EARTHQUAKE RESISTANT DESIGN OF BUILDING ELEMENTS

A Dissertation

submitted in partial fulfilment

of the requirements for the Degree

of

MASTER OF ENGINEERING

in

CIVIL ENGINEERING

By

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July, 1970

: CERTIFICATE :

Certified that the dissertation entitled "EARTH-QUAKE RESISTANT DESIGN OF BUILDING ELEMENTS" which is being submitted by Shri Vineet Prakash Jain in partial fulfilment for the award of degree of Master of Engineering in Civil Engineering with specialisation in 'Earthquake Engineering' of University of Roorkee, Roorkee is a record of student's 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 a period of Six months from Jan '70 to June '70 in preparing this thesis for Master of Engineering degree at this University.

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CHAPTER I

INTRODUCTION

1.1. Considerable knowledge has been gained in the last three decades about the phenomena of ground motion, the characteristics of structures and their behaviour in earthquakes. Whenever an earthquake occurs, it induces forces in structures due to inertia effect of the mass of the structure. These forces tend to fear apart the various parts of the structure.

Since in India, an area of about 6,00,000 Sq. miles falls under heavy earthquake zone which have been subjected to severe earthquakes in the past seventy years, hence a earthquake resistant design of building elements is needed. The basic philosophy of earthquake resistant design is that the structure designed for earthquake region must serve two functions:

(i) For frequent small shocks, they must be capable of controlling damage to non-structural elements in the building.

(ii) For severe earthquakes they must have adequate ductility to accomodate large lateral deflections whereby the energy given by the earthquake can be absorbed.

Thus, to achieve an earthquake resistant design of the structure, they should be so designed that all parts of the structure are tied together and that combined stresses at any point. due to both static and dynamic forces should not exceed the strength of material at that point.

Since earthquake loadings are dynamic in nature, it becomes necessary to consider the possible effects of rate of loading on the strength and deformation capacity of structural element. Investigations have revealed that reinforced concrete exhibits an increase in strength which is primarily due to increased yield strength of steel. These increases can reach 40 percent for intermediate grade steel under the fastest laboratory loadings, with yielding occuring within 0.005 second of application of load(14)+. But in case of an earthquake, the loads are, however, applied much slowly and increase in yield strength is not more than 5 to 10 percent. It is not advised that this increase be considered directly in design, because not all components of structure will yield together, consequently, it is recommended that static strength of structural elements be used in design for earthquake loads, an approach that should be conservative (9).

1.2. To make a structure earthquake resistant, it should be so designed that its energy absorption capacity should not be more than the energy fed by the ground motion. This aim can be achieved by allowing the structure to go in inelastic range. In steel we can allow

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+ Numbers refers to correspondingly numbered items in the list of references.

strains upto 2.5 percent and in concrete the strains could be upto 0.3 percent to 0.5 percent and consequently the energy absorption capacity of the structure can be increased. But with this, many other factors come into picture e.g. percentage of steel, cover, stresses in building materials etc. and these factors also impose certain restrictions in accomodating these strains.

In India about 65 percent of population live in villages and build houses in brick masonary with mud, lime surkhi or cement sand mortar. Since economic conditions do not permit the use of costly and expensive materials, we look forward to reinforced brick masonary. In past few years efforts are going on for strengthening the buildings in seismic zones. As such the lateral force which a building can stand is very small. To increase this some methods have been tested. These methods consists of providing horizontal and vertical steel at:(3)

a) lintel band

b) lintel and plinth bands

c) vertical steel at corners only

d) vertical steel at Jambs only

e) vertical steel at Jambs as well as at corners.

f) lintel band in combination with (c), (d), (e).

In present study, a theoretical analysis of brick shear walls with openings is carried out. The moments shear and axial forces in piers are worked out using Bent method for various level of lateral loads. Equal amount of reinforcement is placed on both faces in piers and, by carrying out an elastic analysis stresses in brick and steel, position of neutral axis etc. are worked out.

Further, these columns are analysed in bending by taking into account the ductility considerations. Generalized expressions for the equilibrium equations are developed for reinforced brick and reinforced concrete sections. Their ultimate moment of resistance is obtained in terms of ductility in steel and concrete. In an effort to achieve the desired ductility in materials and to verify the developed expressions, some test specimens of reinforced brick and reinforced concrete are constructed. The experimental results are compared with the theoretical values for verification of the expressions developed.

CHAPTER II

BRIEF REVIEW

2.1. GENERAL

A brief description of the theoretical and experimental work done in this direction is given here to prepare a background for the work undertaken by the author at the School of Research and Training in Earthquake Engineering.

2.2.

2.2.1. In year 1952, Portland Cement Association(1) of America gave an approximate method to analyse shear walls with openings. According to this method, the doors and windows divide the wall into piers. These piers have deflections due to bending and shear and for a lateral load P, it is given by

$$\epsilon = \frac{Ph^3}{12EI} + \frac{1.2\,Ph}{GA} \qquad ..(2.1)$$

in which

ϵ	=	Horizontal deflection at top of piers.
h	=	Height of pier.
E	=	Modulus of Elasticity of pier.
G	=	Modulus of rigidity of pier.
I	=	Equivalent moment of Inertia about centroidal axis.
A	=	Cross-sectional area of pier.

For brickwork, Poisson's ratio is too small and hence by taking $E = 2G$

$$\epsilon = \left[\frac{P}{E} \quad \frac{h^3}{12I} + \frac{2.4h}{A}\right] \qquad ..(2.2)$$

and, the proportion of load sheared by each pier can be cal-culated from

$$p_i = \frac{\frac{1}{\epsilon_i}}{\sum \frac{1}{\epsilon_i}} \quad P \qquad ..(2.3)$$

Knowing this, the distribution of shears and moments in piers and resulting bending and shear stresses can be computed easily. Beside these stresses, the piers carry direct stresses due to vertical load and overturning effect of horizontal forces which must be taken into account.

2.2.2. Agnihotri,V.K.(2), concluded that if the opening moves upwards, shear in the piers decreases thereby increasing the strength of wall. If the size and vertical placing of opening remains fixed, then for smaller openings (approx. upto 40 percent of the height and length of wall), the strength of wall increases as opening moves towards the centre of wall.

2.2.3. Chandra,B.(5), concluded that an unreinforced brick building can resist earthquakes having a fairly high value of the seismic coefficient, if care is taken to see that the openings are, as far as possible, centrally and symet-rically placed.

2.2.4. The authors(15) concluded that the wall exhibits its capacity to take more load even after all the piers crack and this is the reason which enable a designer to choose a seismic coefficient much lower than the actual force to which a structure is subjected during an earthquake.

2.2.5. Jai Krishna and B.Chandra(3) studied the effect of reinforcement at various positions on the lateral strength of brick building through model tests. The authors conclude that:-

1. Horizontal steel alone at lintel level does not contribute to strength as failure occurs at plinth level.
2. Vertical steel at corners is very effective and increases the strength of str-ucture considerably. It will delay the inital cracking and take much more load before the final collapse.
3. Vertical steel at jambs only does not prevent the initial failure of the structure but does increase the overall resistance of the structure since corners near jambs are vulnerable to failure due to diagnol tension.

4. Combination of horizontal steel at lintel level and vertical steel at corners is still stronger a combination and of course, if vertical steel at jambs is also present, the effect is very much pronounced.

2.2.6. M.Lal (13) suggested bent method because in Portland Cement Association method, the shear force in piers is obtained by pier action and axial force is obtained by treating the wall as cantilever. These two assumptions are inconsistent. Moreover, it is assumed that the depth of rigid common element connecting the top of piers, does not have any influence on the pier action, but effects only the overturning moment and hence the axial forces in piers.

2.3 After the distribution of forces has been obtained, the problem remains of designing R.B.Section subjected to direct and bending forces.

Jai Krishna and B.Chandra(4) analysed the R.B. Sections taking into account the ductility of brick and steel. If the resistance of brickwork in tension was as good as in compression, the columns would have taken large horizontal forces without damage. It, therefore, appears necessary that its energy capacity is increased by providing steel reinforcement on tension faces. Energy absorption capacity can be increased appreciably by accepting some damage through yielding of steel and some inelastic deformations. Also energy of steel should not exceed the energy of brickwork 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 brickwork, from the following equations:

$$N = \frac{1}{1 + \frac{\mu \sigma_{st}}{\mu' m \sigma_b}} \qquad ..(2.4)$$

$$p = \frac{\sigma_b N}{2\sigma_{st}} \left[2 - \frac{1}{\mu'} \right] \qquad ..(2.5)$$

$$= \frac{m \cdot N}{3(2\mu-1)} \left\{ \frac{1}{\mu'} + 3(\mu' - 1) \right\} \left(\frac{\sigma_b}{\sigma_{st}} \right)^2$$

$$..(2.6)$$

in which,

p = percentage of steel

σ_b = stress in brickwork

σ_{st} = stress in tensile steel

μ' = ductility in brick work.

μ = ductility in tensile steel

m = modular ratio $\frac{E_s}{E_b}$

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

Hence the percentage of steel should lie in between these two limits to have a full utilization of steel and brickwork.

The above work was done for singly reinforce sections to begin with. One must consider however that in order to have a section which is earthquake resistant, it must be reinforced with equal amount of reinforcement on both the faces, as tension can occur on any face in the event of an earthquake.

C H A P T E R - III

ANALYSIS OF BRICK SHEAR WALLS WITH OPENINGS

3.1 ANALYSIS:

Fig. 3.1 shows the model chosen for the analysis. The openings divide the wall into a series of piers which will have moment, and force (thrust or tension) and shear due to vertical and lateral load (due to wind and earthquake force). To analyse these piers, a method similar to the one used for analysis of bents, has been used. This is described in the following paragraphs.

THE BENT METHOD:

According to this method, piers are assumed to be tied together by the upper and lower portions of the wall. The portion above (called spandrel) and below piers is assumed to be rigid. The lateral force is carried to bottom by shear and moments in piers (Fig. 3.2) and this action is similar to the action in a continuous beam. The spandrel of equivalent bent has much greater flexural rigidity as compared to piers.

For analysing a bent, it is convenient to make use of an equivalent frame concept in which the bending

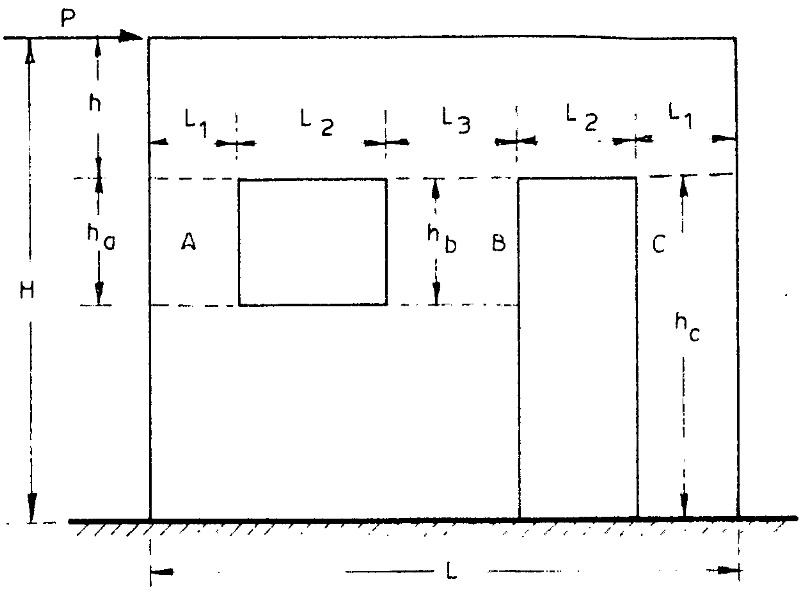

FIG.3.1 _ WALL DIMENSIONS

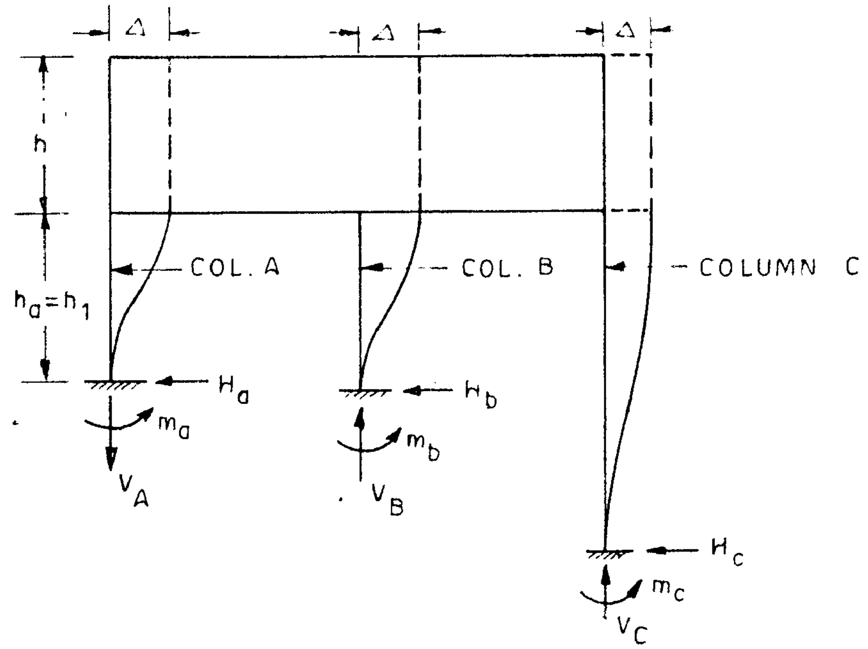

FIG.3.2_ EQUIVALENT BENT

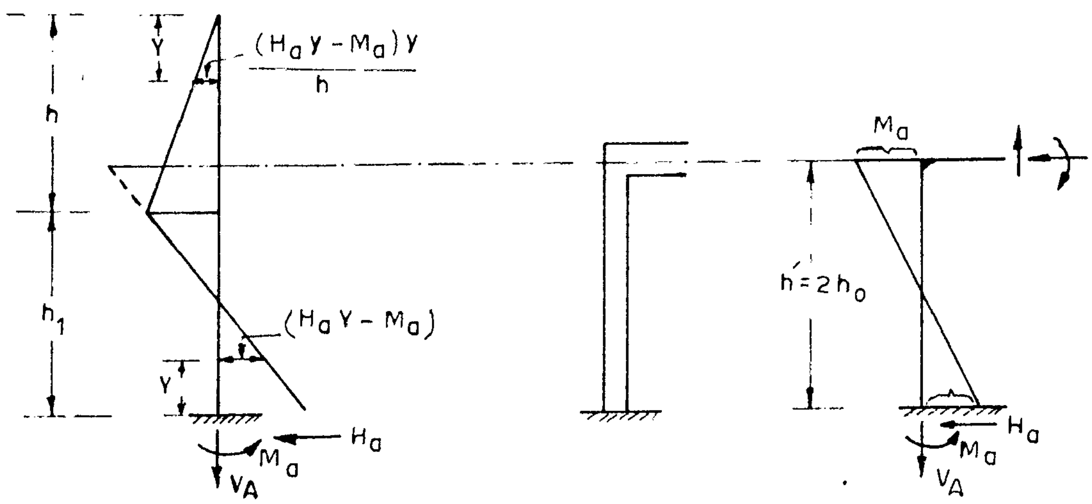

FIG. 3.3 _ EQUIVALENT FRAME

moment diagram for the bent is modified as shown by the dotted line in Fig. 3.3. This modification is made such that the shearing force produces the same strain energy for the columns of substitute frame as that for the original bent. The columns have uniform cross section along their heights. The strain energy due to flexure in bent columns is given by

$$U = \int_0^{h_1} \frac{(H_a y - M_a)^2 \, dy}{2\,EI} + \int_0^{h} \frac{(H_a h_1 - M_a)^2 \left(\frac{y}{h}\right)^2}{2EI} dy. \qquad ..(3.1)$$

Case I.

If the slope at the base of piers is zero

$$\theta_a = \frac{\partial U}{\partial M_a} = 0$$

or,

$$\theta_a = \int_0^{h_1} \frac{(H_a \; y - M_a)\;(-1)\;dy}{2\;EI} + \int_0^{h_1} \frac{(H_a \; h_1 - M_a)\left(\frac{y}{h}\right)^2(-1)}{2\;EI} dy$$

Since EI = constant

$$\left(-\frac{H_a \; h^2{}_1}{2} + M_a \; h_1\right) - \frac{H_a h_1 - M_a}{h^2} \cdot \frac{h^3}{3} = 0$$

$$M_a \left(h_1 + \frac{h}{3}\right) = \frac{H_a \; h^2{}_1}{2} + \frac{H_a \cdot h_1 \; h}{3}$$

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

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

$$= \frac{H_a \, h'_1}{2}$$

Therefore, for the corrosponding column in continuous frame, the height h'_1 is equal to

$$h'_1 = 2h_o = h_1 \left(\frac{1 + \frac{2}{3} \frac{h}{h_1}}{1 + \frac{1}{3} \frac{h}{h_1}} \right)$$

But

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

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

$$h'_1 = h_1 \left(1 + \frac{h}{h_1} \right)^{1/3} \qquad ..(3.2a)$$

Case II

If the column at based is not restrained i.e.

$M_a = 0$

$$\int_0^{h_1} \frac{(Hy)^2\,dy}{2\,EI} + \int_0^{h} \frac{\left(\frac{Hh_1}{h}\,y\right)^2}{2\,EI}\,dy = \int_0^{h_1'} \frac{(Hy)^2}{2\,EI}\,dy$$

Solving it we obtain

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

or $$h'_1 = h_1 \left(1 + \frac{h}{h_1} \right)^{1/3} \qquad ..(3.2b)$$

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.

$$P = H_a + H_b + H_c \qquad ..(3.3)$$

Now, if a horizontal force is applied at top of spandral, then all the piers will move by the same amount unless any crack or failure occurs in the system. As long as the system is in elastic range, the distribution of force in each pier will be proportional to their stiffness ($\frac{1}{\epsilon}$) against deflection. The deflection is due to bending and shear and so.

$$\epsilon = P' \left[\frac{h'^3}{12EI} + \frac{1.2\ h'}{G\ A} \right] \qquad ..(3.4)$$

where $G = 0.5\ E$.

$$\therefore \quad \epsilon = \frac{P'}{12E} \left[\frac{h'^3}{I} + \frac{28.8\ h'}{A} \right] \qquad ..(3.5)$$

in which,

ϵ = Deflection at top of piers

E = Modulus of elasticity in compression

G = Modulus of rigidity

I = Moment of inertia.

A = Cross-sectional area of pier

h' = Equivalent height of piers

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

$$H_i = \frac{\frac{1}{\epsilon_i}}{\sum \left(\frac{1}{\epsilon_i} \right)} \cdot P \qquad ..(3.6)$$

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

$$M_a = H_a \cdot \frac{h_a}{2} \qquad ..(3.7)$$

Besides this, piers will have axial forces due to vertical load and overturning moments. To calculate vertical reaction due to overturning moment, the following steps are involved.

Considering the spandral as a continuous beam, the applied couples are -

$$\left.\begin{aligned} M_A &= M_a + H_a (h + h_a) \\ M_B &= M_b + H_b (h + h_b) \\ M_C &= M_c + H_c (h + h_c) \end{aligned}\right] \qquad ..(3.8)$$

Here M_a, M_b, M_c are all -ve but M_A , M_B and M_C are all + ve.

Hence, the vertical reactions are -

$$V_A \downarrow \quad = \frac{\frac{5}{4} M_A + \frac{1}{2} M_B - \frac{1}{4} M_c}{\frac{1}{2} (L - L_1)}$$

$$= \frac{5M_A + 2 M_B - M}{2 (L - L_1)} \qquad ..(3.9)$$

$$V_C \uparrow \quad = \frac{\frac{1}{4} M_A + \frac{1}{2} M_B + \frac{5}{4} M_C}{\frac{1}{2} (L - L_1)}$$

$$= \frac{M_A + 2 M_B + 5M_C}{2 (L - L_1)} \qquad ..(3.10)$$

$$V_B \downarrow = \frac{-\frac{1}{4}M_A + \frac{1}{2}M_B + \frac{5}{4}M_C}{\frac{1}{2}(L-L_1)} - \frac{\frac{5}{4}M_A + \frac{1}{2}M_B - \frac{1}{4}M_C}{\frac{1}{2}(L-L_1)}$$

$$= \frac{3(M_C - M_A)}{(L - L_1)}$$

$$V_B \uparrow = \frac{3(M_A - M_C)}{(L - L_1)} \quad \text{..(3.11)}$$

And, if the reactions due to dead load be R_A, R_B and R_C, then

$$R_A = \frac{3}{16} W \uparrow = R_C$$

$$R_B = \frac{5}{8} W \uparrow \quad \text{..(3.12)}$$

Where W consists the weight of top spandral and a part of slab. Hence, total vertical force on each pier can be calculated by the algebraic sum of these reactions (i.e. due to vertical load and due to overturning moment.)

3.2. EXAMPLE - A SINGLE ROOM BUILDING

To have an idea about the stress condition, a single room single storey building is chosen. To simulate

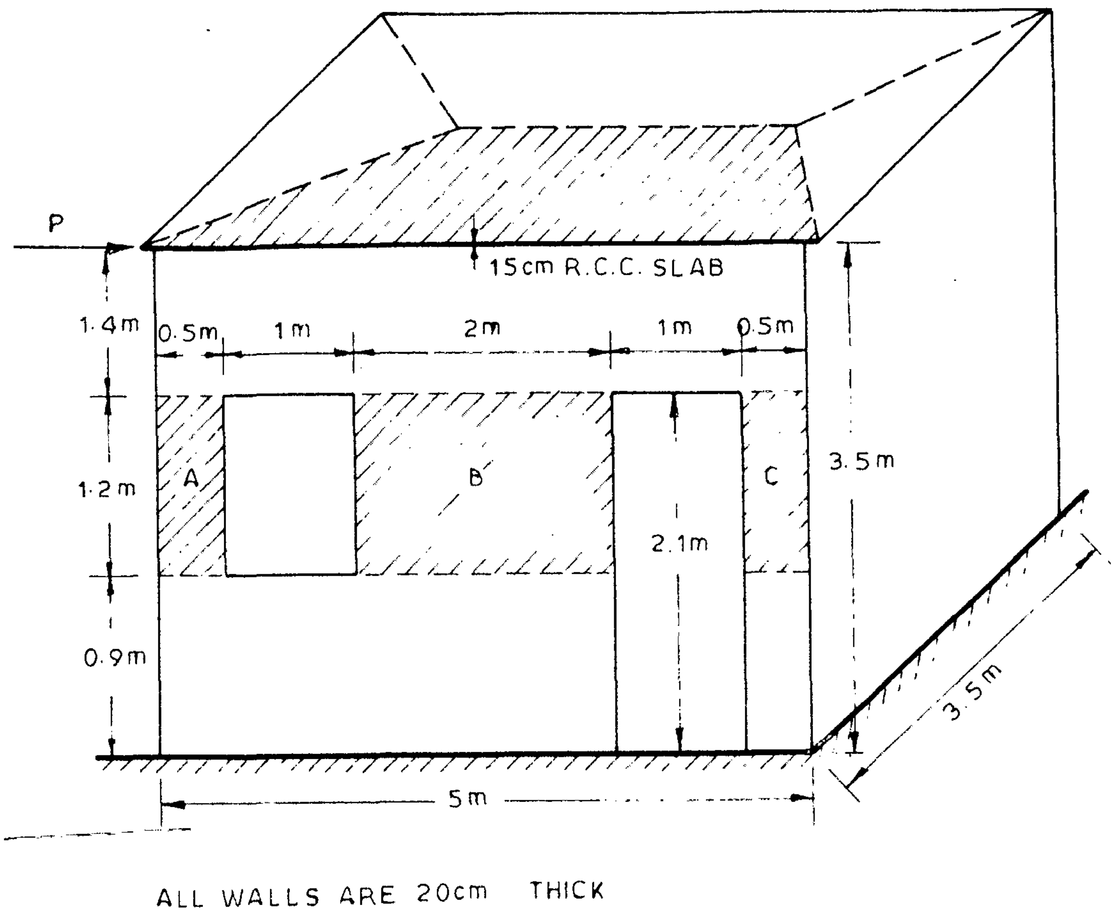

FIG. 3.4 _ WALL DIMENSIONS

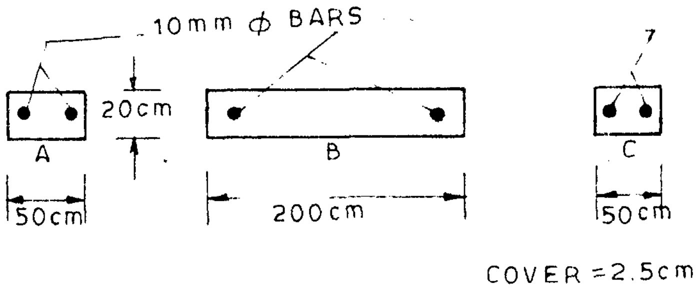

FIG. 3.5 _ REINFORCEMENT IN PIERS

the forces during an earthquake, the lateral force is applied in both directions separately.

The building adopted for analysis is shown in Fig. 3.4. The lateral force is varied from 4 percent to 20 percent g in increments of 2 percent g. To make an elastic analysis of the piers, a computer program was made which is given in the end of this thesis.

IS: 4326-1967 recommends 12 mm diameter bar for $1\frac{1}{2}$ brick thick walls for single storey building and for any other thickness of wall, the area of bar should be increased or decreased accordingly. In present case, since walls are 20 cm thick, hence a 10 mm dia. bar is provided on each face of pier.

The axial forces, moments and shears due to different lateral forces are tabulated in Table 3.1. Table 3.2 gives moments and vertical reaction including reaction due to vertical loads of spandral and slab; and when lateral force is applied from left hand side.

Calculation of lateral force and reaction due to vertical load:

$$\text{Wt. of slab} = 5 \times 3.5 \times 0.15 \times 2400$$

$$= 6300 \text{ kg.}$$

Wt. of front wall $= \left[5 \times 3.5 - 1.2 \times 1 - 1 \times 2.1\right] \times 0.2 \times 1900$

$= 5400$ Kg.

Wt. of side wall $= 3.5 \times 3.5 \times 0.2 \times 1900$

$= 4650$ Kg.

Wt. of all 4 walls $= 20100$ Kg.

$$P = \frac{\text{Accln.}}{2}\left[6300 + \frac{20100}{3}\right]$$

$$P = \text{Acceleration} \times 6500$$

Wt. of Spandral $= 1.4 \times 5 \times 0.2 \times 1900$

$= 2660$ Kg.

Wt. of part of slab $= 2050$ Kg.

Total load $= 4710$ Kg.

$$R_A = R_C = \frac{3}{16} W$$

$= 883$ Kg.

and, $R_B = \frac{5}{8} W$

$= 2944$ Kg.

TABLE 3.1

Accln.	Lateral Force (Kg.)	Pier A			Pier B			Pier C		
		H_A(Kg)	V_A(Kg)	M_A(Kg.M)	H_B(Kg)	V_B(Kg)	M_C(Kg.M)	H_C(Kg)	V_C(Kg)	M_C(Kg.M)
4% g	260	15.0	122.9	30	240.9	13.3	481.6	4.1	109.7	10.1
6% g	390	22.5	184.3	45	361.3	19.9	722.5	6.2	164.5	15.2
8% g	520	30.0	245.7	60	481.75	26.5	963.4	8.25	219.3	20.2
10%g	650	37.5	307.1	75	602.1	30.1	1204.3	10.35	274.1	25.3
12%g	780	45.0	368.6	90	722.6	39.8	1445.0	12.4	329.0	30.4
14%g	910	52.5	430.0	105	843.0	46.3	1685.9	14.35	383.7	35.5
16%g	1040	60.0	491.4	120	963.4	52.9	1926.8	16.5	438.5	40.6
18%g	1170	67.5	552.8	135	1083.8	59.5	2167.7	18.6	493.3	45.7
20%g	1300	75.0	614.2	150	1204.2	66.1	2408.6	20.7	548.1	50.8

TABLE 3.2

Acceln.	Pier A		Pier B		Pier C		REMARKS
	M_A(Kg.cm)	V_A(Kg)	M_B(Kg.cm)	V_B(K(Kg)	M_C(Kg.cm)	V_C(Kg)	
4% g	3000	760.10	48160	2957.3	1010	992.7	Pier A = 20cm x 50cm
6% g	4500	698.70	72250	2963.9	1520	1047.50	Pier B = 20cm x 200cm
8% g	6000	637.30	96340	2970.5	2020	1102.30	Pier C = 20cm x 50cm
10% g	7500	575.90	120430	2977.1	2530	1157.10	Cover = 2.5 cm
12% g	9000	514.40	144500	2983.8	3040	1212.0	
14% g	10500	453.00	168590	2990.3	3550	1266.7	$A_t = A_{sc} =$ 10mm bar = 0.785 cm^2
16% g	12000	391.60	192680	2996.9	4060	1321.5	
18% g	13500	330.20	216770	3003.5	4570	1376.30	Forced applied from left hand side.
20% g	15000	268.80	240860	3010.10	5080	1431.10	

TABLE 3.4

FORCE FROM LEFT SIDE

Force	Pier A Ecen. (cm)	Pier A N.A.	Pier A Stress in brick (Kg/cm2)	Pier A Stress in tensile steel	Pier B Ecen. (cm)	Pier B N.A.	Pier B Stress in brick (kg/cm2)	Pier B Stress in tensile steel	Pier C Ecen. (cm)	Pier C N.A.	Pier C Stress in brick (kg/cm2)	Pier C Stress in tensile steel (kg/cm2)
4%	4.4636	1.000	0.826	42.828	18.116	1.000	0.953	40.560	1.0174	1.000	0.876	85.403
6%	7.3685	1.000	0.886	22.657	27.111	1.000	1.110	21.421	1.5841	1.000	0.962	86.462
8%	10.9222	1.020	0.94911	+2.3280	36.062	1.000	1.272	2.2821	1.9915	1.000	1.048	87.614
10%	15.372	0.83195	1.0457	26.403	44.968	0.89378	1.4479	21.509	2.3664	1.000	1.140	88.674
12%	21.106	0.67538	1.1733	70.493	53.821	0.78342	1.6614	57.415	2.7046	1.000	1.225	89.744
14%	28.767	0.56150	1.3115	128.02	62.642	0.6856	1.9159	109.80	3.0117	1.000	1.312	90.793
16%	39.525	0.48208	1.4437	193.87	71.418	0.6041	2.2039	180.54	3.2914	1.000	1.401	91.853
18%	55.739	0.42483	1.5703	265.76	80.151	0.53848	2.5170	269.66	3.5473	1.000	1.492	92.913
20%	82.964	0.38259	1.6845	339.80	88.842	0.48719	2.8426	374.01	3.7822	1.000	1.58	93.973

TABLE - 3.5

FORCE FROM RIGHT SIDE

Force	Pier A				Pier B				Pier C				
	Ecen. (cm)	N.A.	Stress in brick (Kg/cm^2)	Stress in tensile steel (Kg./cm2)	Ecen. (cm)	N.A.	Stress in brick (Kg/cm^2)	Stress in tensile steel (Kg/cm^2)	Ecen. (cm)	N.A.	Stress in brick (Kg/cm^2)	Stress in tensile steel (Kg/cm^2)	
4%	3.2683	1.000	1.04	68.546	18.299	1.000	0.948	39.767	1.4738	1.000	0.683	62.447	
6%	4.5951	1.000	1.209	61.224	27.522	1.000	1.10	20.235	2.4107	1.000	0.676	52.039	
8%	5.7655	1.000	1.382	53.901	36.792	1.000	1.25	0.69058	3.5087	1.000	0.665	41.723	
10%	6.8051	1.000	1.55	46.579	46.106	0.8788	1.4367	24.767	4.8569	1.000	0.655	31.315	
12%	7.7346	1.000	1.723	39.267	55.465	0.76378	1.6553	63.993	6.5376	1.000	0.646	20.793	
14%	8.5714	1.000	1.895	31.944	64.874	0.66382	1.9152	121.23	8.6311	1.000	0.637	10.50	
16%	9.3283	1.000	2.06	24.622	74.336	0.5810	2.2116	199.36	11.388	0.99754	0.63	0.19367	
18%	10.016	1.000	2.23	17.3	83.84	0.51548	2.5327	297.57	15.147	0.83995	0.63984	15.239	
20%	10.644	1.000	2.41	9.9775	93.396	0.46444	2.8691	413.55	20.575	0.6866	0.66509	37.947	

TABLE 3.3

Accln.	Pier C		Pier B		Pier A		Remarks
	M_C (Kg.cm)	V_C (Kg)	M_B (Kg.cm)	V_B (Kg)	M_A (Kg.cm)	V_A (kg)	
4% g	1010	773.3	48160	2931.7	3000	1005.90	Pier A = 20cm x 50cm
6% g	1520	718.5	72250	2925.1	4500	1067.30	Pier B = 20cm x 200cm
8% g	2020	663.7	96340	2918.5	6000	1128.70	Pier C = 20cm x 50cm
10% g	2530	608.9	120430	2912.0	7500	1190.10	Cover = 2.5 cm
12% g	3040	553.0	144500	2905.2	9000	1251.6	A_t = A_{sc} = 10mm bar
14% g	3550	499.3	168590	2898.7	10500	1313.0	= 0.785 cm^2
16% g	4060	444.5	192680	2892.0	12000	1374.4	Force applied from
18% g	4570	399.7	216770	2885.5	13500	1435.8	right hand side.
20% g	5080	334.9	240860	2878.9	15000	1497.2	

Table 3.4 and 3.5 give the stresses in brick and steel and also the position of neutral axis in each case. These are shown graphically in Figs. 3.6 to 3.11.

A study of the above analysis shows that:

1. Upto a lateral load corrosponding to 8 percent of acceleration, all the piers are in compression.

2. Pier B attracts the largest force and gives rise to worst conditions of stresses when the lateral load is applied from right hand side.

3. Stresses in reinforcing steel and brickwork under worst conditions are well within the permissible range of stress even at a lateral load corrosponding to 20 percent g acceleration.

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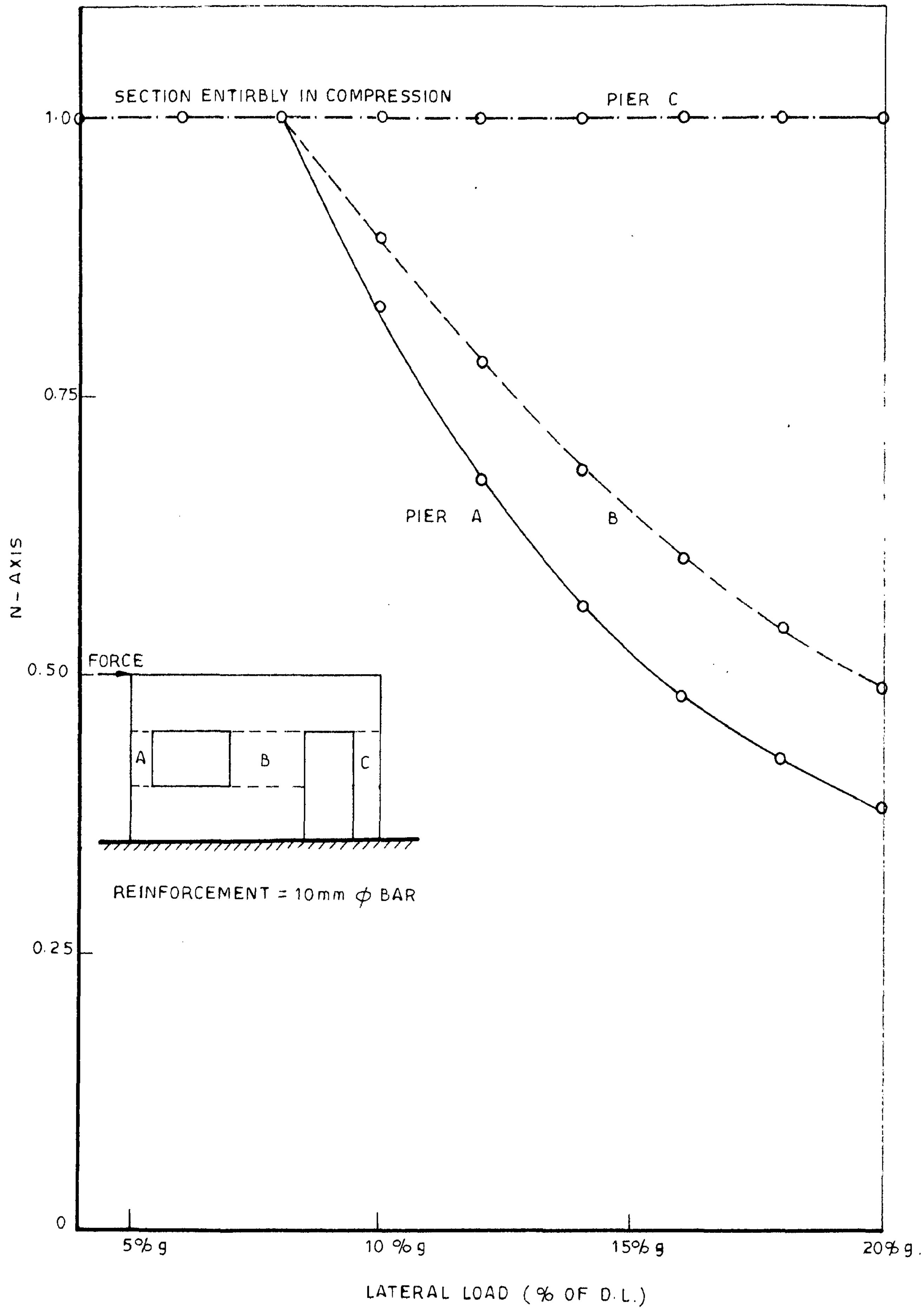


FIG. 3.6_VARIATION OF DEPTH OF N-AXIS WITH LATERAL FORCE

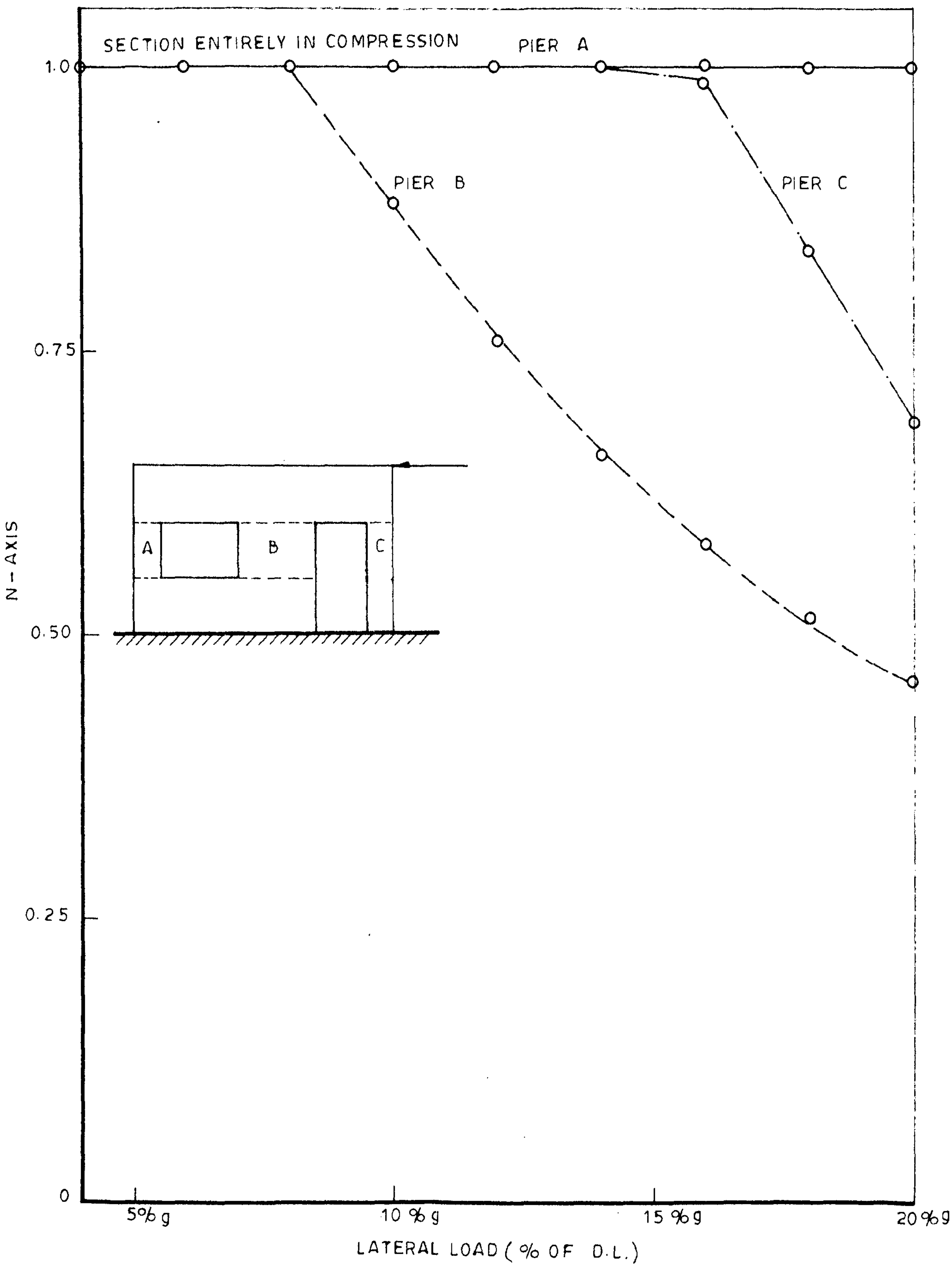


FIG. 3. 7 _ VARIATION OF DEPTH OF N AXIS WITH LATERAL FORCE

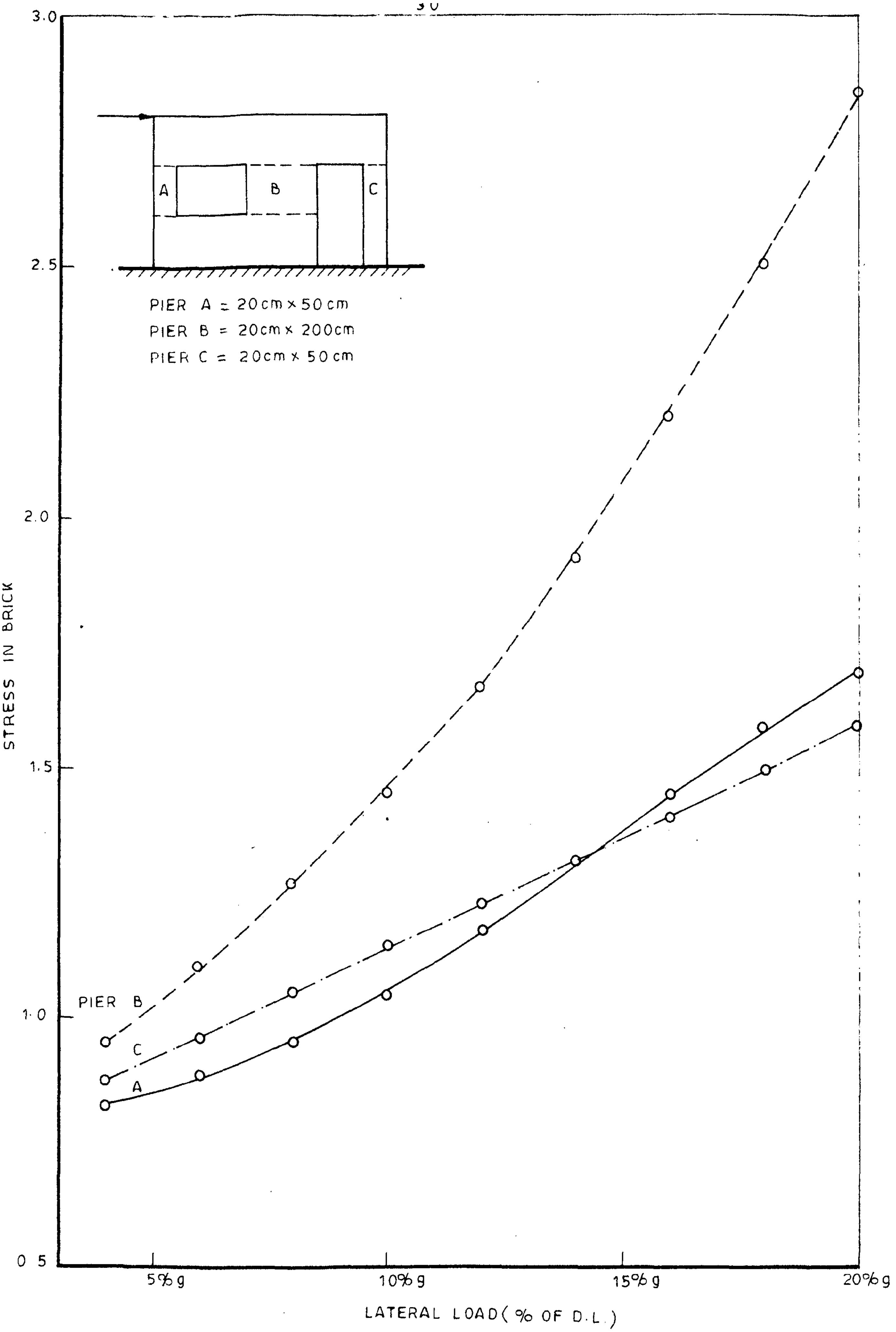


FIG. 3.8 _ VARIATION IN BRICK STRESS WITH LATERAL FORCE

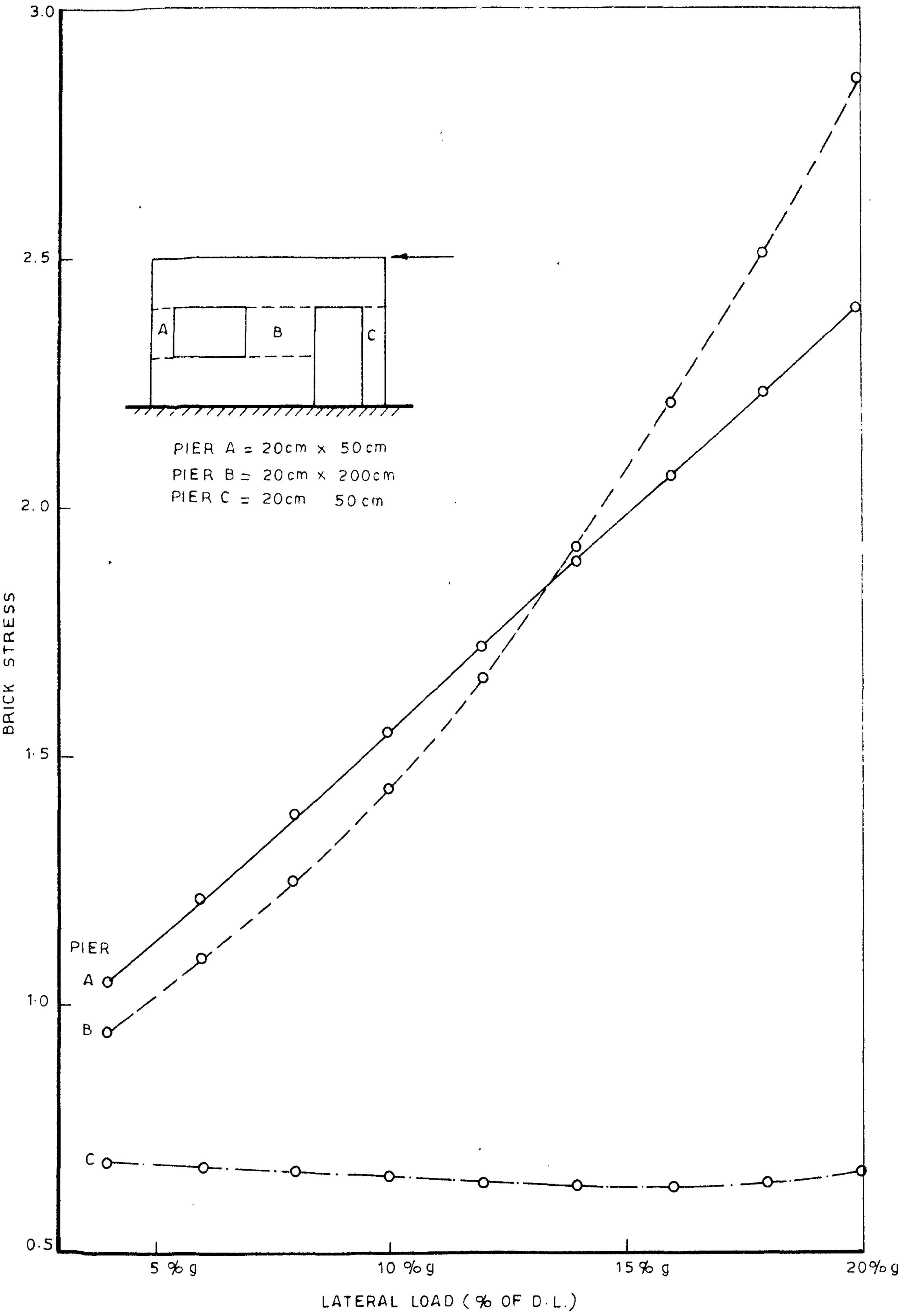


FIG. 3.9_VARIATION OF BRICK STRESS WITH LATERAL FORCE

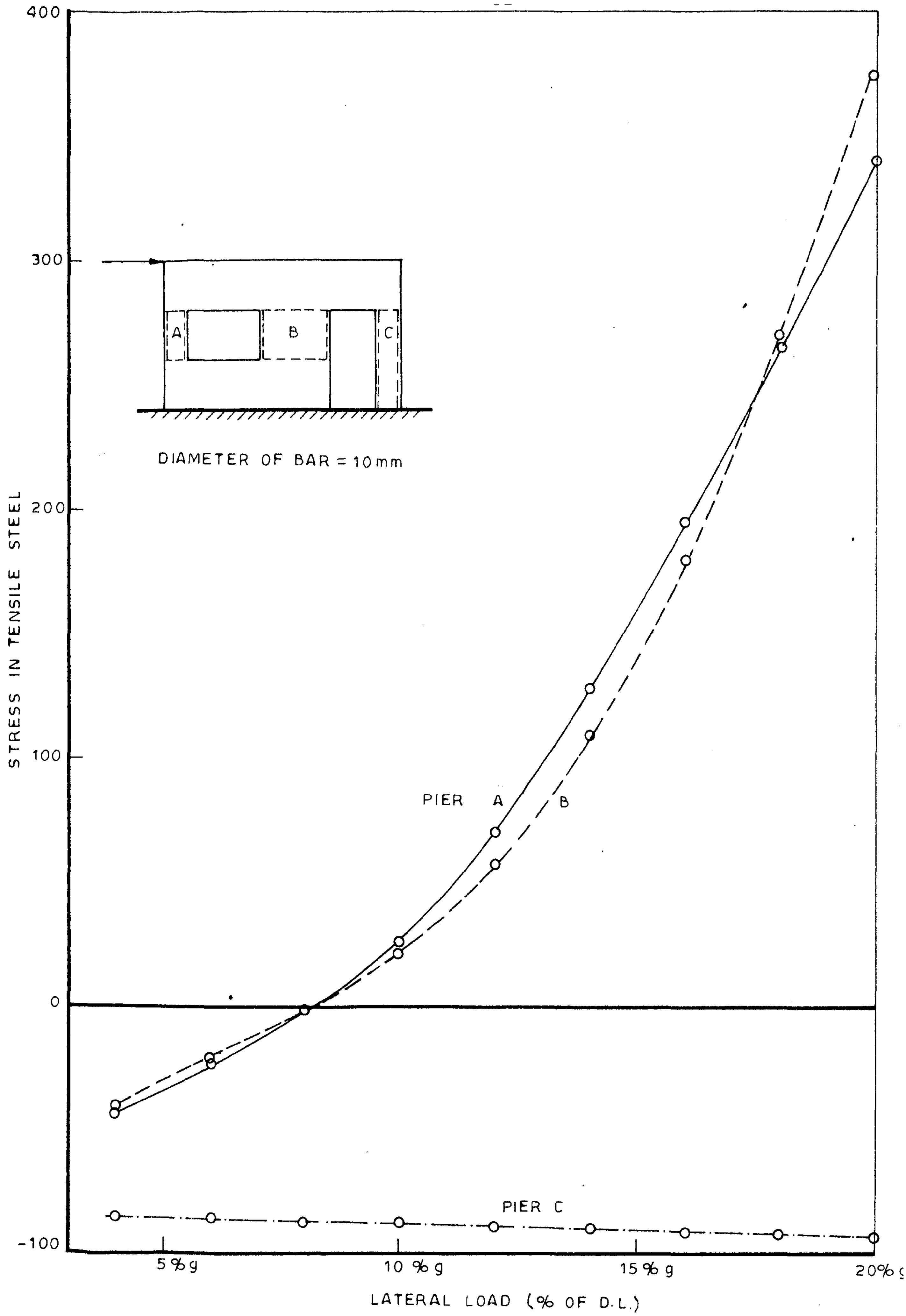


FIG. 3.10_VARIATION OF STRESS IN TENSILE STEEL WITH LATERAL FORCE

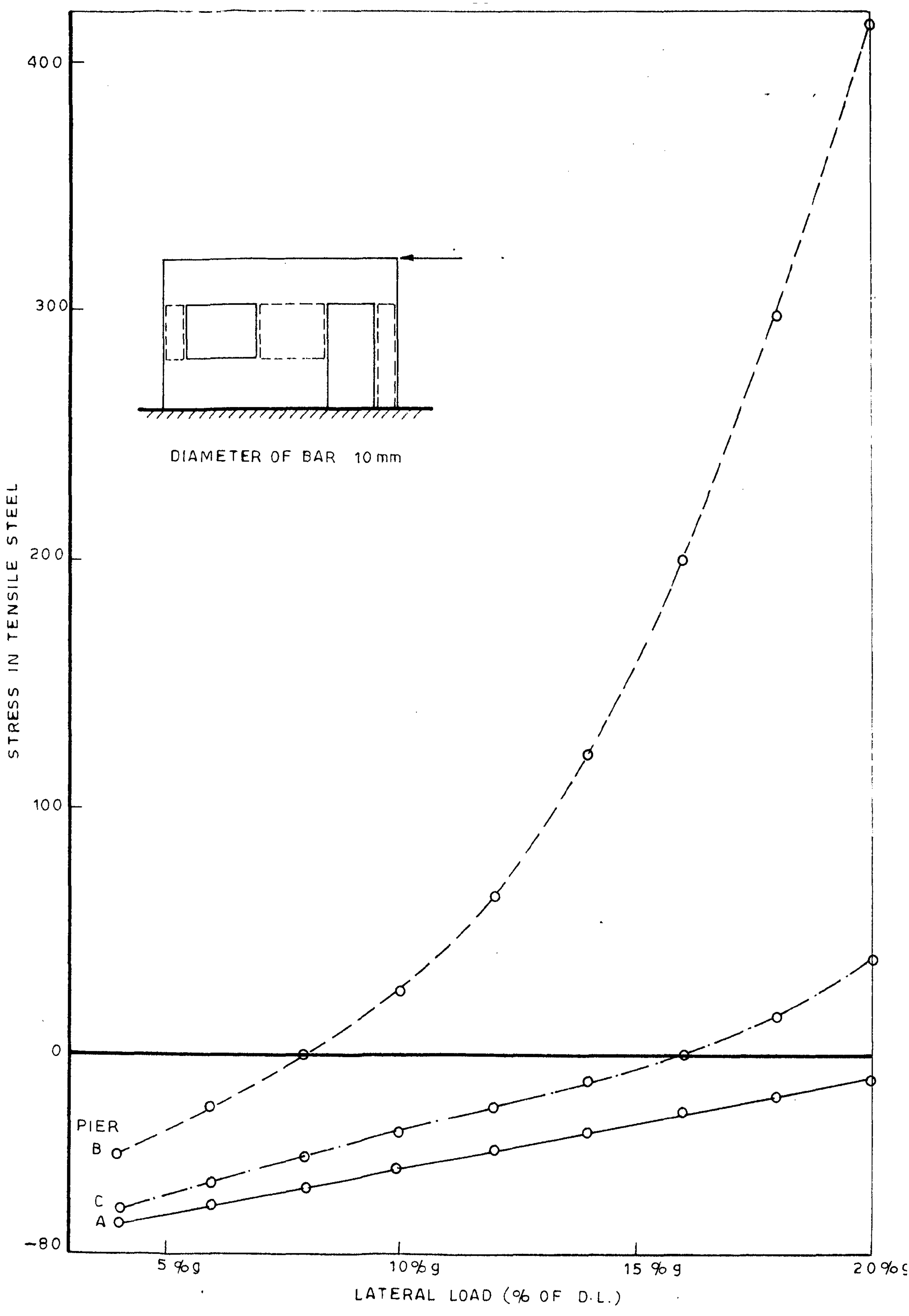


FIG.3.11_VARIATION OF STRESS IN TENSILE STEEL WITH LATERAL FORCE

CHAPTER IV

INELASTIC BEHAVIOUR OF PIERS

4.1. GENERAL

A knowledge of stress strain relationship is essential for the understanding of the dynamic behaviour of structures since it provides a link between the deformation and external forces. In the linear range, it is sufficient to know only the initial slope of the stress strain curve i.e. the modulus of elasticity. However, in order to understand and describe the response of the structure beyond the elastic limits, complete stress - strain curve must be known to us. This Chapter describes the behaviour of reinforced brick and reinforced concrete piers which form part of an earthquake resistant building.

4.2. REINFORCED BRICK SECTIONS

In present case, the stress-strain for reinforcing steel is assumed to be elasto-plastic (Fig. 4.2), and for brick, a linear relationship between stress and strain is adopted (Fig. 4.3). Also in a dynamic case, since any face can be a tension face, so equal amount of reinforcement has to be provided on both faces.

Fig. 4.1 shows the section chosen for the

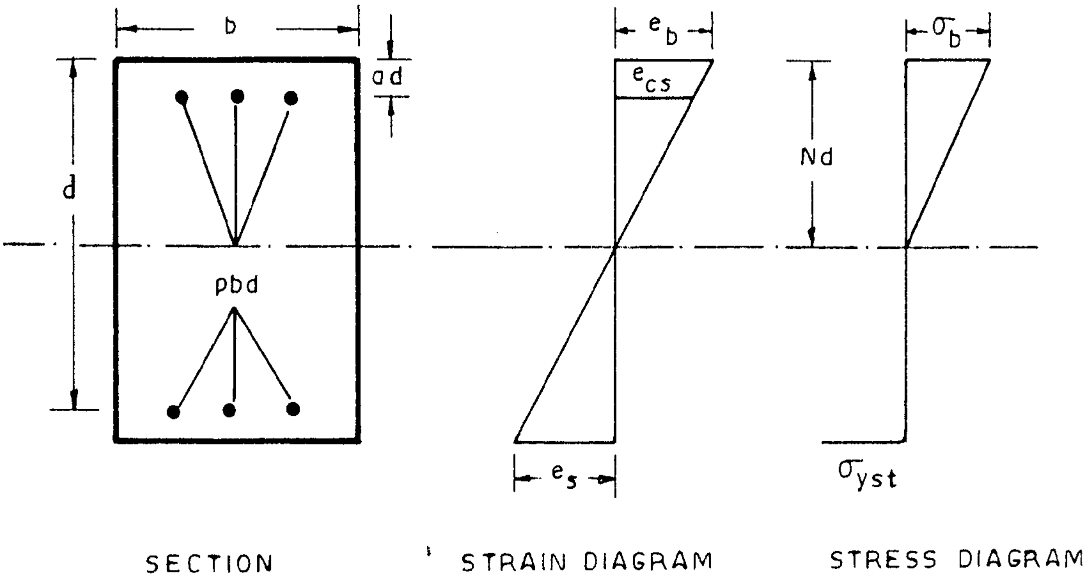
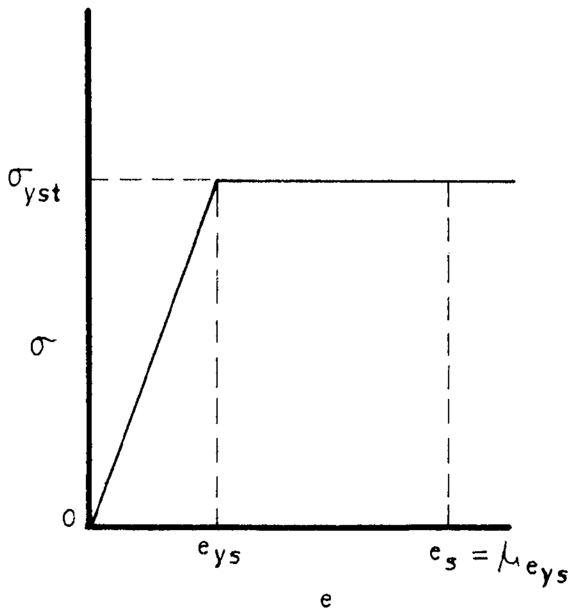
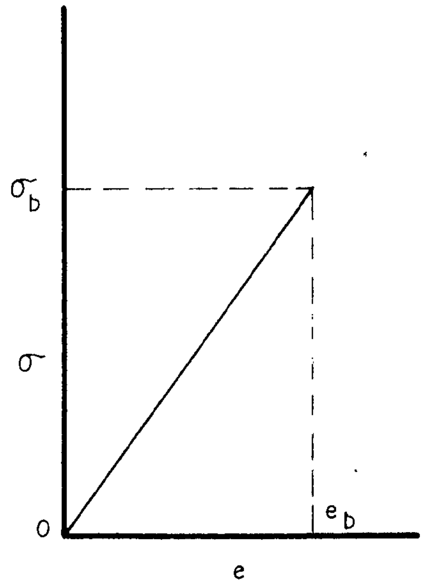


FIG. 4.1



STRESS STRAIN DIAGRAM FOR STEEL

FIG. 4.2



STRESS STRAIN DIAGRAM FOR BRICK

FIG. 4.3

purpose of study. Equal percentage of reinforcement is placed on each face at a cover 'ad'.

From the strain diagram, we get

$$\sigma_{sc} = m \; \sigma_b \; \frac{N - a}{N} \qquad ..(4.1)$$

in which,

σ_{sc} = Stress in compressive steel

m = Modular ratio $\frac{E_s}{E_b}$

σ_b = Stress in brick

N = Distance of N-axis from compression edge (fraction of d).

a = cover of steel (fraction of d)

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

$$N = \frac{1}{1 + \frac{\mu \, \sigma_{yst}}{m \; \sigma_b}} \qquad ..(4.2)$$

in which,

σ_{yst} = Yield stress in tensile steel

μ = ductility in steel.

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

$$\frac{1}{2} \; bNd \; \sigma_b + pbd \; \sigma_{sc} = pbd \; \sigma_{yst}$$

Rearranging

$$p = \frac{N \sigma_b}{2(\sigma_{yst} - \sigma_{sc})} \qquad ..(4.3)$$

From this equation it can be seen that the stress in compressive steel should always be less than the stress in tensile steel. In other words, the stress in compressive steel should not reach upto its yield limit. Hence from Equation 4.1 it can be argued that the value of N should not be less than a and the maximum value of N should be such that compressive steel does not reach its yield value.

Taking moment about tensile steel, the ultimate moment of resistance (M_{bu}) can be worked out as follows:

$$M_{bu} = \frac{1}{2} bNd \sigma_b \left(d - \frac{Nd}{3} \right) + m\, pbd\, \sigma_b \left(\frac{N-a}{N} \right) (d - ad) \qquad ..(4.4)$$

Rearranging

$$M_{bu} = \left[\frac{1}{2} b\, Nd^2 \left(1 - \frac{N}{3} \right) + m\, p\, b\, d^2 \left(\frac{N-a}{N} \right) (1 - a) \right] \sigma_b \qquad ..(4.5)$$

From above five equations a section can be designed for any desired ductility in steel and for maximum stress level in brickwork.

For illustration certain values of p, a, and σ_b are adopted and corresponding values of σ_{sc}, N, μ and M_{bu} are worked out. The results are tabulated in Table 4.1.

The assumed values are:

σ_b = 61 Kg/cm² (for 1:3 ratio)

σ_{yst} = 2600 Kg/cm².

m = 125

TABLE 4.1

a	p	N	μ	M_{bu}
0.1	1 percent	0.14	18	23.65 bd^2
	1.5 "	0.145	17.3	36.2 bd^2
	2.0 "	0.150	16.6	50.0 bd^2
0.15	1 "	0.2	11.7	21.9 bd^2
	1.5 "	0.205	11.3	31.9 bd^2
	2.0 "	0.215	10.7	45.2 bd^2
0.2	1 "	0.26	8.4	21.0 bd^2
	1.5 "	0.27	7.9	31.2 bd^2
	2.0	0.28	7.5	42.6 bd^2

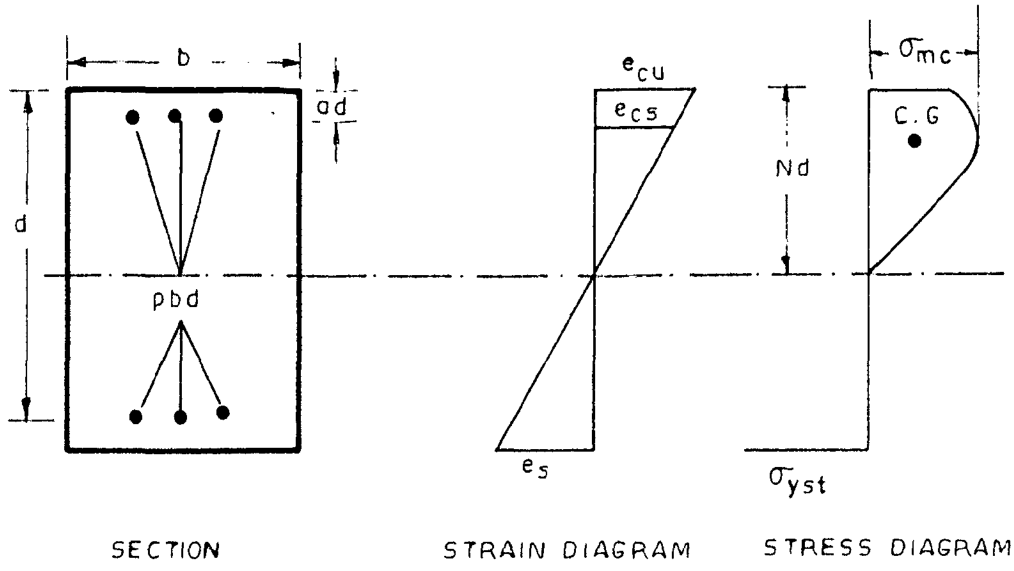

b
αd
d
pbd
e_{cu}
e_{cs}
e_s
Nd
σ_{mc}
C.G
σ_{yst}
SECTION
STRAIN DIAGRAM
STRESS DIAGRAM

FIG. 4.4

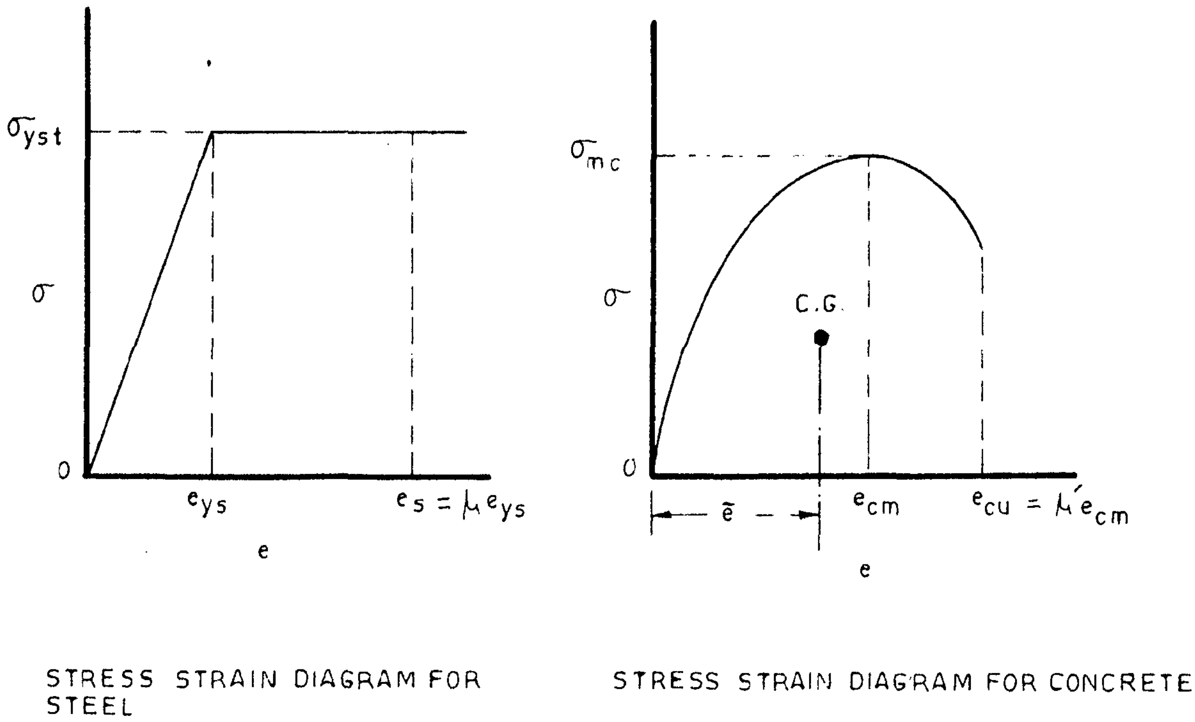

σ_{yst}
σ
0
e_{ys}
$e_s = \mu e_{ys}$
e
σ_{mc}
σ
C.G.
0
$\tilde{e}$
e_{cm}
$e_{cu} = \mu' e_{cm}$
e
STRESS STRAIN DIAGRAM FOR STEEL
STRESS STRAIN DIAGRAM FOR CONCRETE

FIG. 4.5

4.3. REINFORCED CONCRETE SECTIONS

Behaviour of concrete in terms of its stress strain diagram has been assumed parabolic by a number of investigators (10, 11, 12). It is proposed to use this form of stress - strain relationship in the present study also. The reinforced steel is assumed to exhibit elasto-plastic stress-strain curve as in earlier case.

The ductility in concrete μ' is defined as the ratio of ultimate strain in concrete to the strain corrosponding to maximum stress. Normally, the strain (e_{cm}) corrosponding to maximum stress level is about 0.2 percent and the ultimate strain ranges from 0.3 percent to 0.5 percent (7). So in concrete ductility varying from 1.5 to 2.5 can be expected.

A convenient form of the equation for the stress-strain curve for concrete could be as follows: (Fig. 4.5).

$$\sigma = \frac{2\sigma_{mc}}{e_{cm}} e - \frac{\sigma_{mc}}{e^2_{cm}} e^2 \qquad ..(4.6)$$

in which,

σ_{mc} = maximum stress in concrete.

e_{cm} = strain corresponding to maximum stress.

The distance of the centre of gravity ($\bar{e}$)

of the area from the origin can be worked out using the following integral expression:

$$\bar{e} = \frac{\int_0^{e_{cu}} e.\sigma.\, de}{\int_0^{e_{cu}} \sigma de} \quad ..(4.7)$$

using equations(4.6) and (4.7), one obtains

$$\bar{e} = \mu' e_{cm} \left[\frac{\frac{2}{3} - \frac{1}{4}\mu'}{1 - \frac{1}{3}\mu'} \right] \quad ..(48)$$

in which μ' denotes the ductility in concrete.
From Fig.(4.4), stress in compressive steel is

$$\sigma = \mu' \; E_s \; e_{cm} \; \frac{N - a}{N} \quad ..(4.9)$$

From equation (4.9), it is clear that this will be valid only if the value of N is greater than a. In other words, the minimum value of N is fixed from this consideration.

Again from Fig. (4.4)

$$\frac{e_{cu}}{e_s} = \frac{N}{1-N} \quad ..(4.10)$$

or,

$$\mu = \mu' \cdot \frac{e_{cm}}{e_{ys}} \left(\frac{1}{N} - 1 \right) \quad ..(4.11)$$

in which

e_{ys} = yield strain in tensile steel

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

$$F_c = \int_0^{Nd} b\ \sigma(x)\ dx \qquad ..(4.12)$$

or

$$F_c = bNd\ \mu'\ \sigma_{mc}\left[1 - \frac{\mu'}{3}\right] \qquad ..(4.13)$$

For equilibrium, the force of tension must be equal to force of compression. So equating them, one obtains

$$pbd\ \sigma_{yst} = pbd\ \sigma_{sc} + bNd\ \mu'\ \sigma_{mc}\left[1 - \frac{\mu'}{3}\right] \qquad ..(4.14)$$

Rearranging,

$$\sigma_{sc} = \sigma_{yst} - \frac{N\mu'\ \sigma_{mc}}{p}\left[1 - \frac{\mu'}{3}\right] \qquad ..(4.15)$$

In this equation, if the stress in compression steel becomes equal to yield stress, then the contribution of concrete would be nothing. So, the stress in compression steel should be such that the concrete is utilized upto the required ductility. Hence, again from Equation (4.9), the maximum value of N is also fixed. And any value of N between these limits will keep the stress in compression steel below the yield limit and will allow the concrete to reach its desired ductility.

The ultimate moment of resistance (M_{cu}) in such cases can be obtained as follows:

$$M_{cu} = \sigma_{yst}\, pbd^2 \left[1 - N + N.K.\right] + F_{cs} \cdot d \left[N - a N.K.\right] \qquad ..(4.15)$$

in which,

$$K = \left[\frac{0.666 - 0.25\,\mu'}{1 - 0.333\,\mu'}\right]$$

Hence, making use of above equations, a section in bending can be designed for the required ductilities in concrete and steel.

4.4. ILLUSTRATION

To illustrate the method certain values of p, a and μ' are adopted and corrosponding values of μ, N, σ_{sc} , F_{cs} and M_{cu} are worked out for two different grades. These values are tabulated in table 4.3 and 4.4. Table 4.2 gives the maximum and minimum values of N in terms of cover for different ductilities in concrete.

The values adopted are:

σ_{mc} = 150 (for M 150 Grade)

σ_{mc} = 250 Kg/cm^2 (for M 250 Grade)

σ_{yst} = 2600 Kg/cm^2

e_{cm} = 0.2 percent

E_s = 2.1×10^6 Kg/cm^2.

TABLE 4.2

μ'	N_{min}	$N_{max.}$
1.0	a	2.63 a
1.2	a	2.1 a
1.5	a	1.71 a

TABLE 4.3

GRADE: M150

a = 0.10

p	μ'	N	μ	σ_{sc} (Kg/cm²)	F_{cs} x bd (Kg)	M_{cu} x bd² (Kg.cm)
1%	1.0	0.14	9.9	1200	12.0	24.10
	1.2	0.131	12.8	1190	11.9	24.20
	1.5	0.123	17.2	1170	11.7	24.30
1.5%	1.0	0.158	8.6	1540	23.10	35.70
	1.2	0.146	11.3	1590	23.80	35.80
	1.5	0.133	15.5	1560	23.40	35.90
2%	1.0	0.172	7.7	1760	35.2	47.34
	1.2	0.156	10.4	1810	36.2	47.46
	1.5	0.140	14.8	1820	36.25	47.50

Contd/...

Table -4.3(contd.)

a = 0.15

p	μ'	N	μ	σ_{sc} (Kg/cm²)	F_{cs} xbd (kg)	M_{cu} xbd² (kg. cm)
1%	1.0	0.184	7.15	775	7.75	23.55
	1.2	0.175	9.10	720	7.20	23.60
	1.5	0.169	11.9	708	7.08	23.65
1.5%	1.0	0.21	6.05	1200	18.0	34.60
	1.2	0.197	7.85	1200	18.00	34.70
	1.5	0.185	10.60	1190	17.80	34.80
2	1.0	0.23	5.40	1460	29.20	45.56
	1.2	0.213	7.15	1490	29.8	45.70
	1.5	0.197	9.80	1500	30.0	45.80

a = 0.20

p	μ'	N	μ	σ_{sc} (Kg/cm²)	F_{cs} xbd (kg)	M_{cu} xbd² (kg. cm)
1%	1.0	0.22	5.70	382	3.82	23.36
	1.2	0.213	7,10	308	3.08	23.45
	1.5	$<$ a	- not acceptable.			-

TABLE 4.3(Contd)

p	μ'	N	μ	σ_{sc} (Kg/cm^2)	F_{cs}xbd (Kg)	M_{cu}xbd^2 (Kg/cm)
1.5%	1.0	0.254	4.74	893	13.40	33.80
	1.2	0.242	6.05	875	13.10	33.90
	1.5	0.230	8.0	820	12.40	34.0
2.0%	1.0	0.28	4.15	1200	24.0	44.24
	1.2	0.262	5.45	1190	23.80	44.40
	1.5	0.246	7.40	1180	23.60	44.55

TABLE 4.4

GRADE: M250

a = 0.10						
p	μ'	N	μ	σ (Kg/cm²)	F_{cs} x bd (Kg)	M_{cu}xbd² (Kg. cm)
1.0%	1	0.117	12.5	640	6.40	24.45
	1.2	0.113	15.2	580	5.80	24.50
	1.5	0.109	19.8	515	5.15	24.52
1.5%	1	0.135	10.6	1090	16.35	36.0
	1.2	0.127	13.3	1070	16.0	36.2
	1.5	0.120	17.7	1050	15.75	36.4
2.0%	1	0.150	9.1	1440	28.80	47.6
	1.2	0.138	12.1	1390	27.80	48.0
	1.5	0.128	16.4	1370	27.40	48.10
2.5%	1	0.158	8.6	1540	38.5	59.50
	1.2	0.146	11.3	1590	39.8	59.60
	1.5	0.133	15.5	1560	39.0	59.70

contd/-

Table 4.4 (Contd.)

a = 0.15

p	μ'	N	μ	σ_{sc} (Kg/cm²)	F_{cs}xbd (Kg)	M_{cu}xbd² (Kg. cm)
1%	1	a	- not	acceptable		
1.5%	1	0.176	7.5	620	9.30	35.40
	1.2	0.169	9.45	566	8.50	35.60
	1.5	0.164	12.30	537	8.05	35.70
				a = 0.15		
2%	1	0.195	6.6	970	19.40	46.60
	1.2	0.184	8.55	930	18.60	46.70
	1.5	0.176	11.30	920	18.40	46.80
2.5%	1	0.21	6.05	1200	30.0	57.60
	1.2	0.197	7.85	1200	30.0	57.75
	1.5	0.185	10.60	1190	29.8	57.90
				a = 0.20		
1%	1	$<$ a	- not	acceptable -		
1.5	1	$<$ a	- not	acceptable -		

Table 4.4 (Contd.)

p	μ'	N	μ	σ_{sc} (Kg/cm²)	F_{cs}xbd (Kg.)	M_{cu}xbd² (Kg.cm)
2%	1	0.236	5.20	640	12.80	46.40
	1.2	0.226	6.50	580	11.60	46.60
	1.5	0.219	9.15	545	10.90	46.75
2.5%	1	0.254	4.70	894	21.10	56.70
	1.2	0.237	6.20	788	19.70	57.0
	1.5	0.225	8.35	700	17.50	57.30

Fig. 4.6 and 4.7 show the variation in steel ductility with cover. Fig. 4.8 to 4.23 show the variation of steel ductility and N -axis with concrete ductility and variation of moment of resistance with ductility in steel.

A study of these curves reveals that the ductility in tensile steel decreases with the increase in cover. Secondly, there is decrease in moment of resistance with the increase in cover. However, it is seen that by increasing the ductility in concrete, the ductility in steel increases.

These curves will be found useful in designing R.C. sections. With the help of these curves, for the desired ductility in steel and concrete, the value of N, p, cover a and M_{cu} can be directly obtained.

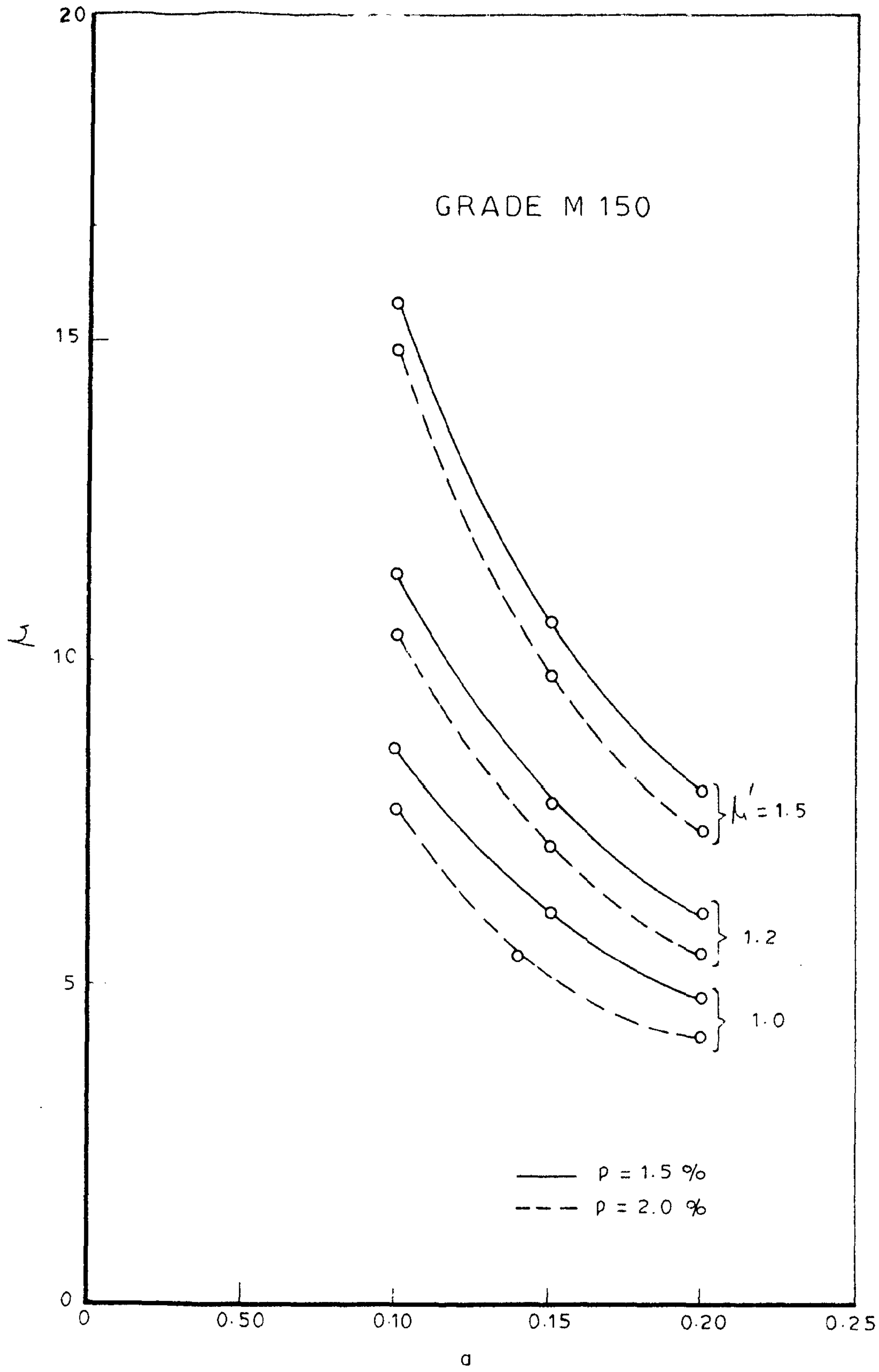


FIG. 4.6 VARIATION IN STEEL DUCTILITY WITH COVER

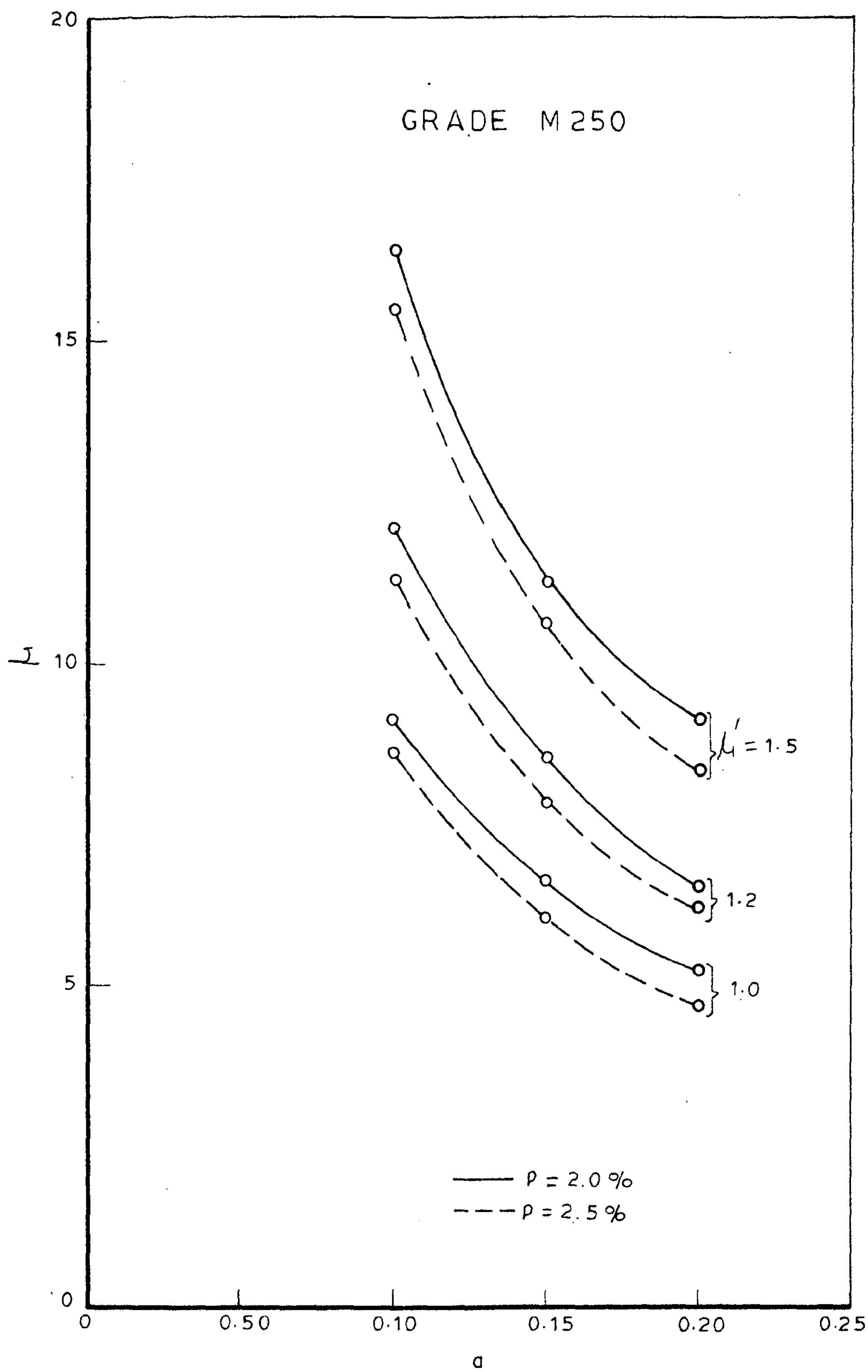
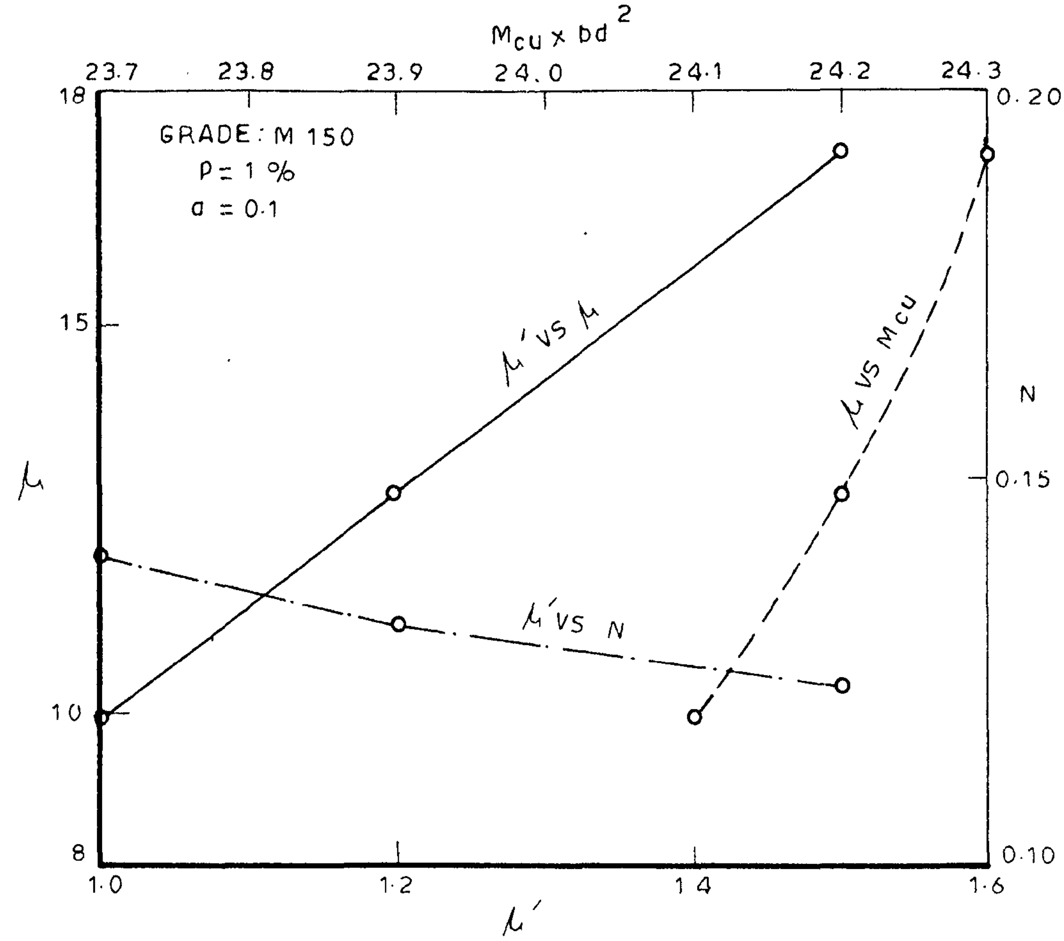
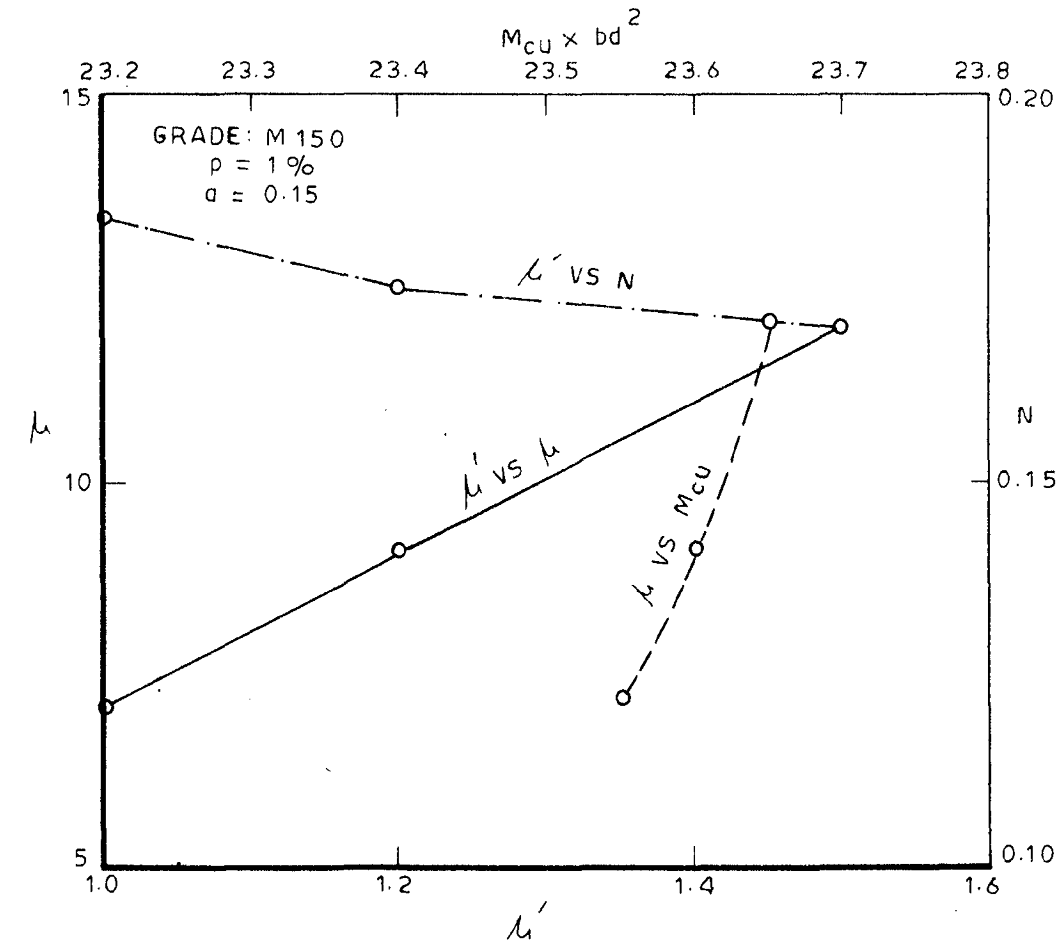


FIG. 4.7 _ VARIATION IN STEEL DUCTILITY WITH COVER



$M_{cu} \times bd^2$
23.7
23.8
23.9
24.0
24.1
24.2
24.3
18
15
10
8
0.20
0.15
0.10
GRADE: M 150
p = 1 %
a = 0.1
μ' vs μ
μ vs Mcu
μ' vs N
μ
N
1.0
1.2
1.4
1.6
μ'

FIG. 4.8



$M_{cu} \times bd^2$
23.2
23.3
23.4
23.5
23.6
23.7
23.8
15
10
5
0.20
0.15
0.10
GRADE: M 150
p = 1 %
a = 0.15
μ' vs N
μ' vs μ
μ vs Mcu
μ
N
1.0
1.2
1.4
1.6
μ'

FIG. 4.9

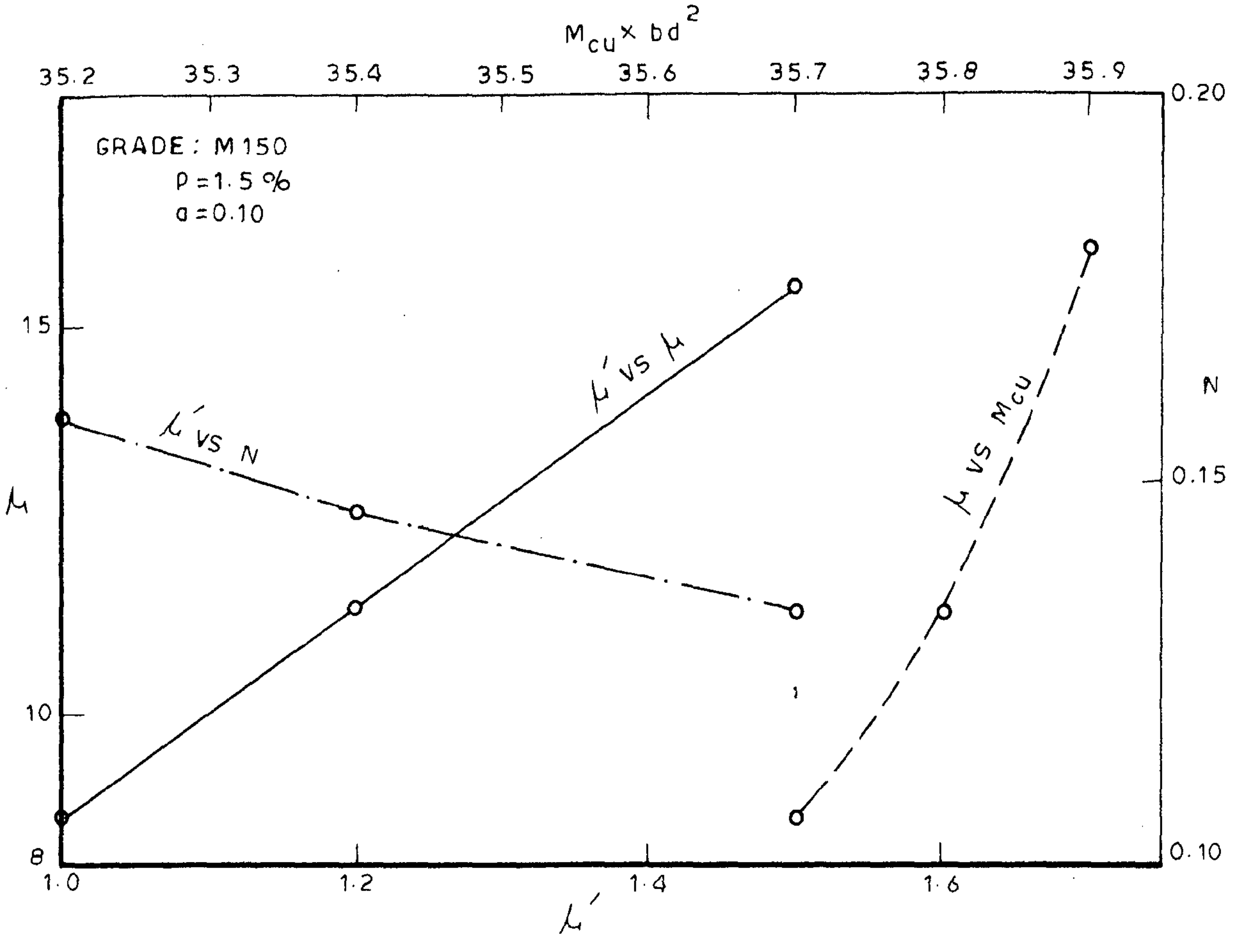


FIG. 4.10

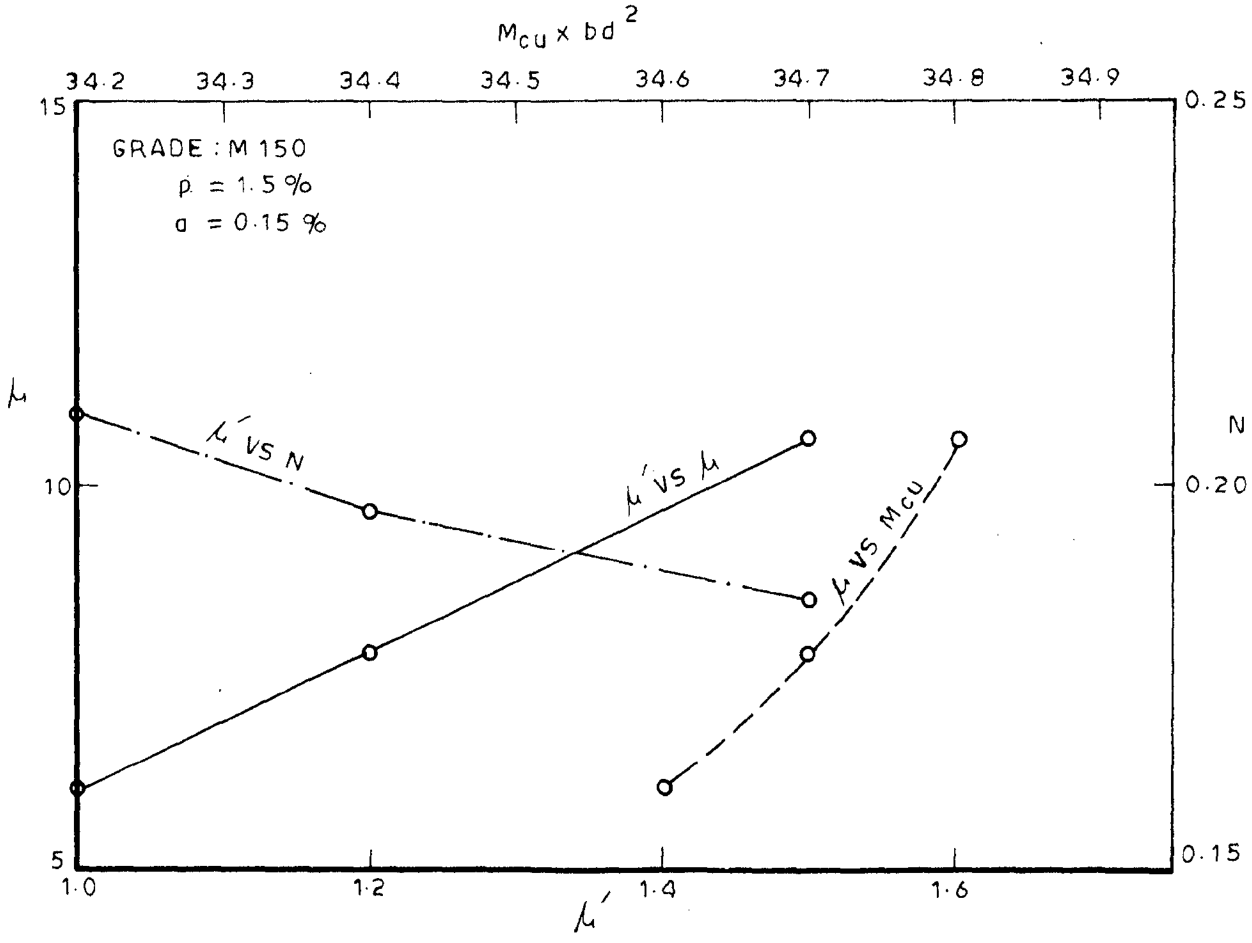
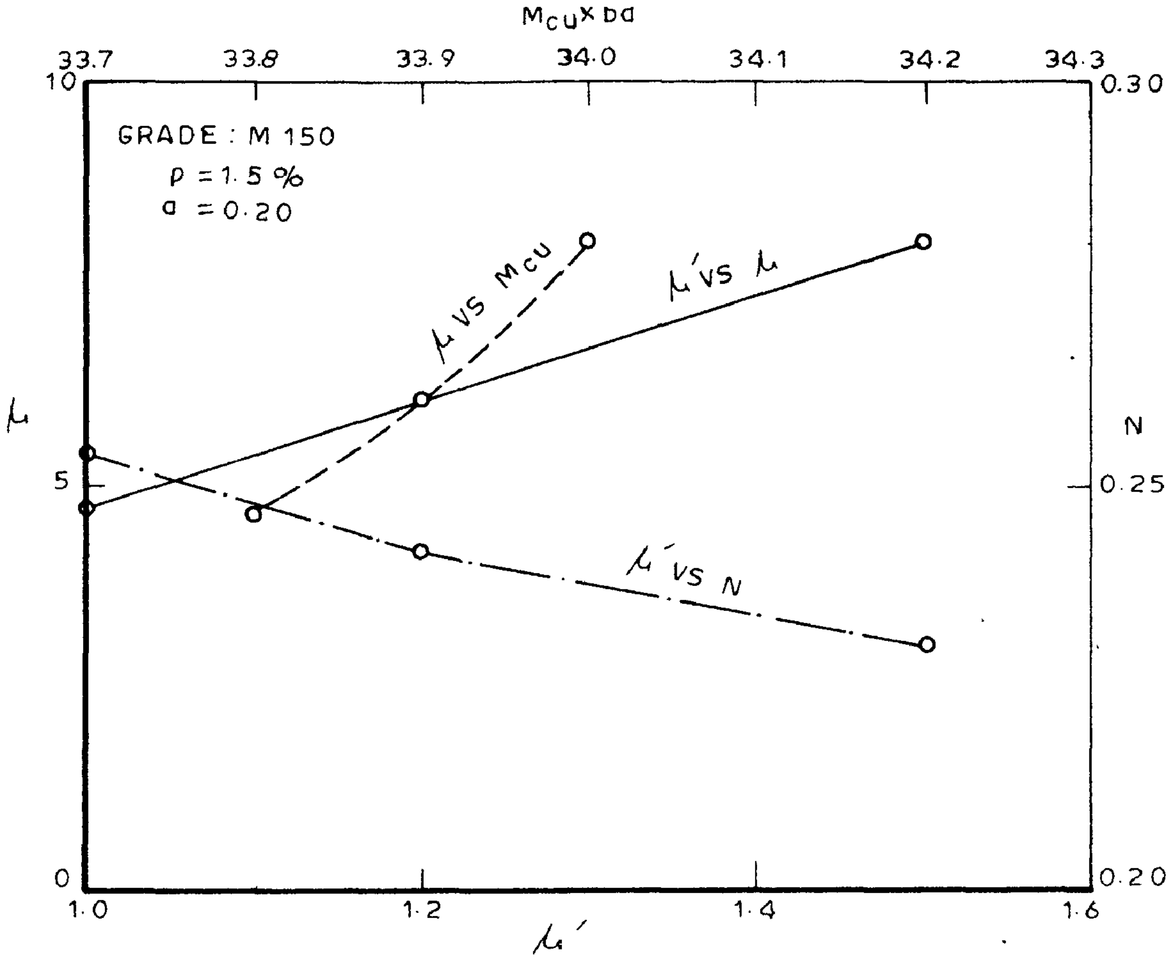
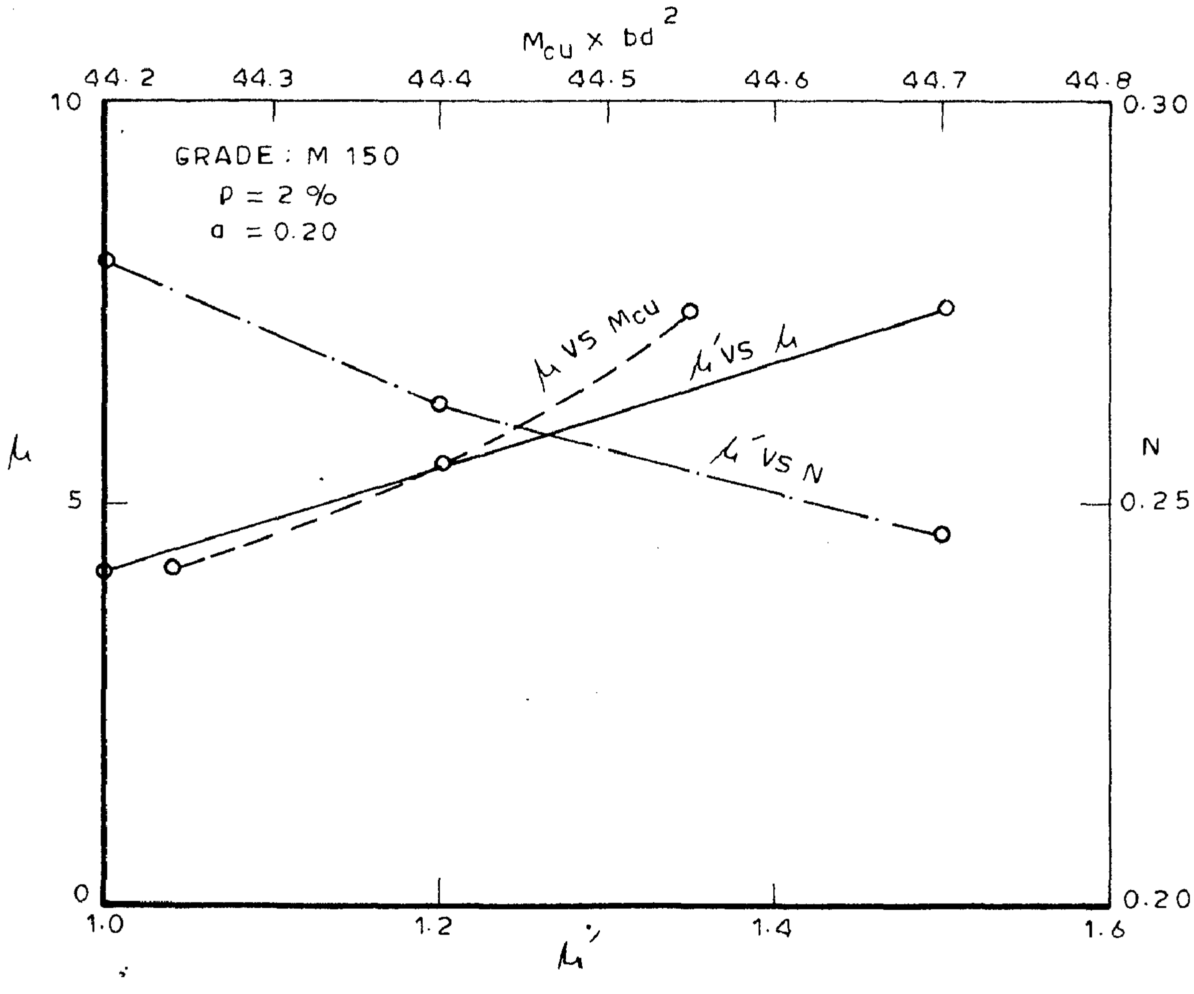


FIG. 4.11



$M_{cu} \times bd$
33.7
33.8
33.9
34.0
34.1
34.2
34.3
10
5
0
0.30
0.25
0.20
GRADE : M 150
p = 1.5 %
a = 0.20
μ vs Mcu
μ′ vs μ
μ′ vs N
μ
N
1.0
1.2
1.4
1.6
μ′

FIG. 4.12



$M_{cu} \times bd^2$
44.2
44.3
44.4
44.5
44.6
44.7
44.8
10
5
0
0.30
0.25
0.20
GRADE : M 150
p = 2 %
a = 0.20
μ vs Mcu
μ′ vs μ
μ′ vs N
μ
N
1.0
1.2
1.4
1.6
μ′

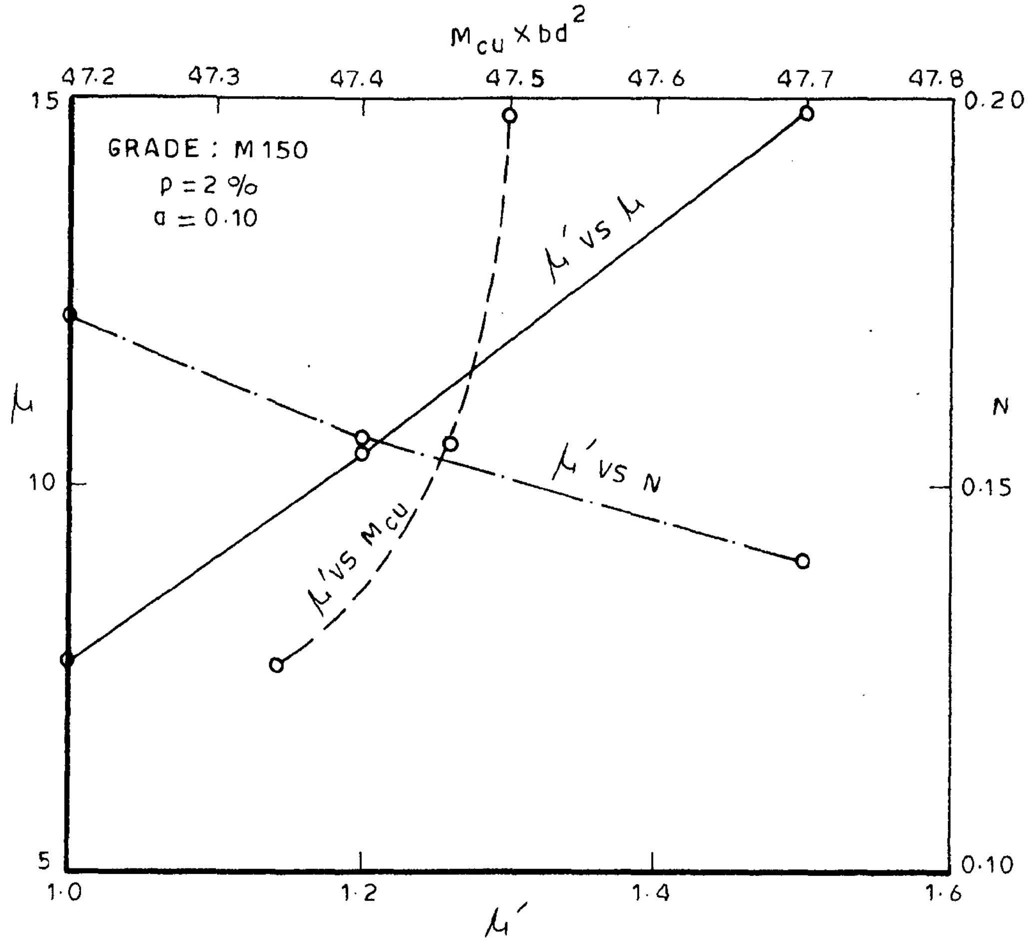


FIG. 4.14

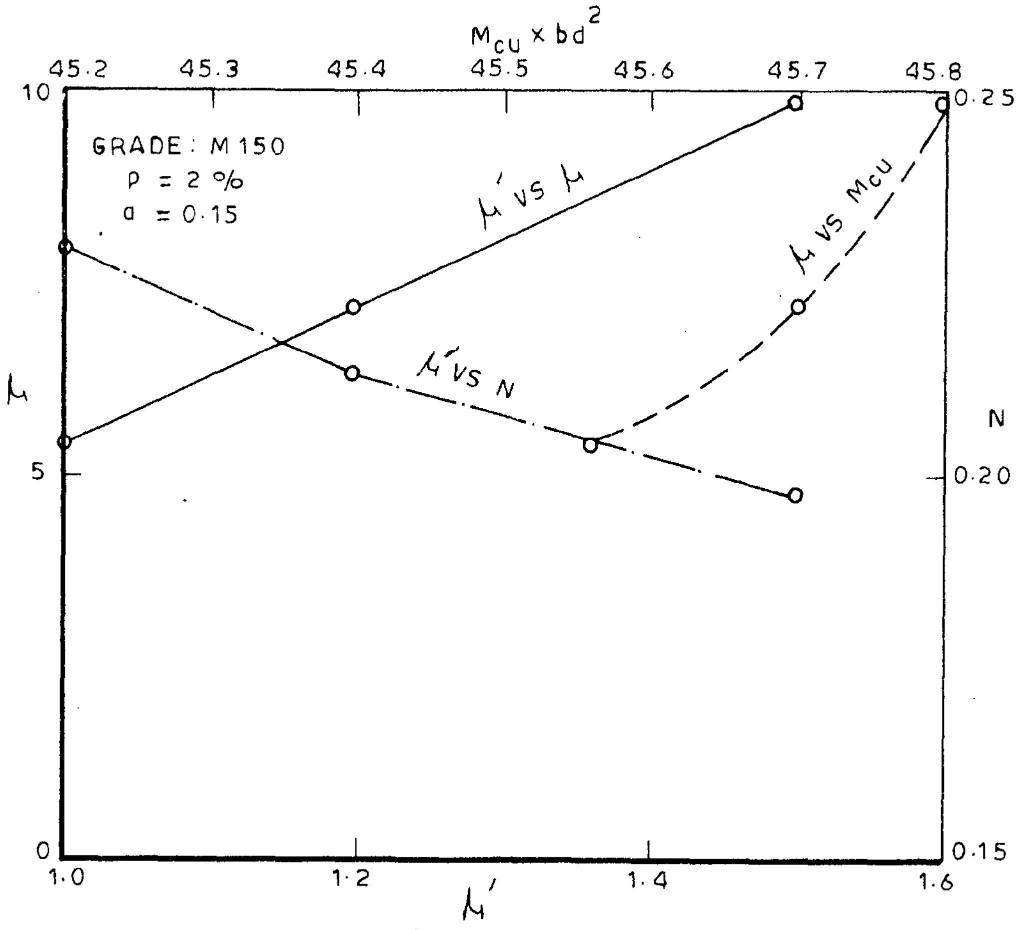


FIG. 4.15

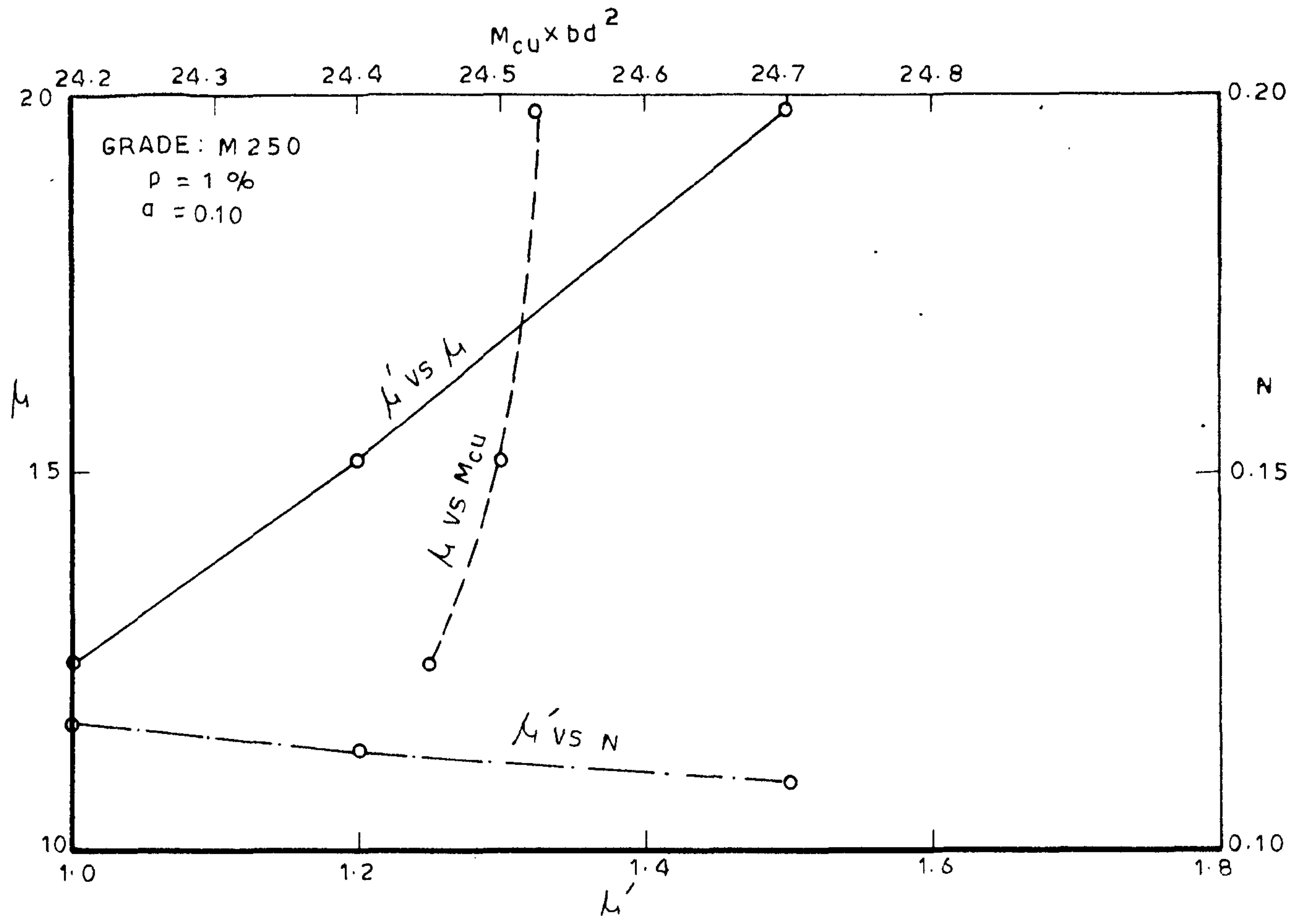


FIG. 4.16

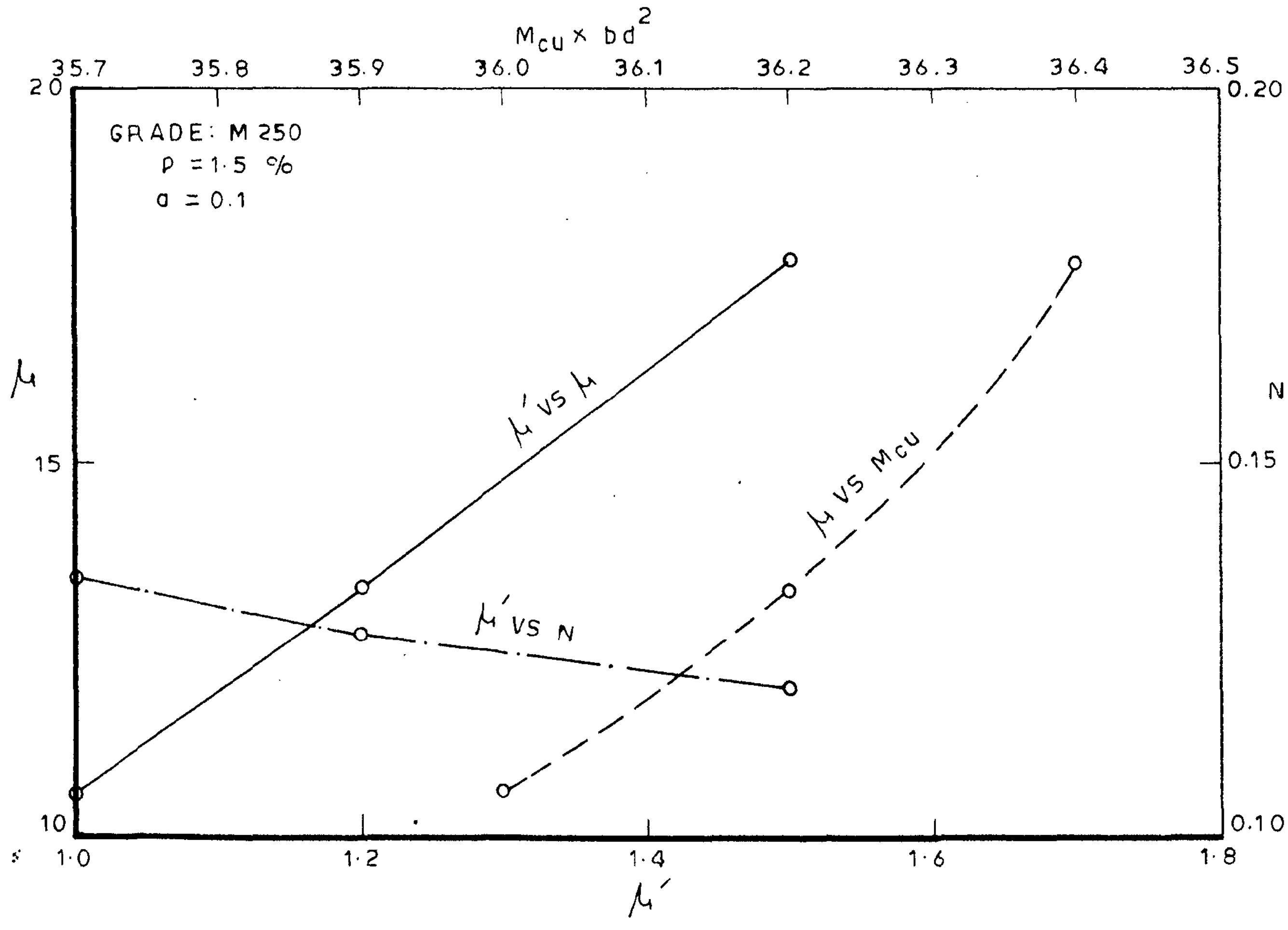


FIG. 4.17

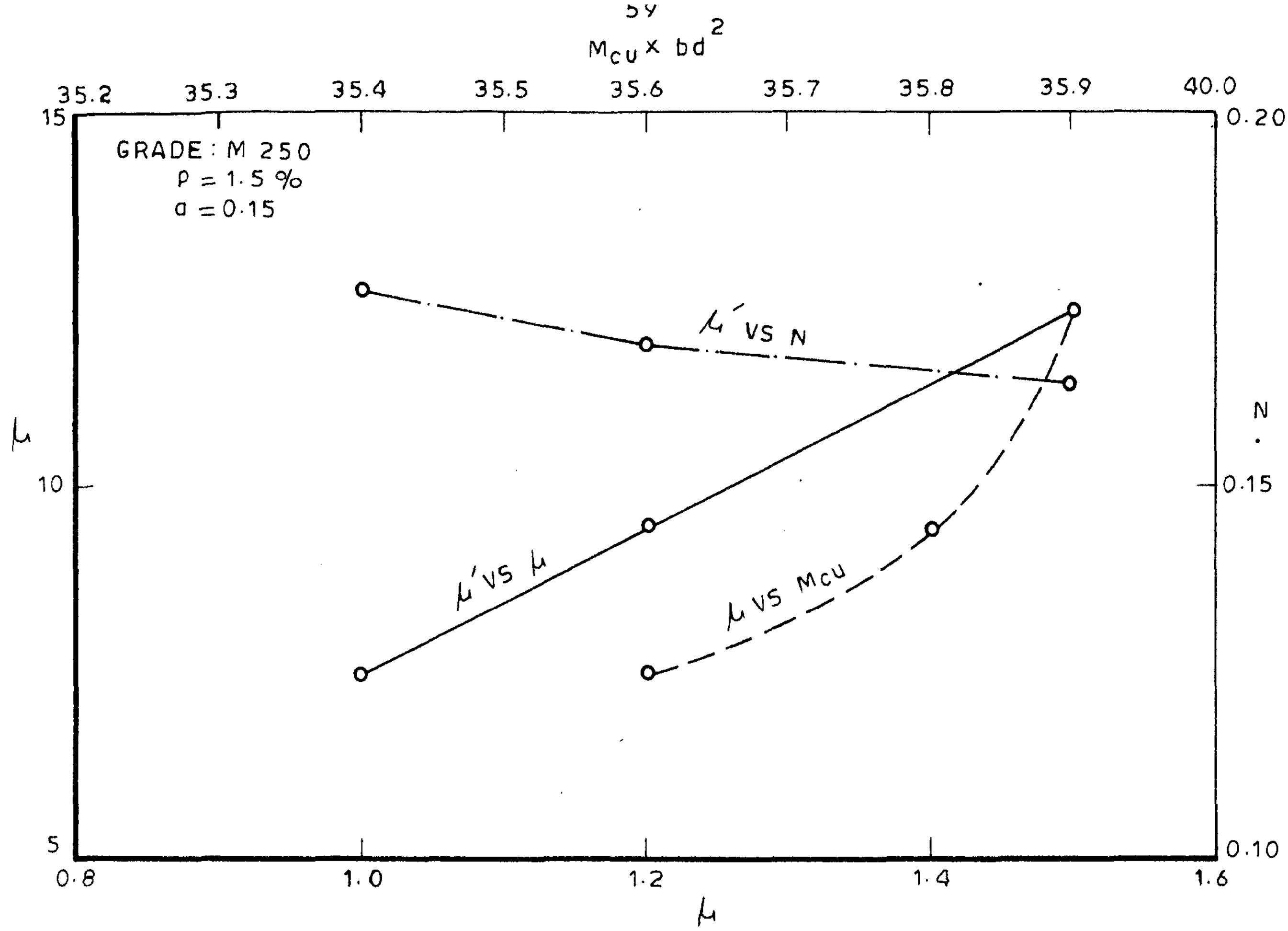
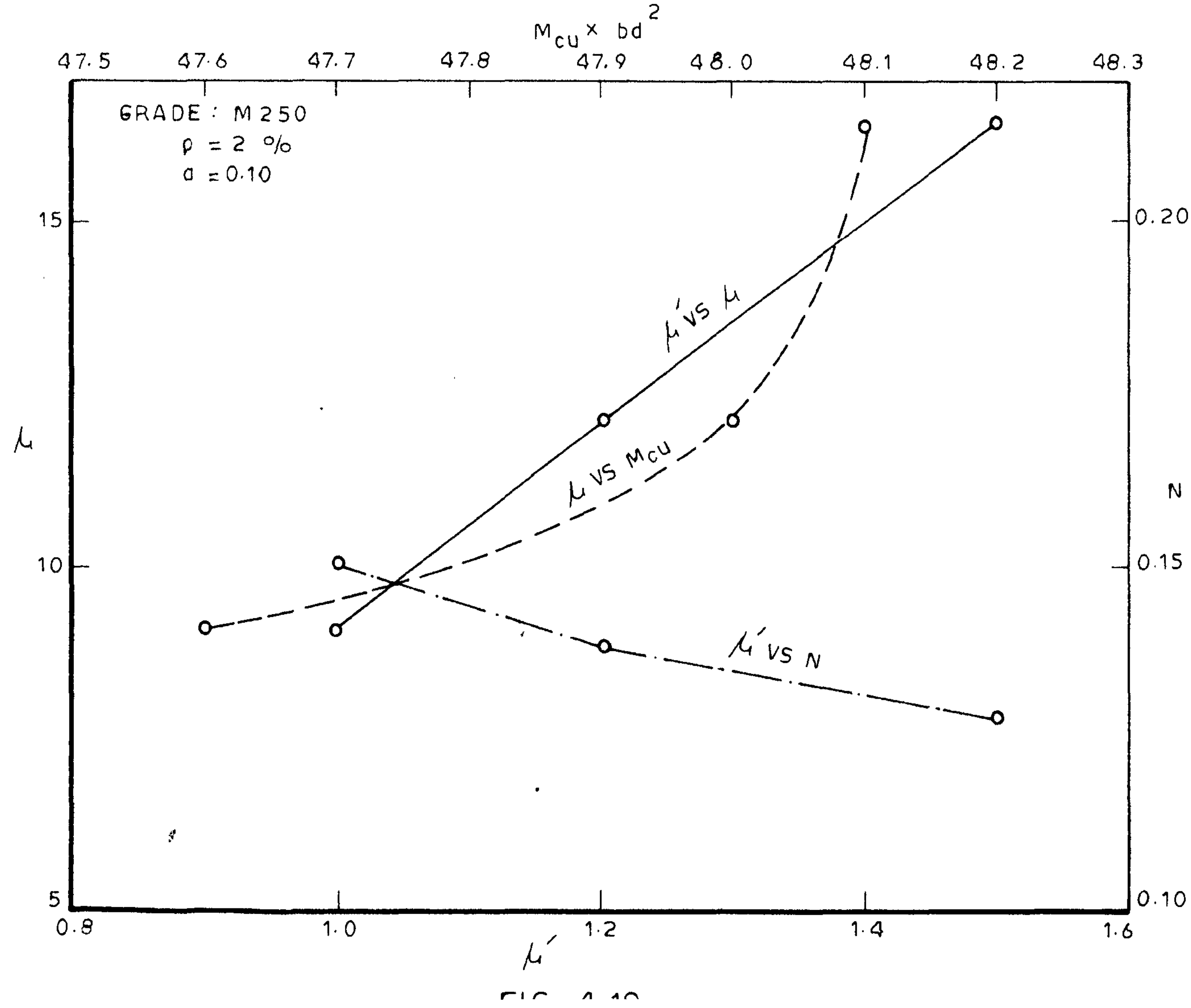

$M_{cu} \times bd^2$
35.2
35.3
35.4
35.5
35.6
35.7
35.8
35.9
40.0
GRADE : M 250
p = 1.5 %
a = 0.15
μ' vs N
μ' vs μ
μ vs Mcu
μ
N
15
10
5
0.20
0.15
0.10
0.8
1.0
1.2
1.4
1.6
μ

FIG. 4.18


$M_{cu} \times bd^2$
47.5
47.6
47.7
47.8
47.9
48.0
48.1
48.2
48.3
GRADE : M 250
p = 2 %
a = 0.10
μ' vs μ
μ vs Mcu
μ' vs N
μ
N
15
10
5
0.20
0.15
0.10
0.8
1.0
1.2
1.4
1.6
μ'

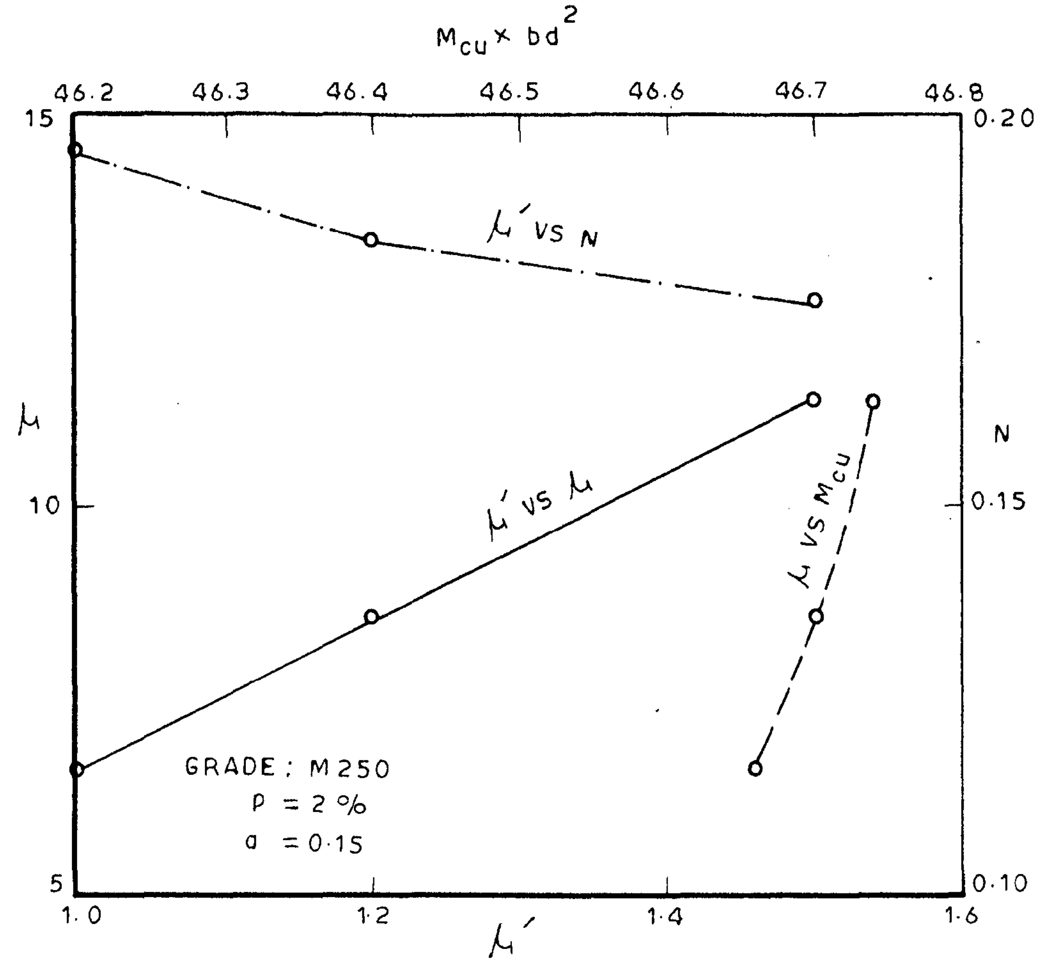


FIG. 4.20

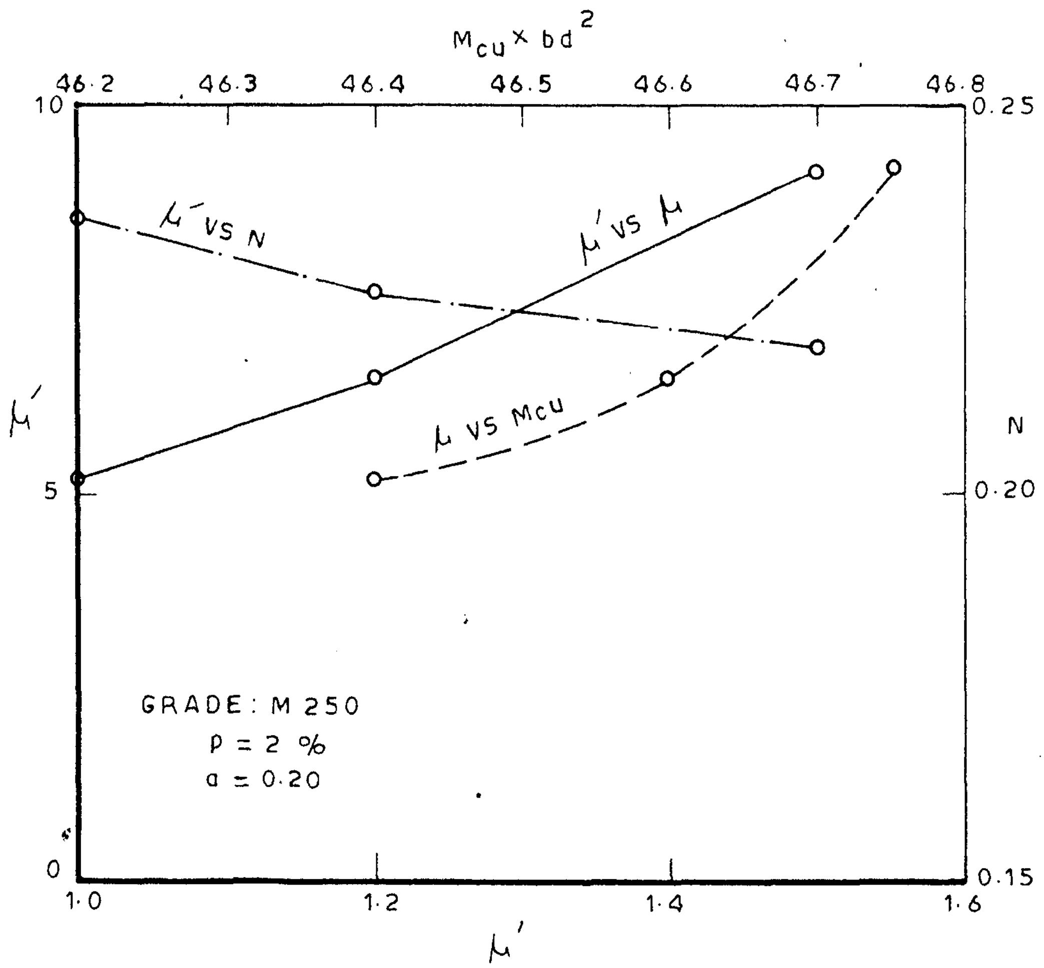


FIG. 4.21

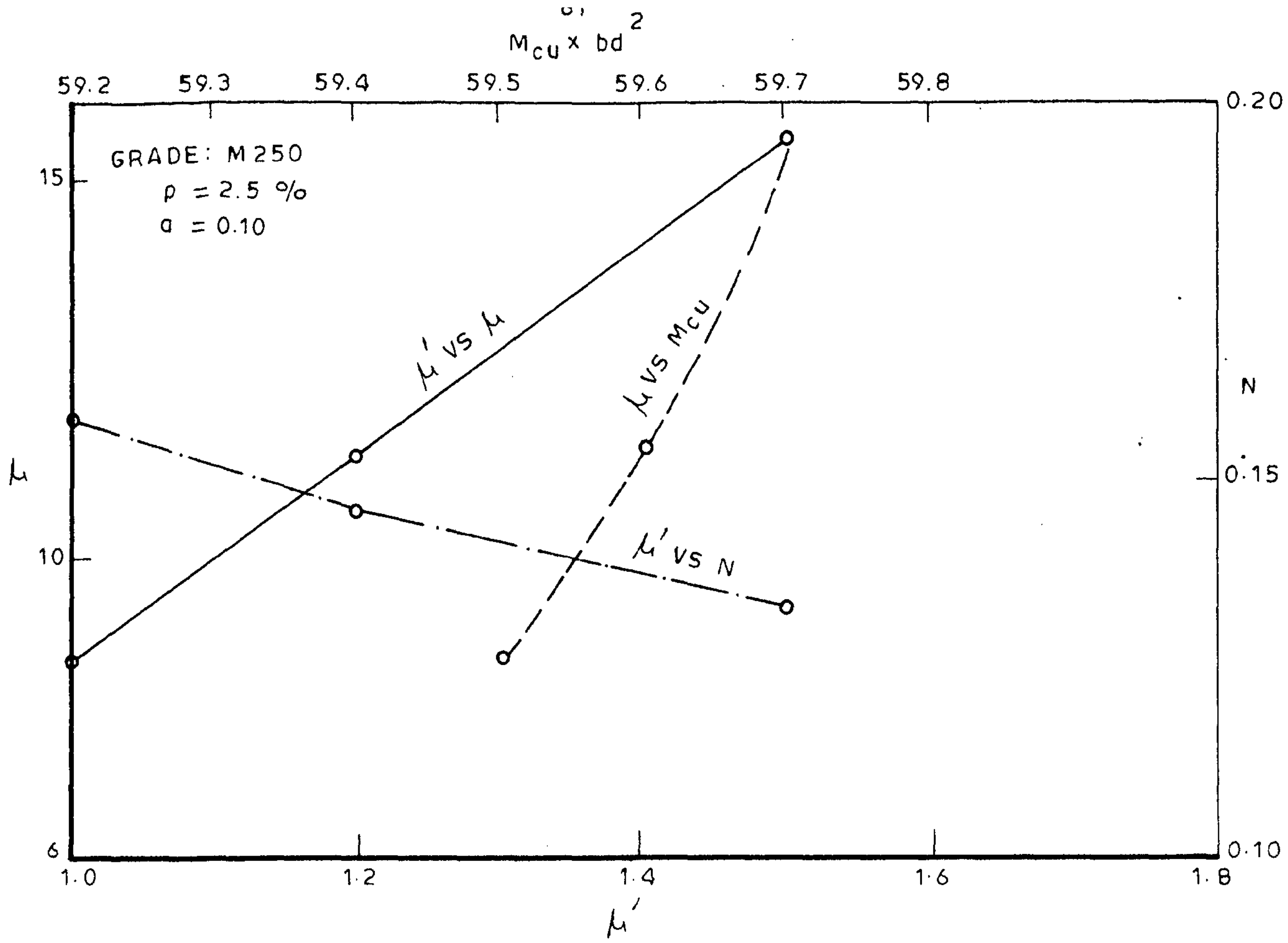


FIG. 4.22

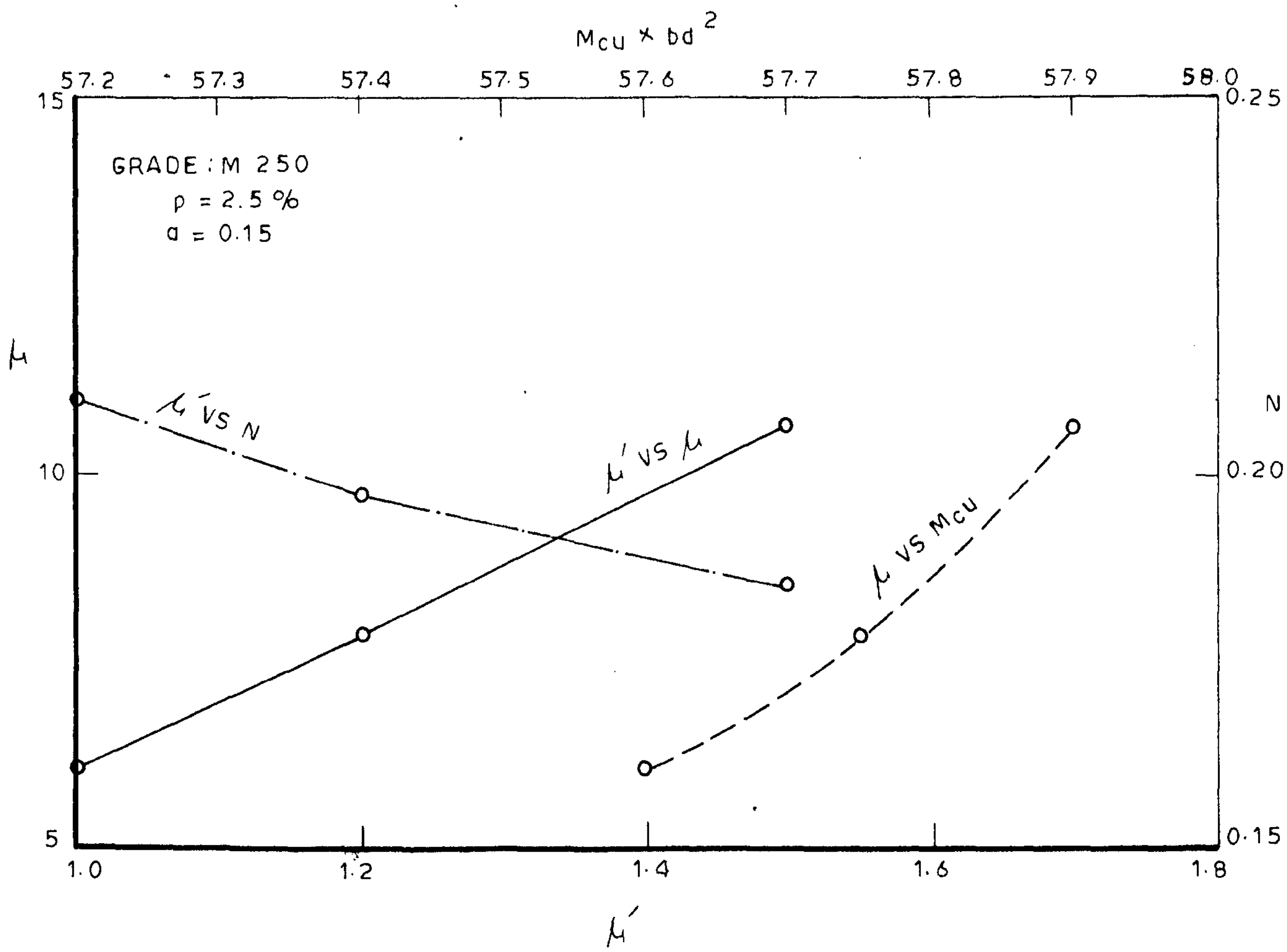


FIG. 4.23

CHAPTER V

EXPERIMENTAL STUDY

5.1 Very little information is available about the actual performance of R.B. and R.C. sections when certain ductility is allowed in both reinforcing steel and concrete or brick. It, therefore, becomes necessary to study experimental behaviour and compare the results with the theoretical values. With this objective, models were constructed and experiments were performed.

5.2. DESCRIPTION OF MODELS

The concrete columns (10 cm x 18 cm) were made in two different grades i.e. M 150 and M 250 giving cement, sand; aggregate proportion as 1:2:4 and 1:1:2 respectively. For reinforcement, 16 mm dia. mild steel bar was placed at a cover of 0.15 d. To fix the column at base, a base plate of 30 cm x 30 cm x 1.2 cm was used. The reinforcing bars were welded to this plate and to make a bond between plate and concrete, some steel hooks were also welded to plate in a staggered fashion. At base, a cut was left in column to, expose the tension reinforcement to fix the strain- gauge. The companion specimens were also prepared while casting the columns to obtain the basic properties of mortar.

The bricks used for brick columns were of nominal

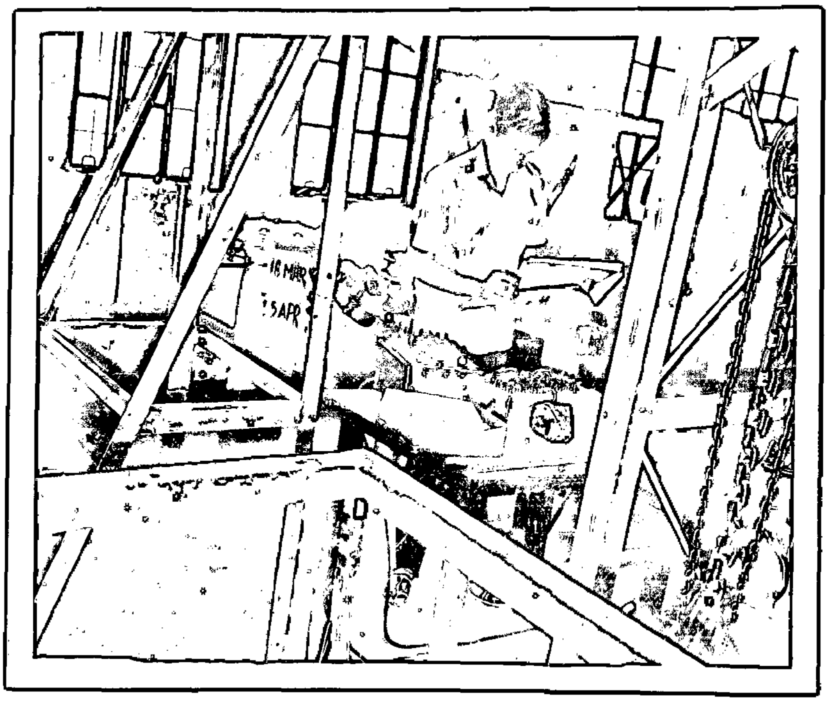


Fig5 .1

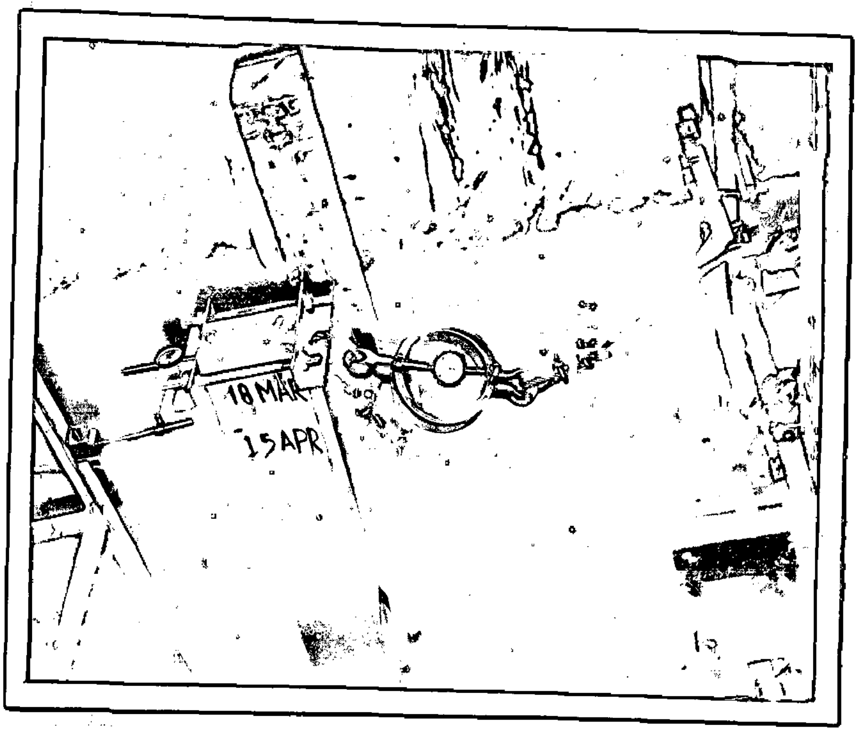


Fig 5 .2

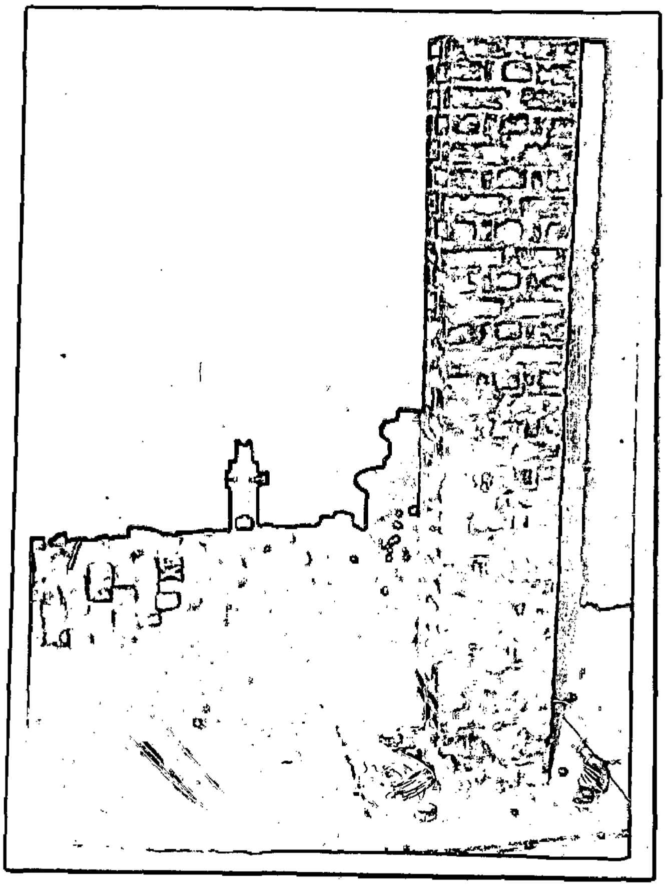


Fig 5.3

size 3" x 1.5" x 1" and the mortar used had proportion of cement sand as 1:3 by weight. In these columns 12 mm dia. mild steel bar was used as reinforcing steel at a cover of 0.2 d. The size of column was 10 cm x 12.5 cm keeping in view the size of brick. The fixing arrangement of column to base plate is similar to concrete column.The water cement ratio was kept constant and same mason was employed to construct all models in order to minimise the variation in workmanshop. The models were cured for 28 days.

After curing two strain gages (Type : CA-10, Gauge factor = 2.06) were fixed on each column i.e. one on tension steel and other on concrete or brick in compression (Fig. 5.3)

5.3. TESTING APPARATUS

The load on the column was applied with the help of a chain pulley block system. The loading system was hung on a frame as shown in Fig.(5.2). The load was measured with the help of a proving ring.

A long arm dial-gauge was fixed on a refrence frame to measure the deflection at the top of the column for the applied lateral load. The dial gauge was having a least want of 0.01 mm.

The strains in concrete or brick and tensile steel were measured with the help of a strain indicator, multi-channel switch and a transformer, (Fig. 5.1 and 5.2).

5.4 EXPERIMENTAL RESULTS

The companion specimens of M150 and M250 grade exhibited the following properties:

M150 : σ_{mc} = 180 Kg/cm^2

e_{cm} = 0.3 percent

M250 : σ_{mc} = 230 Kg/cm^2

e_{cm} = 0.3 percent.

TABLE 5.1

CONCRETE COLUMN (M 150)

COLUMN I

Strain in concrete	Load (Kg) Experi-mental	Load (Kg) Theore-tical.	Strain in ten-sile steel Experi-mental	Strain in ten-sile steel Theor-itical	Deflection top in m.m.
0	0	0	0	0	0
.00013	140	118	.00024	.000195	2.3
.00043	250	370	.00077	.000645	4.21
.00075	370	607	.00135	.00105	7.85
.00109	480	712	.00228	.00244	11.88
.00149	570	720	.00350	.00488	15.5
.00188	640	726	.00671	.00700	19.85
.00229	680	733	.00822	.0101	27.80
.00263	700	740	.0117	.0124	40.50

TABLE 5.2

(M 150) COLUMN II

Strain in Concrete	Load (Kg)		Strain in tensile steel		Deflectio of top in m.m.
	Experi-mental	Theore-tical	Experi-mental	Theori-tical	
0	0	0	0	0	0
.00012	140	109	.00022	.00018	1.8
.00021	250	188	.00040	.000315	3.70
.00051	370	432	.00090	.000765	5.50
.00086	480	705	.00160	.00133	8.92
.00101	570	708	.00239	.00204	12.20
.00125	640	715	.0035	.00332	15.70
.00156	680	721	.00565	.00518	20.0
.00190	700	730	.00748	.00710	25.20

TABLE 5.3

(M250) COLUMN I

Strain in concrete	LOAD (Kg)		Strain in tensile steel		Deflection at top in m.m.
	Experimental	Theoritical	Experimental	Theoritical	
0	0	0	0	0	0
.00019	150	206	.00025	.000285	2.2
.00032	300	340	.00048	.00048	4.10
.00050	450	520	.00098	.00075	7.30
.00071	500	750	.00130	.00123	8.70
.00108	550	758	.00279	.00319	11.90
.00146	600	769	.00540	.00588	15.20
.00197	650	780	.00721	.00910	19.30
.00242	700	790	.00855	.0119	22.20
.00263	725	794	.00983	.0128	27.70
.00270	750	797	.0126	.0139	32.50

TABLE 5.4

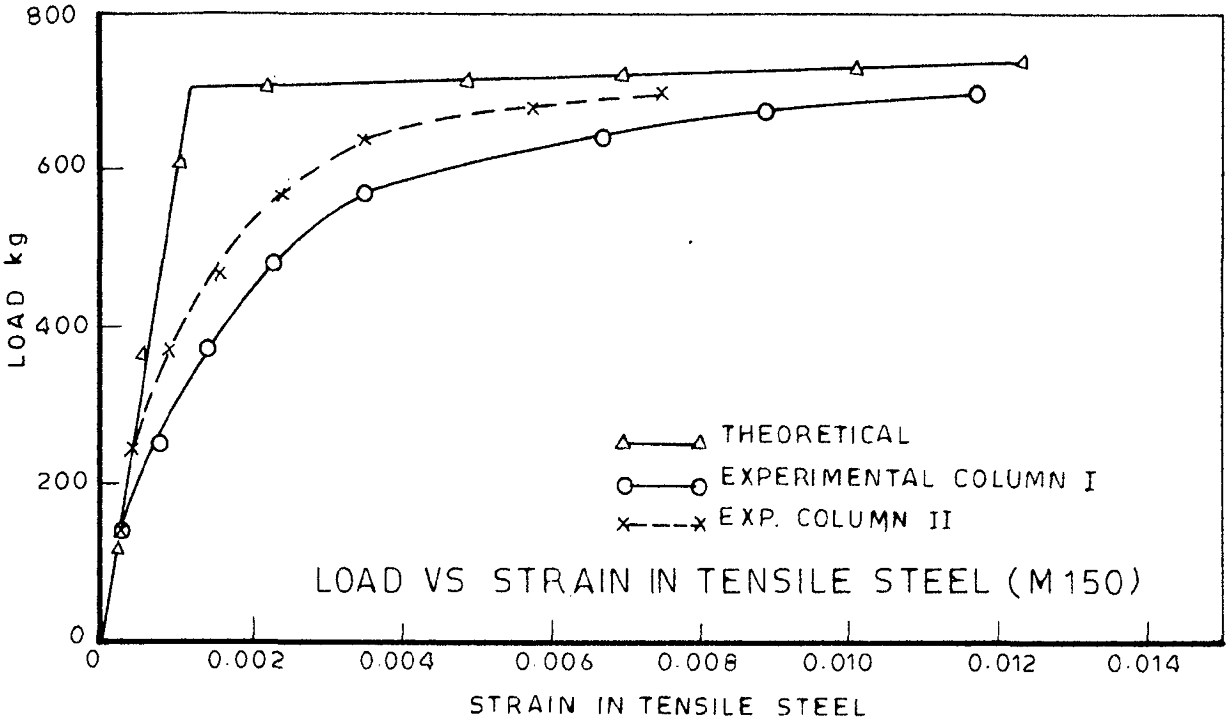
BRICK COLUMN (1:3) I

Strain in Brick	Load (Kg). Experi-mental	Load (Kg). Theori-tical	Strain in tensile steel Experi-mental	Strain in tensile steel Theori-tical	Deflection of top in m.m.
0	0	0	0	0	0
.00025	100	67.2	.00034	.000231	2.78
.00057	200	153	.00080	.000526	6.04
.00920	300	248	.00130	.00085	9.66
.001320	350	346	.00187	.00133	14.46
.001940	400	380	.00292	.00345	20.11

TABLE 5.5

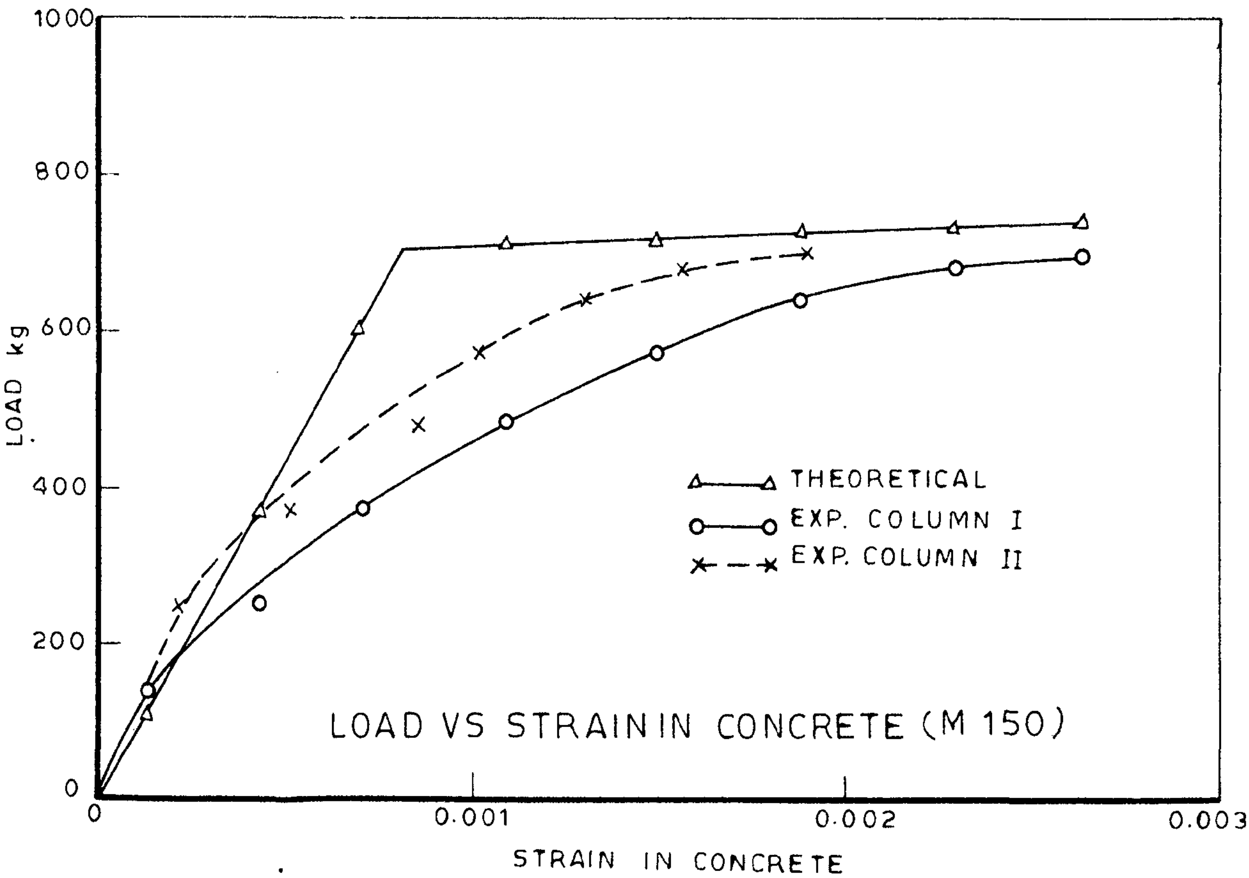
BRICK COLUMN (1:3) II

0	0	0	0	0	0
.00023	100	61.8	.00029	.000212	2.20
.00051	200	137	.000725	.00047	5.40
.00088	300	236	.00120	.000813	8.0
.00130	350	350	.00163	.00130	13.00



800
600
400
200
0
LOAD kg
0
0.002
0.004
0.006
0.008
0.010
0.012
0.014
STRAIN IN TENSILE STEEL
THEORETICAL
EXPERIMENTAL COLUMN I
EXP. COLUMN II
LOAD VS STRAIN IN TENSILE STEEL (M 150)

FIG. 5.4



1000
800
600
400
200
0
LOAD kg
0
0.001
0.002
0.003
STRAIN IN CONCRETE
THEORETICAL
EXP. COLUMN I
EXP. COLUMN II
LOAD VS STRAIN IN CONCRETE (M 150)

FIG. 5.5

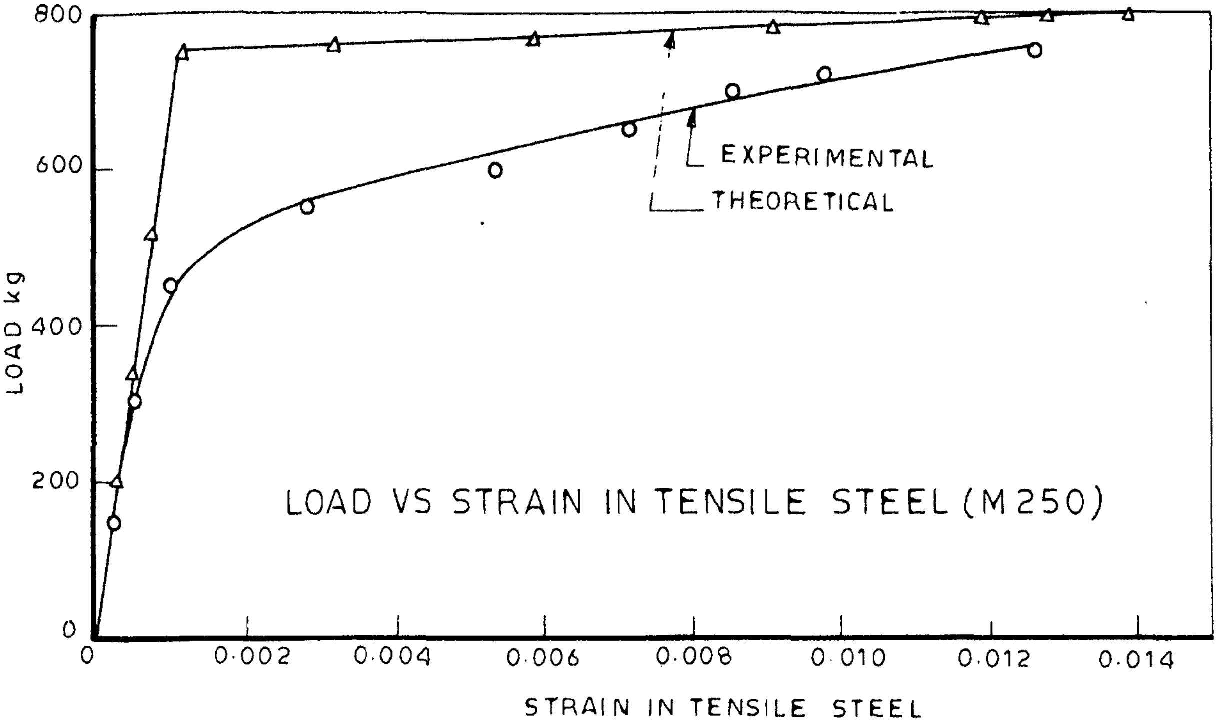

800
600
400
200
0
LOAD kg
0
0·002
0·004
0·006
0·008
0·010
0·012
0·014
STRAIN IN TENSILE STEEL
EXPERIMENTAL
THEORETICAL
LOAD VS STRAIN IN TENSILE STEEL (M 250)

FIG. 5.6

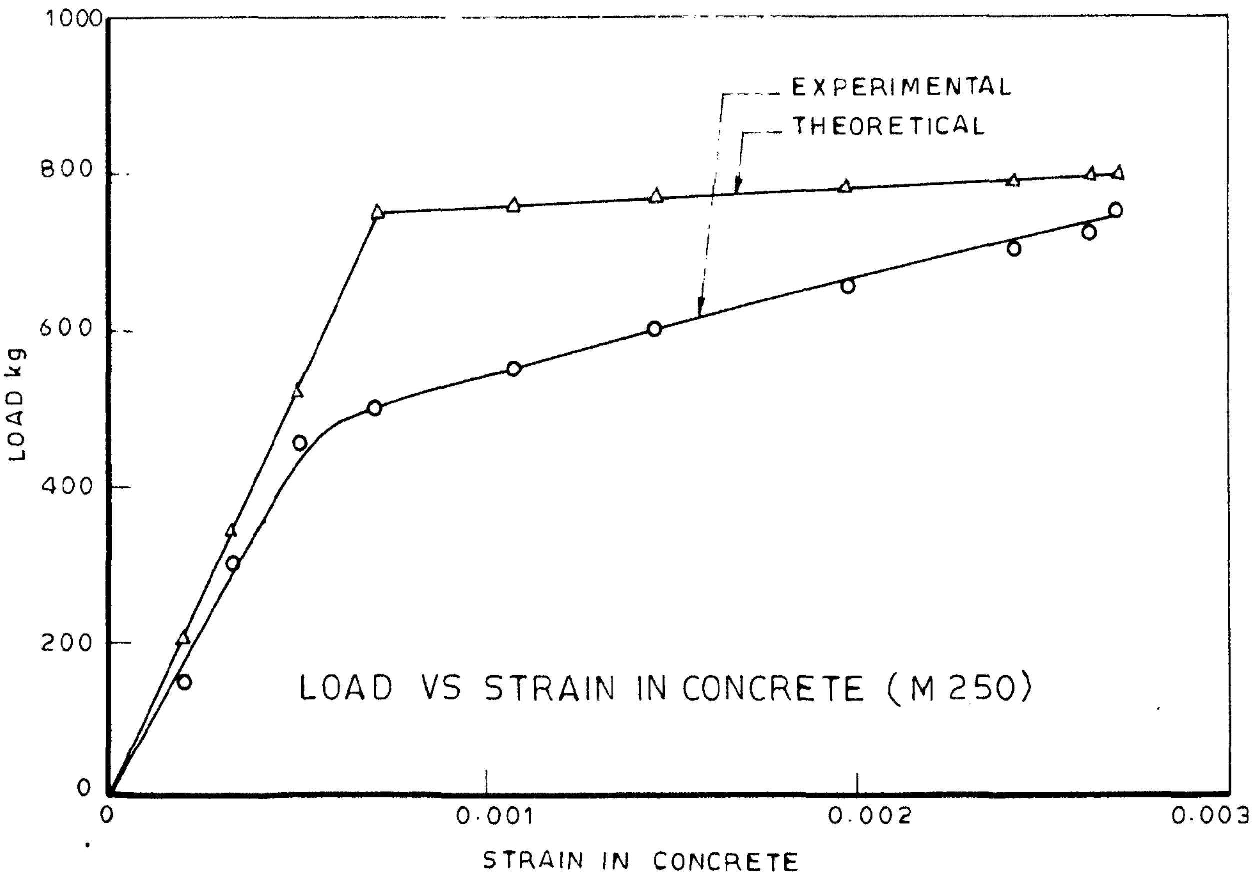

1000
800
600
400
200
0
LOAD kg
0
0·001
0·002
0·003
STRAIN IN CONCRETE
EXPERIMENTAL
THEORETICAL
LOAD VS STRAIN IN CONCRETE (M 250)

FIG. 5.7

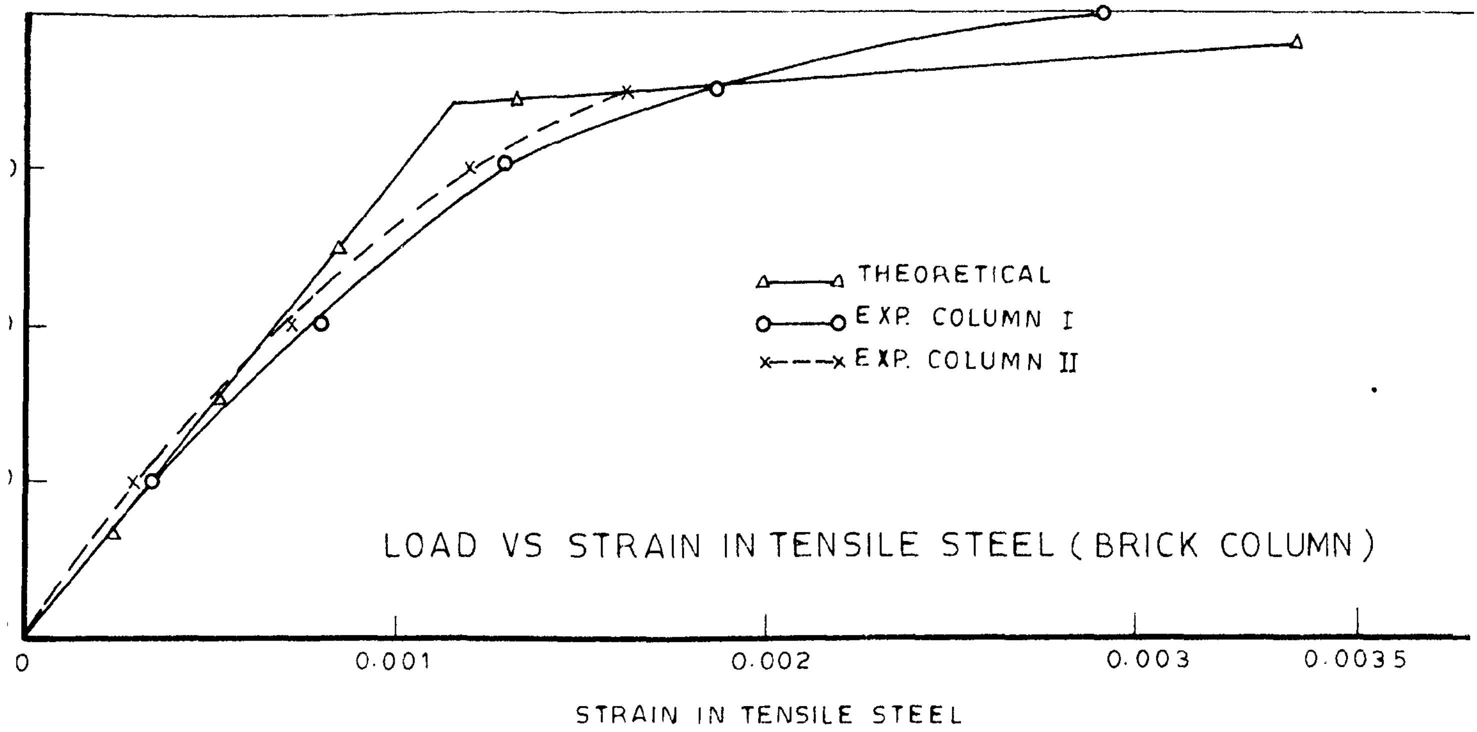

THEORETICAL
EXP. COLUMN I
EXP. COLUMN II
LOAD VS STRAIN IN TENSILE STEEL (BRICK COLUMN)
0
0.001
0.002
0.003
0.0035
STRAIN IN TENSILE STEEL

FIG. 5.8

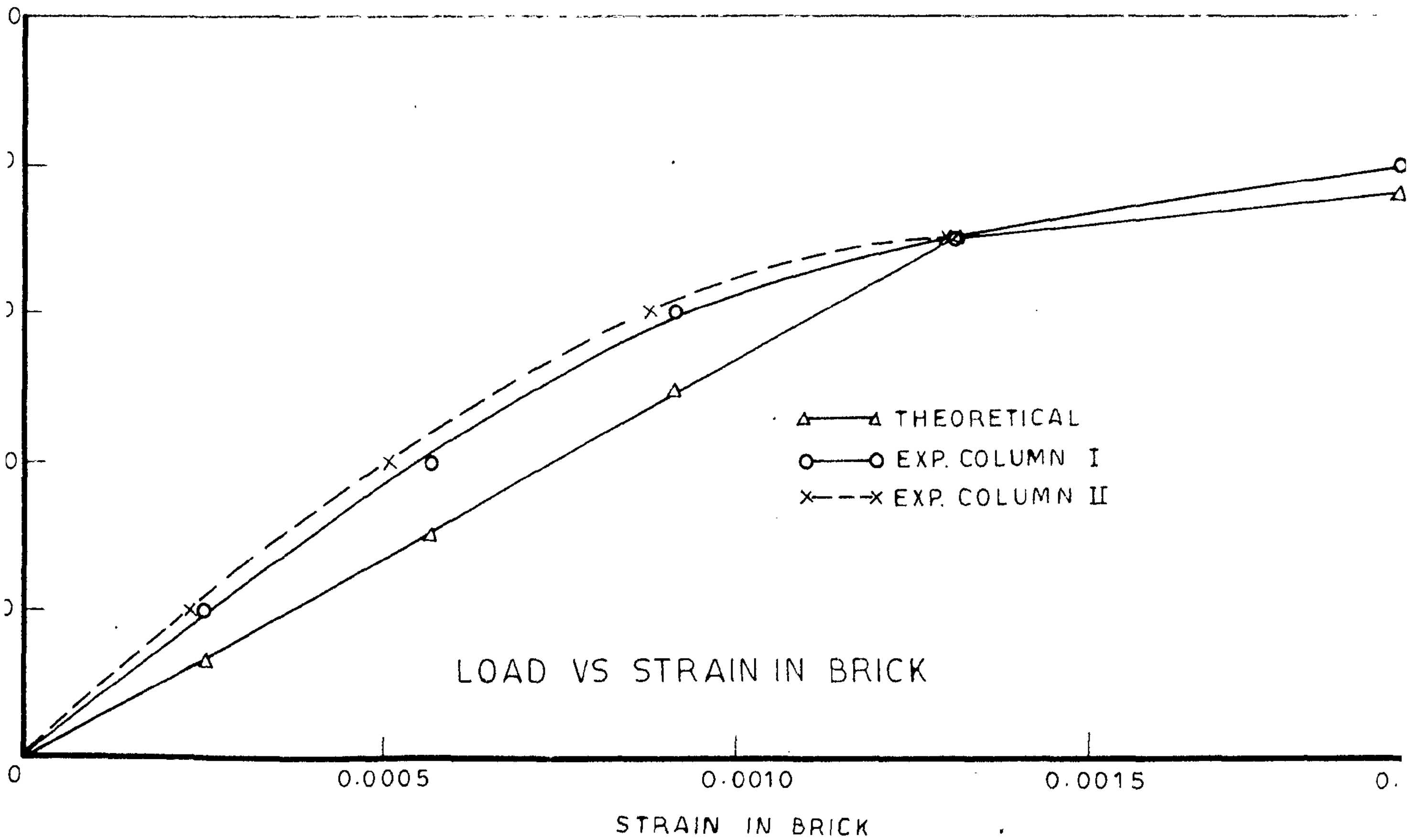

THEORETICAL
EXP. COLUMN I
EXP. COLUMN II
LOAD VS STRAIN IN BRICK
0
0.0005
0.0010
0.0015
STRAIN IN BRICK

FIG. 5.9

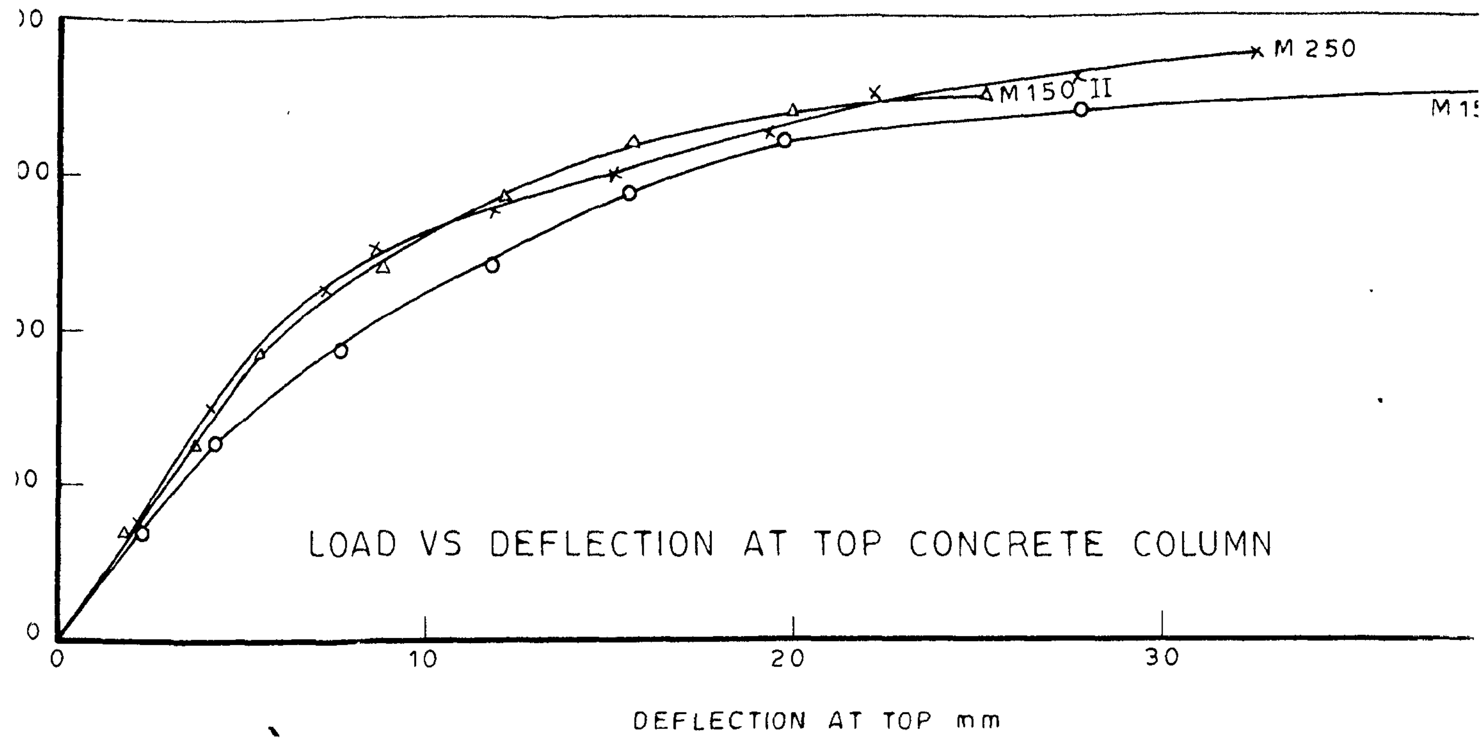


FIG. 5.10

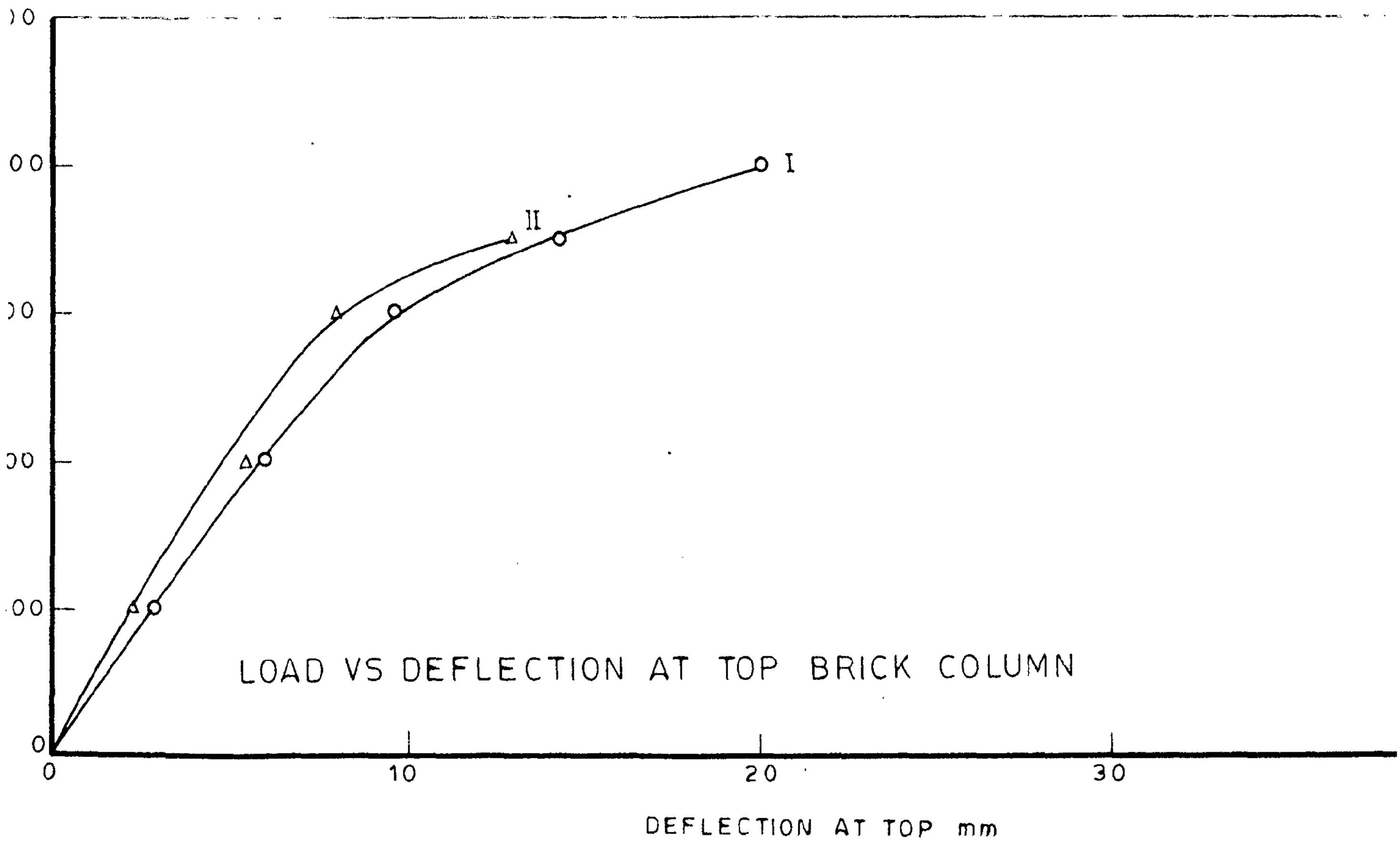


FIG. 5.11

5.5 DISCUSSION OF RESULTS

Figs. 5.4 to 5.9 show the variation of strain in various materials (concrete, brick and tensile steel) with respect to applied lateral load. Figs. 5.10 and 5.11 show load vs deflection at top of columns.

It may be observed from Figs. 5.4 to 5.7 that all experimental curves lie below the theoretical curves. However, in case of brick columns (Figs. 5.8 and 5.9), they are above the theoretical curves. This discrepency in results may be attributed to the following reasons:

1. The value of yield stress and yield strain of reinforcing steel may not be 2600 Kg/cm^2 and 0.124 percent respectively, as has been assumed in obtaining the theoretical curves.
2. The strain gauges fixed may not be exactly vertical because a slight inclination of gauges would show strains less than the actual strains.
3. The gauge factor of strain gauges may be somewhat different than the value given by the manufacturer.
4. The proving ring registers a lower load than the actual because of some part of load is lost when the clips get loosened.

5. Perfect fixity of column bases can not be achieved in practice. Any deformations, however small they may be, would reduce the stiffness of the column bringing down the load deflection curve.

CHAPTER VI

CONCLUSIONS

On the basis of results obtained from the theoretical and experimental investigation, reported in earlier chapters, the results can be summarized as follows:

1. A brick building does not develop any tension upto a lateral load corrosponding to 8 percent g acceleration and all its piers remain in compression. This is in agreement with the provisions of IS: 4326-1967, Code of practice for earthquake resistant construction of buildings.

2. The central pier B of the building chosen for study (See Chapter 3) attracts the largest force and gives rise to worst condition of stresses when the the lateral load is applied from right hand side.

3. Stresses in reinforcing steel and brick work under worst conditions are well within the permissible range of stress even at a lateral load corrosponding to 20 percent g acceleration.

4. For withstanding higher forces, use of energy absorption capacity of the structure can be made.

5. By increasing the cover in a reinforced brick or reinforced concrete 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 moment of resistance of the section.

6. A slight increase in ductility in concrete increases the steel ductility appreciably.

COMPUTER PROGRAM

****

NOTATIONS USED-

AM= MODULAR RATIO
DT=DEPTH OF SECTION(EFFECTIVE)
T=WIDTH OF SECTION
ST=AREA OF TENSILE STEEL
SC=AREA OF COMP. STEEL
Y= COVER
TM=MOMENT
VR=AXIAL FORCE
E= ECENTRICITY
AN= N.A.
SB=STESS IN BRICK
SS=STESS IN TENSILE STEEL

```
C  C  REINFORCED BRICK PROJECT  (PIER ANALYSIS )  VINEET  24305
       DIMENSION DT(30),T(30),ST(30),SC(30),Y(30),HT(30),P(30),AR(30)
       DIMENSION TM(30),VR(30),E(30)
      READ11,N,AM
11    FORMAT(I3,F10.0)
      AMM=AM-1.
100   READ61,(DT(I),T(I),ST(I),SC(I),Y(I),I=1,N)
61    FORMAT(5F10.3)
      DO62I=1,N
      DP=DT(I)
      TH=T(I)
      YZ=DP*.5-Y(I)
      YS=YZ*YZ
      TS=ST(I)+SC(I)
      AR(I)=DP*TH+AMM*TS
      TI=TH*DP*DP*DP/12.+AMM*TS*YS
62    P(I)=TI
      READ70,KK
70    FORMAT(I2)
      K=1
63     READ64,(TM(I),VR(I),I=1,N)
64    FORMAT(2F10.4)
      DO66I=1,N
      E(I)=TM(I)/VR(I)
      QR=VR(I)
      VRA=ABSF(QR)
      EE=ABSF(E(I))
      BB=T(I)
      AA=Y(I)
      DD=DT(I)-AA
      YY=0.5*(DD-AA)
      AC=SC(I)
```

```
      AT=ST(I)
      IF(QR)20,19,19
19    Q=-1.
21    EK=2.*P(I)/(AR(I)*DD)
      IF(EE-EK)81,81,121
81    ZA=AM*VRA/AR(I)
      ZI=AM*VRA*EE*YY/P(I)
      SB=ZA+ZI
      SS=ZA-ZI
      AN=1.
      GOTO122
20    Q=1.
      IF(EE-YY)82,82,121
82    SB=VRA*(YY-EE)*0.5/(AC*YY)
      SS=VRA*(YY+EE)*0.5/(AT*YY)
      AN=1.
      GOTO122
121   BDS=BB*DD*DD*0.5
      BDC=BDS*DD
      QEY=Q*(EE-Q*YY)
      QX=QEY*AM*AT*DD
      CT=(AM-1.)*AC*(DD+QEY-AA)
      A=-BDC*0.333333
      B=BDS*(DD+QEY)
      C=DD*CT+QX
      D=-CT*AA-QX
      AN=0.5
3     SQ=AN*AN
      CU=AN*SQ
      U=A*CU+B*SQ+C*AN+D
      Z=3.*A*SQ+2.*B*AN+C
      AY=AN-U/Z
      IF(ABSF(AY-AN)-.01)1,1,2
2     AN=AY
      GOTO3
1     DN=AN*DD
      ANN=(1.-AN)/AN
      DE=AT*AM*ANN-BB*DN*0.5-(AM-1.)*AC*(DN-AA)/DN
      SB=Q*VRA/DE
      SS=ANN*AM*SB
122   PUNCH22,TM(I),VR(I),E(I),AN,SB,SS
22    FORMAT(6E11.3)
66    CONTINUE
      K=K+1
      IF(K-KK)63,63,100
      END
```

: N O T A T I O N S :

...

The notations are defined wherever they first appear. Here they are collected in alphabetical order for convenience of reference:

a	=	Cover of Steel (Fraction of Depth)
b	=	Width of Section
d	=	Effective depth
e	=	Strain
e_{cm}	=	Strain corrosponding to max. stress.
E	=	Modulus of elasticity.
F_c	=	Force of compression in concrete.
G	=	Modulus of rigidity.
H	=	Height of shear wall.
h_i	=	Height of panel or depth of bent spandrel.
h'	=	Equivalent ht. of bent column.
H_i	=	Horizontal force shared by each column.
I	=	Moment of Inertia
L	=	Length of Shear Wall.
L_1	=	Depth of side piers.
L_2	=	Width of opening.
M_o	=	Overturning moment

M_a	=	Moment in column.
m	=	Modular ratio
M_{bu}	=	Ultimate moment of resistance in brick.
M_{cu}	=	Ultimate moment of resistance in concrete
N	=	Distance of N.A. compression edge (fraction of d).
p	=	percentage of steel
U	=	Strain energy due to flexural
V	=	Vertical reaction in columns of bent
y	=	deflection
ϵ_i	=	Horizontal deflection at joint i.
σ_{sc}	=	Stress in compression steel
σ_{mc}	=	Max. compressive stress in concrete
σ_{yst}	=	Yield stress in tensile steel.
μ	=	ductility in tensile steel
μ'	=	ductility in concrete.

...

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