INFLUENCE OF INITIALLY CURVED BARS ON THE BEHAVIOUR OF REINFORCED CONCRETE COLUMNS

A THESIS

PRESENTED BY

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ABSTRACT

When the longitudinal reinforcements in a column are initially curved towards the axis of the column, they tend to bend inward and induce a lateral pressure on the concrete core under load. Advantage of the confining pressure may be taken for partial elimination of lateral ties in a column.

A theoretical study was made to derive expressions for finding optimum amplitude of initial curvature of the longitudinal reinforcement with respect to other parameters for partial elimination of usual ties, and minimum requirement of ties in columns with such reinforcement.

Tests were made on the column reinforced with initially curved bars, and tied and untied straight bars. The results of the experimental works were tabulated and the behaviour of specimens under load described. Comparison between theoretical and experimental values were made with special emphasis on the effect of initially curved bars, concrete strength and percentage of reinforcement on the structural behaviour of columns. The laboratory investigation shows that for developing ultimate load, the columns reinforced with longitudinal bars having adequate initial curvature require ties only at the ends.

Methods for determination of modulus of elastic foundation and tensile bond strength between steel bars and concrete has been suggested.

DEFINITIONS AND NOTATIONS

Plain column - Unreinforced concrete column.

Modulus of elastic foundation - the magnitude of reaction of the foundation per unit length of the bar if the deflection is equal to unity.

Tensile bond strength - the bond strength at the interface of concrete and steel when they are pulled in tension away from the surface of contact.

Amplitude of curvature - the mid-deflection of the curved bar.

f_{yL}	= yield point strength of longitudinal steel
f_{yt}	= yield point strength of tie.
E	= modulus of elasticity of longitudinal steel
E_t	= modulus of elasticity of tie steel.
E_c	= secant modulus of concrete at $0.4\ f'_c$
f'_c	= Ultimate compressive strength of concrete.
f''_c	= the stress developed in concrete due to lateral strain just before crushing failure of concrete.
t_f	= tensile bond stress between steel bars and concrete.
A'	= area formed by the width of the column per unit longitudinal length of the column.
N	= number of longidudinal steel bars on one side of the splitting plane
a_{sc}	= half of the projected lateral surface area of the bar per unit length of the bar.
a	= initial mid-deflection of the longitudinal steel bars
L	= unsupported length of the longitudinal steel bar in between the end ties
P	= the load carried by the longitudinal steel bars at the failure of the column.
q	= distributed lateral load due to lateral strain of concrete and tensile bond at the surface of contact between steel and concrete
a_1	= final mid-deflection of the longitudinal steel bars
a_n	= amplitude of curvature of n number of modes
n	= number of modes of curvature
β	= modulus of elastic foundation
A_t	= cross sectional area of tie
b	= width of core of column
m	= effective number of tie bars.

CONTENTS

CHAPTER - I

INTRODUCTION

GENERAL REMARKS

From the earliest days of reinforced concrete design, much research activity has been directed towards a better understanding of the behaviour of concrete columns. All this work, however, limited its attention to columns reinforced with steel rods which were straight and supported laterally by relatively widely spaced ties or a continuous closely spaced spiral. It was observed by James F. Pfister (1) that in adequately tied columns the concrete stress at ultimate strength of the columns was between 12-15 per cent higher than the value for the columns with no intermediate tie in between the ends. This increase in strength is mainly due to the lateral support given by the ties. The ties prevent highly stressed, slender longitudinal bars from buckling outward bursting the thin concrete cover. Still the tied column failures show bulging of concrete and buckling of the rods. Bresler (2) and few other researchers carried out investigations on concrete columns reinforced laterally only with end ties and no intermediate tie in an attempt to alleviate the difficulties in casting columns with conventional ties supplied in accordance with 1957 A.C.I. code. Based on their works and subsequent discussions 1963 A.C.I. code reduced much the use of ties. But the ties supplied according to 1963 A.C.I. code still pose problems relating to detailing and fixing of reinforcement, and compaction and settlement of concrete during casting of columns. Investigations have so far been made towards reducing the conventional use of ties. But none of them was directed to find an alternative or replacement of the lateral reinforcement.

Statement of the Problem

In an early investigation made by the author attempt was made to prevent premature buckling of the longitudinal bars by giving an initial curvature to each of the bars towards the axis of the column and providing ties at the ends. The columns showed no such sudden failure as observed in normally tied columns. The study, limited to testing of few columns indicated that the initially curved bars serve both as longitudinal and lateral reinforcement. Further studies, it is believed, will prevent failures resulting from instability of the longitudinal bars and help the engineers avoid the difficulties arising from cries crossing of the column section by too many ties necessary to position and hold all straight bars and prevent them from buckling.

An investigation is necessary to have a basic understanding for developing mathematical and emperical relationship necessary to enable engineers to design columns reinforced with bars having initial curvature. To achieve this, a theoretical study is made and an experimental investigation on the behaviour of columns with respect to ductility and load carrying capacity corresponding to different initial curvatures of the bars and a few other parameters like concrete strength, and diameter, spacing and percentage of the longitudinal bars is conducted. The analysis of the test results is made to explore relationships between different parameters as encountered in this investigation.

Objectives of the research

It was observed by the author that under failure load the single tie provided at each end of the columns shows a premature failure at its welded joint. In this investigation several end ties are used to position and hold the initially curved bars during casting. These ties offer restraint to the outward movement of the ends of the curved bars and lateral straining of concrete. In structural frames the beams and slabs at their level would generally restrain lateral straining of concrete and end movement of the curved bars. Hence end ties are only required to hold the bars in position during casting. The main objectives of the research are to investigate into the use of initially curved bars which are expected to serve as longitudinal and lateral reinforcement.

The objectives of the research are as follows :

1. To make a comparative studybetween the behaviour of columns reinforced with initially curved bars and straight bars. The straight bars are laterally supported either by usual ties or only end ties.

2. To investigate the influence of variation of the following on the structural behaviour of columns :

a. initial curvature of the bars

b. percentage of curved bars.

c. diameter and spacing of curved bars.

d. concrete strength.

Scope of the Experiment

The investigation was undertaken to find the effect of inward initial curvature of the bar on the behaviour and ultimate strength of concentrically loaded reinforced concrete columns. The variables considered were amount of initial curvature of the rods, size and spacing of longitudinal bars, shape of column cross sections and strength of concrete. In practice concrete columns are usually 'short' from the point of design. These tests were, therefore, limited to short columns.

Five series of tests was conducted in this investigation. The series of tests were classified according to the objectives of the research The investigation included 40 specimens, ten of which were plain and the others reinforced.

Compressive stresses were evaluated on the basis of both initial cracking load and ultimate load. Deformation of the column was measured at different stages of loading. Finally an attempt was made to put forward an emperical design within the limits of this investigation.

Outline of the research

An attempt has been made to achieve the objectives of the project through the following main steps :

1. Review of the pertinent literature.

2. Theoretical investigation to find mathematical relationships between different parameters.

3. Laboratory investigation to ascertain physical properties of the materials employed in the project.

4. Laboratory investigation to evaluate the influence of initial curvature, diameter and spacing of the bars on the ultimate strength and ductility of the column.

5. Analysis of the test results to ascertain the validity of assumptions made during theoretical investigations and finally putting forward an emperical design.

CHAPTER II

REVIEW OF LITERATURES

It appears that no work except that done by the author earlier has been done in an attempt to use the initially curved bar as reinforcement in concrete column. However, there are many published works on laterally reinforced concrete columns. A few selected papers on the pertinent topic is summarised below :

Functions of ties

Prof. R. H. Evans (4) reported that the function of closely spaced links in a column are to restrain the shearing of the concrete and to limit the buckling of the main bars. One important function of ties is to set up a state of triaxial stresses in the core of the column which enhances the compressive strength of the concrete in the core.

A preliminary study of tests on columns was reported by J. F. Pfister (1) and the following conclusions were drawn :

1. As the axial load on the column increases, the local strains or stresses in the outer concrete increases until they reach a limiting value.

2. At this limiting strain or stress, the exterior cover cracks, so that it can no longer support appreciable load or it spalls off.

3. The loss of the cross sectional area of the concrete cover imposes additional stresses on the remaining concrete core and the steel reinforcement.

4. The steel reinforcement under this additional stress begins to yield or buckles outward.

5. Loss of stiffness of the steal due to yielding and for impending buckling causes additional strain in the concrete area.

6. The core sub-divided into short prisms, maintains its integrity until its ultimate strength is reached and then fails suddenly.

7. At this point the entire column fails and load bearing capacity is lost. The above events occur rapidly and the failure is often characterised as occuring instantaneously.

Pfister (1) further reported that the size and spacing of ties influence the following factors :

8. Local state of stress (or strain) in the concrete in the zone near the ties.

9. Strength of concrete core.

10. Buckling strength of longitudinal reinforcement, particularly when the concrete cover may be lost due to imposed load or accidental damage.

Effects of ties

In non seismic areas ties have traditionally been provided with a view towards reducing the possibility of local buckling of the longitudinal reinforcement under load approaching the ultimate strength of the column. To this end, 1963 A. C. I. code requires that the ties of at least $\frac{1}{4}''$ diameter be provided at intervals not exceeding 16 bar diameters, 48 tie diameters, or the least lateral dimention of the column. In the square and rectangular column with more than 4 rods the rods in between the corners will try to bend sideways and exert a lateral pressure which is normal to the straight sides of the bands. The bulging of the concrete does likeways. However the bands are not effective in withstanding such beam action without bending outward so much that concrete may crack. So A. C. I. code futher requires that the ties shall be so arranged that every corner bar and alternate longitudinal bar shall have lateral support provided by the corner of a tie having an included angle of not more than 135° and no bar shall be farther than 6 inches from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular ring may be used. These requirements lead to tie arrangements of the kind shown in Fig. 2.1 which is taken from the 1963 A. C. I. detailing manual. James F. Pfister (1) reported that the tie arrangements involving several interior ties probably impede the compaction and settlement of concrete in the column core to a considerable degree. Such interior ties decrease total column strength under practical conditions. These views

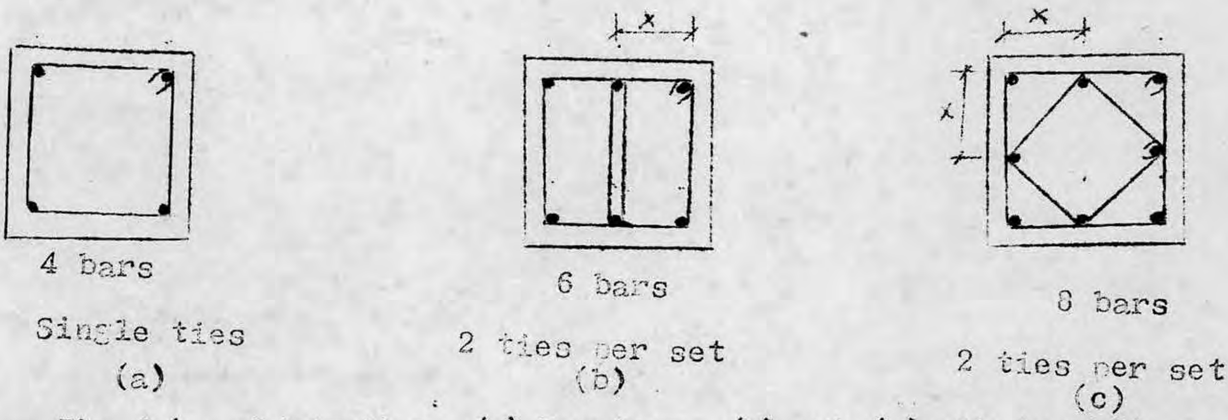


Fig. 2.1 - Column ties. (a) Four bars. (b) and (c) Six to eight bars. If x is not over 6 in., the single tie of (a) is permissible.

are also supported by the following statements made by E. M. Rensa(11) based on the information obtained from the references (12,13,14) and his own theoretical and practical views and experience.

Early investigators showed that the ties has an influence on the ultimate strength of reinforced concrete columns and this influence might be both positive and negative. The ties will to some extent hinder compaction of the concrete and its settlement in the form after it has been placed. This condition cannot be completely eliminated regardless how well the concrete is placed and compacted. For concrete of wet consistancy and badly compacted, the weakening effect is quite high. The square or rectangular ties with hook anchorage at a corner most often used seem to be one of the least favourable in this respect. In addition, it does not give a complete anchorage, unless the ends are bent very tight around the vertical corner bars. A column with such ties and vertical reinforcement may even have less bearing power than an unreinforced column of the same cross-section. A similar weakening effect on walls with light vertical reinforcement has been found by swedish tests.

Hajnal Kony (15) pointed the effect of links in low strength concrete, namely that maximum measured strain of a column with links is substantially greater than that in prisms of plain concrete. Prof. R. H. Evans (4) is reported to confirm the statement. He also added that at normal spacings the ties are relatively ineffective.

According to Soretz and Stumf (12) it is uneconomical to use ties as a direct means of increasing the load carrying capacity of a column. However, the ties has also the function to prevent buckling of the vertical reinforcement and what perhaps is less known, a reasonable amount of ties will increase the ultimate strain considerably. This last condition is important if high strength steel is used and if a brittle or sudden failure is to be guarded against. The unit strain of reinforced concrete column at ultimate load is around 2 percent. The steel strain at 100,000 psi yield point would, however, be about .35 percent. It has been for this reason held that high strength vertical bars could not be fully utilized in tied columns. The investigations of Soretz and Stumf indicate, however, that with proper amount of ties the compressibility of the concrete column will increase to such an extent that vertical steel with yield point of 140,000 psi may be fully utilized.

Influence of initially curved bars

The author(3) reported earlier that the initial curvature given to the longitudinal reinforcement induces the following phenomena in the failure of columns.

1. The column fails like spiral columns whose failures are characterised by spalling of concrete cover and high longitudinal ultimate strain.

2. Buckling of the rods at the failure of the column is almost absent.

It was also contended by the author that the use of ties can be reduced by giving initial curvature to the longitudinal reinforcement.

CHAPTER III

THEORETICAL INVESTIGATION

The failure of a reinforced concrete column may start with the buckling of the reinforcing bars, crushing, splitting or shearing of concrete, tension failure of the tie bars placed at the ends and the yielding of the main reinforcing bars either in tension or in compression.

The inward curvature should be given to the longitudinal rods reinforcing only those columns which are not subjected to eccentric loadings which cause tension in any part of the column. The inwardly curved bar, stressed in tension, may move outward bursting the thin concrete cover and reducing the ultimate strength of the columns. An outwardly curved bars subjected to compressive load would behave in the same manner.

No consideration was given within the scope of the investigation to the effect of eccentricity on the column strength. All the probable modes of failure except tension failure of steel or shearing of the concrete was considered.

The behaviour at working loads has been analysed by elastic theory assuming that the initially curved bar and the supporting concrete behave as elastic materials. The validity of these assumptions can only be established by a comparison with test data.

Buckling failure of the Main Bars

The steel bars in the column under load will be subjected to the following :

1. The longitudinal compressive force that is, the part of

the load carried by the steel.

2. The laterally distributed load which results from the initial curvature of the main bars and the lateral strain in the concrete.

If the spalling of the shell concrete in the column precedes the buckling of the main bars, the bars will be under a lateral load due to tensile bond between the main bars and the shell concrete. After the spalling of the shell concrete, if any, the core concrete under load will induce distributed axial loads on the bars. Before spalling, there will be produced little distributed axial load on the steel bar as the core concrete is supported by the shell concrete. Two cases, therefore arises, one before the spalling and the other after the spalling of the shell concrete.

It should be noted that as the initially curved bars will have a tendency to buckle inward, it will be laterally supported by a farm elastic foundation rendered by concrete. In the analysis of the buckling of the main reinforcing bars, the strains in the concrete and the steel are assumed to be equal. On the basis of this assumption it can be concluded that bond between concrete and steel does not come into play. It is further assumed that the ends of the bars are hinge supported. Two cases as above may be considered separately in analysing the mode of buckling of the main bars.

Case A - Before the loss of the concrete cover

As stated in the above paragraphs, the mode of buckling of the longitudinal bars depends on the magnitude of the following :

(a) Part of the imposed longitudinal load carried by the steel bars.

(b) Outward lateral load due to lateral strain of concrete

(c) Inward equivalent lateral load as a result of initial curvature of the bar.

(d) Outward lateral load due to presence of tensile bond between the rods and the concrete.

The lateral loads encountered above are distributed all through the length of the bar. It was experimentally found that the tensile bond strength between steel bars and concrete is between 25 to 33 per cent of the tensile strength of concrete. So even if the outer shell does not spall off, there might be local failure of the bond between shell concrete and the bars. This conclusion justifies the use of lateral load due to tensile bond in the analysis of the mode of buckling of the bars.

As shown in the Figure 3.1, a segment having a length 'L' on two simple supports with continuous elastic support all through the length due to core concrete is considered. The bar carries longitudinal force P and a distributed lateral load 'q', representing those caused by the lateral strain of the core concrete and tensile bond between the concrete and the steel bars. If 'a' be the maximum deflection of the bar at the mid height the initial shape of the bar can be represented by the equation (16)

$Y_o = a \sin \frac{\pi x}{L}$ and the equivalent lateral load due to deflection is given by $\frac{\pi^2 aP}{L^2} \sin \frac{\pi X}{L}$.

The deflection curve of the reinforcement can be represented by the equation :

$$Y = a_1 \ \mathrm{Sin} \frac{\pi x}{L} + a_2 \ \mathrm{Sin} \frac{2\pi x}{L} + a_3 \ \mathrm{Sin} \frac{3\pi x}{L} + \ldots\ldots + a_n \ \mathrm{Sin} \frac{n\pi x}{L} \quad \ldots(a)$$

In the following discussion, strain energy method has been used for solving the buckling problem. According to this

$$\Delta U - \Delta W = 0 \quad \ldots\ldots\ldots (b)$$

Where ΔU = strain energy of bending of the bar and deformation of elastic medium of concrete

ΔW = work done by the external forces.

Fig. 3.1 - Load acting on an individual rod.

The strain energy of bending of the bar is

$$\Delta U_1 = \frac{EI}{2} \int_0^L \left(\frac{d^2 y}{dx^2} \right)^2 dx = \frac{\pi^4 EI}{4L^3} \sum_{n=1}^{n=\infty} n^4 a_n^2 \quad \ldots (c)$$

If β is the modulus of elastic foundation, that is, the magnitude of the reaction of the foundation per unit length of the bar if deflection of the bar is unity, the total energy of deformation of the elastic medium is

$$\Delta U_2 = \frac{\beta}{2} \int_0^L Y^2 \, dx = \frac{\beta L}{4} \sum_{n=1}^{n=\infty} a_n^2 \quad \ldots\ldots\ldots (d)$$

The work done by P is $\Delta W_1 = \frac{P\pi^2}{4L} \sum_{n=1}^{n=\infty} n^2 a_n^2 \quad \ldots\ldots\ldots (e).$

In calculating the work done by the lateral load q, it is noted that the lateral load on an elementary length dx is q.dx and the corresponding work done by the lateral load is

$$W_2 = \int_0^L qy\,dx = q\int_0^L \sum_{n=1}^{n=\infty} a_n \sin\frac{n\pi x}{L}\,dx$$

$$= \sum_{n=1}^{n=\infty} \frac{qa_n}{\frac{n\pi}{L}} \left[-\cos\frac{n\pi x}{L}\right]_0^L$$

$$= \sum_{n=1}^{n=\infty} \frac{(1-\cos n\pi)qLa_n}{n\pi} = \sum_{n=1,3,5....}^{n=\infty} \frac{2qLa_n}{n\pi} \quad .... \text{(f)}$$

The work done by the equivalent lateral load $\frac{\pi^2 ap}{L^2} \sin\frac{\pi x}{L}$ is

$$\Delta W_3 = \int_0^L \frac{\pi^2 ap}{L^2} \sin\frac{\pi x}{L} \left(a_n \sin\frac{n\pi x}{L}\right) dx \quad ..... \text{(g)}$$

since $\int_0^L \sin\frac{\pi x}{L} \cdot \sin\frac{n\pi x}{L}\,dx = 0$ when $n \neq 1$

and $\int_0^L \sin^2 \frac{\pi x}{L}\,dx = \frac{L}{2}$

all the terms of the above integral, except the first, disappear and ΔW_3 is finally obtained as

$$\Delta W_3 = \frac{\pi^2 ap}{L^2} a_1 \frac{L}{2} = \frac{\pi^2 ap}{2L} a_1 \quad ... \text{(h)}$$

Now using the relation (b) gives

$$\pi^4 \frac{EI}{4L^3} \sum_{n=1}^{n=\alpha} n^4 a_n^2 + \frac{\beta L}{4} \sum_{n=1}^{n=\alpha} a_n^2$$

$$= \frac{P\pi^2}{4L} \sum_{n=1}^{n=\alpha} n^2 a_n^2 - \sum_{n=1,3,5..}^{n=\alpha} \frac{2qL}{n\pi} a_n + \frac{\pi^2 aP}{2L} a_1 \qquad \text{....(i).}$$

with all terms except ' a' transfereed to one side

$$a = \frac{2L}{\pi^2 a_1 P} \left[\frac{\pi^4 EI}{4L^3} \sum_{n=1}^{n=\alpha} n^4 a_n^2 + \frac{\beta L}{4} \sum_{n=1}^{n=\alpha} a_n^2 - \frac{P\pi^2}{4L} \sum_{n=1}^{n=\alpha} n^2 a_n^2 + \sum_{n=1,3,5..}^{n=\alpha} \frac{2qL}{n\pi} a_n \right] \text{....(j)}$$

As for values of n other than 1 the last term of eq.(i) containing ' a ' does not exist, so the values of n althrough the eq.(j) will not have values other than 1.

$$\therefore \quad a = \frac{2L}{a_1 \pi^2 P} \left[\frac{\pi^4 EI}{4L^3} a_1^2 + \frac{\beta L}{4} a_1^2 - \frac{P\pi^2}{4L} a_1^2 + \frac{2qL}{\pi} a_1 \right] \quad \text{... (k)}$$

To obtain the maximum effectiveness of the longitudinal reinforcement, the initial deflection of the bars should be sufficient to permit development of yield point stress f_y just before buckling.

Long prisms subjected to compression, or short specimens with friction at the ends greatly reduced or eliminated, fail along plains parallel to the longitudinal axis, as shown in the Fig.5.2.

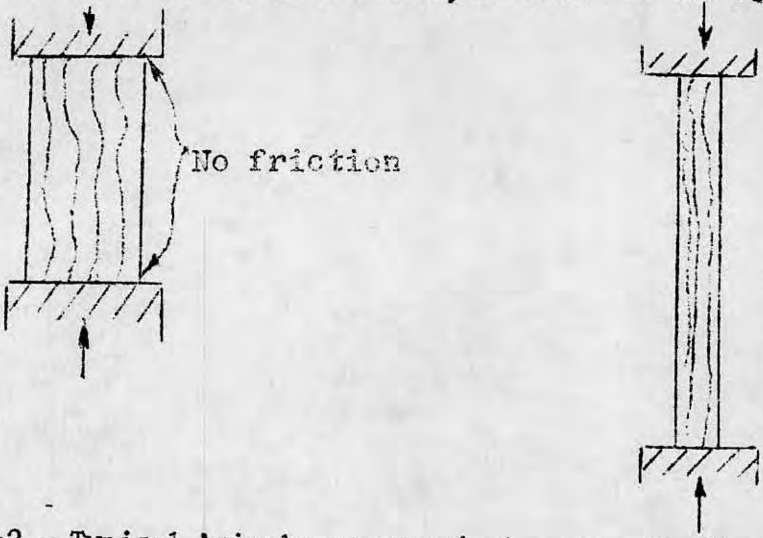


Fig.5.2 - Typical 'simple compression' or longitudinal splitting failure. (2)

The strength corresponding to the longitudinal splitting failure is about 20% lower than that of 'shear cone' failure which is generally found in short compression specimens laterally restrained at the ends of the testing machine (2). To obtain the maximum affectiveness of the concrete core, the steel bars should be initially curved so as to prevent failure by splitting. It the splitting failure develops all through the lateral dimension of the column, dividing it in two parts, the maginute of the lateral load q acting on the bar will be summation of (1) stress due to lateral strain just before crushing of the concrete, multiplied by the area formed by the width of the column per unit longitudinal length of the column and divided by the number of the steel bars on one side of the splitting plane, and (ii) the tensile bond stress between steel bars and concrete multiplied by half of the circumferential area of the bar per unit length. Accordingly the magnitude of lateral load q is given by

$$q = \frac{f''_c A'}{N} + t_f \, a_{sc} \quad \ldots\ldots\ldots \quad (L)$$

where q is as explained earlier,

f''_c = The stress developed in concrete due to lateral strain just before crushing failure of concrete

A' = Area formed by the width of the column per unit longitudinal length of the column

N = The number of the steel bars on one side of the splitting plane

t_f = tensile bond stress between steel bars and concrete

a_{sc} = half of the circumferential area of the bar per unit length.

Case B : After the loss of the concrete cover.

The mode of buckling of the longitudinal bars after loss of concrete cover does not differ much from that observed before the loss of concrete cover. In this case the steel bars will be subjected to an additional distributed axial load and will be relieved of the lateral load caused by the tensile bond between the bars and the concrete. The magnitude and distribution of the distributed axial load resulting from the loss of support rendered by the outer shell to the core concrete depends partly on the extent of spalling of shell concrete and partly on the arrangement size and initial curvature of the longitudinal bars. Considering the fact that the initial curvature is very small, the effect of distributed axial load is neglected in the analysis. Then the system of loading is same as before and the initial curvature can be represented by

$$a = \frac{2L}{\pi^2 Pa_1}\left[\frac{\pi^4 EI}{4L^3} + \frac{\beta L}{4} a_1^2 - \frac{P\pi^2}{4L} a_1^2 + \frac{2qLa_1}{\pi}\right] \quad \ldots\ldots \quad (L)$$

After spalling of concret cover, tensile bond force may act in opposite direction, if the bar buckles outward and in this case

$$q = \frac{f''_c A'}{N} - t_f a_{sc} \quad \ldots\ldots\ldots \quad (m)$$

DETERMINATION OF THE TIE SIZE

The size of ties placed at the two ends of the column is mainly governed by the lateral strain of the concrete and initial curvature of the longitudinal bars. The mode of failure of columns depends partly on the size and arrangement of lateral ties. Lateral displacement of the bar at the tie level would reduce appreciably the magnitude of the critical failure stress and thus would reduce load carrying capacity of the column. Therefore it is advisable to provide an arrangement of ties which will prevent the lateral displacement of the bar at the level of the tie. In the calculations for the initial curvature, as in the previous article, it was assumed that the lateral ties were sufficiently rigid so that the lateral displacement of the two ends of the bar was negligible.

Conforming to the conditions prevailing in the column under investigation, a longitudinal reinforcing bar having a length L is considered to have elastic 'sparing' support at the two ends with concrete shell of the column spalled off and a continuous support along one side which prevents inward buckling and is subjected to longitudinal force P, a lateral load q, equivalent lateral load $\frac{\pi^2 aP}{L^2}$ Sin $\frac{\pi x}{L}$.

Proper solution of the problem requires determination of the smallest value of the spring constant K of the end support which would prevent lateral displacement at the tie level. The final curvature of the bar is as in Fig.3.3, defined by $Y = a_1 \sin \frac{\pi x}{L}$ ..... (n)

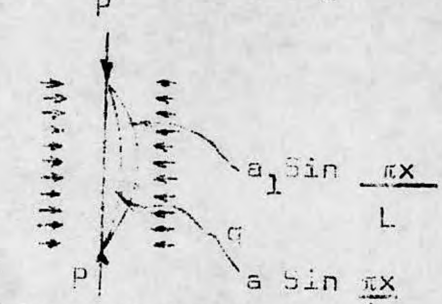


Fig. 3.3 - Loads acting on the bar.

Energy method is applied here to get the spring constant K as a function of a, P, L, q and EI. As in the last article, the strain energy of bending of the bar together with that of the elastic foundation is

$$\frac{\pi^4 EI a_1^2}{4L^3} + \frac{\beta L a_1^2}{4} \quad ..... \text{(o)}$$

The work done by the compressive force P, equivalent lateral load and the lateral load is, from eq(i)

$$\frac{P \pi^2 a_1^2}{4L} - \frac{2qL}{\pi} a_1 + \frac{Pa \pi^2}{2L} a_1 \quad ....\text{(p)}$$

The strain energy of the spring provided by the ties is $2x\frac{1}{2}Ka_o^2$ ....(p')

where a_o is the elongation in the tie.

Substituting the equations (n), (o), (p) and (p') in equation (b) the following equation is obtained,

$$\frac{\pi^4 EI}{4L^3} a_1^2 + \frac{\beta L a_1^2}{4} + Ka_o^2 = \frac{P\,^2 a_1^2}{4L} - \frac{2qLa_1}{\pi} + \frac{P\pi^2 aa_1}{2L}$$

........(q)

From Eq.(q), K may be written as

$$K = \frac{1}{a_o^2}\left[\frac{P\pi^2 a_1^2}{4L} - \frac{2qL}{\pi} a_1 + \frac{Pa\pi^2}{2L} a_1 - \frac{\pi^4 EI}{4L^3} a_1 - \frac{\beta L}{4} a_1^2\right] \quad \text{.... (r)}$$

As the longitudinal bars are supported by direct constraint of a tie at the ends and intermediate elastic foundation, the value of a_o may be assumed to be the strain as the proportional limit of the tie bars. Assuming that the tie acts as an elastic bar, its stiffness is defined by $K = \frac{A'E'}{b'}$ where A' is the effective cross sectional area of the tie, b' is the effective length of the tie. Equating the two expression for K,

$$K = \frac{A'E'}{b'} = \frac{1}{a_o^2}\left[\frac{P\pi^2}{4L} a_1^2 - \frac{2qL}{\pi} a_1 + \frac{Pa\pi^2}{2L} a_1 - \frac{\pi^4 EI}{4L^3} a_1^2 - \frac{\beta L}{4} a_1^2\right] \quad \text{....(s)}$$

$$A' = \frac{b'}{E' a_o^2}\left[\frac{P\pi^2}{4L}a_1^2 - \frac{2qL}{\pi}a_1 + \frac{Pa\pi^2}{2L}a_1 - \frac{\pi^4 EI}{4L^3}a_1^2 - \frac{\beta L}{4}a_1^2\right] \quad ..(t)$$

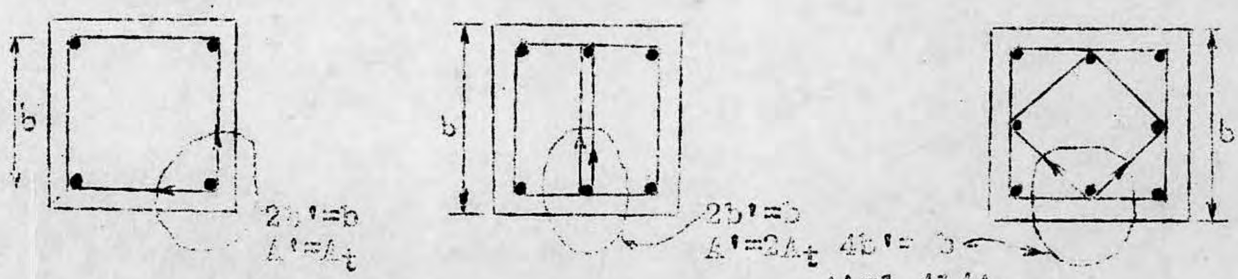
According to Bresler and Gilbert(2) the effective area A' and the effective length b' may be related to the tie-bar cross sectional area A_t and the core dimension b as follows :

$$\frac{A'}{b'} = m\left(\frac{A_t}{b}\right) \qquad \text{i.e.} \qquad A' = mb\ (A_t / b)$$

$$A_t = \frac{b}{mE' a_o^2}\left[\frac{P\pi^2}{4L}a_1^2 - \frac{2qL}{\pi}a_1 + \frac{Pa\pi^2}{2L}a_1 - \frac{\pi^4 EI}{4L^3}a_1^2 - \frac{\beta L}{4}a_1^2\right] \quad ...(t')$$

For the common arrangement of ties, the values of b', A' and m are shown in the following figure :

$\frac{1}{2}m = 1.00$ $\qquad\qquad$ $\frac{1}{2}m = 2.00$ $\qquad\qquad$ $\frac{1}{2}m = 2.82$



The theoretical investigation indicates that the following variables have some influence on the behaviour and carrying capacity of concrete columns reinforced with initially curved bars :

1. Initial curvature of the longitudinal bars
2. Size, spacing and arrangement of the reinforcing bars
3. Shape and size of column
4. Compressive strength and stress strain characteristics of concrete
5. Tensile strength of concrete
6. Size and arrangement of ties
7. Strength and stress-strain characteristics of steel bars

8. Eccentricity of applied load
9. Slenderness ratio of the column
10. Thickness of shell concrete
11. Area of tensile and compressive steel in the case of eccentrically loaded columns
12. Modulus of elastic foundation, rendered by concrete to the steel bars.
13. Tensile bond strength between steel bars and concrete
14. Workability and curing.

Tensile Bond Strength

After the spalling of the concrete cover, the straight longitudinal bars will have a tendency to buckle. Tensile bond at the interface of the rod and the concrete is one of the resistances to buckling. In the column under investigation, concrete shell generally spalls off along a surface passing through the central line of the rod. After the spalling nearly half diameter remains embeded into the concrete core. As explained earlier, tensile bond should be considered in the buckling phenomena of the bars. So tensile strength of concrete and tensile bond strength were determined in this investigation. Two methods were used to find the tensile bond strength. In the first method the rod conforming in diameter to the rods used in the columns was placed centrally in a concrete cylinder during casting. The cylinder with rod inside was subjected to the lateral load. The load was applied parallel to a transverse axis of the specimen in the same manner as in the standard split cylinder test. The tensile bond specimen with rod inside is shown in the Fig. 4.13. Failure surface is assumed to hit the rod at the mid line and tend to divide it into two half cylinders. Since the M. S. rod has a tensile strength much higher than the bond strength, failure will not occur through the rod but will encircle around it. The surface area of the rod is subjected to combined tension and shear. Assuming that only tension exists at the interface, (it is necessary to make such an assumption, since no information on combined tensile and shear bond strength is available), the following equation for the value of the part of the load carried by the tensile bond results from the equation of elasticity for tensile strength of concrete.

$$P_2 = P_1 - \left(P - \frac{2P}{\pi DL} \times dL \right) \quad .... (u)$$

where P_2 = Part of the load carried by the tensile bond

P_1 = Total load at failure of the cylinderical specimen with the rod inside

P = Total load at failure during standard split cylinder test.

D = Diameter of the rod

L = Length of the cylinder.

Hence the tensile bond stress at failure $= \frac{1}{dL} \left[P_1 - \left(P - \frac{2Pd}{\pi D} \right) \right] \quad ..(v)$

The actual mechanism of failure is however much more complicated. A few consideration made below justify this statement.

(i) The failure surface does not necessarily hit the rod along the mid line of the rod; it will follow the weakest path through the concrete mix.

(ii) The surface of contact is not subjected to pure tension, but subjected to combined tension and shear. It is likely that the combined tensile and shear bond strength is higher than the tensile bond strength. However, no information on shear strength of concrete is available to date.

(iii) The tensile bond strength is a function of surface roughness and shape of the rod.

For the simplification of the problem, the above considerations were not made in the derivation of the above formula.

The tensile bondsstrength can also be determined by the method illustrated in Fig. 4.5 in which a moment was applied to a two unit couplet by loading lever arm clamped to the top unit mainly of the rod. The rod is embeded half diameter into concrete block. This method is an adaptation of a method previously used by Saeman and others in masonry structures. Tensile bond is determined by the formula

$$s = \frac{LD}{CA}$$

where s = average tensile bond strength, psi.

L = applied load, lbs. (including equivalent weight due to lever arm dead weight)

D = lever arm, in.

C = distance between centre lines of the contact area and the pivot, in.

A = effective contact area between the steel bar and concrete, sq. in.

CHAPTER IV

LABORATORY INVESTIGATION

The laboratory investigation was carried out in three distinct phases. The first phase was concerned with the determination of physical properties of materials used in this investigation. These include specific gravity and absorption capacities of coarse and fine aggregates, yield strength of steel, compressive and tensile strength of concrete, tensile bond strength between concrete and steel bars, modulus of elastic foundation and poisson's ratio of concrete. The second phase consists of placing curing and testing of concrete columns. The second phase was subdivided into five series of tests involving a total of forty columns of square and rectangular shapes having corss-sectional area equal to thrity six square inches. Each column was identified by two letters and a number. The first letter identified the group, the second letter the type of reinforcement and the third the serial number in the group. All tests were performed in the structural and concrete laboratory of civil Engineering department of East Pakistan University of Engineering and Technology, Dacca.

Concrete

Brick aggregate and sand were used for concreting. First class bricks were crushed to conform to the A.S.T.M. gradation requirement. Local Savar sand was used as fine aggregate and screened before use. The physical properties of the aggregates are shown in Table 4.1 & 4.2. Ordinary portland cement of Gilpak and Five Ram brands were used for concreting. Nominal concrete strengths of 2000 psi, 2500 psi and 4000 psi at 28 days were used for the columns, proportioning of the mixes was done by weight of the day materials as given in table 4.3.

TABLE 4.1 - PHYSICAL PROPERTIES OF AGGREGATES

Properties	Brick Aggregates	Sand
Eineness Modulus	6.08	2.30
Unit weight, lb.per cft. (Dry loose)	59.00	102.00
Absorption ** (per cent dry weight)	14.00	2.98
Bulk specific Gravity	1.85	3.87

** Absorption at the end of three days

Table 4.2 - GRADING OF AGGREGATES

Sieve Size	Coarse Aggregate (per cent retained)	Fine Aggregate (per cent retained)
3/4 in.	16.3	0.0
3/8 in.	85.0	0.0
3/16 in	97.0	0.48
No.8	99.91	0.48
No.16	99.92	8.48
No.30	99.93	42.92
No.50	99.95	81.48
No.100	100.00	97.28
	608.01	230.007
	F. M. = 6.08	F. M. = 2.307

TABLE 4.3 – PROPORTIONING OF CONCRETE MIXES

Design Strength (28 day) lb. per sq. in.	Proportions by weight			
	Cement	Sand	Brick Aggregate	Water/Cement ratio
2000	1	4.0	4.5	0.65
2500	1	3.5	4.0	0.58
4000	1	2.0	2.5	0.40

Reinforcement

Properties : Mild steel plain bars were used for longitudinal and lateral reinforcement. Two specimens of each bar size were tested in tension to find its yield point strength and modulus of elasticity. Tensile properties of the bars are tabulated in Table 4.4.

Initial curvature : For large number of tests the reinforcements were given initial curvature at mid height which varied between the range 0.0625 inch to 0.500 inch.

Size : The longitudinal reinforcement consisted of four bars of diameters ranging from 0.500 inch to 0.875 inch and 0.25 inch ties were used to hold the longitudinal reinforcement which were in some cases welded to it.

Fabrication : Proper initial curvature was given to the longitudinal reinforcement by tieing them closer at the middle. Studs were welded to the reinforcement to form two inch gage length and help in the measurement of the lateral movement of the steel bars during testing. The studs were coated by plastic tubes during casting. The removal of the tubes left a space in between the studs and the concrete. This system ensured proper measurement of longitudinal strain and lateral movement of the steel bars. The studs were placed at a distance of 15 inches from the ends and mid-height of the column. Studs were used also to hold the reinforcement in position during casting.

TABLE 4.4 - PHYSICAL PROPERTIES OF REINFORCEMENT

Nominal dia. inches	Yield strength lb.per.sq.in.	Ultimate strength psi.	Young's Modulus of Elasticity, million psi.
.250	49100	70800	30.00
.500	49200	70700	30.00
.625	46000	69200	29.95
.750	44800	68250	29.99
.875	45450	67820	29.97

Form work and casting of columns

Some previous investigators like Professor R. H. Evans (4) observed that columns in which end surfaces were given a finish after casting for evenness failed at loads which average 10 per cent below the theoretical loads. In this investigation attempt was made to avoid preparation of column after casting. For uniformity, columns of each group were cast from one mix.

Form work of columns : Form works with hoppers mounted at the side near the top of the mold as shown in Figs. 4.4. & 4.5, were used for the vertical casting of square columns. Each of the two sets of the form work with hoppers consisted mainly of twelve 2' ft. 6 inch long, 2 inch by 6 inch channels and 0.125 in. thick four 10 inch by 60 inch plates. Four inch square parts of the upper channels were removed to accomodate the hopper. A top plate and a base plate whose planes were perpendicular to the longitudinal axis of the forms were fitted to it. Assembly of the form was made by placing the different parts around the reinforcement. The outer walls had holes of $\frac{1}{2}$ inch diameter in the proper position to accomodate the studs coated by plastic tubes.

Few square and all rectangular columns were cast horizontally in forms as shown in Fig. 4.6. Mainly 1 inch thick wooden plank and 0.25 inch thick steel plates formed the molds.

Preparation of concrete : Concrete for all the specimens was prepared in a nontilting drum type concrete mixer with a capacity of mixing 6.5 cu.ft. of concrete at a time. Practice recommended in A.C.I. standard 614-59 was followed in measuring, mixing and placing concrete. The concrete was vibrated in the mold by a vibrator having long flexible shaft.

Casting of columns : In the vertical casting of square columns fitting the form by parts and filling them with concrete followed each other. When the mold was filled nearly to the full height top plate was placed and concrete was pushed through the hopper by internal vibration to fill up the space left below the top plate. After 24 hours the parts of the hopper and overhanging parts of concrete were stripped gradually.

In the horizontal casting the top surface of concrete was finished with a trowel to give a uniform and plane surface.

Fabrication of Test blocks

Fabrication of specimens for compression test : Methods of casting, curing and testing of 6 inch cubes were followed as per specifications made in B.S.1881:1952.

The casting of 6 inch by 12 inch concrete cylinders was made by two different methods. One method was followed according to A.S.T.M. C-192-57. The top surface of a standard cylinder finished with a float is not smooth enough for testing and requires further preparations. Neat cement paste is generally used for finishing. To alleviate the difficulty, investigation was made to use an alternative means of casting cylindrical specimens. After the top surface of the cylinder was finished by a trowel a plate was placed at the top and the whole mold was inverted upside down and kept for 24 hours before the mold was stripped.

Fabrication of test specimen for modulus of elastic foundation.

For determining the modulus of elastic foundation, two steel bars were embeded half diameter into each of two opposite faces of a 4 inch by 4 inch by 12 inch concrete prism. The mold and specimen are shown in Figs. 4.7. & 4.12. The steel bars were embeded half diameter into wooden wall of the mold during casting and were exposed after stripping off the mold. Two specimens were prepared along with each group of columns.

Fabrication of specimens for
tensible bond strength :

The first method for the determination of tensile bond strength at the interface of concrete and steel bars is similar to split cylinder test for tensile strength of concrete. In this method the rod having same dimension as used in columns was gripped along central axis of the 6 inch by 12 inch cylindrical mold by the use of holes in the bottom plate and top plate bar as shown in Fig. 4.8.

In the second method, one 2 inch long rod welded to a 2 inch by 3 inch by 3 inch angle was embeded half diameter into the top of 6 inch cubic concrete during casting.

Two specimens of each type were prepared along with each group of column.

Curing of columns

Each column and test block were removed from the mold 24 hours after casting and were cured with wet jute bags for 7 days after which they were exposed to the air following the common practice in the construction work.

Test Specimens

The details of the test specimens of columns were prepared in tabular form in Table 4.5.

TABLE 4.5 - TEST SPECIMENS

Group	Column No.	Type of Reinforcement	Target cylinder strength (psi)	Initial deflection of bars in inches	Amount of steel, 4 nos. of bars with diameter of	Steel in percent of concrete area	Length of column in inch.	Cross sectional dimension of column	L/D ratio
A	AS-O	Curved bars		.125	.625"		60"	6" x 6"	1
	AT-O	Straight bars tied by $\frac{1}{4}$" dia ties @ 6" c/c	2500	-	.625"	3.40	60"	6"x6"	10
	AC-O	Straight bars							
B	BS-O	Curved bars							
	BC-O	Straight bars	2000	.125	.50"	2.17	60"	5"x7.2"	12
C	CS-O	Curved bars							
	CO-O	Straight bars	4000	.125	.625"	3.40	60"	6"x6"	10
D	DS-1	Curved bars	2500	.062	.625"	3.40	60"	6"x6"	10
	DS-2	Curved bars		.125					
	DS-3	Curved bars		.187	,,	,,	,,	,,	,,
	DS-4	Curved bars		.25	,,	,,	,,	,,	,,
	DS-5	Curved bars		.5	,,	,,	,,	,,	,,
	DS-6	Straight bars		.0	,,	,,	,,	,,	,,
					,,	,,	,,	,,	,,
E	ES-1	Curved bars		.0937	.625"	3.40			
	ES-2	Curved bars	4000	.0937	.75"	4.90	60"	6"x6"	10
	ES-3	Curved bars		.0937	.875"	6.68			
P	P-1		2500				60"	6"x6"	10
	P-2		2000				60"	5"x7.2"	12
	P-3		4000				60"	6"x6"	10
	P-4		2500				60"	6"x6"	10
	P-5		4000				60"	6"x6"	10

Testing

Testing of columns : The lateral movement of longitudinal steel bars and lateral bulging of concrete were measured by deflectometers which were put in contact with steel studs and concrete surface. Measurement of longitudinal strain of steel bars were made on the 2 inch spaced steel studs kept for the purpose. Longitudinal deformation of columns were measured at quarter and three quarter height by 10 inch Whittemore strain gauge and at mid height by deflectometers fitted with sides of a frame. Lateral strains were measured by 2 inch Whittemore strain gauge. A set of reading were taken for each measurement and the average of all the readings were recorded.

The columns were tested to failure in a 200 ton universal testing machine. Before each column was loaded, a complete set of strain gage readings was taken to check the presence of any unevenness. When the strain gage readings on the two opposite faces did not tally with each other, the column was removed from the machine, and prepared to remove eccentricity prior to final testing. The load was applied in increaments of approximately 5000 lbs. A complete set of strain observations was made for each load.

Test for compressive strength of concrete : Fig. - 4.1 shows the relation between cube strength and cylinder strength of concrete made of brick aggregates. Upto approximately 3000 lbs. per sq. inch of cylinder strength the ratio of the cylinder strength to the cube strength is about 0.75. Beyond 3000 psi of cylinder strength, the ratio increases and becomes about 0.8 for cylinder strength of 4000 psi.

The average strengths of control cylinders of different group of columns is shown in Table 4.6. The strengths noted are the average of compressive strength of six cylinders.

TABLE 4.6 - STRENGTH OF CONTROL CYLINDERS

Group of Column	Design strength	Average Cylinder strength
A	2500	2380
B	2000	2080
C	4000	3960
D	2500	2460
E	4000	4200

Test for Modulus of elastic foundation : The specimen with four rods half embeded into opposite surfaces of the specimen was placed in between the platens of the testing machine. The rods were in contact with the platens of the testing machine. The strains were recorded at intervals of load increaments of 200 lbs.

Fig. 4.20 shows the plotting of the load per unit length of the rod and the deflection for the determination of modulus of elastic foundation.

Split cylinder test for tensile strength of concrete and tensile bond strength : Specimens for tensile splitting tests were 6 in. by 12 inch concrete cylinders cast and cured in accordance with ASTM C-192. As the diameter of the top platen of the machine was shorter than the length of the cylinder, a $\frac{1}{4}$ inch thick steel plate stiffened by angles was placed over the cylinder. Rubber strips 1/8 inch thick 1 inch wide and 12 inch long were placed along the planes of contact of the cylinder with the top plate and hottom platform. These strips were soft and conformed to the small irregularities on the surface of the specimens, thus allowing a more uniform distribution of stress along the concrete cylinder. The specimen then was loaded to failure, with the load applied at the rate of 30,000 lbs per min. This is equivalent to a unit stress over the longitudinal section of the cylinder of about 265 psi. per min. The tensile splitting strength was computed from the following equation which defines the theoretical stress distribution for a homogenious materials

$$T = \frac{2p}{\pi DL}$$

Where T = Tensile splitting, strength, psi

P = Load at rapture, lbs.

D = Diameter of cylinder, in

L = Length of cylinder, in.

After the failure of the tensile bond specimen the amount of the exposed surface of the central rod was noted.

Two unit couplet method for tensile bond strength : This is the second method applied in this investigation for the determination of tensile bond strength. The concrete cube in which the experimental rod was embeded was clamped with a heavy basement. The free leg of the angle, welded to the experimental bar was connected to a lever arm. Some weights around 50 lbs were put into a bucket ... from the other end of the lever arm. Then water was poured into the bucket until the failure of the tensile bond at the interface of the rod and the concrete occured. The weight of the bucket with water and weights inside was measured by a spring balance.

Table 4.7 & 4.8 show the tensile bond strengths by two methods. The tensile bond strengths determined by both the methods increased with the increase in the compressive strength of concrete. The strength determined by split cylinder method was a bit lower than that by two unit couplet method.

TABLE - 4.7

TENSILE BOND STRENGTH DETERMINATION

BY SPLIT CYLINDER TEST

Compressive strength psi	Load P Kips	Load P_1 Kips	Tensile Strength psi.	Tensile Bond Strength psi.
2300	30.10	28.76	260	100.2
3380	38.00	36.40	337	120.0
3600	49.00	46.76	433	128.1
3960	56.50	54.00	500	133.1

TABLE -4.8

TENSILE BOND STRENGTH DETERMINATION

BY TWO UNIT COUPLET METHOD

Cylinder Strength psi	L, Value of applied load		A, contact area, sq.in.	Average Tensile bond strength, psi.
	Observed	Average		
2300	129,131	130	1.25	104
3380	152,156	154	1.25	123
3600	160,164	162	1.25	130
3960	171,179	175	1.25	140

CHAPTER - V

TEST RESULTS

Summary of Test Results

The test results are presented in Table 5.1 and load-strain curves are given in figures of Appendix - A.

Table 5.1 shows that the maximum longitudinal strain observed in tied column was about 0.0015. However, it was not the ultimate strain. They were in fact measured while the column was still holding its ultimate load and was the last possible observation made before it collapses. A decrease in load of about 40 percent of the ultimate load was usually observed before the column with longitudinal rods having initial curvatures failed. These hold their ultimate loads for a minute or two, during which time a considerable shortening was observed. If the ultimate load was reached when the load was being increased it was only possible to record the strains just before or at ultimate load.

The lateral strains measured in the columns with straight bars are shown in Fig. 5.40. The poisson's ratio was determined by refering the lateral strain to the longitudinal strain for the same load. The poisson's ratios were found to be within the range of 0.15 to 0.199. The strain measurements could not be made accurately because only 2 inch strain gages having a least count of 0.0001 inch were available.

The bulging of square and rectangular columns are shown in Figs 5.20 and 5.30. The readings indicated that the rods in the top part of the column moved laterally a short distance. The bulging occured similarly in square and rectangular columns under only small loads.

The longitudinal strains measured in columns of Groups A and B are shown on the unit load strain curves of Fig. 5.50 & 5.60. The strains in concrete and steel at the different heights of the column were observed to be similar, except near the failure load.

TABLE 5.1 - SUMMARY OF TEST RESULTS

Column	Cylinder strength psi	Load at 1st crack, kips	Strain at 1st crack,	Max. load kips	Strain at max. load	Load at final failure kips	Mode of failure
AS	2380	120	.0011	142	.0032	82	Spalling
AC	2380	105	.00065	120	.00125	20	Splitting
AT	2380	131	.00135	131	.0015	00	Shearcone
BS	2080	90	.0010	113	.0035	65	Spalling
BO	2060	90	.0014	97	.0020	62	Splitting
CS	3960	150	.0011	180	.0030	2[illegible]	Spalling
CO	3960	160	.0014	164	.00135	00	Buckling of rod
DS-1	2470	110	.0016	145	.0048	90	Spalling
DS-2	2470	100	.0017	133.7	.0029	10	Shearing
DS-3	2470	120	.0014	142	.0035	70	Spalling
DS-4	2470	115	.0019	137	.0028	15	Shearing
DS-5	2470	115	.0022	142	.0034	73	Spalling and rod buckling
DO-6	2470	110	.0010	117	.0012	00	Buckling of rod
ES-1	4200	170	.00165	246.3	.00295	155	Spalling
ES-2	4200	190	.00150	216.7	.0038	123	Spalling
ES-3	4200	160	.00141	196	.0042	105	Spalling

Behaviour of Columns under Loading

The column in general did not show non-homogeneity or large eccentricity of loading at any particular section. There was no particular part at which all the columns failed. Figures 5.1 to 5.6 show the appearance of some columns after failure. The behaviour of different columns during failure is described below.

Columns AS, BS, DS-1, DS-3, DS-5, ES-1, ES-2 and ES-3 with initially curved rods failed generally at the top and bottom. Little lateral deformation was noted until the yield point of the steel in the column was reached and no appreciable spalling occured until shortly before the ultimate load was applied. After the spalling of concrete cover the loading dial showed a downward trend and remained fixed at an average of sixty per cent of the ultimate load and the column underwent high longitudinal strain. No buckling of the rods was observed after the failure of the column.

In Columns-AT with tied bars no crack was observed on the specimens until failure load was reached. As soon as the first signs of crushing appeared on the lateral surface, the specimen failed without any warning. The concrete in the core of the specimen showed signs of crushing with the characteristic cones, typical of a compression failures in a prismatic concrete specimen as shown in Fig. 5.1(c). The failure showed some signs of buckling of the reinforcement. However it was believed that this was a secondary effect of the crushing failure of the concrete. The ties did not show any sign of tension failure.

In one of columns CS with initially curved bars the failures started with a crack which was inclined at an angle of about 30° with longitudinal axis. The failure plane then propagated downward by splitting along the plane of two rods. Only one rod buckled within the exposed length. In the other specimen which is shown in Fig.5.3(a) all the longitudinal rods buckled.

In columns AO, BO, DS-6 with straight bar and end ties only concrete compressive stress at ultimate strengths were $0.76f'_c$, $0.72f'_c$ and $0.718f'_c$ respectively. The modes of failure are shown in figures 5.1(b), 5.2(b) and 5.5(f). In these columns without intermediate tie, longitudinal splitting was observed practically over the length of each column. The crack appeared at the top and extended downard along the longitudinal axis. The loading dial indicated maximum loading when the crack reached the mid height of the column. The splitting crack widened and propagated until it came within 1/5th of height of the bottom part. The split up parts behaved as long columns with characteristic buckling.

The columns DS-2 and DS-4 having initially curved bars and 3.4 per cent of longitudinal reinforcement failed with a wedge formation. These columns failed shortly after the formation of an inclined crack. The failure in Column DS-2 occured at the middle. As illustrated in Fig.5.5(b) the upper wedge formed a seat in between two wedges and splitted the lower concrete along the central line. The splitting was a secondary effect of the movement of the upper wedge. The failure in column DS-4 occured within the lower half of the column. The plane of failure made an angle of 30^{o} with the longitudinal axis of the column.

Concrete columns without reinforcement failed either by shearing diagonally or by shearing into a wedge which splits the surrounding concrete. Failure was violent, specially for the higher strength concrete. The plane of failure generally made an angle of about 30 degrees with the axis of the column as illustrated in Fig.5.6. The columns with 4000 psi concrete failed abruptly almost without warning. In the columns with 2500 lb. concrete, the failure was gradual.

CHAPTER - VI

ANALYSIS AND DISCUSSION OF TEST RESULTS.

The ultimate strength of the plain columns and their control cylinders and the strength of the columns stated as a percentage of the strength of the control cylinders, are shown in Table 6.1. The average strength of the plain columns was found to be approximately 85% of the strength of the cylinders.

The stress-strain curves for the cylinders and the columns tested are given in Fig.6.19,6.20 and 6.30 for the two different strengths of the concrete. Each curve is the average for all the specimens of its kind. There is close agreement between the curves for the cylinders and the columns upto a stress of 800 psi and consistantly greater deformation in the columns as the stress is increased.

TABLE - 6.1

RESULTS FOR PLAIN COLUMNS

Column	Ultimate load (Av.of two) Kips.	Column strength CP Psi	Cylinder Strength Cu.(Av.of six)Psi	$\frac{CP}{Cu}$
AP	78	2170	2380	91.00
BP	60.1	1670	2080	80.01
CP	121.3	3360	3960	85.00
DP	74.8	2070	2460	84.00
EP	128	3550	4200	85.00

Fig.6.80 shows the relation between the strength of the reinforced columns and the sum of the strength of the concrete and the reinforcement used. All the reinforced columns used in this figure had 3.4 percent longitudinal reinforcement. The sectional area of the concrete within the outer circumference of the column, that is, the total concrete area minus the sectional area of the longitudinal bars was 34.76 and the sectional area of the plain columns was 36. sq. inch. The plotted values in the lower curve are 34.76/36.0 times the strength of the plain columns, and represent the strength of the concrete in the reinforced columns. To these strengths have been added the total yield point strengths of the longitudinal reinforcement. The strengths thus obtained on columns reinforced with intermediate grades of reinforcement are shown in dotted lines. The strengths of the columns so reinforced but having the reinforcing bars with an initial deflection of 0.125 inch at mid-height are shown as solid lines. Based upon the assumption that the longitudinal reinforcement is stressed to its yield point strength in the column, the vertical distance between the solid and the dotted lines for each grade of steel represents the strength added by the curvature of the reinforcing bars. The indication of the foregoing paragraphs is that the strength of the column having initially curved longitudinal bars was made up of (1) 85 percent of the cylinder strength times the cross sectional area of concrete (2) the yield point stress of the longitudinal bars times their area and (3) the strength added by the curvature.

It has been assumed that the steel stresses are initially zero but there will be some compressive stresses caused by shrinkage of columns while drying out. Shrinkage has no effect on the ultimate load of a column and the strains recorded are the strains measured during the tests.

In Fig. 6.4 to 6.7 inclusive, the average deformation are plotted for the different loads on the columns. In addition, the loads corresponding to the strains in the steel are represented by a broken line. The assumption made was that the concrete, the longitudinal steel and the confinement to the concrete by the initially curved bars contributed to the load carrying capacity of the column. The assumed division of the load carried by each of these factor is shown on the diagram. The above data has been presented in a tabular form in Table 6.1.

Table 6.1 shows that over the summation of the loads carried by the concrete and steel, Group A (square with 2380 psi concrete) columns show an increase in capacity by 7.6%, Group B (rectangular with 2080 psi concrete) columns 9.2%, Group C (square with 3960 psi concrete) columns 3.45% column ES 1 (square with 6.68 p.c. of steel and 4200 psi concrete) 7.9% column ES-2 (square with 4.9 precent of steel and 4200 psi concrete) 7.8% and column ES-3 (square with 3.4 percent of steel and 4200 psi. concrete), 8.8%.

The percentage was lowest for the columns of group-C probably becanse of the failure of the ties at the ends and the violent failure of the concrete.

In the columns DS-1 and ES-3 in which there was no failure of the ties and only spalling of the shell concrete occured the increase of capacity over the load carried by steel and concrete was observed to be the highest.

It is observed that in case of straight bars with proper and

Table 6.1 DIVISION OF LOAD BETWEEN CONCRETE AND REINFORCEMENT

Group	Column No.	Initial deflection in inches	Cylinder strength	Percentage of steel	Load carried by the column		Plain column strength	Load carried by (in kips)		Increase/Decrease of capacity above columns 8&9		Failure of ties.
					Kips	Percent above with straight bar & ties.		Concrete	Steel	Kips	Percent	
1	2	3	4	5	6	7	8	9	10	11	12	13
A	AS	.125	2380	3.4	142	118	78	75.0	57	10	7.6	NO
	AT	-	2380	3.4	133	111	78	75.0	57	1.0	0.76	NO
	AO	-	2330	3.4	120	100	78	75.00	57	-12.0	-9.1	NO
B	BS	.125	2080	2.18	113	116.5	63.1	65	38.5	9.5	9.2	NO
	BO	-	2080	2.18	97	100	63.1	65	38.5	-6.5	-6.3	NO
C	CS	.125	3960	3.4	180	110	121.3	117	57	6	3.45	Yes
	CO	-	3960	3.4	164	100	121.3	117	57	-10	-5.75	NO
D	DS-1	.0625	2470	3.4	145	124	74.8	72	57	16	12.4	NO
	DS-2	.125	2470	3.4	133	114	74.8	72	57	1.7	3.65	NO
	DS-3	.1875	2470	3.4	142	121	74.8	72	57	13	10.1	Yes
	DS-4	.25	2470	2.4	137	117	74.8	72	57	8	6.2	NO
	DS-5	.50	2470	3.4	142	121	74.8	72	57	13	10.1	Yes
	DS-6	-	2470	3.4	117	100	74.8	72	57	-12	-9.3	NO
E	ES-1	.0937	4200	6.68	246.3	-	128	120.0	108.3	-18	7.9	NO
	ES-2	.0937	4200	4.9	216.7	-	128	122	79	15.7	7.8	NO
	ES-3	.0937	4200	3.4	196	-	128	124	57	15.9	8.8	NO

ties the lateral straining of the concrete was not restrained. The loss of capacity due to the lateral straining of the concrete was 5.75 to 9.30 percent of the load carried by the concrete and the reinforcement. The lateral straining resulted in the premature splitting failure of the columns reinforced with straight bars and end tie only.

In comparison with columns with straight bars and end ties load carrying capacity of columns with initially curved bars and ties is higher by about 10 to 21 percent.

In Table 6.2 an analysis has been made with the experimental data for the determination of the initial mid deflection of the longitudinal bars, with its end tie requirement for maximum column capacity. Analysis has also been made for the tie requirement with the supplied initial curvature. In the analysis it was assumed that at the maximum load, there is no change in the maximum initial given deflection of the bars at mid-height due to lateral strain in the concrete.

The analysis given in the Table 6.2 shows that to satisfy the above assumption the initial mid-deflection of the bars lies between .056 inch to .093 inch within the limits of the investigation, the minimum value being for the rectangular columns. It is observed that the supplied initial curvature was more than the minimum required in all the columns.

The tie requirement for the minimum necessary mid deflection of the bar falls in the range of 0.00-0096 sq.inch. Of course small increase in the deflection over the minimum required one results in exeessive increase in the tie requirement which is in the range of .0170 to 2.48 sq. inch in this investigation. It is observed that only two columns namely DS-1 and ES-3 are adequately reinforced with tie for the supplied initial curvature of the longitudinal bars. When the ties are distributed, instead of being supplied at one section, the tie requirement fall in the range of 1-.25 inch dia. to 4-.25 inch dia. tie @ 5 inch c/c near each end. These amounts of ties are necessary when the strain in the tie is equal to the strain at propertional limit. If the supplied amount of tie is less than the required, the strain in the tie is expected to exceed the yield point but does not approach the strain at failure of the tie. Before the strain in the tie approach failure, the concrete will crack resulting in low load carrying capacity. In this investigation the end ties of three columns CS, DS-3, 5 failed at the welded joint probably because of improper welding.

In the Table 6.1 with theoretical analysis it is observed that excepting the column DS-1 and ES-1 with an initial deflection of .0625 of longitudinal bars all the other columns are inadequately tied. All the columns were inferior in comparison to the columns DS-1, and ES-3, probaly due to the reason that strain in the tie exceeded the strain at proportional limit.

In the table of summary of test results, the load at the first crack of all the columns are noted. The load was increased in increaments of 5 kips and cracks were only observed in reaching a definite loading stage. Consequently the load at the first crack might be as much as 5 kips smaller than the observed load.

For the columns reinforced with initially curved bars, the ratio

TABLE 6.2-THEORETICAL ANALYSIS OF EXPERIMENTAL DATA

Column No.	AS	BS	CS	DSI	DS2	DS3	DS4	DS5	ES1	ES2	ES3
Concrete strength, lbs. per sq. in	2380	2080	3960	2470	2470	2470	2470	2470	4200	4200	4200
Diameter of rod, lbs	.625	.500	.625	.625	.625	.625	.625	.625	.875	.750	.625
E. million psi.	29.9	30.0	29.9	29.9	29.9	29.9	29.9	29.9	29.9	29.9	29.9
Et. million psi	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
Ec. Million psi	3.42	3.25	3.60	3.30	3.30	3.30	3.30	3.30	3.90	3.90	3.90
Unsupported length. inches	60	60	60	60	60	60	60	60	60	60	60
6_{YI} Kips per sq.in.	46.0	49.0	46.0	46.0	46.0	46.0	46.0	46.0	45.4	44.8	46.0
6_{Y_2}, Kips per sq.in.	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0
Strain at proportional limit of tie bars, millionth	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700
Load P. Kips	14.2	9.84	14.2	14.2	14.2	14.2	14.2	14.2	27.6	19.6	14.2
Lateral strain before splitting, millionths	130	100	150	110	110	110	110	110	165	165	165
Tensile bond strength, lb.per sq.in.	100	99	140	100	100	100	100	100	145	145	145
Load q before spalling lbs.	1402	860	1707	1158	1158	1158	1158	1158	2052	2034	2015
Load q after spalling, lbs.	1277	760	1532	1032	1032	1032	1032	1032	1798	1916	1875
B. lbs.per sq. in.x106	3.75	3.30	4.05	3.70	3.70	3.70	3.70	3.70	5.20	4.65	4.16
Total width, inches	6	5	6	6	6	6	6	6	6	6	6
Core width, inches	4	3	4	4	4	4	4	4	4	4	4
Value of m x 1/2	1	1	1	1	1	1	1	1	1	1	1
I, moment of inertia of longitudinal steel, hundredth of in.	7.50	3.07	7.50	7.50	7.50	7.50	7.50	7.50	3.27	1.56	7.50
a. required initial deflection, from Eq.(k)	.067	.056	.080	.057	.057	.057	.057	.057	.068	.079	.091
Necessary tie for required initial deflection, sq.in.	.0038	.0027	.0043	-	-	-	-	-	.0095	.0046	.0051
Supplied initila deflection of longitudinal steel inches	.125	.125	.125	.063	.125	.188	.250	.500	.125	.125	.125

Et. million psi	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	
Ec. Million psi	3.42	3.25	3.60	3.30	3.30	3.30	3.30	3.30	3.90	3.90	3.90	
Unsupported length. inches	60	60	60	60	60	60	60	60	60	60	60	
σ_{YI} Kips per sq.in.	46.0	49.0	46.0	46.0	46.0	46.0	46.0	46.0	45.4	44.8	46.0	
σ_{Y}, Kips per sq.in.	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	49.0	
Strain at proportional limit of tie bars, millionth	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	1700	
Load P. Kips	14.2	9.84	14.2	14.2	14.2	14.2	14.2	14.2	27.6	19.6	14.2	
Lateral strain before splitting, millionths	130	100	150	110	110	110	110	110	165	165	165	
Tensile bond strength, lb.per sq.in.	100	99	140	100	100	100	100	100	145	145	145	
Load q before spalling lbs.	1402	860	1707	1158	1158	1158	1158	1158	2052	2034	2015	
Load q after spalling, lbs.	1277	760	1532	1032	1032	1032	1032	1032	1798	1816	1875	
B. lbs.per sq. in.x106	3.75	3.30	4.05	3.70	3.70	3.70	3.70	3.70	5.20	4.65	4.16	
Total width, inches	6	5	6	6	6	6	6	6	6	6	6	
Core width, inches	4	3	4	4	4	4	4	4	4	4	4	
Value of m x 1/2	1	1	1	1	1	1	1	1	1	1	1	
I, moment of inertia of longitudinal steel, hundredth of in.	7.50	3.07	7.50	7.50	7.50	7.50	7.50	7.50	3.27	1.56	7.50	
a. required initial deflection, from Eq. (k)	.067	.056	.080	.057	.057	.057	.057	.057	.068	.079	.091	
Necessary tie for required initial deflection, sq.in.	.0038	.0027	.0043	-	-	-	-	-	.0095	.0046	.0051	
Supplied initila deflection of longitudinal steel inches	.125	.125	.125	.063	.125	.188	.250	.500	.125	.125	.125	
Tie requirement at one section for given deflection, in.	.112	.130	.193	.053	.125	.284	.99	2.48	.180	.106	.017	
Distributed tie requirement for given deflection No.of .25 inch dia. at a rate of 6 inch c/c.	2	2	2	1	2	2	3	4	2	2	1	
Supplied tie, No.of 0.25 inch diameter ties	2	2	2	2	2	2	2@6"	2	3	2	2	
Increased capacity in percent of .85 $f'_c A_c + A_s y l$	7.6	9.2	3.4	12.4	3.65	10.1	6.2	10.1	7.9	7.8	8.8	

of the load at the first crack to the ultimate load varied from .770 to .88, thus giving warning before failure . For the columns reinforced with straight bars and end ties the ratio was in the range of .83 to .98 and for the tied column the ratio was as high as .98. The observation of the cracking load and ultimate load indicates that the behaviour of the column reinforced with longitudinal rods having sufficient initial curvature and adequate tie is superior to the columns reinforced with straight bars and ties regarding the warning before failure by cracking.

The longitudinal maximum observed deformation of columns with initially curved bars was between .0028 to .0048. For the column reinforced longitudinally with straight bars and laterally ties, the maximum strain was observed to be .0015 at which the strain dials had to be removed because of sudden failure of column. In the column reinforced with straight bars and end ties, the ultimate strain was observed to be within the range of .00135 to .0020. The much higher strains observed in the columns reinforced with longitudinal bars having initial curvature and lateral ties indicated that the ductility of the columns is increased when initially curved bars are used. The high ultimate strains also ensure much energy absorbing capacity of the columns. The energy absorbing capacity is important in the design of structures in seismic regions. The high ultimate strain of the column is also necessary in the proper use (by adequate straining) of high strength steel as longitudinal reinforcement.

CONCLUSION

The following conclusions may be drawn from the investigation :

1. The use of ties in reinforced concrete columns can be reduced by giving initial curavture to the longitudinal bars. The characteristic spalling failure of the shell concrete and almost absence of buckling of the longitudinal bars indicates that the longitudinal bars with initial curvature serve both as longitudinal and lateral reinforcement.

2. In all cases columns reinforced with initially curved bars was observed to undergo about 187-300 per cent higher deformation before failure than columns with straight bars. At the same time ample warning of failure is given after the appearance of cracks in the concrete in columns reinforced with initially curved bars.

3. The initial mid deflection of the reinforcement is a function of A_s, Y_s, 1, β, μ, ϵ, d, b, N, t_f and EI, flexural rigidity. It can be determined by the use of the following equation :

$$Y = / a^2 \left(\frac{3\pi^2 P}{41} - \frac{\pi^4 EI}{41^3} \right) = \frac{\beta_1 a^2 \beta}{4} + \frac{2 q1}{\pi} a_q \quad \ldots\ldots (1)$$

4. In order that the stress in the end ties does not exceed the yield point, the cross sectional area of the tie at one end of the column can be represented by the equation :

$$A_t = \frac{b}{m E' a_o^2} \left[\frac{P \pi^2 a^2}{41} - \frac{P a^2 \pi^2}{2q} - \frac{2qL}{\pi} aq - \frac{\pi^4 EI}{AL^3} a^2 \frac{\beta I}{4} a_\beta^2 \right. \quad \ldots (2)$$

5. The ultimate load of axially loaded columns reinforced with initially curved bar and the cross sectional area of the tie in accordance with equations (1) and (2), shows an increase of about 16 per cent over the load carried by the concrete of equivalent cross sectional area in plain concrete column and the steel at its yield point.

SUGGESTION FOR FUTURE STUDIES

It was observed in the analysis of the experimental data for the determination of amplitude of initial curvature that only two columns were adequately reinforced with end ties for the given initial curvature of the rod. As the other columns were not adequately reinforced with ties, the ties were strained causing premature failure of the column. Conclusion cannot be drawn on the basis of two adequately designed test columns. Further research should be made with columns with initial curvature of the bar and area of ties given in accordance with equation (y) and (z) before drawing final conslusion.

The conclusion stated were limited by the scope of the research. The effect of the different variables stated in the chapter III may also be studied for a comprehensive knowledge about the behaviour of columns with initially curved longitudinal bars.

CHAPTER VIII

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APPENDIX A - COMPUTATIONS

The initial curvature given to the rod will be most effective when the rod remains in the original position under the ultimate load. As β represents the magnitude of the reaction of the foundation per unit length of the bar for unit deflection and the lateral load due to lateral strain of concrete, , q and a_1 associated with β and q will depend on the amount of lateral deformation of concrete which is prevented by the initially curved bars. In the following analysis a_1 associated with and q will be represented by a_β and a_q. The Eq.(k) and Eq.(u) are transformed into the following equations if the following equations are made :

$$a2\left[\frac{\pi^2 P}{2L}-\frac{\pi^4 EI}{4L^3}+\frac{P\pi^2}{4L}\right]=\frac{\beta L}{4}a_\beta^2+\frac{2qL}{\pi}a_q \quad \ldots (1)$$

$$A_t=\frac{b}{mE_t a_o^2}\left[\frac{P\pi^2a^2}{4L}-\frac{2qL}{\pi}a_q+\frac{Pa^2\pi^2}{2L}-\frac{\pi^4 EI}{4L^3}a^2-\frac{\beta L}{4}a^2\right] \ldots (2)$$

$$=\frac{b}{mE_t a_o^2}\left[\frac{3}{4}\frac{P\pi^2}{4L}a^2-\frac{2qL}{\pi}a_q-\frac{\pi^4 EI}{4L^3}a^2-\frac{\beta La^2}{4}\right]$$

The following values were obtained from experimental observations

$E = 29.95 \times 10^6$ psi., $P = f_{y1} \times A_s = 46{,}000 \times 0.31 = 14200$ lbs

$$q=\frac{f''_c A'}{N}\pm t_f a_{sc}=\frac{.00013\times 3.42\times 10^6\times 6}{2}\pm 1000\times\frac{5}{8}=1402$$

and 1277 lb/in, $\beta = 3.75 \times 10^6$ lb/sq.in., $a_q = a_\beta = .0013$ in.

$E_t = 30 \times 10^6$ psi., m=2, $a_o = .0017 \times 2 = .0034$, b=4 in., $I = \frac{\pi 5^4}{64\times 8^4}$

$= .0075$ in.^{4}

L = 60 in.

Applying the ~~observed~~ experimental data the required initial mid-deflection of the longitudinal bar is obtained as follows :

From Eq.(1)

$$a^2 \left[\frac{3P\pi^2}{4L} - \frac{\frac{4}{\pi}EI}{4L^3} \right] = \frac{\beta L}{4} a^2 + \frac{2qL}{\pi} a_q$$

Substituting the values, it is found

$$a^2 \left[\frac{3x14200x9.85}{4x60} - \frac{97.1x29.95x10^6x.0075}{4x60x60x60} \right]$$

$$= \frac{3.75x10^6x60x(.00013)^2}{4} + \frac{2x1402.5x60x.00015}{}$$

from which

$$a^2 \;(1755 - 25\;) = 0.95 + 7.00 = 7.95 \qquad \therefore \;\underline{a = .0678 \text{ in.}}$$

Accumulated Tie Requirement for the required Initial Mid-Deflection

As,

$$\frac{b}{m\,E_t\,a_o^2} \qquad \frac{4}{2x30x10^6x2.9x10^{-6}x4} = .00575$$

$$\frac{3P\pi^2 a^2}{4L} = \frac{3x14200x9.85x.0046}{4 \; x \; 60} = 8.03$$

$$\frac{2qL}{\pi} a_q = \frac{2x1277.5x60x.00013}{\pi} = 6.3$$

$$\frac{\pi^4 EI}{4L^3} a^2 = \frac{^4x29.95x10^6x.0075 \; x \; (.0678)^2}{4x60x60x60} = .116$$

$$\frac{La^2}{4} = \frac{3.75x10^6x60x(.00013)^2}{4} = 0.95$$

Substituting the above values in Eq(2),

$$A_t = .00575\;(2.62 - 6.30 + 5.35 - 0.116 - 0.95\;)$$

$$= \underline{0.0385 \text{ sq.in.}}$$

Accumulated Tie Requirement for the given
Initial Mid-Deflection

In the columns of Group A the initial deflection given to the rod was 0.125 in.

with a = 0.125 in.,

$$A_t = 0.00575 \left[\frac{3P\ ^2a^2}{4L} - \frac{2qL}{\pi} a_q - \frac{\pi^4 EI\ a^2}{4L^3} - \frac{L\ \beta a_\beta^2}{4} \right]$$

$$= 0.00575 \left[\frac{3x14200x9.85x0.0156}{4x60} - \frac{2x1277.5\ x60x.0013}{\pi} - \frac{97x29.95x10^6x.0075\ x(.0678)^2}{4x60x60x60} - .95 \right]$$

$$= 0.00575\ (\ 27.35 - 6.3 - 0.394 - 0.95\ )$$

$$= 0.112 \text{ sq.in.}$$

Distributed tie Requirement for the given
Initial Mid Deflection

In an attempt to supply the required tie, only two rods are assumed to be supplied at each end at a spacing of 6 inches.

Then the unsupported length of the bar becomes 60-6x2-1x2 = 46 in. and the amplitude of curvature is 0.125 - 0.125xSin 180x7/60

$$= 0.125(1-0.359) = 0.0802 \text{ in.}$$

With the above values of 'a' and 'L'

$$A_t = 0.00575 \quad \frac{3x14200x9.85x(.0802)^2}{4x46x46} - 4.85 - \frac{97x29.95x10^6x.0075}{4x46x46x46} x\ (0.0802)^2 - 0.95$$

= 0.043 sq.in. As A_t is less than 0.05 sq.in. supplied by 1-¼ in dia. tie, so assumed distribution of tie is O.K.

APPENDIX B - GRAPHIC FIGURES

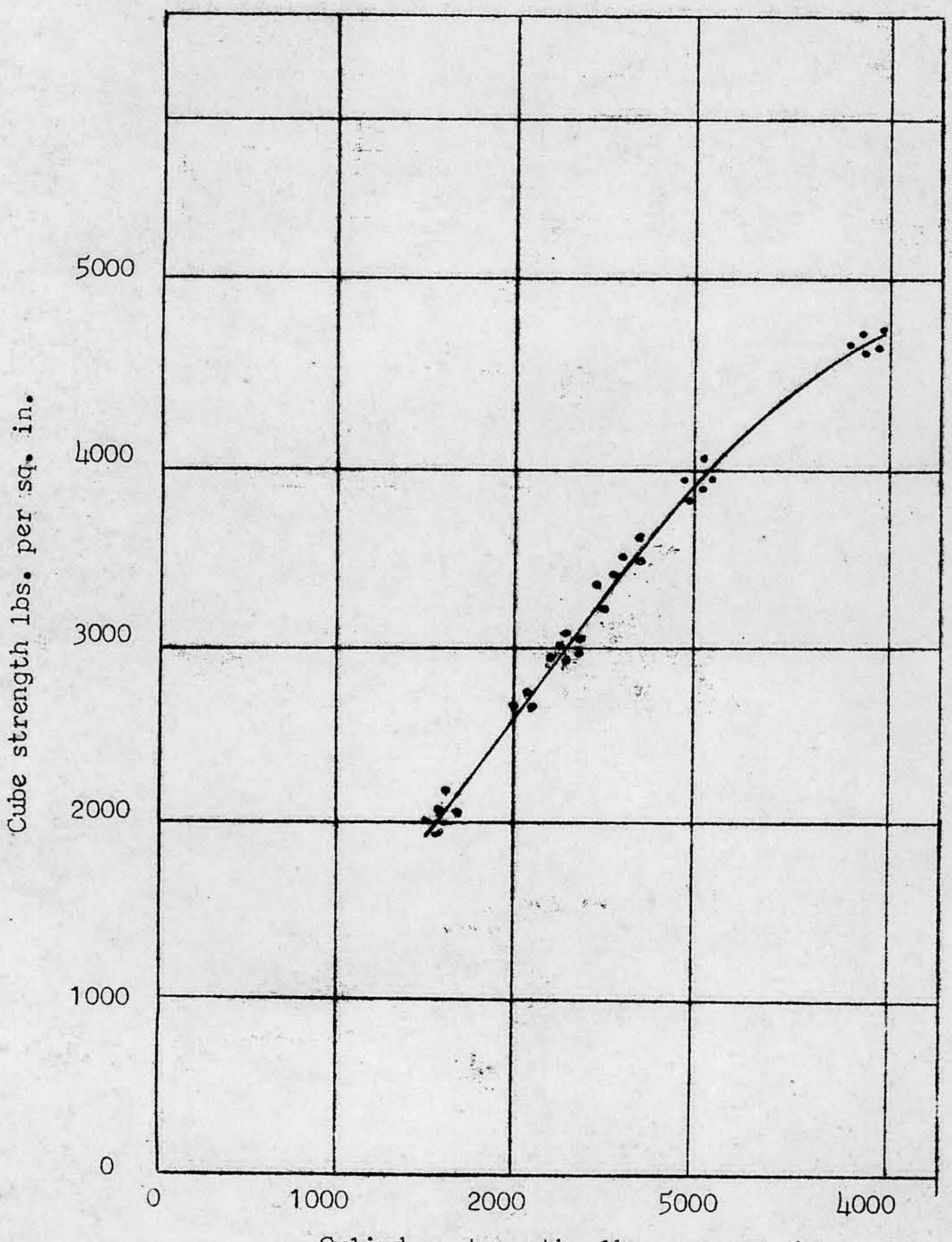

FIG. 4.10 CUBE STRENGTH Vs CYLINDER STRENGTH OF CONCRETE

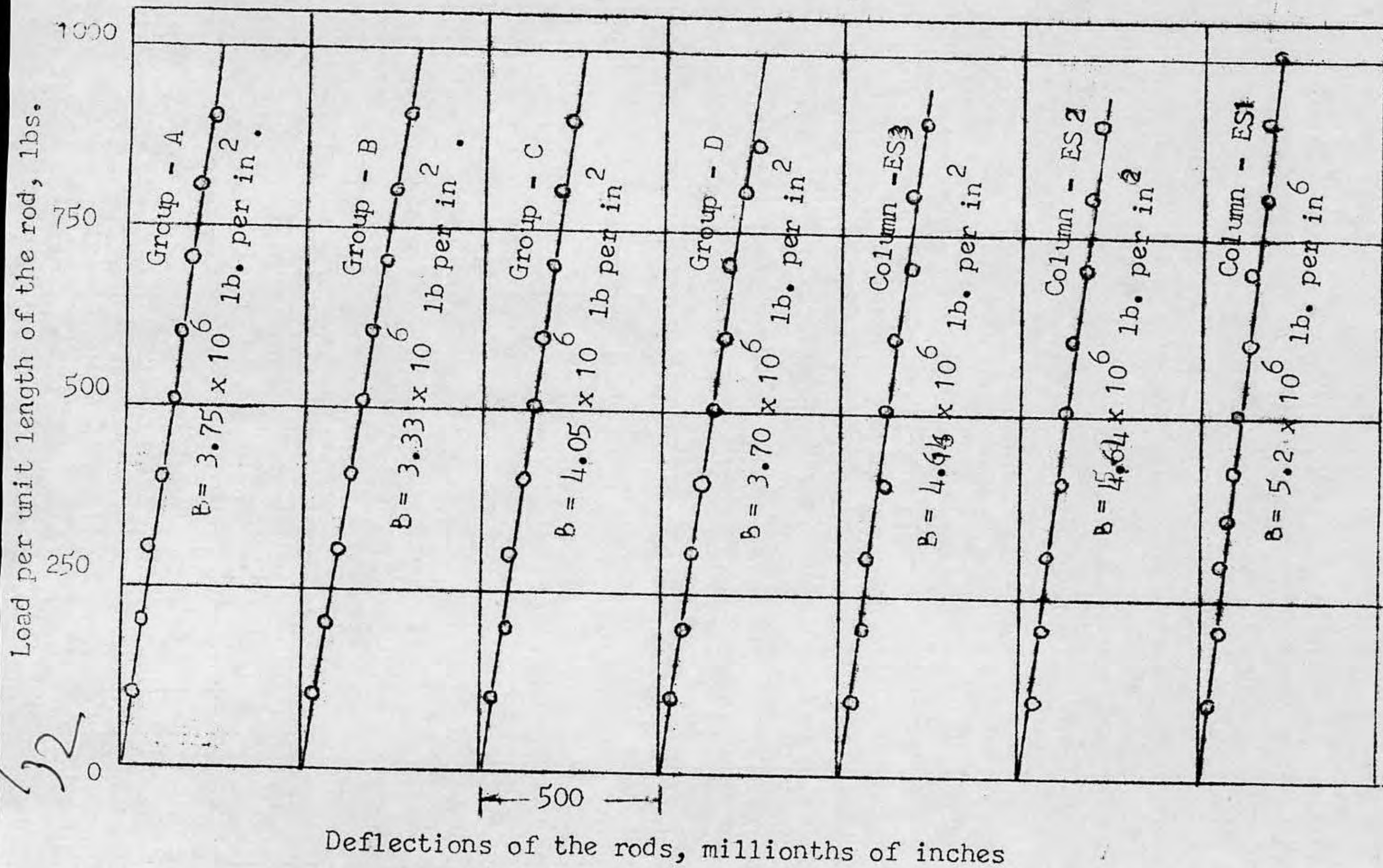

Fig. 4.20 LOAD PER UNIT LENGTH OF THE RODS PER UNIT DEFLECTION IN THE DETERMINATION OF B OF COLUMNS OF DIFFERENT GROUPS

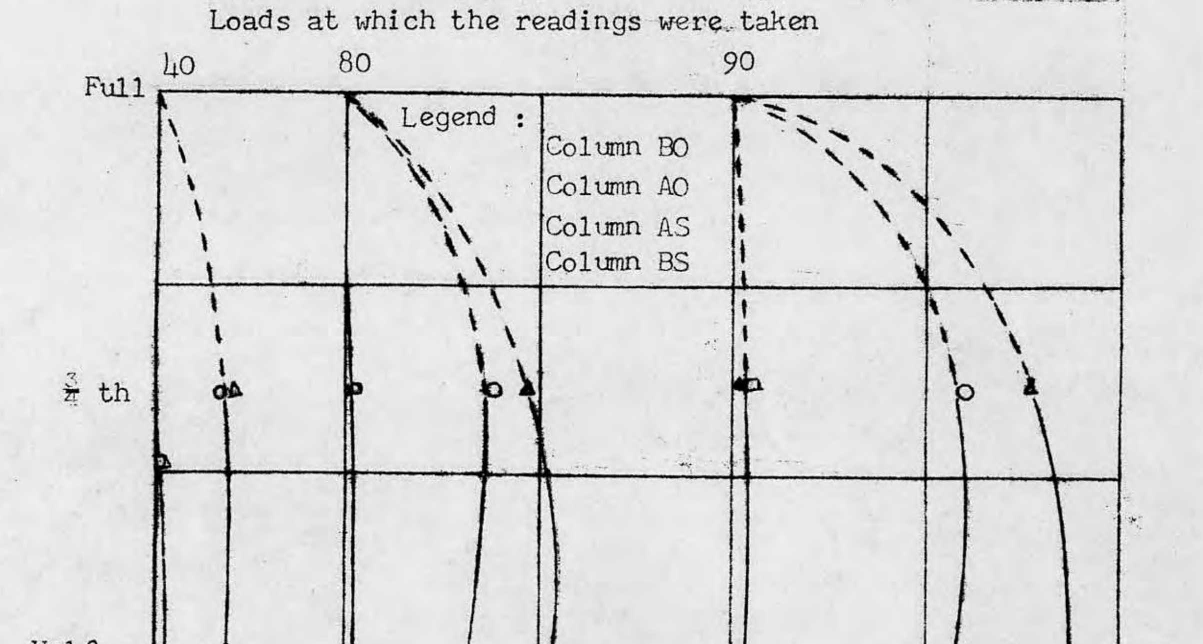

FIG5.10 - OUTWARD LATERAL MOVEMENT OF LONGITUDINAL

STEEL RODS OF THE COLUMNS OF GROUPS A AND B.

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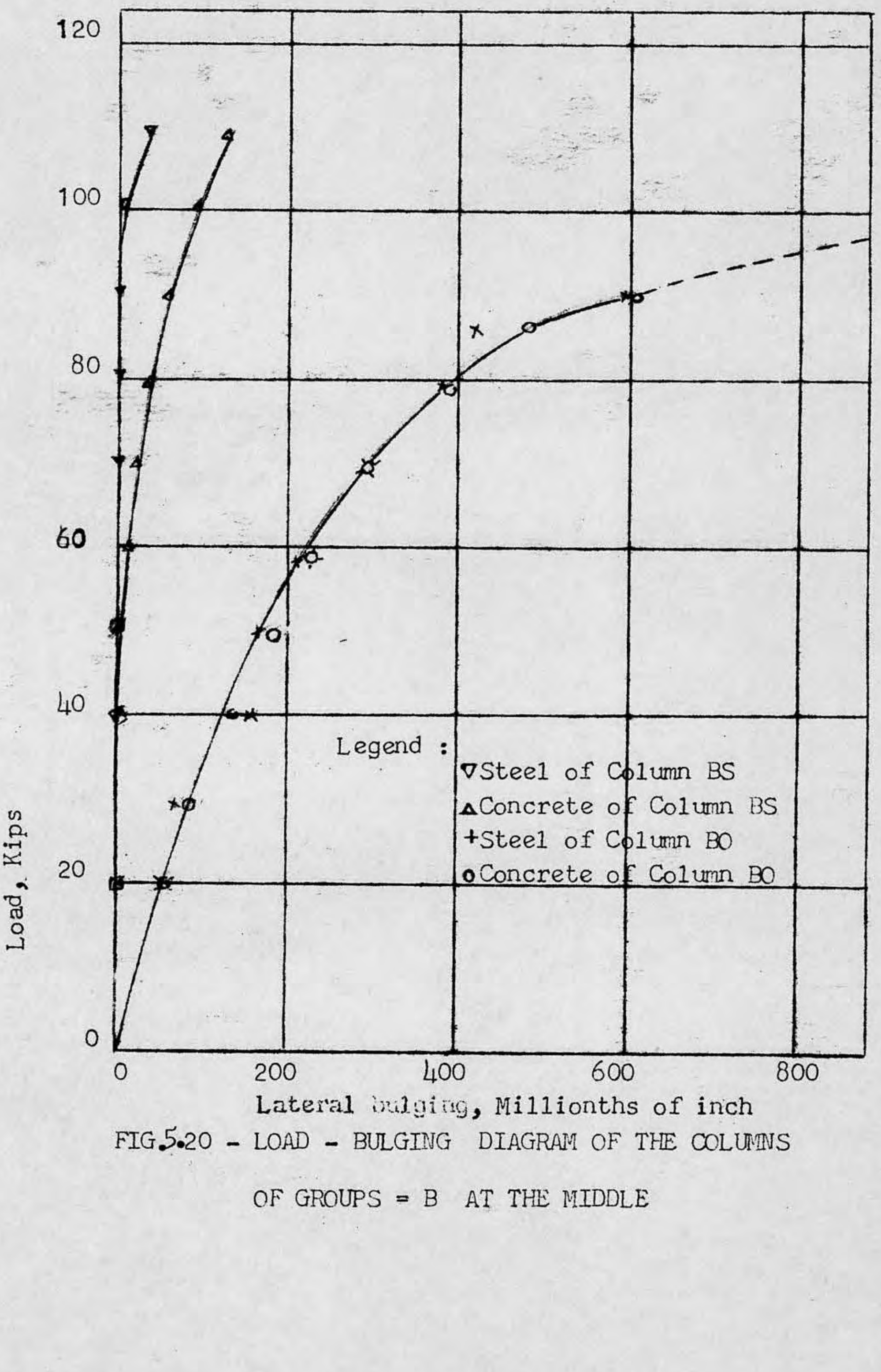


FIG.5.20 - LOAD - BULGING DIAGRAM OF THE COLUMNS

OF GROUPS = B AT THE MIDDLE

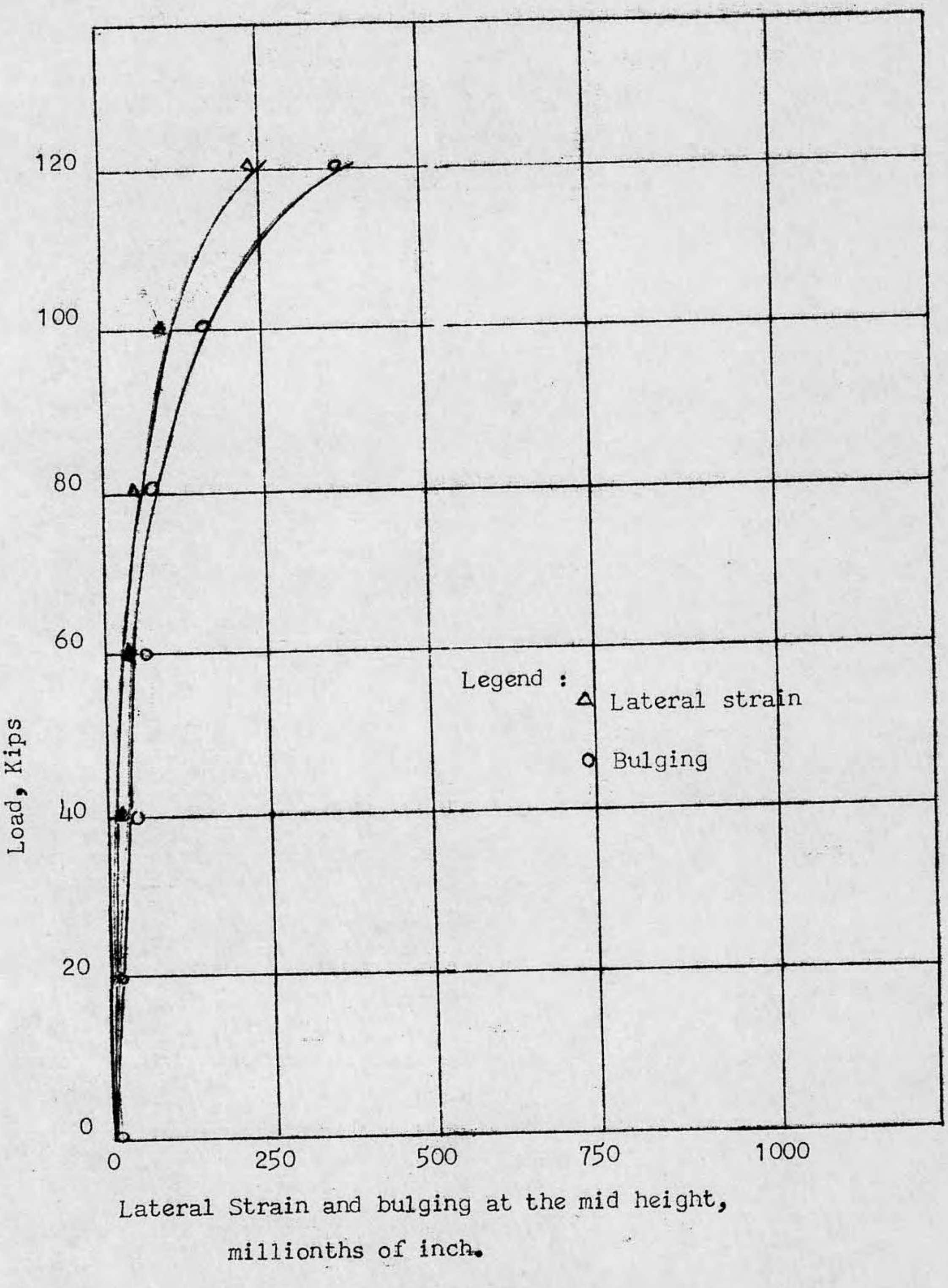


FIG.5.30 - LOAD-STRAIN and LOAD-BULGING DIAGRAM

OF THE PLAIN COLUMN OF GROUP - C

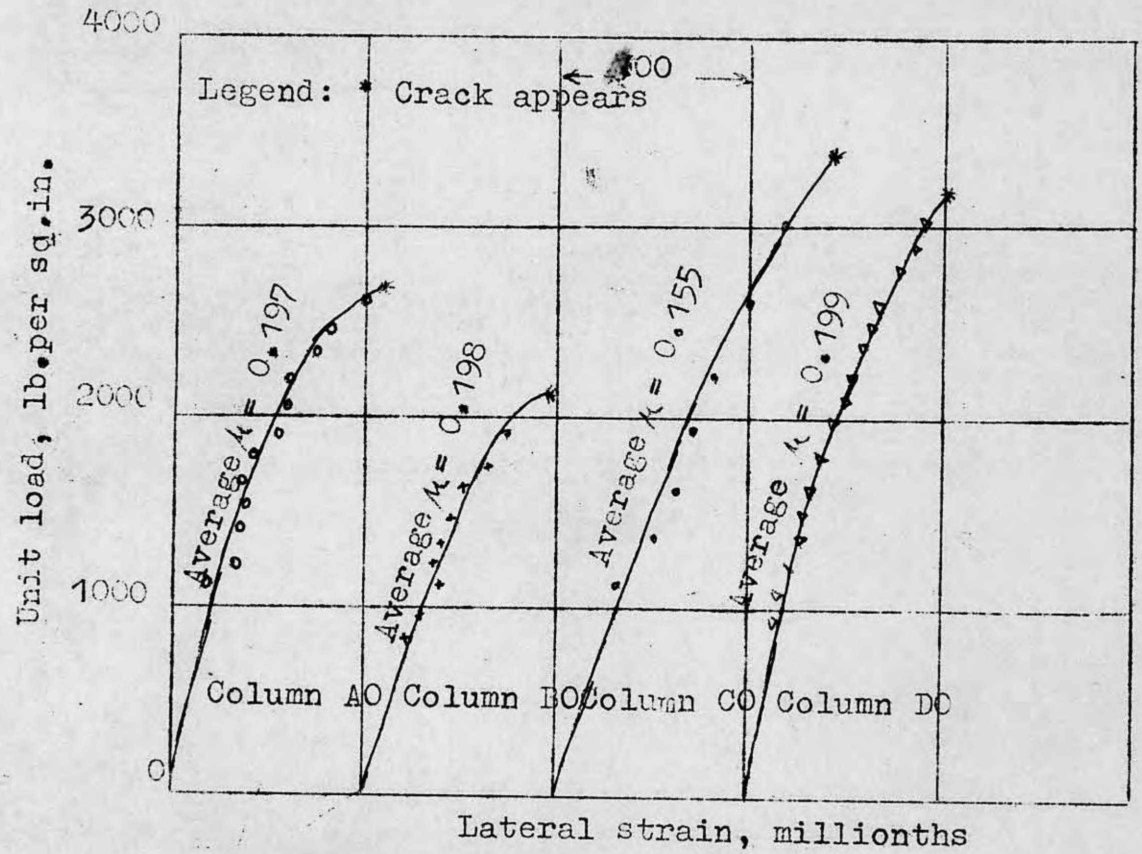


FIG. 5.10- UNIT LOAD- UNIT LATERAL STRAIN CHARACTERISTICS OF COLUMNS OF GROUPS- A,B,C,D.

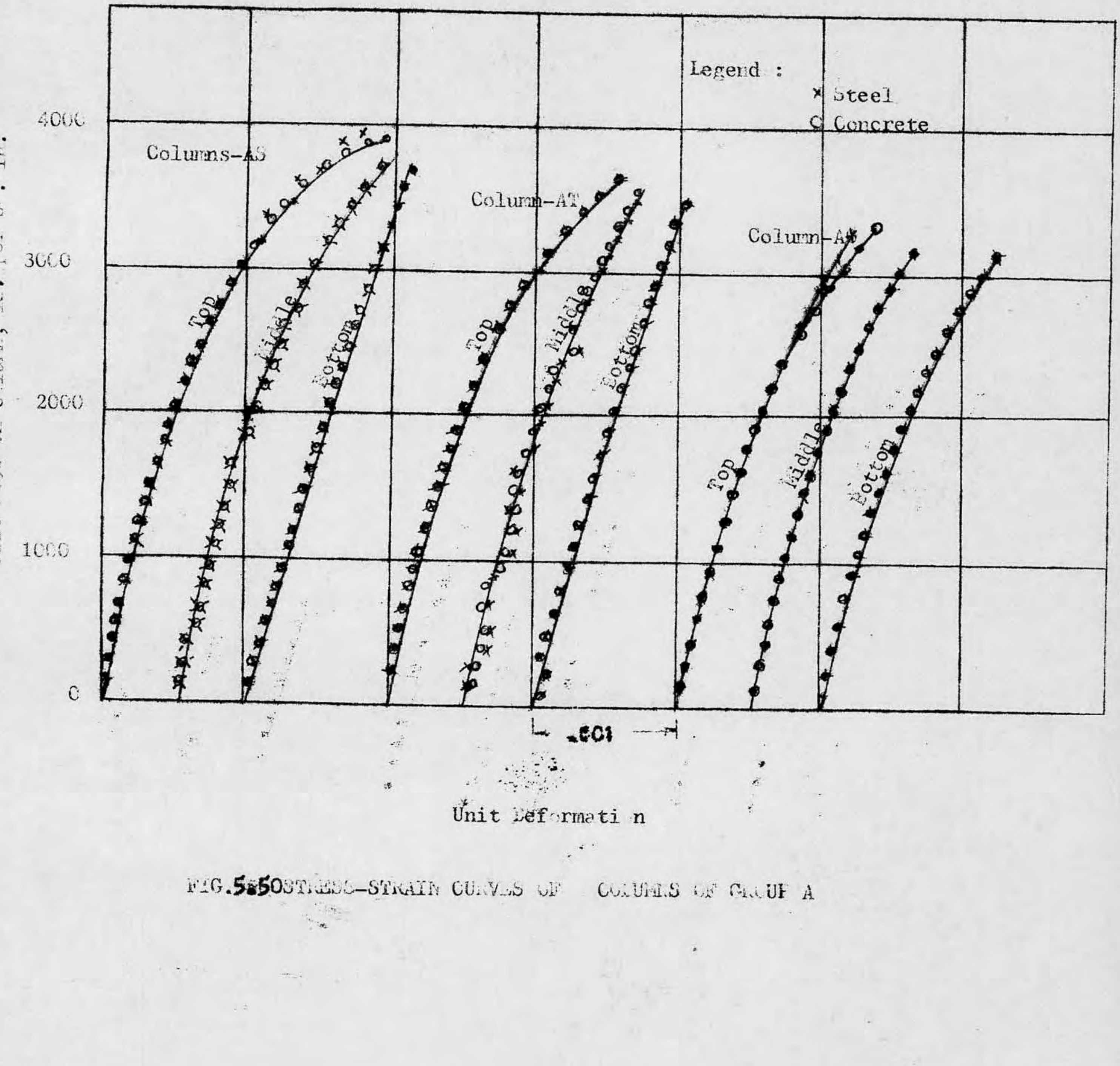


FIG.5.50 STRESS-STRAIN CURVES OF COLUMNS OF GROUP A

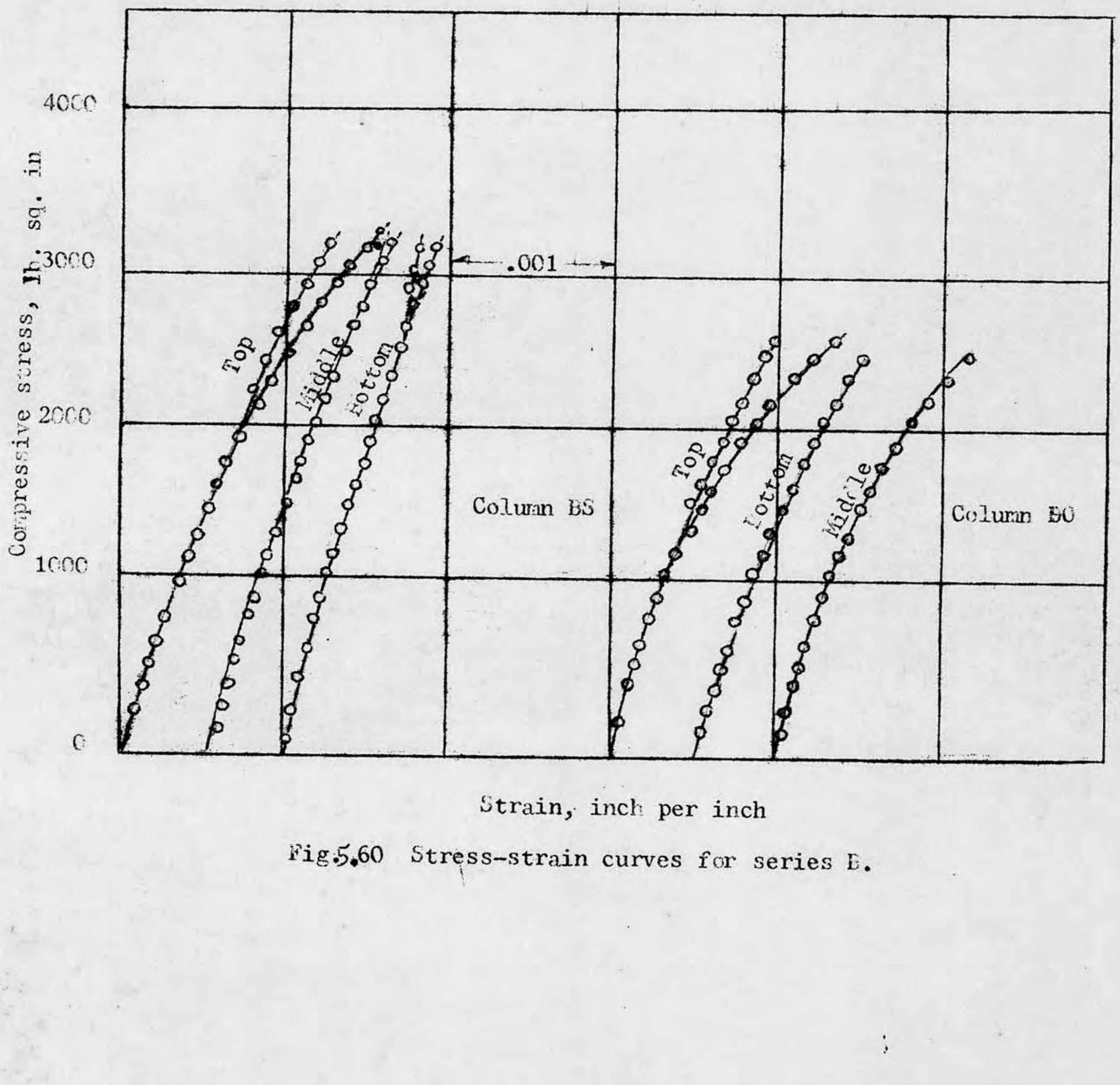


Fig.5.60 Stress-strain curves for series B.

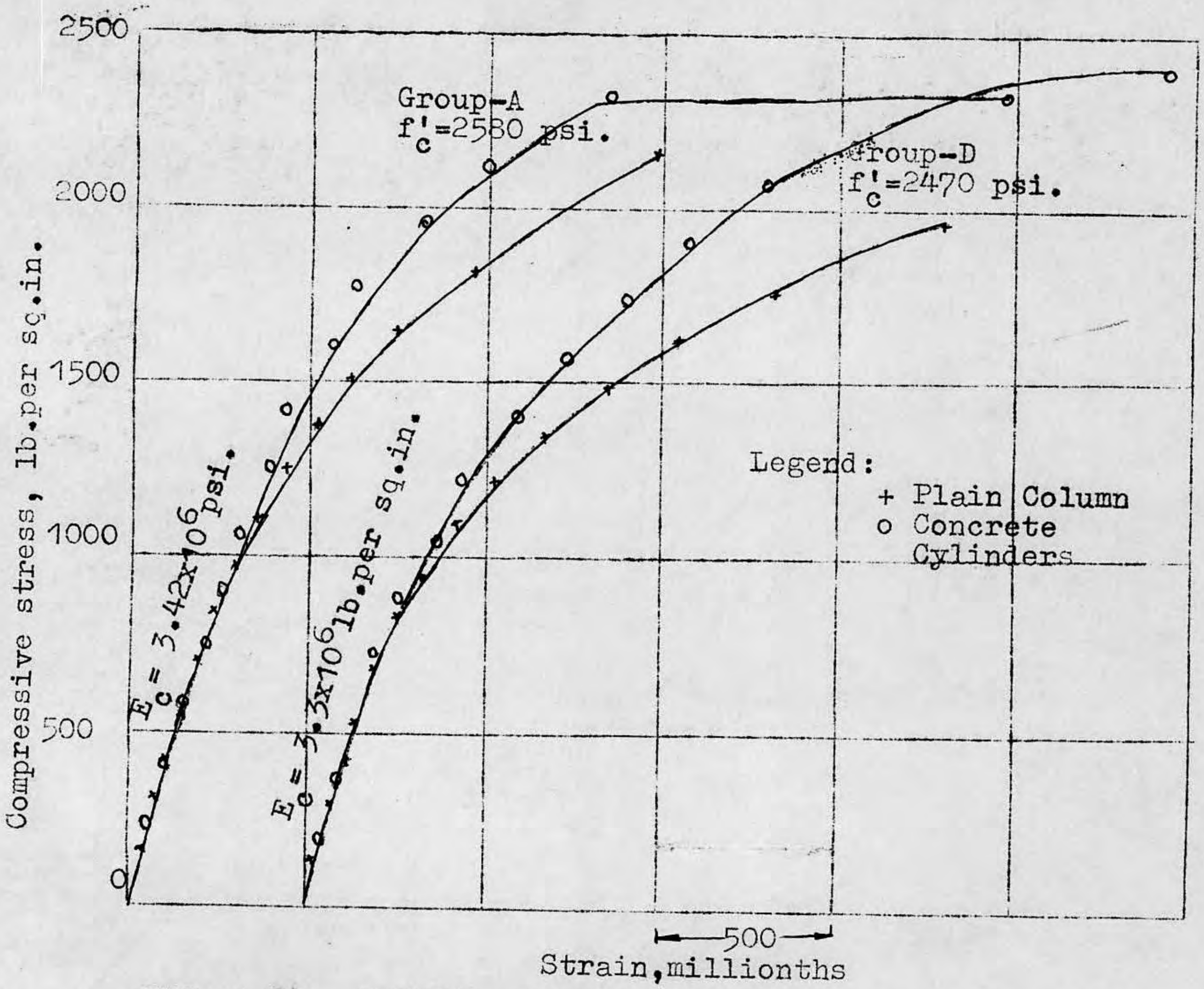


FIG. 6.10 - STRESS - STRAIN CHARACTERISTICS OF CONCRETE CYLINDERS AND PLAIN COLUMNS OF GROUPS-A & D

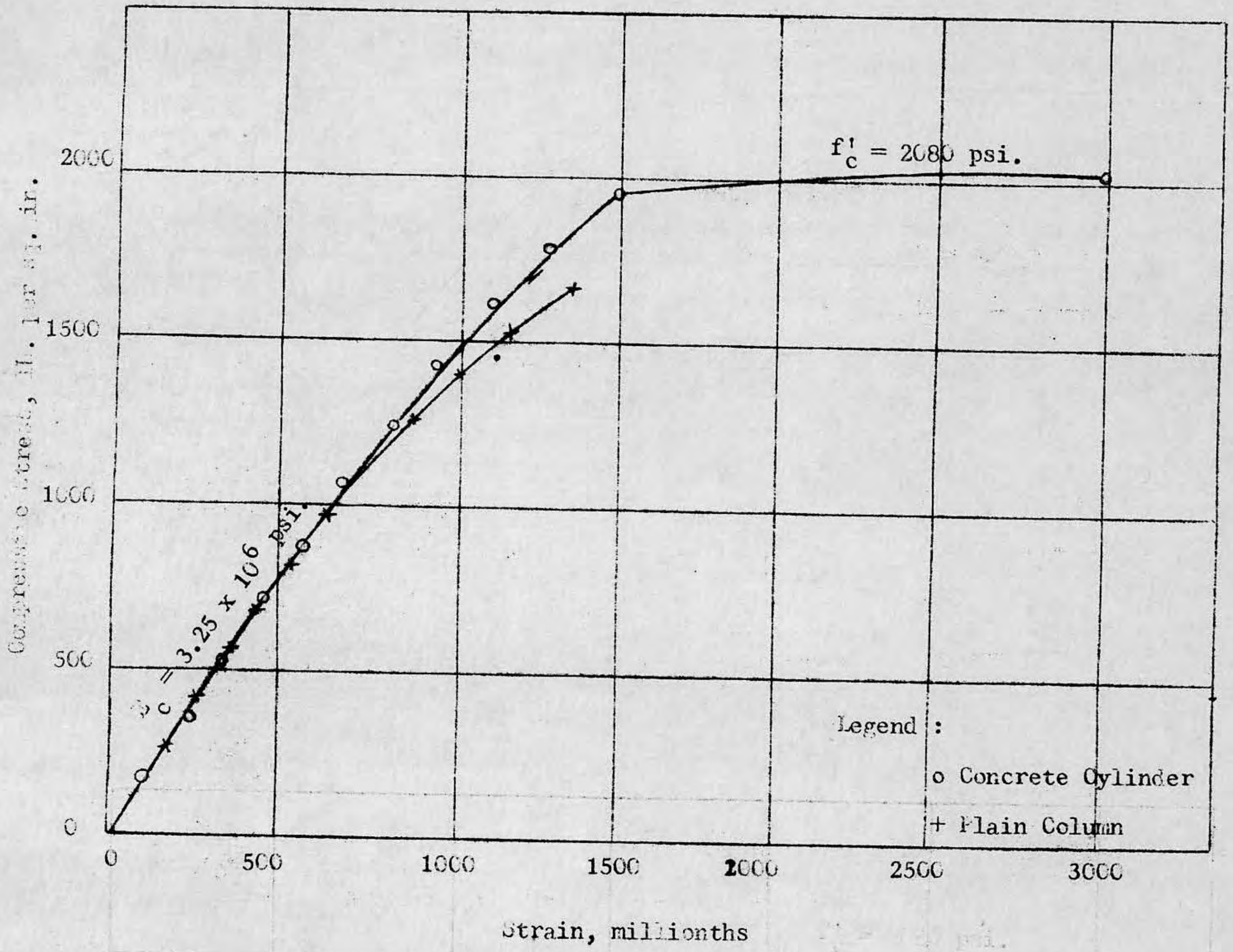


fig.6.20 STRESS - STRAIN CHARACTERISTICS OF CONCRETE CYLINDER AND PLAIN COLUMNS OF GROUP - B

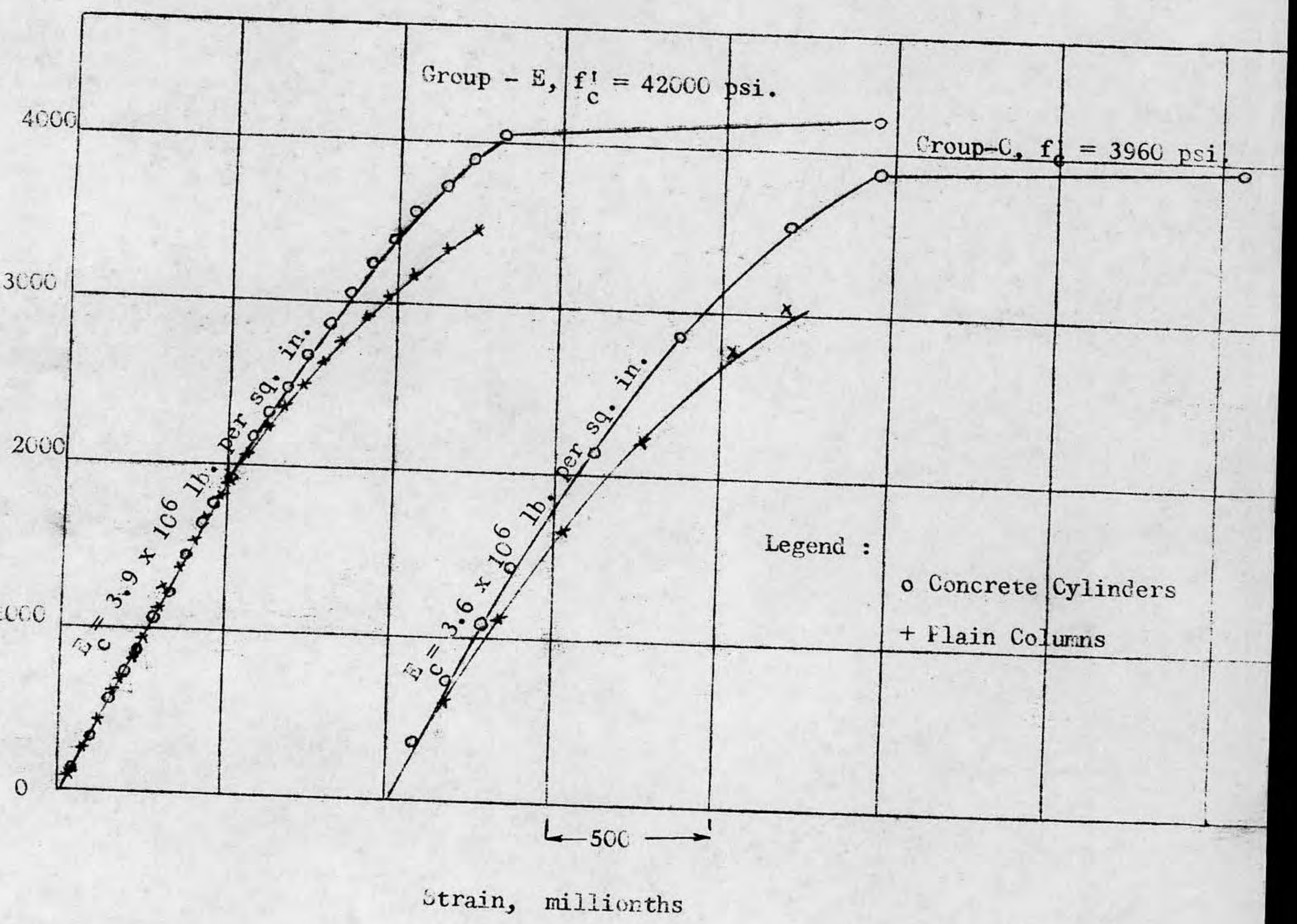


FIG. 630 STRESS- STRAIN CHARACTERISTICS OF CONCRETE CYLINDERS AND PLAIN COLUMNS OF GROUPS C AND E.

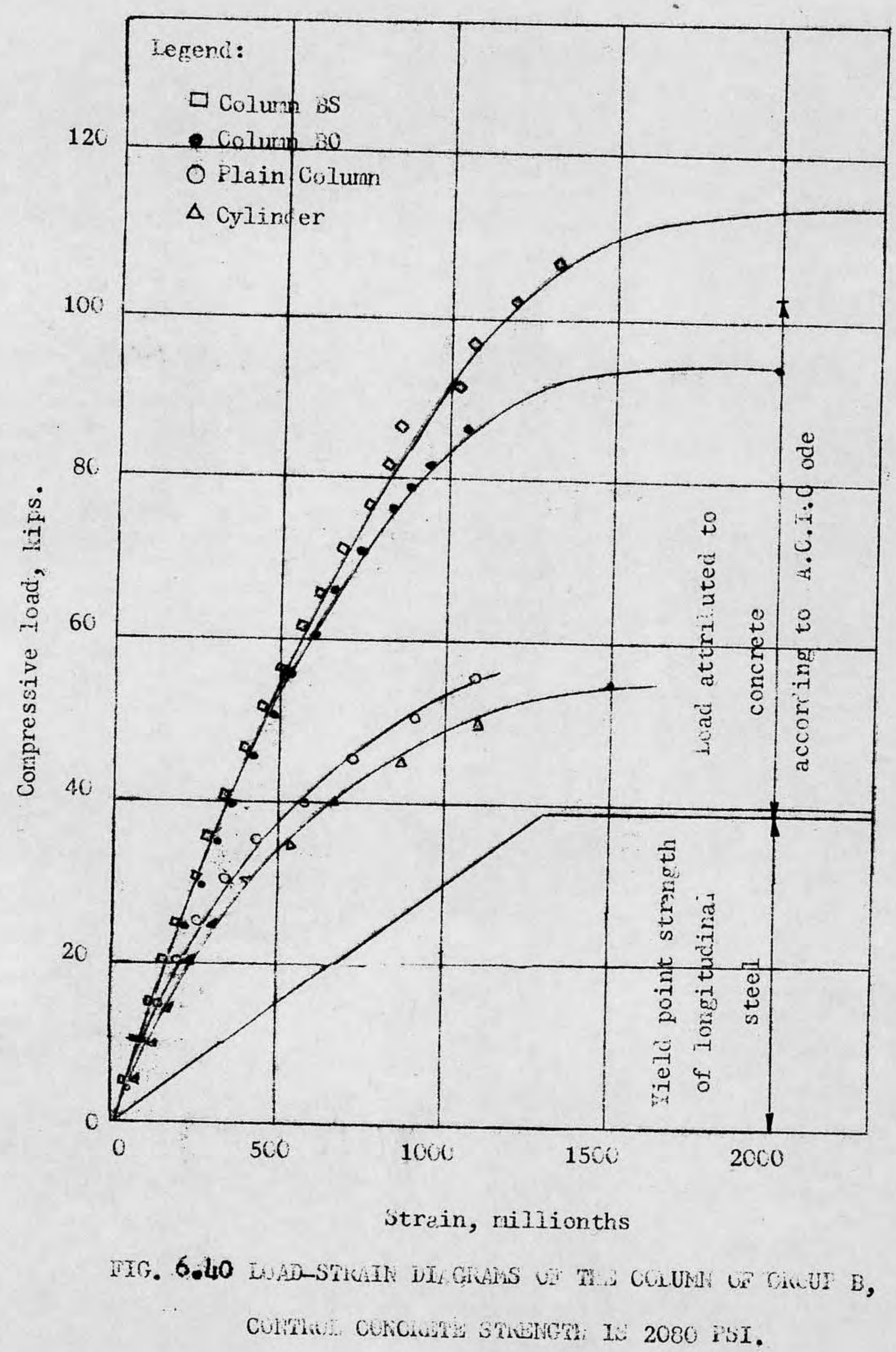


FIG. 6.40 LOAD-STRAIN DIAGRAMS OF THE COLUMN OF GROUP B, CONTROL CONCRETE STRENGTH IS 2080 PSI.

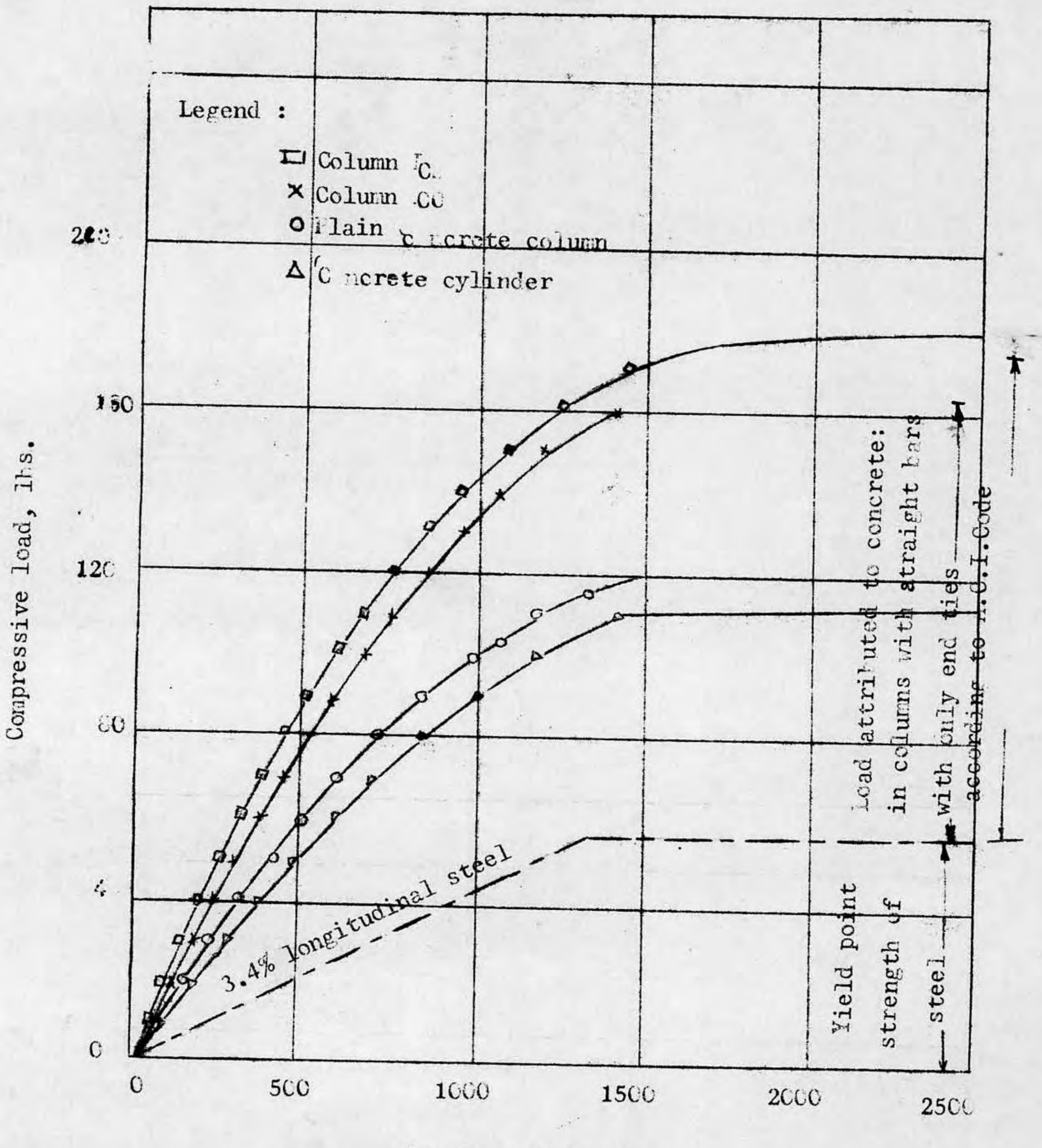


FIG.6.50 LOAD-STRAIN DIAGRAM OF THE COLUMN OF GROUP C,

[illegible] STRENGTH [illegible] LB. PER SQ. IN.; 3.4

PER CENT LONGITUDINAL REINFORCEMENT IN COLUMNS CS AND CO.

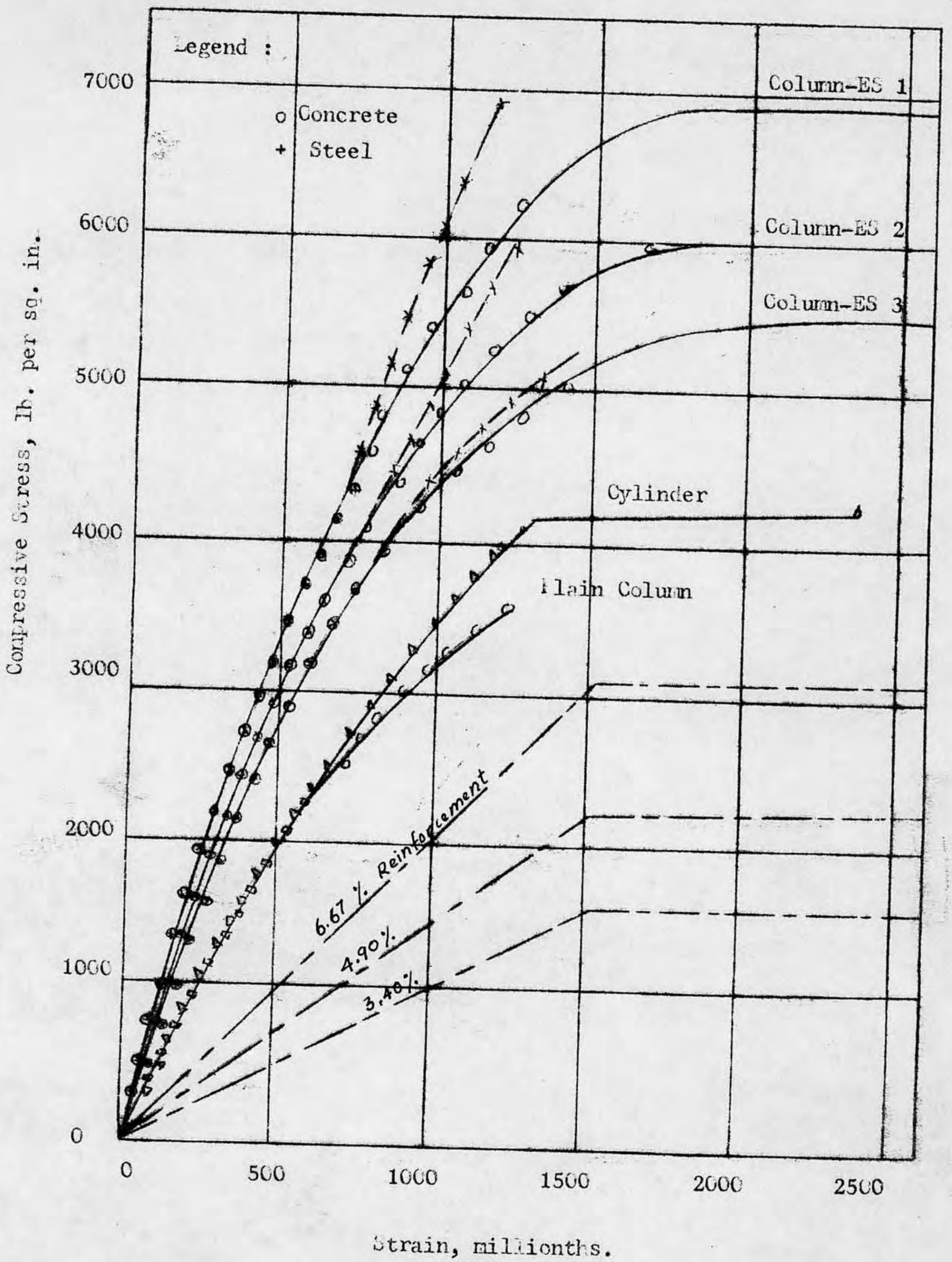


FIG. 6.60 STRESS – STRAIN DIAGRAM FOR COLUMNS OF GROUP – E WITH VARYING PERCENTAGES OF LONGITUDINAL REINFORCEMENT.

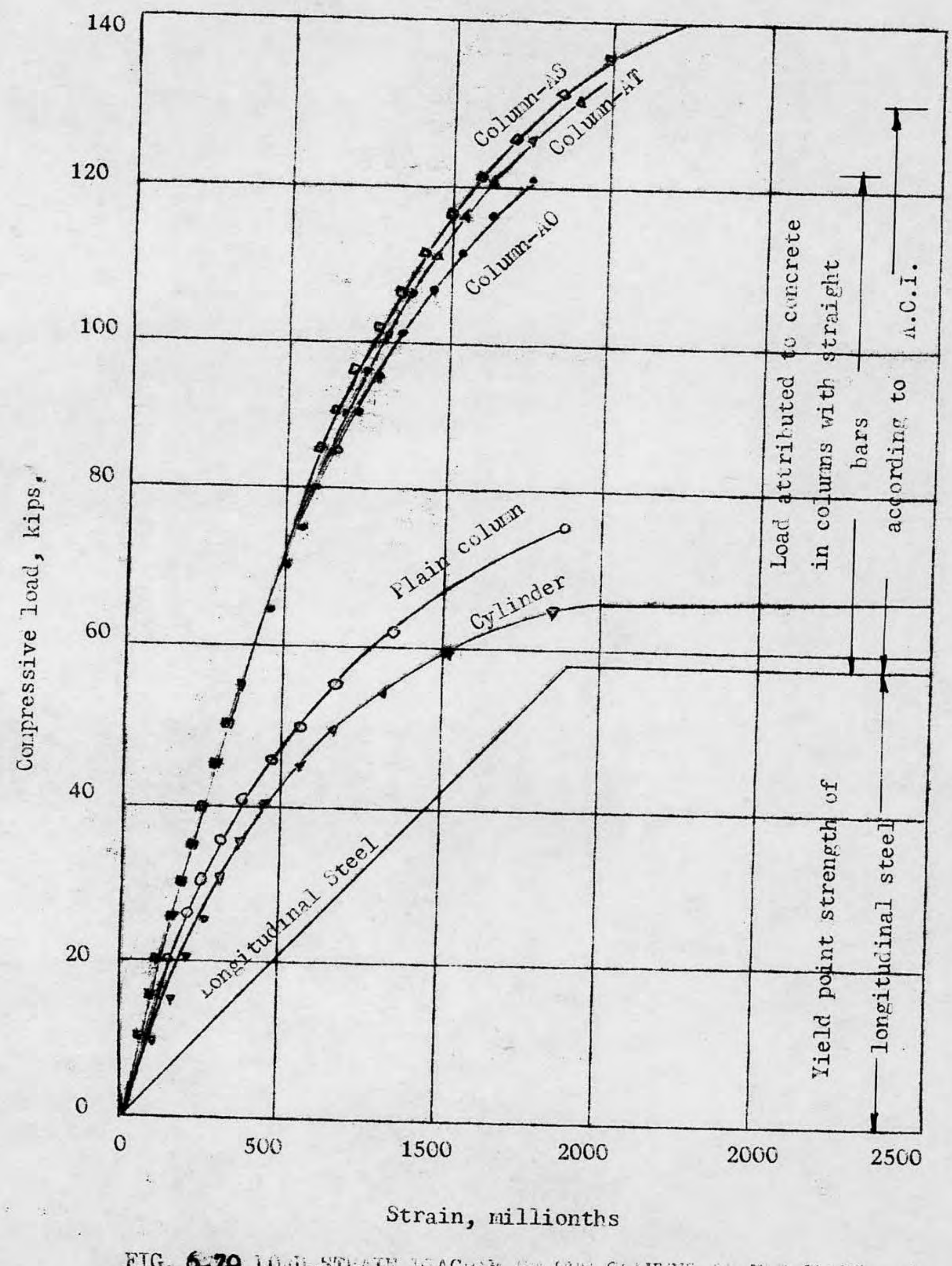


FIG. 6.70 LOAD-STRAIN DIAGRAM OF THE COLUMNS OF THE GROUP - A.

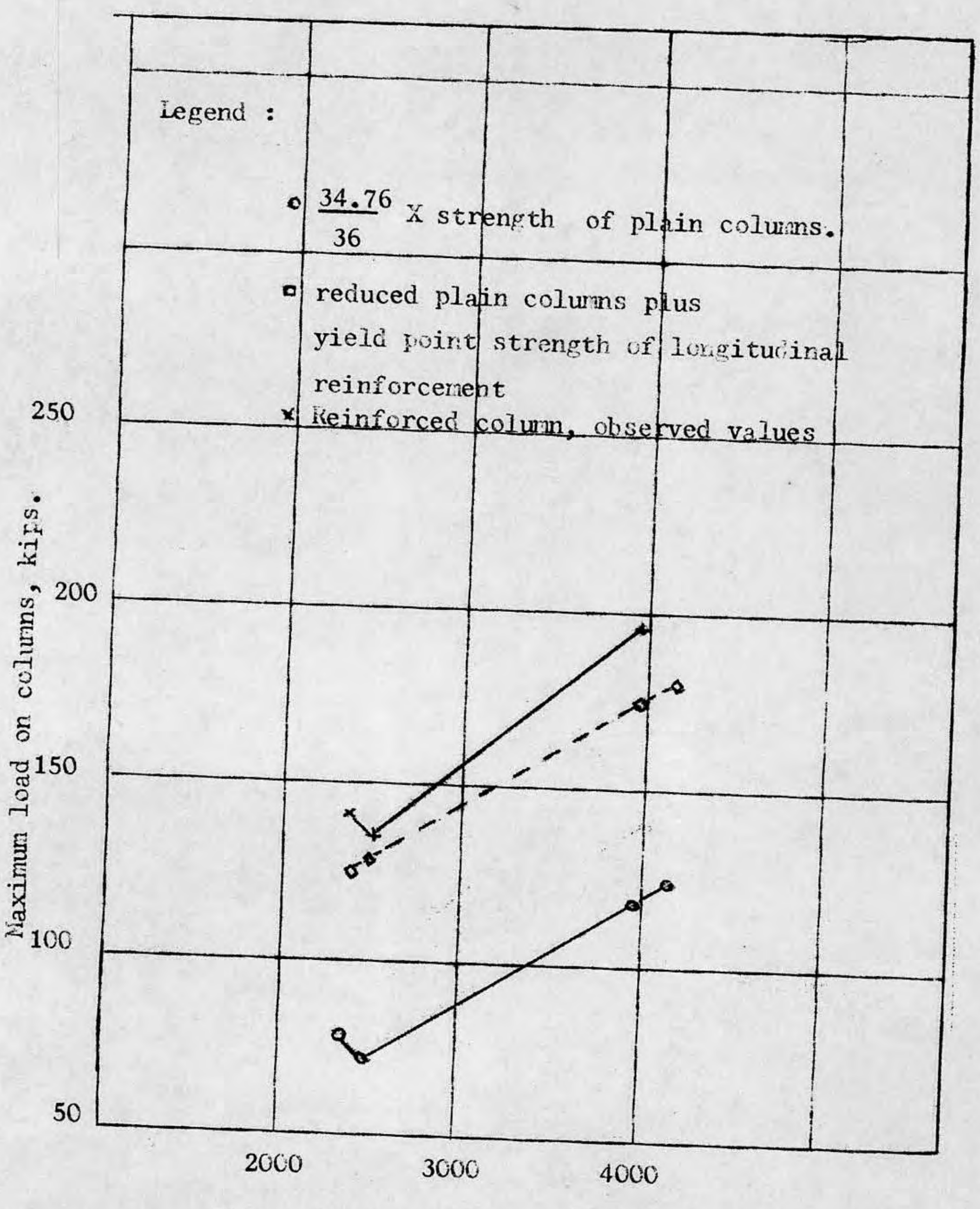


FIG 6.20 : DIVISION OF LOAD BETWEEN CONCRETE & REINFORCEMENT.
THE COLUMNS WERE REINFORCED WITH LONGITUDINAL BARS HAVING MID DEFLECTION = 1/8 IN.

APPENDIX C - PHOTOGRAPHIC FIGURES

Fig. 4.1. Various types of Reinforcement.

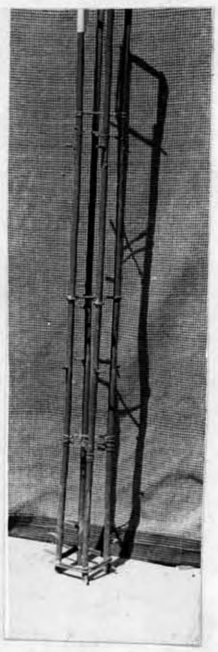


Fig. 4.2 Details of reinforcement with initial curvature.

Fig. 4.3. Reinforcement with tie in the Formwork.

Fig. 4.4 - Formwork for vertical casting of square columns.

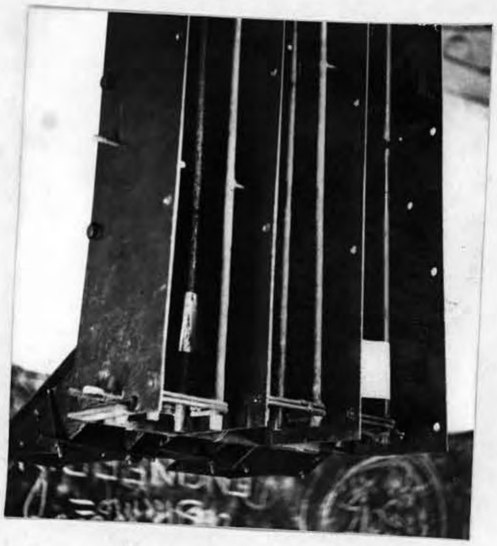


Fig. 4.5-Formwork with reinforcement.

Fig.4.6-Formwork for horizontal casting of square and rectangular column.

Fig. 4.7 Mold for Modulus of elastic foundation test specimen.

Fig. 4.9-Curing of columns

Fig. 4.8. Mold for tensile bond strength test specimen.

Fig. 4.10- Typical test set up for testing of columns.

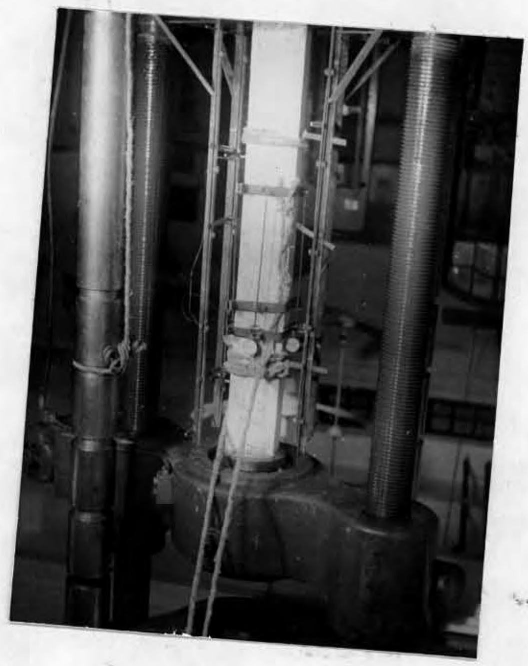
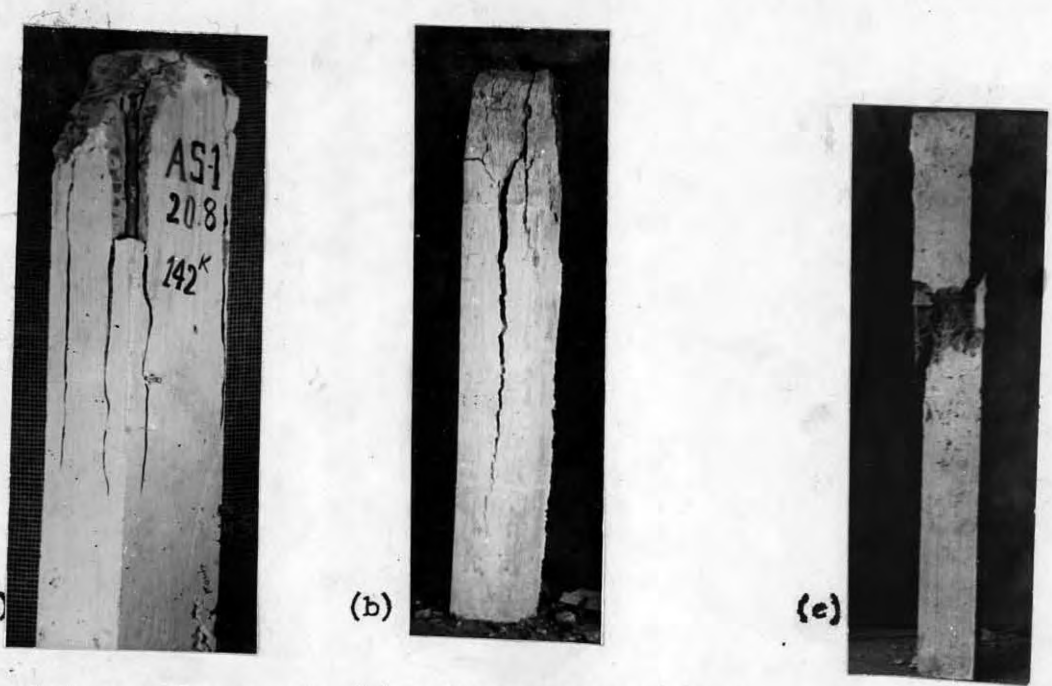


Fig. 4.11- Typical test set up for Modulus of elastic foundation.

Fig.4.12-Test specimen for modulus of elastic foundation.

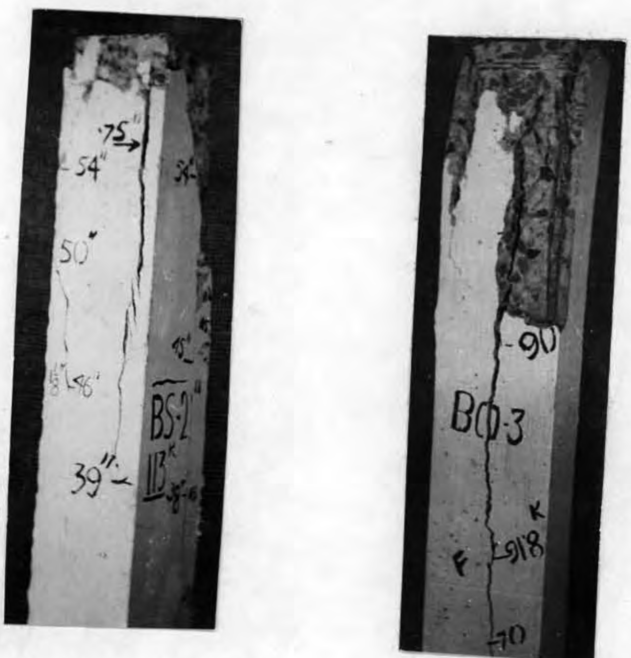
Fig.4.13 - Test specimen for tensile bond strength.

Fig. 4.14 - Typical splitting of a cylinder with the rod inside.



(a) (b) (c)

Fig. 5.1. Columns of group A. (a) Initially curved bars (b) straight bars. (c) Tied bars.



(a) (b)

Fig. 5.2. Columns of Group B.
(a) Initially curved bars. (b) Straight bars.

(a) (b)

Fig.5.3. Failure of columns of group C. (a) Column CS- with initially curved bars. (b)Column CO- with straight bars.

(a) (b) (c)

Fig.5.4. Failure of columns of group E. (a) Column ES1-with 4 -7/8 in. dia. bars. (b) Column ES2-with 4-3/4 in. dia. bars. (c) Column ES3 - with 4-5/8 in. dia. bars.

(a)

(b)

(c)

(d)

Fig.5.5 Failure of columns of group D. (a) Column-DS1.
(b) Column-DS2. (c) Column-DS3.
(d) Column-DS4
(e) Column-DS5.
(f) Column-DS6.

(e)

(f)

(a)

(b)

(c)

(d)

(e)

Fig. 5.6. Failure of concrete columns without reinforcement.
(a) Group A. (b) Group B.
(c) Group C. (d) Group D.
(e) Group E.