200	30.00	0.5494	0.240099
0.2200	31.00	0.5494	0.224859
0.2200	32.00	0.5494	0.211025
0.2200	33.00	0.5494	0.198429
0.2200	34.00	0.5494	0.186928
0.2200	35.00	0.5494	0.176399
0.2200	36.00	0.5494	0.166735
0.2200	37.00	0.5494	0.157844
0.2200	38.00	0.5494	0.149646
0.2200	39.00	0.5494	0.142070
0.2200	40.00	0.5494	0.135056
0.2200	42.00	0.5494	0.122499
0.2200	44.00	0.5494	0.111616
0.2200	46.00	0.5494	0.102121
0.2200	48.00	0.5494	0.093789
0.2200	50.00	0.5494	0.086436
0.2200	55.00	0.5494	0.071434
0.2200	60.00	0.5494	0.060025
0.2200	65.00	0.5494	0.051145
0.2200	70.00	0.5494	0.044100
0.2200	75.00	0.5494	0.038416
0.2200	80.00	0.5494	0.033764

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.2400	5.00	0.0731	1.000000
0.2400	6.00	0.0843	1.000000
0.2400	7.00	0.1003	1.000000
0.2400	8.00	0.1198	0.936000
0.2400	9.00	0.1422	0.877500
0.2400	10.00	0.1289	0.835714
0.2400	11.00	0.1495	0.804375
0.2400	12.00	0.1729	0.780000
0.2400	13.00	0.1994	0.760500
0.2400	14.00	0.2295	0.744545
0.2400	15.00	0.2639	0.583787
0.2400	16.00	0.3035	0.577536
0.2400	17.00	0.3498	0.565615
0.2400	18.00	0.4055	0.547005
0.2400	19.00	0.4760	0.519170
0.2400	20.00	0.5421	0.475891
0.2400	21.00	0.5421	0.431647
0.2400	22.00	0.5421	0.393298
0.2400	23.00	0.5421	0.359842
0.2400	24.00	0.5421	0.330480
0.2400	25.00	0.5421	0.304570
0.2400	26.00	0.5421	0.281592
0.2400	27.00	0.5421	0.261120
0.2400	28.00	0.5421	0.242801
0.2400	29.00	0.5421	0.226345
0.2400	30.00	0.5421	0.211507
0.2400	31.00	0.5421	0.198082
0.2400	32.00	0.5421	0.185895
0.2400	33.00	0.5421	0.174799
0.2400	34.00	0.5421	0.164668
0.2400	35.00	0.5421	0.155393
0.2400	36.00	0.5421	0.146880
0.2400	37.00	0.5421	0.139043
0.2400	38.00	0.5421	0.131826
0.2400	39.00	0.5421	0.125152
0.2400	40.00	0.5421	0.118973
0.2400	42.00	0.5421	0.107912
0.2400	44.00	0.5421	0.098325
0.2400	46.00	0.5421	0.089960
0.2400	48.00	0.5421	0.082620
0.2400	50.00	0.5421	0.076143
0.2400	55.00	0.5421	0.062928
0.2400	60.00	0.5421	0.052877
0.2400	65.00	0.5421	0.045055
0.2400	70.00	0.5421	0.038848
0.2400	75.00	0.5421	0.033841
0.2400	80.00	0.5421	0.029743

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.2600	5.00	0.0792	1.000000
0.2600	6.00	0.0913	1.000000
0.2600	7.00	0.1087	0.945000
0.2600	8.00	0.1298	0.864000
0.2600	9.00	0.1540	0.810000
0.2600	10.00	0.1414	0.771426
0.2600	11.00	0.1632	0.742500
0.2600	12.00	0.1880	0.720000
0.2600	13.00	0.2162	0.702000
0.2600	14.00	0.2486	0.687273
0.2600	15.00	0.2860	0.538675
0.2600	16.00	0.3296	0.531734
0.2600	17.00	0.3819	0.518127
0.2600	18.00	0.4473	0.495882
0.2600	19.00	0.5354	0.459034
0.2600	20.00	0.5354	0.414279
0.2600	21.00	0.5354	0.375763
0.2600	22.00	0.5354	0.342379
0.2600	23.00	0.5354	0.313254
0.2600	24.00	0.5354	0.287693
0.2600	25.00	0.5354	0.265138
0.2600	26.00	0.5354	0.245135
0.2600	27.00	0.5354	0.227313
0.2600	28.00	0.5354	0.211367
0.2600	29.00	0.5354	0.197041
0.2600	30.00	0.5354	0.184124
0.2600	31.00	0.5354	0.172436
0.2600	32.00	0.5354	0.161828
0.2600	33.00	0.5354	0.152168
0.2600	34.00	0.5354	0.143349
0.2600	35.00	0.5354	0.135275
0.2600	36.00	0.5354	0.127864
0.2600	37.00	0.5354	0.121046
0.2600	38.00	0.5354	0.114759
0.2600	39.00	0.5354	0.108949
0.2600	40.00	0.5354	0.103570
0.2600	42.00	0.5354	0.093941
0.2600	44.00	0.5354	0.085595
0.2600	46.00	0.5354	0.078314
0.2600	48.00	0.5354	0.071923
0.2600	50.00	0.5354	0.066285
0.2600	55.00	0.5354	0.054781
0.2600	60.00	0.5354	0.046031
0.2600	65.00	0.5354	0.039222
0.2600	70.00	0.5354	0.033819
0.2600	75.00	0.5354	0.029460
0.2600	80.00	0.5354	0.025892

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.2800	5.00	0.0864	1.000000
0.2800	6.00	0.0996	0.990000
0.2800	7.00	0.1186	0.866250
0.2800	8.00	0.1416	0.792000
0.2800	9.00	0.1680	0.742500
0.2800	10.00	0.1558	0.707143
0.2800	11.00	0.1790	0.680625
0.2800	12.00	0.2055	0.660000
0.2800	13.00	0.2359	0.643500
0.2800	14.00	0.2712	0.630000
0.2800	15.00	0.3124	0.493113
0.2800	16.00	0.3616	0.484636
0.2800	17.00	0.4229	0.467838
0.2800	18.00	0.5063	0.438106
0.2800	19.00	0.5294	0.394069
0.2800	20.00	0.5294	0.355647
0.2800	21.00	0.5294	0.322582
0.2800	22.00	0.5294	0.293923
0.2800	23.00	0.5294	0.268920
0.2800	24.00	0.5294	0.246977
0.2800	25.00	0.5294	0.227614
0.2800	26.00	0.5294	0.210442
0.2800	27.00	0.5294	0.195142
0.2800	28.00	0.5294	0.181453
0.2800	29.00	0.5294	0.169154
0.2800	30.00	0.5294	0.158065
0.2800	31.00	0.5294	0.148032
0.2800	32.00	0.5294	0.138925
0.2800	33.00	0.5294	0.130633
0.2800	34.00	0.5294	0.123061
0.2800	35.00	0.5294	0.116130
0.2800	36.00	0.5294	0.109768
0.2800	37.00	0.5294	0.103914
0.2800	38.00	0.5294	0.098517
0.2800	39.00	0.5294	0.093530
0.2800	40.00	0.5294	0.088912
0.2800	42.00	0.5294	0.080646
0.2800	44.00	0.5294	0.073481
0.2800	46.00	0.5294	0.067230
0.2800	48.00	0.5294	0.061744
0.2800	50.00	0.5294	0.056904
0.2800	55.00	0.5294	0.047028
0.2800	60.00	0.5294	0.039516
0.2800	65.00	0.5294	0.033671
0.2800	70.00	0.5294	0.029032
0.2800	75.00	0.5294	0.025290
0.2800	80.00	0.5294	0.022228

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.3000	5.00	0.0951	1.000000
0.3000	6.00	0.1095	0.900000
0.3000	7.00	0.1305	0.787500
0.3000	8.00	0.1553	0.720000
0.3000	9.00	0.1848	0.675000
0.3000	10.00	0.1725	0.642857
0.3000	11.00	0.1973	0.618750
0.3000	12.00	0.2259	0.600000
0.3000	13.00	0.2592	0.585000
0.3000	14.00	0.2982	0.572727
0.3000	15.00	0.3448	0.448724
0.3000	16.00	0.4026	0.435286
0.3000	17.00	0.4803	0.411928
0.3000	18.00	0.5239	0.370715
0.3000	19.00	0.5239	0.332719
0.3000	20.00	0.5239	0.300279
0.3000	21.00	0.5239	0.272362
0.3000	22.00	0.5239	0.248165
0.3000	23.00	0.5239	0.227054
0.3000	24.00	0.5239	0.208527
0.3000	25.00	0.5239	0.192179
0.3000	26.00	0.5239	0.177680
0.3000	27.00	0.5239	0.164762
0.3000	28.00	0.5239	0.153204
0.3000	29.00	0.5239	0.142820
0.3000	30.00	0.5239	0.133457
0.3000	31.00	0.5239	0.124986
0.3000	32.00	0.5239	0.117297
0.3000	33.00	0.5239	0.110295
0.3000	34.00	0.5239	0.103903
0.3000	35.00	0.5239	0.098050
0.3000	36.00	0.5239	0.092679
0.3000	37.00	0.5239	0.087737
0.3000	38.00	0.5239	0.083180
0.3000	39.00	0.5239	0.078969
0.3000	40.00	0.5239	0.075070
0.3000	42.00	0.5239	0.068090
0.3000	44.00	0.5239	0.062041
0.3000	46.00	0.5239	0.056764
0.3000	48.00	0.5239	0.052132
0.3000	50.00	0.5239	0.048045
0.3000	55.00	0.5239	0.039706
0.3000	60.00	0.5239	0.033364
0.3000	65.00	0.5239	0.028429
0.3000	70.00	0.5239	0.024513
0.3000	75.00	0.5239	0.021353
0.3000	80.00	0.5239	0.018767

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.3200	5.00	0.1056	1.000000
0.3200	6.00	0.1217	0.809999
0.3200	7.00	0.1449	0.708750
0.3200	8.00	0.1731	0.648000
0.3200	9.00	0.2054	0.607500
0.3200	10.00	0.1918	0.578571
0.3200	11.00	0.2188	0.556875
0.3200	12.00	0.2502	0.540000
0.3200	13.00	0.2872	0.526500
0.3200	14.00	0.3315	0.404776
0.3200	15.00	0.3865	0.398569
0.3200	16.00	0.4599	0.381053
0.3200	17.00	0.5191	0.343907
0.3200	18.00	0.5191	0.306756
0.3200	19.00	0.5191	0.275316
0.3200	20.00	0.5191	0.248472
0.3200	21.00	0.5191	0.225372
0.3200	22.00	0.5191	0.205349
0.3200	23.00	0.5191	0.187881
0.3200	24.00	0.5191	0.172550
0.3200	25.00	0.5191	0.159022
0.3200	26.00	0.5191	0.147025
0.3200	27.00	0.5191	0.136336
0.3200	28.00	0.5191	0.126772
0.3200	29.00	0.5191	0.118180
0.3200	30.00	0.5191	0.110432
0.3200	31.00	0.5191	0.103422
0.3200	32.00	0.5191	0.097060
0.3200	33.00	0.5191	0.091266
0.3200	34.00	0.5191	0.085977
0.3200	35.00	0.5191	0.081134
0.3200	36.00	0.5191	0.076689
0.3200	37.00	0.5191	0.072600
0.3200	38.00	0.5191	0.068829
0.3200	39.00	0.5191	0.065345
0.3200	40.00	0.5191	0.062118
0.3200	42.00	0.5191	0.056343
0.3200	44.00	0.5191	0.051337
0.3200	46.00	0.5191	0.046970
0.3200	48.00	0.5191	0.043138
0.3200	50.00	0.5191	0.039756
0.3200	55.00	0.5191	0.032856
0.3200	60.00	0.5191	0.027608
0.3200	65.00	0.5191	0.023524
0.3200	70.00	0.5191	0.020283
0.3200	75.00	0.5191	0.017669
0.3200	80.00	0.5191	0.015530

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.3400	5.00	0.1189	0.900000
0.3400	6.00	0.1369	0.720000
0.3400	7.00	0.1631	0.630000
0.3400	8.00	0.1947	0.576000
0.3400	9.00	0.2311	0.540000
0.3400	10.00	0.2147	0.514285
0.3400	11.00	0.2444	0.495000
0.3400	12.00	0.2795	0.480000
0.3400	13.00	0.3219	0.468000
0.3400	14.00	0.3746	0.358203
0.3400	15.00	0.4452	0.346028
0.3400	16.00	0.5148	0.313350
0.3400	17.00	0.5148	0.277569
0.3400	18.00	0.5148	0.247585
0.3400	19.00	0.5148	0.222209
0.3400	20.00	0.5148	0.200544
0.3400	21.00	0.5148	0.181899
0.3400	22.00	0.5148	0.165739
0.3400	23.00	0.5148	0.151640
0.3400	24.00	0.5148	0.139266
0.3400	25.00	0.5148	0.128348
0.3400	26.00	0.5148	0.118665
0.3400	27.00	0.5148	0.110038
0.3400	28.00	0.5148	0.102318
0.3400	29.00	0.5148	0.095383
0.3400	30.00	0.5148	0.089131
0.3400	31.00	0.5148	0.083473
0.3400	32.00	0.5148	0.078337
0.3400	33.00	0.5148	0.073662
0.3400	34.00	0.5148	0.069392
0.3400	35.00	0.5148	0.065484
0.3400	36.00	0.5148	0.061896
0.3400	37.00	0.5148	0.058596
0.3400	38.00	0.5148	0.055552
0.3400	39.00	0.5148	0.052740
0.3400	40.00	0.5148	0.050136
0.3400	42.00	0.5148	0.045475
0.3400	44.00	0.5148	0.041435
0.3400	46.00	0.5148	0.037910
0.3400	48.00	0.5148	0.034817
0.3400	50.00	0.5148	0.032087
0.3400	55.00	0.5148	0.026518
0.3400	60.00	0.5148	0.022283
0.3400	65.00	0.5148	0.018986
0.3400	70.00	0.5148	0.016371
0.3400	75.00	0.5148	0.014261
0.3400	80.00	0.5148	0.012534

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.3600	5.00	0.1358	0.787500
0.3600	6.00	0.1565	0.650000
0.3600	7.00	0.1864	0.551250
0.3600	8.00	0.2225	0.504000
0.3600	9.00	0.2641	0.472500
0.3600	10.00	0.2421	0.450000
0.3600	11.00	0.2757	0.433125
0.3600	12.00	0.3164	0.420000
0.3600	13.00	0.3674	0.314916
0.3600	14.00	0.4364	0.307477
0.3600	15.00	0.5112	0.278804
0.3600	16.00	0.5112	0.245043
0.3600	17.00	0.5112	0.217062
0.3600	18.00	0.5112	0.193614
0.3600	19.00	0.5112	0.173770
0.3600	20.00	0.5112	0.156827
0.3600	21.00	0.5112	0.142247
0.3600	22.00	0.5112	0.129609
0.3600	23.00	0.5112	0.118584
0.3600	24.00	0.5112	0.108908
0.3600	25.00	0.5112	0.100369
0.3600	26.00	0.5112	0.092797
0.3600	27.00	0.5112	0.086051
0.3600	28.00	0.5112	0.080014
0.3600	29.00	0.5112	0.074591
0.3600	30.00	0.5112	0.069701
0.3600	31.00	0.5112	0.065277
0.3600	32.00	0.5112	0.061261
0.3600	33.00	0.5112	0.057604
0.3600	34.00	0.5112	0.054266
0.3600	35.00	0.5112	0.051209
0.3600	36.00	0.5112	0.048404
0.3600	37.00	0.5112	0.045822
0.3600	38.00	0.5112	0.043442
0.3600	39.00	0.5112	0.041243
0.3600	40.00	0.5112	0.039207
0.3600	42.00	0.5112	0.035562
0.3600	44.00	0.5112	0.032402
0.3600	46.00	0.5112	0.029646
0.3600	48.00	0.5112	0.027227
0.3600	50.00	0.5112	0.025092
0.3600	55.00	0.5112	0.020737
0.3600	60.00	0.5112	0.017425
0.3600	65.00	0.5112	0.014848
0.3600	70.00	0.5112	0.012802
0.3600	75.00	0.5112	0.011152
0.3600	80.00	0.5112	0.009802

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.3800	5.00	0.1585	0.675000
0.3800	6.00	0.1826	0.540000
0.3800	7.00	0.2174	0.472500
0.3800	8.00	0.2596	0.432000
0.3800	9.00	0.3081	0.405000
0.3800	10.00	0.2761	0.385714
0.3800	11.00	0.3155	0.371250
0.3800	12.00	0.3656	0.360000
0.3800	13.00	0.4348	0.266121
0.3800	14.00	0.5081	0.240150
0.3800	15.00	0.5081	0.209197
0.3800	16.00	0.5081	0.183865
0.3800	17.00	0.5081	0.162870
0.3800	18.00	0.5081	0.145276
0.3800	19.00	0.5081	0.130386
0.3800	20.00	0.5081	0.117673
0.3800	21.00	0.5081	0.106733
0.3800	22.00	0.5081	0.097251
0.3800	23.00	0.5081	0.088978
0.3800	24.00	0.5081	0.081718
0.3800	25.00	0.5081	0.075311
0.3800	26.00	0.5081	0.069629
0.3800	27.00	0.5081	0.064567
0.3800	28.00	0.5081	0.060037
0.3800	29.00	0.5081	0.055968
0.3800	30.00	0.5081	0.052299
0.3800	31.00	0.5081	0.048980
0.3800	32.00	0.5081	0.045966
0.3800	33.00	0.5081	0.043223
0.3800	34.00	0.5081	0.040717
0.3800	35.00	0.5081	0.038424
0.3800	36.00	0.5081	0.036319
0.3800	37.00	0.5081	0.034382
0.3800	38.00	0.5081	0.032597
0.3800	39.00	0.5081	0.030946
0.3800	40.00	0.5081	0.029418
0.3800	42.00	0.5081	0.026683
0.3800	44.00	0.5081	0.024313
0.3800	46.00	0.5081	0.022244
0.3800	48.00	0.5081	0.020429
0.3800	50.00	0.5081	0.018828
0.3800	55.00	0.5081	0.015560
0.3800	60.00	0.5081	0.013075
0.3800	65.00	0.5081	0.011141
0.3800	70.00	0.5081	0.009606
0.3800	75.00	0.5081	0.008368
0.3800	80.00	0.5081	0.007355

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.4000	5.00	0.1902	0.562500
0.4000	6.00	0.2191	0.450000
0.4000	7.00	0.2609	0.393750
0.4000	8.00	0.3116	0.360000
0.4000	9.00	0.3697	0.337500
0.4000	10.00	0.3202	0.321429
0.4000	11.00	0.3704	0.309375
0.4000	12.00	0.4427	0.222683
0.4000	13.00	0.5055	0.197507
0.4000	14.00	0.5055	0.170299
0.4000	15.00	0.5055	0.148349
0.4000	16.00	0.5055	0.130385
0.4000	17.00	0.5055	0.115497
0.4000	18.00	0.5055	0.103020
0.4000	19.00	0.5055	0.092462
0.4000	20.00	0.5055	0.083447
0.4000	21.00	0.5055	0.075688
0.4000	22.00	0.5055	0.068964
0.4000	23.00	0.5055	0.063098
0.4000	24.00	0.5055	0.057949
0.4000	25.00	0.5055	0.053406
0.4000	26.00	0.5055	0.049377
0.4000	27.00	0.5055	0.045787
0.4000	28.00	0.5055	0.042575
0.4000	29.00	0.5055	0.039689
0.4000	30.00	0.5055	0.037087
0.4000	31.00	0.5055	0.034733
0.4000	32.00	0.5055	0.032596
0.4000	33.00	0.5055	0.030651
0.4000	34.00	0.5055	0.028874
0.4000	35.00	0.5055	0.027248
0.4000	36.00	0.5055	0.025755
0.4000	37.00	0.5055	0.024382
0.4000	38.00	0.5055	0.023115
0.4000	39.00	0.5055	0.021945
0.4000	40.00	0.5055	0.020862
0.4000	42.00	0.5055	0.018922
0.4000	44.00	0.5055	0.017241
0.4000	46.00	0.5055	0.015774
0.4000	48.00	0.5055	0.014487
0.4000	50.00	0.5055	0.013351
0.4000	55.00	0.5055	0.011034
0.4000	60.00	0.5055	0.009272
0.4000	65.00	0.5055	0.007900
0.4000	70.00	0.5055	0.006812
0.4000	75.00	0.5055	0.005934
0.4000	80.00	0.5055	0.005215

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.4200	5.00	0.2377	0.450000
0.4200	6.00	0.2738	0.360000
0.4200	7.00	0.3261	0.315000
0.4200	8.00	0.3895	0.288000
0.4200	9.00	0.4621	0.177842
0.4200	10.00	0.5843	0.257143
0.4200	11.00	0.4672	0.177323
0.4200	12.00	0.5035	0.151468
0.4200	13.00	0.5035	0.129061
0.4200	14.00	0.5035	0.111282
0.4200	15.00	0.5035	0.096939
0.4200	16.00	0.5035	0.085200
0.4200	17.00	0.5035	0.075472
0.4200	18.00	0.5035	0.067319
0.4200	19.00	0.5035	0.060419
0.4200	20.00	0.5035	0.054528
0.4200	21.00	0.5035	0.049459
0.4200	22.00	0.5035	0.045065
0.4200	23.00	0.5035	0.041231
0.4200	24.00	0.5035	0.037867
0.4200	25.00	0.5035	0.034898
0.4200	26.00	0.5035	0.032265
0.4200	27.00	0.5035	0.029920
0.4200	28.00	0.5035	0.027821
0.4200	29.00	0.5035	0.025935
0.4200	30.00	0.5035	0.024235
0.4200	31.00	0.5035	0.022697
0.4200	32.00	0.5035	0.021300
0.4200	33.00	0.5035	0.020029
0.4200	34.00	0.5035	0.018868
0.4200	35.00	0.5035	0.017805
0.4200	36.00	0.5035	0.016830
0.4200	37.00	0.5035	0.015932
0.4200	38.00	0.5035	0.015105
0.4200	39.00	0.5035	0.014340
0.4200	40.00	0.5035	0.013632
0.4200	42.00	0.5035	0.012365
0.4200	44.00	0.5035	0.011266
0.4200	46.00	0.5035	0.010308
0.4200	48.00	0.5035	0.009467
0.4200	50.00	0.5035	0.008725
0.4200	55.00	0.5035	0.007210
0.4200	60.00	0.5035	0.006059
0.4200	65.00	0.5035	0.005162
0.4200	70.00	0.5035	0.004451
0.4200	75.00	0.5035	0.003878
0.4200	80.00	0.5035	0.003408

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.4400	5.00	0.3169	0.337500
0.4400	6.00	0.3651	0.270000
0.4400	7.00	0.4346	0.236250
0.4400	8.00	0.5019	0.195699
0.4400	9.00	0.5019	0.154626
0.4400	10.00	0.5019	0.125247
0.4400	11.00	0.5019	0.103510
0.4400	12.00	0.5019	0.086977
0.4400	13.00	0.5019	0.074111
0.4400	14.00	0.5019	0.063902
0.4400	15.00	0.5019	0.055665
0.4400	16.00	0.5019	0.048925
0.4400	17.00	0.5019	0.043338
0.4400	18.00	0.5019	0.038657
0.4400	19.00	0.5019	0.034695
0.4400	20.00	0.5019	0.031312
0.4400	21.00	0.5019	0.028401
0.4400	22.00	0.5019	0.025878
0.4400	23.00	0.5019	0.023676
0.4400	24.00	0.5019	0.021744
0.4400	25.00	0.5019	0.020040
0.4400	26.00	0.5019	0.018528
0.4400	27.00	0.5019	0.017181
0.4400	28.00	0.5019	0.015975
0.4400	29.00	0.5019	0.014893
0.4400	30.00	0.5019	0.013916
0.4400	31.00	0.5019	0.013033
0.4400	32.00	0.5019	0.012231
0.4400	33.00	0.5019	0.011501
0.4400	34.00	0.5019	0.010835
0.4400	35.00	0.5019	0.010224
0.4400	36.00	0.5019	0.009664
0.4400	37.00	0.5019	0.009149
0.4400	38.00	0.5019	0.008674
0.4400	39.00	0.5019	0.008235
0.4400	40.00	0.5019	0.007828
0.4400	42.00	0.5019	0.007100
0.4400	44.00	0.5019	0.006469
0.4400	46.00	0.5019	0.005919
0.4400	48.00	0.5019	0.005436
0.4400	50.00	0.5019	0.005010
0.4400	55.00	0.5019	0.004140
0.4400	60.00	0.5019	0.003479
0.4400	65.00	0.5019	0.002964
0.4400	70.00	0.5019	0.002556
0.4400	75.00	0.5019	0.002227
0.4400	80.00	0.5019	0.001957

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.4600	5.00	0.4754	0.089719
0.4600	6.00	0.5008	0.157823
0.4600	7.00	0.5008	0.115951
0.4600	8.00	0.5008	0.088775
0.4600	9.00	0.5008	0.070143
0.4600	10.00	0.5008	0.056816
0.4600	11.00	0.5008	0.046956
0.4600	12.00	0.5008	0.039456
0.4600	13.00	0.5008	0.033619
0.4600	14.00	0.5008	0.028988
0.4600	15.00	0.5008	0.025252
0.4600	16.00	0.5008	0.022194
0.4600	17.00	0.5008	0.019660
0.4600	18.00	0.5008	0.017536
0.4600	19.00	0.5008	0.015739
0.4600	20.00	0.5008	0.014204
0.4600	21.00	0.5008	0.012883
0.4600	22.00	0.5008	0.011739
0.4600	23.00	0.5008	0.010740
0.4600	24.00	0.5008	0.009864
0.4600	25.00	0.5008	0.009091
0.4600	26.00	0.5008	0.008405
0.4600	27.00	0.5008	0.007794
0.4600	28.00	0.5008	0.007247
0.4600	29.00	0.5008	0.006756
0.4600	30.00	0.5008	0.006313
0.4600	31.00	0.5008	0.005912
0.4600	32.00	0.5008	0.005548
0.4600	33.00	0.5008	0.005217
0.4600	34.00	0.5008	0.004915
0.4600	35.00	0.5008	0.004638
0.4600	36.00	0.5008	0.004384
0.4600	37.00	0.5008	0.004150
0.4600	38.00	0.5008	0.003935
0.4600	39.00	0.5008	0.003735
0.4600	40.00	0.5008	0.003551
0.4600	42.00	0.5008	0.003221
0.4600	44.00	0.5008	0.002935
0.4600	46.00	0.5008	0.002685
0.4600	48.00	0.5008	0.002466
0.4600	50.00	0.5008	0.002273
0.4600	55.00	0.5008	0.001878
0.4600	60.00	0.5008	0.001578
0.4600	65.00	0.5008	0.001345
0.4600	70.00	0.5008	0.001160
0.4600	75.00	0.5008	0.001010
0.4600	80.00	0.5008	0.000888

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.4800	5.00	0.5002	0.057981
0.4800	6.00	0.5002	0.040265
0.4800	7.00	0.5002	0.029582
0.4800	8.00	0.5002	0.022649
0.4800	9.00	0.5002	0.017895
0.4800	10.00	0.5002	0.014495
0.4800	11.00	0.5002	0.011980
0.4800	12.00	0.5002	0.010066
0.4800	13.00	0.5002	0.008577
0.4800	14.00	0.5002	0.007396
0.4800	15.00	0.5002	0.006442
0.4800	16.00	0.5002	0.005662
0.4800	17.00	0.5002	0.005016
0.4800	18.00	0.5002	0.004474
0.4800	19.00	0.5002	0.004015
0.4800	20.00	0.5002	0.003624
0.4800	21.00	0.5002	0.003287
0.4800	22.00	0.5002	0.002995
0.4800	23.00	0.5002	0.002740
0.4800	24.00	0.5002	0.002517
0.4800	25.00	0.5002	0.002319
0.4800	26.00	0.5002	0.002144
0.4800	27.00	0.5002	0.001988
0.4800	28.00	0.5002	0.001849
0.4800	29.00	0.5002	0.001724
0.4800	30.00	0.5002	0.001611
0.4800	31.00	0.5002	0.001508
0.4800	32.00	0.5002	0.001416
0.4800	33.00	0.5002	0.001331
0.4800	34.00	0.5002	0.001254
0.4800	35.00	0.5002	0.001183
0.4800	36.00	0.5002	0.001118
0.4800	37.00	0.5002	0.001059
0.4800	38.00	0.5002	0.001004
0.4800	39.00	0.5002	0.000953
0.4800	40.00	0.5002	0.000906
0.4800	42.00	0.5002	0.000822
0.4800	44.00	0.5002	0.000749
0.4800	46.00	0.5002	0.000685
0.4800	48.00	0.5002	0.000629
0.4800	50.00	0.5002	0.000580
0.4800	55.00	0.5002	0.000479
0.4800	60.00	0.5002	0.000403
0.4800	65.00	0.5002	0.000343
0.4800	70.00	0.5002	0.000296
0.4800	75.00	0.5002	0.000258
0.4800	80.00	0.5002	0.000226

FINAL COMPUTED RESULTS

STRESS REDUCTION FACTOR

WALLS IN SINGLE CURVATURE
ECC. AT ONE END EQUAL ZERO

E/T	H/T	PHY	V
0.4999	5.00	0.5000	0.000001
0.4999	6.00	0.5000	0.000001
0.4999	7.00	0.5000	0.000001
0.4999	8.00	0.5000	0.000001
0.4999	9.00	0.5000	0.000000
0.4999	10.00	0.5000	0.000000
0.4999	11.00	0.5000	0.000000
0.4999	12.00	0.5000	0.000000
0.4999	13.00	0.5000	0.000000
0.4999	14.00	0.5000	0.000000
0.4999	15.00	0.5000	0.000000
0.4999	16.00	0.5000	0.000000
0.4999	17.00	0.5000	0.000000
0.4999	18.00	0.5000	0.000000
0.4999	19.00	0.5000	0.000000
0.4999	20.00	0.5000	0.000000
0.4999	21.00	0.5000	0.000000
0.4999	22.00	0.5000	0.000000
0.4999	23.00	0.5000	0.000000
0.4999	24.00	0.5000	0.000000
0.4999	25.00	0.5000	0.000000
0.4999	26.00	0.5000	0.000000
0.4999	27.00	0.5000	0.000000
0.4999	28.00	0.5000	0.000000
0.4999	29.00	0.5000	0.000000
0.4999	30.00	0.5000	0.000000
0.4999	31.00	0.5000	0.000000
0.4999	32.00	0.5000	0.000000
0.4999	33.00	0.5000	0.000000
0.4999	34.00	0.5000	0.000000
0.4999	35.00	0.5000	0.000000
0.4999	36.00	0.5000	0.000000
0.4999	37.00	0.5000	0.000000
0.4999	38.00	0.5000	0.000000
0.4999	39.00	0.5000	0.000000
0.4999	40.00	0.5000	0.000000
0.4999	42.00	0.5000	0.000000
0.4999	44.00	0.5000	0.000000
0.4999	46.00	0.5000	0.000000
0.4999	48.00	0.5000	0.000000
0.4999	50.00	0.5000	0.000000
0.4999	55.00	0.5000	0.000000
0.4999	60.00	0.5000	0.000000
0.4999	65.00	0.5000	0.000000
0.4999	70.00	0.5000	0.000000
0.4999	75.00	0.5000	0.000000
0.4999	80.00	0.5000	0.000000

OP

3 KBYTES USED JOB-STEP RETURN CODE = 0

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
C       STRESS REDUCTION FACTORS
C       WALLS IN SINGLE CURVATURE
C        EQUAL ECC. AT BOTH ENDS
C       PROGRAMMER: ADNAN A. AWNI
C
C
        REAL E(28)
        REAL SLEND(50)
    001 FORMAT(7F10.4)
    002 FORMAT(10F7.2)
    003 FORMAT('1',// T21,'FINAL    COMPUTED    RESULTS')
    004 FORMAT(    22X,'WALLS IN SINGLE CURVATURE')
    005 FORMAT(    /13X,'E/T',12X,'H/T',10X,'PHY',13X,'V'/)
    006 FORMAT(    12X,F6.4,8X,F6.2,8X,F6.4,8X,F8.6)
    007 FORMAT(    23X,'STRESS REDUCTION FACTOR')
    008 FORMAT(    24X,'EQUAL END ECCENTRICITY')
    023 READ(5,1) (E(I),I=1,28)
    024 READ(5,2) (SLEND(J),J=1,50)
    025 DO 237 I=1,28
    026 DO 237 J=1,50
    027 REALK=(SLEND(J))**2*((1.0)/(55.5))
    028 BKERN=0.1666
    029 IF(E(I)-BKERN)40,120,120
    040 A1=(3.0*9.869)
    041 B1=(12.0*9.869*E(I))+(2.0*9.869)-(3.0*REALK)
    042 C1=(-12.0*REALK*E(I))
    043 IF(B1)44,46,46
    044 BPOST=-(B1)
    045 GOTO  47
    046 BPOST=(B1)
    047 XROOT=(BPOST**2)-(4.0*(A1)*(C1))
    048 IF(XROOT)200,49,49
    049 PHY1=(-(B1)+(BPOST**2-4.0*(A1)*(C1))**0.5)/(2.0*A1)
    050 IF(PHY1)86,60,60
    060 PHY2=(-(B1)-(BPOST**2-4.0*(A1)*(C1))**0.5)/(2.0*A1)
    061 PHYB=((2.0)/(3.0))*(1.0-2.0*E(I))
    062 IF(PHY2)63,63,65
    063 PHY2=0
    064 GOTO 68
    065 PHY2=PHY2
    066 PHYMIN=(PHY1-PHY2)
    067 IF(PHYMIN)68,68,70
    068 PHY1=PHY1
    069 GOTO 80
    070 PHY1=PHY2
    080 CHECK1=(PHY1-PHYB)
    081 IF(CHECK1)82,84,84
    082 PHY=PHY1
    083 GOTO 87
    084 PHY=PHYB
    085 GOTO 112
    086 PHY=PHY1
C
C
C
C
C
C
C
```

```
C
C
C
C
      087 V1=(3.0)/(2.0*(1.0+6.0*E(I)+((3.0*PHY)/(2.0))))
      088 VMAX=1.0
      089 IF(V1-VMAX)90,110,110
      090 V=V1
      100 GOTO 236
      110 V=VMAX
      111 GOTO 236
      112 Z3=(9.869*PHY)/(4.0*E(I)+PHY)
      113 V5=(Z3)/(REALK)
      114 VMAX=1.0
      115 IF(V5-VMAX)116,116,118
      116 V=V5
      117 GOTO 236
      118 V=VMAX
      119 GOTO 236
      120 A2=16.655
      130 B2=66.62*E(I)-33.31
      131 C2=((9.0)/(8.0))*REALK
      132 IF(B2)133,135,135
      133 BPOST=-(B2)
      134 GOTO 136
      135 BPOST=(B2)
      136 XROOT=(BPOST**2)-(4.0*(A2)*(C2))
      137 IF(XROOT)200,138,138
      138 PHY1=(-(B2)+(BPOST**2-4.0*(A2)*(C2))**0.5)/(2.*A2)
      139 IF(PHY1)175,140,140
      140 PHY2=(-(B2)-(BPOST**2-4.0*(A2)*(C2))**0.5)/(2.*A2)
      150 PHYB=((2.0)/(3.0))*(1.0-2.0*E(I))
      151 IF(PHY2)152,152,154
      152 PHY2=0
      153 GOTO 157
      154 PHY2=PHY2
      155 PHYMIN=(PHY1-PHY2)
      156 IF(PHYMIN)157,157,159
      157 PHY1=PHY1
      158 GOTO 160
      159 PHY1=PHY2
      160 CHECK2=(PHY1-PHYB)
      170 IF(CHECK2)171,173,173
      171 PHY=PHY1
      172 GOTO 176
      173 PHY=PHYB
      174 GOTO 192
      175 PHY=PHY1
      176 V2=((9.0)/(8.0))*(1.0-2.0*E(I)-((PHY)/(2.0)))
      177 VMAX=1.0
      178 IF(V2-VMAX)179,179,190
      179 V=V2
      180 GOTO 236
      190 V=VMAX
      191 GOTO 236
      192 Z4=(33.31*PHY)*(1.0-2.0*E(I)-(PHY/2.0))**2
      193 V6=(Z4)/(REALK)
      194 VMAX=1.0
      195 IF(V6-VMAX)196,196,198
      196 V=V6
      197 GOTO 236
C
C
C
C
```

```
C
C
C
C
C
C
      198 V=VMAX
      199 GOTO 236
      200 PHY=((2.0)/(3.0))*(1.0-2.0*E(I))
      210 BKERN=0.1666
      211 IF(E(I)-BKERN)212,220,220
      212 Z1=(9.869*PHY)/(4.0*E(I)+PHY)
      213 V3=(Z1)/(REALK)
      214 VMAX=1.0
      215 IF(V3-VMAX)216,216,218
      216 V=V3
      217 GOTO 236
      218 V=VMAX
      219 GOTO 236
      220 Z2=(33.31*PHY)*(1.0-2.0*E(I)-(PHY/2.0))**2
      230 V4=(Z2)/(REALK)
      231 VMAX=1.0
      232 IF(V4-VMAX)233,233,235
      233 V=V4
      234 GOTO 236
      235 V=VMAX
      236 IF(SLEND(J)-5.0)9,9,221
      009 WRITE(6,3)
      010 WRITE(6,7)
      020 WRITE(6,8)
      021 WRITE(6,4)
      022 WRITE(6,5)
      221 WRITE(6,6) (E(I),SLEND(J),PHY,V)
      237 CONTINUE
      238 STOP
      239 END
```

```
C         END OF PROGRAM
C
C         INPUT DATA FOR ABOVE PROGRAM :
C
C         ECCENTRICITY/WALL THICKNESS 'E/T' VALUES :
C  00.01        .02         .04         .06         .08         .10
C    .14        .16         .1666       .18         .20         .22
C    .26        .28         .30         .32         .3333       .34
C    .38        .40         .42         .44         .46         .48
C          SLENDERNESS RATIO 'H/T' VALUES :
C  5.0     6.0     7.0     8.0     9.0    10.0    11.0    12.0    13.0
C 15.0    16.0    17.0    18.0    19.0    20.0    21.0    22.0    23.0
C 25.0    26.0    27.0    28.0    29.0    30.0    31.0    32.0    33.0
C 35.0    36.0    37.0    38.0    39.0    40.0    42.0    44.0    46.0
C 50.0    52.0    54.0    56.0    58.0    60.0    65.0    70.0    75.0
C         END OF INPUT DATA.
```

CODE+GLA+SYMTABS+ARRAYS = 4680+ 760+ 392+ 312= 6144 BYTES

*COMPILATION SUCCESSFUL

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.0100	5.00	0.0028	1.000000
0.0100	6.00	0.0041	1.000000
0.0100	7.00	0.0057	1.000000
0.0100	8.00	0.0078	1.000000
0.0100	9.00	0.0104	1.000000
0.0100	10.00	0.0136	1.000000
0.0100	11.00	0.0176	1.000000
0.0100	12.00	0.0226	1.000000
0.0100	13.00	0.0289	1.000000
0.0100	14.00	0.0371	1.000000
0.0100	15.00	0.0478	1.000000
0.0100	16.00	0.0620	1.000000
0.0100	17.00	0.0811	1.000000
0.0100	18.00	0.1067	1.000000
0.0100	19.00	0.1403	1.000000
0.0100	20.00	0.1831	1.000000
0.0100	21.00	0.2353	1.000000
0.0100	22.00	0.2963	0.997062
0.0100	23.00	0.3650	0.933140
0.0100	24.00	0.4405	0.871751
0.0100	25.00	0.5219	0.813978
0.0100	26.00	0.6086	0.760284
0.0100	27.00	0.6533	0.707997
0.0100	28.00	0.6533	0.658329
0.0100	29.00	0.6533	0.613710
0.0100	30.00	0.6533	0.573478
0.0100	31.00	0.6533	0.537076
0.0100	32.00	0.6533	0.504033
0.0100	33.00	0.6533	0.473948
0.0100	34.00	0.6533	0.446479
0.0100	35.00	0.6533	0.421330
0.0100	36.00	0.6533	0.398248
0.0100	37.00	0.6533	0.377012
0.0100	38.00	0.6533	0.357431
0.0100	39.00	0.6533	0.339336
0.0100	40.00	0.6533	0.322581
0.0100	42.00	0.6533	0.292591
0.0100	44.00	0.6533	0.266596
0.0100	46.00	0.6533	0.243918
0.0100	48.00	0.6533	0.224015
0.0100	50.00	0.6533	0.206452
0.0100	52.00	0.6533	0.190876
0.0100	54.00	0.6533	0.176999
0.0100	56.00	0.6533	0.164582
0.0100	58.00	0.6533	0.153427
0.0100	60.00	0.6533	0.143369
0.0100	65.00	0.6533	0.122161
0.0100	70.00	0.6533	0.105333
0.0100	75.00	0.6533	0.091756
0.0100	80.00	0.6533	0.080645

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.0200	5.00	0.0052	1.000000
0.0200	6.00	0.0076	1.000000
0.0200	7.00	0.0107	1.000000
0.0200	8.00	0.0145	1.000000
0.0200	9.00	0.0191	1.000000
0.0200	10.00	0.0248	1.000000
0.0200	11.00	0.0317	1.000000
0.0200	12.00	0.0401	1.000000
0.0200	13.00	0.0505	1.000000
0.0200	14.00	0.0633	1.000000
0.0200	15.00	0.0792	1.000000
0.0200	16.00	0.0989	1.000000
0.0200	17.00	0.1233	1.000000
0.0200	18.00	0.1534	1.000000
0.0200	19.00	0.1900	1.000000
0.0200	20.00	0.2337	1.000000
0.0200	21.00	0.2847	0.969578
0.0200	22.00	0.3430	0.917670
0.0200	23.00	0.4083	0.865789
0.0200	24.00	0.4802	0.815113
0.0200	25.00	0.5580	0.766478
0.0200	26.00	0.6400	0.720223
0.0200	27.00	0.6400	0.667861
0.0200	28.00	0.6400	0.621009
0.0200	29.00	0.6400	0.578919
0.0200	30.00	0.6400	0.540968
0.0200	31.00	0.6400	0.506629
0.0200	32.00	0.6400	0.475460
0.0200	33.00	0.6400	0.447081
0.0200	34.00	0.6400	0.421169
0.0200	35.00	0.6400	0.397445
0.0200	36.00	0.6400	0.375672
0.0200	37.00	0.6400	0.355640
0.0200	38.00	0.6400	0.337168
0.0200	39.00	0.6400	0.320099
0.0200	40.00	0.6400	0.304294
0.0200	42.00	0.6400	0.276004
0.0200	44.00	0.6400	0.251483
0.0200	46.00	0.6400	0.230090
0.0200	48.00	0.6400	0.211315
0.0200	50.00	0.6400	0.194748
0.0200	52.00	0.6400	0.180056
0.0200	54.00	0.6400	0.166965
0.0200	56.00	0.6400	0.155252
0.0200	58.00	0.6400	0.144730
0.0200	60.00	0.6400	0.135242
0.0200	65.00	0.6400	0.115236
0.0200	70.00	0.6400	0.099361
0.0200	75.00	0.6400	0.086555
0.0200	80.00	0.6400	0.076074

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.0400	5.00	0.0092	1.000000
0.0400	6.00	0.0136	1.000000
0.0400	7.00	0.0189	1.000000
0.0400	8.00	0.0254	1.000000
0.0400	9.00	0.0332	1.000000
0.0400	10.00	0.0425	1.000000
0.0400	11.00	0.0536	1.000000
0.0400	12.00	0.0667	1.000000
0.0400	13.00	0.0822	1.000000
0.0400	14.00	0.1006	1.000000
0.0400	15.00	0.1222	1.000000
0.0400	16.00	0.1475	1.000000
0.0400	17.00	0.1772	0.996099
0.0400	18.00	0.2118	0.962981
0.0400	19.00	0.2516	0.927431
0.0400	20.00	0.2970	0.889930
0.0400	21.00	0.3483	0.851075
0.0400	22.00	0.4056	0.811527
0.0400	23.00	0.4688	0.771935
0.0400	24.00	0.5378	0.732883
0.0400	25.00	0.6125	0.694852
0.0400	26.00	0.6133	0.642613
0.0400	27.00	0.6133	0.595893
0.0400	28.00	0.6133	0.554089
0.0400	29.00	0.6133	0.516535
0.0400	30.00	0.6133	0.482674
0.0400	31.00	0.6133	0.452036
0.0400	32.00	0.6133	0.424225
0.0400	33.00	0.6133	0.398904
0.0400	34.00	0.6133	0.375784
0.0400	35.00	0.6133	0.354617
0.0400	36.00	0.6133	0.335190
0.0400	37.00	0.6133	0.317316
0.0400	38.00	0.6133	0.300835
0.0400	39.00	0.6133	0.285606
0.0400	40.00	0.6133	0.271504
0.0400	42.00	0.6133	0.246262
0.0400	44.00	0.6133	0.224383
0.0400	46.00	0.6133	0.205296
0.0400	48.00	0.6133	0.188544
0.0400	50.00	0.6133	0.173762
0.0400	52.00	0.6133	0.160653
0.0400	54.00	0.6133	0.148973
0.0400	56.00	0.6133	0.138522
0.0400	58.00	0.6133	0.129134
0.0400	60.00	0.6133	0.120668
0.0400	65.00	0.6133	0.102818
0.0400	70.00	0.6133	0.088654
0.0400	75.00	0.6133	0.077228
0.0400	80.00	0.6133	0.067876

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.0600	5.00	0.0125	1.000000
0.0600	6.00	0.0184	1.000000
0.0600	7.00	0.0255	1.000000
0.0600	8.00	0.0340	1.000000
0.0600	9.00	0.0442	1.000000
0.0600	10.00	0.0562	1.000000
0.0600	11.00	0.0701	1.000000
0.0600	12.00	0.0864	1.000000
0.0600	13.00	0.1053	0.988198
0.0600	14.00	0.1271	0.967370
0.0600	15.00	0.1521	0.944461
0.0600	16.00	0.1809	0.919505
0.0600	17.00	0.2137	0.892601
0.0600	18.00	0.2508	0.863927
0.0600	19.00	0.2927	0.833740
0.0600	20.00	0.3396	0.802365
0.0600	21.00	0.3917	0.770173
0.0600	22.00	0.4492	0.737563
0.0600	23.00	0.5119	0.704925
0.0600	24.00	0.5801	0.672621
0.0600	25.00	0.5867	0.621938
0.0600	26.00	0.5867	0.575017
0.0600	27.00	0.5867	0.533212
0.0600	28.00	0.5867	0.495805
0.0600	29.00	0.5867	0.462201
0.0600	30.00	0.5867	0.431902
0.0600	31.00	0.5867	0.404486
0.0600	32.00	0.5867	0.379601
0.0600	33.00	0.5867	0.356943
0.0600	34.00	0.5867	0.336256
0.0600	35.00	0.5867	0.317315
0.0600	36.00	0.5867	0.299932
0.0600	37.00	0.5867	0.283938
0.0600	38.00	0.5867	0.269191
0.0600	39.00	0.5867	0.255563
0.0600	40.00	0.5867	0.242945
0.0600	42.00	0.5867	0.220358
0.0600	44.00	0.5867	0.200781
0.0600	46.00	0.5867	0.183701
0.0600	48.00	0.5867	0.168711
0.0600	50.00	0.5867	0.155484
0.0600	52.00	0.5867	0.143754
0.0600	54.00	0.5867	0.133303
0.0600	56.00	0.5867	0.123951
0.0600	58.00	0.5867	0.115550
0.0600	60.00	0.5867	0.107975
0.0600	65.00	0.5867	0.092003
0.0600	70.00	0.5867	0.079329
0.0600	75.00	0.5867	0.069104
0.0600	80.00	0.5867	0.060736

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.0800	5.00	0.0153	0.998065
0.0800	6.00	0.0223	0.991116
0.0800	7.00	0.0308	0.982789
0.0800	8.00	0.0410	0.973032
0.0800	9.00	0.0531	0.961790
0.0800	10.00	0.0671	0.949010
0.0800	11.00	0.0833	0.934641
0.0800	12.00	0.1019	0.918645
0.0800	13.00	0.1232	0.900999
0.0800	14.00	0.1475	0.881705
0.0800	15.00	0.1751	0.860794
0.0800	16.00	0.2062	0.838341
0.0800	17.00	0.2411	0.814459
0.0800	18.00	0.2803	0.789309
0.0800	19.00	0.3238	0.763094
0.0800	20.00	0.3719	0.736051
0.0800	21.00	0.4249	0.708445
0.0800	22.00	0.4827	0.680548
0.0800	23.00	0.5456	0.652630
0.0800	24.00	0.5600	0.605130
0.0800	25.00	0.5600	0.557688
0.0800	26.00	0.5600	0.515614
0.0800	27.00	0.5600	0.478128
0.0800	28.00	0.5600	0.444586
0.0800	29.00	0.5600	0.414453
0.0800	30.00	0.5600	0.387284
0.0800	31.00	0.5600	0.362701
0.0800	32.00	0.5600	0.340386
0.0800	33.00	0.5600	0.320069
0.0800	34.00	0.5600	0.301518
0.0800	35.00	0.5600	0.284535
0.0800	36.00	0.5600	0.268947
0.0800	37.00	0.5600	0.254606
0.0800	38.00	0.5600	0.241382
0.0800	39.00	0.5600	0.229162
0.0800	40.00	0.5600	0.217847
0.0800	42.00	0.5600	0.197594
0.0800	44.00	0.5600	0.180039
0.0800	46.00	0.5600	0.164724
0.0800	48.00	0.5600	0.151283
0.0800	50.00	0.5600	0.139422
0.0800	52.00	0.5600	0.128904
0.0800	54.00	0.5600	0.119532
0.0800	56.00	0.5600	0.111146
0.0800	58.00	0.5600	0.103613
0.0800	60.00	0.5600	0.096821
0.0800	65.00	0.5600	0.082498
0.0800	70.00	0.5600	0.071134
0.0800	75.00	0.5600	0.061965
0.0800	80.00	0.5600	0.054462

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.1000	5.00	0.0176	0.922301
0.1000	6.00	0.0256	0.915519
0.1000	7.00	0.0353	0.907436
0.1000	8.00	0.0469	0.898022
0.1000	9.00	0.0604	0.887252
0.1000	10.00	0.0761	0.875101
0.1000	11.00	0.0940	0.861554
0.1000	12.00	0.1145	0.846607
0.1000	13.00	0.1378	0.830270
0.1000	14.00	0.1640	0.812573
0.1000	15.00	0.1935	0.793571
0.1000	16.00	0.2264	0.773345
0.1000	17.00	0.2631	0.752006
0.1000	18.00	0.3038	0.729692
0.1000	19.00	0.3486	0.706568
0.1000	20.00	0.3979	0.682819
0.1000	21.00	0.4516	0.658642
0.1000	22.00	0.5100	0.634243
0.1000	23.00	0.5333	0.591660
0.1000	24.00	0.5333	0.543382
0.1000	25.00	0.5333	0.500781
0.1000	26.00	0.5333	0.463000
0.1000	27.00	0.5333	0.429339
0.1000	28.00	0.5333	0.399220
0.1000	29.00	0.5333	0.372162
0.1000	30.00	0.5333	0.347765
0.1000	31.00	0.5333	0.325690
0.1000	32.00	0.5333	0.305653
0.1000	33.00	0.5333	0.287409
0.1000	34.00	0.5333	0.270751
0.1000	35.00	0.5333	0.255501
0.1000	36.00	0.5333	0.241503
0.1000	37.00	0.5333	0.228626
0.1000	38.00	0.5333	0.216751
0.1000	39.00	0.5333	0.205778
0.1000	40.00	0.5333	0.195618
0.1000	42.00	0.5333	0.177431
0.1000	44.00	0.5333	0.161668
0.1000	46.00	0.5333	0.147915
0.1000	48.00	0.5333	0.135846
0.1000	50.00	0.5333	0.125195
0.1000	52.00	0.5333	0.115750
0.1000	54.00	0.5333	0.107335
0.1000	56.00	0.5333	0.099805
0.1000	58.00	0.5333	0.093041
0.1000	60.00	0.5333	0.086941
0.1000	65.00	0.5333	0.074080
0.1000	70.00	0.5333	0.063875
0.1000	75.00	0.5333	0.055642
0.1000	80.00	0.5333	0.048904

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.1200	5.00	0.0196	0.857473
0.1200	6.00	0.0284	0.850989
0.1200	7.00	0.0392	0.843289
0.1200	8.00	0.0519	0.834365
0.1200	9.00	0.0666	0.824204
0.1200	10.00	0.0836	0.812805
0.1200	11.00	0.1031	0.800171
0.1200	12.00	0.1251	0.786317
0.1200	13.00	0.1499	0.771269
0.1200	14.00	0.1777	0.755072
0.1200	15.00	0.2087	0.737787
0.1200	16.00	0.2432	0.719495
0.1200	17.00	0.2813	0.700297
0.1200	18.00	0.3232	0.680312
0.1200	19.00	0.3692	0.659676
0.1200	20.00	0.4194	0.638539
0.1200	21.00	0.4739	0.617057
0.1200	22.00	0.5067	0.581129
0.1200	23.00	0.5067	0.531695
0.1200	24.00	0.5067	0.488310
0.1200	25.00	0.5067	0.450027
0.1200	26.00	0.5067	0.416075
0.1200	27.00	0.5067	0.385825
0.1200	28.00	0.5067	0.358758
0.1200	29.00	0.5067	0.334443
0.1200	30.00	0.5067	0.312519
0.1200	31.00	0.5067	0.292681
0.1200	32.00	0.5067	0.274674
0.1200	33.00	0.5067	0.258280
0.1200	34.00	0.5067	0.243310
0.1200	35.00	0.5067	0.229605
0.1200	36.00	0.5067	0.217027
0.1200	37.00	0.5067	0.205454
0.1200	38.00	0.5067	0.194783
0.1200	39.00	0.5067	0.184922
0.1200	40.00	0.5067	0.175792
0.1200	42.00	0.5067	0.159448
0.1200	44.00	0.5067	0.145282
0.1200	46.00	0.5067	0.132924
0.1200	48.00	0.5067	0.122077
0.1200	50.00	0.5067	0.112507
0.1200	52.00	0.5067	0.104019
0.1200	54.00	0.5067	0.096456
0.1200	56.00	0.5067	0.089690
0.1200	58.00	0.5067	0.083611
0.1200	60.00	0.5067	0.078130
0.1200	65.00	0.5067	0.066572
0.1200	70.00	0.5067	0.057401
0.1200	75.00	0.5067	0.050003
0.1200	80.00	0.5067	0.043948

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.1400	5.00	0.0213	0.801330
0.1400	6.00	0.0309	0.795198
0.1400	7.00	0.0425	0.787939
0.1400	8.00	0.0561	0.779553
0.1400	9.00	0.0720	0.770042
0.1400	10.00	0.0901	0.759413
0.1400	11.00	0.1108	0.747684
0.1400	12.00	0.1341	0.734879
0.1400	13.00	0.1602	0.721034
0.1400	14.00	0.1894	0.706197
0.1400	15.00	0.2217	0.690430
0.1400	16.00	0.2574	0.673810
0.1400	17.00	0.2967	0.656428
0.1400	18.00	0.3398	0.638387
0.1400	19.00	0.3867	0.619803
0.1400	20.00	0.4378	0.600799
0.1400	21.00	0.4800	0.573239
0.1400	22.00	0.4800	0.522311
0.1400	23.00	0.4800	0.477880
0.1400	24.00	0.4800	0.438886
0.1400	25.00	0.4800	0.404477
0.1400	26.00	0.4800	0.373962
0.1400	27.00	0.4800	0.346774
0.1400	28.00	0.4800	0.322447
0.1400	29.00	0.4800	0.300593
0.1400	30.00	0.4800	0.280887
0.1400	31.00	0.4800	0.263058
0.1400	32.00	0.4800	0.246874
0.1400	33.00	0.4800	0.232138
0.1400	34.00	0.4800	0.218684
0.1400	35.00	0.4800	0.206366
0.1400	36.00	0.4800	0.195061
0.1400	37.00	0.4800	0.184659
0.1400	38.00	0.4800	0.175068
0.1400	39.00	0.4800	0.166205
0.1400	40.00	0.4800	0.157999
0.1400	42.00	0.4800	0.143310
0.1400	44.00	0.4800	0.130578
0.1400	46.00	0.4800	0.119470
0.1400	48.00	0.4800	0.109722
0.1400	50.00	0.4800	0.101119
0.1400	52.00	0.4800	0.093491
0.1400	54.00	0.4800	0.086694
0.1400	56.00	0.4800	0.080612
0.1400	58.00	0.4800	0.075148
0.1400	60.00	0.4800	0.070222
0.1400	65.00	0.4800	0.059834
0.1400	70.00	0.4800	0.051592
0.1400	75.00	0.4800	0.044942
0.1400	80.00	0.4800	0.039500

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.1600	5.00	0.0228	0.752208
0.1600	6.00	0.0330	0.746445
0.1600	7.00	0.0453	0.739637
0.1600	8.00	0.0598	0.731793
0.1600	9.00	0.0766	0.722921
0.1600	10.00	0.0958	0.713038
0.1600	11.00	0.1175	0.702165
0.1600	12.00	0.1419	0.690334
0.1600	13.00	0.1692	0.677582
0.1600	14.00	0.1994	0.663961
0.1600	15.00	0.2329	0.649530
0.1600	16.00	0.2697	0.634361
0.1600	17.00	0.3101	0.618534
0.1600	18.00	0.3541	0.602141
0.1600	19.00	0.4019	0.585280
0.1600	20.00	0.4533	0.567769
0.1600	21.00	0.4533	0.514983
0.1600	22.00	0.4533	0.469230
0.1600	23.00	0.4533	0.429315
0.1600	24.00	0.4533	0.394284
0.1600	25.00	0.4533	0.363372
0.1600	26.00	0.4533	0.335958
0.1600	27.00	0.4533	0.311533
0.1600	28.00	0.4533	0.289678
0.1600	29.00	0.4533	0.270045
0.1600	30.00	0.4533	0.252342
0.1600	31.00	0.4533	0.236324
0.1600	32.00	0.4533	0.221785
0.1600	33.00	0.4533	0.208547
0.1600	34.00	0.4533	0.196460
0.1600	35.00	0.4533	0.185394
0.1600	36.00	0.4533	0.175237
0.1600	37.00	0.4533	0.165893
0.1600	38.00	0.4533	0.157277
0.1600	39.00	0.4533	0.149315
0.1600	40.00	0.4533	0.141942
0.1600	42.00	0.4533	0.128746
0.1600	44.00	0.4533	0.117308
0.1600	46.00	0.4533	0.107329
0.1600	48.00	0.4533	0.098571
0.1600	50.00	0.4533	0.090843
0.1600	52.00	0.4533	0.083989
0.1600	54.00	0.4533	0.077883
0.1600	56.00	0.4533	0.072419
0.1600	58.00	0.4533	0.067511
0.1600	60.00	0.4533	0.063085
0.1600	65.00	0.4533	0.053753
0.1600	70.00	0.4533	0.046348
0.1600	75.00	0.4533	0.040375
0.1600	80.00	0.4533	0.035486

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.1666	5.00	0.0232	0.737089
0.1666	6.00	0.0337	0.731190
0.1666	7.00	0.0463	0.724091
0.1666	8.00	0.0612	0.715715
0.1666	9.00	0.0785	0.705966
0.1666	10.00	0.0985	0.694719
0.1666	11.00	0.1215	0.681809
0.1666	12.00	0.1478	0.667014
0.1666	13.00	0.1780	0.650032
0.1666	14.00	0.2128	0.630426
0.1666	15.00	0.2535	0.607532
0.1666	16.00	0.3020	0.580255
0.1666	17.00	0.3620	0.546513
0.1666	18.00	0.4425	0.501221
0.1666	19.00	0.4445	0.449856
0.1666	20.00	0.4445	0.405995
0.1666	21.00	0.4445	0.368249
0.1666	22.00	0.4445	0.335533
0.1666	23.00	0.4445	0.306991
0.1666	24.00	0.4445	0.281941
0.1666	25.00	0.4445	0.259837
0.1666	26.00	0.4445	0.240234
0.1666	27.00	0.4445	0.222768
0.1666	28.00	0.4445	0.207140
0.1666	29.00	0.4445	0.193101
0.1666	30.00	0.4445	0.180442
0.1666	31.00	0.4445	0.168989
0.1666	32.00	0.4445	0.158592
0.1666	33.00	0.4445	0.149126
0.1666	34.00	0.4445	0.140483
0.1666	35.00	0.4445	0.132570
0.1666	36.00	0.4445	0.125307
0.1666	37.00	0.4445	0.118625
0.1666	38.00	0.4445	0.112464
0.1666	39.00	0.4445	0.106771
0.1666	40.00	0.4445	0.101499
0.1666	42.00	0.4445	0.092062
0.1666	44.00	0.4445	0.083883
0.1666	46.00	0.4445	0.076748
0.1666	48.00	0.4445	0.070485
0.1666	50.00	0.4445	0.064959
0.1666	52.00	0.4445	0.060058
0.1666	54.00	0.4445	0.055692
0.1666	56.00	0.4445	0.051785
0.1666	58.00	0.4445	0.048275
0.1666	60.00	0.4445	0.045111
0.1666	65.00	0.4445	0.038437
0.1666	70.00	0.4445	0.033142
0.1666	75.00	0.4445	0.028871
0.1666	80.00	0.4445	0.025375

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.1800	5.00	0.0242	0.706371
0.1800	6.00	0.0352	0.700201
0.1800	7.00	0.0484	0.692762
0.1800	8.00	0.0641	0.683966
0.1800	9.00	0.0823	0.673700
0.1800	10.00	0.1034	0.661813
0.1800	11.00	0.1278	0.648105
0.1800	12.00	0.1559	0.632300
0.1800	13.00	0.1884	0.614008
0.1800	14.00	0.2264	0.592643
0.1800	15.00	0.2715	0.567256
0.1800	16.00	0.3269	0.536118
0.1800	17.00	0.3994	0.495313
0.1800	18.00	0.4267	0.443189
0.1800	19.00	0.4267	0.397765
0.1800	20.00	0.4267	0.358983
0.1800	21.00	0.4267	0.325608
0.1800	22.00	0.4267	0.296680
0.1800	23.00	0.4267	0.271443
0.1800	24.00	0.4267	0.249294
0.1800	25.00	0.4267	0.229749
0.1800	26.00	0.4267	0.212416
0.1800	27.00	0.4267	0.196973
0.1800	28.00	0.4267	0.183155
0.1800	29.00	0.4267	0.170741
0.1800	30.00	0.4267	0.159548
0.1800	31.00	0.4267	0.149421
0.1800	32.00	0.4267	0.140228
0.1800	33.00	0.4267	0.131858
0.1800	34.00	0.4267	0.124216
0.1800	35.00	0.4267	0.117219
0.1800	36.00	0.4267	0.110797
0.1800	37.00	0.4267	0.104889
0.1800	38.00	0.4267	0.099441
0.1800	39.00	0.4267	0.094407
0.1800	40.00	0.4267	0.089746
0.1800	42.00	0.4267	0.081402
0.1800	44.00	0.4267	0.074170
0.1800	46.00	0.4267	0.067861
0.1800	48.00	0.4267	0.062323
0.1800	50.00	0.4267	0.057437
0.1800	52.00	0.4267	0.053104
0.1800	54.00	0.4267	0.049243
0.1800	56.00	0.4267	0.045789
0.1800	58.00	0.4267	0.042685
0.1800	60.00	0.4267	0.039887
0.1800	65.00	0.4267	0.033987
0.1800	70.00	0.4267	0.029305
0.1800	75.00	0.4267	0.025528
0.1800	80.00	0.4267	0.022436

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.2000	5.00	0.0259	0.660422
0.2000	6.00	0.0377	0.653796
0.2000	7.00	0.0519	0.645780
0.2000	8.00	0.0689	0.636265
0.2000	9.00	0.0887	0.625100
0.2000	10.00	0.1118	0.612086
0.2000	11.00	0.1388	0.596943
0.2000	12.00	0.1702	0.579272
0.2000	13.00	0.2072	0.558467
0.2000	14.00	0.2515	0.533533
0.2000	15.00	0.3065	0.502610
0.2000	16.00	0.3799	0.461289
0.2000	17.00	0.4000	0.409402
0.2000	18.00	0.4000	0.365176
0.2000	19.00	0.4000	0.327748
0.2000	20.00	0.4000	0.295793
0.2000	21.00	0.4000	0.268293
0.2000	22.00	0.4000	0.244457
0.2000	23.00	0.4000	0.223662
0.2000	24.00	0.4000	0.205412
0.2000	25.00	0.4000	0.189307
0.2000	26.00	0.4000	0.175025
0.2000	27.00	0.4000	0.162301
0.2000	28.00	0.4000	0.150915
0.2000	29.00	0.4000	0.140686
0.2000	30.00	0.4000	0.131464
0.2000	31.00	0.4000	0.123119
0.2000	32.00	0.4000	0.115544
0.2000	33.00	0.4000	0.108648
0.2000	34.00	0.4000	0.102350
0.2000	35.00	0.4000	0.096585
0.2000	36.00	0.4000	0.091294
0.2000	37.00	0.4000	0.086426
0.2000	38.00	0.4000	0.081937
0.2000	39.00	0.4000	0.077789
0.2000	40.00	0.4000	0.073948
0.2000	42.00	0.4000	0.067073
0.2000	44.00	0.4000	0.061114
0.2000	46.00	0.4000	0.055915
0.2000	48.00	0.4000	0.051353
0.2000	50.00	0.4000	0.047327
0.2000	52.00	0.4000	0.043756
0.2000	54.00	0.4000	0.040575
0.2000	56.00	0.4000	0.037729
0.2000	58.00	0.4000	0.035172
0.2000	60.00	0.4000	0.032866
0.2000	65.00	0.4000	0.028004
0.2000	70.00	0.4000	0.024146
0.2000	75.00	0.4000	0.021034
0.2000	80.00	0.4000	0.018487

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.2200	5.00	0.0279	0.614329
0.2200	6.00	0.0406	0.607167
0.2200	7.00	0.0561	0.598471
0.2200	8.00	0.0745	0.588092
0.2200	9.00	0.0963	0.575831
0.2200	10.00	0.1219	0.561407
0.2200	11.00	0.1522	0.544411
0.2200	12.00	0.1881	0.524218
0.2200	13.00	0.2315	0.499784
0.2200	14.00	0.2860	0.469103
0.2200	15.00	0.3606	0.427162
0.2200	16.00	0.3733	0.375766
0.2200	17.00	0.3733	0.332859
0.2200	18.00	0.3733	0.296902
0.2200	19.00	0.3733	0.266471
0.2200	20.00	0.3733	0.240490
0.2200	21.00	0.3733	0.218132
0.2200	22.00	0.3733	0.198752
0.2200	23.00	0.3733	0.181845
0.2200	24.00	0.3733	0.167007
0.2200	25.00	0.3733	0.153914
0.2200	26.00	0.3733	0.142302
0.2200	27.00	0.3733	0.131956
0.2200	28.00	0.3733	0.122699
0.2200	29.00	0.3733	0.114383
0.2200	30.00	0.3733	0.106885
0.2200	31.00	0.3733	0.100100
0.2200	32.00	0.3733	0.093942
0.2200	33.00	0.3733	0.088334
0.2200	34.00	0.3733	0.083215
0.2200	35.00	0.3733	0.078527
0.2200	36.00	0.3733	0.074225
0.2200	37.00	0.3733	0.070267
0.2200	38.00	0.3733	0.066618
0.2200	39.00	0.3733	0.063245
0.2200	40.00	0.3733	0.060123
0.2200	42.00	0.3733	0.054533
0.2200	44.00	0.3733	0.049688
0.2200	46.00	0.3733	0.045461
0.2200	48.00	0.3733	0.041752
0.2200	50.00	0.3733	0.038478
0.2200	52.00	0.3733	0.035576
0.2200	54.00	0.3733	0.032989
0.2200	56.00	0.3733	0.030675
0.2200	58.00	0.3733	0.028596
0.2200	60.00	0.3733	0.026721
0.2200	65.00	0.3733	0.022768
0.2200	70.00	0.3733	0.019632
0.2200	75.00	0.3733	0.017102
0.2200	80.00	0.3733	0.015031

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.2400	5.00	0.0301	0.568052
0.2400	6.00	0.0440	0.560256
0.2400	7.00	0.0609	0.550738
0.2400	8.00	0.0812	0.539301
0.2400	9.00	0.1055	0.525661
0.2400	10.00	0.1344	0.509404
0.2400	11.00	0.1691	0.489884
0.2400	12.00	0.2115	0.466004
0.2400	13.00	0.2656	0.435596
0.2400	14.00	0.3415	0.392894
0.2400	15.00	0.3467	0.342311
0.2400	16.00	0.3467	0.300860
0.2400	17.00	0.3467	0.266505
0.2400	18.00	0.3467	0.237716
0.2400	19.00	0.3467	0.213352
0.2400	20.00	0.3467	0.192550
0.2400	21.00	0.3467	0.174649
0.2400	22.00	0.3467	0.159132
0.2400	23.00	0.3467	0.145596
0.2400	24.00	0.3467	0.133715
0.2400	25.00	0.3467	0.123232
0.2400	26.00	0.3467	0.113935
0.2400	27.00	0.3467	0.105652
0.2400	28.00	0.3467	0.098240
0.2400	29.00	0.3467	0.091582
0.2400	30.00	0.3467	0.085578
0.2400	31.00	0.3467	0.080146
0.2400	32.00	0.3467	0.075215
0.2400	33.00	0.3467	0.070726
0.2400	34.00	0.3467	0.066626
0.2400	35.00	0.3467	0.062873
0.2400	36.00	0.3467	0.059429
0.2400	37.00	0.3467	0.056260
0.2400	38.00	0.3467	0.053338
0.2400	39.00	0.3467	0.050638
0.2400	40.00	0.3467	0.048138
0.2400	42.00	0.3467	0.043662
0.2400	44.00	0.3467	0.039783
0.2400	46.00	0.3467	0.036399
0.2400	48.00	0.3467	0.033429
0.2400	50.00	0.3467	0.030808
0.2400	52.00	0.3467	0.028484
0.2400	54.00	0.3467	0.026413
0.2400	56.00	0.3467	0.024560
0.2400	58.00	0.3467	0.022895
0.2400	60.00	0.3467	0.021394
0.2400	65.00	0.3467	0.018230
0.2400	70.00	0.3467	0.015718
0.2400	75.00	0.3467	0.013692
0.2400	80.00	0.3467	0.012034

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.2600	5.00	0.0328	0.521541
0.2600	6.00	0.0480	0.512975
0.2600	7.00	0.0668	0.502445
0.2600	8.00	0.0895	0.489669
0.2600	9.00	0.1169	0.474225
0.2600	10.00	0.1503	0.455449
0.2600	11.00	0.1917	0.432186
0.2600	12.00	0.2452	0.402088
0.2600	13.00	0.3200	0.358452
0.2600	14.00	0.3200	0.309073
0.2600	15.00	0.3200	0.269237
0.2600	16.00	0.3200	0.236634
0.2600	17.00	0.3200	0.209614
0.2600	18.00	0.3200	0.186970
0.2600	19.00	0.3200	0.167807
0.2600	20.00	0.3200	0.151446
0.2600	21.00	0.3200	0.137366
0.2600	22.00	0.3200	0.125162
0.2600	23.00	0.3200	0.114515
0.2600	24.00	0.3200	0.105171
0.2600	25.00	0.3200	0.096925
0.2600	26.00	0.3200	0.089613
0.2600	27.00	0.3200	0.083098
0.2600	28.00	0.3200	0.077268
0.2600	29.00	0.3200	0.072031
0.2600	30.00	0.3200	0.067309
0.2600	31.00	0.3200	0.063037
0.2600	32.00	0.3200	0.059159
0.2600	33.00	0.3200	0.055627
0.2600	34.00	0.3200	0.052403
0.2600	35.00	0.3200	0.049452
0.2600	36.00	0.3200	0.046743
0.2600	37.00	0.3200	0.044250
0.2600	38.00	0.3200	0.041952
0.2600	39.00	0.3200	0.039828
0.2600	40.00	0.3200	0.037861
0.2600	42.00	0.3200	0.034341
0.2600	44.00	0.3200	0.031290
0.2600	46.00	0.3200	0.028629
0.2600	48.00	0.3200	0.026293
0.2600	50.00	0.3200	0.024231
0.2600	52.00	0.3200	0.022403
0.2600	54.00	0.3200	0.020774
0.2600	56.00	0.3200	0.019317
0.2600	58.00	0.3200	0.018008
0.2600	60.00	0.3200	0.016827
0.2600	65.00	0.3200	0.014338
0.2600	70.00	0.3200	0.012363
0.2600	75.00	0.3200	0.010769
0.2600	80.00	0.3200	0.009465

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.2800	5.00	0.0361	0.474720
0.2800	6.00	0.0530	0.465200
0.2800	7.00	0.0740	0.453381
0.2800	8.00	0.0998	0.438839
0.2800	9.00	0.1318	0.420890
0.2800	10.00	0.1719	0.398323
0.2800	11.00	0.2247	0.368581
0.2800	12.00	0.2933	0.324033
0.2800	13.00	0.2933	0.276099
0.2800	14.00	0.2933	0.238065
0.2800	15.00	0.2933	0.207381
0.2800	16.00	0.2933	0.182269
0.2800	17.00	0.2933	0.161456
0.2800	18.00	0.2933	0.144015
0.2800	19.00	0.2933	0.129254
0.2800	20.00	0.2933	0.116652
0.2800	21.00	0.2933	0.105807
0.2800	22.00	0.2933	0.096407
0.2800	23.00	0.2933	0.088206
0.2800	24.00	0.2933	0.081008
0.2800	25.00	0.2933	0.074657
0.2800	26.00	0.2933	0.069025
0.2800	27.00	0.2933	0.064007
0.2800	28.00	0.2933	0.059516
0.2800	29.00	0.2933	0.055482
0.2800	30.00	0.2933	0.051845
0.2800	31.00	0.2933	0.048554
0.2800	32.00	0.2933	0.045567
0.2800	33.00	0.2933	0.042847
0.2800	34.00	0.2933	0.040364
0.2800	35.00	0.2933	0.038090
0.2800	36.00	0.2933	0.036004
0.2800	37.00	0.2933	0.034084
0.2800	38.00	0.2933	0.032314
0.2800	39.00	0.2933	0.030678
0.2800	40.00	0.2933	0.029163
0.2800	42.00	0.2933	0.026452
0.2800	44.00	0.2933	0.024102
0.2800	46.00	0.2933	0.022051
0.2800	48.00	0.2933	0.020252
0.2800	50.00	0.2933	0.018664
0.2800	52.00	0.2933	0.017256
0.2800	54.00	0.2933	0.016002
0.2800	56.00	0.2933	0.014879
0.2800	58.00	0.2933	0.013871
0.2800	60.00	0.2933	0.012961
0.2800	65.00	0.2933	0.011044
0.2800	70.00	0.2933	0.009523
0.2800	75.00	0.2933	0.008295
0.2800	80.00	0.2933	0.007291

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.3000	5.00	0.0400	0.427479
0.3000	6.00	0.0591	0.416734
0.3000	7.00	0.0832	0.403201
0.3000	8.00	0.1135	0.386181
0.3000	9.00	0.1522	0.364402
0.3000	10.00	0.2043	0.335074
0.3000	11.00	0.2667	0.289727
0.3000	12.00	0.2667	0.243451
0.3000	13.00	0.2667	0.207438
0.3000	14.00	0.2667	0.178862
0.3000	15.00	0.2667	0.155809
0.3000	16.00	0.2667	0.136941
0.3000	17.00	0.2667	0.121304
0.3000	18.00	0.2667	0.108200
0.3000	19.00	0.2667	0.097111
0.3000	20.00	0.2667	0.087642
0.3000	21.00	0.2667	0.079494
0.3000	22.00	0.2667	0.072432
0.3000	23.00	0.2667	0.066270
0.3000	24.00	0.2667	0.060863
0.3000	25.00	0.2667	0.056091
0.3000	26.00	0.2667	0.051859
0.3000	27.00	0.2667	0.048089
0.3000	28.00	0.2667	0.044715
0.3000	29.00	0.2667	0.041685
0.3000	30.00	0.2667	0.038952
0.3000	31.00	0.2667	0.036480
0.3000	32.00	0.2667	0.034235
0.3000	33.00	0.2667	0.032192
0.3000	34.00	0.2667	0.030326
0.3000	35.00	0.2667	0.028618
0.3000	36.00	0.2667	0.027050
0.3000	37.00	0.2667	0.025608
0.3000	38.00	0.2667	0.024278
0.3000	39.00	0.2667	0.023049
0.3000	40.00	0.2667	0.021911
0.3000	42.00	0.2667	0.019874
0.3000	44.00	0.2667	0.018108
0.3000	46.00	0.2667	0.016568
0.3000	48.00	0.2667	0.015216
0.3000	50.00	0.2667	0.014023
0.3000	52.00	0.2667	0.012965
0.3000	54.00	0.2667	0.012022
0.3000	56.00	0.2667	0.011179
0.3000	58.00	0.2667	0.010421
0.3000	60.00	0.2667	0.009738
0.3000	65.00	0.2667	0.008297
0.3000	70.00	0.2667	0.007154
0.3000	75.00	0.2667	0.006232
0.3000	80.00	0.2667	0.005478

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.3200	5.00	0.0451	0.379641
0.3200	6.00	0.0671	0.367252
0.3200	7.00	0.0955	0.351285
0.3200	8.00	0.1326	0.330409
0.3200	9.00	0.1839	0.301566
0.3200	10.00	0.2400	0.255565
0.3200	11.00	0.2400	0.211211
0.3200	12.00	0.2400	0.177476
0.3200	13.00	0.2400	0.151222
0.3200	14.00	0.2400	0.130390
0.3200	15.00	0.2400	0.113584
0.3200	16.00	0.2400	0.099830
0.3200	17.00	0.2400	0.088431
0.3200	18.00	0.2400	0.078878
0.3200	19.00	0.2400	0.070794
0.3200	20.00	0.2400	0.063891
0.3200	21.00	0.2400	0.057951
0.3200	22.00	0.2400	0.052803
0.3200	23.00	0.2400	0.048311
0.3200	24.00	0.2400	0.044369
0.3200	25.00	0.2400	0.040890
0.3200	26.00	0.2400	0.037805
0.3200	27.00	0.2400	0.035057
0.3200	28.00	0.2400	0.032598
0.3200	29.00	0.2400	0.030388
0.3200	30.00	0.2400	0.028396
0.3200	31.00	0.2400	0.026594
0.3200	32.00	0.2400	0.024958
0.3200	33.00	0.2400	0.023468
0.3200	34.00	0.2400	0.022108
0.3200	35.00	0.2400	0.020862
0.3200	36.00	0.2400	0.019720
0.3200	37.00	0.2400	0.018668
0.3200	38.00	0.2400	0.017698
0.3200	39.00	0.2400	0.016802
0.3200	40.00	0.2400	0.015973
0.3200	42.00	0.2400	0.014488
0.3200	44.00	0.2400	0.013201
0.3200	46.00	0.2400	0.012078
0.3200	48.00	0.2400	0.011092
0.3200	50.00	0.2400	0.010223
0.3200	52.00	0.2400	0.009451
0.3200	54.00	0.2400	0.008764
0.3200	56.00	0.2400	0.008149
0.3200	58.00	0.2400	0.007597
0.3200	60.00	0.2400	0.007099
0.3200	65.00	0.2400	0.006049
0.3200	70.00	0.2400	0.005216
0.3200	75.00	0.2400	0.004543
0.3200	80.00	0.2400	0.003993

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.3333	5.00	0.0493	0.347360
0.3333	6.00	0.0739	0.333507
0.3333	7.00	0.1064	0.315213
0.3333	8.00	0.1510	0.290128
0.3333	9.00	0.2213	0.250611
0.3333	10.00	0.2223	0.202998
0.3333	11.00	0.2223	0.167767
0.3333	12.00	0.2223	0.140970
0.3333	13.00	0.2223	0.120117
0.3333	14.00	0.2223	0.103570
0.3333	15.00	0.2223	0.090221
0.3333	16.00	0.2223	0.079296
0.3333	17.00	0.2223	0.070241
0.3333	18.00	0.2223	0.062654
0.3333	19.00	0.2223	0.056232
0.3333	20.00	0.2223	0.050749
0.3333	21.00	0.2223	0.046031
0.3333	22.00	0.2223	0.041942
0.3333	23.00	0.2223	0.038374
0.3333	24.00	0.2223	0.035243
0.3333	25.00	0.2223	0.032480
0.3333	26.00	0.2223	0.030029
0.3333	27.00	0.2223	0.027846
0.3333	28.00	0.2223	0.025893
0.3333	29.00	0.2223	0.024138
0.3333	30.00	0.2223	0.022555
0.3333	31.00	0.2223	0.021124
0,3333	32.00	0.2223	0.019824
0.3333	33.00	0.2223	0.018641
0.3333	34.00	0.2223	0.017560
0.3333	35.00	0.2223	0.016571
0.3333	36.00	0.2223	0.015663
0.3333	37.00	0.2223	0.014828
0.3333	38.00	0.2223	0.014058
0.3333	39.00	0.2223	0.013346
0.3333	40.00	0.2223	0.012687
0.3333	42.00	0.2223	0.011508
0.3333	44.00	0.2223	0.010485
0.3333	46.00	0.2223	0.009593
0.3333	48.00	0.2223	0.008811
0.3333	50.00	0.2223	0.008120
0.3333	52.00	0.2223	0.007507
0.3333	54.00	0.2223	0.006962
0.3333	56.00	0.2223	0.006473
0.3333	58.00	0.2223	0.006034
0.3333	60.00	0.2223	0.005639
0.3333	65.00	0.2223	0.004805
0.3333	70.00	0.2223	0.004143
0.3333	75.00	0.2223	0.003609
0.3333	80.00	0.2223	0.003172

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.3400	5.00	0.0517	0.330907
0.3400	6.00	0.0780	0.316150
0.3400	7.00	0.1132	0.296322
0.3400	8.00	0.1635	0.268059
0.3400	9.00	0.2133	0.221595
0.3400	10.00	0.2133	0.179492
0.3400	11.00	0.2133	0.148340
0.3400	12.00	0.2133	0.124647
0.3400	13.00	0.2133	0.106208
0.3400	14.00	0.2133	0.091577
0.3400	15.00	0.2133	0.079774
0.3400	16.00	0.2133	0.070114
0.3400	17.00	0.2133	0.062108
0.3400	18.00	0.2133	0.055399
0.3400	19.00	0.2133	0.049721
0.3400	20.00	0.2133	0.044873
0.3400	21.00	0.2133	0.040701
0.3400	22.00	0.2133	0.037085
0.3400	23.00	0.2133	0.033930
0.3400	24.00	0.2133	0.031162
0.3400	25.00	0.2133	0.028719
0.3400	26.00	0.2133	0.026552
0.3400	27.00	0.2133	0.024622
0.3400	28.00	0.2133	0.022894
0.3400	29.00	0.2133	0.021343
0.3400	30.00	0.2133	0.019944
0.3400	31.00	0.2133	0.018678
0.3400	32.00	0.2133	0.017528
0.3400	33.00	0.2133	0.016482
0.3400	34.00	0.2133	0.015527
0.3400	35.00	0.2133	0.014652
0.3400	36.00	0.2133	0.013850
0.3400	37.00	0.2133	0.013111
0.3400	38.00	0.2133	0.012430
0.3400	39.00	0.2133	0.011801
0.3400	40.00	0.2133	0.011218
0.3400	42.00	0.2133	0.010175
0.3400	44.00	0.2133	0.009271
0.3400	46.00	0.2133	0.008483
0.3400	48.00	0.2133	0.007790
0.3400	50.00	0.2133	0.007180
0.3400	52.00	0.2133	0.006638
0.3400	54.00	0.2133	0.006155
0.3400	56.00	0.2133	0.005724
0.3400	58.00	0.2133	0.005336
0.3400	60.00	0.2133	0.004986
0.3400	65.00	0.2133	0.004248
0.3400	70.00	0.2133	0.003663
0.3400	75.00	0.2133	0.003191
0.3400	80.00	0.2133	0.002805

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.3600	5.00	0.0610	0.280703
0.3600	6.00	0.0940	0.262109
0.3600	7.00	0.1430	0.234552
0.3600	8.00	0.1867	0.187883
0.3600	9.00	0.1867	0.148451
0.3600	10.00	0.1867	0.120245
0.3600	11.00	0.1867	0.099376
0.3600	12.00	0.1867	0.083504
0.3600	13.00	0.1867	0.071151
0.3600	14.00	0.1867	0.061350
0.3600	15.00	0.1867	0.053442
0.3600	16.00	0.1867	0.046971
0.3600	17.00	0.1867	0.041607
0.3600	18.00	0.1867	0.037113
0.3600	19.00	0.1867	0.033309
0.3600	20.00	0.1867	0.030061
0.3600	21.00	0.1867	0.027267
0.3600	22.00	0.1867	0.024844
0.3600	23.00	0.1867	0.022731
0.3600	24.00	0.1867	0.020876
0.3600	25.00	0.1867	0.019239
0.3600	26.00	0.1867	0.017788
0.3600	27.00	0.1867	0.016495
0.3600	28.00	0.1867	0.015337
0.3600	29.00	0.1867	0.014298
0.3600	30.00	0.1867	0.013361
0.3600	31.00	0.1867	0.012513
0.3600	32.00	0.1867	0.011743
0.3600	33.00	0.1867	0.011042
0.3600	34.00	0.1867	0.010402
0.3600	35.00	0.1867	0.009816
0.3600	36.00	0.1867	0.009278
0.3600	37.00	0.1867	0.008783
0.3600	38.00	0.1867	0.008327
0.3600	39.00	0.1867	0.007906
0.3600	40.00	0.1867	0.007515
0.3600	42.00	0.1867	0.006817
0.3600	44.00	0.1867	0.006211
0.3600	46.00	0.1867	0.005683
0.3600	48.00	0.1867	0.005219
0.3600	50.00	0.1867	0.004810
0.3600	52.00	0.1867	0.004447
0.3600	54.00	0.1867	0.004124
0.3600	56.00	0.1867	0.003834
0.3600	58.00	0.1867	0.003574
0.3600	60.00	0.1867	0.003340
0.3600	65.00	0.1867	0.002846
0.3600	70.00	0.1867	0.002454
0.3600	75.00	0.1867	0.002138
0.3600	80.00	0.1867	0.001879

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.3800	5.00	0.0752	0.227724
0.3800	6.00	0.1226	0.201044
0.3800	7.00	0.1600	0.154537
0.3800	8.00	0.1600	0.118317
0.3800	9.00	0.1600	0.093485
0.3800	10.00	0.1600	0.075723
0.3800	11.00	0.1600	0.062581
0.3800	12.00	0.1600	0.052585
0.3800	13.00	0.1600	0.044806
0.3800	14.00	0.1600	0.038634
0.3800	15.00	0.1600	0.033655
0.3800	16.00	0.1600	0.029579
0.3800	17.00	0.1600	0.026202
0.3800	18.00	0.1600	0.023371
0.3800	19.00	0.1600	0.020976
0.3800	20.00	0.1600	0.018931
0.3800	21.00	0.1600	0.017171
0.3800	22.00	0.1600	0.015645
0.3800	23.00	0.1600	0.014314
0.3800	24.00	0.1600	0.013146
0.3800	25.00	0.1600	0.012116
0.3800	26.00	0.1600	0.011202
0.3800	27.00	0.1600	0.010387
0.3800	28.00	0.1600	0.009659
0.3800	29.00	0.1600	0.009004
0.3800	30.00	0.1600	0.008414
0.3800	31.00	0.1600	0.007880
0.3800	32.00	0.1600	0.007395
0.3800	33.00	0.1600	0.006953
0.3800	34.00	0.1600	0.006550
0.3800	35.00	0.1600	0.006181
0.3800	36.00	0.1600	0.005843
0.3800	37.00	0.1600	0.005531
0.3800	38.00	0.1600	0.005244
0.3800	39.00	0.1600	0.004978
0.3800	40.00	0.1600	0.004733
0.3800	42.00	0.1600	0.004293
0.3800	44.00	0.1600	0.003911
0.3800	46.00	0.1600	0.003579
0.3800	48.00	0.1600	0.003287
0.3800	50.00	0.1600	0.003029
0.3800	52.00	0.1600	0.002800
0.3800	54.00	0.1600	0.002597
0.3800	56.00	0.1600	0.002415
0.3800	58.00	0.1600	0.002251
0.3800	60.00	0.1600	0.002103
0.3800	65.00	0.1600	0.001792
0.3800	70.00	0.1600	0.001545
0.3800	75.00	0.1600	0.001346
0.3800	80.00	0.1600	0.001183

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.4000	5.00	0.1022	0.167537
0.4000	6.00	0.1333	0.121725
0.4000	7.00	0.1333	0.089431
0.4000	8.00	0.1333	0.068471
0.4000	9.00	0.1333	0.054100
0.4000	10.00	0.1333	0.043821
0.4000	11.00	0.1333	0.036216
0.4000	12.00	0.1333	0.030431
0.4000	13.00	0.1333	0.025930
0.4000	14.00	0.1333	0.022358
0.4000	15.00	0.1333	0.019476
0.4000	16.00	0.1333	0.017118
0.4000	17.00	0.1333	0.015163
0.4000	18.00	0.1333	0.013525
0.4000	19.00	0.1333	0.012139
0.4000	20.00	0.1333	0.010955
0.4000	21.00	0.1333	0.009937
0.4000	22.00	0.1333	0.009054
0.4000	23.00	0.1333	0.008284
0.4000	24.00	0.1333	0.007608
0.4000	25.00	0.1333	0.007011
0.4000	26.00	0.1333	0.006482
0.4000	27.00	0.1333	0.006011
0.4000	28.00	0.1333	0.005589
0.4000	29.00	0.1333	0.005211
0.4000	30.00	0.1333	0.004869
0.4000	31.00	0.1333	0.004560
0.4000	32.00	0.1333	0.004279
0.4000	33.00	0.1333	0.004024
0.4000	34.00	0.1333	0.003791
0.4000	35.00	0.1333	0.003577
0.4000	36.00	0.1333	0.003381
0.4000	37.00	0.1333	0.003201
0.4000	38.00	0.1333	0.003035
0.4000	39.00	0.1333	0.002881
0.4000	40.00	0.1333	0.002739
0.4000	42.00	0.1333	0.002484
0.4000	44.00	0.1333	0.002263
0.4000	46.00	0.1333	0.002071
0.4000	48.00	0.1333	0.001902
0.4000	50.00	0.1333	0.001753
0.4000	52.00	0.1333	0.001621
0.4000	54.00	0.1333	0.001503
0.4000	56.00	0.1333	0.001397
0.4000	58.00	0.1333	0.001303
0.4000	60.00	0.1333	0.001217
0.4000	65.00	0.1333	0.001037
0.4000	70.00	0.1333	0.000894
0.4000	75.00	0.1333	0.000779
0.4000	80.00	0.1333	0.000685

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.4200	5.00	0.1067	0.089746
0.4200	6.00	0.1067	0.062324
0.4200	7.00	0.1067	0.045789
0.4200	8.00	0.1067	0.035057
0.4200	9.00	0.1067	0.027699
0.4200	10.00	0.1067	0.022436
0.4200	11.00	0.1067	0.018543
0.4200	12.00	0.1067	0.015581
0.4200	13.00	0.1067	0.013276
0.4200	14.00	0.1067	0.011447
0.4200	15.00	0.1067	0.009972
0.4200	16.00	0.1067	0.008764
0.4200	17.00	0.1067	0.007763
0.4200	18.00	0.1067	0.006925
0.4200	19.00	0.1067	0.006215
0.4200	20.00	0.1067	0.005609
0.4200	21.00	0.1067	0.005088
0.4200	22.00	0.1067	0.004636
0.4200	23.00	0.1067	0.004241
0.4200	24.00	0.1067	0.003895
0.4200	25.00	0.1067	0.003590
0.4200	26.00	0.1067	0.003319
0.4200	27.00	0.1067	0.003078
0.4200	28.00	0.1067	0.002862
0.4200	29.00	0.1067	0.002668
0.4200	30.00	0.1067	0.002493
0.4200	31.00	0.1067	0.002335
0.4200	32.00	0.1067	0.002191
0.4200	33.00	0.1067	0.002060
0.4200	34.00	0.1067	0.001941
0.4200	35.00	0.1067	0.001832
0.4200	36.00	0.1067	0.001731
0.4200	37.00	0.1067	0.001639
0.4200	38.00	0.1067	0.001554
0.4200	39.00	0.1067	0.001475
0.4200	40.00	0.1067	0.001402
0.4200	42.00	0.1067	0.001272
0.4200	44.00	0.1067	0.001159
0.4200	46.00	0.1067	0.001060
0.4200	48.00	0.1067	0.000974
0.4200	50.00	0.1067	0.000897
0.4200	52.00	0.1067	0.000830
0.4200	54.00	0.1067	0.000769
0.4200	56.00	0.1067	0.000715
0.4200	58.00	0.1067	0.000667
0.4200	60.00	0.1067	0.000623
0.4200	65.00	0.1067	0.000531
0.4200	70.00	0.1067	0.000458
0.4200	75.00	0.1067	0.000399
0.4200	80.00	0.1067	0.000351

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.4400	5.00	0.0800	0.037861
0.4400	6.00	0.0800	0.026293
0.4400	7.00	0.0800	0.019317
0.4400	8.00	0.0800	0.014790
0.4400	9.00	0.0800	0.011686
0.4400	10.00	0.0800	0.009465
0.4400	11.00	0.0800	0.007823
0.4400	12.00	0.0800	0.006573
0.4400	13.00	0.0800	0.005601
0.4400	14.00	0.0800	0.004829
0.4400	15.00	0.0800	0.004207
0.4400	16.00	0.0800	0.003697
0.4400	17.00	0.0800	0.003275
0.4400	18.00	0.0800	0.002921
0.4400	19.00	0.0800	0.002622
0.4400	20.00	0.0800	0.002366
0.4400	21.00	0.0800	0.002146
0.4400	22.00	0.0800	0.001956
0.4400	23.00	0.0800	0.001789
0.4400	24.00	0.0800	0.001643
0.4400	25.00	0.0800	0.001514
0.4400	26.00	0.0800	0.001400
0.4400	27.00	0.0800	0.001298
0.4400	28.00	0.0800	0.001207
0.4400	29.00	0.0800	0.001125
0.4400	30.00	0.0800	0.001052
0.4400	31.00	0.0800	0.000985
0.4400	32.00	0.0800	0.000924
0.4400	33.00	0.0800	0.000869
0.4400	34.00	0.0800	0.000819
0.4400	35.00	0.0800	0.000773
0.4400	36.00	0.0800	0.000730
0.4400	37.00	0.0800	0.000691
0.4400	38.00	0.0800	0.000655
0.4400	39.00	0.0800	0.000622
0.4400	40.00	0.0800	0.000592
0.4400	42.00	0.0800	0.000537
0.4400	44.00	0.0800	0.000489
0.4400	46.00	0.0800	0.000447
0.4400	48.00	0.0800	0.000411
0.4400	50.00	0.0800	0.000379
0.4400	52.00	0.0800	0.000350
0.4400	54.00	0.0800	0.000325
0.4400	56.00	0.0800	0.000302
0.4400	58.00	0.0800	0.000281
0.4400	60.00	0.0800	0.000263
0.4400	65.00	0.0800	0.000224
0.4400	70.00	0.0800	0.000193
0.4400	75.00	0.0800	0.000168
0.4400	80.00	0.0800	0.000148

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.4600	5.00	0.0533	0.011218
0.4600	6.00	0.0533	0.007790
0.4600	7.00	0.0533	0.005724
0.4600	8.00	0.0533	0.004382
0.4600	9.00	0.0533	0.003462
0.4600	10.00	0.0533	0.002805
0.4600	11.00	0.0533	0.002318
0.4600	12.00	0.0533	0.001948
0.4600	13.00	0.0533	0.001660
0.4600	14.00	0.0533	0.001431
0.4600	15.00	0.0533	0.001246
0.4600	16.00	0.0533	0.001096
0.4600	17.00	0.0533	0.000970
0.4600	18.00	0.0533	0.000866
0.4600	19.00	0.0533	0.000777
0.4600	20.00	0.0533	0.000701
0.4600	21.00	0.0533	0.000636
0.4600	22.00	0.0533	0.000579
0.4600	23.00	0.0533	0.000530
0.4600	24.00	0.0533	0.000487
0.4600	25.00	0.0533	0.000449
0.4600	26.00	0.0533	0.000415
0.4600	27.00	0.0533	0.000385
0.4600	28.00	0.0533	0.000358
0.4600	29.00	0.0533	0.000333
0.4600	30.00	0.0533	0.000312
0.4600	31.00	0.0533	0.000292
0.4600	32.00	0.0533	0.000274
0.4600	33.00	0.0533	0.000258
0.4600	34.00	0.0533	0.000243
0.4600	35.00	0.0533	0.000229
0.4600	36.00	0.0533	0.000216
0.4600	37.00	0.0533	0.000205
0.4600	38.00	0.0533	0.000194
0.4600	39.00	0.0533	0.000184
0.4600	40.00	0.0533	0.000175
0.4600	42.00	0.0533	0.000159
0.4600	44.00	0.0533	0.000145
0.4600	46.00	0.0533	0.000133
0.4600	48.00	0.0533	0.000122
0.4600	50.00	0.0533	0.000112
0.4600	52.00	0.0533	0.000104
0.4600	54.00	0.0533	0.000096
0.4600	56.00	0.0533	0.000089
0.4600	58.00	0.0533	0.000083
0.4600	60.00	0.0533	0.000078
0.4600	65.00	0.0533	0.000066
0.4600	70.00	0.0533	0.000057
0.4600	75.00	0.0533	0.000050
0.4600	80.00	0.0533	0.000044

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.4800	5.00	0.0267	0.001402
0.4800	6.00	0.0267	0.000974
0.4800	7.00	0.0267	0.000715
0.4800	8.00	0.0267	0.000548
0.4800	9.00	0.0267	0.000433
0.4800	10.00	0.0267	0.000351
0.4800	11.00	0.0267	0.000290
0.4800	12.00	0.0267	0.000243
0.4800	13.00	0.0267	0.000207
0.4800	14.00	0.0267	0.000179
0.4800	15.00	0.0267	0.000156
0.4800	16.00	0.0267	0.000137
0.4800	17.00	0.0267	0.000121
0.4800	18.00	0.0267	0.000108
0.4800	19.00	0.0267	0.000097
0.4800	20.00	0.0267	0.000088
0.4800	21.00	0.0267	0.000079
0.4800	22.00	0.0267	0.000072
0.4800	23.00	0.0267	0.000066
0.4800	24.00	0.0267	0.000061
0.4800	25.00	0.0267	0.000056
0.4800	26.00	0.0267	0.000052
0.4800	27.00	0.0267	0.000048
0.4800	28.00	0.0267	0.000045
0.4800	29.00	0.0267	0.000042
0.4800	30.00	0.0267	0.000039
0.4800	31.00	0.0267	0.000036
0.4800	32.00	0.0267	0.000034
0.4800	33.00	0.0267	0.000032
0.4800	34.00	0.0267	0.000030
0.4800	35.00	0.0267	0.000029
0.4800	36.00	0.0267	0.000027
0.4800	37.00	0.0267	0.000026
0.4800	38.00	0.0267	0.000024
0.4800	39.00	0.0267	0.000023
0.4800	40.00	0.0267	0.000022
0.4800	42.00	0.0267	0.000020
0.4800	44.00	0.0267	0.000018
0.4800	46.00	0.0267	0.000017
0.4800	48.00	0.0267	0.000015
0.4800	50.00	0.0267	0.000014
0.4800	52.00	0.0267	0.000013
0.4800	54.00	0.0267	0.000012
0.4800	56.00	0.0267	0.000011
0.4800	58.00	0.0267	0.000010
0.4800	60.00	0.0267	0.000010
0.4800	65.00	0.0267	0.000008
0.4800	70.00	0.0267	0.000007
0.4800	75.00	0.0267	0.000006
0.4800	80.00	0.0267	0.000005

FINAL COMPUTED RESULTS
STRESS REDUCTION FACTOR
EQUAL END ECCENTRICITY
WALLS IN SINGLE CURVATURE

E/T	H/T	PHY	V
0.5000	5.00	0.0000	0.000000
0.5000	6.00	0.0000	0.000000
0.5000	7.00	0.0000	0.000000
0.5000	8.00	0.0000	0.000000
0.5000	9.00	0.0000	0.000000
0.5000	10.00	0.0000	0.000000
0.5000	11.00	0.0000	0.000000
0.5000	12.00	0.0000	0.000000
0.5000	13.00	0.0000	0.000000
0.5000	14.00	0.0000	0.000000
0.5000	15.00	0.0000	0.000000
0.5000	16.00	0.0000	0.000000
0.5000	17.00	0.0000	0.000000
0.5000	18.00	0.0000	0.000000
0.5000	19.00	0.0000	0.000000
0.5000	20.00	0.0000	0.000000
0.5000	21.00	0.0000	0.000000
0.5000	22.00	0.0000	0.000000
0.5000	23.00	0.0000	0.000000
0.5000	24.00	0.0000	0.000000
0.5000	25.00	0.0000	0.000000
0.5000	26.00	0.0000	0.000000
0.5000	27.00	0.0000	0.000000
0.5000	28.00	0.0000	0.000000
0.5000	29.00	0.0000	0.000000
0.5000	30.00	0.0000	0.000000
0.5000	31.00	0.0000	0.000000
0.5000	32.00	0.0000	0.000000
0.5000	33.00	0.0000	0.000000
0.5000	34.00	0.0000	0.000000
0.5000	35.00	0.0000	0.000000
0.5000	36.00	0.0000	0.000000
0.5000	37.00	0.0000	0.000000
0.5000	38.00	0.0000	0.000000
0.5000	39.00	0.0000	0.000000
0.5000	40.00	0.0000	0.000000
0.5000	42.00	0.0000	0.000000
0.5000	44.00	0.0000	0.000000
0.5000	46.00	0.0000	0.000000
0.5000	48.00	0.0000	0.000000
0.5000	50.00	0.0000	0.000000
0.5000	52.00	0.0000	0.000000
0.5000	54.00	0.0000	0.000000
0.5000	56.00	0.0000	0.000000
0.5000	58.00	0.0000	0.000000
0.5000	60.00	0.0000	0.000000
0.5000	65.00	0.0000	0.000000
0.5000	70.00	0.0000	0.000000
0.5000	75.00	0.0000	0.000000
0.5000	80.00	0.0000	0.000000

STOP

93 KBYTES USED JOB-STEP RETURN CODE = 0

CTCTL3/

E.R.C.C. FORTRAN COMPILER RELEASE 6 VERSION 5

```
C        PROGRAM FOR CUBIC EQUATION SOLUTION
C        PROGRAMER  A. AWNI
C        THIS PROGRAM SOLVES 799 CUBIC EQUATION FOR  WALLS
C        BENT IN DOUBLE CURVATURE TO OBTAIN THE
C        WALL END ROTATION 'PHY'
C        SAME PROGRAM CAN BE USED FOR WALLS BENT
C        IN SINGLE CURVATURE BUT THE WALL SLENDERNESS MUST BE DOUBLE
C        I.E.    REALK=((SLEND(J))**2)/(55.5)
C
C
C
      REAL*8 E(17),SLEND(47),X02AAF,COEF(4),REZ(4),IMZ(4),DUMMY,TO
    1 FORMAT(10F5.2)
      READ(5,1) (E(K),K=1,17)
      READ(5,1) (SLEND(J),J=1,47)
      DO 100 K=1,17
      DO 100 J=1,47
      REALK=((SLEND(J))/(2.0))**2*((1.0)/(55.5))
      COEF(1)=3.*9.869
      COEF(2)=3.*9.869*E(K)-3.*REALK+2.*9.869
      COEF(3)=-3.*REALK*E(K)+3.*9.869*(E(K)**2)
      COEF(4)=-3.*REALK*(E(K)**2)
      N=4
      TOL=X02AAF(DUMMY)
      IFAIL=0
      CALL C02AEF(COEF,N,REZ,IMZ,TOL,IFAIL)
      IF(IFAIL.EQ.0) GOTO 10
      WRITE(6,15) IFAIL
   15 FORMAT('IFAIL= ',I4)
      GOTO 101
   10 WRITE(6,30) (REZ(I),IMZ(I),I=1,3)
   30 FORMAT(   T20,'ROOTS OF EQUATION',3(2F12.8//))
  100 CONTINUE
  101 STOP
      END
```

CODE+GLA+SYMTABS+ARRAYS = 1088+ 536+ 112+ 608= 2344 BYTES

*COMPILATION SUCCESSFUL