Inelastic Behaviour of Reinforced Brick and Reinforced Concrete Members under the Action of Lateral Loads

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By

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Certified that the thesis entitled "Inelastic Behaviour of Reinforced Brick and Reinforced Concrete Members . Under the Action of Lateral Loads", which is being submitted by Shri Rajesh Verma in partial fullfilment for the award of Master of Engineering in "Earthquake Engineering" with specialisation in "Structural Dynamics" of the University of Roorkee, Roorkee is a record of students own work carried out by him under my supervision and guidance. The matter embodied in this thesis has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for an effective period of Seven months from 15th December 1970 to 26th July, 1971 for preparing this thesis for Master of Engineering Degree at this University.

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ABSTRACT

Analytical and experimental study of the behaviour of reinforced brick and reinforced concrete members subjected to lateral loads is presented. Theoretical load-deflection curves for such members with certain assumed stress-strain characteristics of steel, brick and concrete is compared with those obtained experimentally. The variable parameters in this study include percentage of steel, the shape of stress - strain curve for brick and concrete and the strain level in these materials. The study is aimed at arriving a set of structural parameters for economical design of members taking into account the energy absorbing capacity of the materials.

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CHAPTER I

INTRODUCTION :

Earthquake resistant design of structures is a vibration problem. The vibrations result from an earthquake shock and can be resolved in any three mutually perpendicular directions. A structure is considered safe if it is designed to with stand the components of the vibrations in the three directions simultaneously. The predominant direction of vibration however, is horizontal. The usual approach for design of ordinary structure is to analyse the structure for a horizontal force acting at the centre of gravity and is given by,

$$F = \alpha_H . W \qquad \ldots \qquad (1.1)$$

where

α_H = a seismic coefficient

W = weight considered to be acting at the centre of gravity of the structure.

For important structures however, more detailed investigation is necessary.

In countries like India which have large areas falling in heavy earthquake zone, buildings must be designed to withstand the effects of large and medium size earthquakes.

Buildings which can be expected to withstand such shocks must be designed with relatively high resistance to lateral forces. Experience from past earthquakes indicates that damage to structure is closely associated with the properties of the material of the construction besides the soil and geological conditions of the site. In India buildings fall mainly under following categories

1. TIMBER HOUSES :- These are generally constructed in hilly regions and if designed properly are perhaps the best suited in earthquake zones, as these are light and are consequently subjected to smaller inertia forces. But the shortage of well seasoned timber, fire hazards, rotting of timber and damage by white ants are serious disadvantages of this form of construction.

2. UNREINFORCED BRICK HOUSES : - These constructions have thick walls and heavy floors. Such buildings are relatively rigid structures and have low natural period of vibration with low damping in the elastic range. During earthquakes, their spect al response is high and since unreinforced brick work has very little resistance, it suffers heavy damage.

3. BRICKWORK WITH TIMBER SCANTLINGS : - This form of construction prevents total collapse of the buildings during an earthquake and the damage is repairable. But timber has

disadvantages of being costly and also it has poor bond with masonry. Therefore such structures are generally not used in modern constructions. In place of timber scantlings, pretensioned concrete members are now used. These are cheaper than timber and bond well with the brick work in cement mortar.

4. REINFORCED CONCRETE/BRICK BUILDINGS : - Experience with reinforced concrete and reinforced brick buildings has been generally good. This indicates the usefulness of introducing steel in brick work or concrete to increase not only their resistance in tension but also their energy absorbing capacity through ductility of tension steel. Since an earthquake forces may occure in any direction and any face may be tension or compression, doubly reinforced brick or concrete sections are to be invariably used in earthquake resistant structures.

About 70% of the population of India live in villages and build houses in brick masonry with mud, lime-surkhi or cement - sand mortar. Since the use of costly and expensive materials such as steel and well seasoned timber under the present economic conditions is not advisable, we look forward to reinforced brick and reinforced concrete members.

A current design procedure is to allow structure to behave elastically for small size shocks and during severe shocks the structures are permitted to undergo inelastic deformations. Thus it is necessary to undertake the study of the inelastic response of reinforced brick and reinforced concrete structural members under the action of lateral loads.

OBJECT AND SCOPE :

The present study aims at determining the lateral load carrying capacity of the building elements taking into account their energy absorbing capacity. Energy absorption can be increased by allowing some damage through yielding of steel and some inelastic deformations. For this reason the behaviour of reinforced brick and reinforced concrete piers in the inelastic range is studied and their load - deflection characteristics as also the strain variation in the section under the action of lateral load are examined analytically and the results checked experimentally. Since a shear wall in a brick building can be considered as a series of piers, this study leads to some useful information regarding the inelastic design of shear walls also.

A theoretical analysis for calculation of deflections of reinforced brick and reinforced concrete piers in the inelastic range is presented here. Calculations of deflection depend on the moment of resistance of the section at different stress conditions. A general expression for computation of moment of resistance of the section is developed taking into account the yielding of tension steel from extreme outer fibre to its complete yielding. Initial flexural rigidity (EI Value) is found out by experiments and is used for estimating EI values for other conditions of yielding of tensile steel and for different positions of neutral axis when tension steel has yielded completely.

Expressions for calculation of moment of resistance of the reinforced concrete section for different strain levels in concrete and tension steel are also developed and presented here. Variation of strain in the section has also been studied for the theoretical model as well as on experimental ones.

The theoretical results of computation of load and deflection characteristics of reinforced brick and concrete piers and corresponding strain variation in the section are checked with experimental results.

Theoretical analysis of reinforced brick shear wall for calculation of ultimate lateral load carrying capacity is also presented in the thesis.

CHAPTER II

BRIEF REVIEW

2.1.: GENERAL : - The problem of earthquake resistant design of brick buildings has received very little attention of investigators in the past. The School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee, has been actively engaged in studying this for the last few years. A brief review of this is presented here.

2.1.1 An important contribution in the field of unreinforced brickwork is due to Agnihotri (2). In the first phase of this investigation, strength of brickwork in different mortars was studied experimentally. This formed the basis of the computation of the strength of brickwall under the action of lateral loads. Effect of size and placing of openings in a brick wall, was also studied. The following important conclusions were drawn.

1. If the geometrical shape of the wall remains the same, the bigger walls are weaker.
2. Strength of wall increases linearly with increase of tensile strength of bond of masonry.
3. For particular value of tensile strength of bond of masonry and width of shear wall, following conclusions

were drawn. :

(I) If size and placing of opening is fixed, the strength of wall increases with increase in width/height ratio.

(II) When width/height ratio of the shear wall is fixed, strength of wall increases for higher and centrally located openings if their size is kept constant, otherwise strength decreases for larger openings with fixed placings.

(III) Strength of wall is practically constant for same placing and area of opening if length/height ratio of opening is varied.

In the above analysis the effect of cross wall was not taken into account. The cross walls also contribute to the forces and also provides the resistance to lateral forces. The resistance of cross wall being small, it was neglected and the strength of single room building was calculated on this basis (3). These studies conclude that in an unreinforced building, sections along the jambs of the openings and along corners are weaker sections and must be safeguarded in the event of an earthquake.

2.1.3 In view of above studies, some methods of strengthening of such buildings(4) were suggested. A relative strength of single room brick house model built in 1:6

cement and sand mortar were tested, when reinforced as follows.

1. Normal construction without any reinforcement
2. Providing a lintel band
3. Providing lintel and plinth band
4. Providing vertical steel at corners only
5. Providing vertical steel at jambs only
6. Providing vertical steel at corners as well as at jambs
7. Lintel band in combination with 4, 5 and 6.

The results of above tests provided the qualitative data for comparing the usefulness of steel reinforcement in different positions and it was found that vertical steel at corners is very effective and that the combination of horizontal steel at lintel level with vertical steel at corners is still stronger, while vertical steel at jambs alone does not increase the over all resistance. This was confirmed by experimental studies (5) on shear walls with openings and incorporating various types of strengthening methods.

2.1.4 In order to understand the behaviour of such walls better a numerical analysis was made in which the wall was treated as a plate with openings (6, 7) and wall deflections and stress distribution at various points in the wall were computed. It was concluded that the wall behaves as a bent for small height/width ratio and as the height of wall increases, the cantilever effect starts becoming predominant. This study has confirmed that shear wall in brick buildings can be treated as a series of piers or bents and the forces in these could be computed accordingly.

2.1.5 After the distribution of forces has been obtained, the problem of designing R.B. section subjected to direct and bending forces was taken up. Krishna and Chandra (8) analysed the R.B. sections taking into account the elastic and inelastic behaviour of brick and steel. If the resistance of brickwork in tension was as good as in compression, the piers could take large horizontal forces without damage, However since it is not so, it appears necessary that its energy absorbing capacity is increased by providing steel reinforcement on tension faces. Energy absorbing capacity can be increased appreciably by accepting some damage through yielding of steel and permitting some inelastic deformations. Also energy absorbing capacity of steel should not exceed the energy absorbing capacity of brick work because it would have no use when brick has failed.

A maximum and minimum percentage of steel was obtained based on criteria that energy of steel is not more than energy of brick work, from the following equations.

$$N = \frac{1}{1 + \frac{\mu_s \cdot \sigma_{st}}{\mu_b \cdot m \cdot \sigma_b}} \qquad \ldots \qquad (2.1)$$

$$p = \frac{\sigma_b \cdot N}{2\,\sigma_{st}} \left[2 - \frac{1}{\mu_b} \right] \qquad \ldots \qquad (2.2)$$

$$= \frac{m \cdot N}{3(2\mu_s - 1)} \left[\frac{1}{\mu_b} + 3(\mu_b - 1) \right] \left(\frac{\sigma_b}{\sigma_{st}} \right)^2 \qquad \ldots \qquad (2.3)$$

in which,

p = percentage of steel

σ_b = stress in brickwork

σ_{st} = stress in tensile steel

μ_s = ductility in tension steel

μ_b = ductility in brick

m = modular ratio $\left(\frac{E_s}{E_b} \right)$

N = distance of neutral axis from the extreme compression fibre.

Hence the percentage of steel should be in between these two limits given by equations 2.2 and 2.3 to have a full utilisation of steel and brickwork.

2.1.6 Goel (9) worked on the behaviour of brick piers subjected to lateral and vertical loads and extended the work to concrete piers also. The effect of reinforcement on ultimate moment capacity and ductility factor of tensile steel of singly reinforced rectangular brick section was investigated. The lateral load carrying capacity of a doubly reinforced rectangular pier under certain axial load was also studied. Similarly behaviour of doubly reinforced concrete section was also studied assuming certain stress - strain characteristic of concrete. Fundamental frequency of vibration of singly and doubly reinforced brick rectangular cantilever piers was studied experimentally and compared with those obtained theoretically. Some experiments carried out to obtain load-deformation characteristics were also reported.

2.1.7 Jain (10) carried out theoretical analysis of brick shear walls with openings. The moments, shears and axial forces in the piers were worked out using Bent method for various level of lateral loads. Equal amount of reinforcement was considered to be placed on both faces in piers and position of neutral axis was found by elastic analysis. The ultimate moment of resistance of the reinforced brick and concrete section with different percentages of steel and various cover of steel was worked out for reinforced brick and concrete. These results were presented in graphical form also which help in designing sections. With the help of these curves, for desired ductility in steel and concrete, the value of neutral axis, percentage of steel, cover and ultimate moment of resistance of the section can be directly obtained.

Experimental verification of theoretical results in the above study revealed discrepancies and experimental load - deflection characteristics of reinforced brick and concrete piers were not tallying with theoretical results. The present work has incorporated modifications in theoretical approach. The approach presented here is more realistic and yields results which compares reasonably well with those of experimental absorvations. The approach is extended for computation of ultimate lateral load carrying capacity and deflections of plain shear wall also. The study may help a designer in working out economical design of members taking into account the energy absorbing capacity of the materials in the inelastic range.

CHAPTER III

THEORETICAL ANALYSIS OF REINFORCED BRICK PIERS

3.1 GENERAL : - As emphasized in chapter II, brick walls must be reinforced with steel in order to increase their resistance to lateral forces. In order to do so efficiently, a knowledge of the stress - strain characteristicsof the two materials is absolutely necessary. For the elastic range, the initial slope of the stress - strain curve may be sufficient for calculations of deflections, but for studying the behaviour in the inelastic range, complete stress - strain curve must be known. This chapter describes the theoretical analysis of reinforced brick piers in the elastic and inelastic range.

3.2 DESCRIPTION OF MODEL :-In present study, the stress - strain relationship for reinforcing steel is assumed to be elasto-plastic (Fig. 3.2) and for brick a linear relationship between stress and strain is adopted (Fig. 3.3). Since in dynamic case any one of two faces of a pier could be a tension face, equal reinforcement has been considered on both the faces. Figure 3.1 shows the section chosen for the purpose of the study. Equal percentage of reinforcement is placed on each face at a cover 'ad'. The analysis is done for both elastic and inelastic case and is presented in following sections.

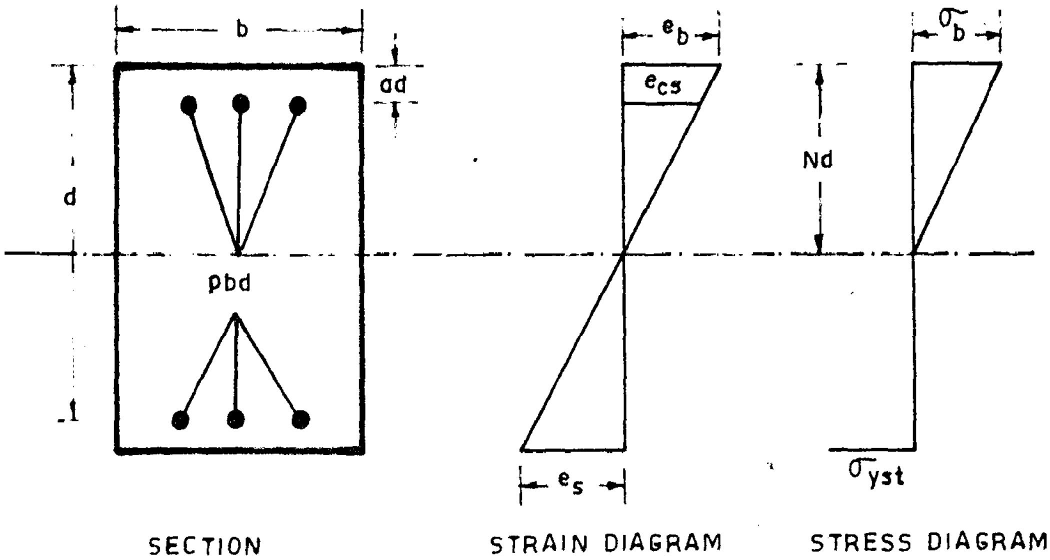
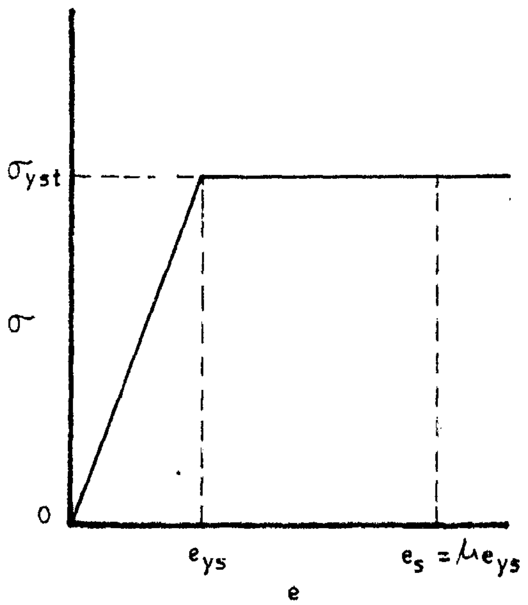
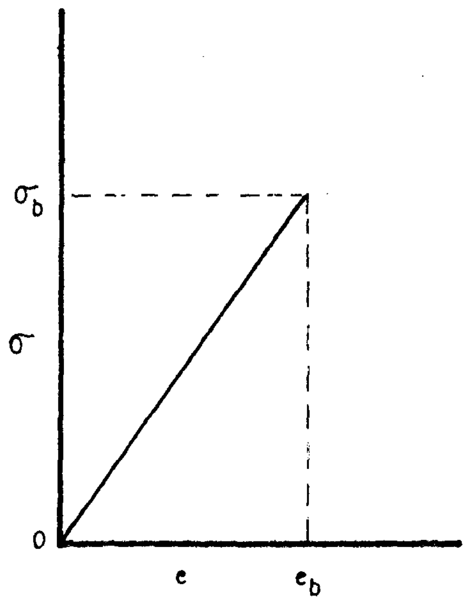


FIG. 3.1



STRESS STRAIN DIAGRAM FOR STEEL

FIG. 3.2



STRESS STRAIN DIAGRAM FOR BRICK

FIG. 3.3

3.3 ANALYSIS IN ELASTIC RANGE : -

3.3.1 ASSUMPTIONS : Following assumptions are made in this study.

1. Brickwork has the stress - strain curve shown in Fig. 3.3 and it behaves as an elastic material upto its failure.

2. Tension is resisted by steel only and its stress strain diagram is assumed to be elasto-plastic as shown in figure 3.2.

3. Plane section remains plane after bending. Figure 3.1 shows the distribution of strains in the section of the brick column.

3.3.2 POSITION OF NEUTRAL AXIS : - Let the neutral axis lie

at a distance 'Nd' below the top fibre of brick. The position of neutral axis is found by taking moments of the compression and tension areas about the neutral axis. This gives.

$$b.Nd. \frac{Nd}{2} + (m - 1).\ pbd\ (Nd - ad) = m.pbd.(d - ad) \quad \ldots (3.1)$$

in which

d = effective depth of the section

b = width of the section

m = Modular ratio$\left(\frac{Es}{Eb}\right)$

p = percentage of steel

a = cover for steel (fraction of d)

This on simplification gives,

$$N^2 + 2(m-1).p.(N-a) - 2\,mp\,(1-N) = 0 \qquad \ldots \quad (3.2)$$

If approximately $(m-1) = m$, N is given by

$$N = -2mp + \sqrt{4m^2p^2 + 2mp(1+a)} \qquad \ldots \quad (3.3)$$

Equation (3.3) holds good for sections with equal percentage of reinforcement in the compression and tension zone.

3.3.3 MOMENT OF RESISTANCE : - Lever arm coefficient for equally reinforced section is given by

$$j = (1 - \frac{N}{3}) + (\frac{N}{3} - a)\ \frac{N - a}{(1 - N)} \qquad \ldots \quad (3.4)$$

Knowing the lever arm, moment of resistance of the section can be found from following expressions. With respect to compression side taking moment of all compressive forces about the tension steel,

$$M_1 = \left[1/2\ N + (m-1).p.\ \frac{N-a}{N}\right] \sigma_b.bd.jd \quad ..(3.5)$$

in which,

σ_b = stress in brick

and with respect to tension side, taking moment of force of tension about the centre of gravity of compressive forces we get

$$M_2 = p.\ \sigma_s.\ j.\ bd^2 \qquad \ldots \quad (3.6)$$

in which σ_s = stress in tension steel

The smaller of the values of M_1 or M_2 gives the value of the external bending moment to which the section can be subjected upto the elastic limit.

If the section is subjected to an external bending moment M, then, stress in tension steel is given by

$$\sigma_{s'} = \frac{M}{p.bd.\ jd} \qquad \ldots (3.7)$$

and strain in tension steel by

$$\epsilon_s = \frac{\sigma_s}{E_s} \qquad \ldots (3.8)$$

where E_s = Modulus of elasticity of steel

From strain diagram (Fig. 3.1), strain in brick

$$\epsilon_b = E_s \frac{N}{(1 - N)} \qquad \ldots (3.9)$$

From these equations strain levels in tension steel and brick in the elastic range for different lateral load can be calculated and variation of strain in the section obtained.

3.4 ANALYSIS IN INELASTIC RANGE ;

3.4.1 POSITION OF NEUTRAL AXIS : - From Fig. 3.1 showing strain diagram, we get

$$\sigma_{sc} = m.\sigma_b.\frac{N-a}{N} \qquad \ldots (3.10)$$

where σ_{sc} = stress in the compression steel

Also the distance of neutral axis from the compression fibre is given by,

$$N = \frac{1}{1 + \frac{\mu . \sigma_{yst}}{m . \sigma_b}} \quad \ldots \quad (3.11)$$

in which,

σ_{yst} = yield stress in tensile steel

μ = ductility in steel, which is ratio of strain in steel to its yield strain.

Simplifying equation 3.11 we get,

$$\frac{\sigma_{yst}}{\sigma_b} = \frac{m}{\mu} \left(\frac{1-N}{N} \right) \quad \ldots \quad (3.12)$$

Further, equating the force of tension to force of compression, one obtains

$$1/2\ b.Nd.\ \sigma_b + p.\ bd.\ \sigma_{sc} = p.bd.\ \sigma_{yst} \quad \ldots \quad (3.13)$$

Substituting the value of σ_{sc} from equation 3.10 in 3.13 we get,

$$1/2\ \ N + p.m \frac{N-a}{N} = p. \frac{\sigma_{yst}}{\sigma_b} \quad \ldots \quad (3.14)$$

From equation (3.12) and (3.14) we get,

$$\frac{N}{2} + p.m \frac{N-a}{N} = p \frac{m}{\mu} \left(\frac{1-N}{N} \right) \quad \ldots \quad (3.15)$$

From equation 3.15, one concludes that the position of neutral axis in the inelastic range depends on the value of μ alone for a given value of p, a, and m. Thus neutral axis for any value of ductility of steel μ_s, can be calculated in the inelastic range from equation 3.15.

3.4.2. MOMENT OF RESISTANCE : - Before ultimate moment of resistance is reached, tension steel starts yielding in stages from the extreme outer fibre to extreme inner fibre and the moment of resistance of the section increases from the first stage and reaches its ultimate value at full yield of the steel. The analysis given below takes into account the actual behaviour of the member and is therefore more realistic.

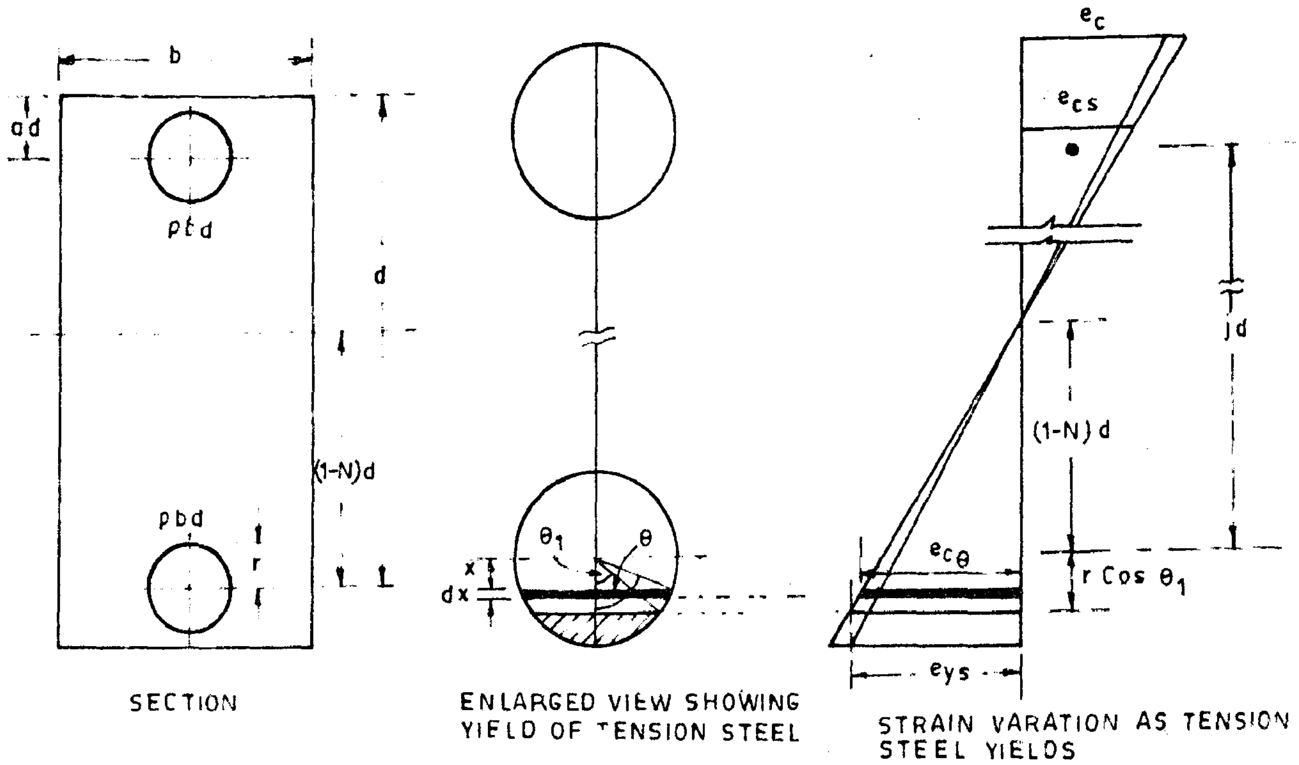
Consider that yielding of tension steel has occured upto an angle θ_1, as shown in Fig. No. 3.4. The stress in the yielded portion of the tension steel will be equal to the yield stress of the steel. The moment of resistance of the section is given by

$$M_{r\theta} = \int_0^{\theta_1} 2r.\sin\theta \; dx.\sigma_y(jd + X) + \int_{\theta_1}^{\pi} 2r.\sin\theta \; dx \; \frac{e_y \; (1-N)d+x \; E_s}{(1-N)d+r\cos\theta_1)} .(jd + x) \quad \ldots (3.16)$$

equation (3.16) on Integration gives the solution

$$M_{r\theta} = \Bigg[\frac{jd}{2}(\theta_1 - 1/2.\sin 2\theta_1) + \frac{r}{3}\sin^3\theta_1 + \frac{1}{(1-N)d+r\cos\theta_1} \Bigg\{ \frac{(1-N)j.d^2}{2}(\pi - \theta_1 + 1/2.\sin 2\theta_1) - \frac{(1-N+j)\;d.r}{3}.\sin^3\theta_1 + r^2(1/8(\pi-\theta_1 - \frac{\sin 2\theta_1}{2}) - \sin^3\theta_1.\cos\theta_1 \Bigg\} \Bigg] 2r^2.\sigma_y \quad \ldots\ldots (3.17)$$

It is assumed in the analysis that the position of the neutral axis remains unchanged throughout the charge of



MOMENT OF RESISTANCE OF THE SECTION TAKING TRANSITION OF TENSION STEEL FROM ELASTIC TO INELASTIC

FIG. 3.4

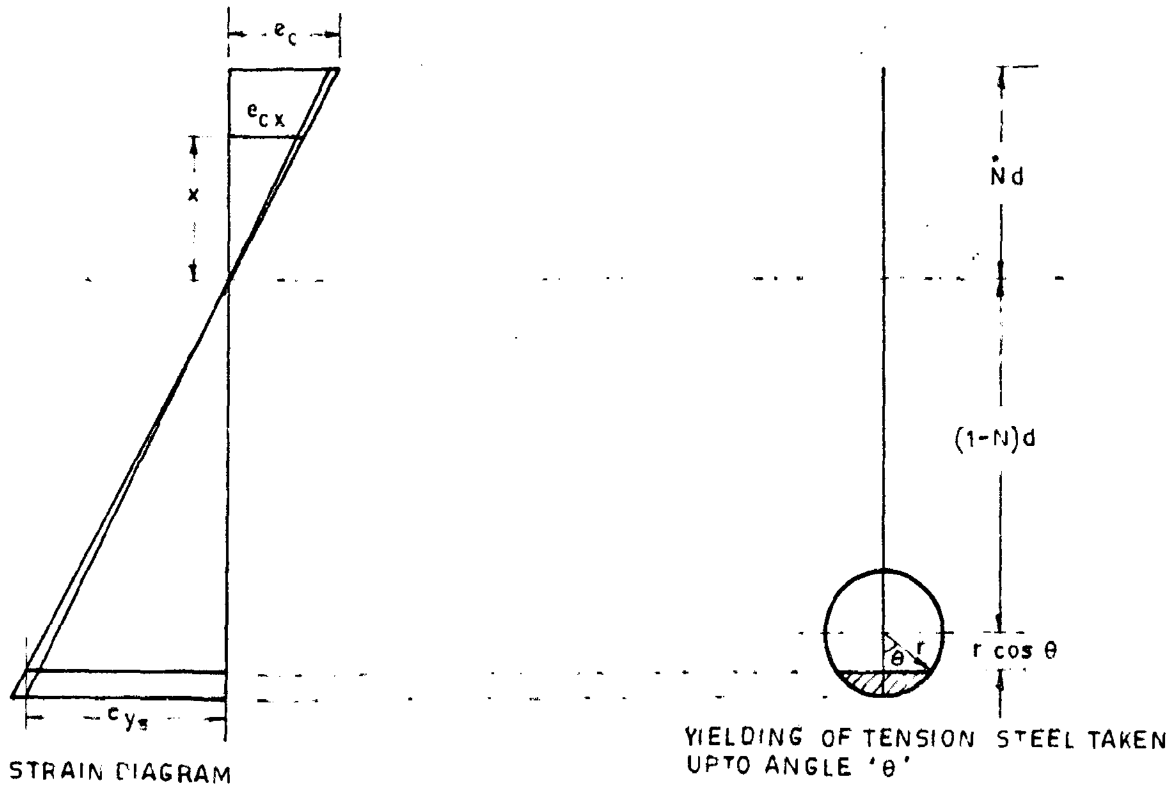


FIG. 3.5

tension steel from elastic state to inelastic state. However the position of neutral axis does shift upwards when tension steel starts yielding i.e. as θ_1 varies from 0^0 to π, but it is assumed that this shift is very small and has little effect on the moment of resistance of the section. Thus for all calculations the neutral axis coefficient 'N' is taken same as in the case of elastic conditions

The ultimate moment of resistance of the section by taking moment about tension steel of force of compression in brick and compression steel is worked out as follows ,

$$M_{bu} = 1/2.b\ Nd.\ \sigma_b(d - \frac{Nd}{3}) + m\ p.bd.\ \sigma_b(\frac{N-a}{N})(d-ad) \quad ...(3.18)$$

or

$$M_{bu} = \sigma_{b.}bd^2 \left[1/2\ \ N(1 - \frac{N}{3}) + m.p(\frac{N-a}{N})\ (1-a) \right] \quad ...(3.19)$$

It is obse ved that the moment of resistance of the section does not increase as the column goes in the inelastic range as any increase in applied moment results in the cracking of the section and to balance the force of compression and force of tension, the neutral axis merely shifts upwards.

3.4.3. COLUMN DEFLECTIONS UNDE THE ACTION OF LATERAL LOAD IN THE ELASTIC RANGE :

Case 1 BOTH ENDS OF COLUMN FIXED AND NO ROTATION ALLOWED AT A AND B.

Deflection of the point B with respect to tangent at point A as shown in Fig. 3.11 is given by,

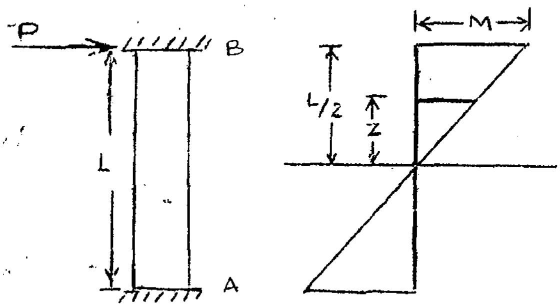


Fig. 3-11

$$\Delta = \int_A^B \frac{M}{EI} Z.dZ \quad \ldots(3.20)$$

in which EI = flexural rigidity of the column. On taking limits, equation (3.20) gives,

$$\Delta = \frac{M.L^2}{6EI} \quad \ldots \quad \ldots \quad (3.21)$$

Now since $M = \frac{PL}{2}$, equation (3.21) may be written as

$$\Delta = \frac{PL^3}{12\ EI} \quad \ldots \quad (3.22)$$

From equation (3.22) deflection of the column fixed at top and bottom under the action of lateral load P can be calculated easily.

Case 2 COLUMN ACTS AS CANTILEVER : - Deflection of point B, with respect to tangent at A is given by the well known. expression,

$$\Delta = \frac{PL^3}{3EI} \quad \ldots \quad \ldots \quad (3.23)$$

From equation (3.23) deflection of the cantilever column can be calculated. Equation (3.22) and (3.23) on comparison show that deflection of the cantilever column is 4 times the deflection of column fixed at top and bottom under the action of equal horizontal lateral load P.

Case 3 : DEFLECTION OF PORTAL FRAME WITH DIFFERENT END CONDITIONS

Generally the end conditions of the piers in a structure are such that it can be considered as fixed at top and bottom. The theoretical analysis for lateral loads and corresponding deflections of such piers is presented in this section. To verify these theoretical results experimentally, it is necessary to simulate conditions of fixity at top and bottom of the pier. For this it was decided to have two columns suitably placed and joined rigidly at top by a beam. However in the experiments, the rigidity of the joint could not be achieved inspite of best efforts. Therefore for the purpose of obtaining more realistic theoretical results, top end joint is considered as imperfect. This has been done through use of a parameter α, such that it relates the rotations of the column end with that of the beam end. This is described in the following paragraphs.

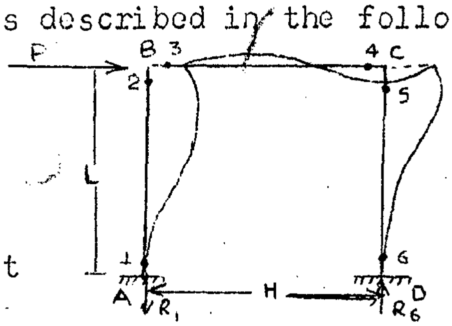


Fig. 3.12

Consider portal frame ABCD as shown in the figure 3.12. Let rotations of points 1, 2, 3, 4, 5 and 6 be θ_1, θ_2, θ_3, θ_4, θ_5 and θ_6 respectively. In this case as base is fixed and no rotation is allowed,

$$\theta_1 = \theta_6 = 0$$

Also $\theta_2 = \theta_5$

Also as top end joint B and C are not perfect, Let

$\theta_3 = \theta_4 = \alpha\,\theta_2$

where α = a constant varying between 0 and 1 and for perfect joint $\alpha = 1.0$

Applying slope deflection equations to the various members we have

$$M_1 = \frac{2EI_c}{L}\left[\theta_2 - \frac{3\Delta}{L}\right] \quad \ldots\ldots \quad (3.24)$$

$$M_2 = \frac{2EI_c}{L}\left[2\theta_2 - \frac{3\Delta}{L}\right] \quad \ldots\ldots \quad (3.25)$$

$$M_3 = \frac{2EI_b}{H}\left[2\theta_3 + \theta_4\right] \quad \ldots\ldots \quad (3.26)$$

$$M_4 = \frac{2EI_b}{H}\left[2\theta_4 + \theta_3\right] \quad \ldots \quad (3.27)$$

$$M_5 = \frac{2EI_c}{L}\left[2\theta_5 - \frac{3\Delta}{L}\right] \quad \ldots \quad (3.28)$$

$$M_6 = \frac{2EI_c}{L}\left[\theta_5 - \frac{3\Delta}{L}\right] \quad \ldots \quad (3.29)$$

In these equations there are two unknowns θ_2 and Δ as θ_3, θ_4 and θ_5 can be expressed in terms of θ_2, and where,

EI_c = Flexural rigidity of the column

EI_b = Flexural rigidity of the beam

H = Span of beam

Now the conditions of equilibrium are, at joint B, and C,

$$M_2 + M_3 = M_4 + M_5 = 0 \quad \ldots \quad (3.30)$$

and $$M_1 + M_2 + M_5 + M_6 + P.L. = 0 \quad \ldots \quad (3.31)$$

Let $\frac{2EI_c}{L} = K_c$ and $\frac{2EI_b}{H} = K_b$ Then from condition 3.30 we get After substitution of

$$\theta_3 = \alpha.\theta_2$$

$$\theta_4 = \alpha.\theta_2$$

$$\theta_5 = \theta_2, \quad \text{gives}$$

$$M_1 = Kc \left[\theta_2 - \frac{3\Delta}{L} \right] \quad \ldots \quad (3.32)$$

$$M_2 = K_c \left[2\theta_2 - \frac{3\Delta}{L} \right] \quad \ldots \quad (3.33)$$

$$M_3 = 3\alpha\, K_b\, \theta_2 \quad \ldots \quad (3.34)$$

$$M_4 = 3\alpha\, K_b\, \theta_2 \quad \ldots \quad (3.35)$$

$$M_5 = K_c \left[2\theta_2 - \frac{3\Delta}{L} \right] \quad \ldots \quad (3.36)$$

$$M_6 = K_c \left[\theta_2 - \frac{3\Delta}{L} \right] \quad \ldots \quad (3.37)$$

Now the conditions of equilibrium at joints B and C, as given in equations (3.30) and (3.31) yields on substitution,

$$\theta_2 \left(2 + 3\alpha \frac{K_b}{K_c}\right) = 3.\frac{\Delta}{L} \quad \ldots\ldots \quad (3.38)$$

and $$\theta_2 = 2\frac{\Delta}{L} - \frac{PL}{6K_c} \quad \ldots\ldots \quad (3.39)$$

Substitution of the value of θ_2 from equation 3.38 in equatio 3.39 and taking $\frac{K_b}{K_c}$ = b, yields on simplification,

$$\Delta = \frac{PL^3}{24\ EI_c} \cdot \frac{2(2 + 3\alpha b)}{(1+6\alpha b)} \qquad \ldots. \qquad (3.40)$$

Now for pier fixed at top and bottom, its deflection under lateral load is given by euation 3.22 as

$$\Delta_{TBF} = \frac{PL^3}{24EI_c} \qquad \ldots \qquad (3.41)$$

where Δ_{TBF} = Deflection of pier which is fixed at top and bottom and 'P' is the applied load on two columns.

Thus equation 3.40 may be rewritten as

$$\Delta = \Delta_{TBF}. \frac{2(2 + 3\alpha b)}{(1+6\alpha b)} \qquad \ldots \qquad (3.42)$$

Equation 3.42 is a general equation giving deflection of the portal for any end condition at top of the pier in the elastic range. The effect of variation of 'α' on the deflection of the portal frame is illustrated in table 3.1 taking ratio of K_b/K_c as 0.5, 1.0, 1.5 and 2.0.

Table 3.1 indicate that as the top joint of the piers with the beam mores from perfect to imperfect with increase in lateral loads, the deflections of the pier goes on increasing.

TABLE 3.1

EFFECT OF VARIATION OF α AND k_b/k_c RATIO ON R B PORTAL DEFLECTIONS

α	Deflection = Δ_{TBF} x				
	$\frac{k_b}{k_c} = 0.25$	$\frac{k_b}{k_c} = 0.5$	$\frac{k_b}{k_c} = 1.0$	$\frac{k_b}{k_c} = 1.5$	$\frac{k_b}{k_c} = 2.0$
0	4.0	4	4	4	4
0.1	3.5	3.32	2.87	2.58	2.36
0.2	3.32	2.87	2.36	2.08	1.88
0.3	3.08	2.56	2.07	1.81	1.65
0.4	2.88	2.36	1.88	1.65	1.52
0.5	2.72	2.20	1.75	1.55	1.43
0.6	2.58	2.08	1.65	1.47	1.37
0.7	2.46	1.97	1.57	1.41	1.32
0.8	2.36	1.88	1.51	1.37	1.28
0.9	2.28	1.81	1.46	1.33	1.25
1.0	2.20	1.75	1.43	1.30	1.23

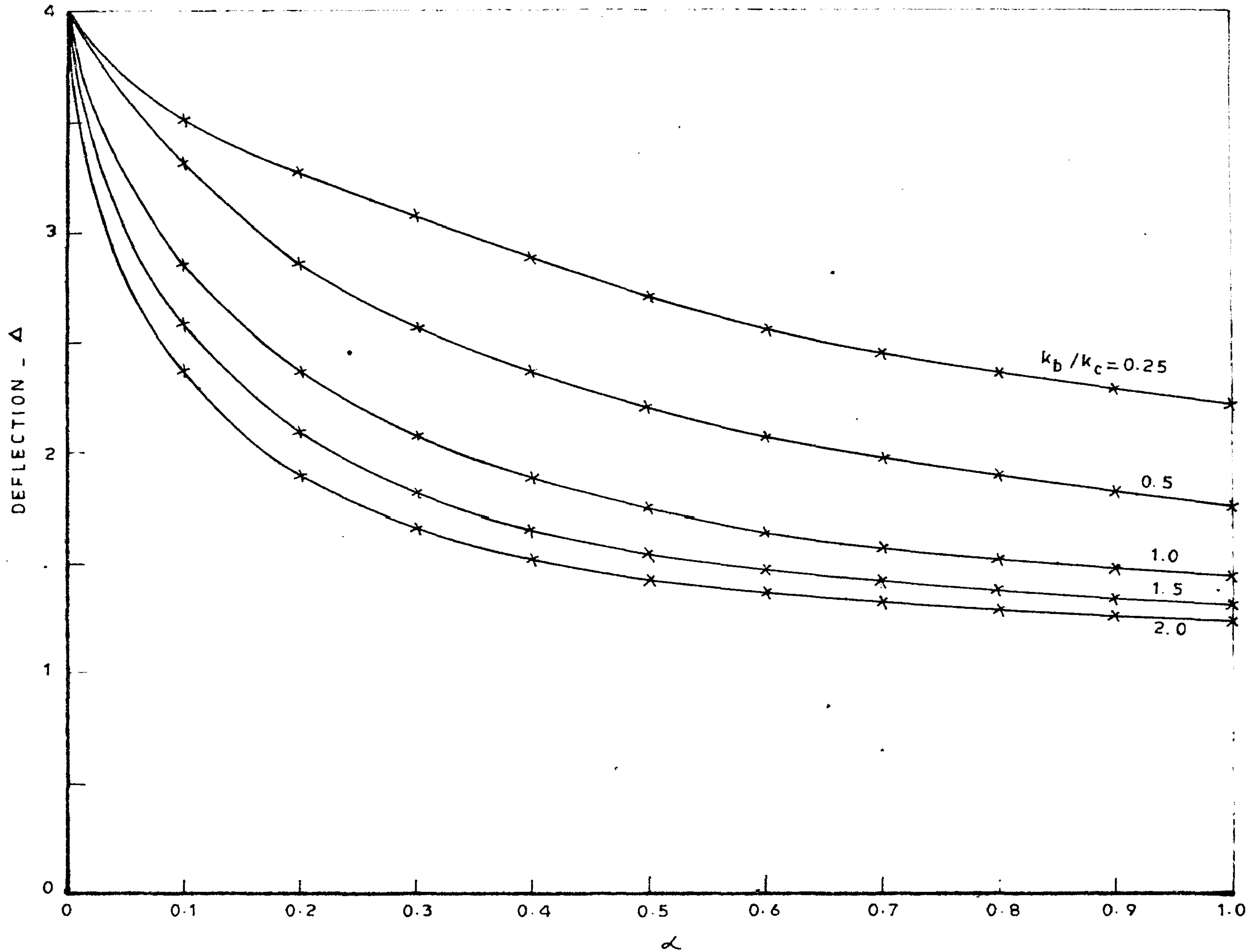


FIG. 3.6_EFFECT OF VARIATION OF α AND k_b/k_c RATIO ON R. B. PORTAL DEFLECTIONS

At $\alpha_1 = 0$, which means that pier is independent of beam and acts as cantilever, the deflection is obtained as 4 times the deflection of pier which is fixed at top and bottom. This is evident from equations 3.22 and 3.23 also.

Effect of change of beam and column stiffness ratios, which may be due to change of modulus of Elasticity of brick, is also taken into account in the above analysis. This indicate that as k_b/K_c ratio increases, the R.B. portal deflection goes on decreasing. The variation of portal deflection with change in value of α_1 for different K_b/K_c ratio is presented in graphical form in Fig. 3.6. which shows the order of difference expected in deflection due to variation of K_b/K_c.

3.4.4 CALCULATION OF MOMENT OF INERTIA OF THE SECTION :

Knowing moment of resistance of the section at any stage of loading, theoretical deflections can be calculated if moment of inertia of the section is known. In the elastic range the position of neutral axis remains unchanged and both tension and compression steels are effective and contribute towards the moment of inertia of the section. However when the tension steel yields then it is assumed that the yielded port of the steel does not contribute towards the moment of inertia of the section. Considering yielding to have occured upto an angle θ_1 the moment of Inertia of the section is found as follows :

Moment of inertia due to compression side about the neutral axis

$$I_c = b. \frac{(Nd)^3}{3} + m.p.bd(Nd.-ad)^2 \quad \ldots \quad (3.43)$$

$$\text{or} \quad I_c = 1/3\ b.(Nd)^3 + m.p.bd^3(N-a)^2 \quad \ldots \quad (3.44)$$

and due to tension steel which has yielded upto an angle θ_1

$$I_t = \int_{\theta_1}^{\pi} 2r \sin\theta\ dx \left[(1 - N)\ d+x\right]^2 \quad \ldots \quad (3.45)$$

equation 3.45 on solution gives,

$$I_t = mr^2(1-N)^2d^2(\pi-\theta_1+1/2 \sin 2\theta_1)+m.2r^4\left\{\frac{1}{8}(\pi-\theta_1 + \right.$$

$$\left. + \frac{\sin 2\theta_1}{2}) - \sin^3\theta_1 \cos\theta_1\right\}$$

$$-\frac{4}{3}r^3(1-N).d.m. \sin^3\theta_1 \quad \ldots.. \quad (3.46)$$

The total flexural rigidty $(E_b.I)$ of the section is thus given by $I = I_c + I_t$

$$E_bI = \frac{1}{3} E_b\ b(Nd)^3 + E_s\left[p.bd^3(N-a)^2 + r^2(1-N)^2d^2 . \right.$$

$$(\pi-\theta_1 + \frac{\sin 2\theta_1}{2}) + 2r^4\left\{\frac{1}{8}(\pi - \theta_1 + \frac{\sin 2\theta_1}{2})\right.$$

$$\left.\left. - \sin^3\theta_1 \cos\theta_1\right\} - \frac{4}{3} r^3 (1 - N)d \sin^3\theta_1\right] \ldots (3.47)$$

The above expression is of the form

$$I = + n.b. \text{ or } E_b I = E_b . a + E_s . b \quad ....(3.48)$$

The value of I for different values of θ_1 can be calculated from above expression.

The value of $E_b.I$ is directly required in calculation of deflection. This value is experimentally found out by taking the free vibration record of the cantilever calumn with the help of a pen recorder. From the record frequency of vibration is calculated. The formula for frequency of cantilever free vibration is,

$$f = 0.56 \sqrt{\frac{EI\ g}{w\ L4}}$$

where E = modulus of Elasticity of the material of the calcumn

g = acceleration due to gravity

w = weight of calumn per cms. height and

L = height of the calumn

This expression gives the value of EI as f, L, w and g are known quantities.

3.4.5 CALCULATION OF AREA OF THE SECTION : The expression for calculation of shear deflection is

$$\Delta_{Shear} = 1.2 \frac{P.L}{G.A} \simeq 2.4 \frac{P.L}{E.A} \quad .... \quad (3.49)$$

where

P = lateral load acting on the column top

E = modulus of elasticity

A = area of uncracked brick section, compression steel and unyielded tension steel.

Calculation of shear deflections require the value of A, which changes from elastic to inelastic range. It can be calculated for different stages of yielding of tension steel as given below.

If tension steel has yielded upto an angle θ_1 as shown in Fig. 3.5 we have,

Area of Strip = $2r, \sin\theta. dx.$

$X = r \cos\theta, \quad dx = -r \sin\theta$

$$A_{t\theta} \quad \text{Area of unyielded steel} = \int_{\theta_1}^{\pi} 2r^2 \sin^2\theta. d\theta$$

$$= r^2 \left(\pi - \theta_1 + \frac{\sin 2\theta_1}{2}\right) \quad ... \quad (3.50)$$

For different values of θ_1 area of unyielded steel can be calculated and effective area of the section calculated as given below,

$$E_b A = E_b b. \ Nd + E_s \cdot A_c + A_{t\theta} \cdot E_s \quad .... \quad (3.51)$$

where A_c = area of compression steel

$A_{t\theta}$ = area of unyielded steel

The total deflection of the calum under the action of lateral load can be obtained as sum of the deflection due to shear shear and that due to bending.

3.4.6 THEORETICAL COMPUTATIONS OF LOAD. - DEFLECTION CHARACTERISTICS OF R.B. PIER USING ABOVE APPROACH

A comutor programme, presented in Apendix ,A, , is made for computation of lateral loads and corresponding deflections of reinforced brick cantilever pier using the above theoretical approach. The variable parameters considered in the analysis are percentage of steel and 'ad' cover for steel. The value of 'a' was varied from 0.10 to 0.25 in steps of 0.05. The quantity of steel reinforcement considered are 0.25, 0.15, 1.0, 1.5 and 2.0 percent. The load - deflection characteristics of such piers were obtained for different percentages of reinforcing steel while keeping the cover for steel as constant. Effect of variation in cover on the column behaviour is also examined. The theoretical results obtained are presented in graphical form in Fig. 3.7 to 3.10 and are also listed in tabular form in appendix ,B, Following conclusions can be drawn from the study of Fig.3.7

to 3.10.

1. The shape of the load - deflection curves is of the general non - linear type and similar to skelton curves given by Jennings (1). Thess curves can be approximated into an elasto - plastic system. However this approach being more realistic, predicts more exact behaviour of reinforced brick piers unde the action of lateral loads.

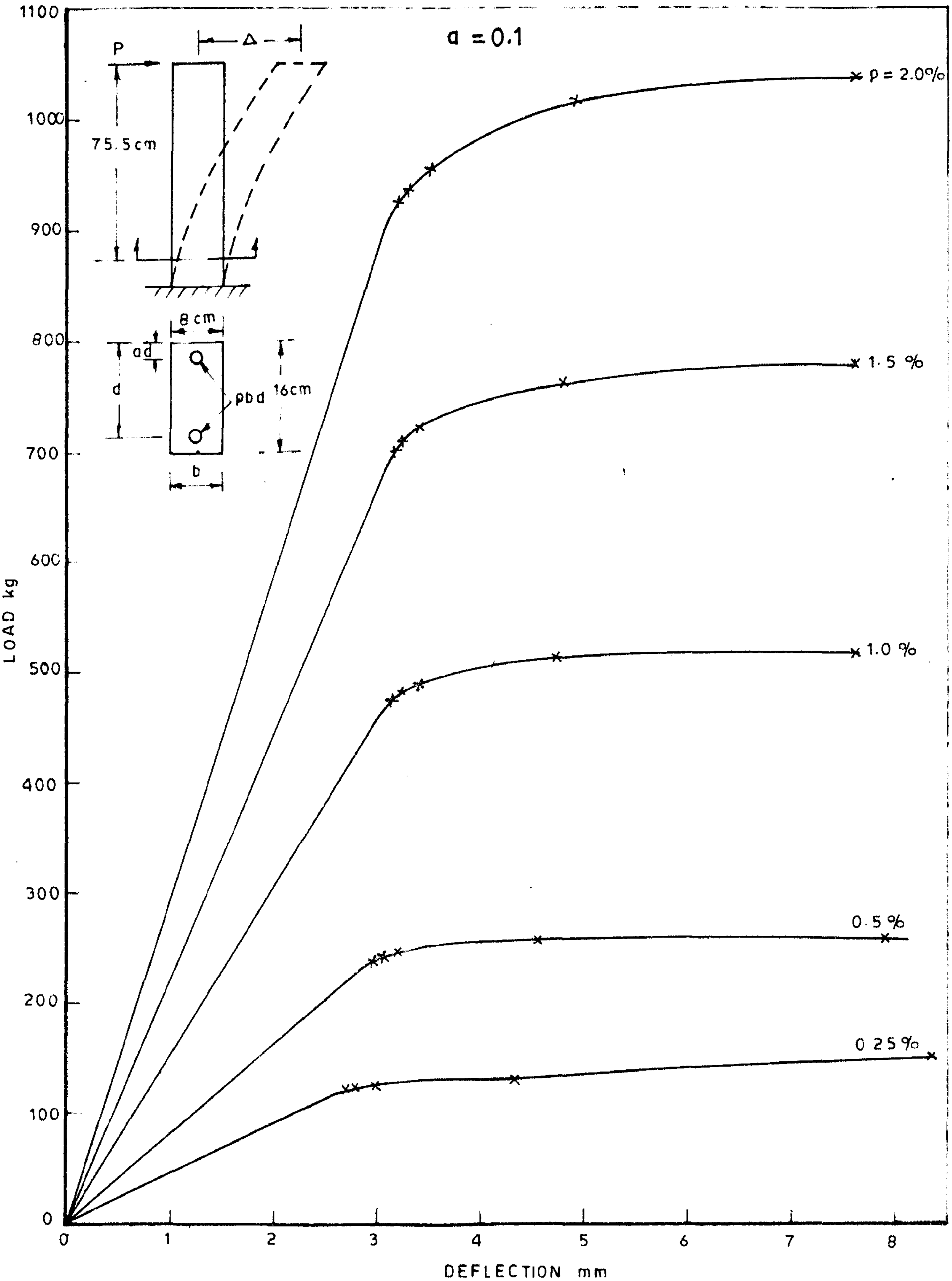
2. The lateral load carrying capacity of the pier is increased in proportion to the percentage of steel for constant cover for steel.

3. The figures also indicate that the increase in lateral load carrying capacity of reinforced brick column may be 10 to 15 percent when tension steel goes from elastic to inelastic range. This increase is found to be more for higher percentages of steel compared to the lower ones for the same cover. Load factors are given in Apendix C.

4. An increase in 'cover' for steel decreases the ultimate lateral load carrying capacity and yielding of tension steel also starts at a lower load and corrosponding deflections are more. Also it can be concluded that by increasing the cover in a

reinforced brick section, the ductility in tensile steel can be decreased if required. However this will have to be done at the cost of some reduction in ultimate lateral load carrying capacity of the section.

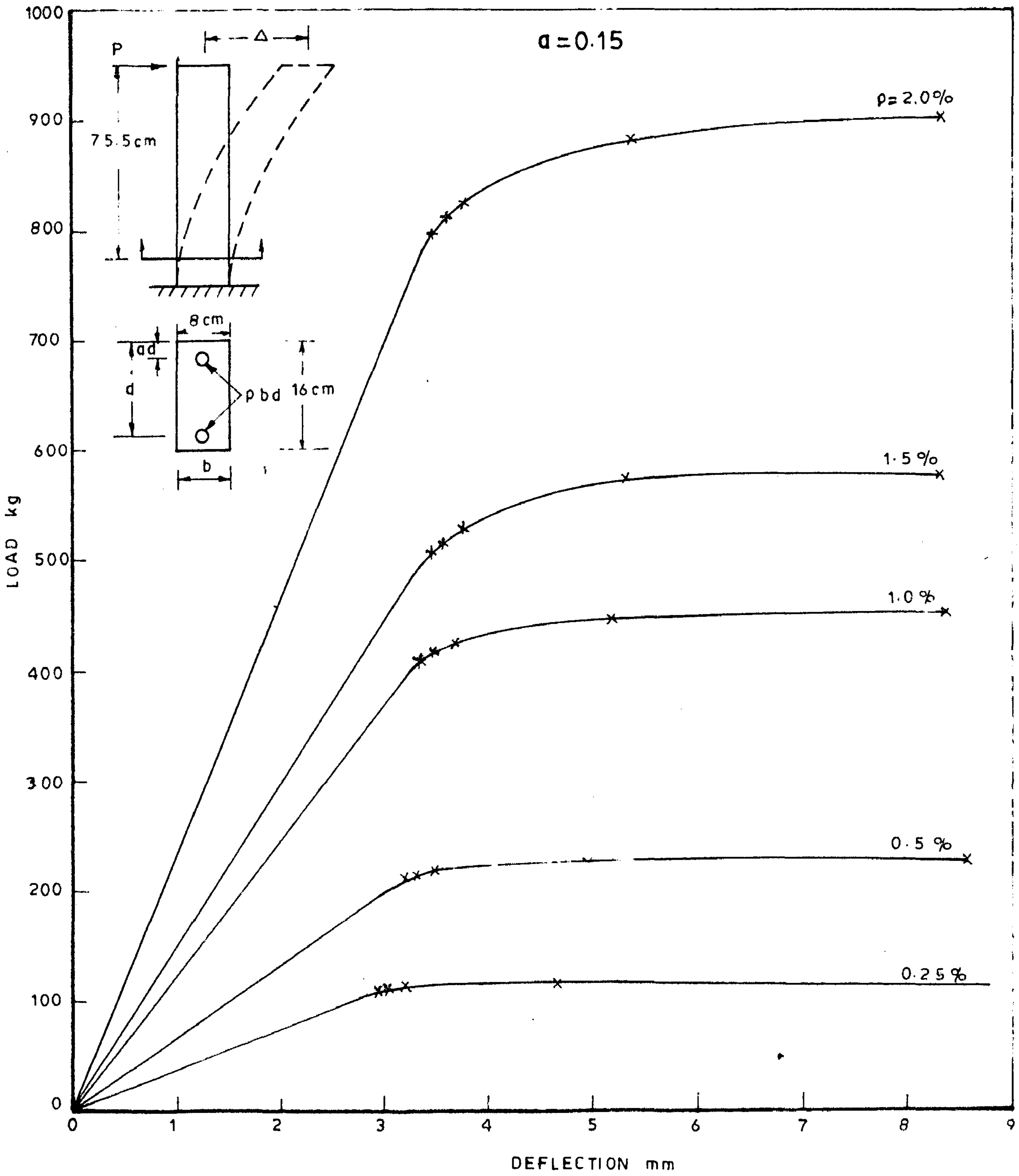
5. When the tension steel yields completely, the compressive stress in brick is not reached to its ultimate and on further loading, the compressive stress in brick increases, neutral axis shifts upwards and strains in the tension steel increases and its goes in the inelastic range. However the increase in load carrying capacity in case of reinforced brick masonry work is not appreciable. Thus for full utilisation of the brick masonry and steel reinforcement, the section is to be so designed, that when brick reaches to ultimate stress, tension steel has some workable ductility.



THEORETICAL LOAD VS DEFLECTION CURVE

REINFORCED BRICK CANTILEVER COLUMN

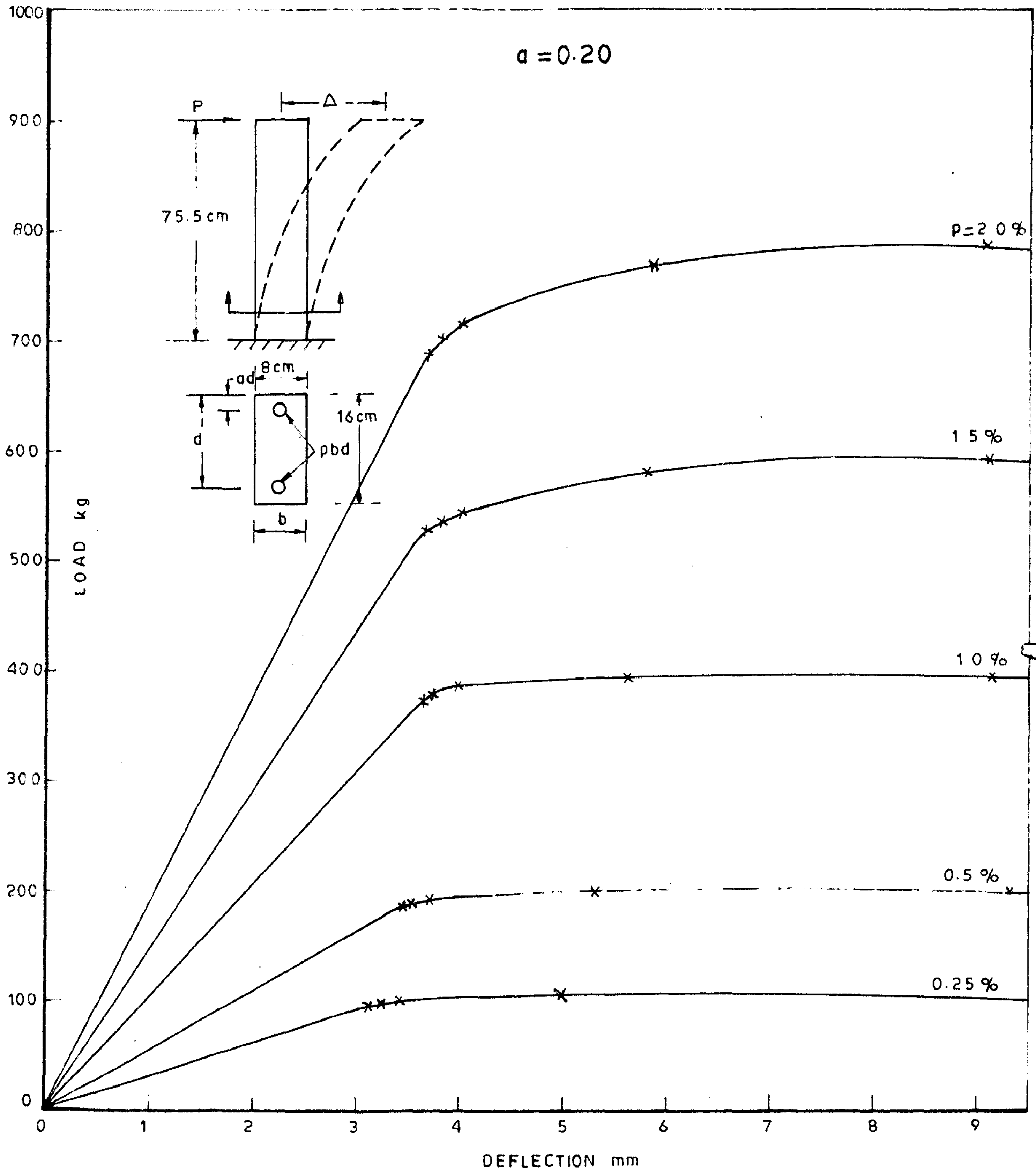
FIGURE_3.7



LOAD VS DEFLECTION CURVE

THEORETICAL REINFORCED BRICK CANTILEVER COLUMN

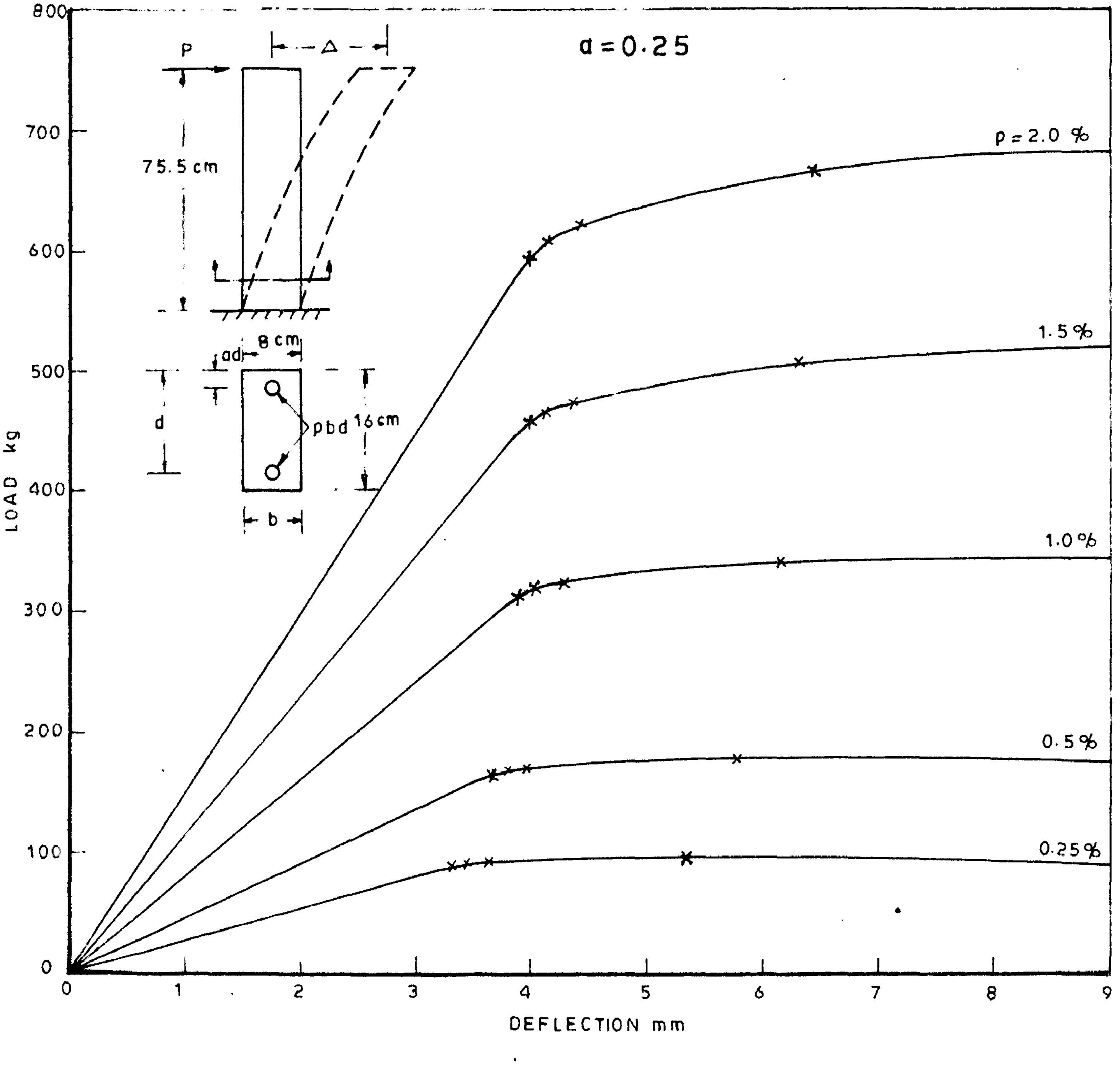
FIGURE_ 3.8



LOAD VS DEFLECTION CURVE

THEORETICAL REINFORCED BRICK CANTILEVER COLUMN

FIGURE _ 3.9



LOAD VS DEFLECTION CURVE

THEORETICAL REINFORCED BRICK CANTILEVER COLUMN

FIGURE_3.10

CHAPTER IV

ANALYSIS OF REINFORCED CONCRETE SECTION

4.1 Expressions in this chapter, have been developed to calculate ductility in tension steel, position of neutral axis and moment of resistance of the section in the inelastic range, for various parameters of the section.

Typical stress - strain curve for unconfined concrete in compression is assumed to be parabolic for the present study of the response of Reinforced concrete sections to applied bending moment which is increased to ultimate. Let μ_c be the ductility in concrete. μ_c is defined as (refer Fig. 4.3)

$$\mu_c = \frac{\text{Ultimate strain in concrete } 'e_{cu}'}{\text{Strain in concrete corrosponding to Maximum stress } 'e_{cm}'}$$

Normally 'e_{cm}' is about 0.002 and ultimate strain 'e_{cu}' varies from 0.003 to 0.005. So in concrete ductility varying from 1.5 to 2.5 can be expected.

Assume the stress - strain curve for concrete has equation

$$\sigma_c = 2\left(\frac{\sigma_{mc}}{e_{cm}}\right) e_c - \left(\frac{\sigma_{mc}}{e_{cm}^2}\right) e_c^2 \quad \ldots. \quad (4.1)$$

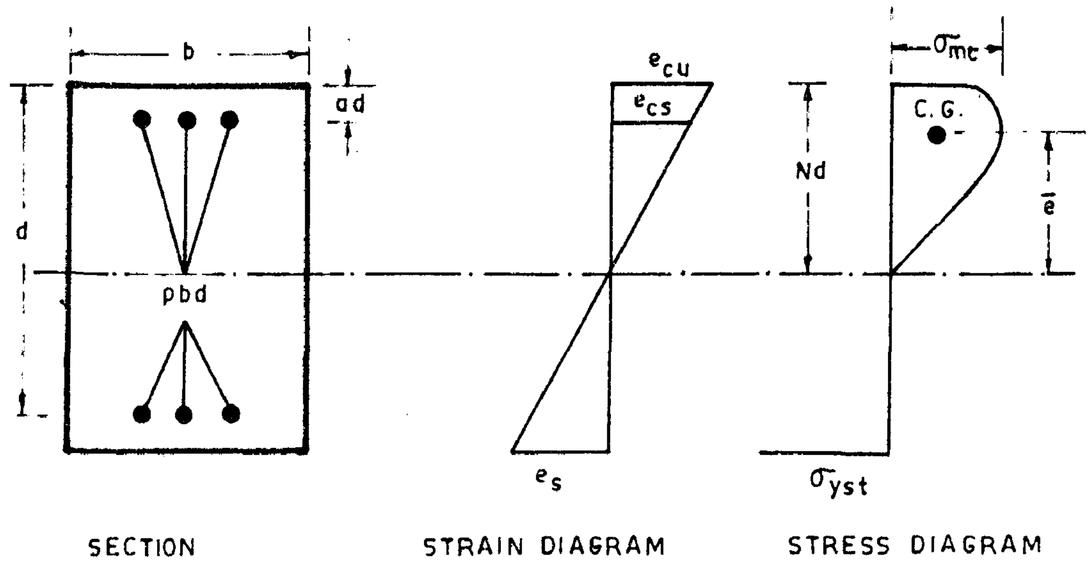


FIG. 4.1

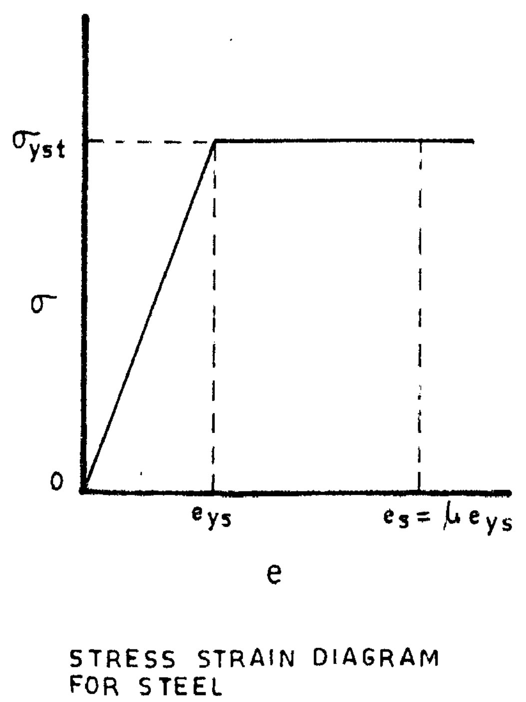


FIG. 4.2

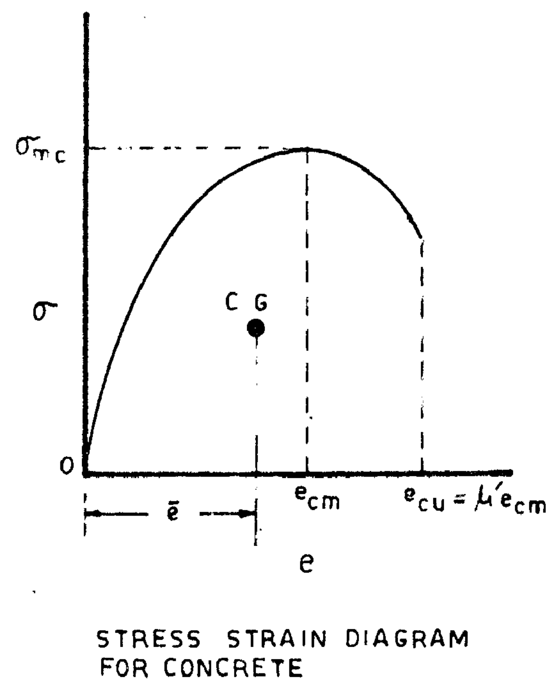


FIG. 4.3

where,

σ_{mc} = maximum stress in concrete

e_{cm} = strain corrosponding to maximum stress

σ_c = stress in concrete

e_c = strain in concrete at σ_c, stress level

Fig. 4.1 shows the section and stress - strain variation in the section. Strain in compression steel is given by ,

$$e_{cs} = e_c \cdot \frac{N - a}{N} \quad \ldots\ldots (4.2)$$

and

$$e_c = e_s \frac{N}{1 - N} \quad \ldots\ldots (4.3)$$

stress in compression steel, $\sigma_{cs} = E_s \cdot e_{cs}$

or

$$\sigma_{cs} = E_s \cdot e_c \cdot \frac{N - a}{N} \quad \ldots\ldots (4.4)$$

From fig. 4.2, $e_s = \mu_s \cdot e_{ys}$

where μ_s is ductility in steel and e_{ys} is the yield strain of tensionsteel, also, $e_c = \mu_c \cdot e_{cm}$

From equation 4.3 we have,

$$\mu_c \cdot e_{cm} = \mu_s \, e_{ys} \frac{N}{(1 - N)}$$

or

$$\mu_s = \mu_c \frac{e_{cm}}{e_{ys}} \cdot \frac{(1 - N)}{N} \quad \ldots\ldots (4.5)$$

also from equation (4.4),

$$\sigma_{cs} = \mu_c E_s . e_{cm} \left(\frac{N - a}{N}\right) \quad \ldots\ldots (4.6)$$

The C.G. of the stress diagram of compression concrete from the neutral axis can be worked out as follows

$$\bar{\sigma} = \frac{\int_0^{Nd} e.\ \sigma.\ de}{\int_0^{Nd} \sigma\ de} \qquad \ldots \qquad (4.7)$$

Substituting the value of σ from equation (4.1) in (4.6) we get,

$$\bar{e} = \frac{\int_0^{Nd} e\left(2\frac{\sigma_{mc}}{e_{cm}}.e - \frac{\sigma_{cm}}{e_{cm}^2}.e^2\right) de}{\int_0^{Nd}\left(2\frac{\sigma_{mc}}{e_{cm}}.e - \frac{\sigma_{cm}}{e_{cm}^2}.e^2\right) de}$$

Solving we get,

$$\bar{e} = N.d.\ \frac{(2/3 - 1/4.\mu_c)}{(1 - 1/3\ \mu_c)}$$

or $$\bar{e} = N.d.\ K \qquad \ldots \qquad (4.8)$$

where $$K = \frac{(2/3 - 1/4\ \mu_c)}{(1-1/3\ \mu_c)}$$

The force of compression in concrete F_c, can be obtained as

$$F_c = \int_0^{Nd} b\ \sigma.\ de \qquad \ldots \qquad (4.9)$$

or $$F_c = bNd.\ \mu_c.\ \sigma_{mc}.\ \left(1 - \frac{\mu_c}{3}\right) \qquad \ldots \qquad (4.10)$$

Equating the force of compression to force of tension we get,

$$p.bd\ \sigma_{yst} = p\ bd.\ \sigma_{sc} + b\ Nd\ \ \mu_c.\sigma_{mc}\ (1 - \frac{\mu_c}{3})$$

$$\ldots\ldots \qquad (4.11)$$

$$\text{or, } \sigma_{sc} = \sigma_{yst} - N.\mu_c \frac{\sigma_{mc}}{p} (1 - \frac{\mu_c}{3}) \quad \ldots \qquad (4.12)$$

Substitute the value of σ_{sc}, from equation (4.6) in (4.12) we get,

$$\mu_c.E_s.e_{cm} (\frac{N-a}{N}) = \sigma_{yst} - \frac{N.\mu_c.\sigma_{mc}}{p} (1 - \frac{\mu_c}{3}) \quad ..(4.13)$$

Taking moment of the force in tension steel and compression steel about the C.G. of the force of compression in concrete, we get the moment of resistance of the section as,

$$M_r = pbd^2\ \sigma_{yst}\ (1-N+N.K) + F_{cs}.d\ (N - a - N.K) \quad \ldots \quad (4.14)$$

The values of E_s, e_{cm}, a, σ_{yst}, σ_{mc} and p, are known in an experiment. The equation (4.13) contains unknowns μ_c and N. Thus the position of neutral aixs for any ductility in concrete (μ_c) can be calculated from equation (4.13).

Knowing the value of N, the value of ductility in steel (μ_s) can be worked out from equation (4.5). The value of moment of resistance of the section in the inelastic range and for varying values of ductility in concrete (μ_c) and steel (μ_s) can be calculated from equation (4.14). The moment of resistance of the section in the transition phase of tension steel from elastic to inelastic can be done as in the case of anlysis given for reinforced brick column.

4.2 CALCULATION OF DFLEXURAL RIGIDITY OF THE SECTION : - For the purpose of calculations of deflections, E I value at each stage is required. As the stress - strain curve for concrete is assumed to be parabalic, so at each stress level, the value of E_c will be changing. Thus to calculate contribution of concrete section, taking into account the variation of E_c, towards E_cI, value of the section following procedure is to be adopted.

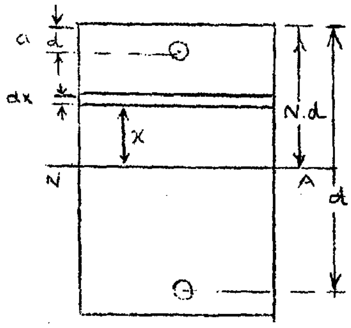


Fig.4.4

The parabolic equation of stress-strain curve for concrete is from equation No. (4.1) as,

$$E_c = 2\left(\frac{\sigma_{mc}}{e_{cm}}\right) - \left(\frac{\sigma_{mc}}{e_{cm}^2}\right) . e_c \qquad \ldots \qquad (4.15)$$

Equation (4.15) contains strain in concrete (e_c) as the only variable e_c depends on the position of the fibre under consideration. Reffering figure No. 4.4, If strain at $(1 - N)d + r \cos\theta$ is yield strain then strain in concrete at distance x, will be given by,

$$e_{cx} = \frac{e_{ys}}{(1-N)d + r \cos\theta} . x \qquad \ldots . \qquad (4.16)$$

Substituion of the value of e_c in equation (4.15) yields,

$$E_{c(x)} = 2\left(\frac{\sigma_{mc}}{e_{cm}}\right) - \left(\frac{\sigma_{mc}}{e_{cm}^2}\right) \frac{e_{ys}}{(1-N)d+r\,\mathrm{Cos}\theta} \cdot x \quad \ldots \quad (4.17)$$

Let dx be the thickness of concrete fibre at distance x from the Neutral axis then contribution of compression concrete is,

$$E_c \cdot I = \int_0^{Nd} b.dx\, x^2 \left[2\left(\frac{\sigma_{mc}}{e_{cm}}\right) - \left(\frac{\sigma_{mc}}{e_{cm}^2}\right) \frac{e_{ys}}{[(1-N)d+r\mathrm{Cos}\theta]} \right] \quad \ldots \quad (4.18)$$

In the above expression θ will change as the tension steel yields from elastic stage to inelastic stage. Contribution of compression steel can be calculated directly as $A_{cs}E_s$.

The yielding of tension steel can be taken into account in calculation of $E_s \cdot A_{ts}$ as described earlier in the case of R.B. pier calculations. When ductility in tension steel is grater then 1, its contribution towards $E_s \cdot I$ value is zero. Contribution of concrete and compression steel can however be calculated as described earlier.

4.3 CALCULATION OF $E_c \cdot A$:- For calculation of shear deflections we require the value $E_c \cdot A$ where A is the effective area of the section. For concrete, again E_c will change with distance from neutral axis. This can be taken into account as described below:

$$E_c . A_\theta = \int_0^{Nd} b.dx.\ E_{c(x)} \quad \ldots \quad (4.19)$$

From equation (4.17) and (4.18) we get,

$$E_c\ A_\theta = \int_0^{Nd} b. \left[2\left(\frac{\sigma_{mc}}{e_c} \right) - \left(\frac{\sigma_{mc}}{e_{cm}^2} \right) \frac{e_{ys}}{[(1-N)d + r\ Cos\theta]} \right] dx \quad \ldots\ldots \quad (4.20)$$

From equation (4.20) we can calculate $E_c . A_\theta$ taking into account variation of E_c.

Area of tension steel reduces as it yields. This can be calculated as given earlier. $E_s . A_{cs}$ also can be calculated easily. Thus shear deflections at each stage can be calculated.

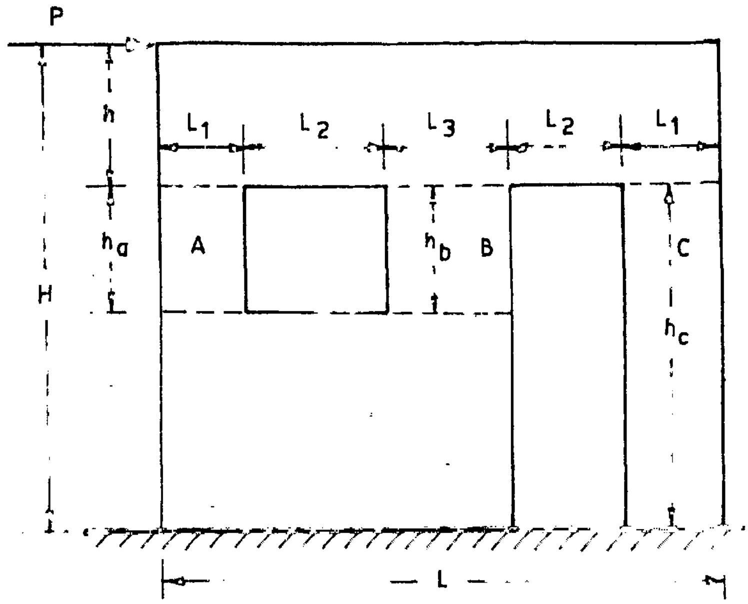
Using the above approach, the effect of variation of percentage of steel and cover for steel on the lateral load - deflection characteristics of reinforced concrete columns can be studied similar to reinforced brick sections as described in chapter III. The use of the expressions developed is illustrated in chapter VI along with the results of experimental studies.

CHAPTER V

ULTIMATE LOAD CARRYING CAPACITY OF SHEAR WALLS

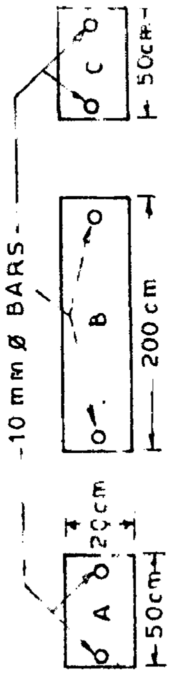
5.1 GENERAL :- Preceeding chapters describe method of computation of ultimate lateral load carrying capacity and deflections of reinforced brick and reinforced concrete piers taking into account the ductility of steel. However such piers only form a part of a shear wall which is the main lateral load carrying member of a building. This approach is extended, in this chapter, for estimating the lateral load carrying capacity of a plane shear wall. This is illustrated by an example presented in the subsequent sections.

5.2 THEORETICAL MODEL AND FORMULATION OF PROBLEM :- The model selected for the present study is shown in the figure 5.1 The wall contains two openings (a) window (b) Door, which generally occure in the practice. The openings divide the wall into a series of piers, which for the present analysis is considered to be acted by lateral load which may be due to wind or earthquake. The wall is divided into three piers viz. A, B and C, each of different stiffnesses. The lateral load applied on the top of the shear wall is shared by these piers in the proportion of their stiffness. The load shared by each pier multiplied by half the height of column gives moment applied on the section. In each pier, plastic hinges will be formed, when the load shared by them is such as to cause a plastic bending moment.

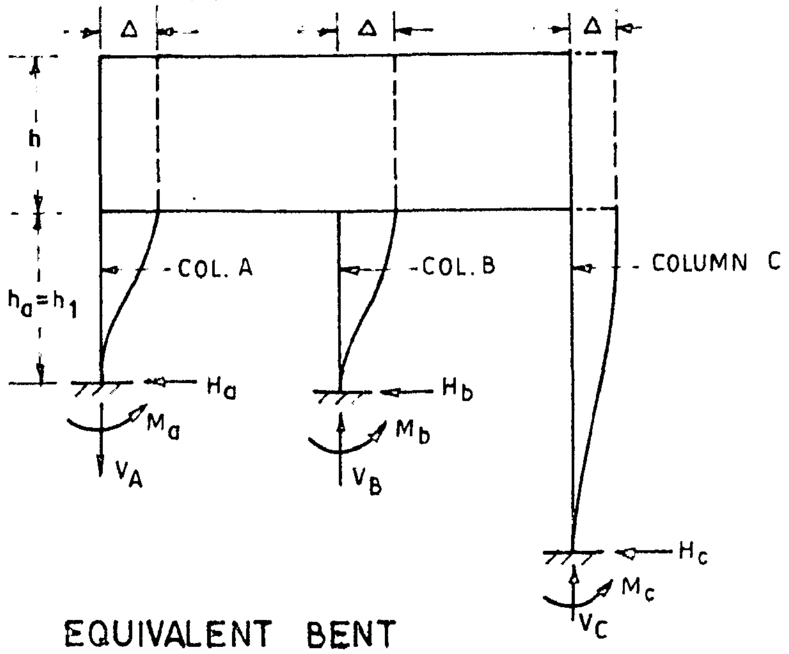


WALL DIMENSIONS

FIG. 5.1

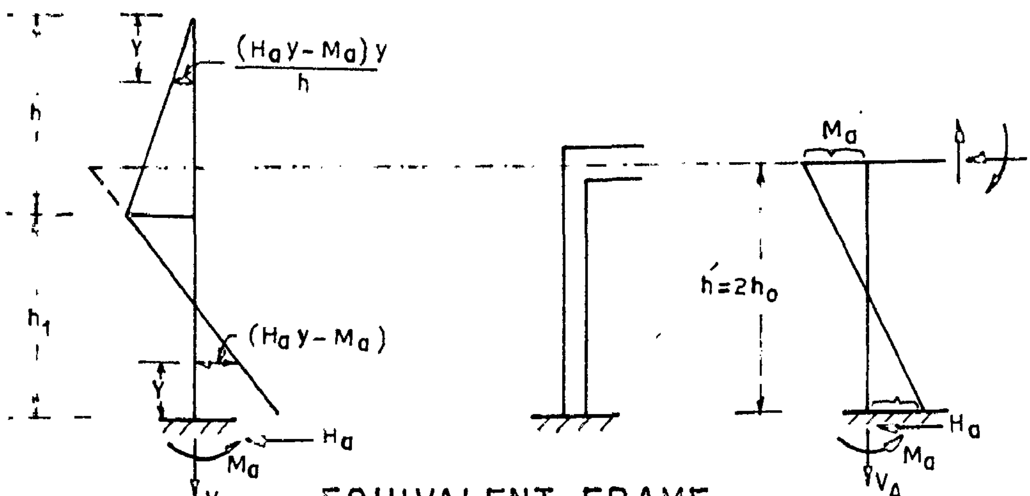


REINFORCEMENT IN PIER



EQUIVALENT BENT

FIG. 5.3



EQUIVALENT FRAME

FIG. 5.4

The problem attempted in this chapter deals with finding out the ultimate load carrying capacity of the shear wall, when plastic hinges have formed in two piers and these are deflecting in the inelastic range and the third pier has just attained yield stress in the tension steel. The load factor, obtained by dividing the ultimate load carrying capacity of the shear wall by the load which causes just yield of first pier is found for the shear wall considered here. This gives an Index of extra load which the wall can take through inelastic deformations of the wall. Lal (7) has shown that the performance of the long shear walls is similar to that of bent and the effect of over turning moments and superimposed load is negligiable and behaviour of the wall is of shear type only. For such type shear walls following analysis of ultimate load carrying capacity will be useful in ascertaining the extra load that the wall can take when plastic winges are considered to have formed in the piers.

5.3 ANALYSIS : - The method for the analysis of these piers is similar to the analysis of bents. The method is described below,

THE BENT METHOD : The piers formed are assumed to be tied together by the upper and lower portions of the wall. The portion above and below pier is assumed to be rigid. The lateral force is carried to bottom by shear and moments in the pier (Fig. 5.2). The spandrel of the equivalent bent has much greater flexural rigidity as compared to piers.

For the analysis of a bent, it is convienient to make use of an equivalent frame concept in which the bending moment diagram for the bent is modified as shown by the dotted line in Fig. 5.3. This modification is made such that the shearing force produces the same strain energy for the columns of substitute frame as that of the origanal bent. The columns have uniform cross section along their heights. The strain energy due to flexure in bent colum is given by,

$$U = \int_0^{h_1} \frac{(H_a y - M_a)^2}{2EI} . dy + \int_0^{h} \frac{(H_a . h_1 - M_a)^2}{2EI} . \left(\frac{y}{h}\right)^2 . dy \quad .... \quad (5.1)$$

Case 1 : If the slope at the base of the pier is zero,

$$\theta_a = \frac{\delta u}{\delta M_a} = 0$$

or

$$\theta_a = \int_0^{h_1} \frac{(H_a . y - M_a)\ dy}{2EI} + \int_0^{h} \frac{(H_a h_1 - M_a)\left(\frac{y}{h}\right)^2}{2\ EI}\ dy$$

As, EI, is constant throughout ,

$$\left(- \frac{H_a . h_1^2}{2} \quad M_a\ h_1\right) - \frac{H_a . h_1 - M_a}{h^2} . \frac{h^3}{3} = 0$$

$$\text{or, } M_a \left(h_1 + \frac{h}{3}\right) = \frac{H_a . h_1^2}{2} + \frac{H_a . h_1 . h}{3}$$

and hence,

$$M_a = \left(\frac{H_a \cdot h_1}{2} \right) \frac{(h_1 + \frac{2}{3} h)}{(h_1 + \frac{h}{3})}$$

$$= \left(\frac{H_a \cdot h_1}{2} \right) \frac{(1 + \frac{2}{3} \cdot \frac{h}{h_1})}{(1 + \frac{1}{3} \frac{h}{h_1})}$$

Also $M_a = \dfrac{H_a \cdot h_1'}{2}$, Therefore for the corresponding column in continuous frame, the height h_1' is equal to

$$h_1' = 2h_o = h_1 \frac{(1 + \frac{2}{3} \frac{h}{h_1})}{(1 + \frac{1}{3} \frac{h}{h_1})}$$

But,

$$\frac{(1 + \frac{2}{3} \frac{h}{h_1})}{(1 + \frac{1}{3} \frac{h}{h_1})} = 1 + \frac{1}{3} \frac{h}{h_1} - \frac{1}{9} \left(\frac{h}{h_1} \right)^2 + \ldots = \left(1 + \frac{h}{h_1}\right)^{\frac{1}{3}}$$

or $$h_1' = h_1 \left(1 + \frac{h}{h_1} \right)^{\frac{1}{3}} \qquad \ldots \qquad \ldots \quad (5.2)$$

Case II : If the column at base is not restrained i.e.

$$M_a = 0$$

$$\int_0^{h_1} \frac{(H \cdot y)^2 \cdot dy}{2\,EI} + \int_0^{h} \frac{\left(\frac{H\,h_1}{h} \cdot y \right)^2}{2\,EI} dy$$

$$= \int_0^{h_1'} \frac{(H \cdot y)^2}{2EI} dy \qquad \ldots\ldots\ldots \quad (5.3)$$

Solution of equation (5.3) gives

$$h_1'^3 = h_1^3 \left(1 + \frac{h}{h_1} \right)$$

$$\text{or} \quad h_1' = h_1 \left(1 + \frac{h}{h_1}\right)^{\frac{1}{3}} \quad \ldots \quad \ldots \quad (5.4)$$

Thus it is seen that conditions of restraints of column at base do not effect the equivalent height **h'** of the column. From this, horizontal reactions and moments in the columns of the bent can be obtained from an equivalent continuous frame for any degree of restraint at base.

For equilibrium, the following condition must be satisfied.

$$P = H_a + H_b + H_c \quad \ldots\ldots \quad (5.5)$$

Now If a horizontal force is applied at top of spandrel, then all piers will move by the same amount unless any crack an failure occures in the system. Till all the piers are in the elastic range, the distribution of force in each pier will be proportional to their stiffness against deflection. When first pier yields and reaches in the inelastic range, the further applied load is shared by rest of the piers in the proportions of their stiffnesses. The yielded column does not take any extra force and it just deflects as plastic hinges are formed in the pier at top and bottom. The load on the shear wall can be increased till all piers reaches upto the yield moment and plastic hinges are formed in them. The deflection due to bending and shear is given by,

$$\Delta = P \left(\frac{h'^3}{12EI} + 1.2 \frac{h'}{GA} \right) \quad .... \quad (5.6$$

when $G = 0.5\ E$

$$\Delta = \frac{P}{12E} \left(\frac{h'^3}{I} + 28.8 \frac{h'}{A} \right) \quad ... \quad (5.7)$$

in which,

Δ = deflection at top of piers

E = Modulus of elasticity

G = Modulus of rigidity

I = Moment of inertia

A = Cross - Sectional area of pier

h' = Equivalent height of piers

Now the part of the load shared by each pier can be calculated as,

$$H_1 = \frac{\frac{1}{\Delta_1}}{\left(\frac{1}{\Delta_1}\right)} \cdot P \quad .. \quad (5.8)$$

From this equation H_a, H_b, H_c ......etc. in each pier can be calculated. Then pier having shear H_a will produce a moment,

$$M_a = H_a \cdot \frac{h_a}{2} \qquad \ldots\ldots (5.9)$$

If the plastic moments of resistances of piers A, B and C are M_{PA}, M_{PB}; and M_{PC} respectively and heights of these piers are h_a, h_b and h_c then, loads required to cause these moments in each piers separately will be,

$$P_A = \frac{2\,M_{PA}}{h_a}$$

$$P_B = \frac{2M_{PB}}{h_b} \qquad \ldots\ldots (5.10)$$

$$P_C = \frac{2M_{PC}}{h_c}$$

Then ultimate load carrying capacity of the shear wall when all piers have reached to yield stage is given by,

$$P_u = P_A + P_B + P_C = \left(\frac{2M_{PA}}{h_a} + \frac{2M_{PB}}{h_b} + \frac{2M_{PC}}{h_c} \right) \ldots (5.11)$$

Now If P is the total load applied at the top of the shear wall and if it is shared by piers in the proportion of stiffnesses say K_a, K_b and K_c then there can be three cases

Case 1 : Let initially P_1 is the load applied then,

If $P_1 K_a = P_A$

or $$P_1 = \frac{P_A}{K_a} \qquad \ldots\ldots (512)$$

equation (5.12) indicates that If P_1, is the total load applied at the top of the shear wall then pier A, will share a load of P_A, which will cause pier A, to yield. At this stage Pier B, and C, are, say, in the elastic range and $(P_1 - P_A)$ load is shared by rest of the piers.

Case 2 : - Now If load is increased to say, P_2 then P_A will be the load taken by pier A, as it has already yielded and $(P_2 - P_A)$ load will be distributed in the proportion of stiffness of rest of the piers and If,

$(P_2 - P_A) = \frac{P_B}{K_b}$ - (5.13) then to cause yield of Pier B, applied load required is given by,

$$P_2 = P_A + \frac{P_B}{K_b} \quad \ldots\ldots \quad (5.14)$$

Now at load P_2 piers A, and B, have yielded and A, is in the inelastic range and B, has just yielded and C, is in the elastic state. Additional load required to make this pier also yield is equal to P_C.

Thus we can calculate the total applied loads at which yielding of pier A, pier B and Pier C, will take place. The ratio of load required to cause yielding of all piers to the load required to cause yielding of first pier is defined as load factor. This load factor estimates the extra load carried by the shear wall due to energy absorption by inelastic deformations.

5.4 EXAMPLE - PLAIN REINFORCED SHEAR WALL : - To have an idea of load factor a plain reinforced brick shear wall fixed at bottom is considered. the shear wall adopted for analysis is shown in the figure 5.1.

IS 4326 - 1967(11) recommends 12 mm. diameter bars for $1\frac{1}{2}$ brick thick walls for single storey buildings and for any other thickness of the wall, the area of the bar is to be increased or decreased accordingly. In present case, since walls are 20 cms. thick, 10 mm diameter bars are provided on each face of piers as shown in Fig. 5.2

5.4.1 CALCULATION OF MOMENT OF INERTIA : For reinforced shear walls,

$$I_A = I_a + \frac{1}{2} m. A_s. (L_1 - 2a)^2 \qquad \ldots. \qquad (5.15)$$

$$I_B = I_b + \frac{1}{2} m. A_s (L_2 - 2a)^2 \qquad \ldots \qquad (5.16)$$

$$I_C = I_c + \frac{1}{2} m. A_s (L_1 - 2a)^2 \qquad \ldots. \qquad (5.17)$$

$$\text{Area} = A_a + 2m. A_s \quad \text{etc.} \qquad \ldots. \qquad (5.18)$$

where,

A_a = Area of unreinforced pier

A_s = Area of reinforcing bar

In present case,

$A_S = 0.785 \times 10^{-4} \quad m^2$

$L_1 = 0.5$ m

$L_2 = 1.0$ m

$L_3 = 2.0$ m

$a = 0.025$ m

$m = 125$

Thickness of wall = 0.2 m

On calculation we have,

$$I_A = I_C = 0.003075 \; m^4 \quad \ldots \quad (5.19)$$

and $$I_B = 0.1517 \; m^4 \quad \ldots \quad (5.20)$$

Also,

$$A_a = A_c = 0.1196 \; m^2 \quad \ldots \quad (5.21)$$

and $$A_b = 0.4196 \; m^2 \quad \ldots \quad (5.22)$$

5.4.2 CALCULATION OF EQUIVALENT HEIGHT OF PIERS.

1) PIER A, and B,

$h = 1.4$ m

$h_1 = 1.2$ m

Therefore $$h'_A = h'_B = 1.2 \left(1 + \frac{1.4}{1.2}\right)^{\frac{1}{3}} = 1.56 \text{ m} \quad ..(5.23)$$

2) PIER C

$h = 1.4$ m

$h_1 = 2.1$ m

therefore $h_c' = 2.1 \left(1 + \frac{1.4}{2.1}\right)^{\frac{1}{3}} = 2.49$ m .. (5.24)

5.4.3. CALCULATION OF STIFFNESS

1. Pier A, and C,

Equation (5.7) gives bending and shear deflection of the shear well,

From this deflection of pier A and C is given by,

$$\Delta_A = \frac{P}{12\,E}\left(\frac{1.56^3}{0.003075} + 28.8 \times \frac{1.56}{0.1196}\right)$$

or $\Delta_A = \frac{P}{12\,E} \cdot 1616$

therefore $\frac{P}{\Delta_A} = K_A = \frac{12\,E}{1616} = 12E \times 0.00062$ ..(5.25)

similarly

$K_B = 12E \times 0.00752$ ... (5.26)

$K_C = 12E \times 0.000176$ ... (5.27)

$\Sigma K = K_A + K_B + K_C = 12E \times 0.008316$ ... (5.28)

if P is the total load applied then from equation (5.8), If load shared by pier A, B and C be P_1, P_2 and P_3, we have

$P_1 = 0.074 \times P$ ... (5.29)

$P_2 = 0.906 \times P$ ... (5.30)

and $P_3 = 0.02 \times P$ ... (5.31)

5.4.4 CALCULATION OF MOMENT OF RESISTANCE OF THE PIER SECTIONS AT YIELD POINT

PIER A, and C,

Percentage of steel = 0.0785

Cover = 2.5 cms

m = 125

The nateral axis coefficient 'N' is calculated to be 0.299 and lever arm coefficient 'J' = 0.916. If yield stress of tension steel = 2600 kg/cm² then,

The moment of resistance of the section when θ = 0 is calculated from equation 3.18 and is as given below,

$$M_{r_{\theta = 0}} = 89,000 \text{ Kg. cms.}$$

Load required to cause this moment in pier A is given by,

$$P_1 = \frac{2M}{L} = \frac{2 \times 89,000}{120} = 1483.33 \text{ Kg.} \quad .. \quad (5.32)$$

similarly for pier C

$$P_3 = \frac{2 \times 89000}{210} = 850 \text{ Kg} \quad ... \quad (5.33)$$

Similarly moment of resistance of the section when pier B reaches yielding is

M = 3,84,000 kg. cms and load required to be applied is

$$P_2 = 6,400 \text{ Kg.} \quad .... \quad (5.34)$$

5.4.5 CALCULATION OF TOTAL LOAD REQUIRED TO CAUSE HINGE FORMATION IN ALL PIERS

Now we know that applied load P_1 is shared by each piers as given in equations (5.29) to (5.30). There can be three cases.

1) Total load required to cause yielding of pier A alone when all other piers are in elastic state is calculated as below,

$$P \times 0.074 = 1483.33$$

$$\text{or } P = 20{,}000 \text{ Kg} \qquad \ldots \qquad (5.35)$$

2) Similarly total load required to cause yielding of pier B when all other piers are in elastic state is given by,

$$P = 7060 \text{ Kg} \qquad \ldots \qquad (5.36)$$

3) Also total load required to cause yield of pier C when all other piers are in elastic state is,

$$P = 42{,}500 \text{ kg} \qquad \ldots \qquad (5.37)$$

From equations (5.35) to (5.37) it is observed that as applied load is increased, first of all, the strongest pier B will yield, as the load shared by it is the maximum. Next yielding will take place in pier A, and lastly in pier C. But pier B, when it has yielded will not take any additional load and increased load will be shared by piers A and C alone in proportion of their stiffnesses.

Thus when a load of 7060 kg is applied a load of 6,400 kg is taken by pier B and if yields, at this stage pier A and C are in the elastic stage.

Now load will be shared by piers A and C in proportions of their stiffnesses viz.

$$K_A : K_C \quad :: \quad 0.79 : 0.21$$

Now let total load required to cause yield of pier A is P then for yield of pier A

$$P \times 0.79 = 1483.33$$

or $\quad P = 1880$ kg.

Thus total load required to cause hinge formation in pier B and A is given by

$$P_{AB} = 6,400 + 1,880 = 8,280 \text{ kg}$$

At this load pier A and B reaches yield limit and pier C is in elastic state. For hinge formation in this pier also, additional load of 850 kg is to be applied. Thus ultimate load carrying capacity of the shear wall when plastic hinges are formed in all piers is,

$$P_{ABC} = 1483.33 + 6,400 + 850 = 8,733.33 \text{ kg.} \quad (5.38)$$

From equation (5.37) applied load for first yielding of the shear wall is calculated. From equations (5.37) and (5.38) the load factor for the case considered here is given by.

$$\text{Load factor} = \frac{8733.33}{7060} = 1.236 \quad \ldots\ldots (5.39)$$

Thus there is an increase of 23.0% (approximately) in the load carrying capacity of the shear wall.

5.3.6 CALCULATION OF DEFLECTION : Since all piers will deflect to the same amount, the deflection of pier C will also give the deflection of the shear wall as a whole.

Load shared by pier C, when pier B, yield = 0.02 x 7060

= 141.2 Kg. and

Load shared by pier C, when it just yield = 850 kg.

Deflection of pier C in first case from equation (5.7) in the elastic range = 0.0398 cms and similarly

Deflection of pier C when it has just yielded = 0.238 cms.

$$\text{The ratio of these deflections} = \frac{0.238}{0.0398} = 5.98$$

Thus the diflection of the shear wall when last pier yields is 5.98 times the deflection of the shear wall when only first pier yields. Thus extra 23 % load is taken by the shear wall by inelastic absorption of energy due to deflection.

CHAPTER VI

EXPERIMENTAL STUDY

6.1 GENERAL : The experiments were performed on the reinforced brick and reinforced concrete columns under the action of action of lateral loads and the results of these investigations are reported in this chapter. The results comprise of

1. Load vs. deflection curves
2. Strain variation in the section in the elastic and inelastic range.

6.2 DESCRIPTION OF MODELS : - The reinforced brick columns (8 cms x 16 cms) and height 75.5 cms. with 12 mm/ 6 mms diameter mild steel reinforcing bars one each in tension and compression at a cover of 0.15d were made. Using the special small bricks of size 3" x 1.5" x 1" and the mortar used had proportions of cement sand as 1:3 by weight. To fix the calumn at base, a base plate of 30 cms x 30 cms x 1.2 cms was used. The reinforcing bars were welded to this plate and to make a bond between plate and brick few steel hooks were also welded to plate in a staggered fashion. At top of the reinforcing bars a steel plate 8 cms x 16 cms and 1.0 cms thick was welded. This steel plate had two 12 mm Φ threaded holes so as to enable to fix steel beam made of two channels welded together on the top of two columns with the help of long bolts and also for making arrangement for application of load. At base a cut was left in column to expose the tension reinforcement to fix the strain gage. The companion specimens were also prepared while each column was made to obtain the basic properties of the mortar used.

Two sets of concrete columns (8 cms x 16 cms) were made in M 150 grade giving cement, sand and aggregate proportion as 1:2:4 using 12 mm and 6 mm diameter mild steel reinforcing bars at a cover of 0.15d. To fix the column at base again 30 cm x 30 cm x 1.2 cms base plates were used and reinforcing bars were welded to this plate. To make a bond between plate and brick few steel hooks were also welded to plate in a staggered fashion. At top of the reinforcing bars a steel plate 8 cms x 16 cms and 1.0 cms. thick was welded. This plate had two 12 mm. dia. threaded holes, so that steel plate of size 11 cm x18 cm and 0.5 cms thick cn be fixed over it, with the help of two 12 mm Φ bolts. This plate had a hole of 12 mm dia. to fix the proving ring. The water-cement ratio was kept constant and same mason was exployed to construct all models in order to minimise the variation of work-manship. The models were cured for 28 days. After curing , two strain gages type CA10,(gage factor 2.08)

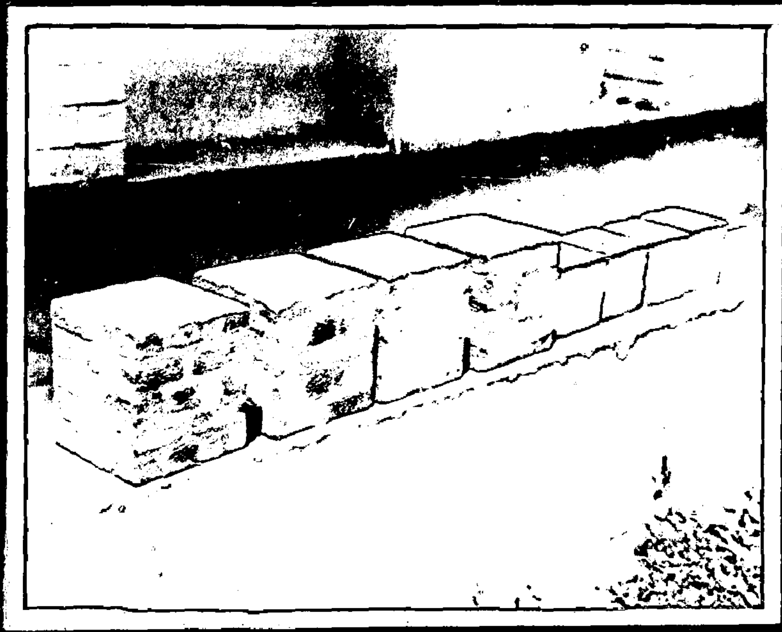


FIG. 6.1

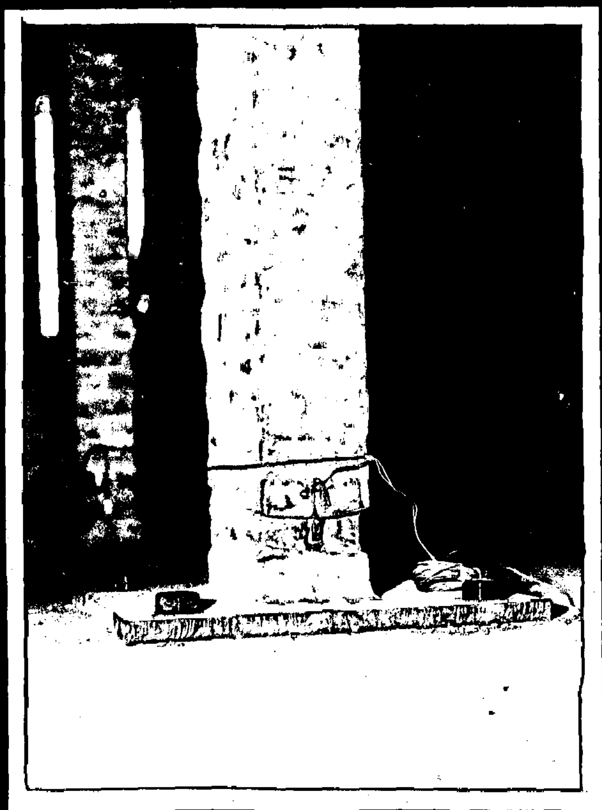


FIG. 6.2

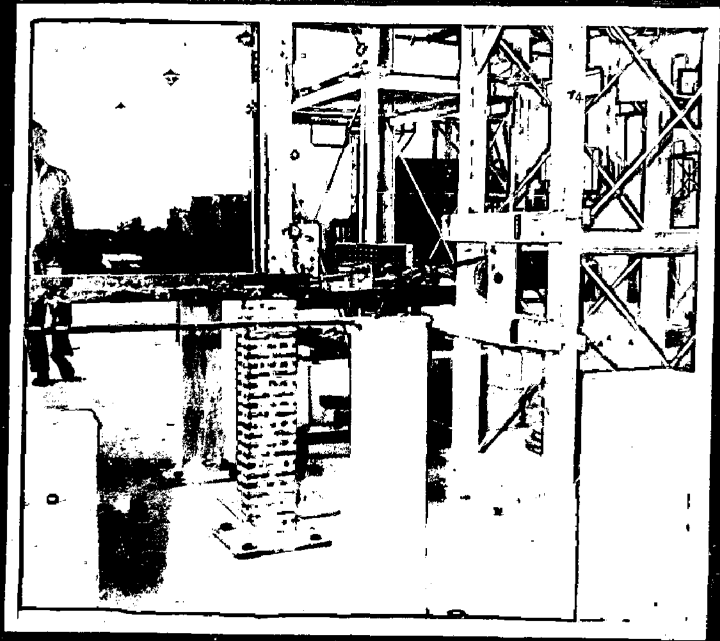


FIG. 6.3

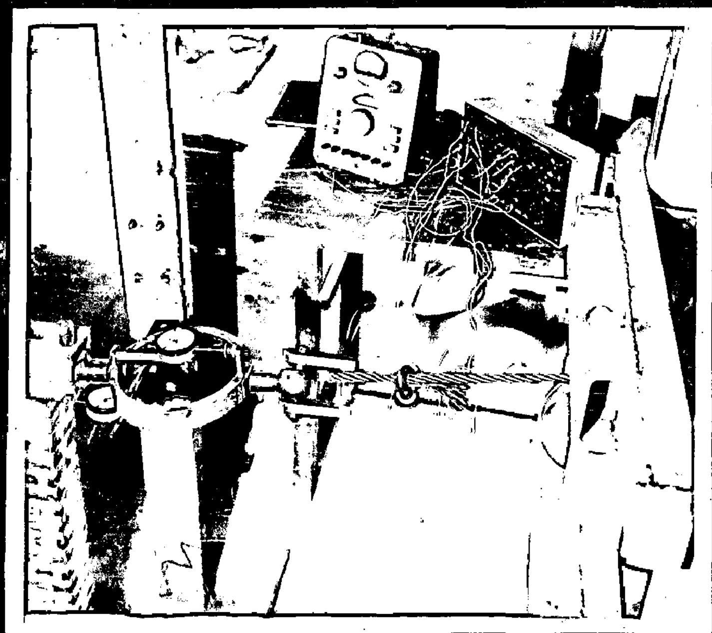


FIG. 6.4

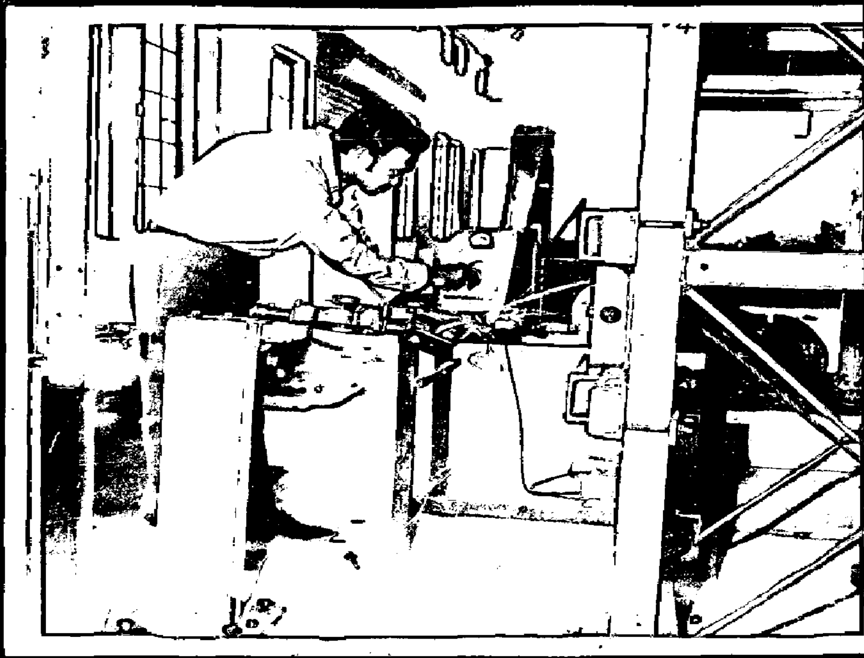


FIG. 6.5

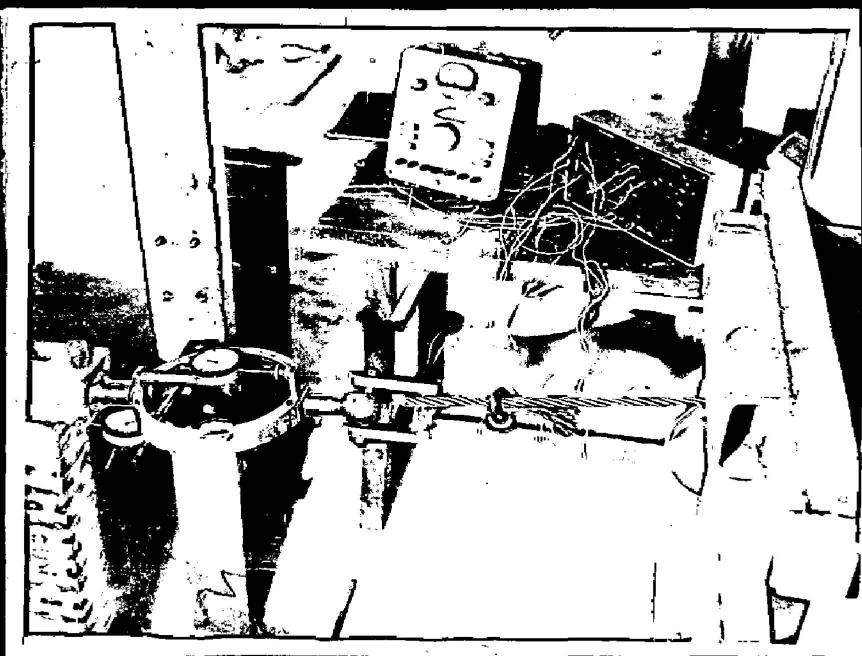


FIG. 6.6

and type L - 10, (gage factor 2.75) were used. Two strain gages were fixed on one column i.e. one on tension steel and other on concrete or brick in compression. The strain gages fixed on the tension steel and compression brick are shown in Fig. 6.2. companion specimens of brick and concrete were also made to find the basic properties of the material used. These are shown in the Fig. 6.1. To test the reinforced brick pier fixed at top, R.B. piers were connected rigidly at top with a steel beam made of two channels welded face to face.

6.3 TESTING APPARATUS : - For testing of columns the load was applied with the help of chain pulley block system shown in Fig 6.3 and load was measured with the help of a proving ring of 2 Ton. capacity. The proving ring arrangement is shown in Fig. 6.6.

A long travel dial gage was fixed on a reference frame with the help of magnetic base as shown in Fig. 6.6 to measure the deflection at the top of the column caused due to the applied load. The dial gage had a least count of 0.01 mms.

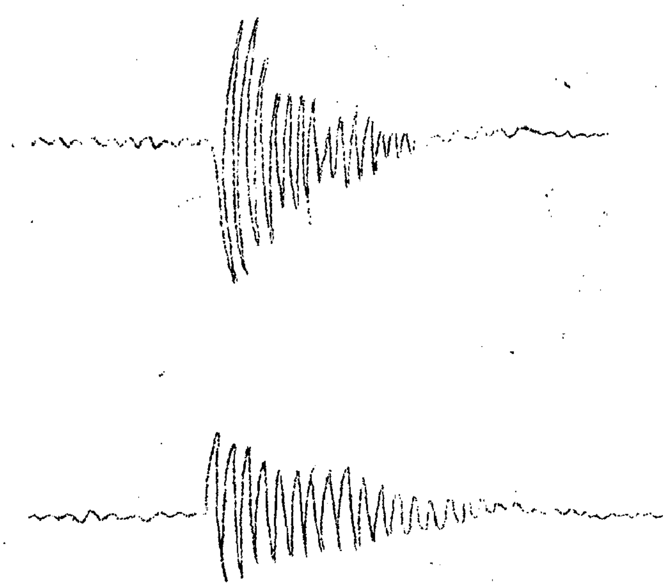
Strains in concrete/brick and tensile steel were measured with the help of a strain Indicator through a multichannel switch. The arrangement is shown in Figs. 6.4 and 6.5.

Cover for reinforcement = 2.4 cms

Height of cantilever = 75.5 cms.

Density of material (brick work) = 1920 kg/m^3

Fig. 6.7 shows the frequency records taken for R.B. cantilever column with 12 mm and 6 mm diameter reinforcements. Flexural rigidity is calculated as given below,



Freqency Records

Fig. 6.7

1. BRICK PIER WITH 12 MM DIA.REINFORCEMENT

Frequency obtained from free vibration test = 68.75 cps

Value of EI from equation(6.1) = 124×10^6 $kg.cm^2$

6.4 DETERMINATION OF FLEXURAL RIGIDITY : - Frequency of a freely vibrating uniform cantilever is given by ,

$$f_b = 0.56 \sqrt{\frac{E_b \; I.g}{\sigma \; AL}} \quad \ldots \quad (6.1)$$

where

f_b = frequency in cycle per second.

I = Equivalent moment of Inertia of the column section.

g = Acceleration due to gravity

σ = weight density

A = Equivalent area of the section

L = Height of cantilever column

The column section were 8 cm x 16 cms in section and height was 75.5 cms. Leads from the strain gage fixed on the tension steel was connected to the brush amplifier. Connection from the amplifier were taken to a pen recorder.

Free vibrations were created by hand tapping and records were taken on the pen recorder. From these records the frequency of vibration of the column was calculated. Knowing the properties of the contilever column we can calculate the flexeral rigidity (EI) of the column from equation (6.1). The properties of cantilever column are as follow :

Dimension of the section (B x D) = 8 cms x 16 cms

2. 6 MM. DIA. REINFORCED BRICK COLUMN

Frequency obtained from free vibration test = 50 cps

Value of EI from equation(6.1) = 65.5×10^{6} kg. cm^2

6.5 BASIC PROPERTIES OF MATERIALS USED : The companion specimens of brick (1:3) and M 150 grade concrete used in construction of models of columns were tested for finding the compressive stresses. The average values of the properties exhibited are listed below.

BRICKWORK IN 1:3

σ_b = 61 kg/cm^2

M150 GRADE CONCRETE

σ_{mc} = 170 kg/cm^2

e_{cms} = 0.3 percent

To find the yield stress of reinforcing steel used, tensile strength test was carried out on samples of reinforcing steel used in 10 Ton universal testing machine. These exhibited following properties

6 MM. DIA. BAR

Yield stress = 6,200 kg/cm^2

12 MM. DIA. BAR

Yield stress = 5,000 kg/cm^2

6.6 PRESENTATION AND DISCUSSION OF RESULTS : - The theoretical results for models were computed first and checked with experimental observations. Fig. 6.8 to 6.17 presents theoretical and experimental load - deflection characteristics of the reinforced brick and reinforced concrete piers in graphical form and strain variation in the section under the action of increasing lateral loads. A reasonably good comparison of theoretical and experimental results can be seen in the above figures. However it is observed that all experimental curves initially lie above the theoretical curve but when load level is increased these lie below the theoretical curves and the ultimate load carrying capacity of the experimental model is also less than theoretical value. Also, at initial stages of loading the neutral axis is found to lie below the theoretically calculated value and shifts upwards with increase in lateral load. The discrepancies in results may be attributed to the following reasons.

1. Initially the section is uncracked and also brick/concrete can take some tension. Theoretical analysis does not take this into account. Thus initially theoretical curve is below the experimental curve.

2. The proving ring may register a lower load than the actually applied, because some part of the load is lost when clips gets loosened.

3. Perfect fixity of pier bases can not be achieved in practice. Any deformations, how-so-ever small they may be, would reduce the stiffness of the column bringing down the load - deflection curve.

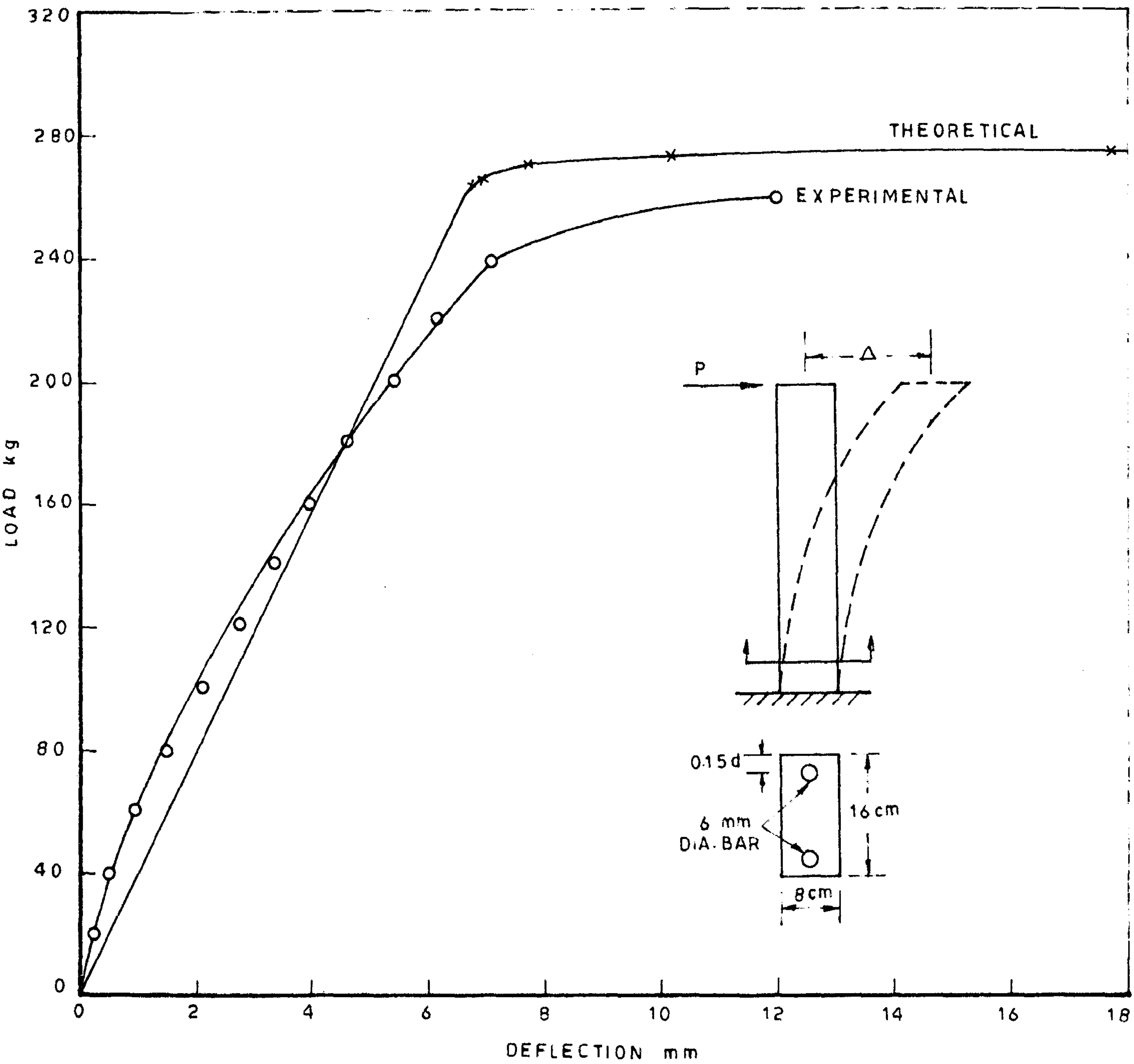
Some other important factors which contribute to these discrepancies are workmanship and ability of mortar to form a strong and durable bond with brick/concrete and reinforcement.

It is observed from the experimental results that a slight increase of ductility in concrete increases the steel ductility considerably. However strain in brick/ concrete is not reached to its ultimate and is less than the theoretical value. This may be due to lack of bond between mortar and brick/concrete. Due to this brick and concrete may not be utilized to full extent. This indicates the need of properly spaced stirrups in the piers for achieving large ductilities.

Consideration of transition of tension steel from elastic to inelastic in the theoretical approach presented gives theoretical load - deflection curve a general nonlinear shape instead of elasto - plastic, which would have been obtained, if no transition was considered. This tallies very well with the actual experimental results obtained.

The experimental load - deflection curve in case of R.B. portal is not close to the theoretical one and lies much below it. This is because perfect fixity at junction of the top of the pier with beam is not maintained when load is increased and top joint gradually changes condition from perfect to imperfect. This is taken into account in the theoretical analysis presented in chapter III and curves for changing end condition and different beam and pier stiffness ratios are drawn.

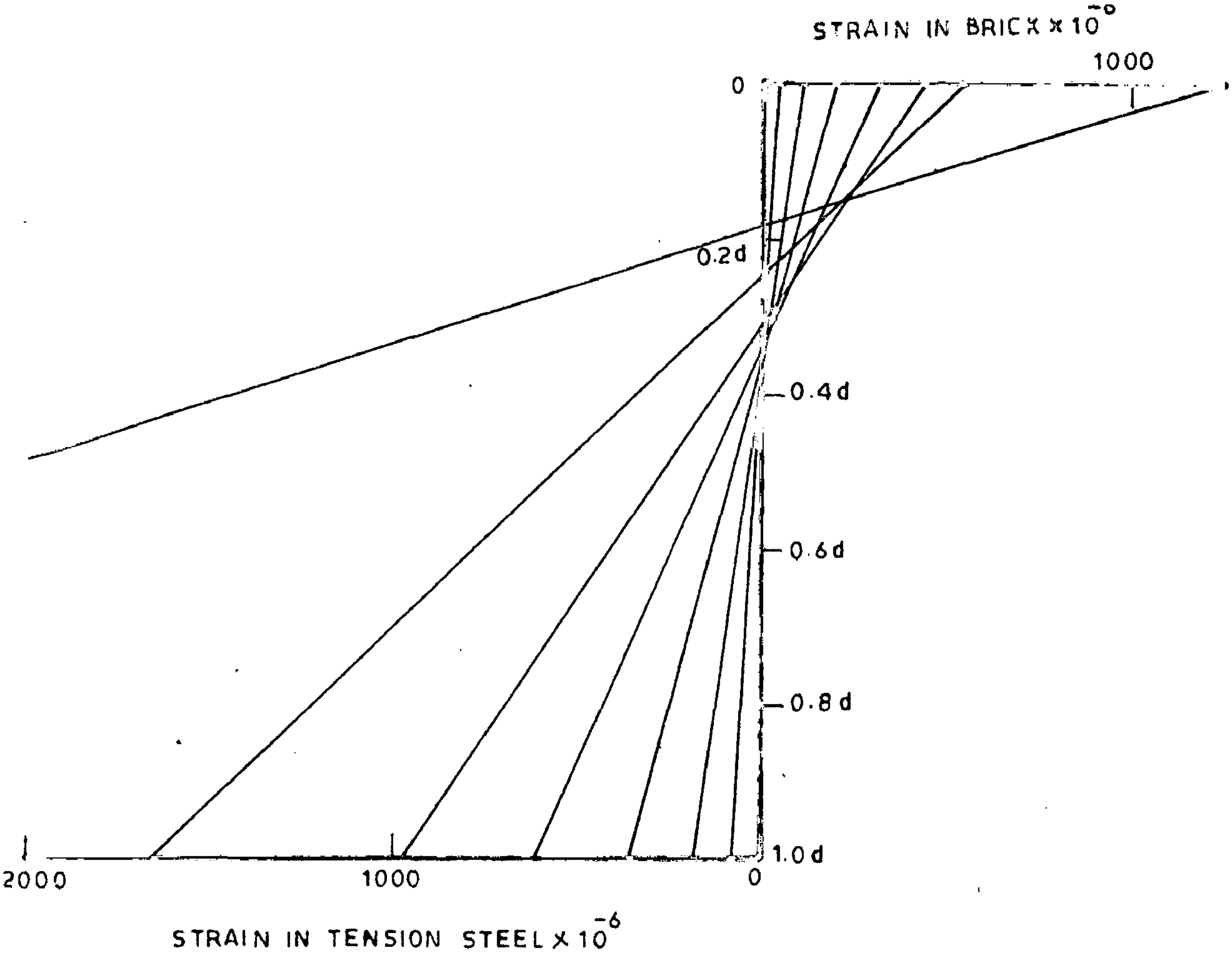
Since structure did not maintain rigid connection at the junction of column and beam, the rotations of the ends of these two members were different. This means that 'α', must not have had a value equal to unity (which refers to perfectly rigid joint) and must have assumed a value smaller than this. Also this value could be changing during the process of loading. The fig. 6.12 shows the response of various frame types with different 'α' and 'K_b/K_c' ratios which envelope the experimentally obtained load - deflection curve. Actual conditions of the frame can not be taken care of in such a situation. The actual value of K_b/K_c could be different than that taken for theoretical curves, because E, value of masonry could be very much different. Fig. 3.6, however indicate the order of difference expected in deflection due to variation of K_b/K_c



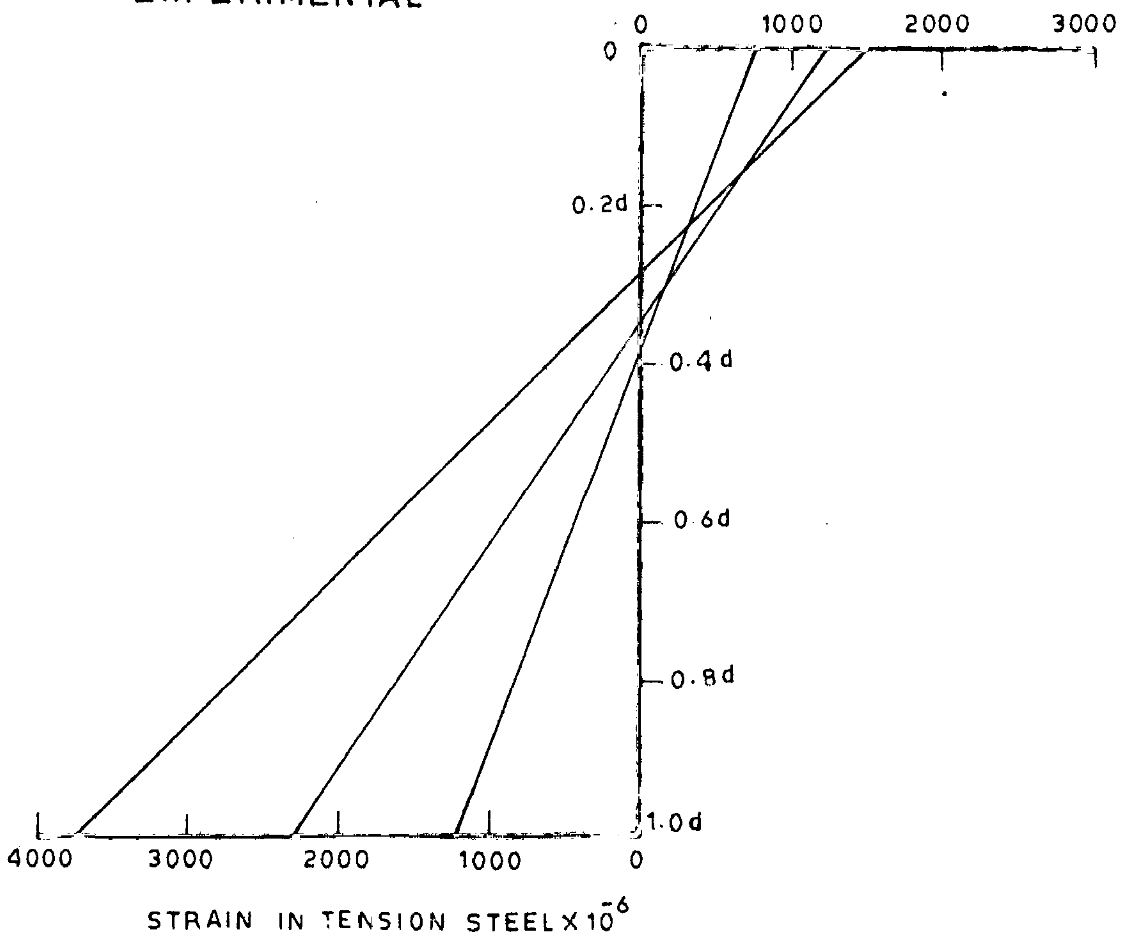
LOAD VS DEFLECTION CURVE

REINFORCED BRICK CANTILEVER COLOUMN WITH 6mm DIA. BARS

FIGURE _ 6.8

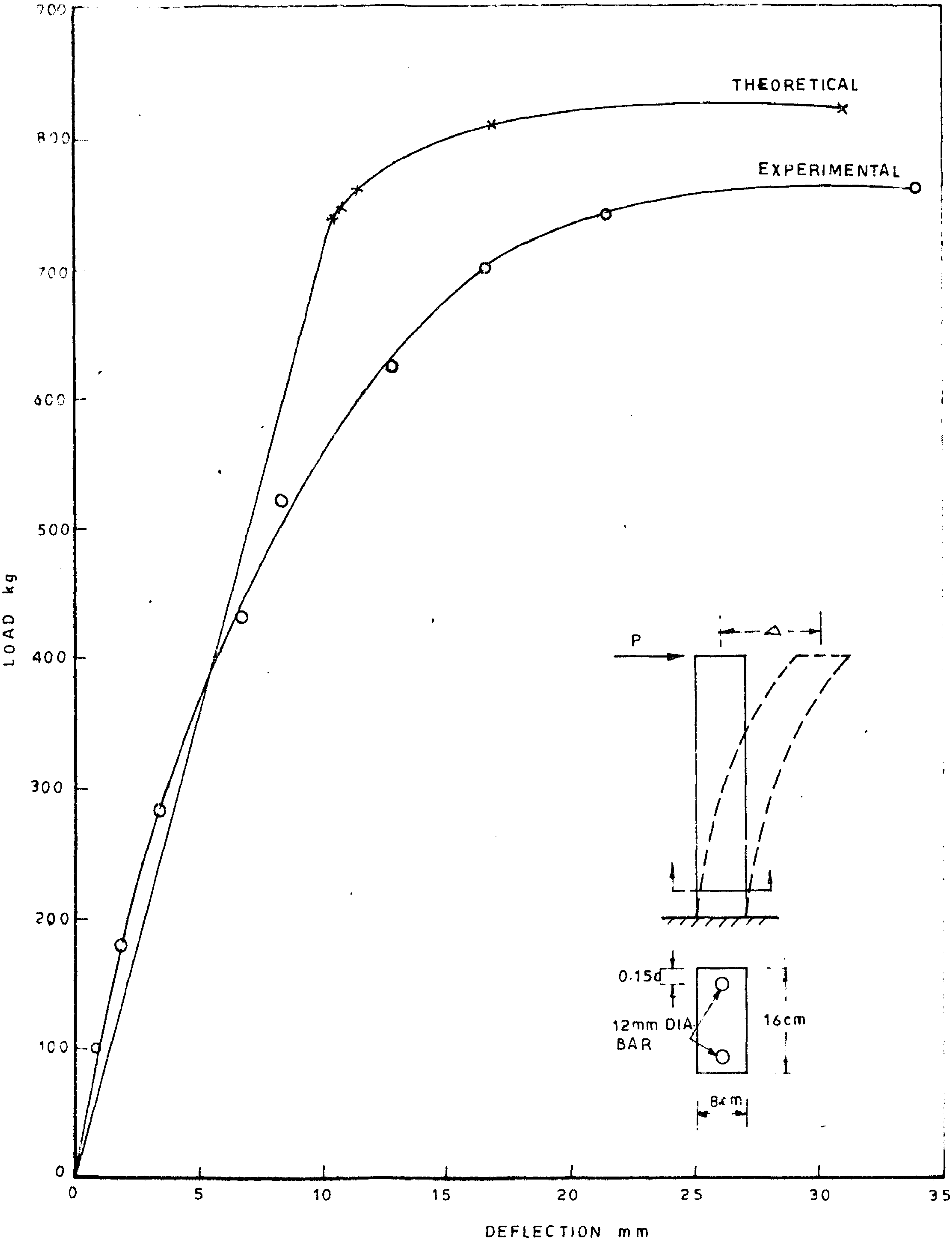


EXPERIMENTAL



THEORETICAL

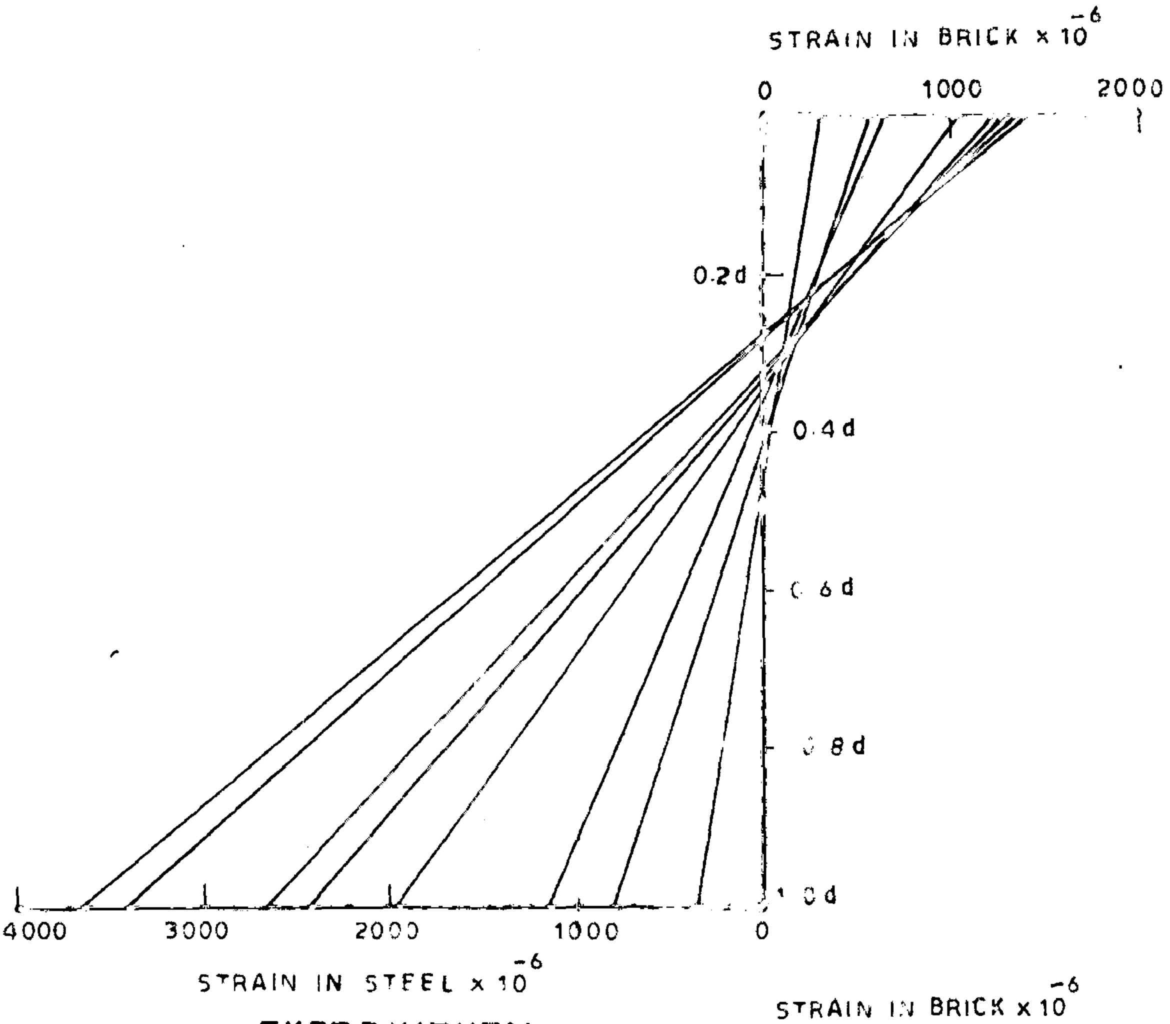
FIG. 6.9 _ STRAIN VARIATION IN THE SECTION WITH 6mm DIA. BAR R.B. CANTILEVER COLUMN



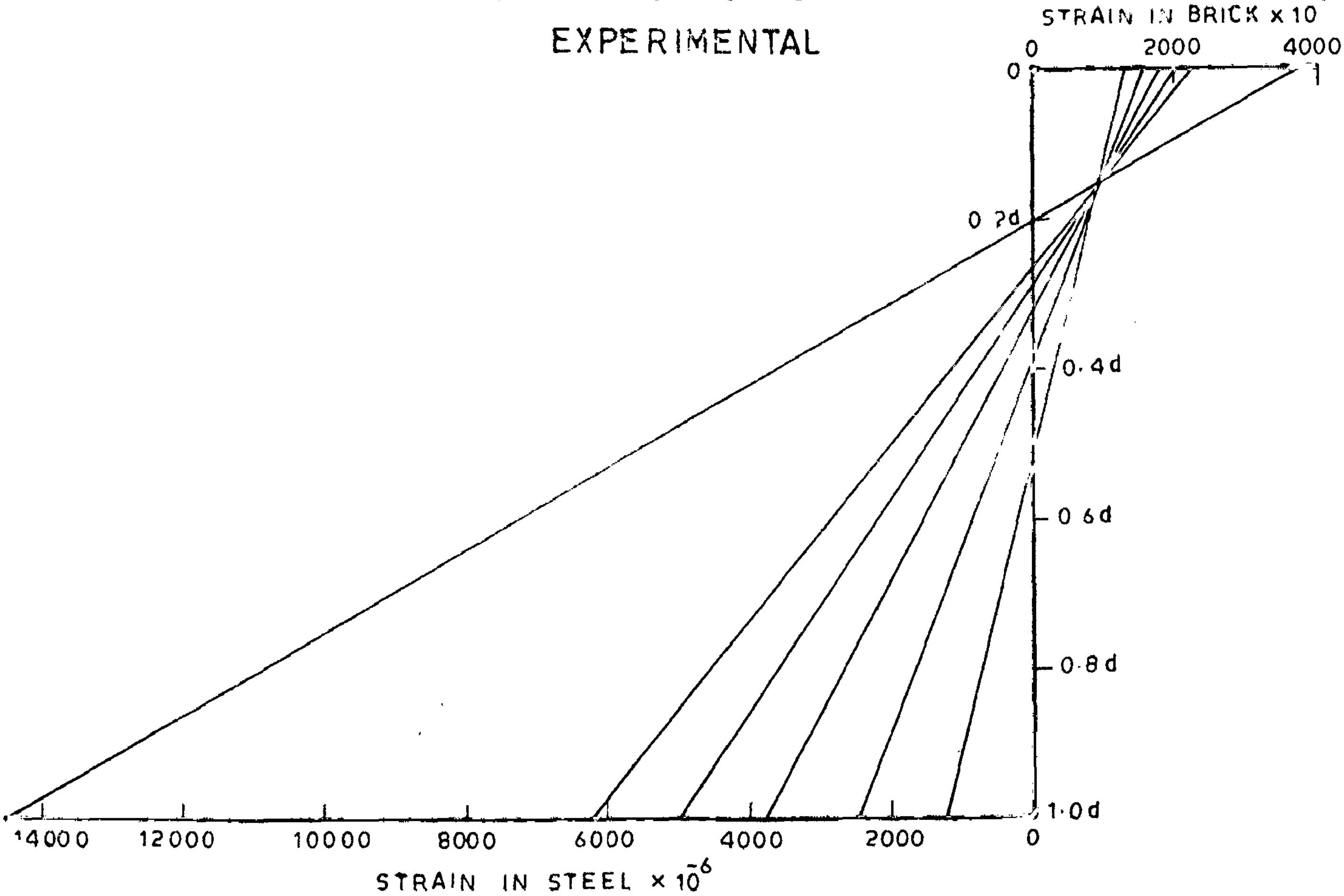
LOAD VS DEFLECTION CURVE

REINFORCED BRICK CANTILEVER COLUMN WITH 12mm DIA BAR

FIGURE_6.10

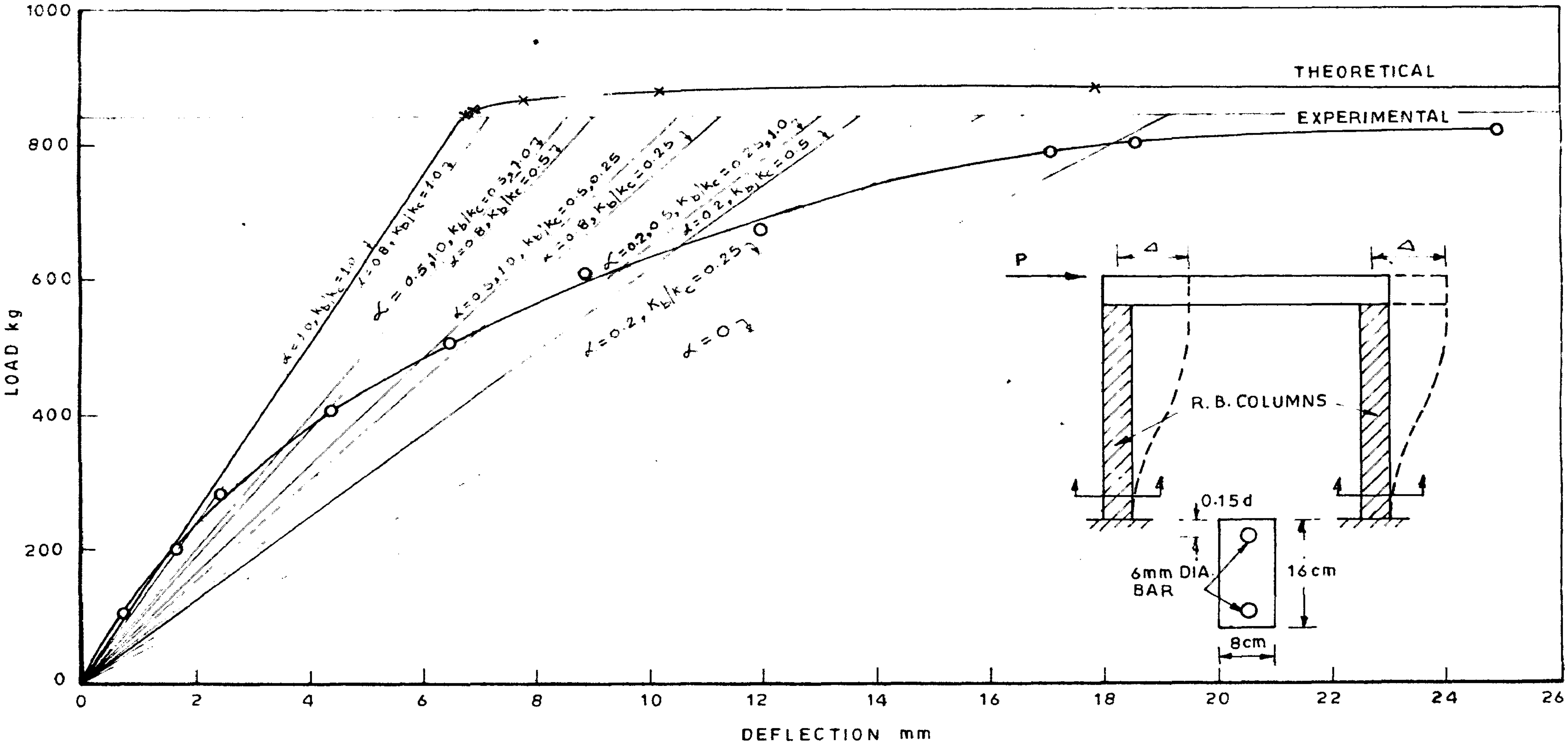


EXPERIMENTAL



THEORETICAL

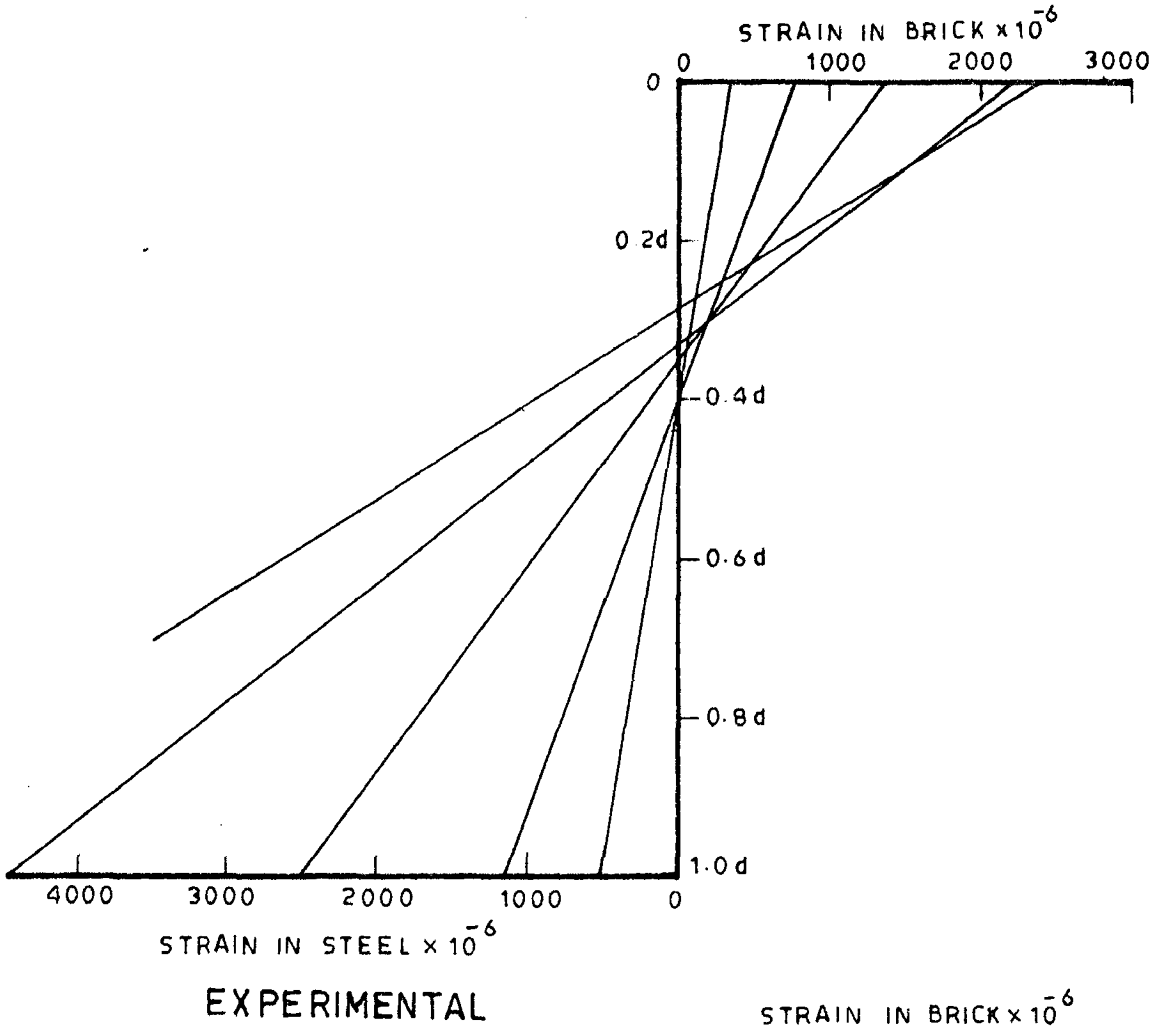
FIG. 6.11_STRAIN VARIATION IN THE SECTION WITH 12mm DIA. BARS R.B. CANTILEVER COLUMN



LOAD VS DEFLECTION CURVE

REINFORCED BRICK COLUMN (PORTAL) WITH 6 mm DIA. BARS

FIGURE _6.12



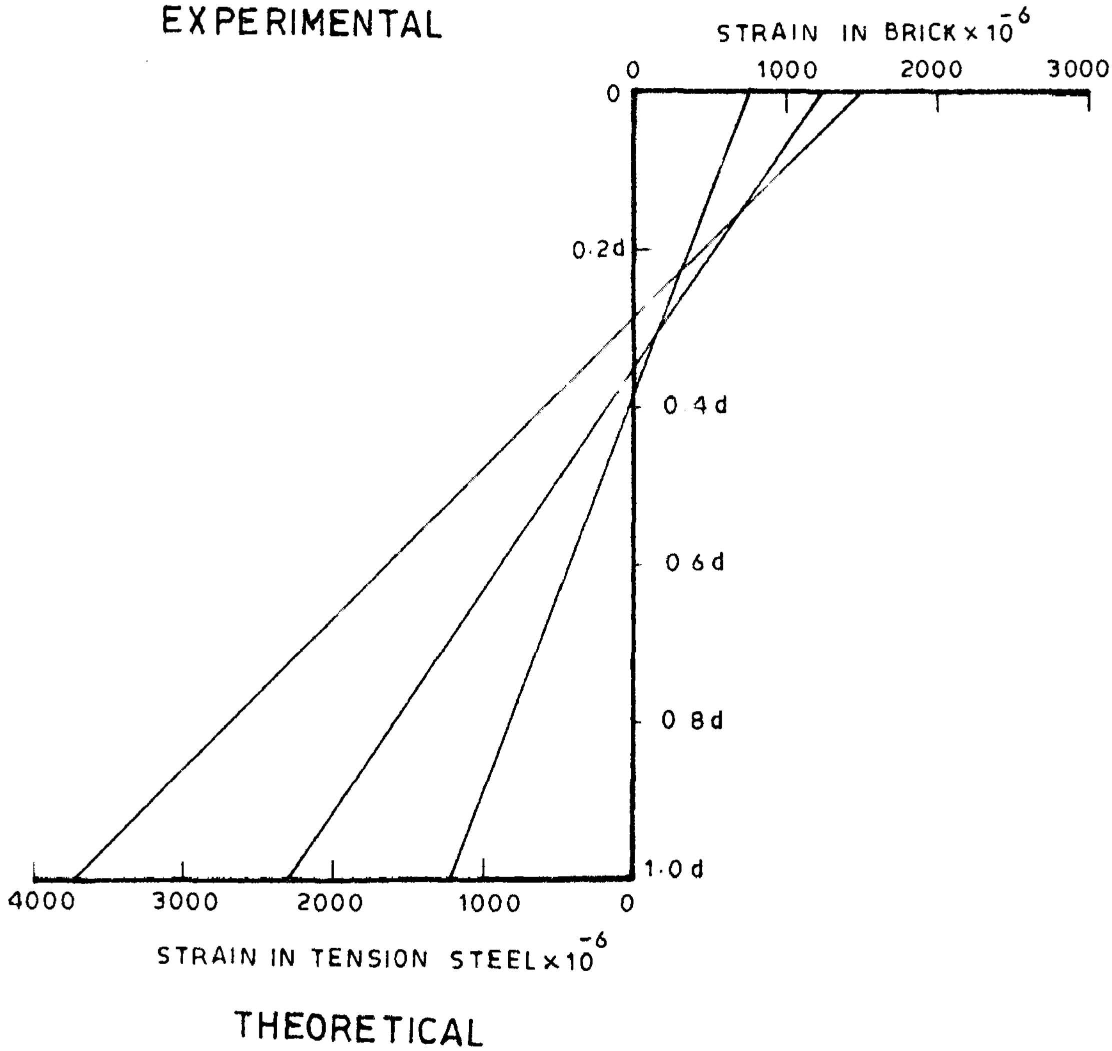
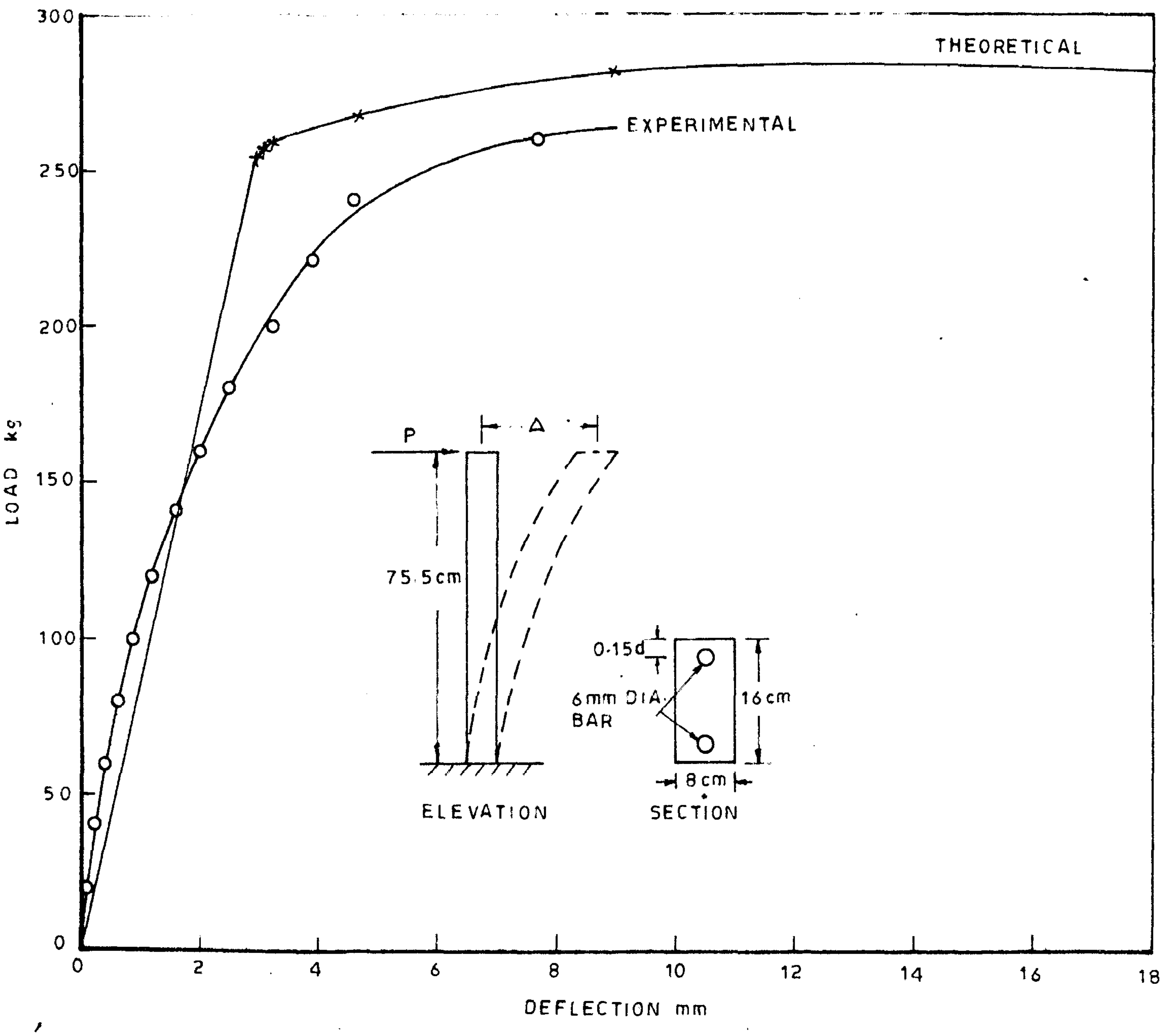


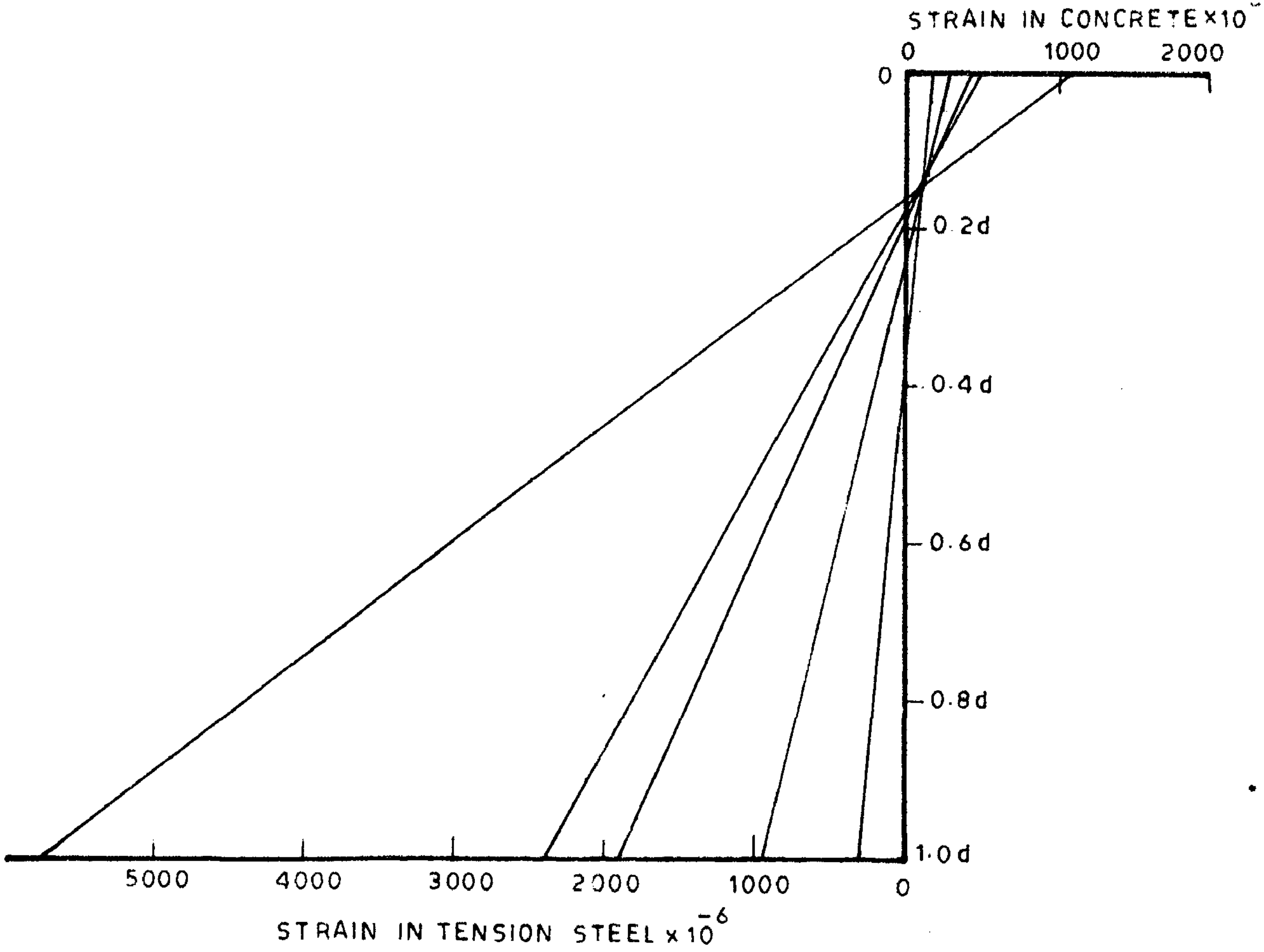
FIG. 6.13_STRAIN VARIATION IN THE SECTION WITH 6 mm DIA. BARS (R.B. PORTAL)



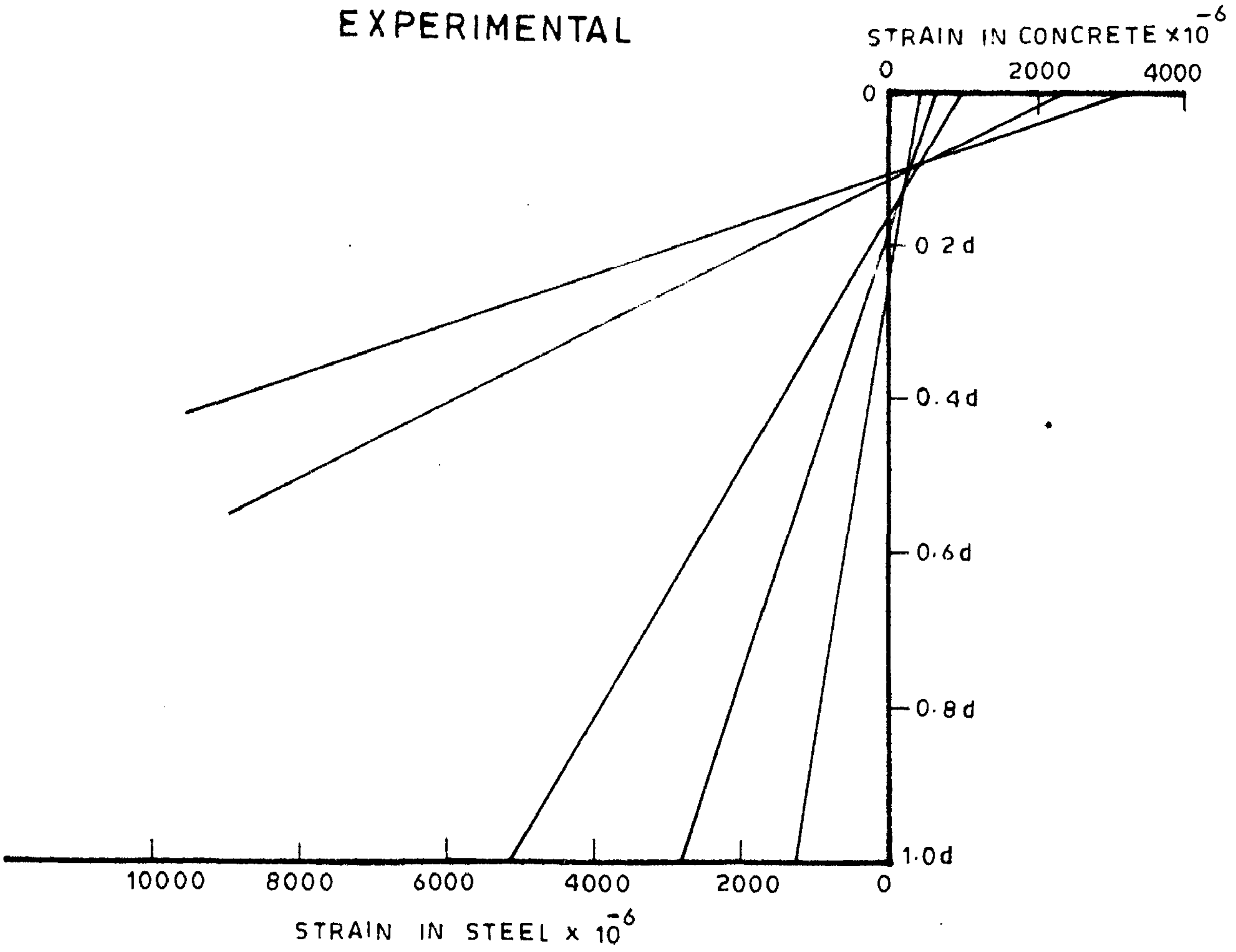
LOAD VS DEFLECTION CURVE

REINFORCED CONCRETE CANTILEVER COLUMN WITH 6mm DIA. BARS

FIGURE _ 6.14

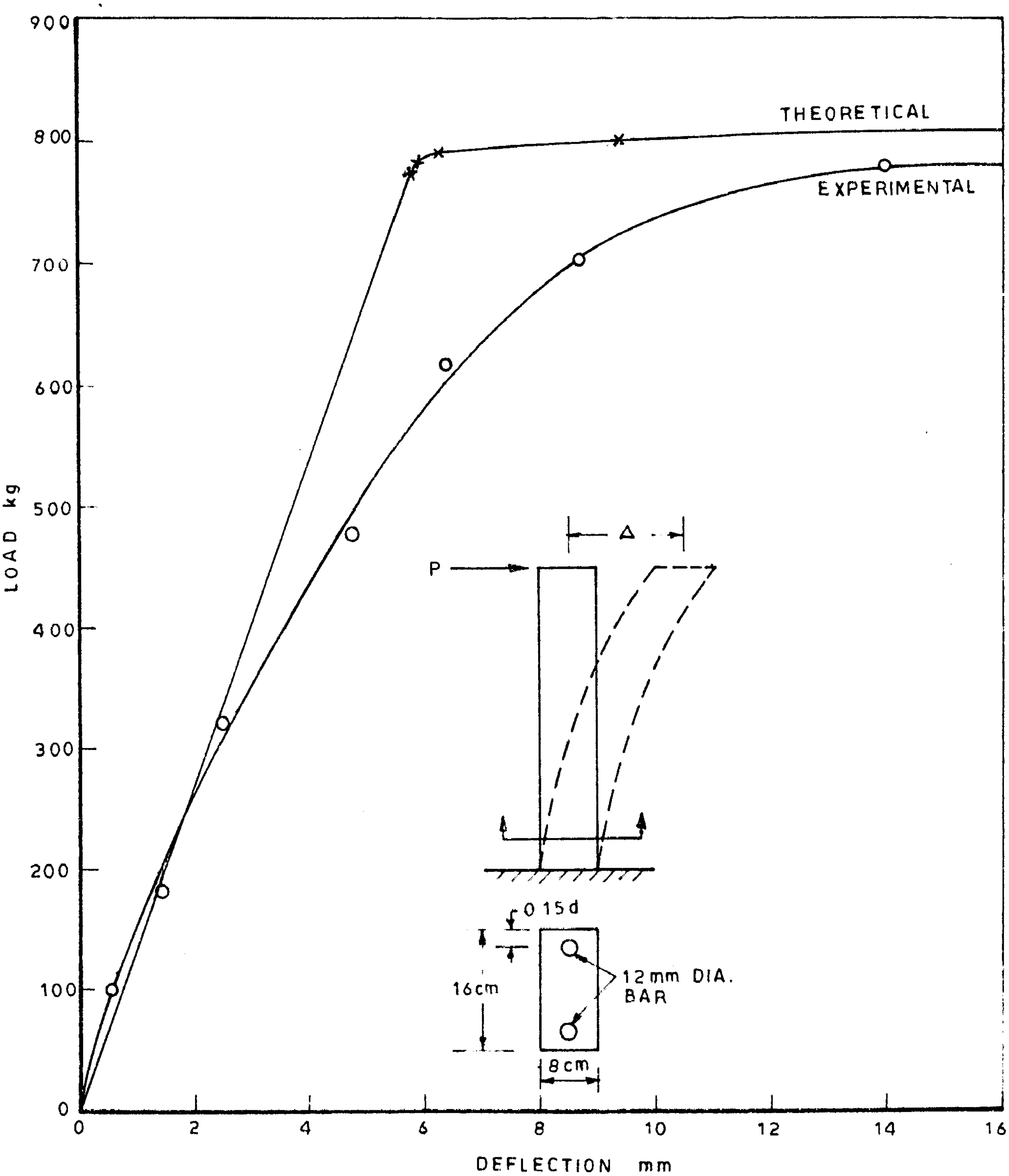


EXPERIMENTAL



THEORETICAL

FIG. 6.15_ STRAIN VARIATION IN THE SECTION WITH 6 mm DIA. BARS R.C. CANTILEVER COLUMN



LOAD VS DEFLECTION CURVE

REINFORCED CONCRETE CANTILEVER COLUMN WITH 12 mm DIA. BARS

FIGURE _6.16

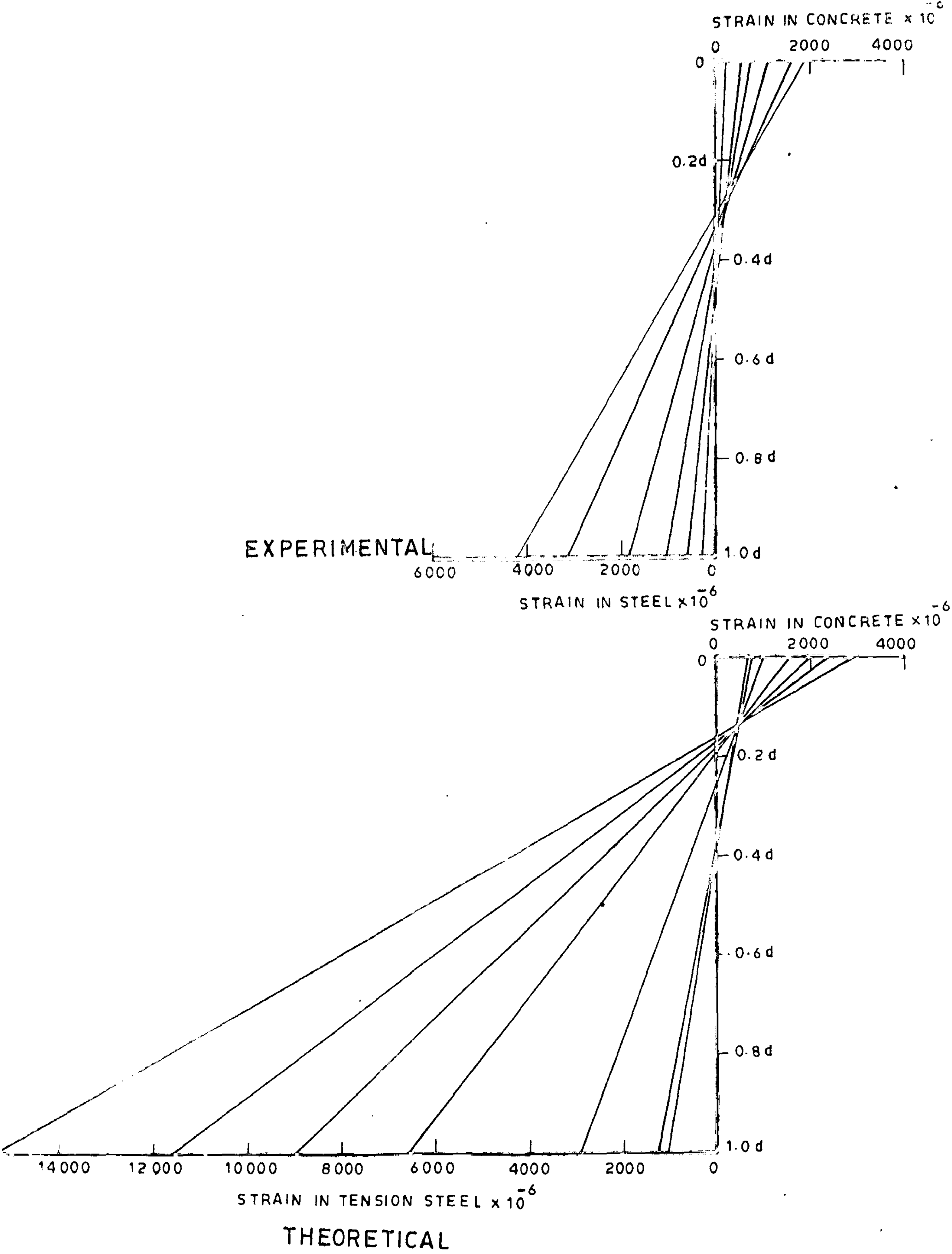


FIG. 6.17_ STRAIN VARIATION IN THE SECTION WITH 12 mm DIA. BAR R.C. CANTILEVER COLUMN

CHAPTER VII

CONCLUSIONS

On the basis of results obtained from the theoretical and experimental investigation, reported in earlier chapters, these can be summarised as below ;

I. In reinforced brick/concrete piers as the cover of reinforcement increases, the yielding of tension steel takes place at lesser load and there is reduction of ultimate moment of resistance of the section. However due to this ductility requirement in tension steel can be reduced if required.

II Ultimate load carrying capacity of piers increases with percentage of steel and is proportional to it for same cover for steel. The increase in percentage of steel decreases the ductility of tensile steel and for lower value of percentage of steel, large ductility should be expected.

III A slight increase in maximum strain level of concrete increases the ductility in steel appreciably and Since the load deflection curves are of general nonlinear type, the energy absorbing capacity of the members is considerably large. This would help the structure to withstand earthquake shocks better.

IV Properly spaced stirrups should be provided in the column in order to avoid slipping of bars and to achieve large ductility in tension steel

V The ultimate lateral load carrying capacity of reinforced brick and reinforced concrete column is proportional to the yield stress of tension steel.

VI The section is to be so designed that at ultimate load carrying capacity of the section, brick has reached to yield stress and tension steel has some workable ductility. In case of reinforced concrete section, the design is to be such that at ultimate stage, concrete reaches the maximum strain level while tension steel at that stage has some workable ductility.

VII The increase in lateral load carrying capacity of the shear wall (chosen for study) over its elastic strength, is about 23 percent, considering yielding in all the constituent piers (see chapter V).

VIII The deflection of the shear wall at the ultimate stage works out to about six times its elastic deflection.

IX The elastic and inelastic behaviour of reinforced brick/concrete members as envisaged in the theoretical approach presented in this thesis compares reasonably well with the experimental observations.

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APPENDIX A (COMPUTOR PROGRAMME)

```
C  C  PROGRAMME TO CALCULATE DEFLECTION OF R.B CANTILEVER COLUMN RAJESH
      DIMENSION TH(5)
      READ 1,B,TD,ES,EB,AL,YY,AI
      READ 2,L,M,N
      READ 1,(TH(K),K=1,5)
    1 FORMAT (7F10.1)
    2 FORMAT  (3I4)
      PI=3.14285
      A=AI
      AM=ES/EB
      DO 40  I=1,L
      D=TD/(1.+A)
      P=0.0
      DO50  J=1,M
      IF(P-.0025) 10,10,11
   10 P=P+.0025  $GO TO 12
   11 P=P+.005
   12 C=2.*AM*P
      R=SQRTF(P*B*D/PI)
      AN=-C+SQRTF(C*C+C*(1.+A))
      AJ=1.-AN/3.+(AN/3.-A)*(AN-A)/(1.-AN)
      DO 60 K=1,N
      T=TH(K)
      T=T*PI/180.0
      T2=2.*T
      X=SINF(T)
      Y=SINF(T2)
      Z=COSF(T)
      E=(1.-AN)*D
      BM1=0.5*AJ*D*(T-0.5*Y)+0.3333*X**3
      BM2=0.5*(1.-AN)*AJ*D*D*(PI-T+0.5*Y)
      BM3=(1.-AN+AJ)*D*R*0.3333*X**3
      BM4=.125*R*R*(PI-T+.5*Y)-Z*X**3
      BMR=2.*R*R*YY*(BM1+(BM2-BM3+BM4)/(E+R*Z))
      PLOAD=BMR/AL
      EB1=0.3333*EB*B*(AN*D)**3
      EB2=ES*B*P*D*D*D*(AN-A)**2
      EB3=ES*R*R*(E*E+R*R*.25)*(PI-T+.5*Y)
      RX=R*X
      EB4=2.*ES*(Z*R+2.*E*.3333)*RX**3
      EBI=EB1+EB2+EB3-EB4
      ATT=R*R*(PI-T+.5*Y)
      EBA=EB*B*AN*D+ES*(PI*R*R+ATT)
      DEFB=0.3333*BMR*AL**2/EBI
      DEFS=2.4*BMR/EBA
      TOTD=DEFB+DEFS
      PUNCH4,A,P,T,PLOAD,BMR,EBI,EBA,TOTD
    4 FORMAT(3F6.3,5E12.5)
   60 CONTINUE
   50 CONTINUE
   40 A=A+0.05
      STOP
      END
```

.200	.003	.524	0.98342E+02	0.74248E+04	0.45050E+08	0.18977E+07	0.32252
.200	.003	.786	0.99110E+02	0.74828E+04	0.43010E+08	0.18629E+07	0.34018
.200	.003	1.571	0.10212E+03	0.77098E+04	0.30039E+08	0.16337E+07	0.49895
.200	.003	3.143	0.10157E+03	0.76684E+04	0.15122E+08	0.13541E+07	0.97701
.200	.005	0.000	0.18769E+03	0.14171E+05	0.81018E+08	0.31360E+07	0.34315
.200	.005	.524	0.18926E+03	0.14289E+05	0.79404E+08	0.31037E+07	0.35295
.200	.005	.786	0.19123E+03	0.14438E+05	0.75996E+08	0.30342E+07	0.37236
.200	.005	1.571	0.19892E+03	0.15018E+05	0.54791E+08	0.25758E+07	0.53476
.200	.005	3.143	0.19918E+03	0.15038E+05	0.31213E+08	0.20164E+07	0.93325
.200	.010	0.000	0.36043E+03	0.27213E+05	0.14721E+09	0.54502E+07	0.36320
.200	.010	.524	0.36469E+03	0.27534E+05	0.14431E+09	0.53855E+07	0.37477
.200	.010	.786	0.36980E+03	0.27920E+05	0.13826E+09	0.52465E+07	0.39644
.200	.010	1.571	0.38982E+03	0.29431E+05	0.10183E+09	0.43297E+07	0.56542
.200	.010	3.143	0.39428E+03	0.29768E+05	0.63291E+08	0.32111E+07	0.91584
.200	.015	0.000	0.52697E+03	0.39786E+05	0.21213E+09	0.77207E+07	0.36871
.200	.015	.524	0.53452E+03	0.40357E+05	0.20789E+09	0.76237E+07	0.38152
.200	.015	.786	0.54345E+03	0.41030E+05	0.19911E+09	0.74152E+07	0.40479
.200	.015	1.571	0.57846E+03	0.43673E+05	0.14767E+09	0.60400E+07	0.57925
.200	.015	3.143	0.58968E+03	0.44521E+05	0.95274E+08	0.43620E+07	0.91230
.200	.020	0.000	0.68889E+03	0.52011E+05	0.27671E+09	0.99774E+07	0.36962
.200	.020	.524	0.70016E+03	0.52862E+05	0.27106E+09	0.98482E+07	0.38341
.200	.020	.786	0.71337E+03	0.53859E+05	0.25944E+09	0.95702E+07	0.40793
.200	.020	1.571	0.76526E+03	0.57777E+05	0.19288E+09	0.77365E+07	0.58703
.200	.020	3.143	0.78526E+03	0.59287E+05	0.12721E+09	0.54992E+07	0.91131
.250	.003	0.000	0.87840E+02	0.66319E+04	0.38722E+08	0.18623E+07	0.33395
.250	.003	.524	0.88386E+02	0.66732E+04	0.37914E+08	0.18468E+07	0.34307
.250	.003	.786	0.89109E+02	0.67277E+04	0.36198E+08	0.18134E+07	0.36202
.250	.003	1.571	0.91979E+02	0.69444E+04	0.25299E+08	0.15934E+07	0.53196
.250	.003	3.143	0.91427E+02	0.69027E+04	0.12802E+08	0.13249E+07	0.10369E+01
.250	.005	0.000	0.16628E+03	0.12554E+05	0.66680E+08	0.30411E+07	0.36761
.250	.005	.524	0.16774E+03	0.12664E+05	0.65342E+08	0.30101E+07	0.37833
.250	.005	.786	0.16957E+03	0.12803E+05	0.62522E+08	0.29434E+07	0.39949
.250	.005	1.571	0.17687E+03	0.13354E+05	0.45031E+08	0.25033E+07	0.57622
.250	.005	3.143	0.17704E+03	0.13367E+05	0.25687E+08	0.19664E+07	0.10050E+01
.250	.010	0.000	0.31513E+03	0.23792E+05	0.11874E+09	0.52674E+07	0.39153
.250	.010	.524	0.31906E+03	0.24089E+05	0.11637E+09	0.52053E+07	0.40440
.250	.010	.786	0.32380E+03	0.24447E+05	0.11143E+09	0.50719E+07	0.42840
.250	.010	1.571	0.34268E+03	0.25873E+05	0.81900E+08	0.41918E+07	0.61500
.250	.010	3.143	0.34670E+03	0.26176E+05	0.50956E+08	0.31178E+07	0.99612
.250	.015	0.000	0.45766E+03	0.34553E+05	0.16950E+09	0.74492E+07	0.39844
.250	.015	.524	0.46462E+03	0.35079E+05	0.16605E+09	0.73561E+07	0.41281
.250	.015	.786	0.47287E+03	0.35702E+05	0.15892E+09	0.71560E+07	0.43878
.250	.015	1.571	0.50575E+03	0.38184E+05	0.11756E+09	0.58358E+07	0.63281
.250	.015	3.143	0.51602E+03	0.38960E+05	0.75975E+08	0.42249E+07	0.99639
.250	.020	0.000	0.59575E+03	0.44979E+05	0.21990E+09	0.96169E+07	0.39983
.250	.020	.524	0.60612E+03	0.45762E+05	0.21531E+09	0.94928E+07	0.41538
.250	.020	.786	0.61829E+03	0.46681E+05	0.20590E+09	0.92259E+07	0.44288
.250	.020	1.571	0.66688E+03	0.50350E+05	0.15263E+09	0.74656E+07	0.64294
.250	.020	3.143	0.68529E+03	0.51739E+05	0.10090E+09	0.53178E+07	0.99761

0 STOP END AT S. 0040 + 01 L. Z

APPENDIX B

DEFLECTION OF R.B PIER FOR DIFFERENT PERCENTAGE AND COVER FOR STEEL

C C PROGRAMME TO CALCULATE DEFLECTION OF R.B CANTILEVER COLUMN RAJESH

.100	.003	0.000	0.12418E+03	0.93756E+04	0.66923E+08	0.20298E+07	0.27725
.100	.003	.524	0.12486E+03	0.94268E+04	0.65551E+08	0.20122E+07	0.28446
.100	.003	.786	0.12574E+03	0.94935E+04	0.62631E+08	0.19743E+07	0.29953
.100	.003	1.571	0.12910E+03	0.97472E+04	0.43992E+08	0.17243E+07	0.43453
.100	.003	3.143	0.12858E+03	0.97080E+04	0.22441E+08	0.14192E+07	0.83831
.100	.005	0.000	0.24384E+03	0.18410E+05	0.12234E+09	0.33505E+07	0.29908
.100	.005	.524	0.24568E+03	0.18549E+05	0.11997E+09	0.33153E+07	0.30719
.100	.005	.786	0.24797E+03	0.18721E+05	0.11493E+09	0.32395E+07	0.32334
.100	.005	1.571	0.25665E+03	0.19377E+05	0.83417E+08	0.27394E+07	0.45831
.100	.005	3.143	0.25718E+03	0.19417E+05	0.48027E+08	0.21292E+07	0.79001
.100	.010	0.000	0.47738E+03	0.36042E+05	0.22943E+09	0.58647E+07	0.31321
.100	.010	.524	0.48240E+03	0.36421E+05	0.22506E+09	0.57942E+07	0.32255
.100	.010	.786	0.48840E+03	0.36874E+05	0.21589E+09	0.56426E+07	0.34019
.100	.010	1.571	0.51124E+03	0.38598E+05	0.16008E+09	0.46424E+07	0.47806
.100	.010	3.143	0.51683E+03	0.39021E+05	0.10000E+09	0.34221E+07	0.76869
.100	.015	0.000	0.70457E+03	0.53195E+05	0.33532E+09	0.83371E+07	0.31671
.100	.015	.524	0.71353E+03	0.53871E+05	0.32887E+09	0.82313E+07	0.32693
.100	.015	.786	0.72405E+03	0.54666E+05	0.31545E+09	0.80038E+07	0.34564
.100	.015	1.571	0.76422E+03	0.57698E+05	0.23552E+09	0.65036E+07	0.48674
.100	.015	3.143	0.77780E+03	0.58724E+05	0.15222E+09	0.46731E+07	0.76310
.100	.020	0.000	0.92656E+03	0.69955E+05	0.44090E+09	0.10796E+08	0.31699
.100	.020	.524	0.93998E+03	0.70968E+05	0.43228E+09	0.10655E+08	0.32790
.100	.020	.786	0.95561E+03	0.72149E+05	0.41445E+09	0.10352E+08	0.34747
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.100	.020	3.143	0.10393E+04	0.78469E+05	0.20451E+09	0.59110E+07	0.76082
.150	.003	0.000	0.10974E+03	0.82854E+04	0.55223E+08	0.19695E+07	0.29515
.150	.003	.524	0.11037E+03	0.83327E+04	0.54078E+08	0.19527E+07	0.30299
.150	.003	.786	0.11119E+03	0.83948E+04	0.51644E+08	0.19164E+07	0.31934
.150	.003	1.571	0.11436E+03	0.86341E+04	0.36135E+08	0.16772E+07	0.46632
.150	.003	3.143	0.11382E+03	0.85935E+04	0.18250E+08	0.13854E+07	0.90949
.150	.005	0.000	0.21329E+03	0.16103E+05	0.99199E+08	0.32388E+07	0.32035
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.150	.005	.786	0.21710E+03	0.16391E+05	0.93112E+08	0.31325E+07	0.34700
.150	.005	1.571	0.22525E+03	0.17006E+05	0.67313E+08	0.26542E+07	0.49538
.150	.005	3.143	0.22563E+03	0.17035E+05	0.38482E+08	0.20706E+07	0.86079
.150	.010	0.000	0.41400E+03	0.31257E+05	0.18336E+09	0.56486E+07	0.33716
.150	.010	.524	0.41862E+03	0.31606E+05	0.17980E+09	0.55811E+07	0.34756
.150	.010	.786	0.42415E+03	0.32023E+05	0.17237E+09	0.54361E+07	0.36711
.150	.010	1.571	0.44548E+03	0.33634E+05	0.12734E+09	0.44794E+07	0.51983
.150	.010	3.143	0.45046E+03	0.34010E+05	0.79276E+08	0.33121E+07	0.83971
.150	.015	0.000	0.60851E+03	0.45942E+05	0.26628E+09	0.80156E+07	0.34155
.150	.015	.524	0.61672E+03	0.46562E+05	0.26106E+09	0.79144E+07	0.35298
.150	.015	.786	0.62640E+03	0.47293E+05	0.25022E+09	0.76968E+07	0.37384
.150	.015	1.571	0.66383E+03	0.50119E+05	0.18616E+09	0.62618E+07	0.53070
.150	.015	3.143	0.67615E+03	0.51049E+05	0.12014E+09	0.45109E+07	0.83446
.150	.020	0.000	0.79813E+03	0.60259E+05	0.34888E+09	0.10369E+08	0.34209
.150	.020	.524	0.81042E+03	0.61187E+05	0.34191E+09	0.10234E+08	0.35435
.150	.020	.786	0.82478E+03	0.62271E+05	0.32754E+09	0.99442E+07	0.37624
.150	.020	1.571	0.88039E+03	0.66470E+05	0.24432E+09	0.80308E+07	0.53675
.150	.020	3.143	0.90221E+03	0.68117E+05	0.16101E+09	0.56963E+07	0.83245
.200	.003	0.000	0.97759E+02	0.73808E+04	0.46009E+08	0.19139E+07	0.31404

APENDIX C

EFFECT OF COVER AND PERCENTAGE OF STEEL ON ULTIMATE LOAD CARRYING CAPACITY OF R.B.PIERS

Cover for Steel	Percentage of Steel	Load Factor
0.10	0.25	1.03
	0.50	1.06
	1.00	1.08
	1.50	1.10
	2.00	1.12
0.15	0.25	1.04
	0.50	1.06
	1.00	1.10
	1.50	1.12
	2.00	1.13
0.20	0.25	1.04
	0.50	1.065
	1.0	1.10
	1.5	1.12
	2.0	1.14
0.25	0.25	1.04
	0.50	1.07
	1.0	1.10
	1.5	1.13
	2.0	1.15

APPENDIX D.

TABLE 6.1

REINFORCED BRICK CANTILEVER COLUMN WITH
6 MM. DIA. BAR

EXPERIMENTAL RESULTS

LOAD (KG)	DEFLECTION (MMS.)	STRAIN IN BRICK x 10^{-6}	STRAIN IN TENSION STEEL x 10^{-6}
0	0	0	0
20	0.22	40	80
40	0.56	70	120
60	0.97	110	180
80	1.47	155	278
100	2.10	190	360
120	2.78	275	485
140	3.32	320	610
160	3.95	340	630
180	4.60	380	720
200	5.38	440	960
220	6.18	495	1250
240	7.0	550	1660
260	12.08	1450	Very large strains

TABLE 6.2

REINFORCED BRICK CANTILEVER COLUMN WITH 12 MM. DIA. BAR.

EXPERIMENTAL RESULT

LOAD (KG.)	DEFLECTION (MMS)	STRAIN IN BRICK $\times 10^{-6}$	STRAIN IN TENSION STEEL $\times 10^{-6}$
0	0	0	0
100	0.85	300	360
180	1.8	590	810
285	3.4	760	1160
428	6.8	1060	1960
520	8.40	1240	2440
620	12.8	1280	2640
700	16.80	1360	3420
740	21.50	1410	3640
760	34.00	1320 (Strain gage chiped off)	Very large strains

TABLE 6.3

R. B. PORTAL WITH 6 MM. DIA. BAR

EXPERIMENTAL RESULT

LOAD (KG.)	DEFLECTION (MMS)	STRAIN IN BRICK $\times 10^{-6}$	STRAIN IN TENSION STEEL $\times 10^{-6}$
0	0	0	0
104	0.7	170	220
200	1.7	370	500
285	2.5	570	600
410	4.4	800	1140
505	6.6	1060	1600
616	9.0	1380	2500
680	12.1	1800	3400
800	17.2	2250	4500
806	18.8	2310	5085
816	24.9	2400	6000

TABLE 6.4

R. C. CANTILEVER WITH 6 MM. DIA. BAR

EXPERIMENTAL RESULT

LOAD (KG.)	DEFLECTION (MMS)	STRAIN IN BRICK $\times 10^{-6}$	STRAIN IN TENSION STEEL $\times 10^{-6}$
0	0	0	0
20	0.12	45	180
40	0.22	150	435
60	0.41	200	660
80	0.59	275	945
100	0.80	350	1260
120	1.20	400	1590
140	1.59	450	1905
160	2.0	480	2145
180	2.48	500	2400
200	3.25	550	2640
220	3.95	650	3255
240	4.61	750	3510
260	7.70	1095	5500

TABLE 6.5

R. C. CANTILEVER WITH 12 MM. DIA BAR

EXPERIMENTAL RESULT

LOAD (KG.)	DEFLECTION (MMS)	STRAIN IN CONCRETE $\times 10^{-6}$	STRAIN IN TENSION STEEL $\times 10^{-6}$
0	0	0	0
100	0.51	186	242
180	1.43	482	650
320	2.42	710	1100
400	3.51	820	1425
475	4.73	1110	1900
620	6.40	1275	2600
710	7.75	1550	3200
785	14.10	1910	4320

NOTATIONS

The notations are defined wherever they first appear. Here they are listed in alphabetical order for convienience of reference.

A	=	Area of the Section
a	=	Cover for steel (Fraction of depth)
b	=	Width of the section
d	=	Effective depth of the section
E	=	Modulus of Elasticity
e_c	=	Strain in concrete
e_{cs}	=	Strain in compressive steel
e_{cm}	=	Strain in concrete corrosponding to Maximum stress
f	=	Frequency of vibration
F_c	=	Force of compression
G	=	Modulus of rigidity
g	=	Acceleration due to gravity
H	=	Span of beam
h'	=	Equivalent height of piers
I	=	Moment of Inertia of the section
j	=	Lever arm coefficient
K_b	=	Stiffness of beam
K_c	=	Stiffness of Pier
L	=	Length of shear wall
L_1	=	Length of side piers

L_2 = Length of window and door opening

M_{bu} = Ultimate Moment of resistance of the section

m = Moduler ratio (E_s/E_b)

N = Distance of Neutral axis from compression edge (Fraction of 'd')

p = Percentage of steel

P = Lateral Load acting on pier

r = Radius of reinforcing bar

U = Strain Energy

w = Weight of column per cms. height

W = Weight of the structure

α_H = a Seismic coefficient

σ_{sc} = Stress in compressive steel

σ_b = Stress in brickwork

σ_{yst} = Yield stress of tension steel

σ_{st} = Stress in tension steel

σ_{mc} = Maximum stress in concrete

σ_c = Stress in concrete

$_s$ = Strain in steel

$_b$ = Strain in brick

θ = Angle upto which tension steel has yielded

Δ = Deflection of pier

Δ_{TBF} = Deflection of pier fixed at top and Bottom

α = Parameter which relates rotation of beam with that of volumn.

μ_s = Ductility in tension steel

μ_b = Ductility in brick

μ_c = Ductility in concrete