

AN EXPERIMENTAL INVESTIGATION OF LATERALLY
PRESTRESSED CONCRETE COLUMNS UNDER ECCENTRIC LOADS

by

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ABSTRACT

This thesis is a study of the effects of laterally stress, eccentricity of applied load and slenderness ratio on the ultimate load of laterally prestressed concrete columns.

An experimental investigation has been carried out on 6" diameter concrete columns with slenderness ratios of 16, 24, 36 and 48. Loads were applied at eccentricities $\frac{e}{D}$ of 0.1, 0.25 and 0.5. Initial laterally prestresses were applied to the columns by wrapping the columns with piano wires giving lateral stresses of 900, 1600 and 3200 psi.

Based on the test results together with those of concentrically loaded laterally prestressed columns from Godse's investigation (16), an empirical formula is proposed for predicting the ultimate load of laterally prestressed concrete columns under either concentric or eccentric loadings. Charts were prepared to simplify use of the proposed formula and a design method using the presented charts is also given.

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NOMENCLATURE

A	Cross-sectional area of the column.
A_c	Net cross-sectional area of concrete, excluding any finishing material and reinforcing steel.
A'_c	Effective area of concrete section to resist compression.
A_e	Concrete area bounded by the edge fiber of the maximum strain and a straight line located at the point of eccentric load and parallel to the neutral axis.
A_w	Cross-sectional area of wire.
a	Empirical constant.
a_1	Depth of equivalent rectangular stress block = $k_1 c$.
C	Resultant of concrete compression of the section.
C_M	Nondimensional factor of ultimate bending moment = $M_{ult}/(D^3 f'_c)$
C_P	Nondimensional factor of ultimate axial load = $P_{ult}/(D^2 f'_c)$
c	Distance from extreme compressive fiber to neutral axis at ultimate strength.
c/c	Center to center.
c_1	Empirical constant.
c_2	Empirical constant.
c_3	Empirical constant.
D	Diameter of concrete column.
D_w	Diameter of wire.
E	Young's modulus.
E_t	Initial tangent modulus.
e	Eccentricity.
e_o	Equivalent eccentricity.
F'_c	Maximum compressive strength.

F_t'	Maximum tensile strength.
$F\{\}$	A function.
f_c'	Unrestrained concrete crushing strength.
f_{sl}'	Increase of compressive strength due to longitudinal reinforcement and longitudinal prestressing.
f_w	Ultimate tensile strength of wire.
f_y	Yield strength or elastic limit.
f_y'	Yield strength of longitudinal steel.
f_y''	Yield strength of spiral ties.
f_y'''	Yield strength of longitudinal prestressing wires.
I	Moment of inertia.
K	Empirical coefficient.
K_1	Empirical coefficient.
K_2	Empirical coefficient.
K_3	Empirical coefficient.
K_4	Empirical coefficient.
K_5	Empirical coefficient.
k_1	A factor, = 0.85 for $f_c' \leq 4000$ psi, and decreases by 0.05 for every 1000 psi above 4000.
kip	One thousand pounds.
ksi	One thousand pounds per square inch.
L	Effective length of column.
l	Over-all length of column.
lb.	One pound.
L/r	Slenderness ratio.
M_{ult}	Ultimate bending moment.

N_1	Number of wires per inch of wrapping.
N_2	Pitch of the wires (inch c/c of wires / layer).
P	Total failure load.
P_c	Load carried by concrete alone.
P_{comp}	Maximum load controlled by compression failure.
P_{cr}	Critical load.
P_E	Euler load.
P_{exp}	Experimental failure load.
P_{sl}	Load carried by longitudinal reinforcement, including spiral ties and prestressing cable.
P_{sp}	Load carried by spiral prestressing wire through lateral pressure.
P_{ten}	Maximum load controlled by tension failure.
P_{ult}	Theoretical ultimate load.
P_y	The yield load of the material.
p'	Ratio of volume of longitudinal reinforcement to $A_c \times$ unit length.
p''	Ratio of volume of spiral ties to $A_c \times$ unit length.
p'''	Ratio of volume of longitudinal prestressing wires to $A_c \times$ unit length.
psi	Pounds per square inch.
q_i	Pressure per unit length of the inside circumference of wrapping wire.
R	Reduction factor due to slenderness ratio and eccentricity.
R_1	Empirical coefficient.
R_2	Empirical coefficient.
R_3	Empirical coefficient.
R_e	Empirical coefficient.
r	Radius of gyration.

T	Tension exists in the wrapping wire.
T^u	Ultimate axial tension of wire.
y_1	Distance from neutral axis to the extreme fiber on the same side of the load.
y_2	Distance from neutral axis to the extreme fiber on the opposite side of the load.
α	Empirical coefficient.
β	Reduction factor for $\bar{\sigma}_3$.
δ	Mid-height lateral deflection.
$\sigma_{\max}$	The maximum principal stress.
$\sigma_{\min}$	The smallest principal stress.
σ_y	Yield stress.
σ_1	Longitudinal stress.
$\bar{\sigma}_1$	Total maximum longitudinal stress.
σ_3	Initial lateral prestress.
$\bar{\sigma}_3$	Final lateral stress at the time of column failure.
σ_3^u	Ultimate lateral stress due to rupture of wires.

CHAPTER 1

INTRODUCTION

It has long been known that concrete under a triaxial state of stress can withstand larger stresses and deformations than the same concrete in its unrestrained state.

Many investigations have demonstrated the above hypothesis by testing concrete in a triaxial cell with the concrete subjected to an uniform cell pressure and a deviator stress applied to fail the concrete. A Coulomb type of failure criteria has been found to hold:-

$$\bar{\sigma}_1 = f'_c + K \bar{\sigma}_3, \text{ where } \bar{\sigma}_2 = \bar{\sigma}_3$$

The constant K varies in the range 2.65 for light weight concrete to nearly 6.0 for very dense concrete. Generally, a value of 4.1 is used for normal weight concrete.

The practical applications of triaxial confining stresses to increase load carrying capacity are in pipe columns and spiral reinforced columns. In these cases the lateral confining stress is provided by the distortion of the pipe or spiral respectively. No confining stress exists until the pipe or spiral distorts; thus the load capacity is increased but the deformations at high load are large.

To exploit this large increase in strength due to triaxial stress, several investigations have been performed on concrete cylinders laterally prestressed by being wrapped with wire under tension. These investigations all showed large increases in load capacity. Due to Poisson's Ratio type effects the lateral stress increases as the cylinder

deforms. Failure is governed by the maximum lateral stress provided by the steel wire; which disappears when the wire breaks. All these investigations were carried out with concentric loading.

One of the great handicaps of reinforced concrete frame buildings is the large diameter needed for the inside columns of the lower stories. In some cases, a reinforced concrete frame building has been decided against in favor of a steel frame building for this very reason. It is possible that the inside column diameters for the lower floors can be made as small as, or smaller than, those of structural steel by using laterally prestressed concrete columns. Lateral external prestressing can be also used to strengthen a column (building) after the building has been damaged by earthquake movement or blast.

Unfortunately, there are no provisions for the design of laterally prestressed columns in either the current (1970) ACI (American Concrete Institute) or NBC (National Building Code of Canada) codes, since not enough experimental data is available on which code recommendations can be based.

This investigation was carried out to determine experimentally the effects of lateral prestressing, slenderness ratio and eccentricity on the failure load of spiral wrapped concrete columns. Hence an empirical design formulae was derived, summarising the experimental results, to predict the behaviour of laterally prestressed concrete columns.

CHAPTER 2

LITERATURE SURVEY

Compression members are an integral part of most civil engineering structures, consequently being able to predict their performance is essential from both the economic and safety points of view. However, the problem is compounded that short columns fail by material failure and long columns by instability or buckling. Columns are typically constructed of reinforced concrete, steel, prestressed concrete, wood or concrete filled steel tubes. In this Chapter, the classical theories of column behaviour will be reviewed followed by a survey of previous investigations of triaxially stressed concrete compression members.

2.1 Theory of Columns

2.1.1 Short Columns

For a very short pin-ended column (27)¹ and neglecting the lateral deflection due to the failure load P and eccentricity e , the failure load (defined as end of elastic behaviour) is given by the two following formulas, whichever has a smaller value:-

$$P_{\text{comp}} = \frac{F_c'}{\left(\frac{1}{A} + \frac{e y_1}{I} \right)} \dots\dots\dots(2.1a)$$

¹ Numerals in parentheses refer to corresponding items in the List of References.

$$P_{\text{ten}} = \frac{F'_t}{\left(\frac{e y_2}{I} - \frac{1}{A} \right)} \dots\dots\dots(2.1b)$$

where P_{comp} is the failure load controlled by compression failure, P_{ten} is that controlled by tension failure, y_1 is the distance from neutral axis to the extreme fibre on the same side as the load, y_2 is that on the opposite side of the load, F'_c is the maximum compressive strength, F'_t is the maximum tensile strength, A is the cross-sectional area, and I is the moment of inertia of the column section.

If the material has the same strength in compression and tension, such as steel, the failure load will be obtained with Eqn.2.1a. However, if the maximum tensile strength of the material, such as concrete, is much lower than its maximum compressive strength, and if the eccentricity is large enough to produce tensile stress in the column, the failure load given by Eqn.2.1b will be very small and will control the failure. This is why concrete columns need reinforcing in the tension zone to take care of the tensile stress.

Many theoretical and empirical formulas have been proposed for the analysis and design of long slender columns (27). The following sections will review those formulas relevant to this work.

2.1.2 Long Columns

Unfortunately the failure (maximum) load of a long column is limited by buckling. The developement of the various long column formulas are followed in this section.

2.1.3 Euler's Column Formula

In 1757, L. Euler obtained a formula for the maximum load of an axially loaded elastic pin-ended column which is assumed to be initially straight. The formula is

$$P_E = \frac{\pi^2 E I}{L^2} \dots\dots\dots(2.2)$$

where L is the effective length, E is the Young's modulus and P_E is known as the Euler load of the pin-ended column. The Euler load is also called "critical load P_{cr} " at which the lateral buckling of the column occurs. This formula was verified by a series of experiments carried out by von Karman and others about 1900 (19).

2.1.4 Tangent Modulus Formula

Since the Modulus of Elasticity of a material is related to the stress in the material, Euler's formula only holds if the Modulus of Elasticity is independent of stress.

In 1889, Engesser (6) proposed the tangent modulus formula to avoid this criticism of Euler formula.

$$P_{cr} = \frac{\pi^2 E_t I}{L^2} \dots\dots\dots(2.3)$$

in which the tangent modulus E_t replaces Young's modulus in the Euler equation.

Failure is now given by the simultaneous solution of the equilibrium equation and the tangent modulus equation. This method can be shown to hold for columns where material failure and instability interact.

In the derivation of the Euler's column formula and the tangent modulus formula, the following assumptions have been made (19):

- a) The column is incompressible.
- b) Plane sections remain plane.
- c) The stress-strain relationship of any fibre is the same as the stress-strain diagram for a small specimen of the material.
- d) The plane of bending is a plane of symmetry.
- e) The effect of shearing stresses in the deflected column when it passes from the stable to the unstable state can be neglected.
- f) The load is applied along the centroidal axis of the column.
- g) The column is perfectly straight.
- h) The cross-sectional area of the column is constant.

The effect of a) is small in rigid material and can be neglected (26). Assumption b) and c) have been verified by experiment, and the effect of e) is small and can be neglected (6). However, most columns in structures and in experimental work are eccentrically loaded and have some initial imperfections such as curvature. Assumptions f), g) and h) are, in general, not valid and the conditions which are most difficult to satisfy are that the load is perfectly axial and that the column is perfectly straight. In the past, this difficulty has led to the use of the secant formula, but it is now considered better to use the tangent modulus formula and to allow for other effects by the use of an appropriate safety factor (8).

2.1.5 The Secant Formula

The secant formula was originally derived to allow calculation of the elastic stresses in columns loaded eccentrically.

When a column is loaded longitudinally with an eccentricity e , the mid-height lateral deflection δ is given by

$$\begin{aligned}\delta &= e \left\{ \sec \left(\frac{L}{2} \sqrt{\frac{P}{EI}} \right) - 1 \right\} \\ &= e \left\{ \sec \left(\frac{L}{2r} \sqrt{\frac{P}{EA}} \right) - 1 \right\} \dots\dots\dots(2.5)\end{aligned}$$

where r is the radius of gyration.

By combining the bending and direct stresses, the maximum compressive stress is then obtained

$$\sigma_1 = \frac{P}{A} \left\{ 1 + \frac{e y_1}{r^2} \sec \left(\frac{L}{2r} \sqrt{\frac{P}{EA}} \right) \right\} \dots\dots\dots(2.6)$$

This equation gives the maximum compressive stress in the column for any value of P and e provided that this stress does not exceed the elastic limit.

2.1.6 Representation of Imperfections by Equivalent Eccentricity

In general, columns have imperfections such as initial curvature, errors in centering the load, uncertainty as to end conditions, etc. Thus, for an axially loaded column, the compressive load will produce more or less bending in addition to direct compression. On this basis it seems logical to conclude that the behaviour of a real imperfect column under load will be similar to that of a perfectly straight ideal column loaded with a suitable eccentricity e_o . This suggests that the secant formula could be used also as a design for

supposedly straight centrally loaded columns simply by choosing an appropriate value of the equivalent eccentricity to account for the effect of imperfections. Then the secant formula for this purpose may be given as follows

$$\sigma_1 = \frac{P}{A} \left\{ 1 + \frac{e_o y_1}{r^2} \sec \left(\frac{L}{2r} \sqrt{\frac{P}{AE}} \right) \right\} \dots\dots\dots(2.7)$$

The value of equivalent eccentricity e_o was recommended to be $e_o = L/400$ for steel columns (27), or $e_o = L/500 + B/50$ for other material (25), where B is the width of the section in the plane of bending.

2.1.7 Empirical Column Formulas

Because of the objections to the transcendental nature of the secant formula, many simpler but wholly empirical formulas have been proposed as substitutes for it (25,27). These formulas usually give the permissible axial load P as a function of the slenderness ratio L/r , without specifically indicating the factor of safety used.

2.1.7.a Rankine-Gordon Formula

One of the empirical formulas for pin-ended, centrally loaded columns is the Rankine-Gordon formula. The derivation was started with the equation

$$\frac{1}{P} = \frac{1}{P_E} + \frac{1}{P_y} \dots\dots\dots(2.8a)$$

where P is the total failure load, P_E is the Euler load and P_y is the yield load of the material. Substituting the equation of P_E (Eqn.2.2) and replacing yield load P_y by yield stress σ_y , the Rankine-Gordon

formula is obtained as

$$P = \frac{\sigma_y A}{1 + \frac{\sigma_y}{2\pi E} \left(\frac{L}{r}\right)^2} \dots\dots\dots(2.8)$$

which is used in British Code of Practice CP114.

2.1.7.b Johnson's Parabolic Formula

In order to make greater allowance for the effect of imperfections for the more slender columns, a parabolic formula is often used (27), such that

$$P = f_y A \left\{ 1 - c_1 \left(\frac{L}{r}\right)^2 \right\} \dots\dots\dots(2.9)$$

where c_1 is a numerical factor and f_y is the yield strength.

2.1.7.c Straight-line Formula

One of the most commonly used empirical column formulas is the Straight-line formula (27)

$$P = f_y A \left\{ 1 - c_2 \left(\frac{L}{r}\right) \right\} \dots\dots\dots(2.10)$$

where c_2 is a numerical factor. The Straight-line formula is used by NBC (National Building Code of Canada) for steel columns.

2.2 Previous Investigations of Triaxial Effects on Plain Concrete and Reinforced Concrete Columns and Cylinders

Many investigations of the load capacity of mortar, plain and reinforced concrete under triaxial compression have been carried out since the beginning of this century by using fluid pressures or mechanical restraints. However, as far as known to the author, all of these

investigations considered the longitudinal load applied concentrically.

2.2.1 Survey of Previous Investigations

In 1903 and 1906, Considere (9) investigated the triaxial behavior of cement mortar. Cement mortar cylinders 11.8" in diameter and 31.5" long were used. The lateral pressures on the mortar were applied by water under pressure. From these results, Considere obtained a formula for the axial stress at failure σ_1 in terms of the unconfined mortar crushing strength f'_c and lateral pressure $\bar{\sigma}_3$

$$\bar{\sigma}_1 = a f'_c + 4.8 \bar{\sigma}_3 \dots\dots\dots(2.11)$$

where a is a constant increasing with $\bar{\sigma}_3$ in the range of 1.0 to 1.5.

In 1903, Considere (9) also reported a research carried out on the behavior of hoop reinforced and restrained concrete. A number of concrete specimens with various amounts of hoop reinforcement were tested. It was found that the hooped specimens initially behaved in a similar manner to nonreinforced specimens. However, the hooped specimens carried much higher loads than the unhooped specimens and only failed after extensive cracking had become apparent and large deformations had occurred. It was also found that the hooped concrete showed no drop off in load with deformation, exhibiting a plastic type of behavior.

Considere also noted that the ductility of restrained mortar and concrete was much increased over the unrestrained cylinders.

In 1928, Richart, Brandtzaeg, and Brown (23) carried out investigations of concrete in triaxial compression at the University of Illinois Engineering Experiment Station. The method used was that of applying a liquid pressure to the sides of a cylinder which was loaded in the axial direction in a testing machine. The specimens were enclosed inside a rubber tube to protect them against penetration of oil during the test. Each specimen was treated by rubbing the surface thoroughly with a paste of plaster of Paris until all surface cavities were filled in order to prevent the edges of small holes or cavities in the surface puncturing the rubber sheath. Two groups of specimens were tested.

In the first group, eighty concrete cylinders, 4" in diameter and 8" long, from three different mixtures, were tested. Sixty-four of these specimens were tested in three-dimensional compression, the two principal stresses produced normal to the axis of the cylinder by liquid pressure being smaller than the axial stress. Eight nominal intensities of oil pressure were used, ranging from 250 to 4000 psi. For comparison, sixteen specimens were tested in simple compression. From the results of the first group, it was concluded that the maximum unit load $\bar{\sigma}_1$ carried may be considered as made up of the strength f'_c of the concrete in simple compression plus an added strength equal to 4.1 times the lateral pressure $\bar{\sigma}_3$ developed at the time of failure. That is

$$\bar{\sigma}_1 = f'_c + K \bar{\sigma}_3 \dots\dots\dots(2.12)$$

where $K = 4.1$, is a coefficient for $\bar{\sigma}_3$.

In the second group, forty-eight concrete cylinders, 4" in diameter and 22" long, were tested with the two principal stresses normal to the axis of the cylinders larger than the axial stress. The second group results, also showed that

$$\sigma_{\max} = f'_c + 4.1 \sigma_{\min} \dots\dots\dots(2.12a)$$

where $\sigma_{\max}$ = the maximum principal stress, and

$\sigma_{\min}$ = the smallest principal stress.

Test results of the two groups were presented in table form and graphs showing a straight-line relationship between axial failure stress and lateral stresses. It was concluded that the rate of increase in the strength with increase in the smallest principal stress was largely independent of the proportions of the concrete mixture, and that the magnitude of the maximum principal stress developed was roughly equal to the strength of the concrete in simple compression plus 4.1 times the lateral pressure.

In 1929, Richart, Brandtzaeg and Brown (24) extended the study to spirally reinforced columns. These results, similar to Considere's investigations, showed that the failure stress of hooped concrete specimens was higher than that of unhooped specimens, and could be predicted with the expression obtained in 1928,

$$\bar{\sigma}_1 = f'_c + 4.1 \bar{\sigma}_3 \dots\dots\dots(2.12)$$

In 1944, Maney (20) investigated short cylindrical columns with artificial triaxial compression. The diameters of specimens varied

from 2" to 8", and the height was approximately four times the diameter. The specimens were confined laterally by a combination of a steel metal pipe-shaped form and spiral steel just outside the steel pipe-shaped form. The prestressing in the spirals was achieved by compacting the concrete with high pressure and vibration after casting. In this way, when the load-application unit was removed from the specimen the concrete had enough strength to keep the enclosed spiral in tension. All the specimens were tested to failure. It was concluded that the compressive load capacity of a pound of steel could be increased by at least 500 per cent and the cost of the structure could be reduced to 1/3 to that of steel structures.

In 1949, Balmer (4) conducted research at the U. S. Bureau of Reclamation similar to that of Richart, Brandtzaeg, and Brown (23). The lateral confining pressure, ranging from 0 to 25000 psi, was applied first and held constant while the axial stress increased until the concrete failed. From the test results of all specimens, 6" in diameter and 12" long, Balmer assumed the same form of equation as proposed by Richart and his associates, but found that the coefficient K for the lateral pressure $\bar{\sigma}_3$ varied from 6.00 to 2.00. For lower compressive loads the coefficient K was closer to 6.00 than to 4.10, while under high loads it varied between 3.00 and 2.00.

In 1950, Johnson (17) carried out an investigation of spirally prestressed short columns 3" in diameter and 12" long. A total of 135 specimens were cast, 27 were of plain concrete without any

reinforcement, while 108 specimens were spirally prestressed. The prestressing was achieved by wrapping the concrete cylinder with wire under tension. The cylinders were spun in a lathe with the wire being drawn through a pair of friction plates. The tensile force applied in the wire was read by means of a spring-scale attached to the friction plates.

The compressive strength of concrete used by Johnson ranged from 3470 to 4870 psi. Three wire sizes, 0.0317", 0.0475" and 0.0625" in diameter, with the ultimate strengths of 253 ksi, 395 ksi and 578 ksi respectively, were used with pitches of 3, 5, 7 and 9 wires per inch. Three different initial lateral prestresses of 300, 1500 and 3000 psi were applied to the specimens. All cylinders were tested to complete failure with a hydraulic testing machine. Most failures were similar in appearance. The typical concrete failure began as local crushing near the weak end of the specimen -- the end which was at the top when cast. On final failure, however, the majority of specimens showed the so-called shearing failure along a conical surface. From the test results, Johnson concluded that all spirally prestressed specimens had higher load carrying capacity than their unprestressed counterparts. The increase in load carrying capacity varied directly with the pitch of the wire wrapped. The magnitude of K in the University of Illinois equation (Eqn.2.12) was calculated from the experimental values of $\bar{\sigma}_1$, f_c , and $\bar{\sigma}_3$ and was found to range from 3.60 to 5.77.

In 1958, Gitman reported Goldenblatt-Ratz investigations (15) on laterally prestressed concrete columns, 14" in diameter and 118"

long. The specimens were produced first as thin-wall concrete pipes ($1\frac{1}{4}$ " - $1\frac{1}{2}$ ") on a spinning machine and afterward filled with concrete. Longitudinal reinforcement of nine $\frac{1}{4}$ " diameter mild steel bars and spiral reinforcement of $\frac{5}{32}$ " diameter steel wire with pitches varying from 4" to 8" on different specimens were used to resist handling and transportation stresses. The circumferential prestress was applied by wrapping $\frac{7}{32}$ " diameter wires around the columns, 9 specimens with a pitch of 0.6" and 5 with a pitch of 1.2". The initial stresses in the wires were 90 ksi for 11 of the 14 columns and 7 ksi for three others. Five specimens of the 11 with full circumferential prestress were covered with a protective shell.

All these specimens were loaded to failure, with spherical hinges between the loading plates and the top and bottom of the specimens. The test results showed that the failure compressive stress of the specimens with a pitch of 0.6" was increased 100 per cent of that of plain concrete specimen, while that of the specimens with a pitch of 1.2" was increased only 50 per cent. It was also found that, with the same pitch of wire, the specimens covered with a protective shell had higher failure stress than the specimens without a protective shell.

In 1961, Akroyd (1) studied the effect of pore-water pressure on the ultimate strength of concrete subjected to triaxial stresses. Cylindrical specimens, 3" in diameter and 6" long were cured in their cast-iron moulds for the first 24 hours after casting, then were cured in various ways until tested at the age of 7 days with a high-pressure triaxial machine. One series was cured in water at 18° C, a second

series was coated with an epoxy resin membrane to keep the water content constant, a third series was cured in water for three days and then dried at 100°C for two days. A fourth series was dried at 100°C and a fifth series was dried in silica gel. From these undrained tests of the specimens, Akroyd concluded that saturated concrete behaves like a material which does not possess the property of internal friction, when tested triaxially under condition of no change of water content (undrained). Akroyd's results were found to be similar to those of Balmer (4). Akroyd obtained the same relationship between the major (vertical) principal stress $\bar{\sigma}_1$ and the minor (lateral) principal stress $\bar{\sigma}_3$, with the form

$$\bar{\sigma}_3 = c_2 \bar{\sigma}_1^{c_3} + c_1 \dots\dots\dots(2.13)$$

which was developed by Balmer (4). For one series of Akroyd's tests, c_1 , c_2 and c_3 had the values of

$$\begin{aligned} c_1 &= -430 \\ c_2 &= 0.0051 \\ c_3 &= 1.37 \end{aligned}$$

In 1961 and 1962, Gambarov (12,13) carried out a research program dealing with the short-time effect of axial load on circumferentially prestressed columns. The program was based on round column specimens 5½" in diameter and 31½" long. Four groups of specimens were tested, one containing no circumferential wire and other three containing 1.08 per cent circumferential prestressing wire of 0.1" diameter. The initial prestresses in the wires were 7.1, 108 and 184.5

ksi. Each group had seven specimens. Three specimens in each group were tested at the age of $1\frac{1}{2}$ months and the remainder at the age of 1 year. It was found that the wrapped specimens had higher rupture strengths than their unwrapped counterparts and the rupture strength increased as the initial prestress increased.

In 1962, Feaser and Chinn (10,11) reported experiments on spirally prestressed concrete columns. Their project can be considered as a continuation of Johnson's work (17). Thirty-four short columns 3" in diameter and 12" long were tested. The average unconfined concrete compressive strength was 5230 psi. Circumferential prestressing was achieved by turning the specimen in a lathe and wrapping the specimen with wire drawn through a friction-plate device. The tension in the wire was measured by strain-recorder and the friction-plate device was calibrated for various tensions in the wire. Two sizes of wire, 0.054" and 0.0595" in diameter, were used. The pitches of wire were 0.0625", 0.1", 0.165" and 0.25". Four different tensions of 10, 100, 200 and 300 lbs. per wire were applied to the wire while wrapping. Thus, the initial lateral pressure on the concrete surface ranged from 27 psi to 2133 psi. In the testing, the failure of all specimens coincided with the rupture of the spiral wire. The lateral pressure at ultimate then ranged from 1533 psi to 6120 psi. From the test results, the average value of K in the University of Illinois equation (Eqn.2.12) was found to be 3.818. Hence the equation for predicting the ultimate strength of wrapped cylinder was given by

$$\bar{\sigma}_1 = f'_c + 3.818 \bar{\sigma}_3 \dots\dots\dots(2.14)$$

It was also shown that the stiffness of the specimens can be increased by increasing the initial tension in the spiral wire. However, the initial tangent modulus of all laterally prestressed concrete was less than that of the unstressed plain concrete. It was also concluded that the ultimate strength of the specimen was independent of the initial tension in the spiral wire but dependent only on the lateral confining stress to the specimen at the time of failure.

In 1965, Chinn and Zimmerman (7,28) extended the work of Balmer (4). Their program consisted of increasing the axial load until the concrete failed while the cell liquid pressure was held constant. Thirty-six concrete cylinders, 6" in diameter and 12" long, were tested using cell liquid pressures of 20000 psi and 60000 psi. From their test results, they concluded that the axial strength increases as the lateral stress increases.

In 1965, Krah1, Victory, Erkmen and Sims (18) reported the research of the behavior of plain mortar and concrete under triaxial stress. Mortar cylindrical specimens, $\frac{1}{2}$ " in diameter and 1" long, were drilled from cast blocks of mortar with a diamond drill and were cut to length with a diamond saw. Four mortar mixes were used, one of which included light weight aggregates. Approximately 50 specimens were tested from each of four mixes. Each specimen was tested under constant hydrostatic confining pressure, as axial load was increased to failure. Some specimens were protected from the confining fluid by a plastic membrane and others were not. From test results, they concluded that

the use of very small cylinders of mortar in triaxial tests is feasible and meaningful results are still obtained provided special care is devoted to the preparation of specimens, and that the effect of confining pressure on the axial strength of a mortar or concrete cylinder can be expressed by the equation

$$\bar{\sigma}_1 = f'_c + c_1 \bar{\sigma}_3^{c_2} \dots\dots\dots(2.15)$$

where c_1 and c_2 are empirical constants. This equation implies that

$$\log (\bar{\sigma}_1 - f'_c) = \log c_1 + c_2 \log \bar{\sigma}_3 \dots\dots\dots(2.16)$$

that is, a linear relationship exists between $\log (\bar{\sigma}_1 - f'_c)$ and $\log \bar{\sigma}_3$. The value of the constants were determined by the method of least square and were presented in table form in their paper.

In 1966, Ben-Zvi, Muller and Rosenthal (5) carried out an investigation of the effects of triaxial stress caused by circumferential prestressing of concrete columns. Prestressing was achieved by wrapping the column with steel wires under tension. Three sizes of concrete specimens were used in their investigation.

In the first series, thirty-two of the total forty-four concrete cylinders, 3" in diameter and 6" long, were circumferentially prestressed with initial tensions in the wire of 7.10, 9.22 and 163.5 ksi. Two sizes of wire were 0.04" and 0.055" in diameter and the pitches were 0.04" and 0.067" respectively. All specimens failed by rupture of the spiral prestressing wire due to the lateral pressure exerted on it through the expansion of the concrete.

In the second series, six specimens, 10" in diameter and 100" long, with average concrete strength of 4260 psi, were cast with longitudinal reinforcement of six 0.4" diameter wires and a spiral of 0.24" diameter wire. The columns were prestressed longitudinally by means of a 0.24" cable, with a force of 32 tons. Four of the specimens were circumferentially prestressed with steel of 0.2" diameter with an initial prestress of 99400 psi in the wire and a pitch of 0.4" or 0.8". In the test of this series, columns without wrapping failed by concrete compression crushing. The wrapped ones failed by buckling and rupture of the winding wire.

In the last series, a total of fifty one concrete columns with an average cube strength of 6800 psi, were cast in three different lengths, 24", 36" and 72". All were 3" in diameter. Specimens were reinforced and prestressed in three different combinations of: longitudinal reinforcement containing six wires 0.12" diameter and a spiral 0.08" diameter, spiral reinforcement of 2.17" diameter spiral with 1" pitch, six longitudinal prestressing wires of 0.2" diameter with a prestressing of 2 tons in each wire, and circumferential prestressing of 0.055" diameter spiral steel wires in one, two or three layers with a pitch of 0.06" and an average initial tension of 7.1 or 163 ksi in the wire. All specimens were tested under axial compression. Failure of the short unwrapped and some short wrapped specimens occurred in vertical and oblique planes while all other specimens generally failed by buckling.

From these tests, Ben-Zvi and his associates concluded that column strength is increased with an increase of initial spiral prestressing, but decreased with the increase of the slenderness ratio of the column, and that longitudinal prestressing does not reduce column

ultimate strength and has a beneficial effect on deformation and transportation stresses. They proposed a new form of the University of Illinois equation (Eqn.2.12)

$$\bar{\sigma}_1 = (f'_c + f'_{sl}) + \beta K \bar{\sigma}_3 \dots\dots\dots(2.17)$$

where f'_{sl} is the compressive strength increase due to longitudinal reinforcement and longitudinal prestress. β is a slenderness correction coefficient. The value of βK varied from 0.15 to 3.52. They also proposed a design formula for triaxially prestressed columns. The failure load P_{ult} of such column is

$$P_{ult} = P_c + P_{sl} + P_{sp} \dots\dots\dots(2.18)$$

$$\text{in which, } P_c = A_c f'_c \dots\dots\dots(2.18a)$$

load carried by concrete alone,

$$P_{sl} = A_c (p' f'_y + 2.5p'' f''_y + p''' f'''_y) \dots\dots\dots(2.18b)$$

load carried by longitudinal reinforcement including spiral ties and prestressing wires,

$$\text{and } P_{sp} = A_c \beta K \bar{\sigma}_3 \dots\dots\dots(2.18c)$$

load carried by spiral prestressing wire through lateral pressure.

where A_c is the cross-sectional area of concrete, excluding any finishing material and reinforcement,

p' is the ratio of volume of reinforcement to $A_c \times$ unit length,

p'' is the ratio of volume of spiral ties to $A_c \times$ unit length,

p''' is the ratio of volume of longitudinal prestressing wire to $A_c \times$ unit length,

and f'_y , f''_y , f'''_y are yield stresses of steel, spiral ties and

longitudinal prestressing wires respectively.

In 1968, Martin (21) reported experiments on 17 spirally prestressed concrete cylinders, 4" in diameter and 8" long. The cylinders were wrapped with steel piano wires of 0.016" diameter in a lathe. The pitches of the wire were 0.016", 0.018" and 0.032" with the initial lateral stresses varying from 140 to 1482 psi. Plain concrete compressive strength was 6490 psi with a standard deviation of 357 psi. Specimens were tested on short-term and long-term axial compression. Martin concluded that the increase in axial ultimate stress and strain of these cylinders depended essentially on the lateral ultimate stress that the spiral wire caused in the concrete. For wrapped specimens, the value of Poisson's ratio varied from 0.16 at small stress to more than 1.0 at large stresses during rapid loading.

In 1969, Gardner (14) reported the use of a soil mechanics theory to predict the behavior of concrete under conditions of triaxial stress. An experimental investigation was carried out which justified the use of the theory which relates the stress in the concrete to the instantaneous Poisson's ratio. Twenty eight concrete cylinders, 3" in diameter and 6" long, with compressive strength of 4000 psi, were tested to failure in a triaxial apparatus under lateral oil pressures of 0, 1250, 2500 and 3750 psi. Gardner concluded that all the mechanical properties of concrete were improved under triaxial loading, and that the failure strength, ductility, and the value of the instantaneous Poisson's ratio at failure all increase with an increase in confining pressure.

In 1969, Godse (16) carried out an experimental investigation of the effects of initial prestress and slenderness ratios on the ultimate load carrying capacity of axially loaded spirally prestressed concrete columns. A total of forty-four 6" diameter concrete cylinders were cast in four series of 12", 24", 42" and 60" long. The compressive strengths of concrete were 4600, 5100, 5000 and 4950 psi respectively. For the two longer series, longitudinal reinforcing of 4-No.3 bars and No.2 ties at 6" c/c were used to take care of stresses due to transportation and handling. From each series, four levels of initial prestress were employed, being 0, 960, 1706 and 3412 psi, with pitches of 0 (unwrapped), 18, 32 and 64 wires per inch. Initial lateral prestressing was achieved by turning the specimen in a lathe and wrapping high strength piano wire, under tension, around the specimens. High strength piano wire of 0.031" diameter with 342 ksi average ultimate stress was used.

All specimens were loaded axially and concentrically to failure. Loads were applied through spherically-seated bearing blocks at the top and bottom of the specimens. From the test results, Godse concluded that the load carrying capacity can be increased by the increase in initial spiral prestress, but will be decreased as the slenderness ratio of the column increased. Empirical coefficient βK of Eqn.2.17, calculated from test results, ranged between 1.23 to 3.90 .

2.2.2 Conclusions on Previous Investigations

From the results of the previous investigations which have been reviewed in section 2.2.1, the following conclusions are made:

- 1) Concrete column strength can be increased by applying a lateral pressure or lateral prestressing.
- 2) For the short plain concrete column, the axial compressive strength can be predicted by the Illinois equation (Eqn.2.12)

$$\bar{\sigma}_1 = f'_c + K \bar{\sigma}_3$$

where K is itself a function of $\bar{\sigma}_3$ which decreases with increase of $\bar{\sigma}_3$.

- 3) The coefficient K is also a decreasing function with increase in slenderness ratio. Hence the Illinois equation (Eqn.2.12) can be modified to

$$\bar{\sigma}_1 = f'_c + \beta K \bar{\sigma}_3$$

The value of β decreases with increase in slenderness ratio.

- 4) For columns where the lateral stress is provided by wire wrapping, the load is governed by the final, not the initial, stress in the wire.

CHAPTER 3

MATERIALS AND TESTING PROCEDURE

The experimental program described in this thesis consisted of casting a number of columns of different lengths, with different amounts of lateral prestress, and loading these columns to failure with the loads being applied at various eccentricities. The columns were laterally prestressed by turning them in a lathe and wrapping them with piano wire under tension. All tests were duplicated to improve reliability.

The casting, curing, lateral prestressing and testing details will be described later in this chapter. Fig.3.1 p.26 gives details of the number and types of the test columns.

3.1 Preparation of Specimens

3.1.1 Casting

3.1.1.a Specimens

A total of 96 cylindrical specimens 6" in diameter were cast in four different lengths of $17\frac{1}{2}$ ", $29\frac{1}{2}$ ", $47\frac{1}{2}$ " and $65\frac{1}{2}$ " and were labelled A, B, C and D series respectively. When attached to the loading plates and spherical bearings, these four lengths would become effective lengths of 24", 36", 54" and 72" respectively.

Two steel stands, accommodating 24 moulds at a time, were used to hold the moulds vertical while casting.

Fig.3.1 Specimens Details

Specimen No.	Length		$\frac{L}{r}$	No. of spec.	f'_c (psi)	Layers of wires	wires inch	σ_3 (psi)	$\frac{e}{D}$
	overall l	effective L							
A- 0-a A- 0-b A- 0-c	$17\frac{1}{2}"$	24"	16	2	7200	nil	0	0	0.10 0.25 0.50
A-18-a A-18-b A-18-c						1	18	900	0.10 0.25 0.50
A-32-a A-32-b A-32-c							32	1600	0.10 0.25 0.50
A-64-a A-64-b A-64-c						2	64	3200	0.10 0.25 0.50
B- 0-a B- 0-b B- 0-c	$29\frac{1}{2}"$	36"	24	2	7460	nil	0	0	0.10 0.25 0.50
B-18-a B-18-b B-18-c						1	18	900	0.10 0.25 0.50
B-32-a B-32-b B-32-c							32	1600	0.10 0.25 0.50
B-64-a B-64-b B-64-c						2	64	3200	0.10 0.25 0.50
C- 0-a C- 0-b C- 0-c	$47\frac{1}{2}"$	54"	36	2	6750	nil	0	0	0.10 0.25 0.50
C-18-a C-18-b C-18-c						1	18	900	0.10 0.25 0.50
C-32-a C-32-b C-32-c							32	1600	0.10 0.25 0.50
C-64-a C-64-b C-64-c						2	64	3200	0.10 0.25 0.50
D- 0-a * D- 0-b * D- 0-c *	$65\frac{1}{2}"$	72"	48	2	5600	nil	0	0	0.10 0.25 0.50
D-18-a * D-18-b * D-18-c *						1	18	900	0.10 0.25 0.50
D-32-a * D-32-b * D-32-c *							32	1600	0.10 0.25 0.50
D-64-a * D-64-b * D-64-c *						2	64	3200	0.10 0.25 0.50

* 4-No.3 longitudinal bars and No.2 ties at 6" c/c were used in series D.

For each series, twenty 6"x12" cylinders were cast to determine the compressive strength.

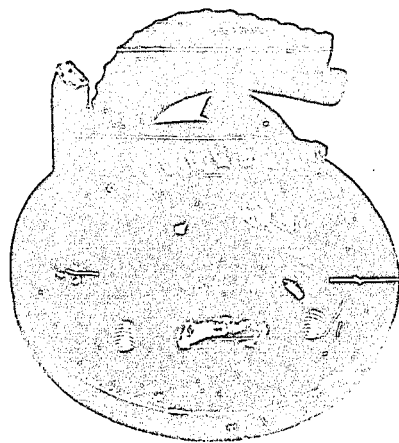
3.1.1.b Moulds and End Caps

The moulds were cut to the required lengths from 6"x8'-0" sono-tubes. To be able to bolt up the specimen to the loading plates or the cap for wire-wrapping, steel anchor caps were placed at both ends of each specimen during the casting of specimens.

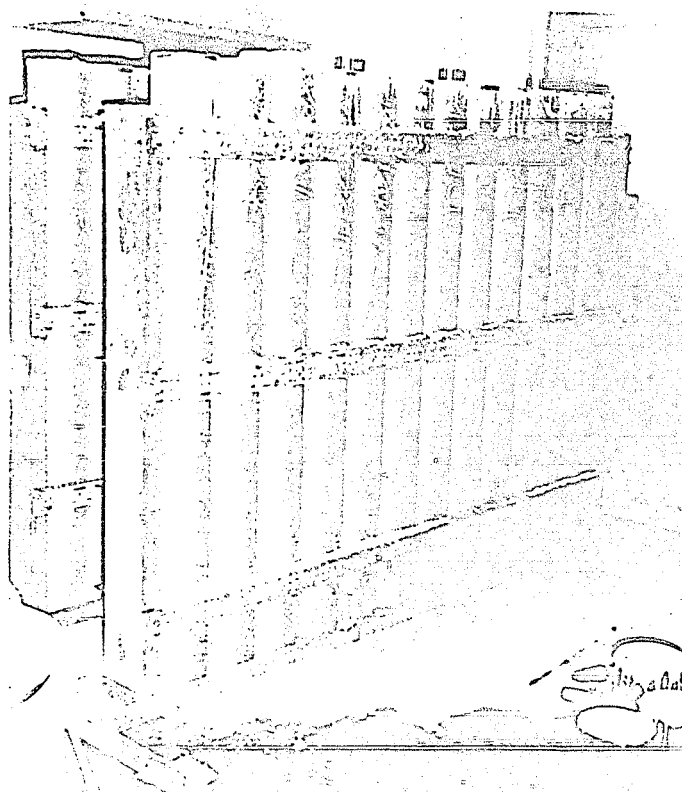
The steel end caps were 6" in diameter, $\frac{3}{4}$ " thick, with four $\frac{5}{8}$ "-diameter threaded holes for bolting up to the loading plates or the cap for wire-wrapping. Two spiral bars and one straight bar were welded to the end cap as an anchor between the end cap and the concrete portion of the specimen. The straight bar was on the tension side of the cylinder to increase the bond when tensile stress occurred. The top end caps had three $\frac{1}{4}$ " holes for bleeding of water and air during casting. At the edge of each end cap, two $\frac{1}{4}$ " threaded holes, $\frac{1}{2}$ " deep were made for anchoring the ends of wrapping wire. The end cap is shown in Fig.3.2 p. 28 .

The eccentric load was applied by the ball seats of the loading plattens of the columns being displaced from the section centroid the desired distance. In order to enable the application of eccentric load, the axes of top and bottom end caps had to be on the same plane. This was achieved by marking the axes on the end caps and one longitudinal reference line along the sono-tube before casting, and matching these axes and the line while casting.

Fig.3.3 p.29 shows the fabrication details of specimen.

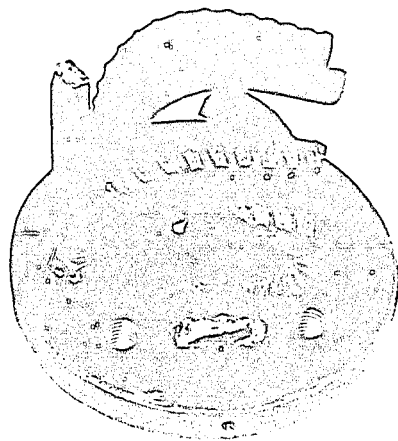


(a) END CAP

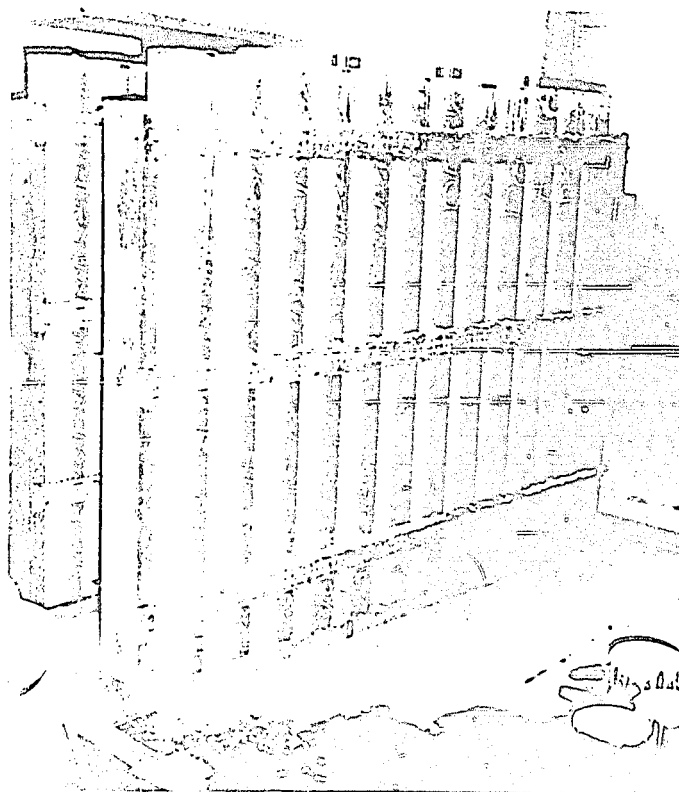


(b) CASTING OF SPECIMENS

FIG. 3-2 END CAP AND CASTING
OF SPECIMENS



(a) END CAP



(b) CASTING OF SPECIMENS

FIG. 3-2 END CAP AND CASTING
OF SPECIMENS

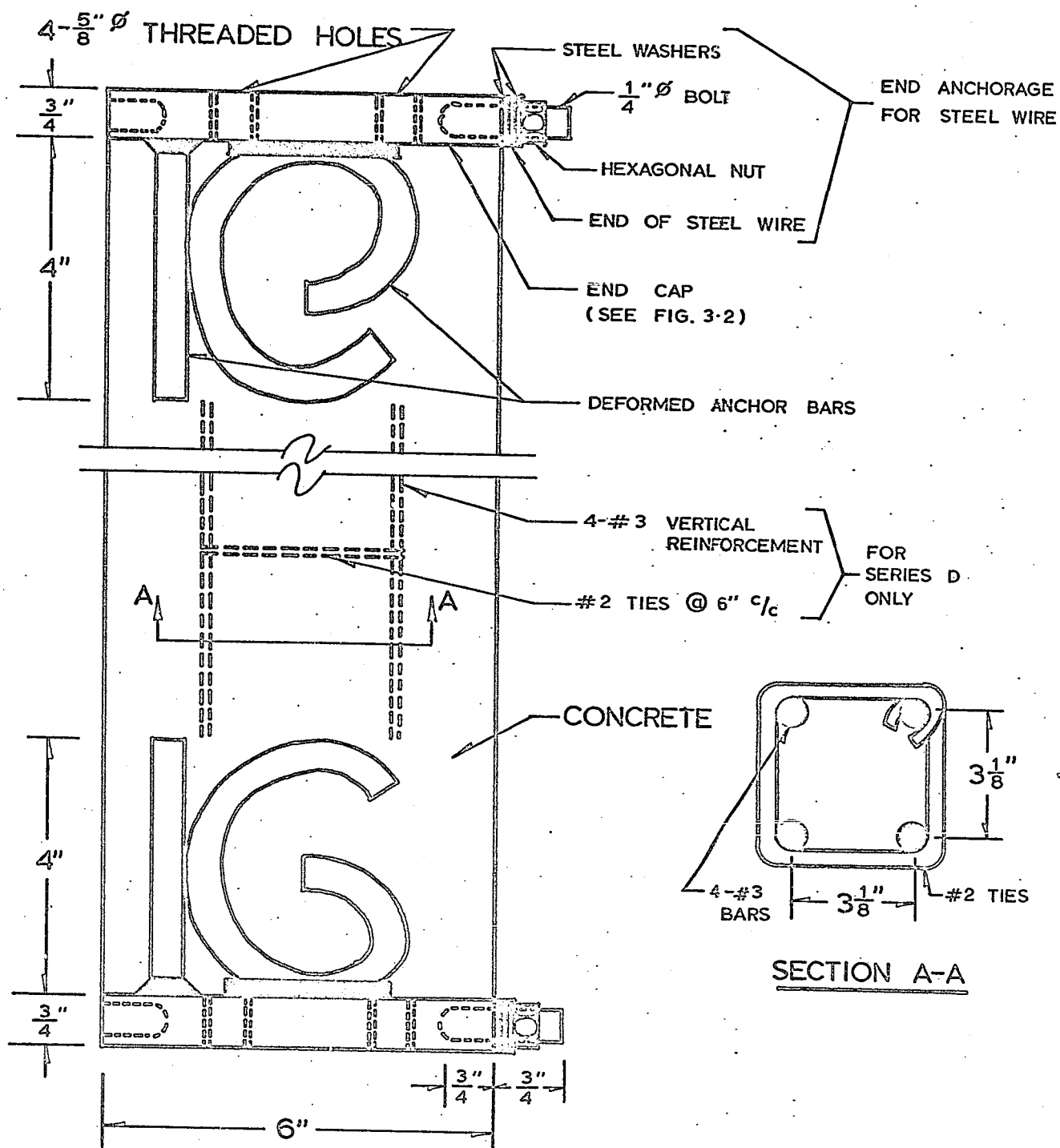


FIG. 3-3 FABRICATION DETAILS OF SPECIMEN

3.1.1.c Concrete Mix

The concrete used in all specimens was obtained from the Dominion Building Material Co., a local ready-mixed concrete manufacturing plant. The concrete was proportioned by weight and machine mixed. The mix proportion of the concrete is given in Fig.3.4 p.30 .

Fig.3.4 Mix Proportions by Weight

(slump = 3")

Material	Weight per cubic yard of concrete
Normal Portland Cement	615 lbs.
Sand (Type 1)	1,570 lbs.
3/8" Aggregate	1,585 lbs.
Water	310 lbs.

3.1.1.d Reinforcement

To avoid damage of the longest columns in transportation and to take care of the stresses due to the self weight of column as well as the lateral force of the prestressing wire while the column was supported in the lathe, longitudinal reinforcing was provided for the series D columns only.

Nominal reinforcing of 4-No.3 bars, $3\frac{1}{8}$ " centre to centre, and No.2 ties at 6" centre to centre (to comply the 1965 National Building Code of Canada requirement, section 4.5.3.67(2); p.22) were provided. The arrangement of these bars is shown in Fig.3.3 p.29 .

Stress-strain characteristic of these bars is shown in Fig.A.2 p.125 in Appendix A.

3.1.2 Curing

After casting, all specimens were left in the moulds for 24 hours. All specimens were cured vertically in a moist room after stripping the moulds. The curing time of all four series was at least 28 days.

3.1.3 Wrapping of Cylinders

The specimens were taken out of the moist room and air dried for at least 7 days before wrapping. The columns were laterally prestressed by wrapping high-strength wire, under tension, around the columns. This was done by turning the column slowly in a lathe, the rotation of the column drawing the wire through a pair of adjustable clamped friction plates (washers) of the wrapping device which was fixed to the tool carriage of the lathe. Fig.3.5 p.32 shows the wrapping of cylinder.

Both ends of the wire were anchored at the end caps by passing the wire between the two mild steel washers and clamping them with a nut. This arrangement is shown in Fig.3.3 p.29 .

The pitches of the wire were chosen to be similar to the cutting threads of a lathe, and the tool carriage could be traversed at the given pitch by adjusting the set up of the lathe. For this experiment, pitches of 18 and 32 threads per inch were used. The wrapping of 64 wires per inch was done by a double layer of 32 wires per inch.

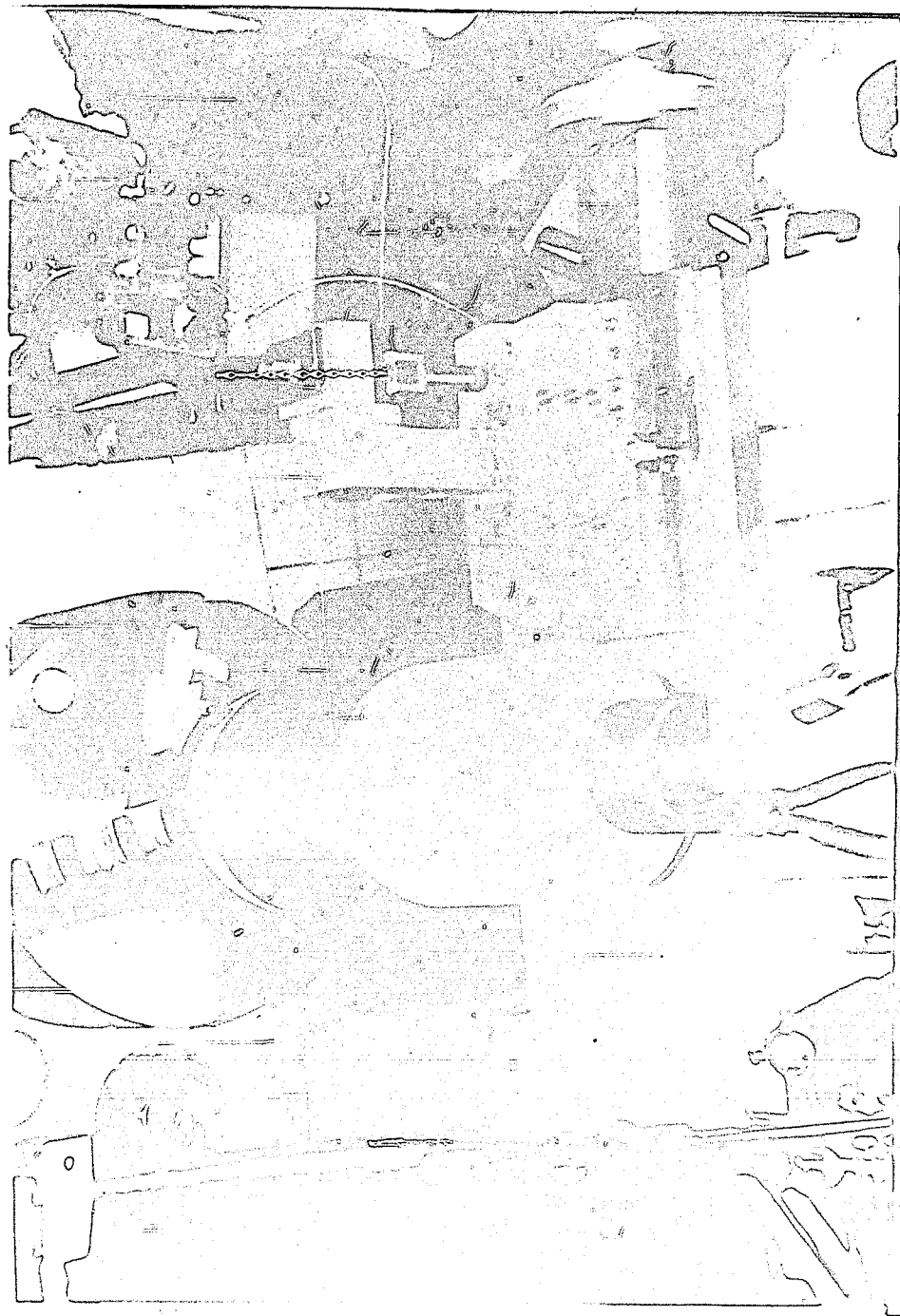


FIG. 3.5 WRAPPING OF CYLINDERS

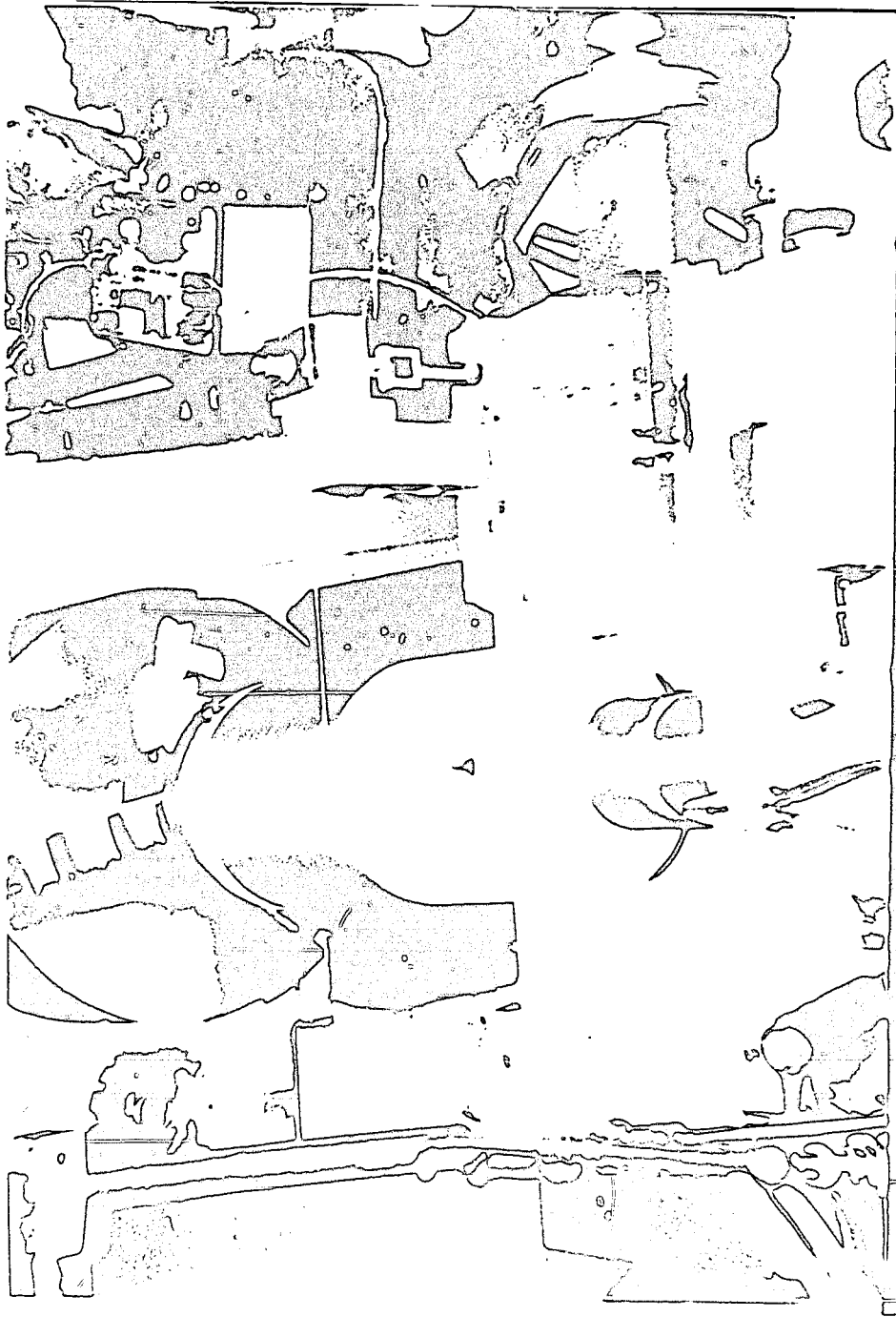


FIG. 3.5 WRAPPING OF CYLINDERS



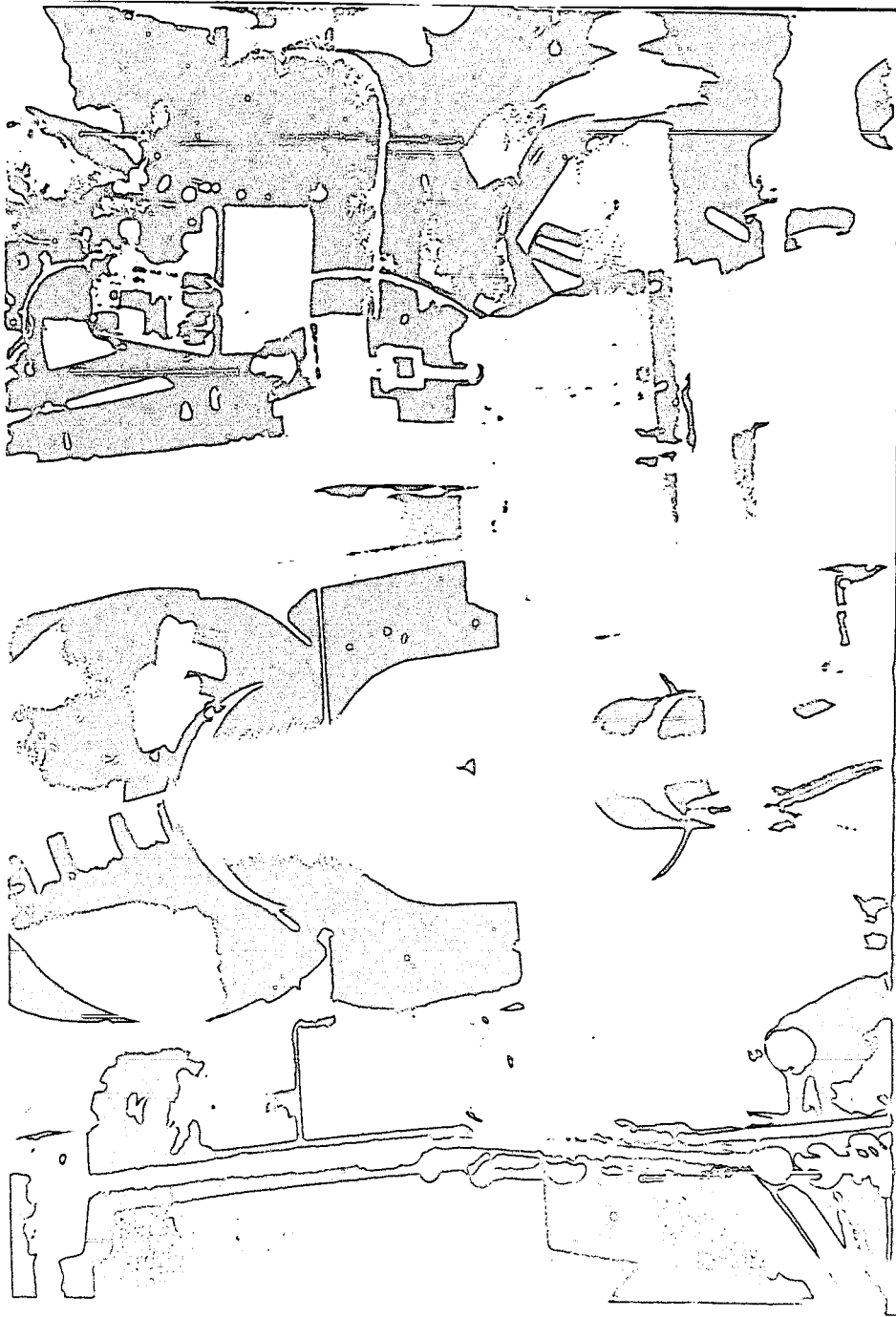


FIG. 3.5 WRAPPING OF CYLINDERS



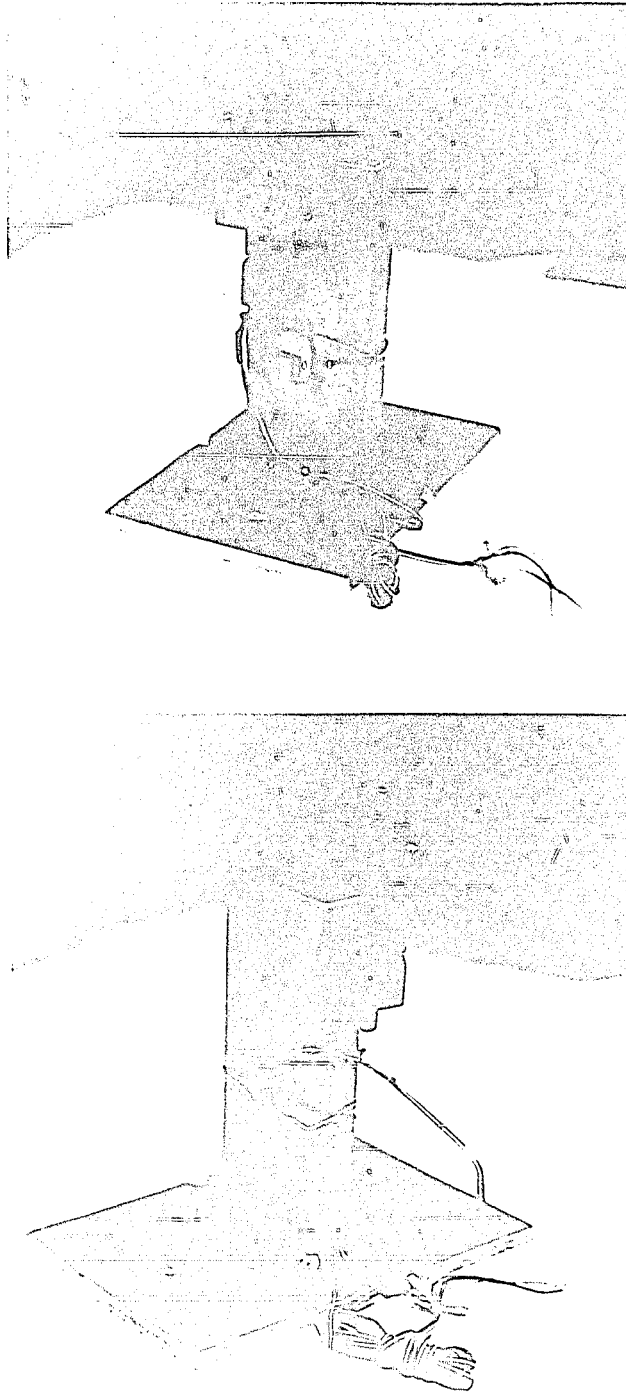


FIG. 3·6 WRAPPING DEVICE

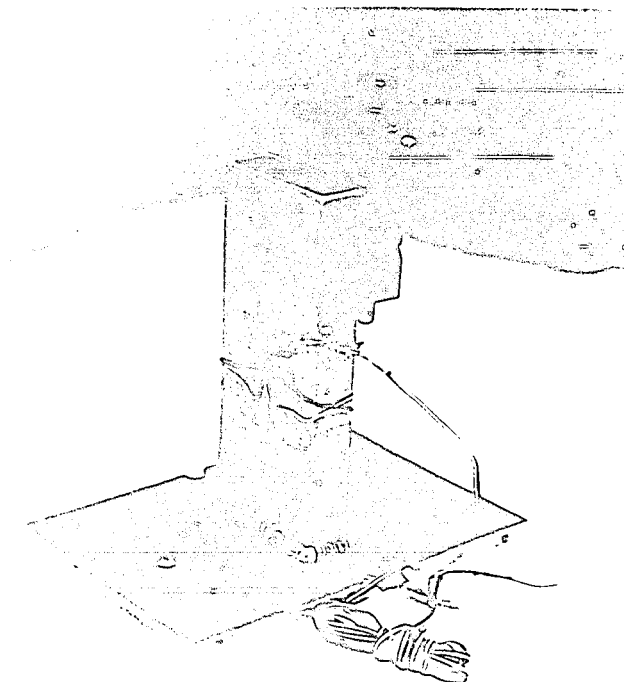
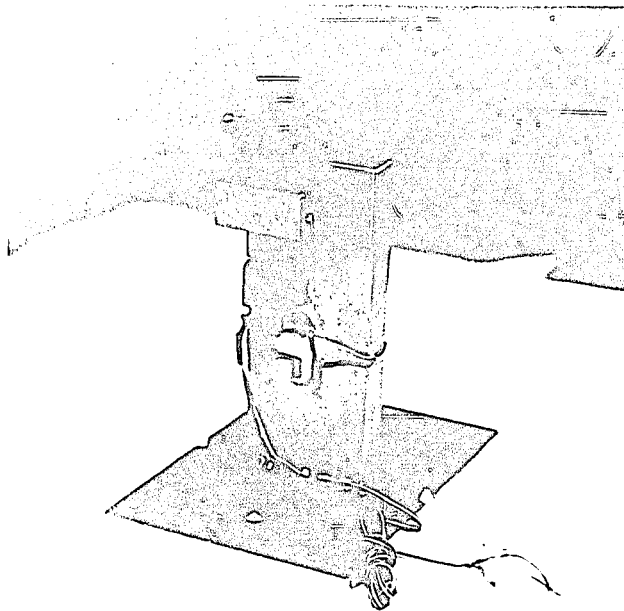


FIG. 3·6 WRAPPING DEVICE

3.1.3.a Wire for Wrapping

The wire used to provide the lateral spiral prestressing was high strength piano wire of 0.031" in diameter. The average ultimate strength of the wire was 342000 psi. The stress-strain characteristic is shown in Fig.A.1 p.124 in Appendix A.

3.1.3.b Wrapping Device

The device for wrapping consisted of three parts which were a mounting plate, a vertical cantilever and a pair of adjustable clamped friction plates. It is shown in Fig.3.6 p.33 .

The aluminium bottom mounting plate was 9"x6"x3/8", with four holes for fixing the device on the tool carriage of the lathe. The aluminium vertical cantilever, a piece of 3"x1"x $\frac{1}{8}$ " aluminium channel, $7\frac{5}{8}$ " long, was welded to the central portion of the bottom plate. An aluminium block, 2"x2"x1", was bolted to the cantilever on one side at the top. A bolt went vertically through the centre of this block and two pieces of steel plate (washers). A horizontal hole in the bolt allowed the wire to pass through the bolt between the two washers. The washers could be tightened by a nut at the top part of the bolt. At the base of the cantilever, four EA-06-250BG-120 type strain gages were mounted in a full-bridge configuration.

3.1.3.c Wrapping of Cylinders

The tensile force applied to the wire was 150 lbs. From this force, the initial lateral prestress applied to the column by the wire was calculated. The tensile force in the wire was obtained from

a Budd-strain-gage indicator which was connected with the strain gages mounted on the wrapping device. The strain gage readings were calibrated to give the force in the wire.

In order to maintain a constant force in the wire, the strain gage reading of the wrapping device was kept constant by adjusting the nut clamping the steel plates between which the wire passed during wrapping.

The wrappings of 18, 32 and 64 wires per inch produced an initial lateral prestresses of 900, 1600 and 3200 psi respectively with a wire tension of 150 lbs.

3.1.4 Installation of Strain Gages

At the mid-height of each column specimen, one longitudinal and one circumferential electrical resistance foil type strain gages, 5 mm. gage length, type KF-5-C8-11 of Kyowa Co. were installed on both the inside and outside faces of the column. An epoxy material was applied over the piano wire of each wrapped specimen to provide a smooth base for the strain gages.

3.2 Testing

All specimens were tested in a hydraulically operated 600000 lbs. capacity Badwin-Sowthwark Tate-Emery Testing Machine, and were loaded to failure. Three different eccentricities, $\frac{e}{D} = 0.10, 0.25$ and 0.50 were used for each combination of length and lateral prestress of the specimens.

All specimens were loaded with ball bearing hinged end plates. The designs of these end bearing loading plates are shown in Fig.3.7 p.39 to Fig.3.9 p.41 . These loading plates were fabricated by a "steel-ball-press" technique. High strength stainless steel balls were utilized. A cone-shaped depression was machined into each of the SPS 245 Chrome Nickel pads. Then, employing one pair of such chrome nickel pads, the steel ball was "pressed" hydraulically into the cone-shaped recess with a total axial force of 500 kips. The resulting plastic deformation formed a spherically shaped recess conforming perfectly to the contours of the steel ball. The seating pads formed were then welded at the required eccentricities onto thick steel plates which could be bolted to the ends of each specimen. Two other thick steel cover plates with the same type of spherically shaped recess were bolted to the testing machine, one at the loading head and one on the base. These two cover plates are shown in Fig.3.10 p.42 .

The lateral deflection of the columns was measured at mid-height. Since it was dangerous to read the deflection directly by means of dial gages, a 8" long cantilever was used to measure the lateral deflection by means of the strain gage at the base of the spring steel cantilever. This cantilever was calibrated by applying a known displacement to the cantilever and measuring the corresponding strain gage reading.

All deformation and lateral deflection measurements were made at essentially equal load increments of 10000 lbs. for specimens of eccentricity $\frac{e}{D} = 0.10$, 5000 lbs. for those of $\frac{e}{D} = 0.25$ and 1000 lbs. for those of $\frac{e}{D} = 0.50$.

The strain gage readings of all deformations and lateral deflections were read with a Digital Strain Indicator, Model 205 of CMS Laboratories, through with a Switch and Balance Unit, Model 305 of Binary Electronics.

Fig.3.11 p. 43 shows the specimen under test.

3.3 Nomenclatures for Specimens

In order to identify all the specimens, each specimen was designated for length, wires per inch of wrapping (i.e. initial lateral prestress), eccentricity and a number denoting the specimen in that particular type. Nomenclatures used in this thesis for specimens are as follows:

For the effective length:

- A : 2'-0" (24")
- B : 3'-0" (36")
- C : 4'-6" (54")
- D : 6'-0" (72")

For the lateral prestress:

- 0 : without lateral prestress
- 18 : 18 wires per inch of wrapping, ($\sigma_3 = 900$ psi)
- 32 : 32 wires per inch of wrapping, ($\sigma_3 = 1600$ psi)
- 64 : 64 wires per inch of wrapping, ($\sigma_3 = 3200$ psi)

For eccentricity:

$$a : e/D = 0.10$$

$$b : e/D = 0.25$$

$$c : e/D = 0.50$$

Thus, as an example:

A-32-b(2) is the second of the columns of 24" in effective length, with 32 wires per inch of wrapping (i.e. initial lateral prestress is 1600 psi) and is loaded with a eccentricity of $\frac{e}{D} = 0.25$.

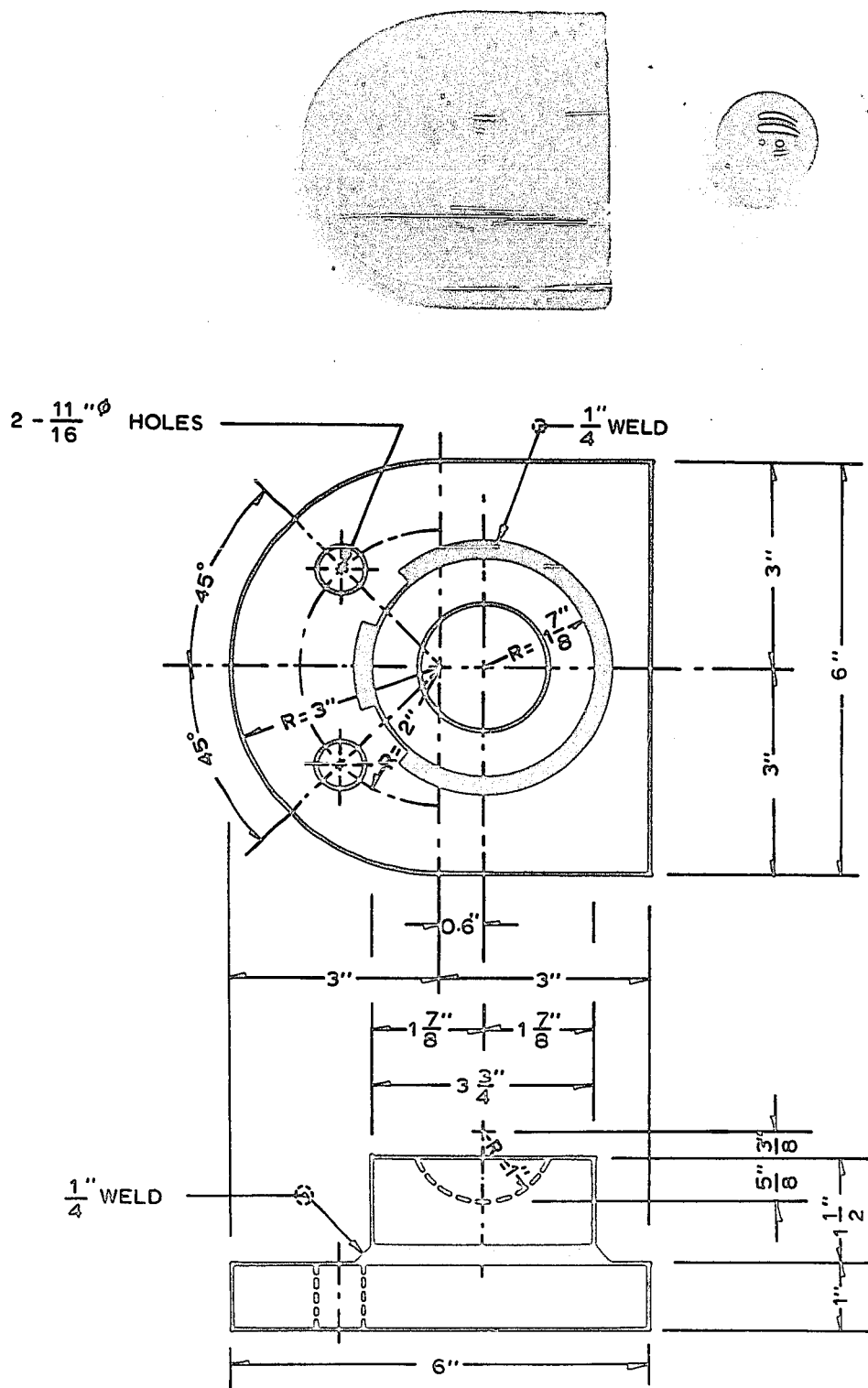


FIG. 3-7 DETAIL OF LOADING PLATE (FOR $\frac{e}{D} = 0.1$)

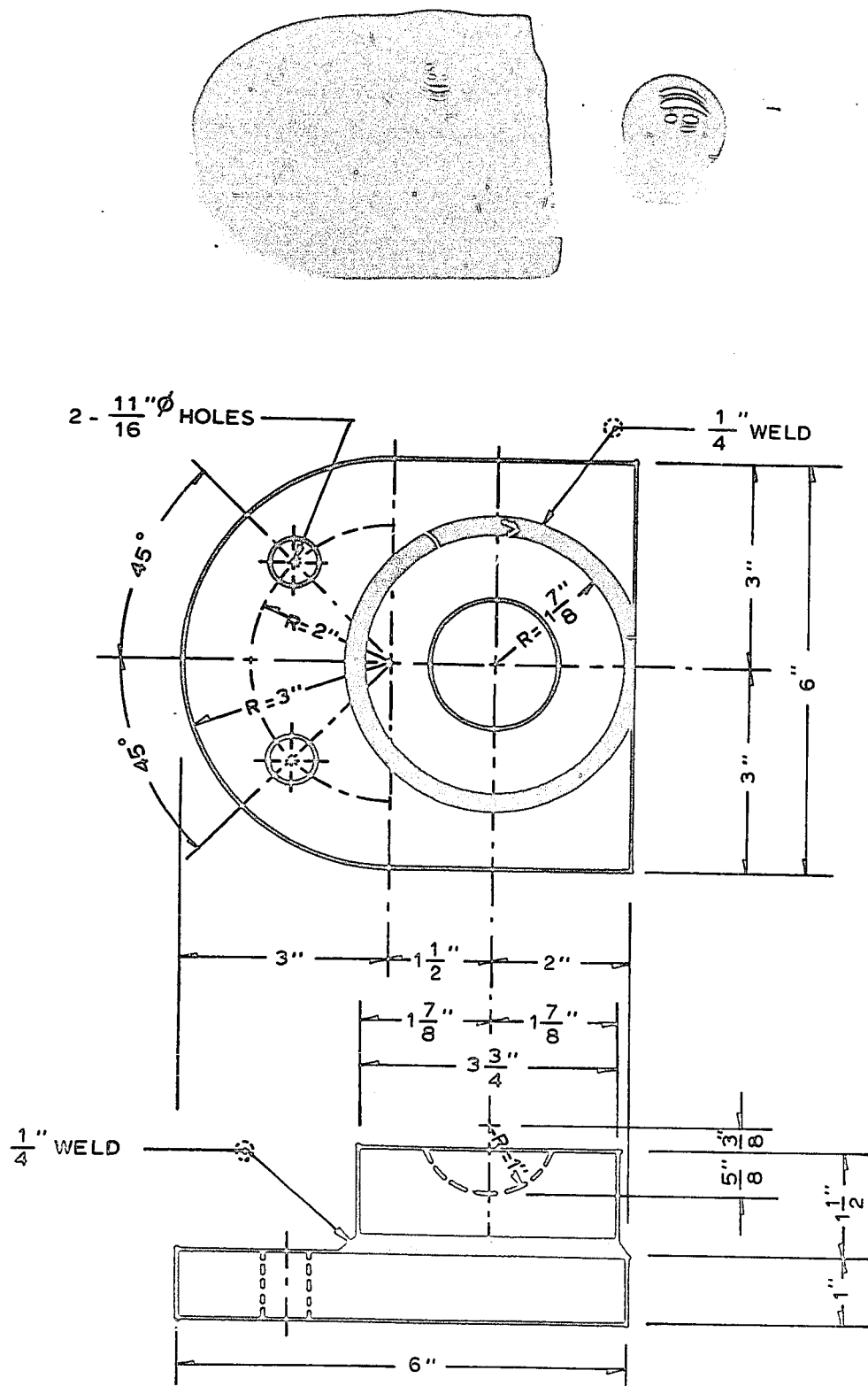


FIG. 3-8 DETAIL OF LOADING PLATE (FOR $\frac{e}{D} = 0.25$)

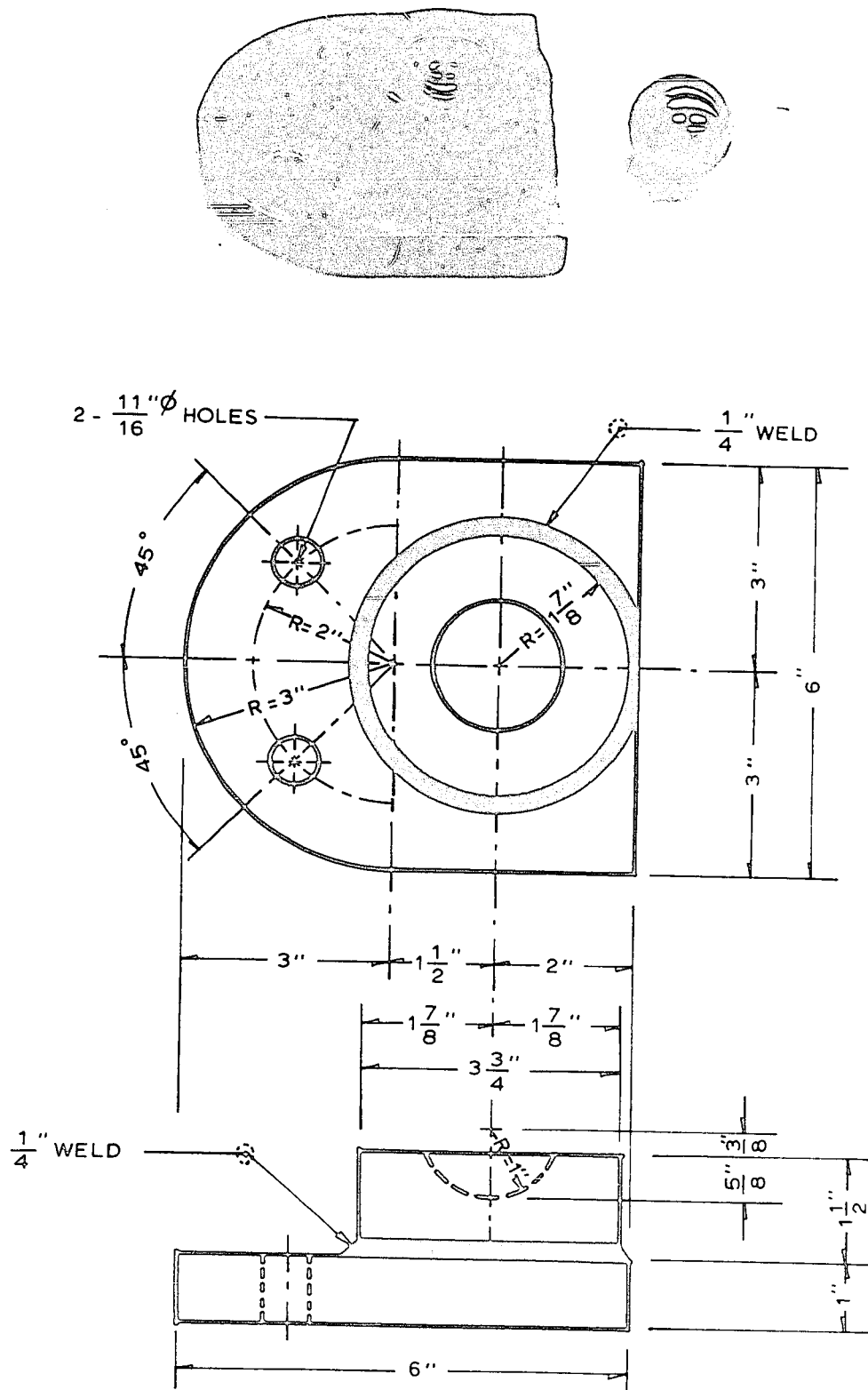


FIG. 3-8 DETAIL OF LOADING PLATE (FOR $\frac{e}{D} = 0.25$)

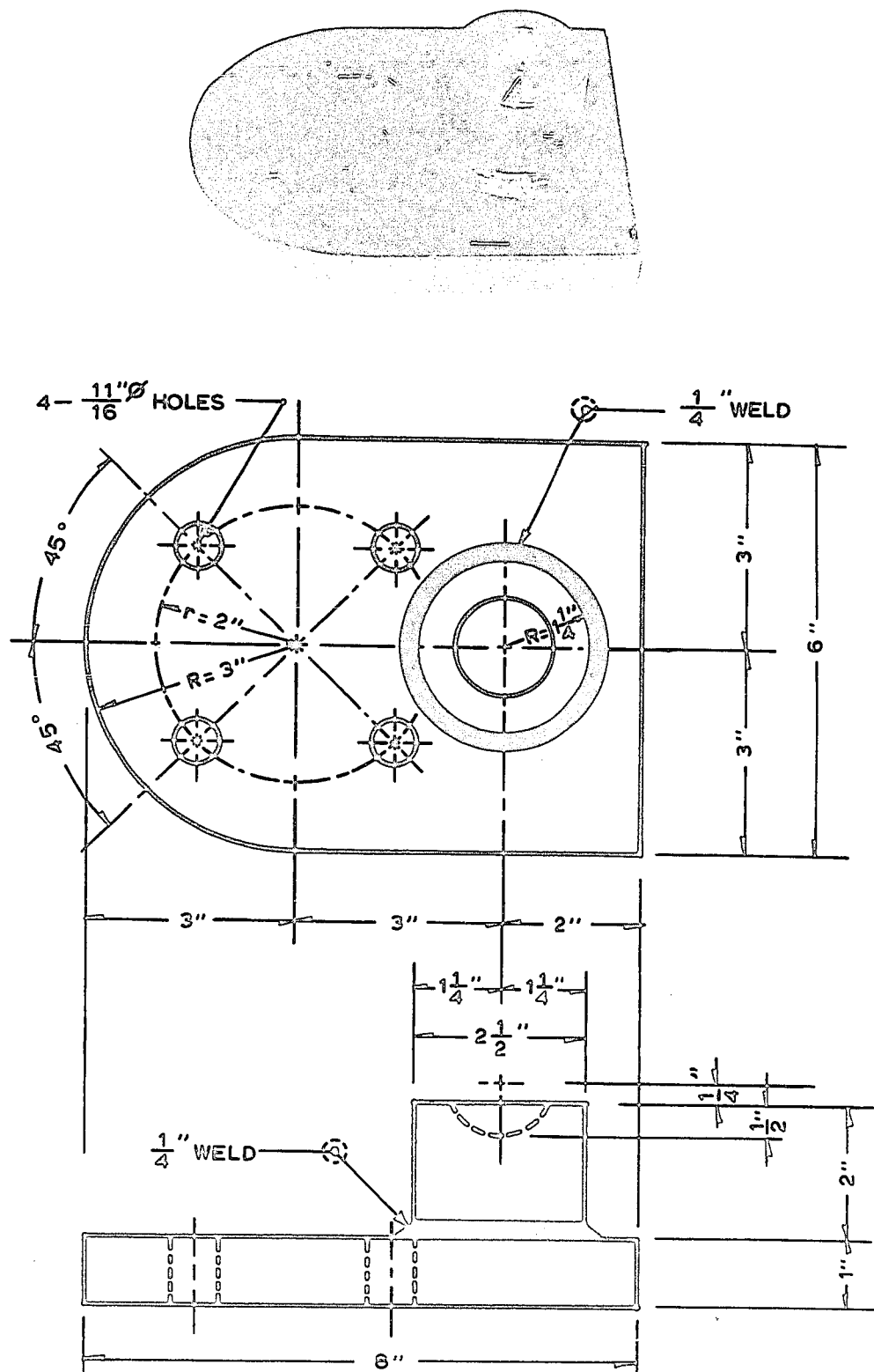


FIG. 3-9 DETAIL OF LOADING PLATE (FOR $\frac{e}{D} = 0.5$)

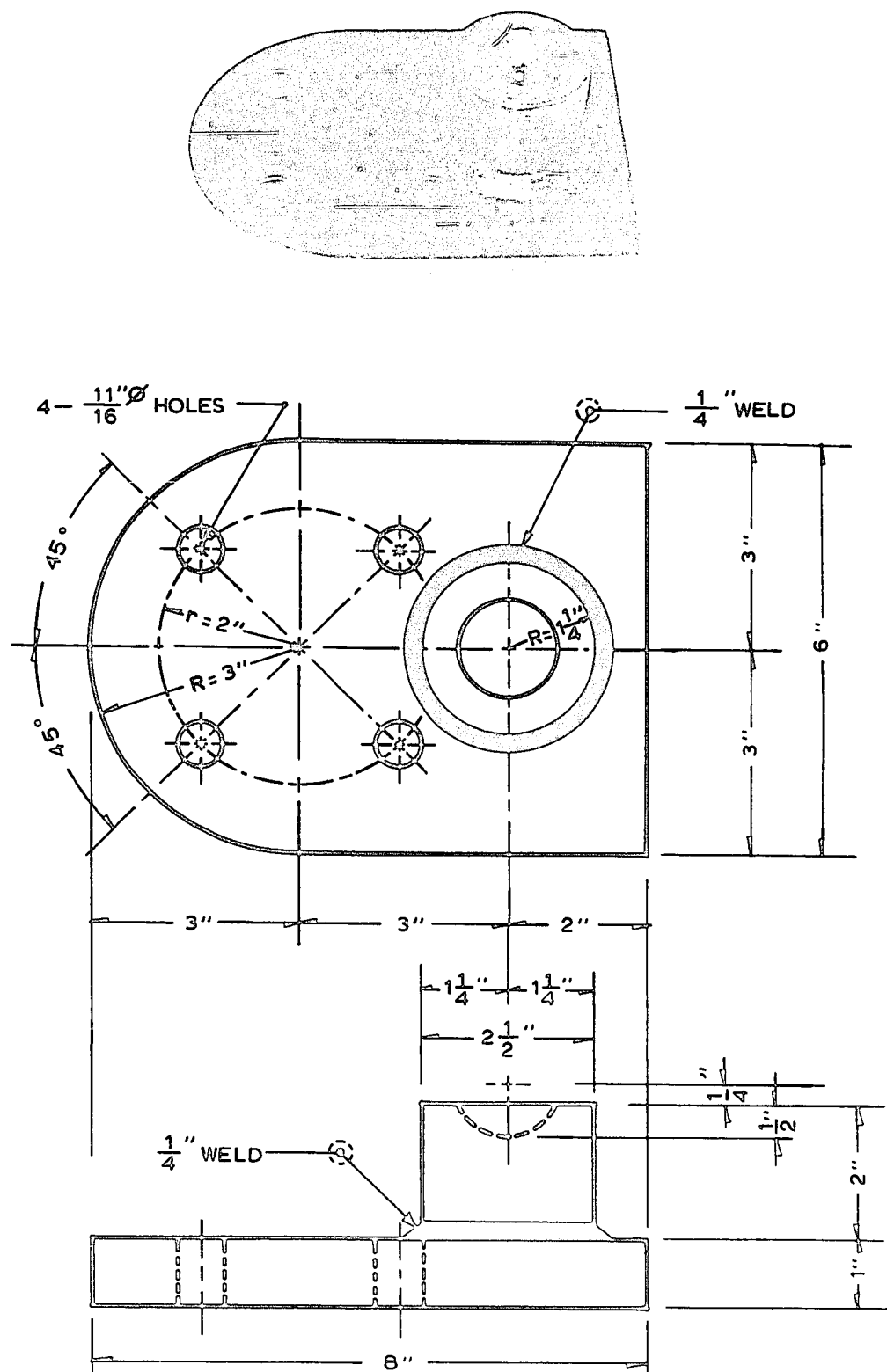
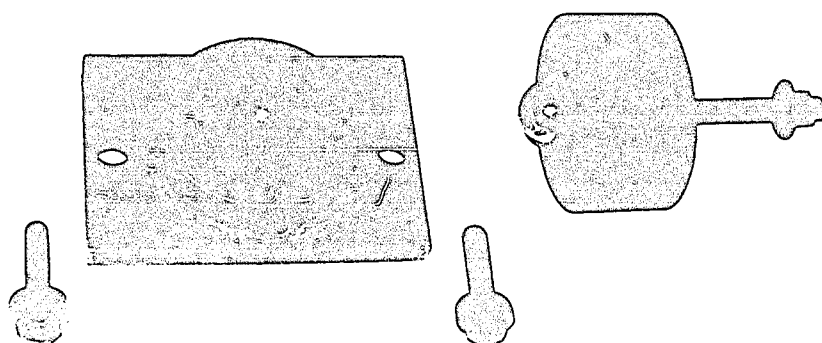


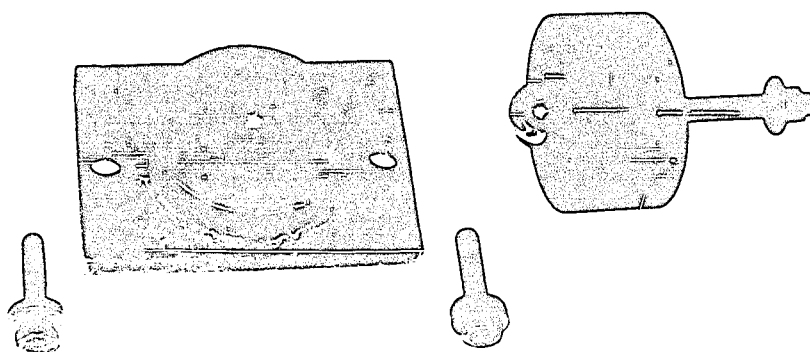
FIG. 3.9 DETAIL OF LOADING PLATE (FOR $\frac{e}{D} = 0.5$)



LEFT : BOTTOM COVER PLATE

RIGHT : TOP COVER PLATE

FIG. 3.10 LOADING COVER PLATES



LEFT : BOTTOM COVER PLATE

RIGHT : TOP COVER PLATE

FIG. 3·10 LOADING COVER PLATES

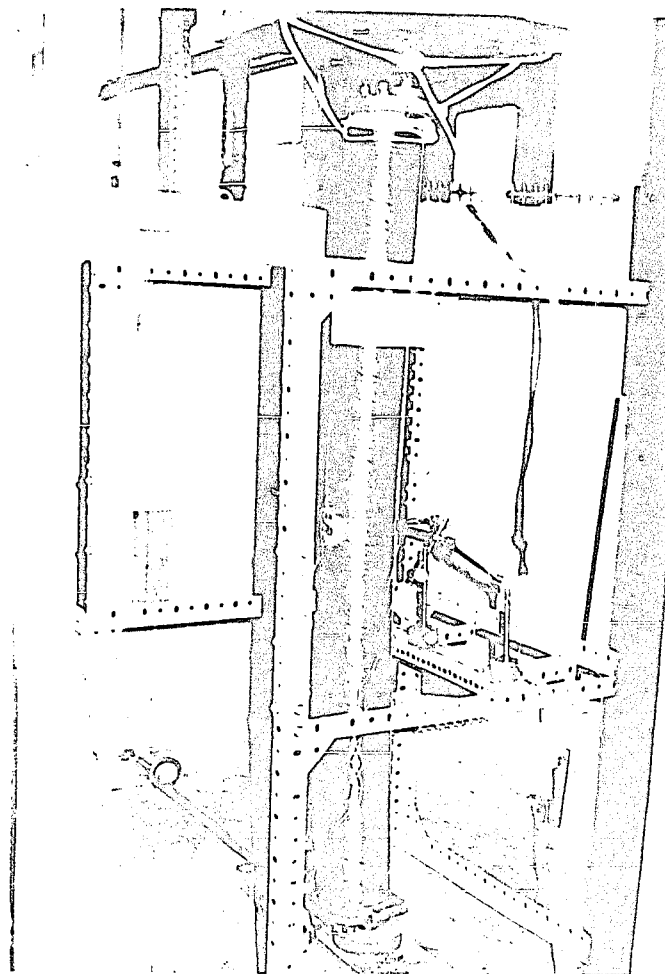


FIG. 3-11 SPECIMEN UNDER TEST

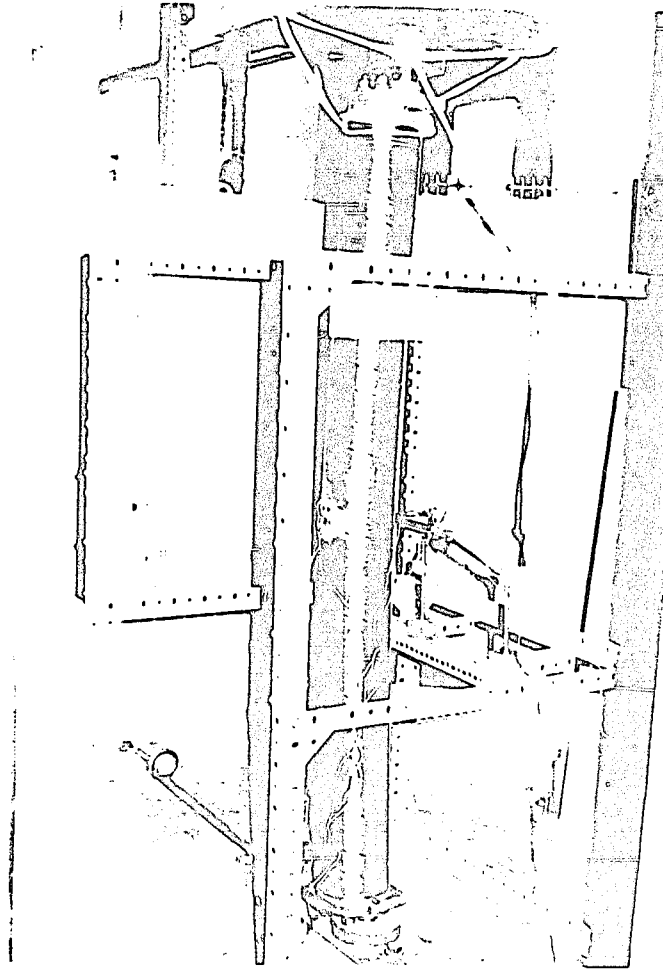


FIG. 3-11 SPECIMEN UNDER TEST

CHAPTER 4

THEORETICAL CONSIDERATIONS AND ANALYSIS

The theoretical analysis and derivation described in this Chapter is based on elementary strength of materials principles, and the conclusions obtained from previous investigations of the triaxial effects on cylindrical concrete columns under concentric load.

4.1 Analysis of Lateral Pressure from Wrapping Wires to Concrete Cylinders

When a concrete cylinder is wrapped with wire under tension, the wire is stretched by the tension T applied to the wire. After the cylinder is wrapped and both ends of the wire are fixed on the cylinder, the wire will have a tendency to return to its original length. This effect produces a pressure interacting between the wire and the cylinder surface.

Fig.4.1 p.45 shows the free body diagrams of the wire and the wrapped cylinder. If N_1 , the number of wires per inch of the wrapping, is known, the diameter D is expressed in inches and the tension T in the wire in pounds, the lateral stress σ_3 (psi) acting on the outside cylindrical surface of concrete cylinder can be calculated with the formula

$$\sigma_3 = \frac{2 N_1 T}{D} \dots\dots\dots(4.1)$$

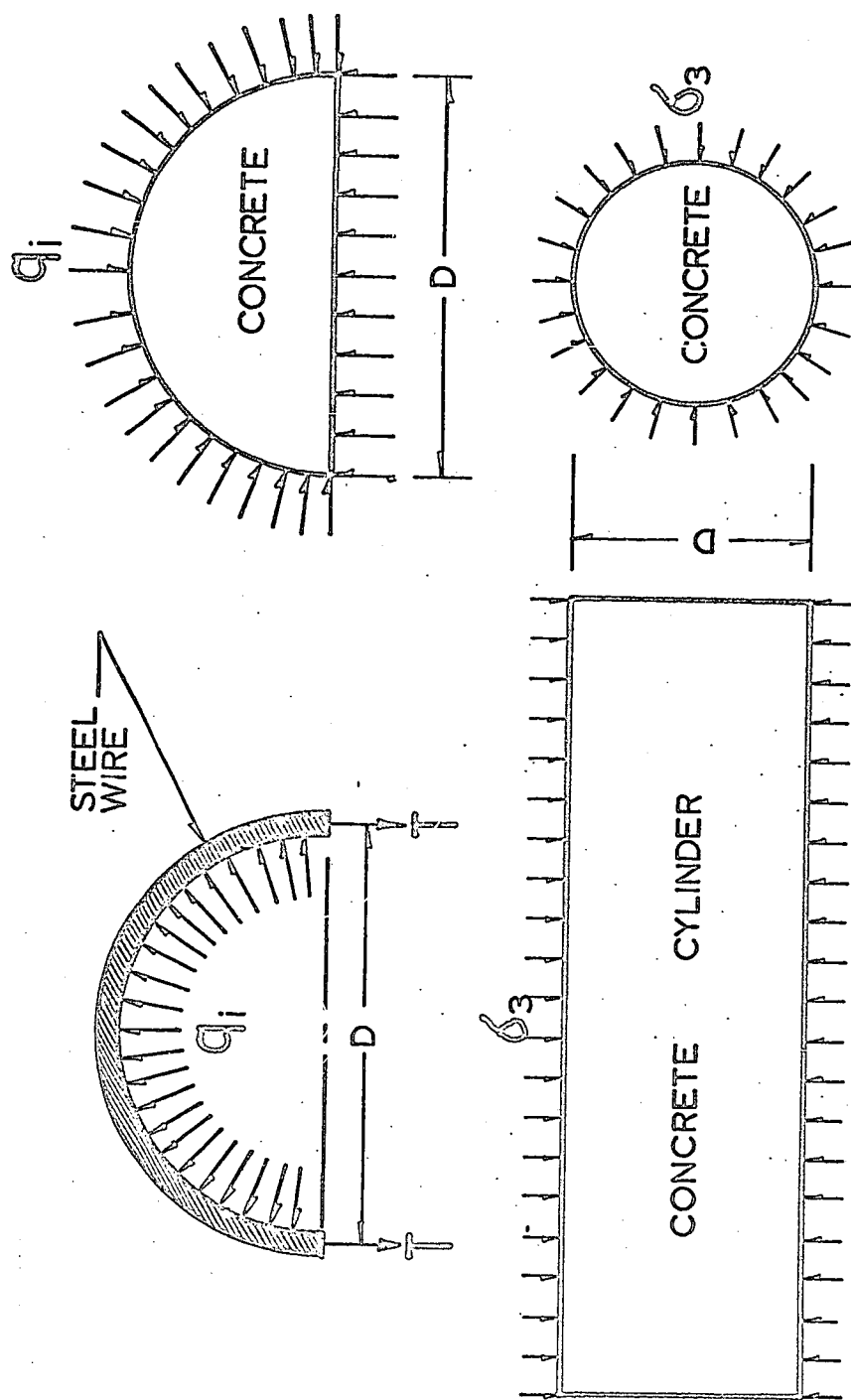


FIG. 4.1 LATERAL PRESSURE FROM WRAPPING WIRES
TO THE CYLINDER

In this thesis, Eqn.4.1 was used to calculate the lateral stress σ_3 .

4.2 Ultimate Strength Analysis of Plain Concrete Column

As the tensile strength of concrete is very weak, it is reasonable and usual to assume that the concrete takes no tensile stress. The tensile stress is usually taken by longitudinal reinforcement in reinforced concrete columns.

The ultimate strength analysis of plain concrete columns shall be based on the two following additional assumptions which are given in both the ACI (2) and NBC (22) building codes:

- a) At ultimate strength, concrete stress is not proportional to strain. The variation of compressive concrete stress distribution may be assumed to be a rectangle, trapezoid, parabola, or any other shape which results in predictions of ultimate strength in reasonable agreement with the results of compressive tests.
- b) The requirements of assumption a) may be considered satisfied by the equivalent rectangular concrete stress distribution which is defined as follows. At ultimate strength, a concrete stress intensity of $0.85f'_c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by the edge of the cross section and a straight line located parallel to the neutral axis at a distance $a = k_1 c$ from the fiber of maximum compressive strain. The distance c from the fiber of maximum strain to the neutral axis is measured in a direction

perpendicular to that axis. The fraction k_1 shall be taken as 0.85 for $f'_c \leq 4000$ psi, and decreases by 0.05 for every 1000 psi above 4000.

4.2.1 Cylindrical Plain Concrete Columns Under Concentric Load

Let Fig.4.2(b) and 4.2(c) p.48 represent the distribution of internal strains and stresses when the plain concrete column is about to fail under a concentric load P_{ult} . The whole section in this case will be under compressive stress (i.e. $a = D$). Using the assumption of equivalent rectangular stress block as shown in Fig.4.2(d) p.48, the magnitude of this stress is $0.85f'_c$ uniformly distributed over the whole section. The failure load P_{ult} will be given by

$$P_{ult} = A_c (0.85 f'_c) \dots\dots\dots(4.2)$$

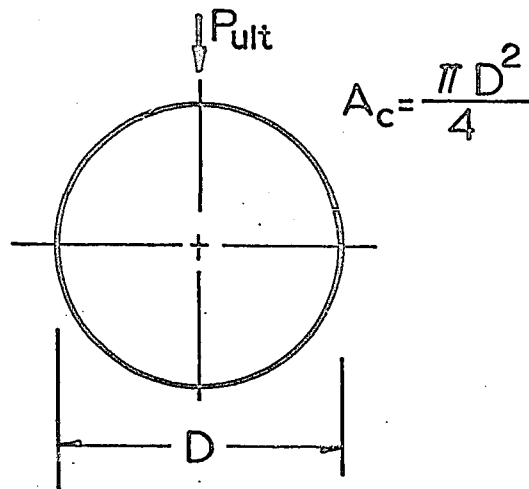
where A_c is the sectional area $= \frac{\pi D^2}{4}$ for cylinder.

4.2.2 Cylindrical Concrete Columns Under Eccentric Load

When a cylindrical plain concrete column is under load P_{ult} at an eccentricity e from the plastic centroid of the section as shown in Fig.4.3 p.49, the resultant C of the equivalent rectangular stress block must be collinear with and equal to P_{ult} for equilibrium. Since the resultant C passes through the centroid of the equivalent compression zone and the point of loading, the area A'_c of the equivalent compression zone will be

$$A'_c = 2 A_e \dots\dots\dots(4.3)$$

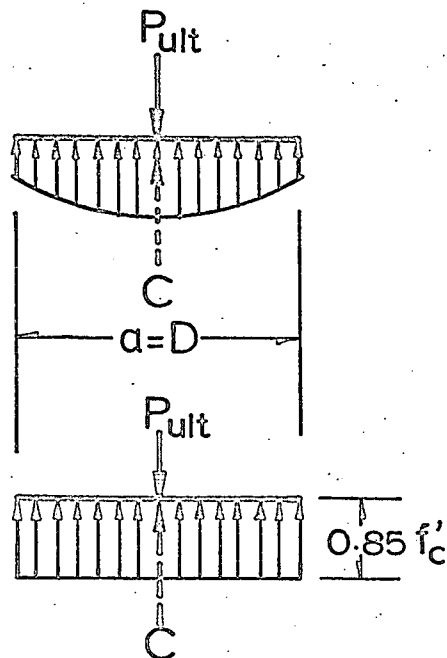
where A_e is the area bounded by the edge fiber of the maximum strain



(a) Cross section and loading



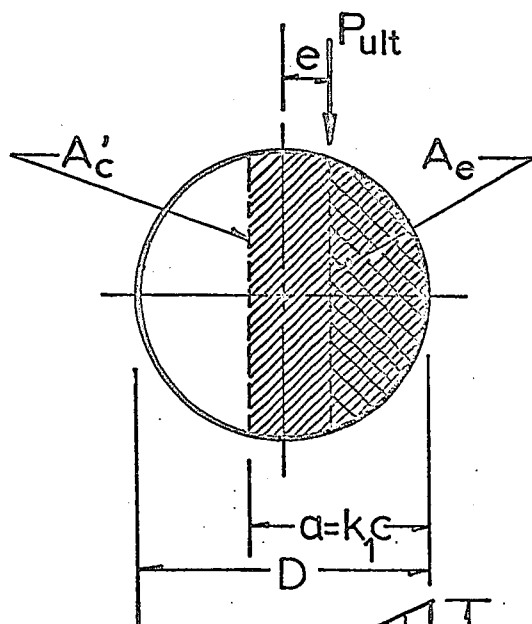
(b) Strain distribution



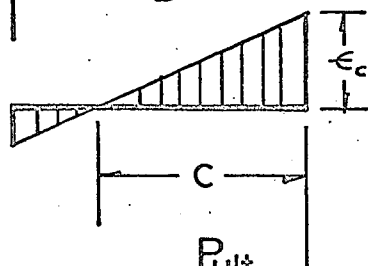
(c) Stress distribution

(d) Equivalent rectangular stress block

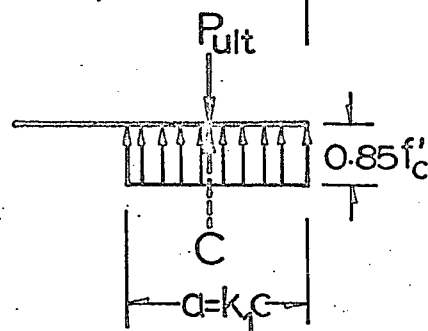
FIG. 4-2 STRESS AND STRAIN DISTRIBUTION OF CYLINDRICAL CONCRETE COLUMN UNDER CONCENTRIC LOAD



(a) Cross section and loading



(b) Strain distribution



(c) Equivalent rectangular stress block

FIG. 4.3 STRESS AND STRAIN DISTRIBUTION OF CYLINDRICAL CONCRETE COLUMN UNDER ECCENTRIC LOAD

and a straight line located at the point of eccentric load and parallel to the neutral axis as shown in Fig.4.3(a) p.49 .

The failure load P_{ult} in this case can be given by

$$P_{ult} = 0.85 f'_c A'_c \dots\dots\dots(4.4)$$

The expression of A'_e in terms of eccentricity e can be derived by integration and found to be

$$A'_e = \frac{\pi D^2}{8} - e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{4} \sin^{-1} \left(\frac{2e}{D} \right) \dots\dots\dots(4.5)$$

Thus, A'_c becomes

$$A'_c = \frac{\pi D^2}{4} - 2e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1} \left(\frac{2e}{D} \right) \dots\dots\dots(4.6)$$

Eqn.4.6 shows that

$$A'_c = \frac{\pi D^2}{4} = A_c \text{ when } \frac{e}{D} = 0, \text{ and}$$

$$A'_c = 0 \text{ when } \frac{e}{D} = 0.50 .$$

For convenience, the values of the effective sectional area A'_c for different $\frac{e}{D}$ ratios, less than 0.50, have been plotted in Fig.6.1 p.84 in Chapter 6.

For long columns with the load applied eccentrically, reduction factors must be applied to Eqn.4.4

$$P_{ult} = A'_c (0.85 f'_c) (R_1 - R_2 \frac{L}{r}) (1 - R_3 \frac{e}{D}) \dots\dots\dots(4.7)$$

where R_1 , R_2 and R_3 are empirical coefficients which will be discussed in Chapter 5, and A'_c is the effective area of the cross section.

Eqn.4.7 can be considered as general form of formula for cylindrical plain concrete columns with an eccentricity ranging from

$$\frac{e}{D} = 0 \text{ to } 0.50.$$

4.2.3 Laterally Prestressed Plain Concrete Columns

Previous investigations of the effects of lateral prestress to the concrete columns found that the lateral prestress increases the load capacity a certain amount. It is logical that Eqn.4.7 can also be used for laterally prestressed plain concrete columns by replacing the unconfined concrete compressive strength f'_c by the confined concrete compressive strength. That is

$$P_{ult} = A'_c \{0.85f'_c + \beta K \bar{\sigma}_3\} \{R_1 - R_2 \frac{L}{r}\} \{1 - R_3 \frac{e}{D}\} \dots\dots(4.8)$$

where βK is an empirical constant which decreases with increases of either lateral prestress or slenderness ratio. As it will be discussed in Chapter 5, βK is also a function of the $\frac{e}{D}$ ratio. Thus, βK can be expressed in the form of

$$\beta K = K_1 \left[1 - K_2 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^\alpha \right] \left[K_3 - K_4 \frac{L}{r} \right] \left[1 - K_5 \frac{e}{D} \right] \dots\dots\dots(4.9)$$

where K_1 , K_2 , K_3 , K_4 , K_5 and α are empirical coefficients which will be determined in Chapter 5.

Thus, the general form of the formula to predict the failure load P_{ult} for laterally prestressed plain concrete columns may be written as

$$P_{ult} = A'_c \left\{ 0.85f'_c + K_1 \left[1 - K_2 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^\alpha \right] \left[K_3 - K_4 \frac{L}{r} \right] \left[1 - K_5 \frac{e}{D} \right] \bar{\sigma}_3 \right\} \left\{ R_1 - R_2 \frac{L}{r} \right\} \left\{ 1 - R_3 \frac{e}{D} \right\} \dots\dots\dots(4.10)$$

where A'_c is the same as before, with $\frac{e}{D}$ ranging from 0 to 0.50.

CHAPTER 5

DISCUSSION OF TEST RESULTS AND THE PROPOSED FORMULA

The individual test results are presented in the form of load/deformation curves. These graphs as well as the load/lateral deflection curves are presented in Appendix B, from Fig.B.1 p.128 to Fig.B.60 p.187. The experimental failure loads of all specimens are presented in Fig.C.1 p.189 in Appendix C.

The average failure loads for each type of specimens are presented in Fig.5.2 p.54 and curves showing the effects of lateral prestress, of slenderness ratio, and of eccentricity are presented in Fig.5.8 p.62, Fig.5.9 p.63 and Fig.5.10 p.64 respectively.

In order to obtain some of the empirical coefficients for the proposed formula, the test results of Godse's investigation (16) of similar specimens under concentric load were also used and are presented in Fig.5.1 p.53.

5.1 Failure Phenomena of Specimens

The various typical failure phenomena are shown in Fig.5.3 p.55 to Fig.5.6 p.58. A summary of failure phenomena observed is given briefly in Fig.5.7 p.59.

It was found that all unwrapped concrete columns failed by crushing of concrete at mid-height or at the weaker end of the cylinder.

Most of the shorter series of wrapped specimens, failed at mid-height portion by crushing of concrete and simultaneous rupture of

wires. The rupture of wires indicated the tensile stress in the wires reached its ultimate value. The final lateral stress $\bar{\sigma}_3$ can then be calculated from the ultimate strength of the wire.

However, most of the longer specimens failed by crushing of the concrete without rupture of the wire. The value of the final lateral stress at failure $\bar{\sigma}_3$ cannot be predicted. In this case, the value $\bar{\sigma}_3$ at failure was obtained from the circumferential strain readings of test results and the characteristic curve of the wire.

All specimens which were loaded with $\frac{e}{D} = 0.50$ failed by tension failure of the concrete at the weaker end.

Those of series D with $\frac{e}{D} = 0.10$ and 0.25 failed by general buckling.

It was concluded that lateral prestressing does not increase the tensile strength of concrete and results for $\frac{e}{D} = 0.50$ were not considered in the development of the empirical equation.

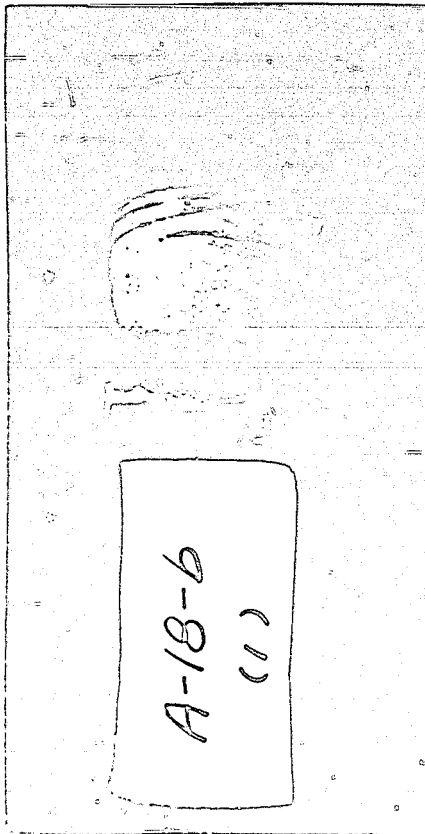
Fig.5.1 Test Results of Laterally Prestressed Concrete Columns Under Concentric Load (obtained from Godse's investigation (16))

Specimen No.	$\frac{L}{r}$	f'_c (psi)	$\bar{\sigma}_3$ (psi)	P_{exp} (kips)	P_{sl} (kips)	P_{exp} (kips)
A- 0-0	16.60	4600	0	112		112.0
A-18-0			1452	290		290.0
A-32-0			2581	374		374.0
A-64-0			5162	573		573.0
B- 0-0	25.40	5100	0	111		111.0
B-18-0			1452	284		284.0
B-32-0			2581	364		364.0
B-64-0			5162	545		545.0
C- 0-0	37.05	5000	0	130	28.3	101.7
C-18-0			1452	290		261.7
C-32-0			2581	352		323.7
C-64-0			5162	407		378.7
D- 0-0	49.40	4950	0	126	28.3	97.7
D-18-0			1452	268		239.7
D-32-0			2581	310		281.7
D-64-0			5162	348		319.7

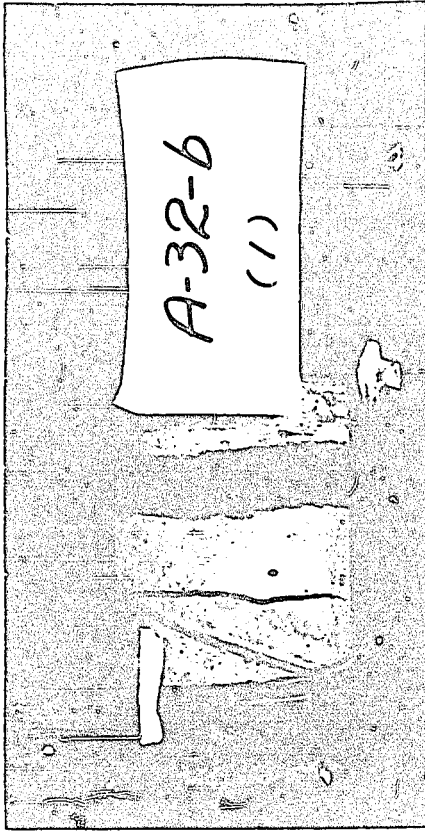
Fig.5.2 Test Results of Laterally Prestressed Concrete Columns
Under Eccentric Load

Specimen No.	$\frac{L}{r}$	f_c' (psi)	σ_3 (psi)	$\bar{\sigma}_3$ (psi)	$\frac{e}{D}$	P_{exp} (kips)
A- 0-a	16	7200	0	0	0.10	120.0
A- 0-b					0.25	63.0
A- 0-c					0.50	10.5
A-18-a			900	1547	0.10	290.0
A-18-b				1547	0.25	139.0
A-18-c				910	0.50	11.8
A-32-a			1600	2750	0.10	358.0
A-32-b				2130	0.25	156.0
A-32-c				1600	0.50	10.6
A-64-a			3200	4750	0.10	460.0
A-64-b				3860	0.25	193.0
A-64-c				3200	0.50	10.6
B- 0-a	24	7460	0	0	0.10	117.0
B- 0-b					0.25	59.0
B- 0-c					0.50	9.0
B-18-a			900	1547	0.10	262.0
B-18-b				1075	0.25	110.0
B-18-c				900	0.50	10.7
B-32-a			1600	2750	0.10	340.0
B-32-b				1950	0.25	140.0
B-32-c				1600	0.50	12.7
B-64-a			3200	4300	0.10	380.0
B-64-b				3700	0.25	170.0
B-64-c				3200	0.50	10.3
C- 0-a	36	6750	0	0	0.10	101.0
C- 0-b					0.25	50.0
C- 0-c					0.50	7.0
C-18-a			900	1100	0.10	192.0
C-18-b				955	0.25	85.0
C-18-c				900	0.50	8.9
C-32-a			1600	2000	0.10	248.0
C-32-b				1730	0.25	105.0
C-32-c				1600	0.50	10.9
C-64-a			3200	3670	0.10	301.0
C-64-b				3410	0.25	119.0
C-64-c				3200	0.50	10.0
D- 0-a	48	5600	0	0	0.10	66.0 *
D- 0-b					0.25	41.0 *
D- 0-c					0.50	9.8 *
D-18-a			900	1065	0.10	140.0 *
D-18-b				980	0.25	79.0 *
D-18-c				900	0.50	10.0 *
D-32-a			1600	1820	0.10	150.0 *
D-32-b				1810	0.25	85.0 *
D-32-c				1600	0.50	10.4 *
D-64-a			3200	3540	0.10	155.0 *
D-64-b				3510	0.25	87.0 *
D-64-c				3200	0.50	11.0 *

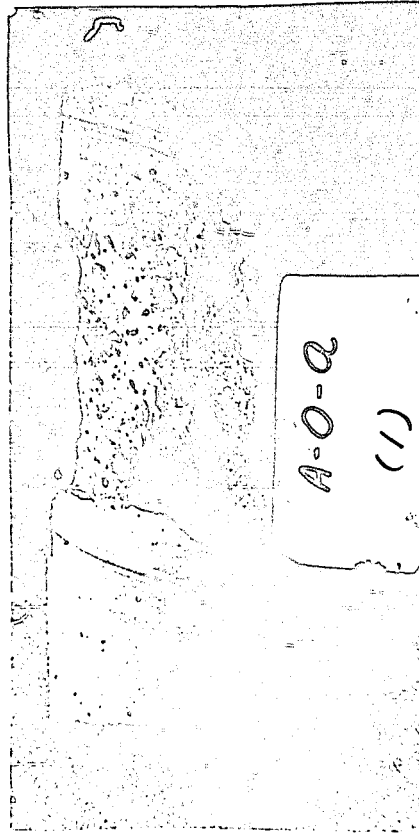
* Since the contribution of longitudinal reinforcement remains unknown, Series D is not included in the analysis.



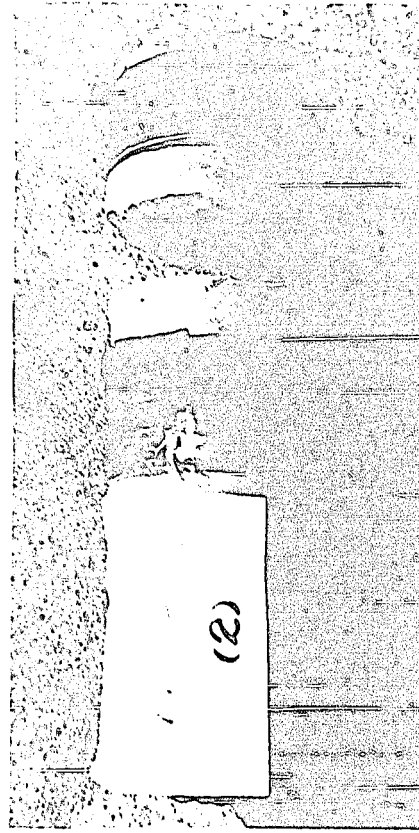
SIMULTANEOUS CRUSHING OF CONCRETE AND RUPTURE OF WIRES.



CRUSHING OF CONCRETE ONLY.



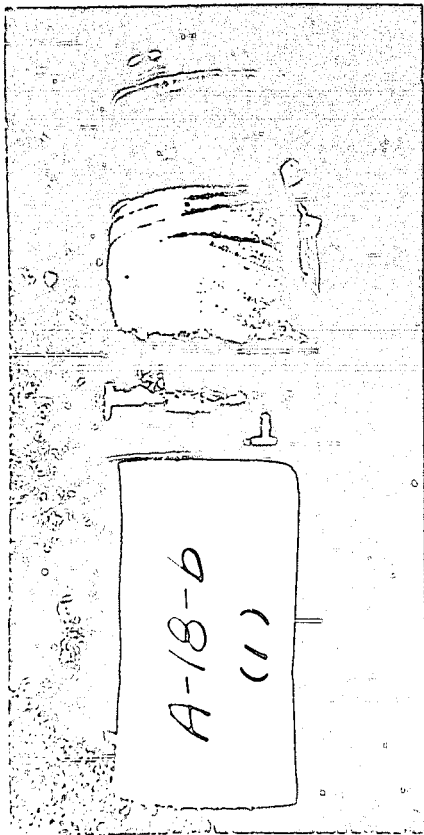
CRUSHING OF CONCRETE AT MID-HEIGHT.



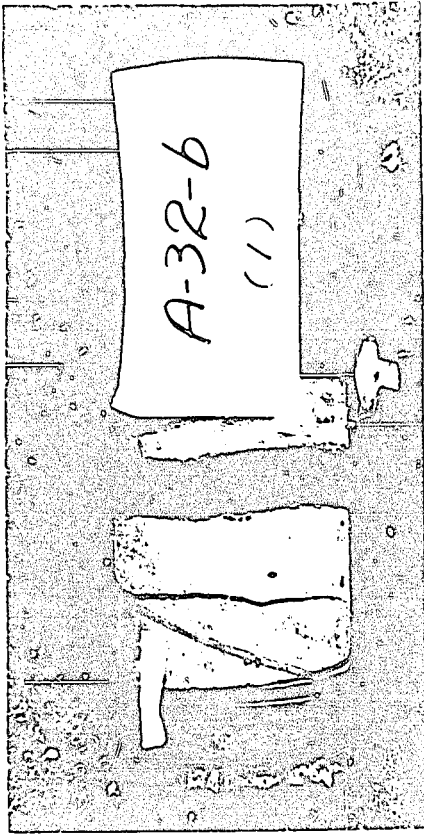
TENSION FAILURE AT WEAKER END.

FIG. 5-3 TYPICAL FAILURE PHENOMENA OF SERIES A

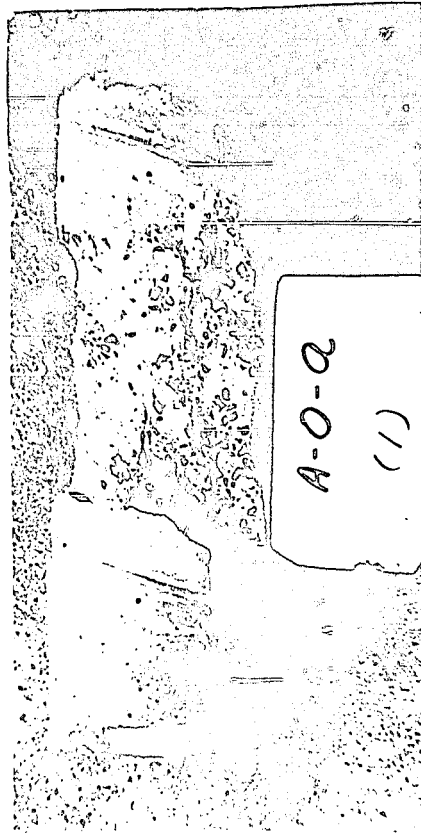
A



SIMULTANEOUS CRUSHING OF CONCRETE
AND RUPTURE OF WIRES.



CRUSHING OF CONCRETE ONLY.



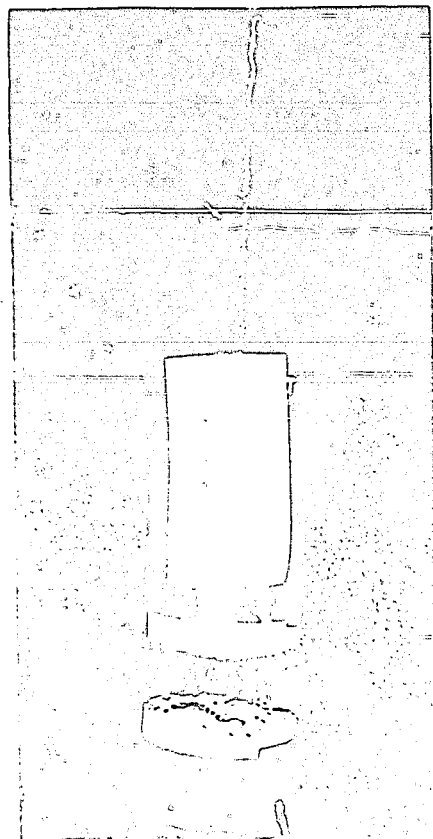
CRUSHING OF CONCRETE AT MID-HEIGHT.



TENSION FAILURE AT WEAKER END.

FIG. 5.3 TYPICAL FAILURE PHENOMENA OF SERIES A

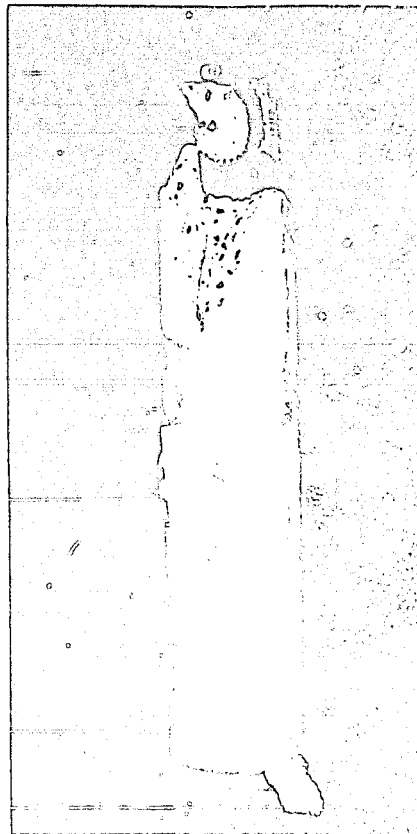
A



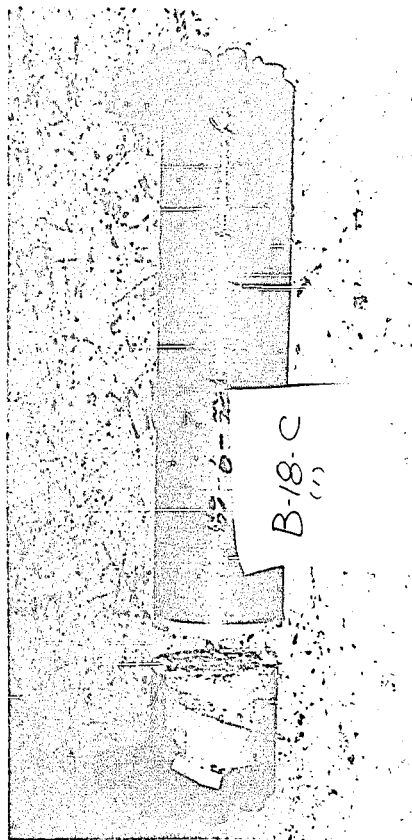
SIMULTANEOUS CRUSHING OF CONCRETE
AND RUPTURE OF WIRES.



CRUSHING OF CONCRETE ONLY.



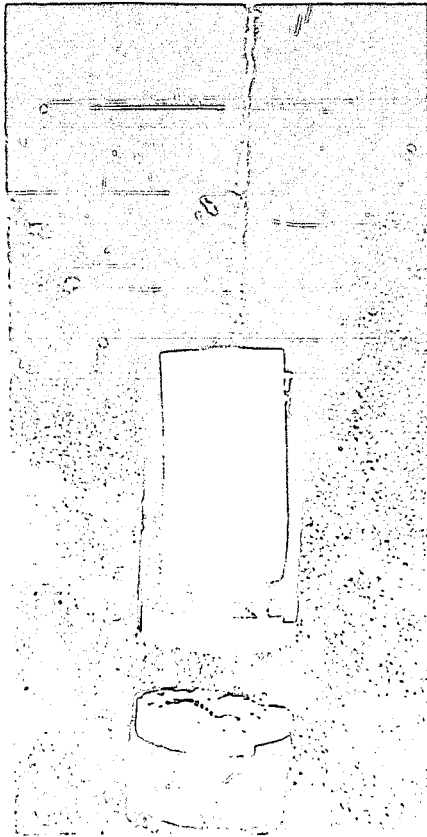
CRUSHING OF CONCRETE AT WEAKER END.



TENSION FAILURE AT WEAKER END.

FIG. 5-4 TYPICAL FAILURE PHENOMENA OF SERIES B

A



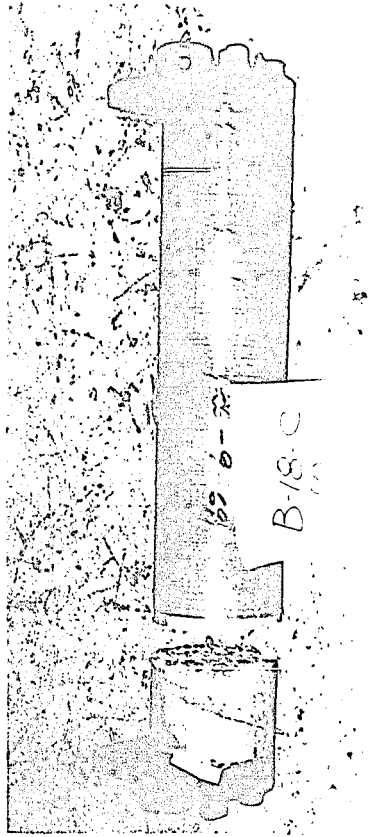
SIMULTANEOUS CRUSHING OF CONCRETE
AND RUPTURE OF WIRES.



CRUSHING OF CONCRETE ONLY.



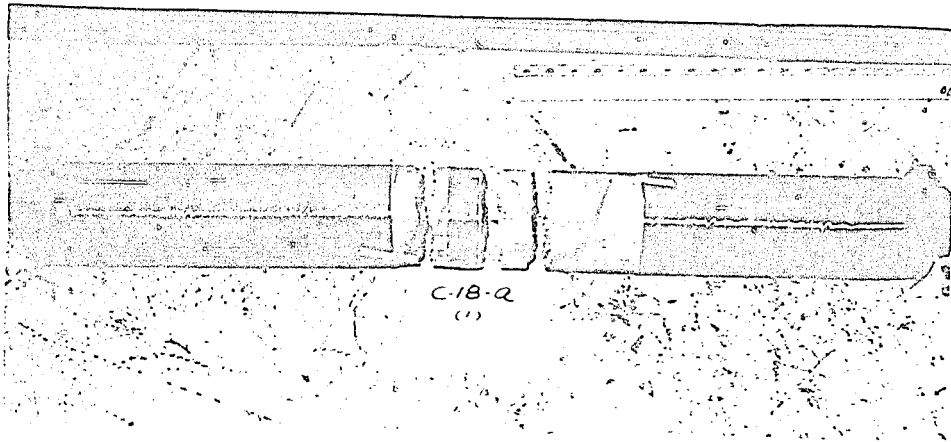
CRUSHING OF CONCRETE AT WEAKER END.



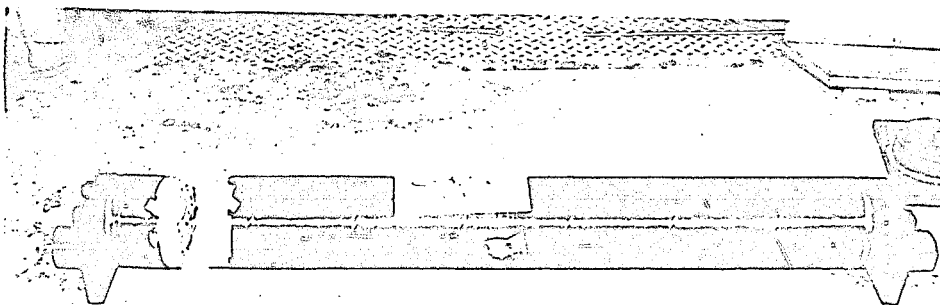
TENSION FAILURE AT WEAKER END.

FIG. 5-4 TYPICAL FAILURE PHENOMENA OF SERIES B

A

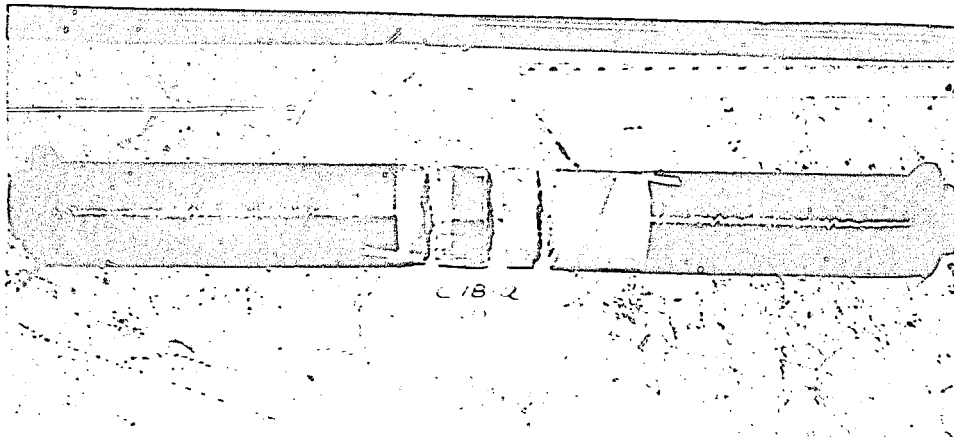


CRUSHING OF CONCRETE ONLY.

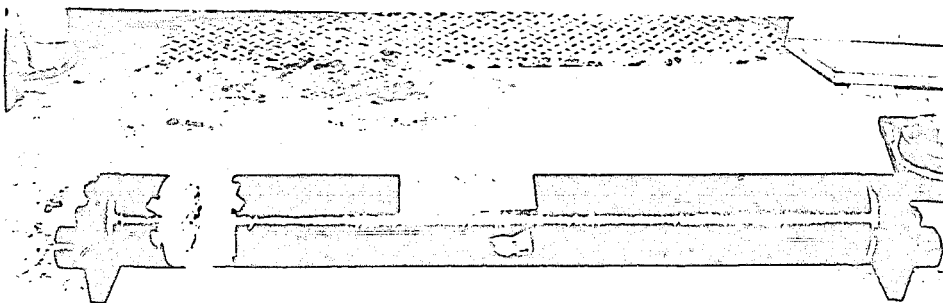


TENSION FAILURE AT WEAKER END.

FIG. 5-5 TYPICAL FAILURE PHENOMENA
OF SERIES C

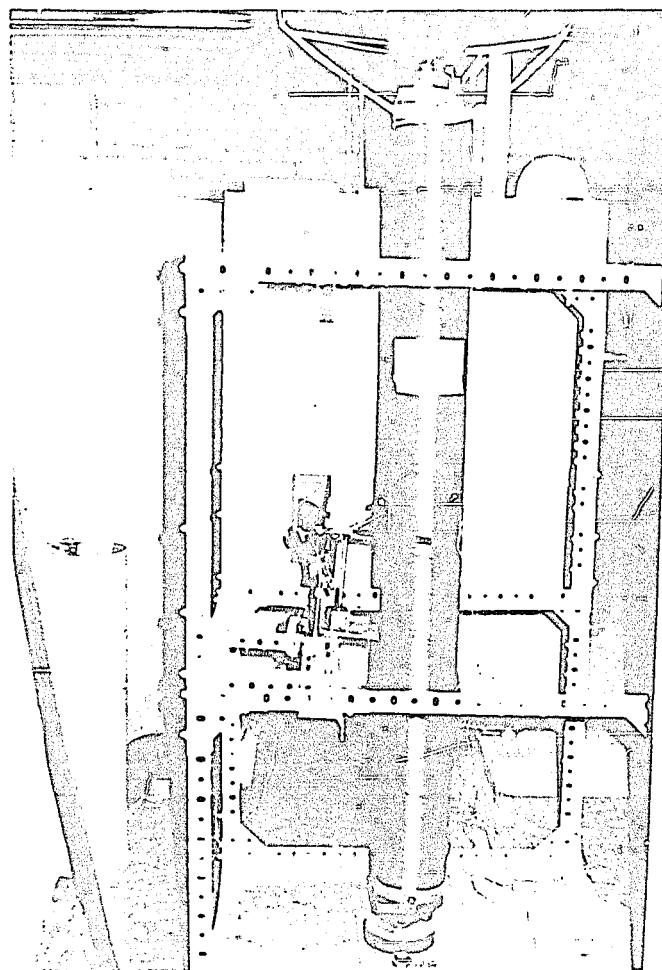


CRUSHING OF CONCRETE ONLY.

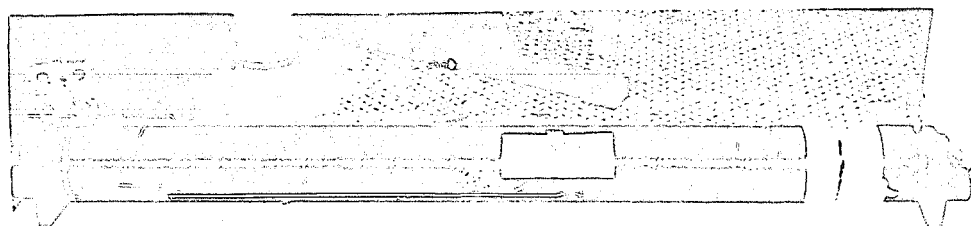


TENSION FAILURE AT WEAKER END.

FIG. 5-5 TYPICAL FAILURE PHENOMENA
OF SERIES C

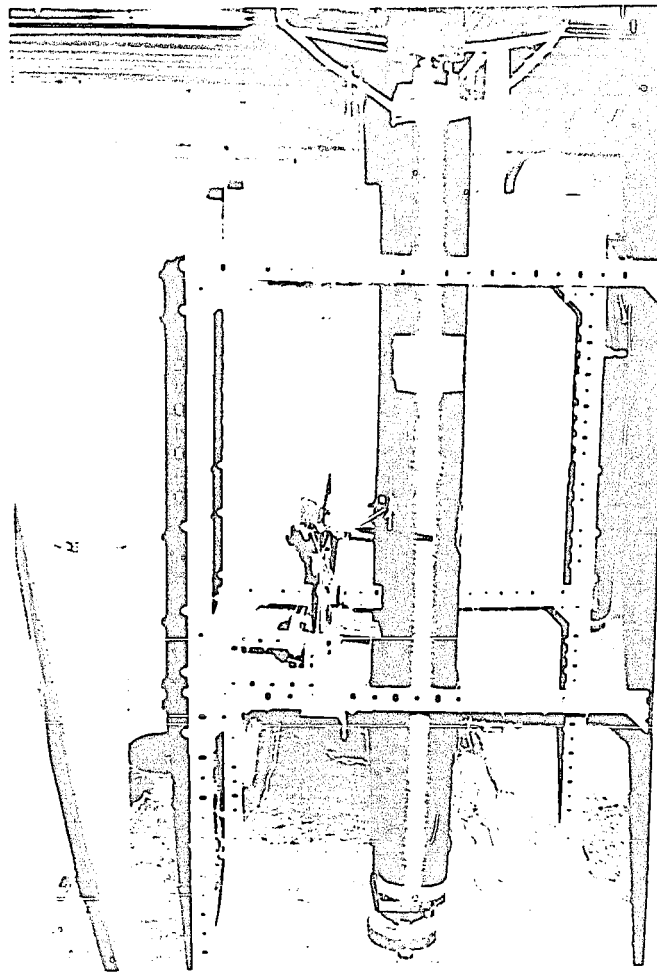


FAILED BY GENERAL BUCKLING.



TENSION FAILURE AT WEAKER END.

FIG. 5-6 TYPICAL FAILURE PHENOMENA
OF SERIES D



FAILED BY GENERAL BUCKLING.



TENSION FAILURE AT WEAKER END.

FIG. 5-6 TYPICAL FAILURE PHENOMENA
OF SERIES D

Fig.5.7 Summary of Failure Phenomena of Test Specimens

$\frac{L}{r}$	$\frac{e}{D}$ \diagup wires in.	0	18	32	64
A (16)	a (0.10)	Crushing of concrete at mid-height	Crushing of concrete and simultaneous rupture of wire	Crushing of concrete	*
	b (0.25)				
	c (0.50)	Tension failure of concrete at weaker end (the top end of the specimen while casting)			
B (24)	a	Crushing of concrete near the weaker end	Crushing of concrete and simultaneous rupture of wire	Crushing of concrete only	
	b		Crushing of concrete only		
	c	Tension failure of concrete at weaker end (the top end of the specimen while casting)			
C (36)	a	Crushing of concrete only			
	b				
	c	Tension failure of concrete at weaker end (the top end of the specimen while casting)			
D (48)	a	Failed by general buckling			
	b				
	c	Tension failure of concrete at weaker end (the top end of the specimen while casting)			

* Since the load was too high, testing not continued up to complete failure.

5.2 Effect of Lateral Prestress

Fig.5.8 p.62 shows the variation of failure load with lateral prestress. It was found that the lateral prestress increased the failure load in each type of specimens except for $\frac{e}{D} = 0.50$. The failure load increased as the lateral prestress increased. The rate of increase of failure load decreased as the lateral prestress increased. This showed the assumption for βK is reasonable in the form

$$\beta K \propto F\left\{ 1 - K_2 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^\alpha \right\} \dots\dots\dots(5.1)$$

where K_2 and α are empirical coefficients.

5.3 Effect of Slenderness Ratio

Fig.5.9 p.63 shows the variation of failure load with slenderness ratio. It was found that the failure load P_{ult} decreased as the slenderness ratio increased. It can be expressed in the form of the straight-line relationship

$$P_{ult} \propto F\left\{ R_1 - R_2 \frac{L}{r} \right\} \dots\dots\dots(5.2)$$

considering the $\frac{L}{r}$ also affects the factor βK ,

$$\beta K \propto F\left\{ K_3 - K_4 \frac{L}{r} \right\} \dots\dots\dots(5.3)$$

where R_1 , R_2 , K_3 and K_4 are empirical coefficients.

5.4 Effect of Eccentricity

Fig.5.10 p.64 shows the variation of failure load with eccentricity $\frac{e}{D}$. It was found that the failure load P_{ult} decreased as $\frac{e}{D}$ increased. When $\frac{e}{D} = 0.50$, P_{ult} was approaching to zero since these specimens with $\frac{e}{D} = 0.50$ failed by tension failure of the concrete. The assumption was that the tensile strength of concrete is zero and is unchanged by lateral prestressing. This shows the application of A'_c in Eqn.4.6 in Chapter 4 is correct, where A'_c is the area of the equivalent compression zone of concrete

$$A'_c = \frac{\pi D^2}{4} - 2e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1}\left(\frac{2e}{D}\right) \dots\dots\dots(5.4)$$

Furthermore, an empirical factor R_e may be applied to the area of the equivalent compression zone A'_c . Let it be in the form of

$$R_e A'_c = A'_c (1 - R_3 \frac{e}{D}) \dots\dots\dots(5.5)$$

where R_3 is an empirical coefficient.

Considering the $\frac{e}{D}$ ratio also affects the factor βK , then it is necessary to assume that

$$\beta K \propto F\{1 - K_5 \frac{e}{D}\} \dots\dots\dots(5.6)$$

where K_5 is an empirical coefficient.

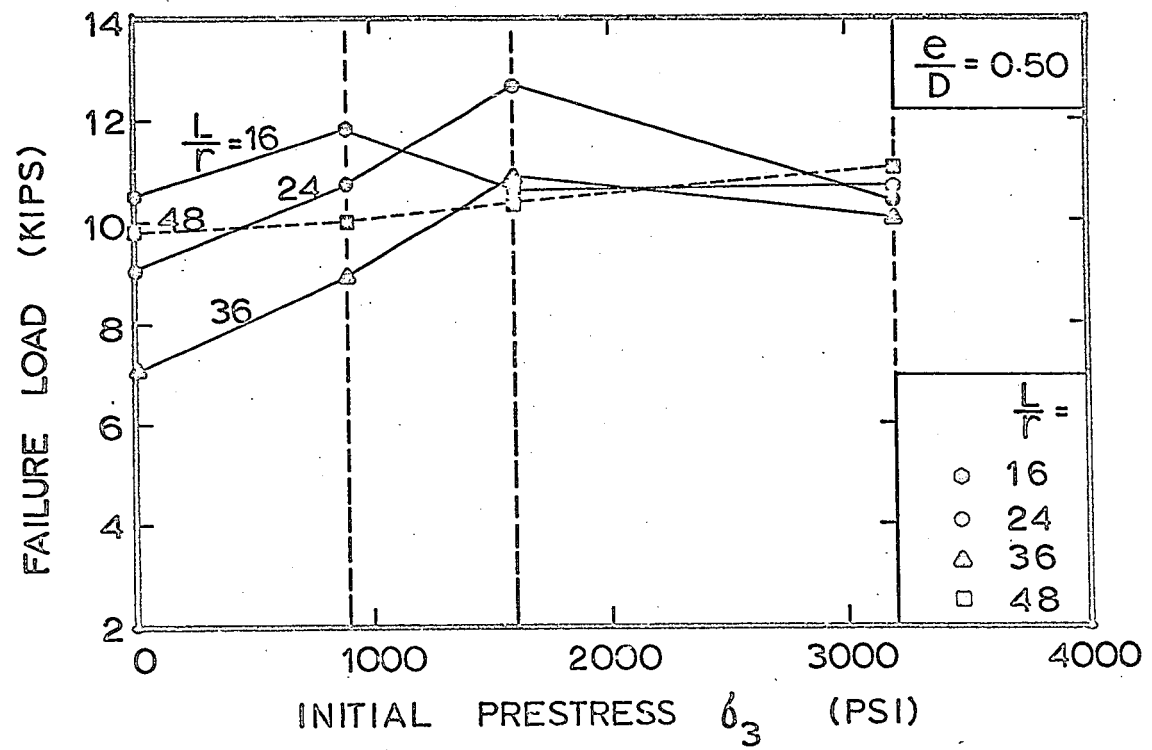
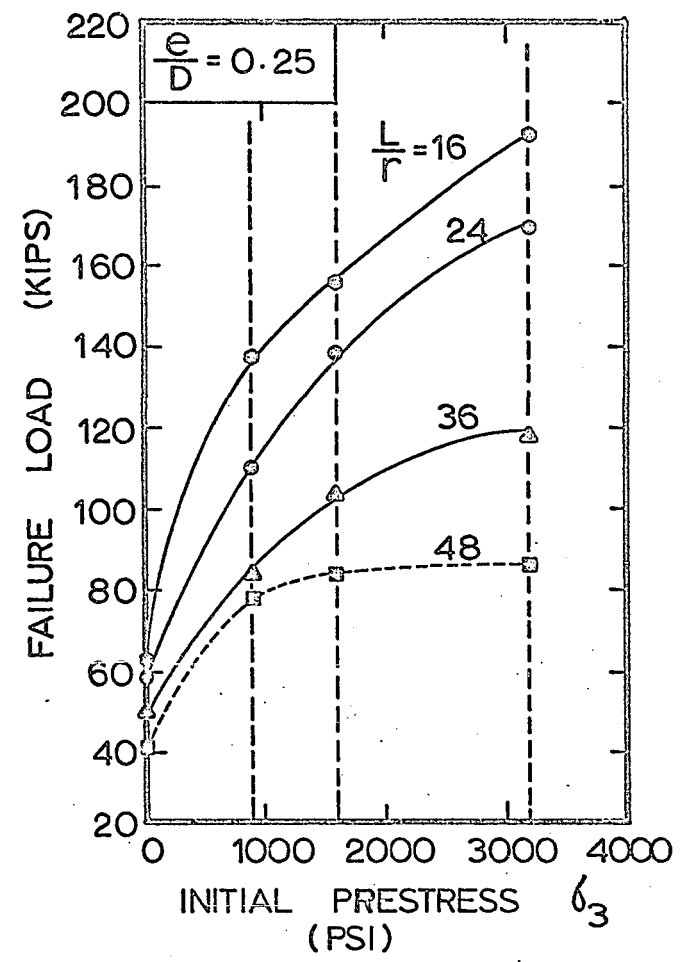
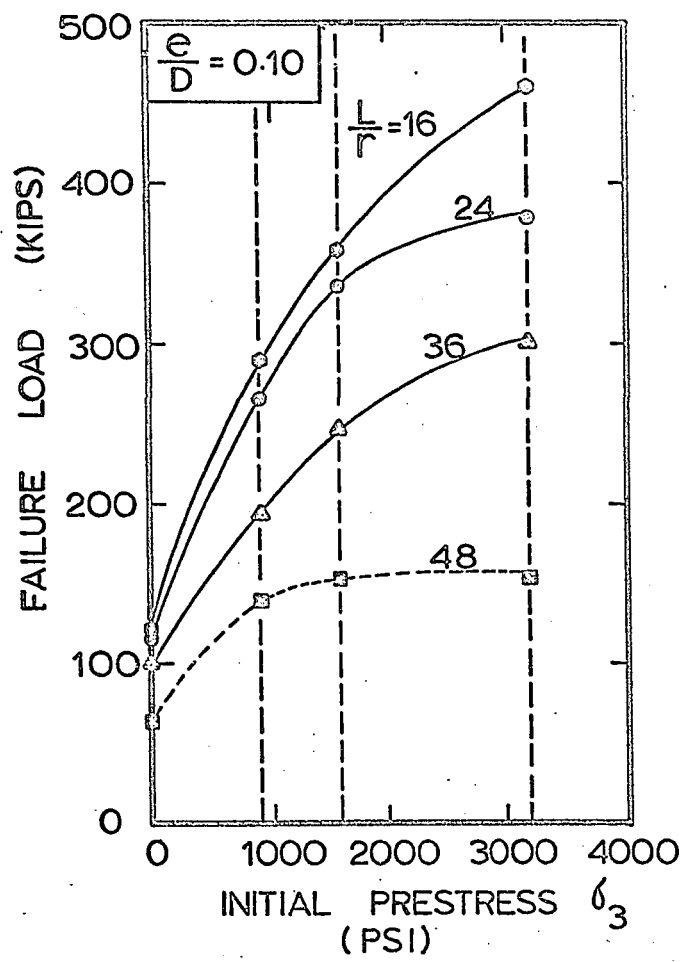


FIG. 5-8 VARIATION OF FAILURE LOAD WITH LATERAL PRESTRESS

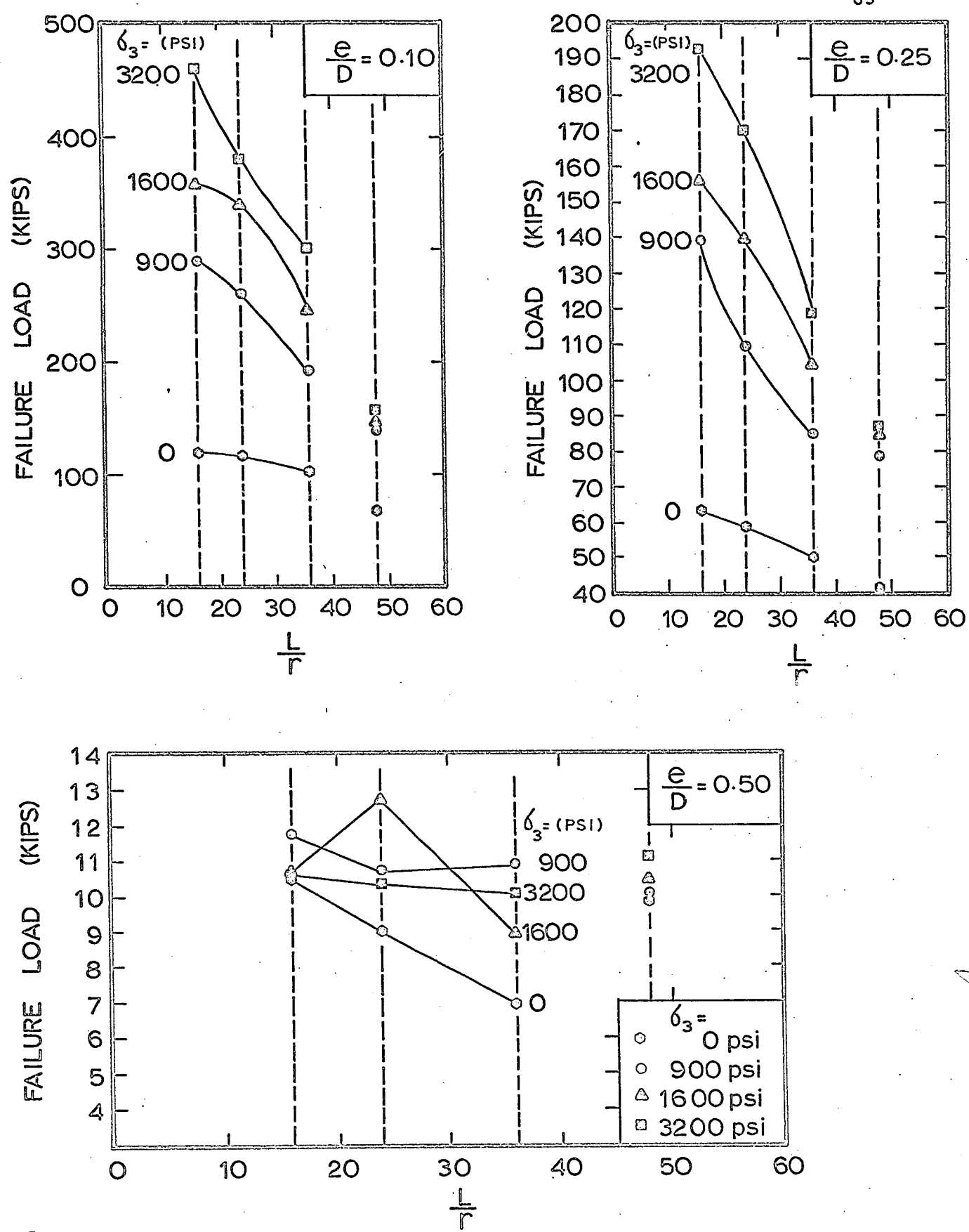


FIG. 5.9 VARIATION OF FAILURE LOAD WITH SLENDERNESS RATIO

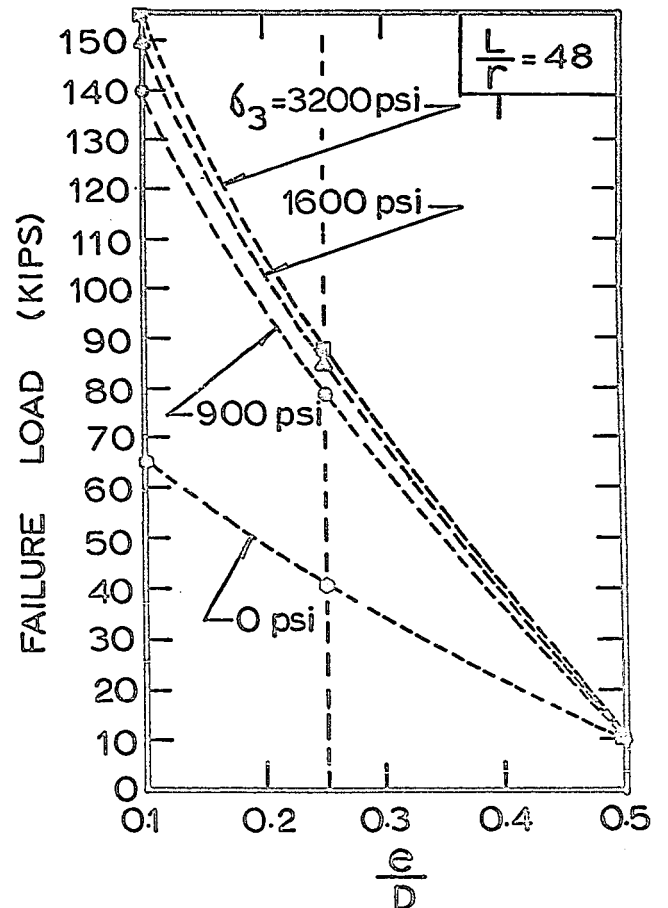
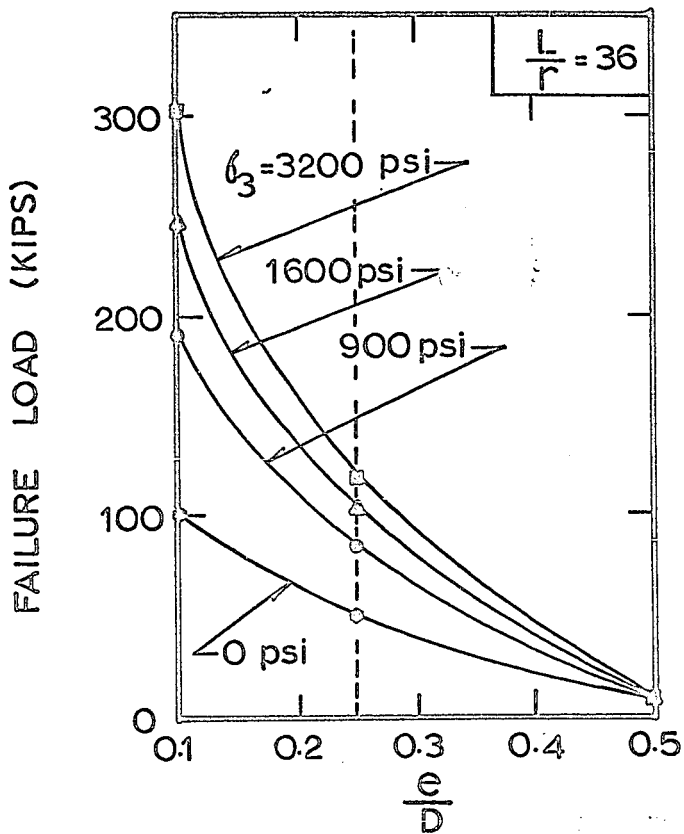
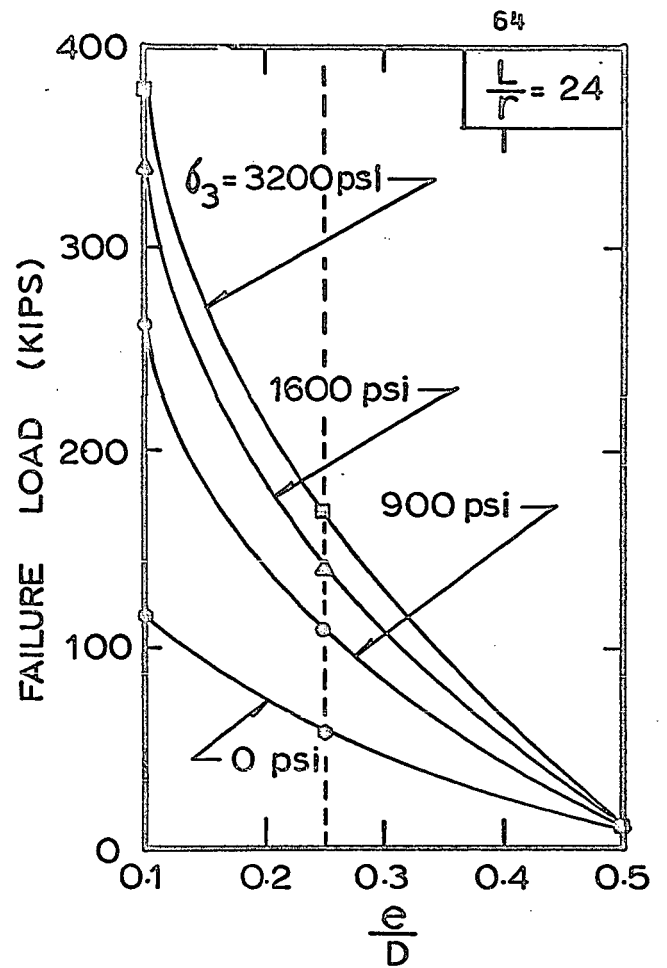
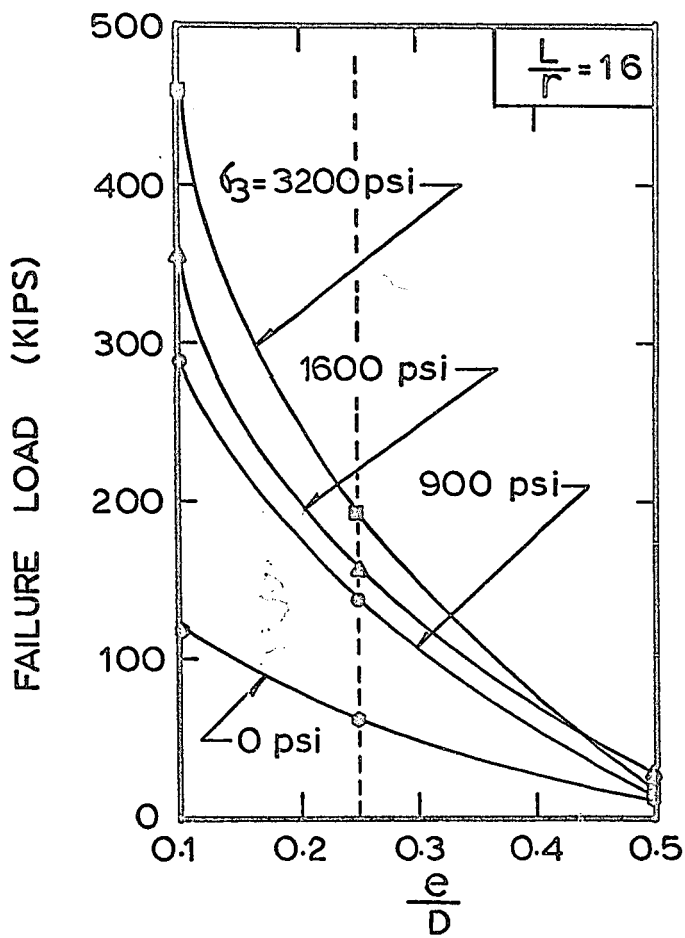


FIG. 5-10 VARIATION OF FAILURE LOAD
WITH ECCENTRICITY

5.5 The Proposed Formula

Assuming the expression of $\bar{\sigma}_1 = 0.85f'_c + K_1\bar{\sigma}_3$ holds and combining Eqns. 5.1, 5.2, 5.3, 5.4, 5.5, 5.6 and 5.7 from the effects of lateral prestress, slenderness ratio and eccentricity, a general expression of the relation between the failure load P_{ult} and the final lateral stress $\bar{\sigma}_3$, slenderness ratio $\frac{L}{r}$ and eccentricity $\frac{e}{D}$, can be written in the following form:

$$P_{ult} = A'_c \left\{ 0.85f'_c + K_1 \left[1 - K_2 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^\alpha \right] \left[K_3 - K_4 \frac{L}{r} \right] \left[1 - K_5 \frac{e}{D} \right] \bar{\sigma}_3 \right\} \left\{ R_1 - R_2 \frac{L}{r} \right\} \left\{ 1 - R_3 \frac{e}{D} \right\} \dots\dots\dots(5.8)$$

Using test results, the coefficients K_1 , K_2 , K_3 , K_4 , K_5 , α , R_1 , R_2 and R_3 for Eqn. 5.8 can be found by trial and error as well as the iteration method using computer programming.

5.5.1 Values of Empirical Coefficients R_1 and R_2

When $\frac{e}{D} = 0$ and $\bar{\sigma}_3 = 0$, Eqn. 5.8 becomes

$$P_{ult} = A'_c \{ 0.85f'_c \} \{ R_1 - R_2 \frac{L}{r} \} \dots\dots\dots(5.9)$$

Since $\frac{e}{D} = 0$, $A'_c = A_c = \pi D^2/4$ for cylindrical section.

The empirical coefficients R_1 and R_2 in Eqn. 5.9 can be determined from the test results of axially loaded unwrapped columns from Fig. 5.1 p. 53 by trial and error.

$R_1 = 1.042$ and $R_2 = 0.0057$ were fitted to the test results with a mean value of $P_{\text{exp}}/P_{\text{ult}}$ of 1.0443, standard deviation of 0.0355 and the coefficient of variation of 3.40 %.

5.5.2 Values of Empirical Coefficients K_1, K_2, K_3, K_4 and α

When $\frac{e}{D} = 0$, $\bar{\sigma}_3 \neq 0$, and R_1, R_2 are replaced by the values found in section 5.5.1, Eqn.5.8 becomes

$$P_{\text{ult}} = A'_c \left\{ 0.85f'_c + K_1 \left[1 - K_2 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^\alpha \right] \left[K_3 - K_4 \frac{L}{r} \right] \bar{\sigma}_3 \right\} \left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \dots\dots\dots(5.10)$$

In this case, $A'_c = A_c = \frac{\pi D^2}{4}$ for cylindrical section as $\frac{e}{D} = 0$.

The empirical coefficients K_1, K_2, K_3, K_4 and α in Eqn.5.10 were found from the test results of axially loaded wrapped columns from Fig.5.1 p.53 by the same methods as in section 5.5.1. The values

$$K_1 = 11.6$$

$$K_2 = 0.715$$

$$K_3 = 1.04$$

$$K_4 = 0.0056$$

$$\alpha = 0.2$$

were fitted to the test results with a mean value of $P_{\text{exp}}/P_{\text{ult}}$ of 1.0177, standard deviation of 0.0805 and the coefficient of variation of 7.92 %.

5.5.3 Value of Empirical Coefficient R_3

When $\frac{e}{D} \neq 0$, $\bar{\sigma}_3 = 0$ and R_1, R_2 are replaced by the values found in section 5.5.1, Eqn.5.8 becomes

$$P_{ult} = A'_c \{0.85f'_c\} \left\{1.042 - 0.0057 \frac{L}{r}\right\} \left\{1 - R_3 \frac{e}{D}\right\} \dots (5.11)$$

In this case,

$$A'_c = \frac{\pi D^2}{4} - 2e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1}\left(\frac{2e}{D}\right)$$

The empirical coefficient R_3 in Eqn.5.11 can be found from the test results of eccentrically loaded unwrapped columns, from Fig.5.2 p.54, by iteration method. The value of the coefficient $R_3 = 0.35$ was found to fit the test results with a mean value of P_{exp}/P_{ult} of 1.0281, standard deviation of 0.0253 and the coefficient of variation of 2.47 %.

5.5.4 Value of Empirical Coefficient K_5

Substituting the values of $R_1, R_2, R_3, K_1, K_2, K_3, K_4$ and α into Eqn.5.8, Eqn.5.8 becomes

$$P_{ult} = A'_c \left\{0.85f'_c + 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c}\right)^{0.2}\right] \left[1.04 - 0.0056 \frac{L}{r}\right] \left[1 - K_5 \frac{e}{D}\right] \bar{\sigma}_3 \right\} \left\{1.042 - 0.0057 \frac{L}{r}\right\} \left\{1 - 0.35 \frac{e}{D}\right\} \dots (5.12)$$

K_5 can be found based on test results of eccentrically loaded wrapped columns (i.e. when $\frac{e}{D} \neq 0$ and $\bar{\sigma}_3 \neq 0$) from Fig.5.2 p.54 by trial and error. Hence K_5 was found to be 0.66 to fit the test results

with a mean value of P_{exp}/P_{ult} of 1.0381, standard deviation of 0.0587 and the coefficient of variation of 5.66 %.

5.5.5 The Proposed Formula

Finally, the proposed formula for the failure load of laterally prestressed cylindrical concrete columns, either concentrically or eccentrically loaded, was determined to be

$$P_{ult} = A'_c \left\{ 0.85f'_c + 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \right. \\ \left. \left[1 - 0.66 \frac{e}{D} \right] \bar{\sigma}_3 \right\} \left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \left[1 - 0.35 \frac{e}{D} \right] \dots\dots\dots(5.13)$$

where $A'_c = \frac{\pi D^2}{4} - 2e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1} \left(\frac{2e}{D} \right)$ for $\frac{e}{D}$ varying from zero to 0.50.

Eqn.5.13 can be written in a simpler form as

$$P_{ult} = A'_c \{ 0.85f'_c + \beta K \bar{\sigma}_3 \} R \dots\dots\dots(5.13a)$$

where

$$\beta K = 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \left[1 - 0.66 \frac{e}{D} \right] \dots\dots(5.13b)$$

and

$$R = \left[1.042 - 0.0057 \frac{L}{r} \right] \left[1 - 0.35 \frac{e}{D} \right] \dots\dots\dots(5.13c)$$

The bending moment M_{ult} due to this failure load P_{ult} at the eccentricity e is then

$$M_{ult} = (P_{ult})(e) \dots\dots\dots(5.14)$$

In order to simplify the calculation of these equations (Eqn.5.13 and Eqn.5.14), some charts were prepared for the terms A'_c ,

βK , R , P_{ult} and M_{ult} , and are presented in Chapter 6.

5.5.6 Comparison Between the Values Calculated from the Proposed Formula and the Test Results

5.5.6.a Comparison with Present Investigation

The comparison between test results P_{exp} of present investigation and the calculated value P_{ult} from the proposed formula (Eqn.5.13) is presented in Fig.5.11 p.70 .

It shows that Eqn.5.13 gives a conservative value of P_{ult} for columns wrapped with single layer. Columns wrapped with double layers, the values of P_{ult} from Eqn.5.13 are a little bit higher than test results. This is because of that the application of double-layer wires has less efficiency than that of single-layer wires. When the outer layer wires were wrapped, it pressed on the inner layer wires and reduced the prestress in the inner layer wires. As the wrapped column was being loaded, the outer wires took more lateral stress and reached the ultimate value before the inner layer wires. After the outer wires had broken, the lateral stress was then taken by the inner wires alone and thus the inner wires failed too. The lateral stress could not be distributed to the two layers of wires equally. Thus longitudinal failure load was then lower than it was expected.

The empirical formula fitted to the results with a mean of P_{exp}/P_{ult} of 1.0311, a standard deviation of 0.061 and the coefficient of variation of 5.83 %. This shows the internal consistency of the proposed equation with all the test results is extremely good.

Fig.5.11 Comparison of Test Results P_{exp} and Calculated Values P_{ult}

Specimen No.	P_{exp} (kips)	P_{ult} (kips)	$\frac{P_{exp}}{P_{ult}}$	Specimen No.	P_{exp} (kips)	P_{ult} (kips)	$\frac{P_{exp}}{P_{ult}}$
A- 0-0	112.0	104.7	1.0694	A- 0-a	120.0	118.6	1.0117
A-18-0	290.0	289.4	1.0020	A-18-a	290.0	265.0	1.0942
A-32-0	374.0	380.5	0.9830	A-32-a	358.0	343.7	1.0415
A-64-0	573.0	512.3	1.1184	A-64-a	460.0	442.9	1.0387
B- 0-0	111.0	110.0	1.0094	A- 0-b	63.0	58.7	1.0732
B-18-0	284.0	280.2	1.0134	A-18-b	139.0	123.5	1.1256
B-32-0	364.0	366.4	0.9935	A-32-b	156.0	141.4	1.1035
B-64-0	545.0	496.3	1.0982	A-64-b	193.0	184.4	1.0466
C- 0-0	101.7	99.8	1.0187	B- 0-a	117.0	117.0	1.0000
C-18-0	261.7	245.3	1.0668	B-18-a	262.0	250.9	1.0444
C-32-0	323.7	318.6	1.0161	B-32-a	340.0	323.3	1.0517
C-64-0	378.7	428.2	0.8844	B-64-a	380.0	396.9	0.9574
D- 0-0	97.7	90.5	1.0800	B- 0-b	59.0	57.9	1.0189
D-18-0	239.7	212.2	1.1294	B-18-b	110.0	102.2	1.0761
D-32-0	281.7	273.4	1.0303	B-32-b	140.0	128.7	1.0878
D-64-0	319.7	364.6	0.8768	B-64-b	170.0	170.1	0.9994
				C- 0-a	101.0	97.9	1.0321
				C-18-a	192.0	183.5	1.0462
				C-32-a	248.0	234.0	1.0597
				C-64-a	301.0	306.6	0.9818
				C- 0-b	50.0	48.4	1.0323
				C-18-b	85.0	82.2	1.0335
				C-32-b	105.0	102.4	1.0249
				C-64-b	119.0	136.4	0.8725
Mean of $\frac{P_{exp}}{P_{ult}}$				= 1.0311			
Standard deviation				= 0.0601			
Coefficient of variation				= 5.827 %			

5.5.6.b Comparison with Previous Investigations

The comparisons between the calculated values P_{ult} from Eqn.5.13 and the test results P_{exp} of Johnson's (17), Goldenbatt-Ratz's (15) and Gambarov's (12,13) investigations are presented in Fig.5.12 p.72, Fig.5.13 p.73 and Fig.5.14 p.73 respectively. The comparisons are limited to $\frac{e}{D} = 0$ since all previous investigations of wire-wrapped columns used concentric loading only.

In Fig.5.12 p.72 , the comparison with Johnson's (17) investigation, shows the mean of P_{exp}/P_{ult} of 1.1094, a standard deviation of 0.0971 and the coefficient of variation of 8.754 %.

In Fig.5.13 p.73 , the comparison with Goldenbatt-Ratz's (15) investigation, shows the mean of P_{exp}/P_{ult} of 1.2222, a standard deviation of 0.0428 and the coefficient of variation of 3.500 %.

In Fig.5.14 p.73 , the comparison with Gambarov's (12,13) investigation, shows the mean of P_{exp}/P_{ult} of 1.0702, a standard deviation of 0.1591 and the coefficient of variation of 14.862 %.

FIG. 5-12 COMPARISON BETWEEN CALCULATED VALUES P_{ult} AND TEST RESULTS P_{exp} OF JOHNSON'S (17) INVESTIGATION

SPECIMENS	f'_c (psi)	$\bar{\delta}_3$ (psi)	P_{exp} (kips)	P_{ult} (kips)	$\frac{P_{exp}}{P_{ult}}$
LENGTH = 12" DIAMETER = 3" $\frac{L}{r} = 16$ $\frac{e}{D} = 0$	4870	400	44.66	44.60	1.0013
	3680		39.60	37.07	1.0684
	3470		40.75	35.71	1.1413
	4870	670	51.60	53.59	0.9628
	3680		49.60	45.42	1.0921
	3470		50.30	43.92	1.1452
	4870	930	56.40	61.35	0.9193
	3680		56.20	52.51	1.0703
	3470		59.60	50.87	1.1716
	4870	1200	65.60	68.69	0.9550
	3680		63.50	59.13	1.0740
	3470		67.30	57.33	1.1740
	3620	790	48.00	48.35	0.9928
	4000		51.40	51.13	1.0053
	3630		55.60	48.42	1.1483
	3620	1320	61.20	61.33	0.9979
	4000		67.90	64.63	1.0506
	3630		70.70	61.42	1.1511
	3620	1850	74.60	72.06	1.0353
	4000		77.70	75.92	1.0234
	3630		87.70	72.16	1.2153
	3620	2370	83.20	80.93	1.0281
	4000		89.10	85.38	1.0436
	3630		94.60	81.05	1.1672
	3800	1160	61.40	59.19	1.0374
	3660		69.60	58.02	1.1996
	3880		71.50	59.85	1.1948
	3800	1930	85.80	75.42	1.1376
	3660		93.00	73.95	1.2576
	3880		91.60	76.25	1.2013
	3800	2710	96.50	88.32	1.0926
	3660		111.20	86.51	1.2854
	3880		109.50	89.34	1.2257
	3800	3480	115.70	98.41	1.1757
	3660		120.20	96.25	1.2489
	3880		124.40	99.62	1.2487
MEAN OF P_{exp} / P_{ult} = 1.1094					
STANDARD DEVIATION = 0.0971					
COEFFICIENT OF VARIATION = 8.754 %					

FIG. 5-13 COMPARISON BETWEEN CALCULATED VALUES P_{ult} & TEST RESULTS P_{exp} OF GOLDENBATT-RATZE'S (15) INVESTIGATION

SPECIMENS	f'_c (psi)	$\bar{\delta}_3$ (psi)	P_{exp} (kips)	P_{ult} (kips)	$\frac{P_{exp}}{P_{ult}}$
LENGTH = 118"	4250	850	1162.00	1001.57	1.1602
DIAMETER = 14"	4400	920	1332.00	1056.31	1.2610
$\frac{L}{r} = 33.7$	4700	460	1016.00	849.91	1.1954
$\frac{e}{D} = 0$	5100	910	1432.00	1147.17	1.2483
	5000	430	1082.00	868.30	1.2461
MEAN OF P_{exp} / P_{ult} = 1.2222 STANDARD DEVIATION = 0.0428 COEFFICIENT OF VARIATION = 3.500 %					

FIG. 5-14 COMPARISON BETWEEN CALCULATED VALUES P_{ult} & TEST RESULTS P_{exp} OF GAMBAROV'S (12,13) INVESTIGATION

SPECIMENS	f'_c (psi)	$\bar{\delta}_3$ (psi)	P_{exp} (kips)	P_{ult} (kips)	$\frac{P_{exp}}{P_{ult}}$
LENGTH = 31.5"	7300	1200	237.00	272.24	0.8706
DIAMETER = 5.5"	7500	1200	278.00	276.66	1.0049
$\frac{L}{r} = 22.9$	7300	1200	311.00	272.24	1.1424
$\frac{e}{D} = 0$	8100	1200	285.00	289.78	0.9835
	8150	1200	316.00	290.87	1.0864
	8050	1200	385.00	288.69	1.3336
MEAN OF P_{exp} / P_{ult} = 1.0702 STANDARD DEVIATION = 0.1591 COEFFICIENT OF VARIATION = 14.862 %					

CHAPTER 6

CHARTS FOR THE PROPOSED FORMULA AND DESIGN METHOD

Because the proposed formula is quite lengthy and cumbersome, computer generated design charts have been prepared to simplify the calculation of the proposed formula. A design method using the presented ultimate load/moment/lateral stress interaction diagrams is also given.

6.1 Charts

The empirical formula summarizing the experimental results is repeated below:

$$P_{ult} = A'_c \left\{ 0.85f'_c + 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \right. \\ \left. \left[1 - 0.66 \frac{e}{D} \right] \bar{\sigma}_3 \right\} \left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \left\{ 1 - 0.35 \frac{e}{D} \right\} \dots\dots\dots (5.13)$$

This equation can be rewritten in the form as given by Eqn.5.13a

$$P_{ult} = A'_c \{ 0.85f'_c + \beta K \bar{\sigma}_3 \} R \dots\dots\dots (6.1)$$

where $\beta K = 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \left[1 - 0.66 \frac{e}{D} \right] \dots\dots (6.1a)$

and $R = \left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \left\{ 1 - 0.35 \frac{e}{D} \right\} \dots\dots\dots (6.1b)$

Four groups of design charts are prepared in nondimensional forms, for the values of A'_c , βK , and R in Eqn.6.1, and also the interaction of axial load P_{ult} and bending moment M_{ult} .

6.1.1 Chart for A'_c Values

In section 4.2.2, A'_c was derived to be

$$A'_c = \frac{\pi D^2}{4} - 2e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1}\left(\frac{2e}{D}\right)$$

Dividing both sides by D^2 , it becomes the nondimensional expression

$$\frac{A'_c}{D^2} = \frac{\pi}{4} - 2e \sqrt{\frac{1}{4} - \left(\frac{e}{D}\right)^2} - \frac{1}{2} \sin^{-1}(2e/D) \dots\dots\dots(6.2)$$

Eqn.6.2 is represented by Fig.6.1 p.84, from which the nondimensional value of A'_c/D^2 can be obtained for e/D values ranging from $e/D = 0$ to 0.50. The magnitude of A'_c can then be obtained by multiplying A'_c/D^2 by the specified D^2 .

6.1.2 Chart for R Values

In Eqn.6.1, the reduction factor R is expressed by Eqn.6.1b

$$R = \left\{1.042 - 0.0057 \frac{L}{r}\right\} \left\{1 - 0.35 \frac{e}{D}\right\} \dots\dots\dots(6.1b)$$

The values of R can be obtained from Fig.6.2 p.85 for the specified L/r and e/D ratios.

6.1.3 Charts for βK Values

The parameter βK representing the increase in axial stress capacity due to lateral stress with L/r , e/D and $\bar{\sigma}_3/f'_c$, is given by

Eqn.6.1a as

$$\beta K = 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \left[1 - 0.66 \frac{e}{D} \right] \dots (6.1a)$$

The values of βK can be obtained for the specified ratios of lateral stress, slenderness and eccentricity ratios from Fig.6.3 p.86 through Fig.6.12 p.95 .

6.1.4 Ultimate Load/Moment/Lateral Stress Interaction Diagrams

Dividing both sides of Eqn.5.13 by $D^2 f'_c$, the load equation becomes

$$\frac{P_{ult}}{D^2 f'_c} = \frac{A'_c}{D^2} \left\{ 0.85 + 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \left[1 - 0.66 \frac{e}{D} \right] \left(\frac{\bar{\sigma}_3}{f'_c} \right) \right\} \left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \left\{ 1 - 0.35 \frac{e}{D} \right\} \dots (6.3)$$

where A'_c/D^2 has been described in section 6.1.1 Eqn.6.2. Based on Eqn.6.3, Figs.6.13 p.96 through 6.30 p.113 were plotted of interaction diagrams with the nondimensional values of C_P and C_M , where

$$C_P = \frac{P_{ult}}{D^2 f'_c} \dots \dots \dots (6.4)$$

and

$$C_M = C_P \left(\frac{e}{D} \right) = \frac{P_{ult}(e)}{D^3 f'_c} = \frac{M_{ult}}{D^3 f'_c} \dots \dots \dots (6.5)$$

For any combination of lateral stress ratio $\bar{\sigma}_3/f'_c$, slenderness ratio $\frac{L}{r}$ and eccentricity $\frac{e}{D}$, the values of C_P and C_M can be obtained from these diagrams. The magnitude of the ultimate load P_{ult} and moment M_{ult} can then be obtained with

$$P_{ult} = C_P D^2 f'_c \dots\dots\dots(6.6)$$

and $M_{ult} = C_M D^3 f'_c \dots\dots\dots(6.7)$

6.1.5 Chart for Axial Tensile Force in the Wrapping Wires

The lateral stress at failure governs the load capacity of those columns at failure. Hence, to predict the failure load, it is necessary to know the lateral stress at failure. It is suggested that the initial lateral prestress be 80 % of the wire ultimate tension and that this stress also be used as the final lateral stress at failure.

For the given values of diameter D_w and ultimate strength f_w of the wrapping wires, the axial tensile force T' developed just before the rupture of the wires is

$$T' = A_w f_w = \frac{\pi D_w^2}{4} f_w \dots\dots\dots(6.8)$$

where A_w is the cross-sectional area of the wire.

Fig.6.31 p.114 shows the values of force T , which is 80 % of T' , against D_w for several selected ultimate strengths of wires.

6.2 Example of Using The Charts to Predict Failure Load

For a plain concrete column with known values of concrete strength, diameter, length, eccentricity and the final lateral stress at the time of failure, the failure load can be predicted by using the design charts presented in this Chapter. Let's take specimen C-32-a as an example:

Specimen C-32-a :

Known : Effective length $L = 54''$

Diameter $D = 6''$

$f'_c = 6750$ psi

$\bar{\sigma}_3 = 2000$ psi

$e = 0.6''$

Solutions :

$$1) \frac{L}{r} = \frac{4L}{D} = 36$$

$$\frac{e}{D} = 0.10$$

$$\frac{\bar{\sigma}_3}{f'_c} = 0.3$$

2) Method 1 : Find the failure load and moment by parts:

From Fig.6.1 p.84 , for $\frac{e}{D} = 0.1$, it gives

$$\frac{A'_c}{D^2} = 0.588$$

then, $A'_c = 0.588(36) = 21.12$ sq.in.

From Fig.6.2 p.85 , for $\frac{e}{D} = 0.1$ and $\frac{L}{r} = 36$, it gives

$$R = 0.808$$

From Fig.6.5 p.88 , for $\frac{\bar{\sigma}_3}{f'_c} = 0.3$, $\frac{e}{D} = 0.1$ and $\frac{L}{r} = 36$,

$$\text{it gives } \beta K = 3.98$$

Then, the failure load and moment are

$$P_{ult} = 21.12(0.85f'_c + 3.98\bar{\sigma}_3)(0.808) = \underline{234000} \text{ lbs.} \quad \text{and}$$

$$M_{ult} = P_{ult} (e) = \underline{140400} \text{ in.-lb.}$$

- 3) Method 2 : Find the failure load and moment from the interaction diagrams :

From Fig.6.23 p.106 , for $\frac{L}{r} = 36$, $\frac{e}{D} = 0.1$ and $\frac{\bar{\sigma}_3}{f'_c} = 0.3$

$$\text{it gives } C_P = 0.965 \quad \text{and} \quad C_M = 0.0965$$

Then

$$P_{ult} = C_P D^2 f'_c = \underline{234000} \text{ lbs.} \quad \text{and}$$

$$M_{ult} = C_M D^3 f'_c = \underline{140400} \text{ in.-lb.}$$

6.3 Design Method of Laterally Prestressed Concrete Columns

From the test results, it is found that the magnitude of the final lateral stress $\bar{\sigma}_3$ at the moment of column failure is between the initial lateral prestress σ_3 and the ultimate lateral stress σ'_3 , depending upon the failure phenomenon of the column. If it is known that the column will fail simultaneously by both crushing of the concrete and rupture of wires, $\bar{\sigma}_3$ can be replaced by σ'_3 in the design formula. However, in general, especially for the long columns with eccentric loads,

the column fails by crushing of the concrete, without rupture of the wire, and the magnitude of $\bar{\sigma}_3$ becomes unpredictable.

For design purposes, this difficulty can be overcome by using the initial lateral prestress σ_3 equal to the final lateral stress $\bar{\sigma}_3$ in the design formula, and by specifying the percentage of σ_3/σ_3' in order to give a reasonable result.

In this thesis, it is suggested to that the initial lateral prestress σ_3 be 80 % of the ultimate lateral stress σ_3' , and to use this initial lateral prestress for design, i.e. use 80 % of the ultimate tensile force T' of the wire as the initial axial tension T applied to the wires. This suggestion will probably give a conservative result but will not be very inaccurate since the maximum $\bar{\sigma}_3$ in this case is just slightly higher than σ_3 , within 20 %.

For a given column height L , concrete strength f'_c , design ultimate axial load P_{ult} and bending moment M_{ult} , the laterally prestressed concrete column can be designed as follows:

- 1) Choose a cross-section (diameter D) of concrete column.
- 2) With given data and chosen cross-section, calculate the values of:

$$e = M_{ult}/P_{ult}$$

$$\frac{L}{r} = \frac{4L}{D}$$

$$C_P = P_{ult}/(D^2 f'_c)$$

$$C_M = M_{ult}/(D^3 f'_c) \quad \text{and}$$

$$e/D$$

If $\frac{e}{D} \geq 0.50$, choose another bigger cross-section until $\frac{e}{D} < 0.50$.

- 3) From the appropriate interaction diagram, from Fig.6.13 p.96 through Fig.6.30 p.113, for the calculated $\frac{L}{r}$, find the required lateral prestress ratio $\bar{\sigma}_3/f'_c$ for calculated C_P , C_M and $\frac{e}{D}$ ratios. Then the required lateral prestress

$$\sigma_3 = (\bar{\sigma}_3/f'_c) f'_c$$

- 4) Choose the diameter D_w and ultimate strength f_w of the wrapping wires. From Fig.6.31 p.114, find the initial tensile force T for the chosen wire.

- 5) Calculate the required pitch N of the wrapping, using Eqn.4.1

$$\text{Total } N_1 = \sigma_3 D / (2T) \quad \text{wires/inch}$$

$$N_2 = 2T / (\sigma_3 D) \quad \text{inch c/c of wires/layer}$$

- 6) Check the pitch N_2 inch c/c of wires/layer

if $N_2 \geq D_w$, (O.K.)

if $N_2 < D_w$, repeat from step 4) with other size D_w of wire,

or redesign from step 1) with another cross-section D of the column section.

Example : Design a laterally prestressed concrete column to carry a design ultimate load of 416 kips and bending moment 83.1 ft.-kips.

The storey height is 10 feet. Concrete strength $f'_c = 5000$ psi.

- 1) Choose $D = 12"$ for the cross section of the column.

$$2) \frac{L}{r} = \frac{4L}{D} = \frac{4(10)(12)}{12} = 40$$

$$e = M_{ult}/P_{ult} = \frac{83.1(12)}{416} = 2.4"$$

$$\frac{e}{D} = \frac{2.4}{12} = 0.2$$

$$C_P = \frac{P_{ult}}{D^2 f'_c} = \frac{416000}{12(12)(5000)} = 0.5775$$

$$C_M = \frac{M_{ult}}{D^3 f'_c} = \frac{83100(12)}{12(12)(12)(5000)} = 0.1155$$

- 3) From Fig.6.25 p.108, for $\frac{L}{r} = 40$, $\frac{e}{D} = 0.2$, $C_P = 0.5775$ and $C_M = 0.1155$, the required prestress ratio is

$$\frac{\sigma_3}{f'_c} = 0.3$$

Then the required lateral prestress is

$$\sigma_3 = 0.3(5000) = \underline{1500} \text{ psi}$$

- 4) Choose the wrapping wire : size $D_w = 0.04''$, $f_w = 375 \text{ ksi}$.

From Fig.6.31 p.114, the required initial tensile force applied to the wire is

$$T = \underline{377.5} \text{ lbs.}$$

- 5) The total pitch of wires

$$N_1 = \frac{D}{2T} = \frac{1500(12)}{2(377.5)} = 23.8 \text{ wires/inch}$$

say $N_1 = 24 \text{ wires/inch}$, wrapped in one layer.

- 6) Check N_2

$$N_2 = \frac{1}{24} = 0.0417 \text{ "c/c} > D_w = 0.04'' \text{ (O.K.)}$$

- 7) Therefore, the designed column is then

$$\text{Diameter } D = 12''$$

$$\text{Wrapping wire : Diameter } D_w = 0.04''$$

ultimate strength $f_w = 375$ ksi

Pitch of wrapping $N_2 = 24$ wires/inch, one layer of wrapping,

Initial tension applied to the wire $T = 377.5$ lbs.

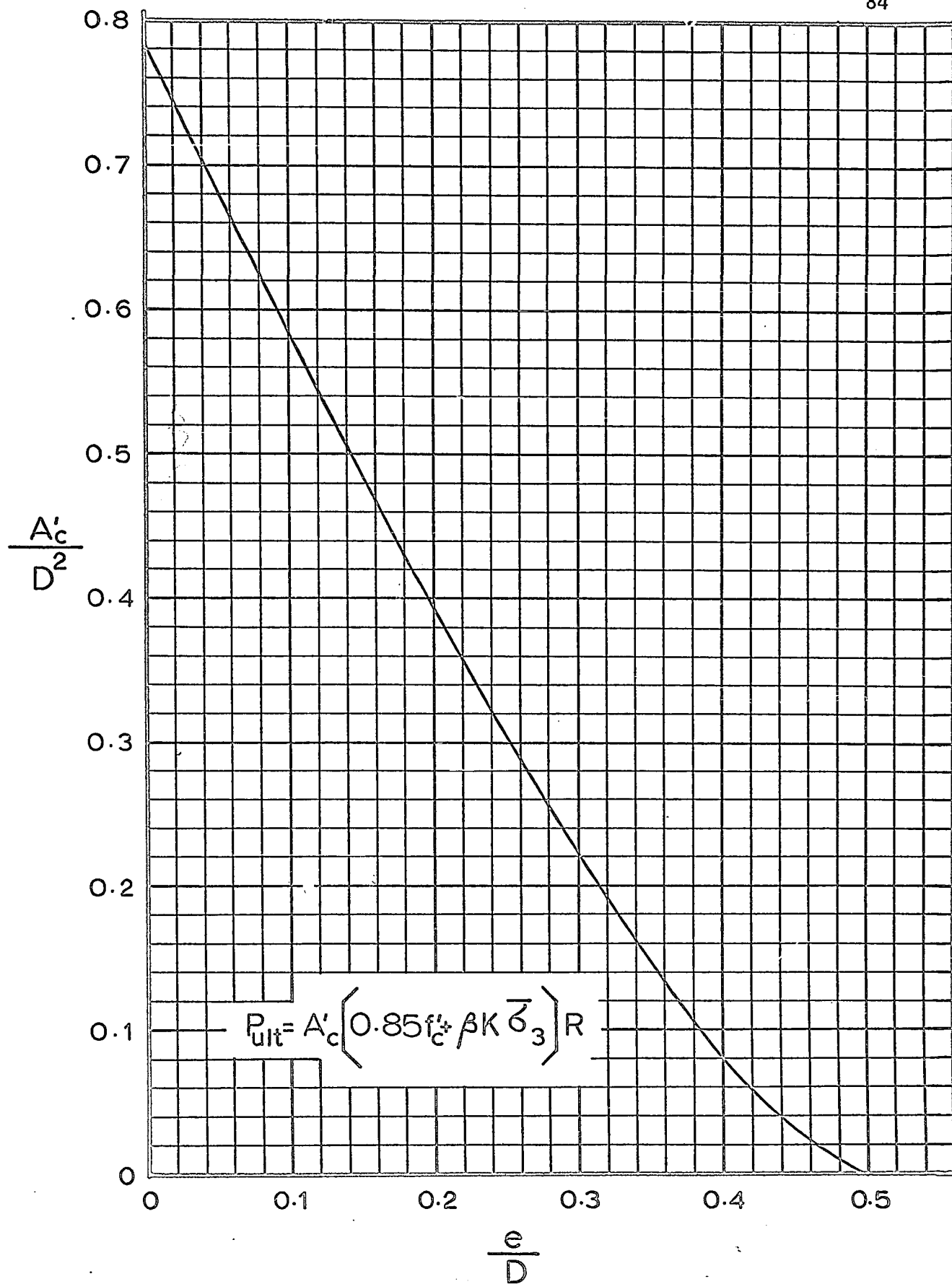


FIG. 6.1 VALUES OF $\frac{A'_c}{D^2}$

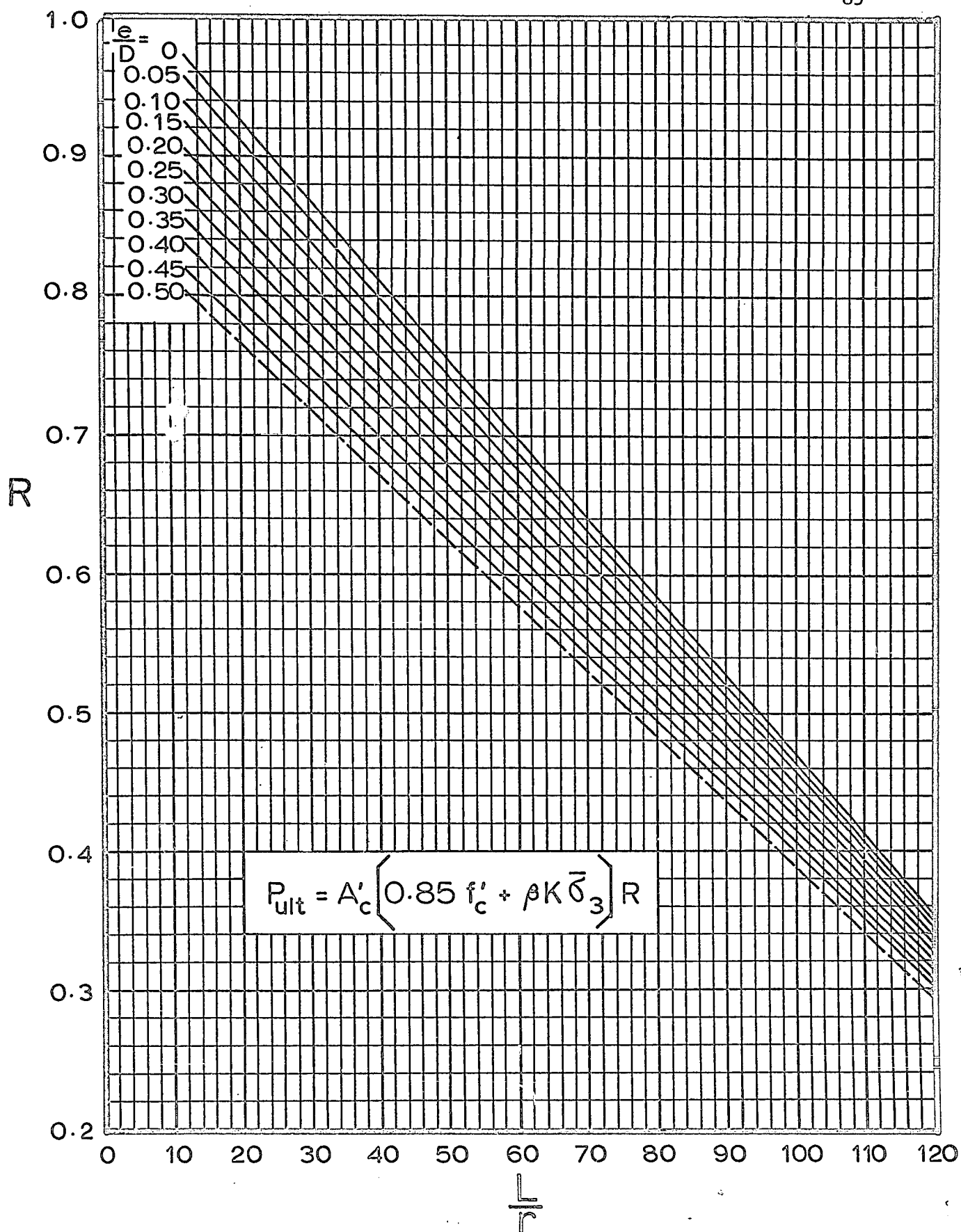


FIG. 6.2 VALUES OF R

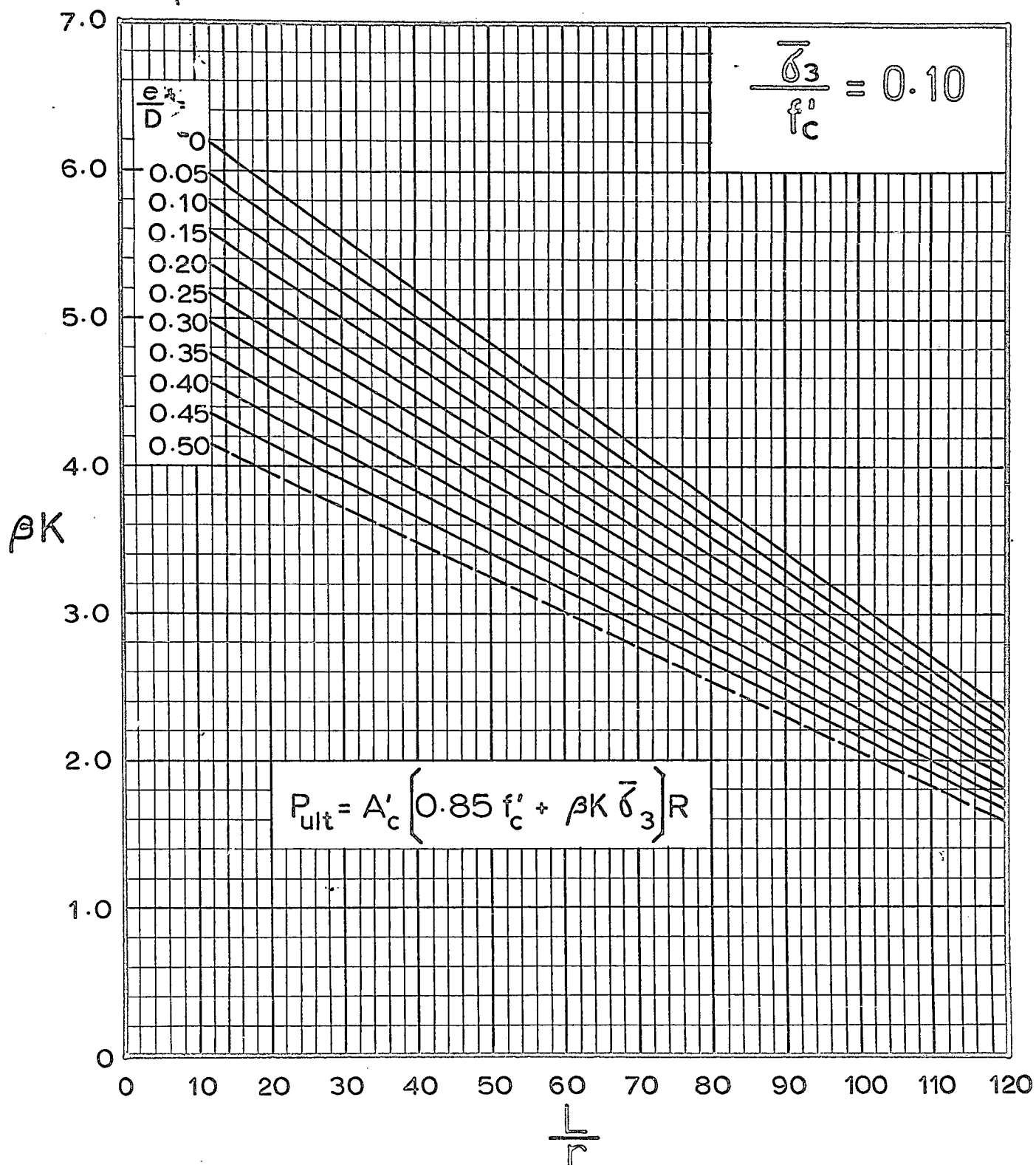


FIG. 6.3 VALUES OF βK
FOR $\frac{\delta_3}{f'_c} = 0.10$

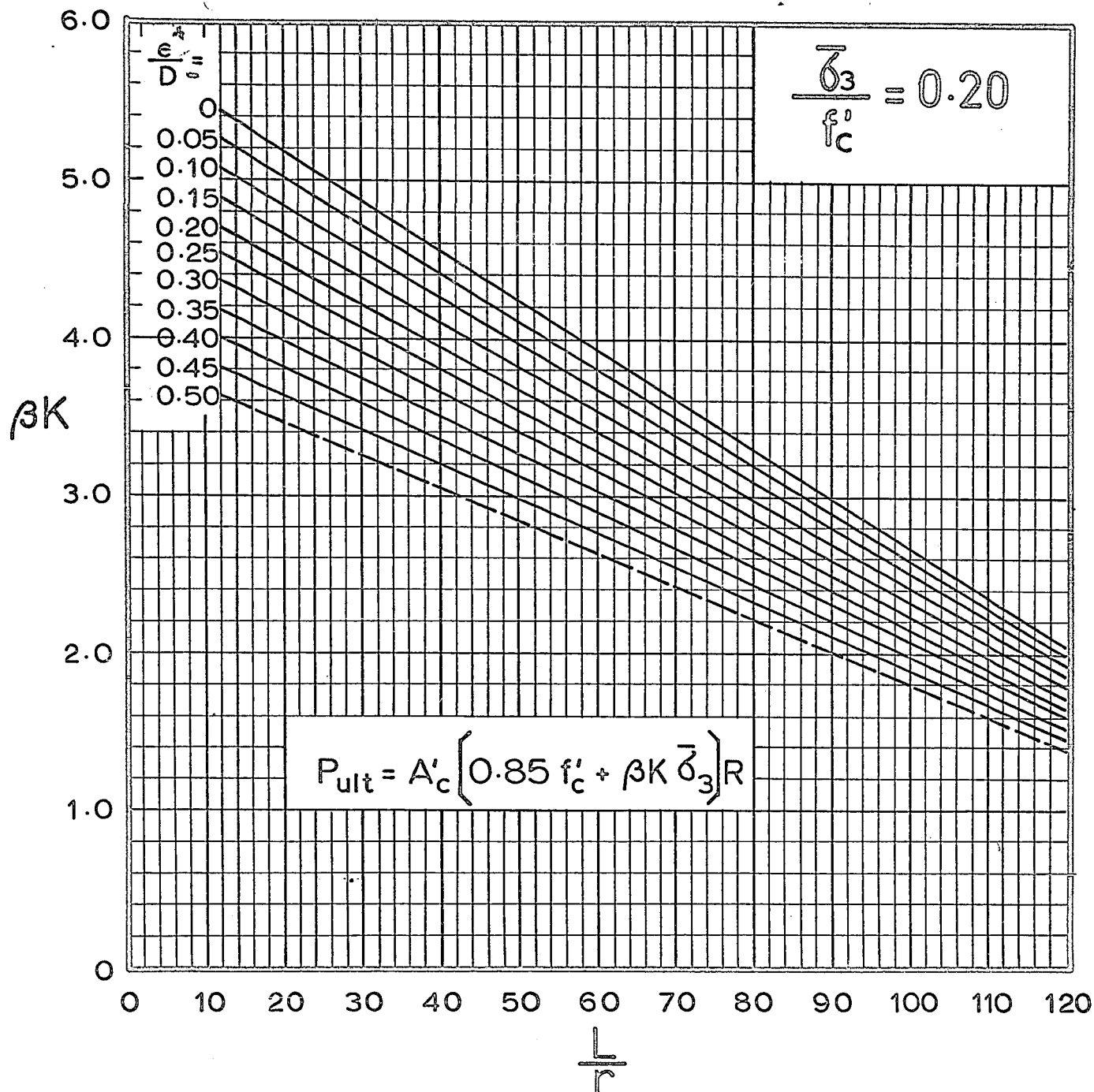


FIG. 6.4 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 0.20$

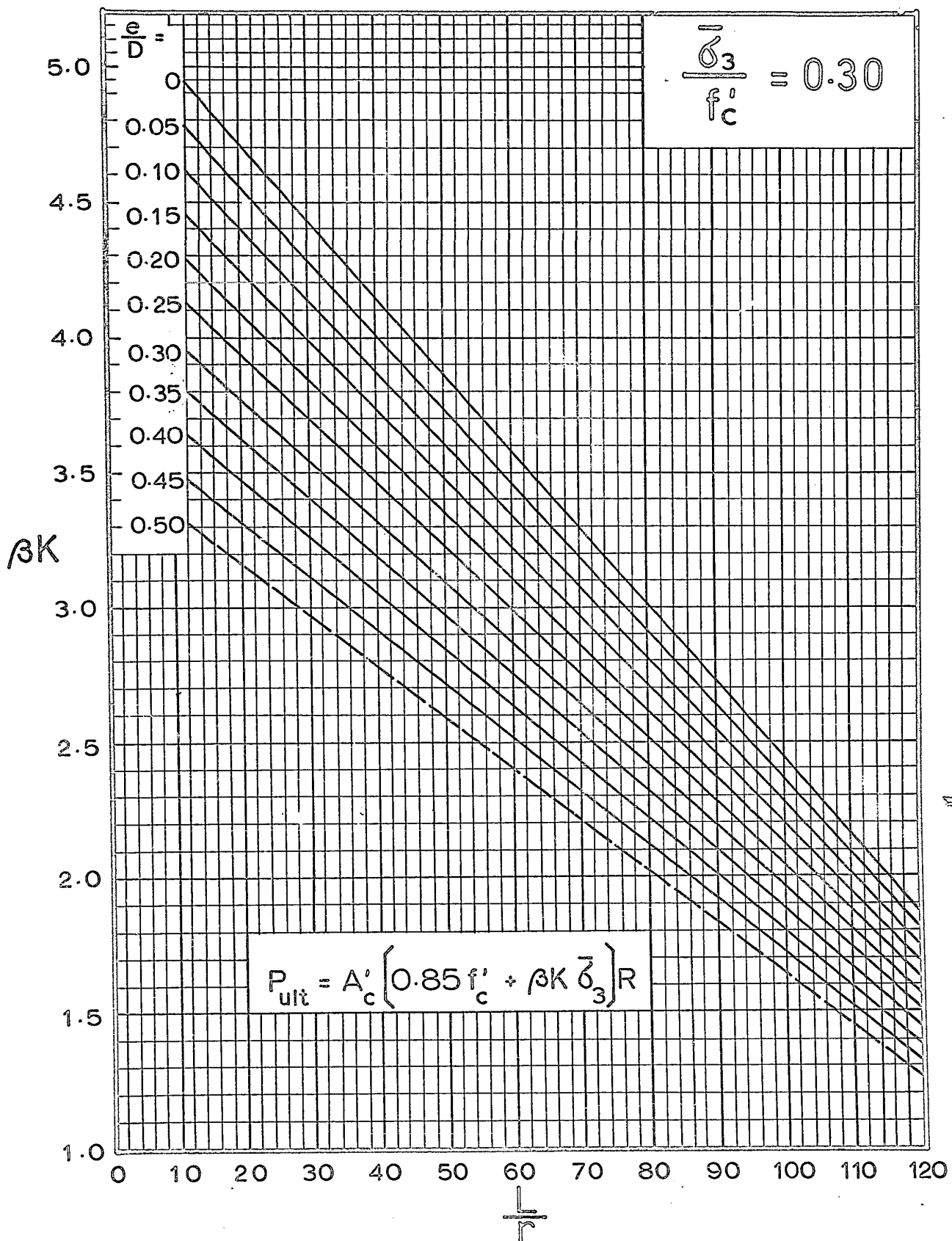


FIG. 6.5 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 0.30$

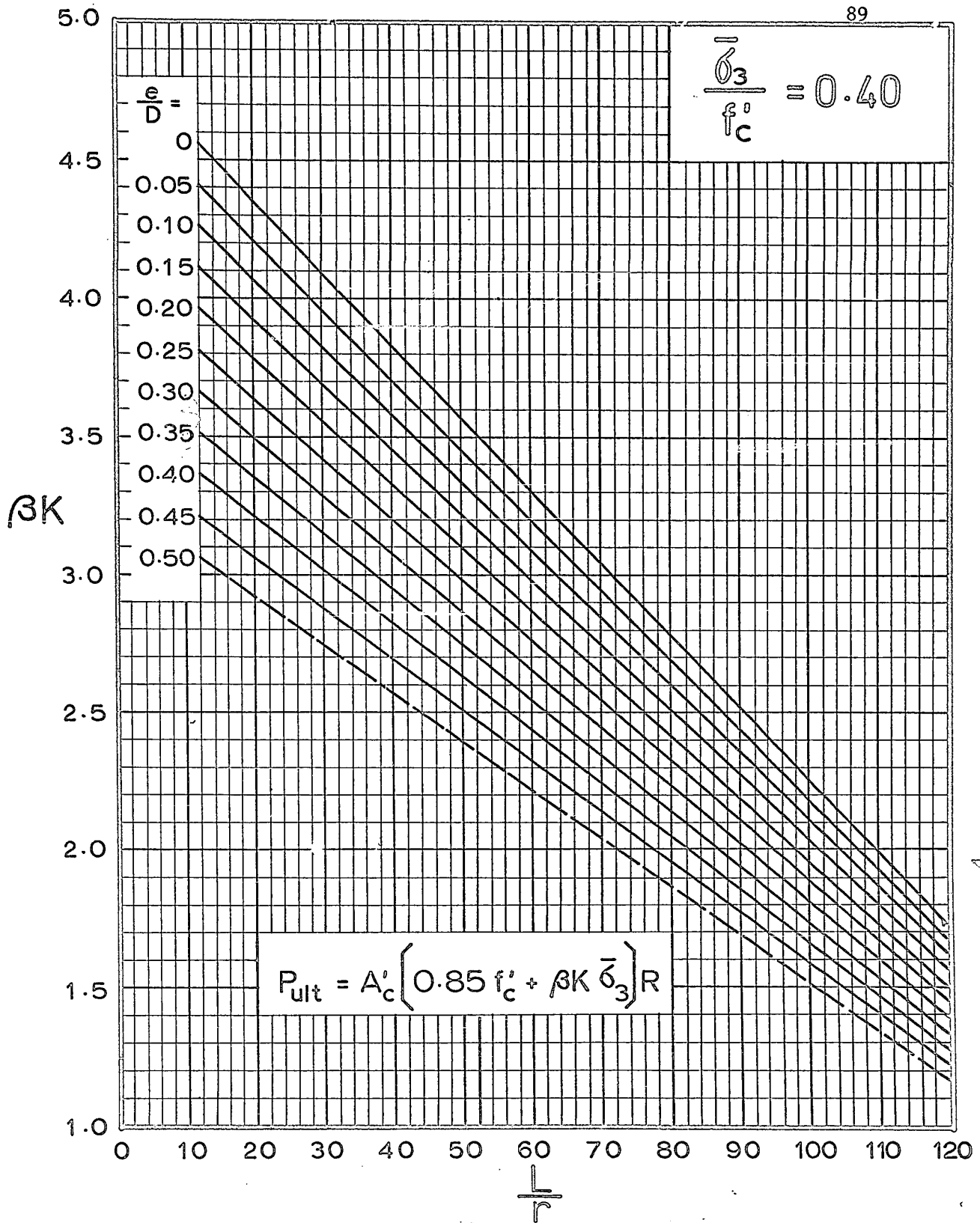


FIG. 6.6 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 0.40$

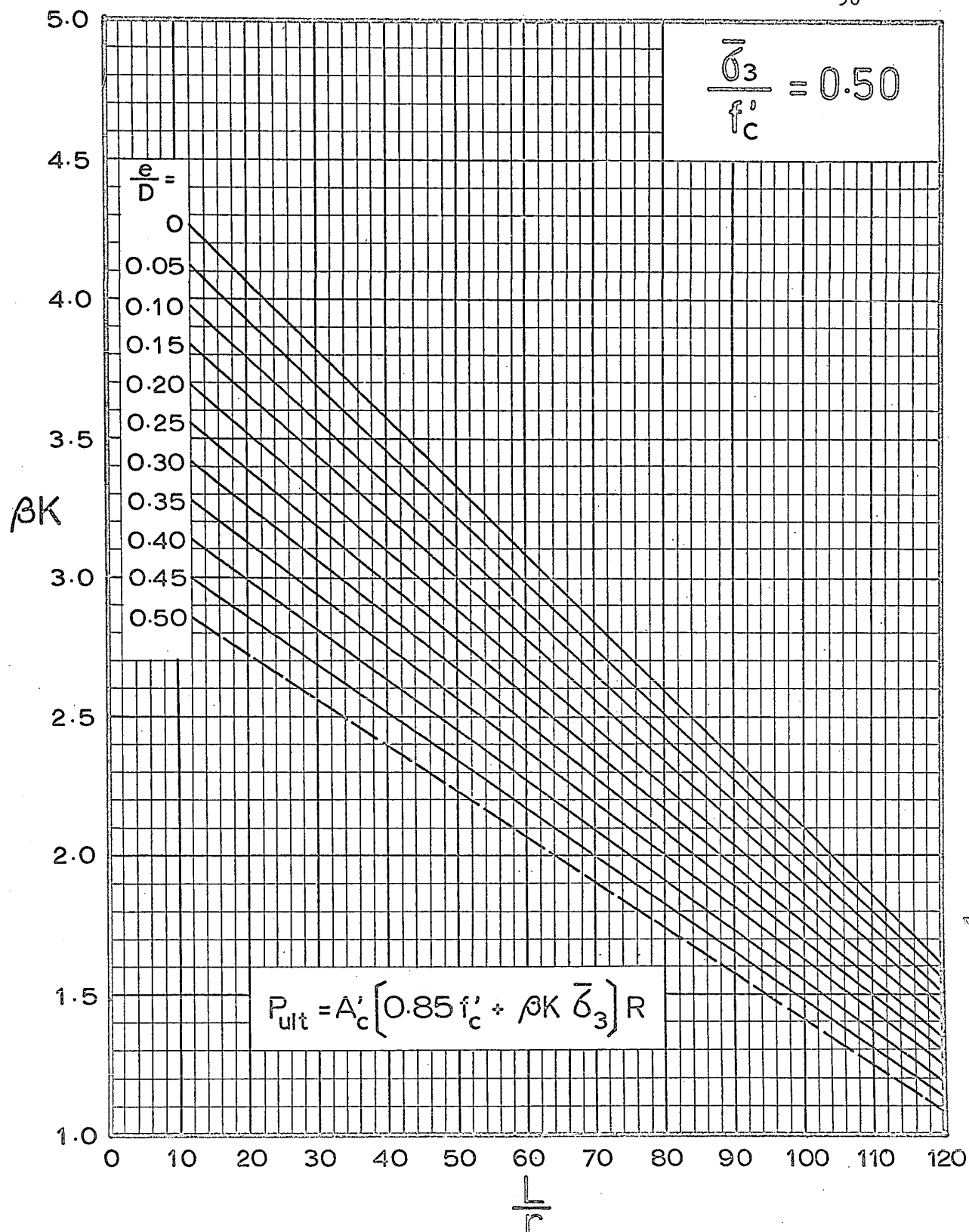


FIG. 6.7 VALUES OF βK
 FOR $\frac{\delta_3}{f'_c} = 0.50$

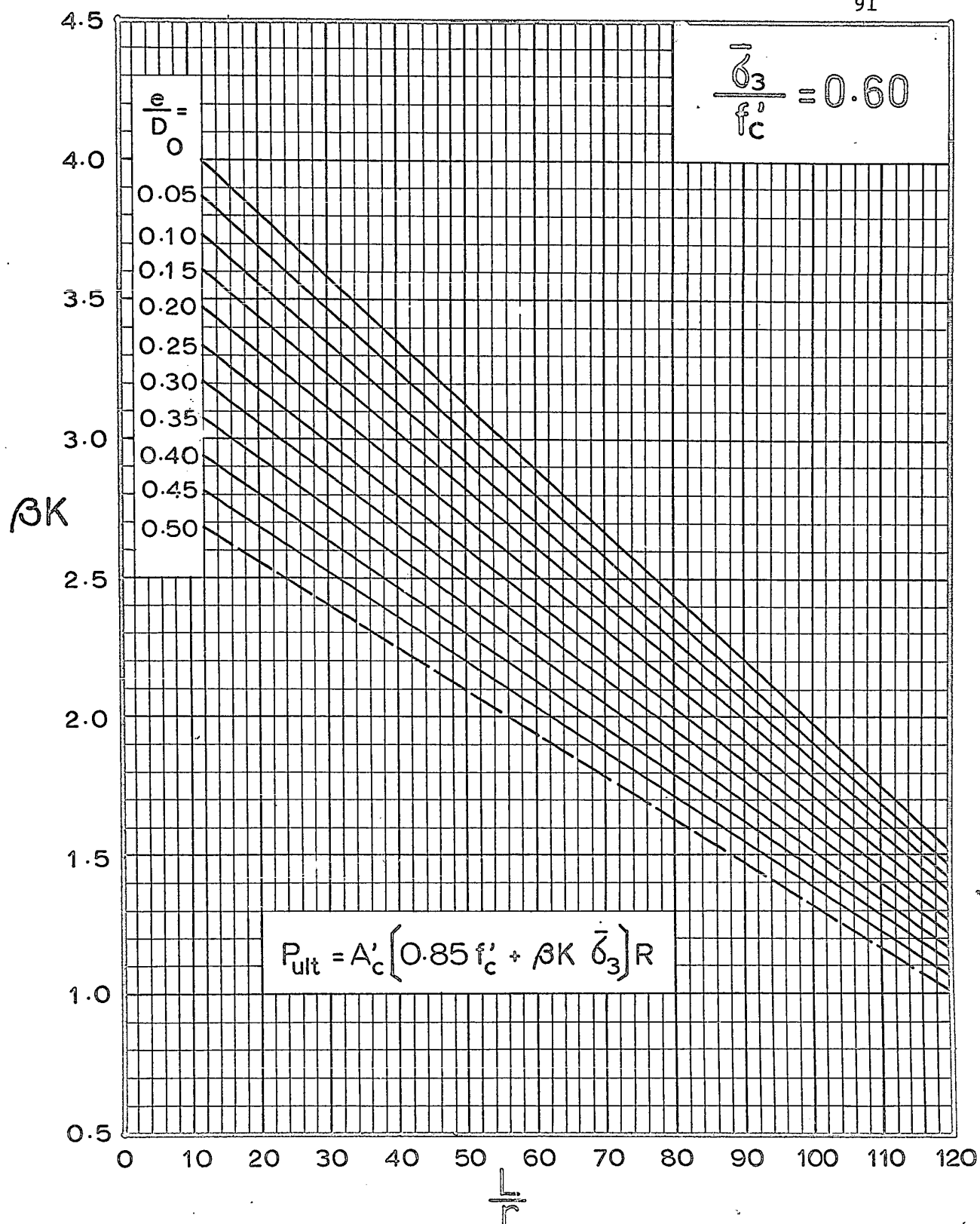


FIG. 6.8 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 0.60$

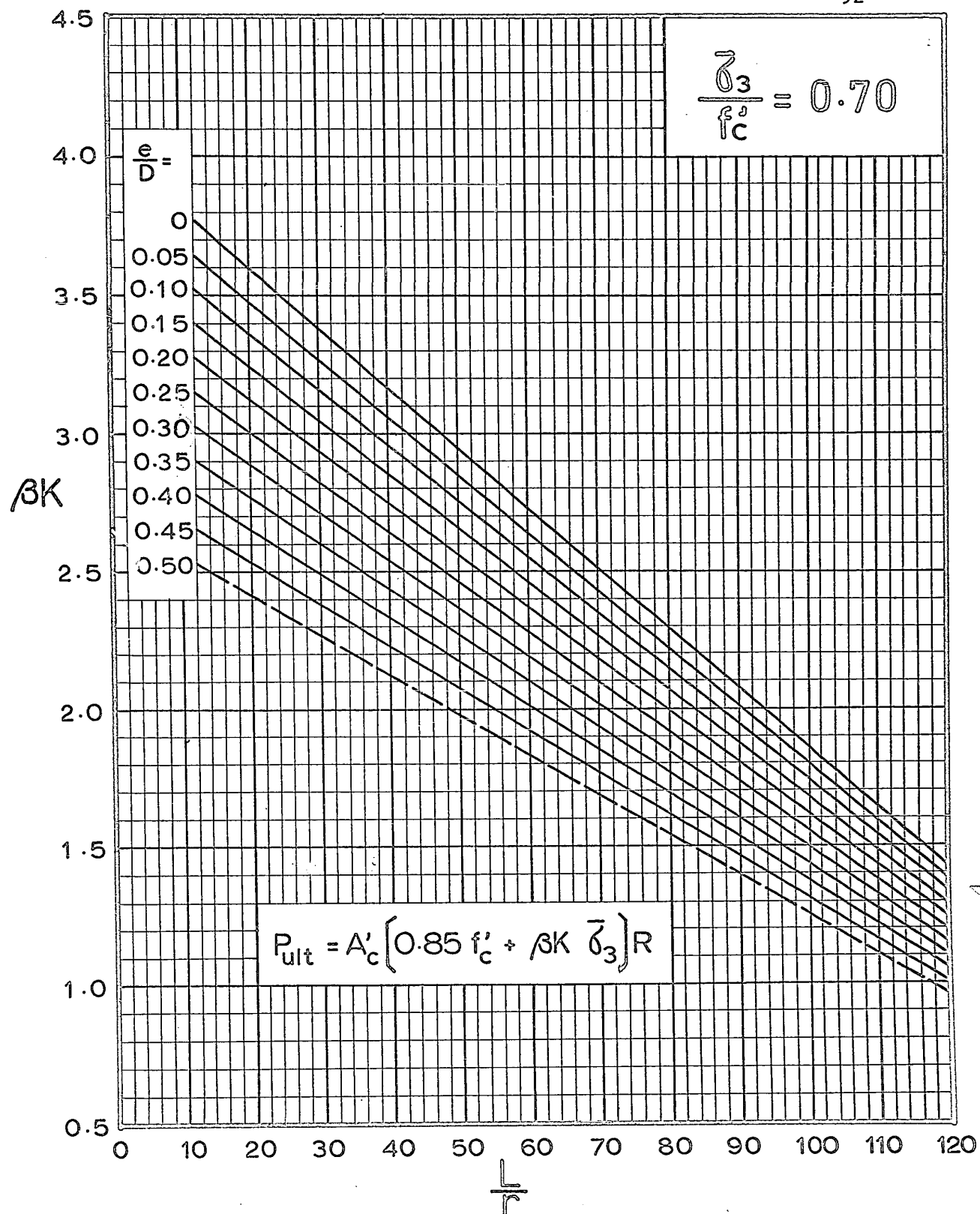


FIG. 6-9 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 0.70$

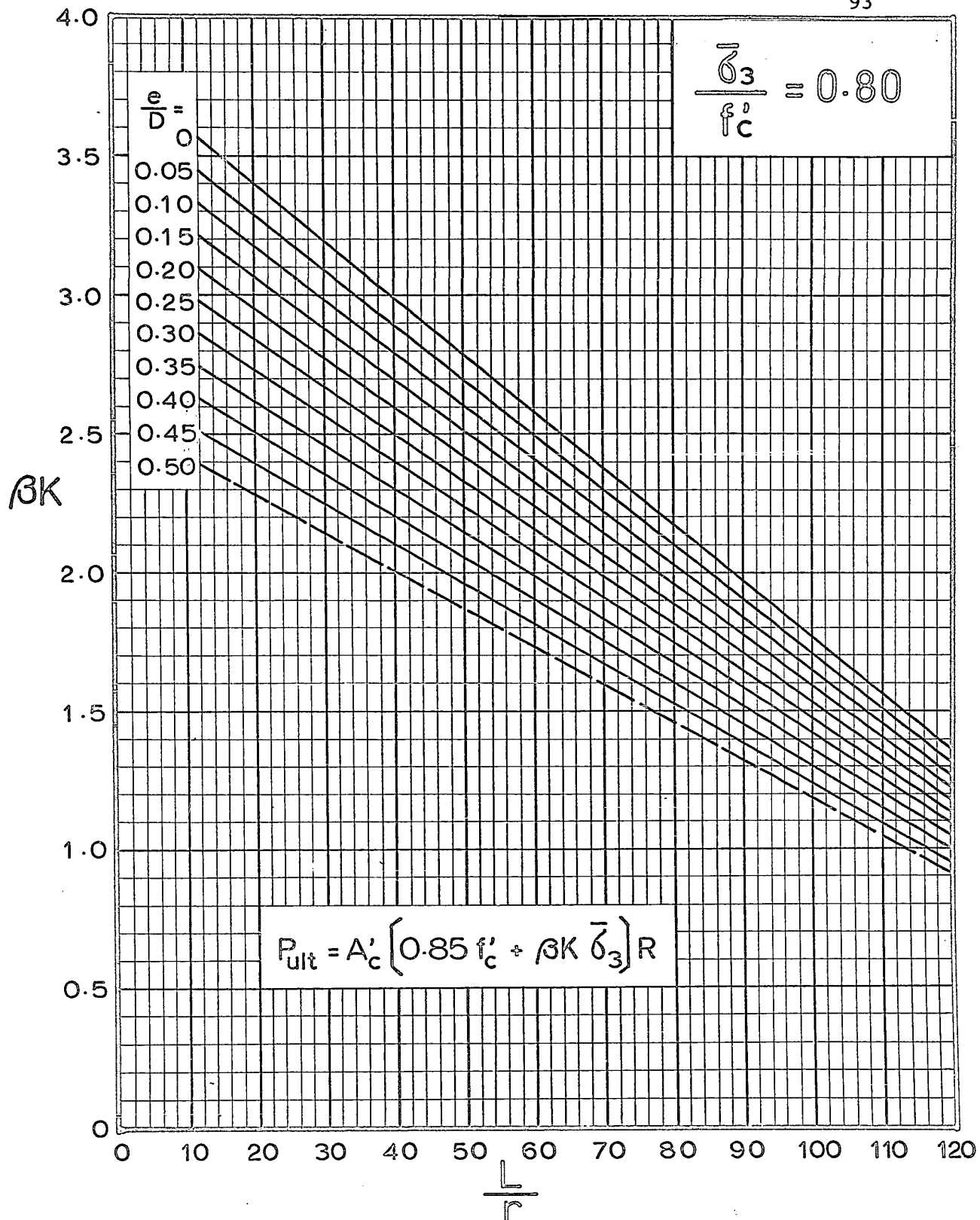


FIG. 6-10 VALUES OF βK
FOR $\frac{\delta_3}{f'_c} = 0.80$

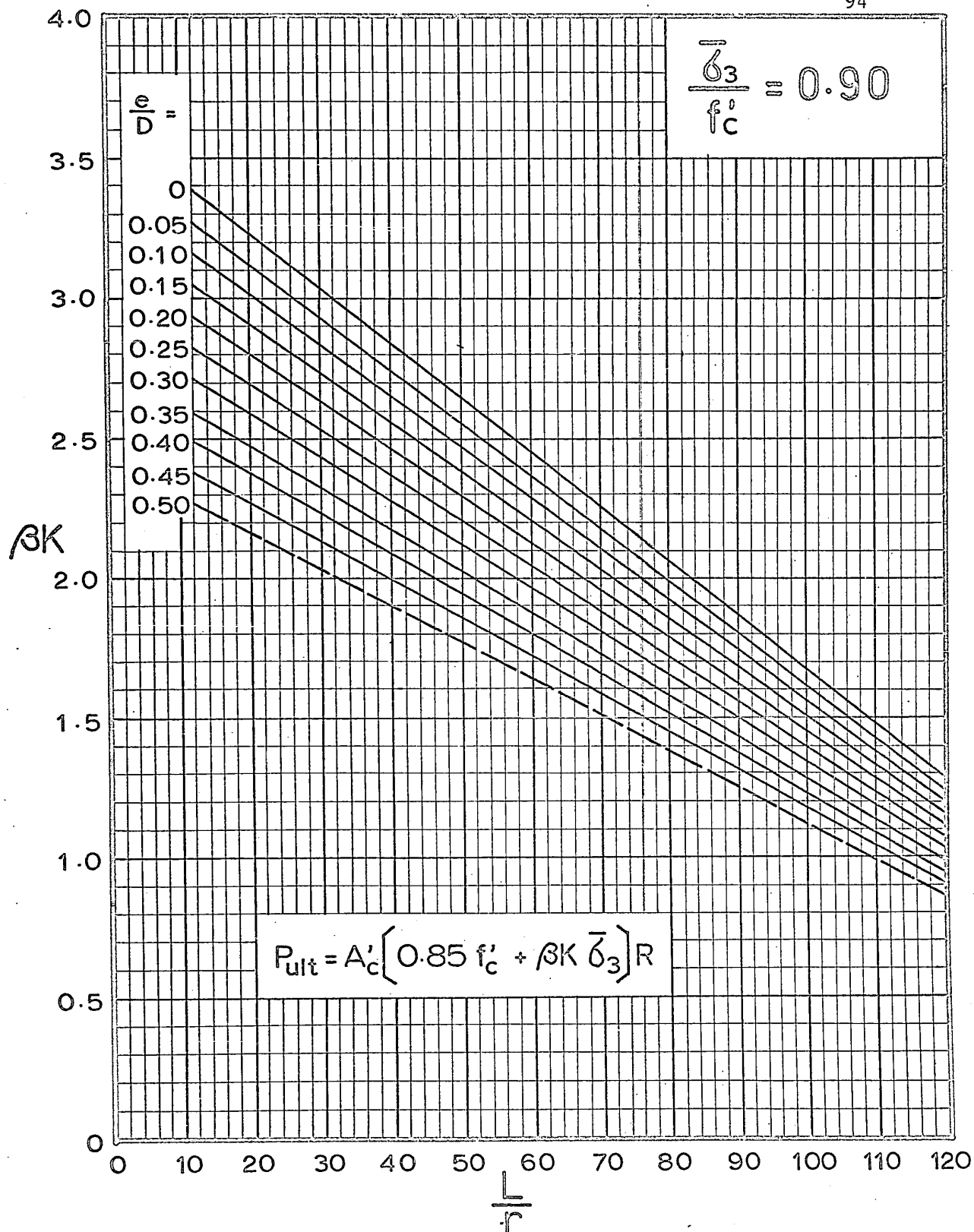


FIG. 6-11 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 0.90$

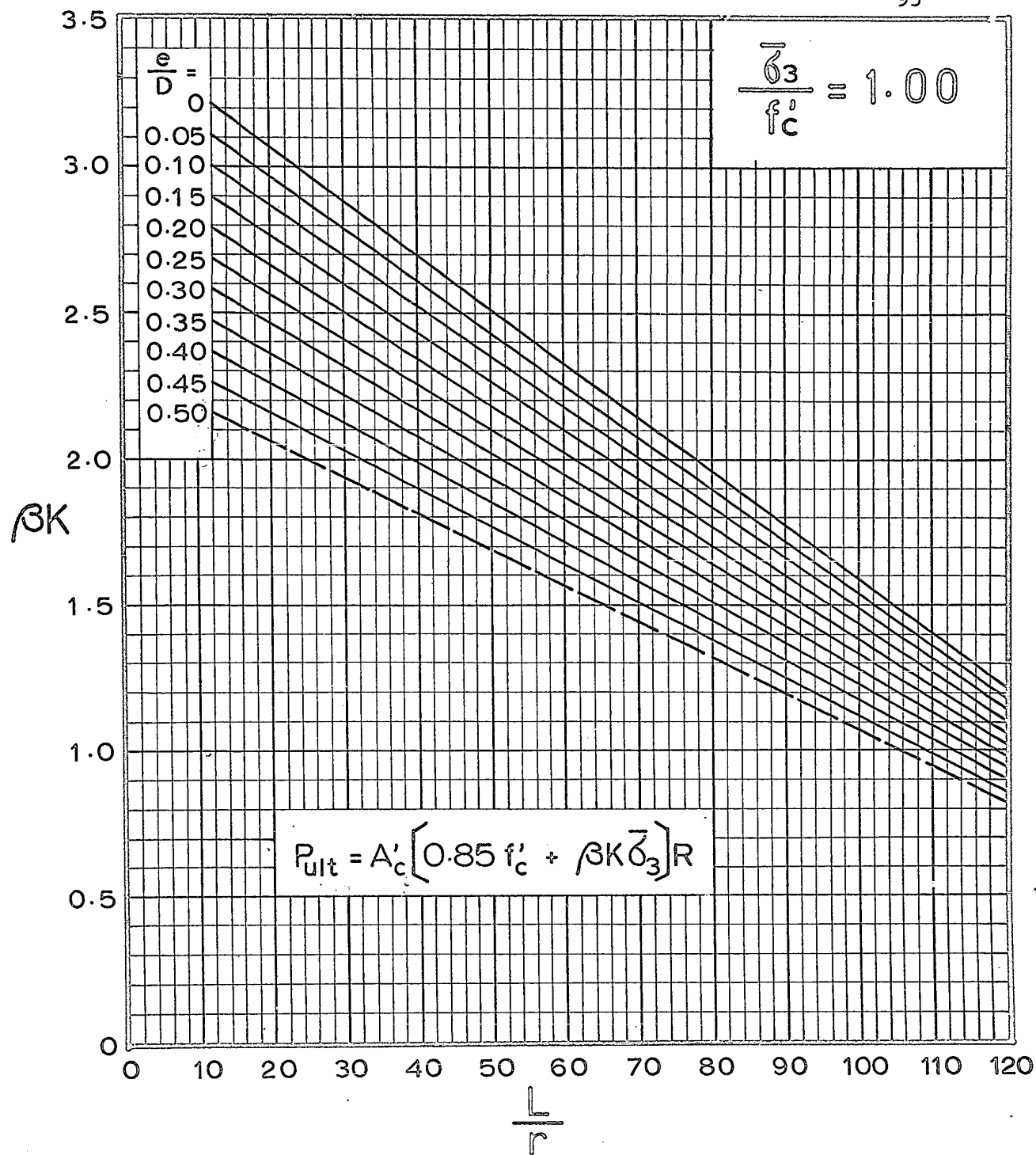


FIG. 6-12 VALUES OF βK
FOR $\frac{\bar{\delta}_3}{f'_c} = 1.00$

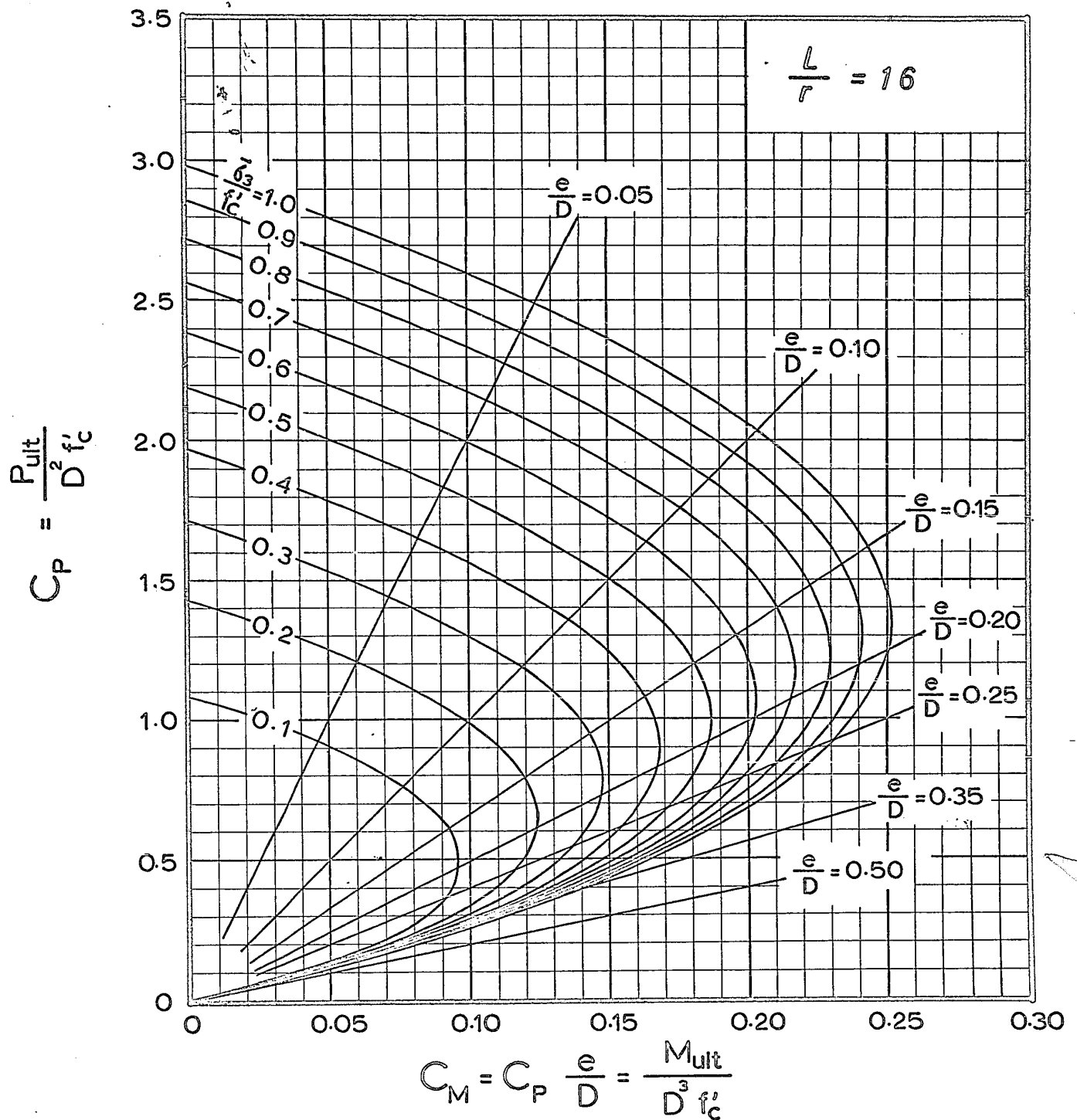


FIG. 6-13 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 16$

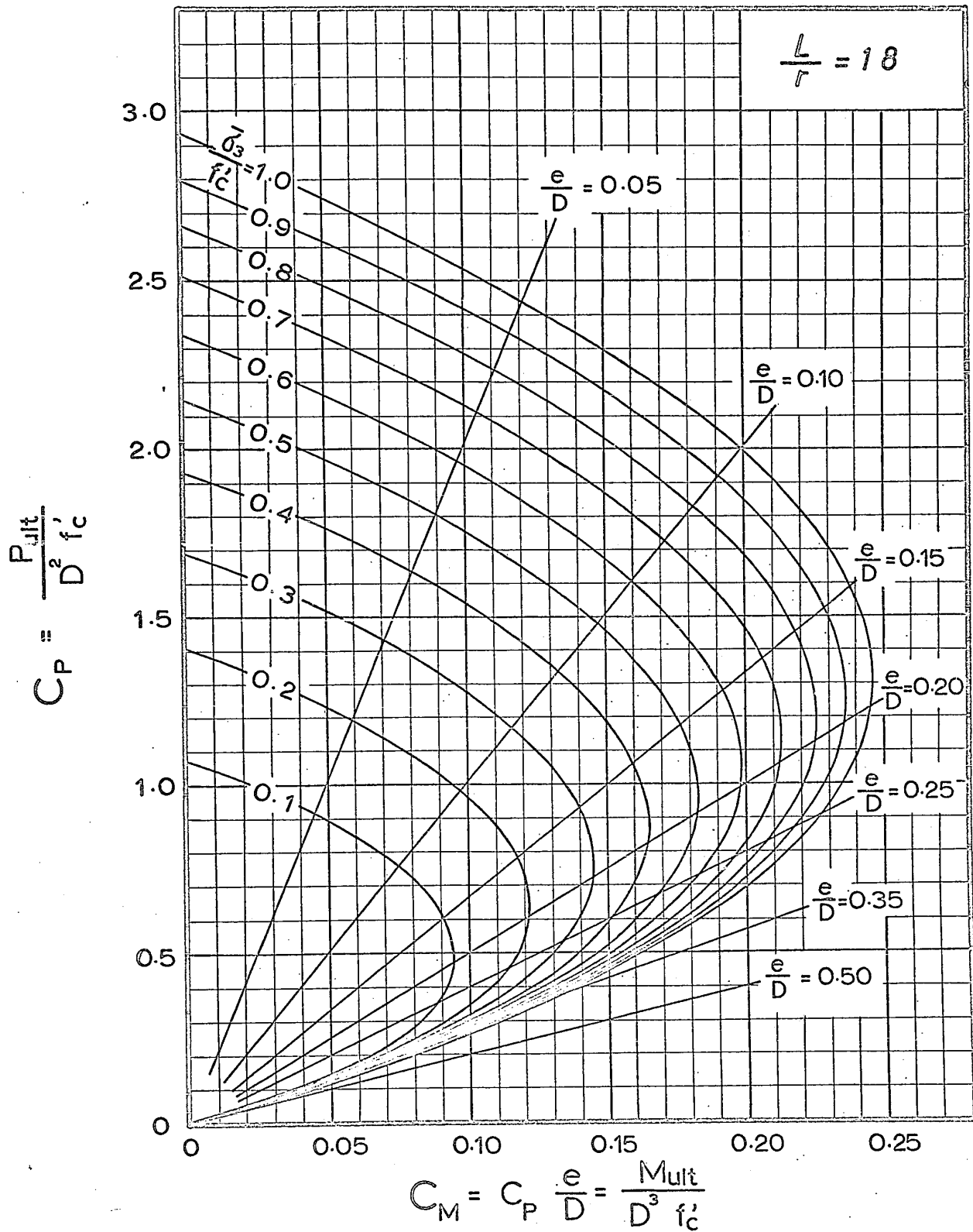


FIG. 6-14 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 18$

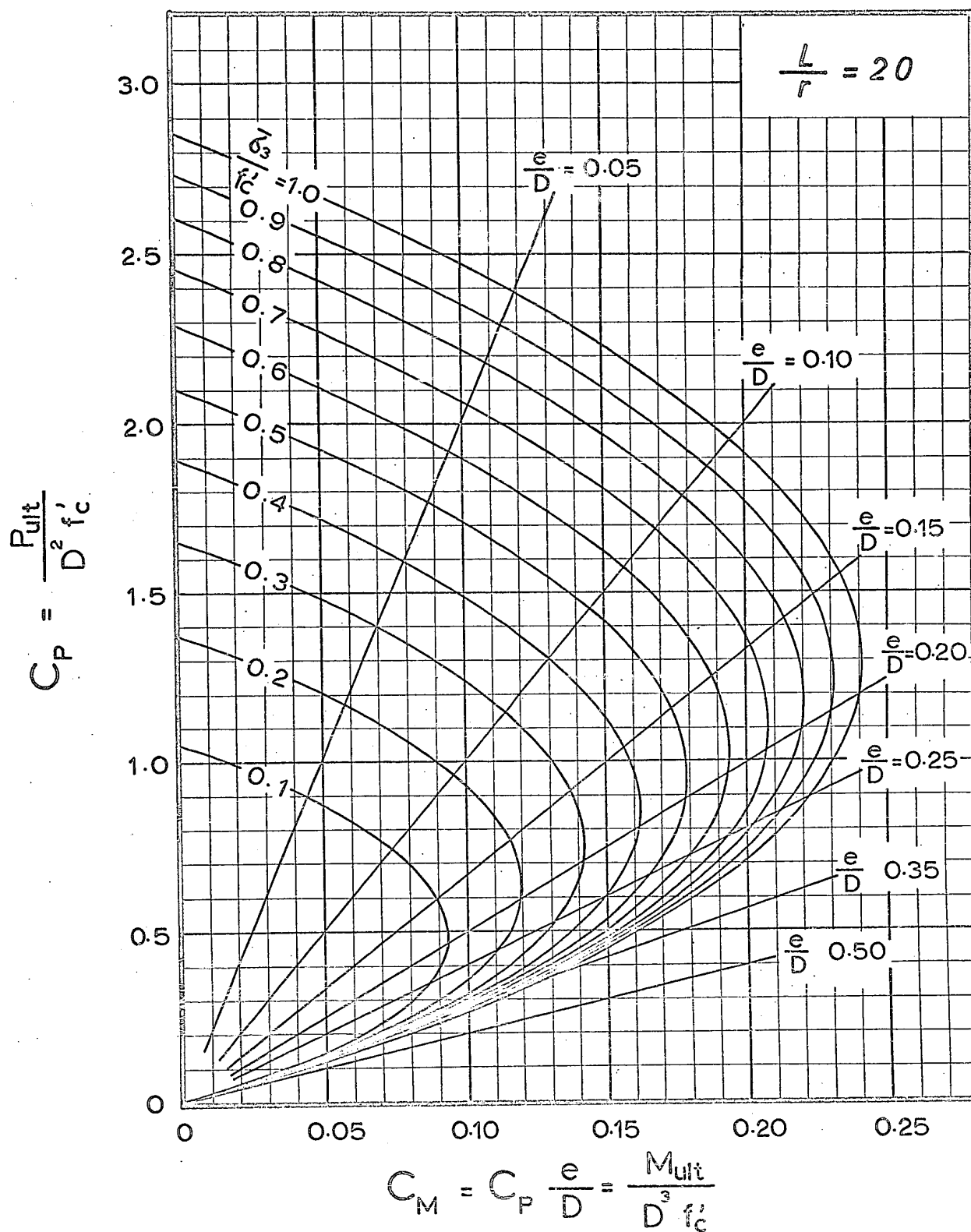


FIG. 6.15 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 20$

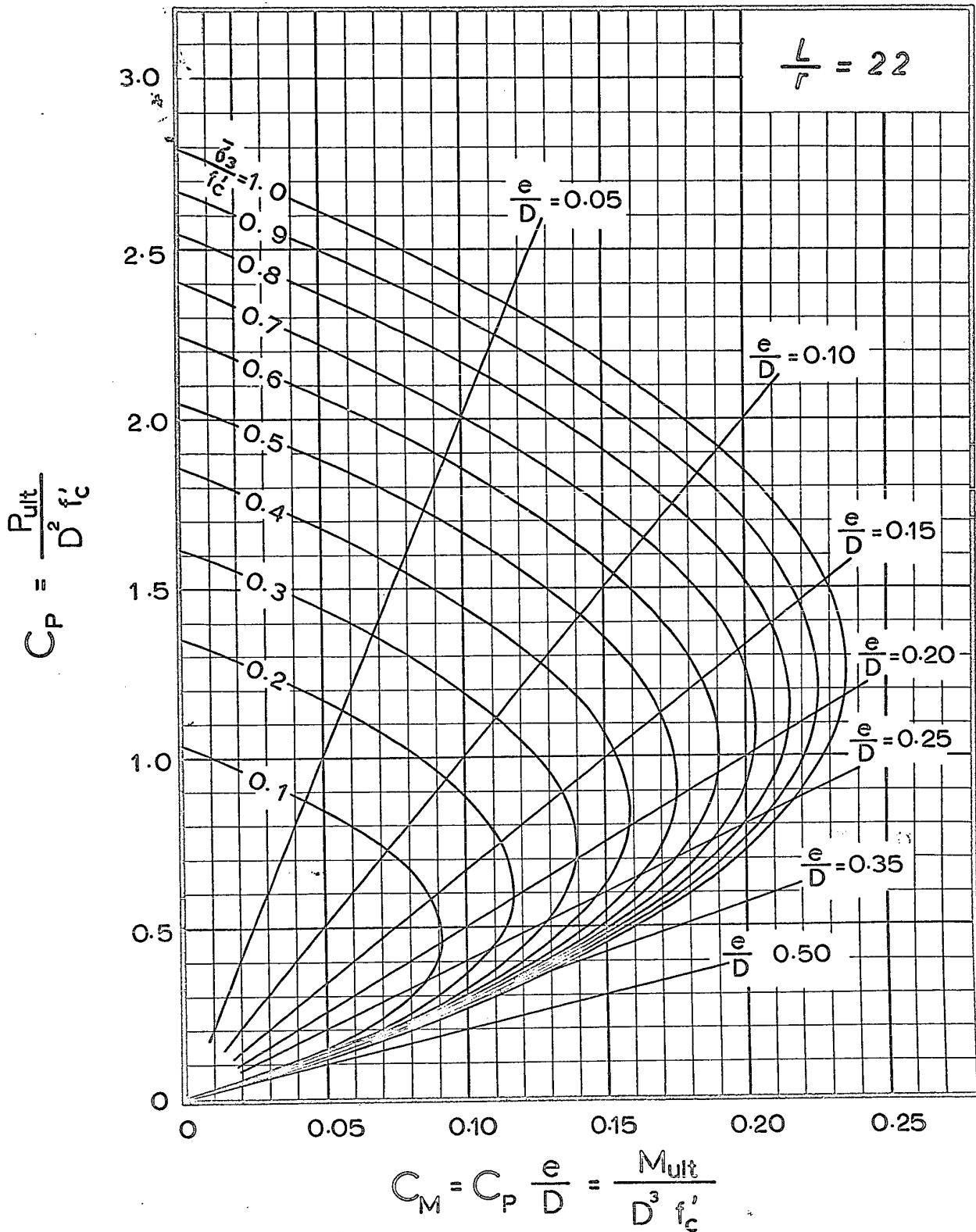


FIG. 6-16 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 22$

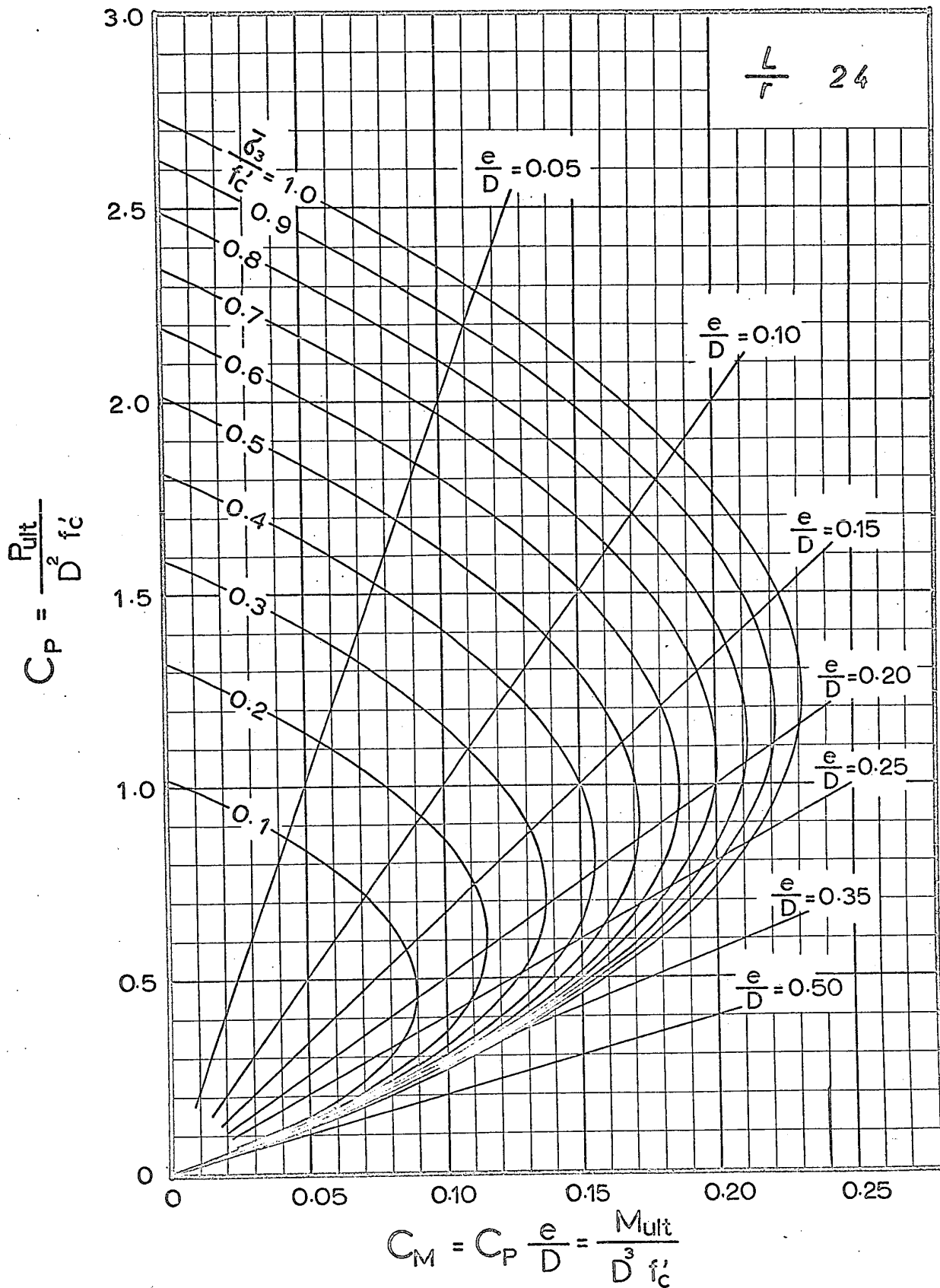


FIG. 6-17 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 24$

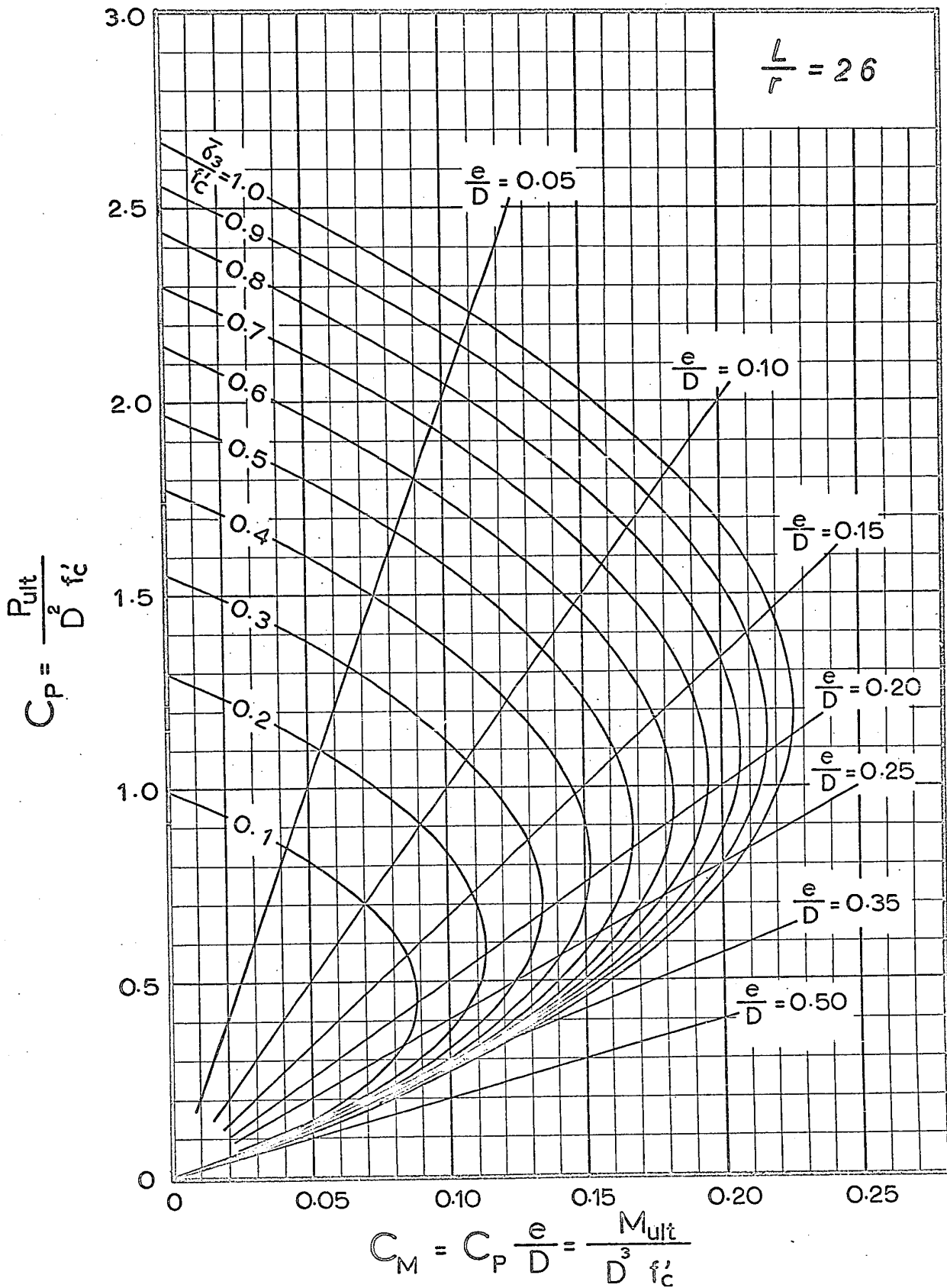


FIG. 6-18 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 26$

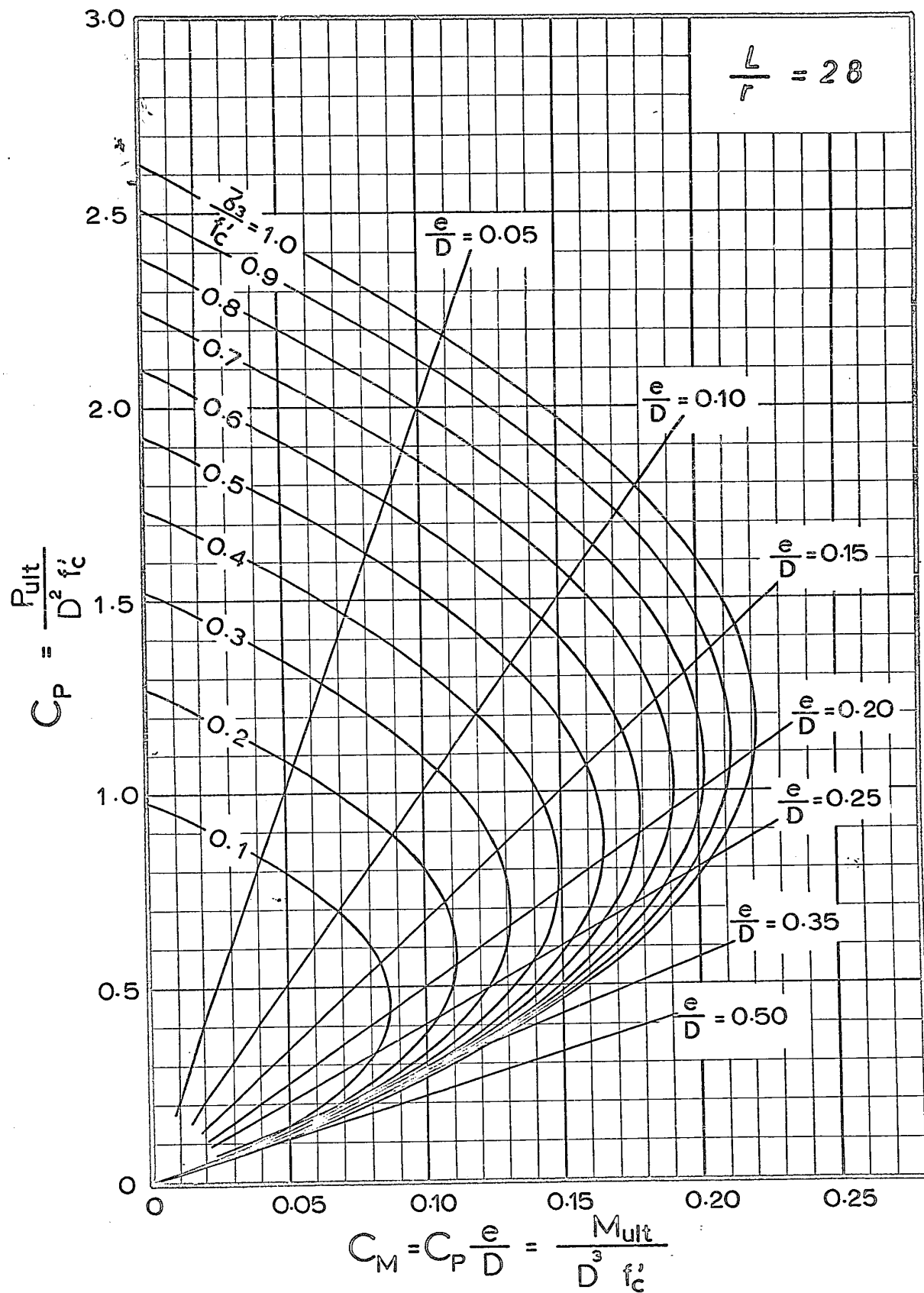


FIG. 6-19 INTERACTION DIAGRAM FOR LAT. PRESTRESSED CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 28$

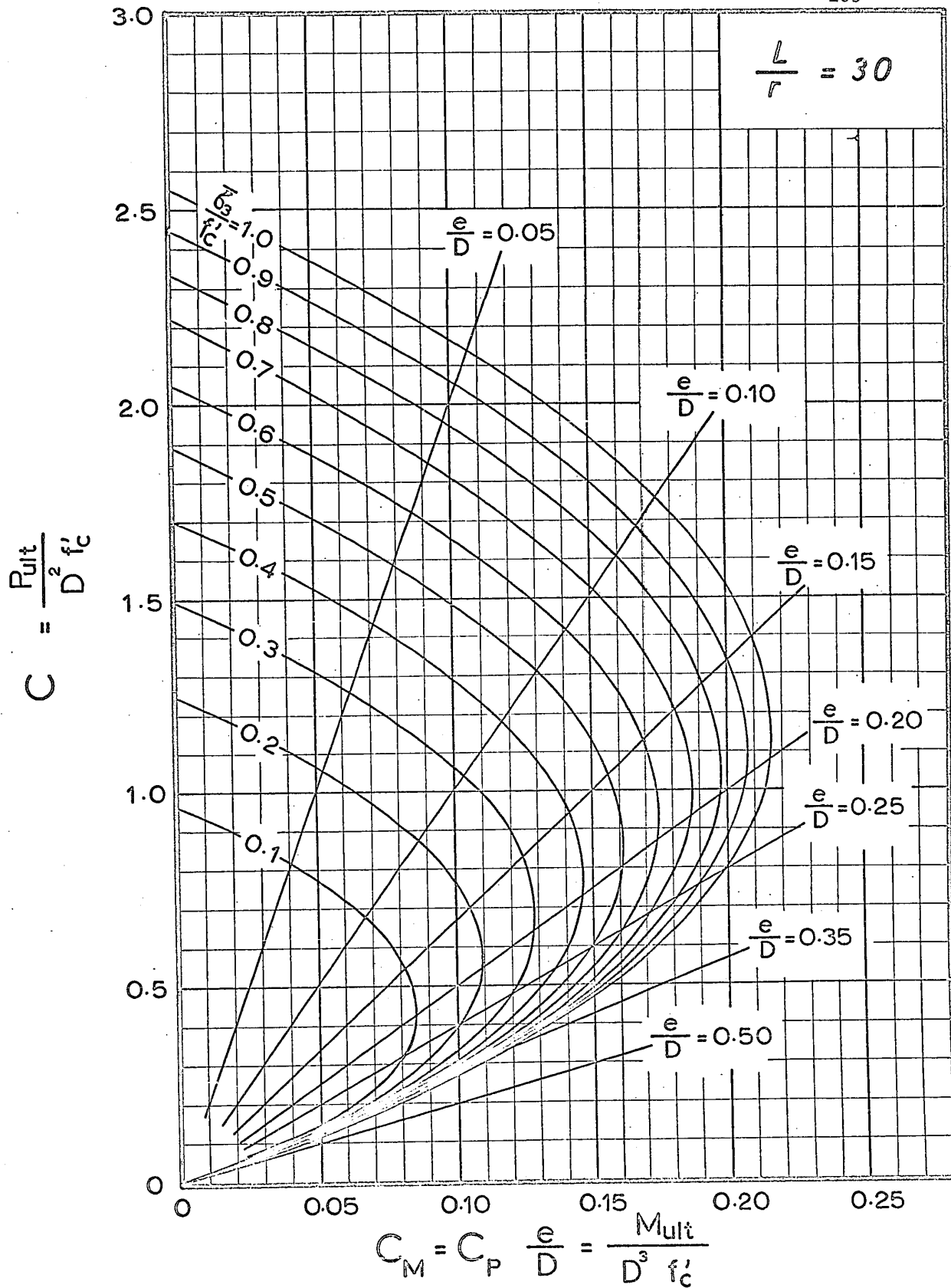


FIG. 6-20 INTERACTION DIAGRAM FOR LAT. PRESTRESSED CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 30$

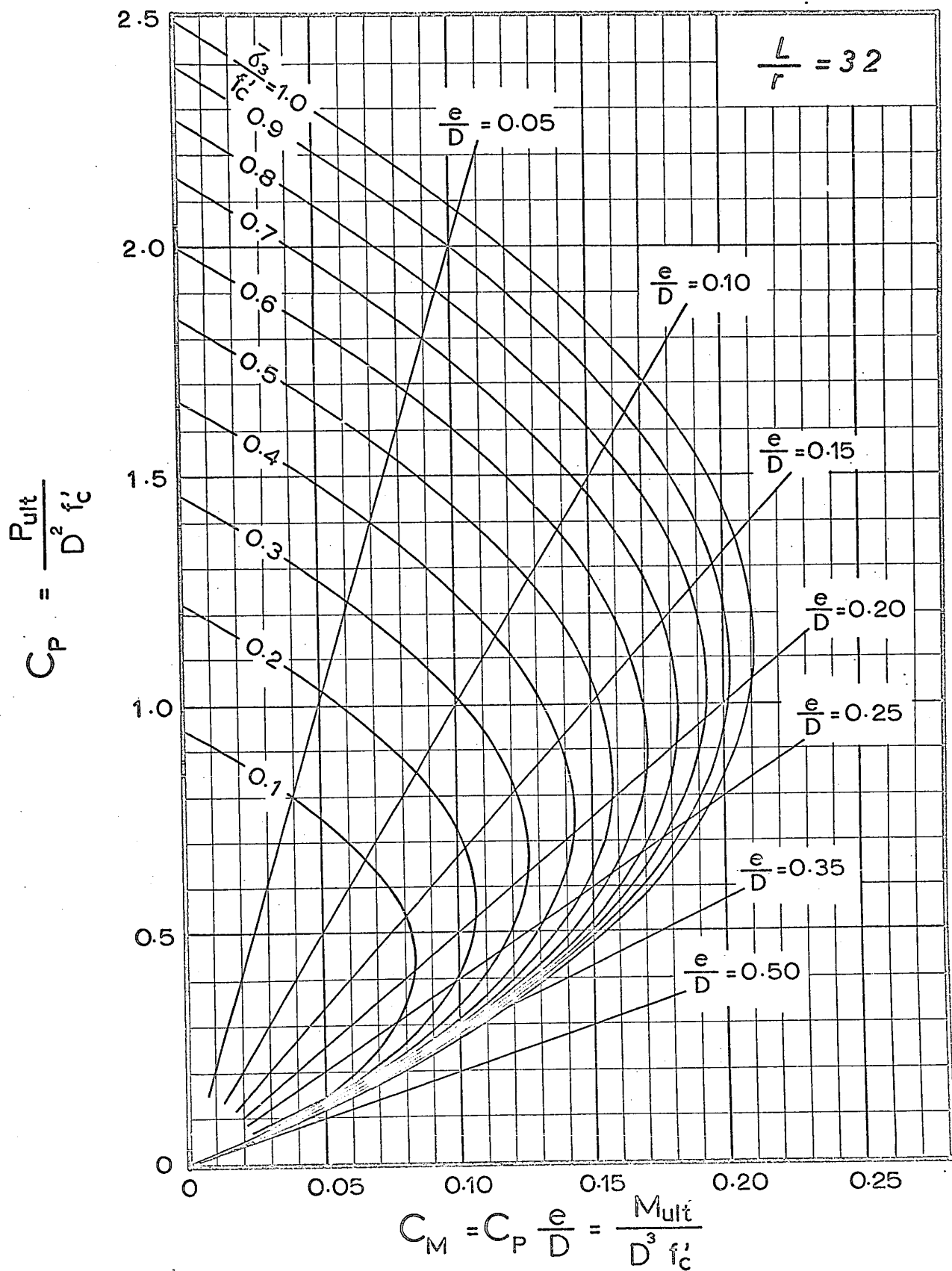


FIG. 6-21 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 32$

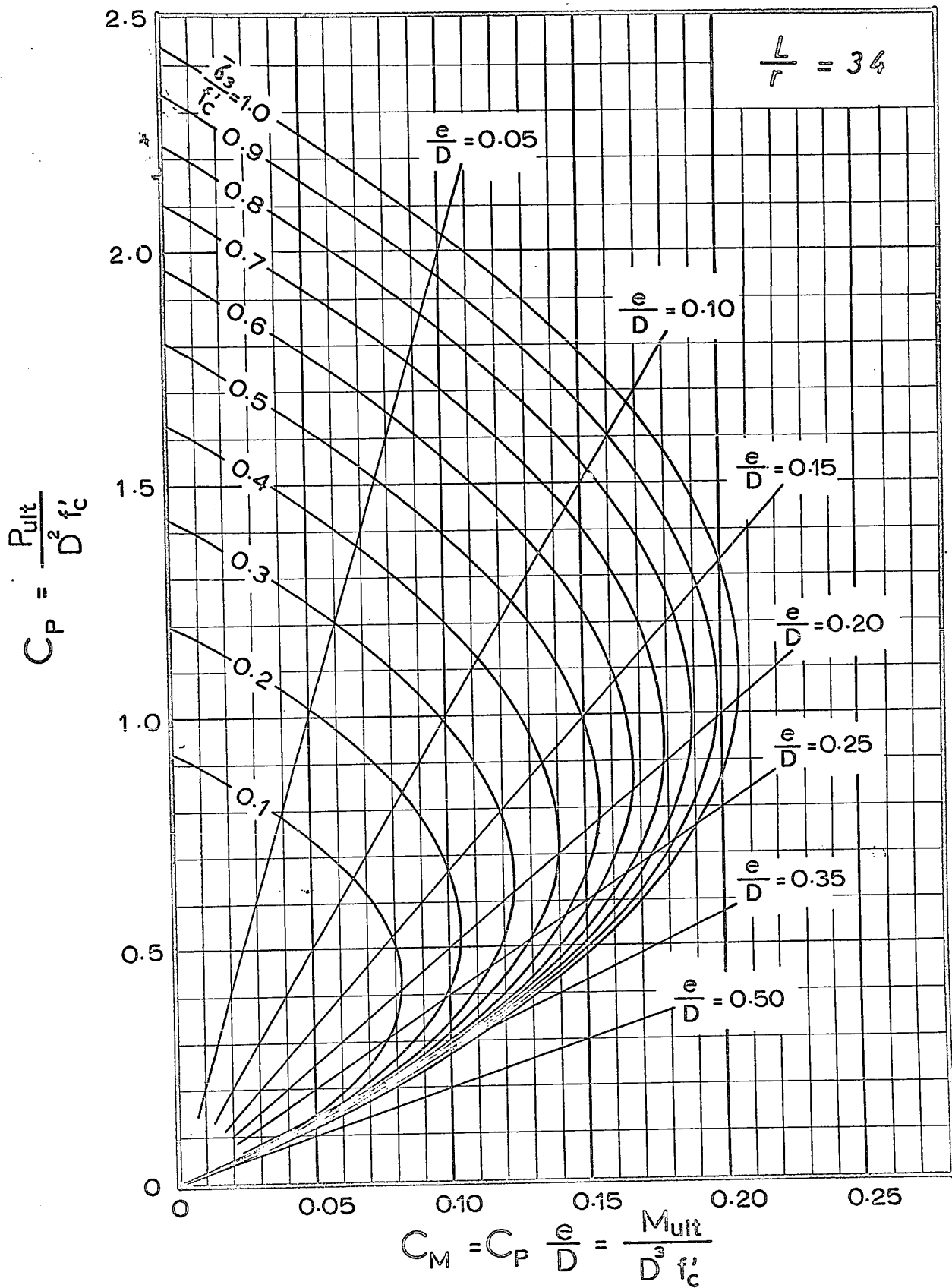


FIG. 6-22 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 34$

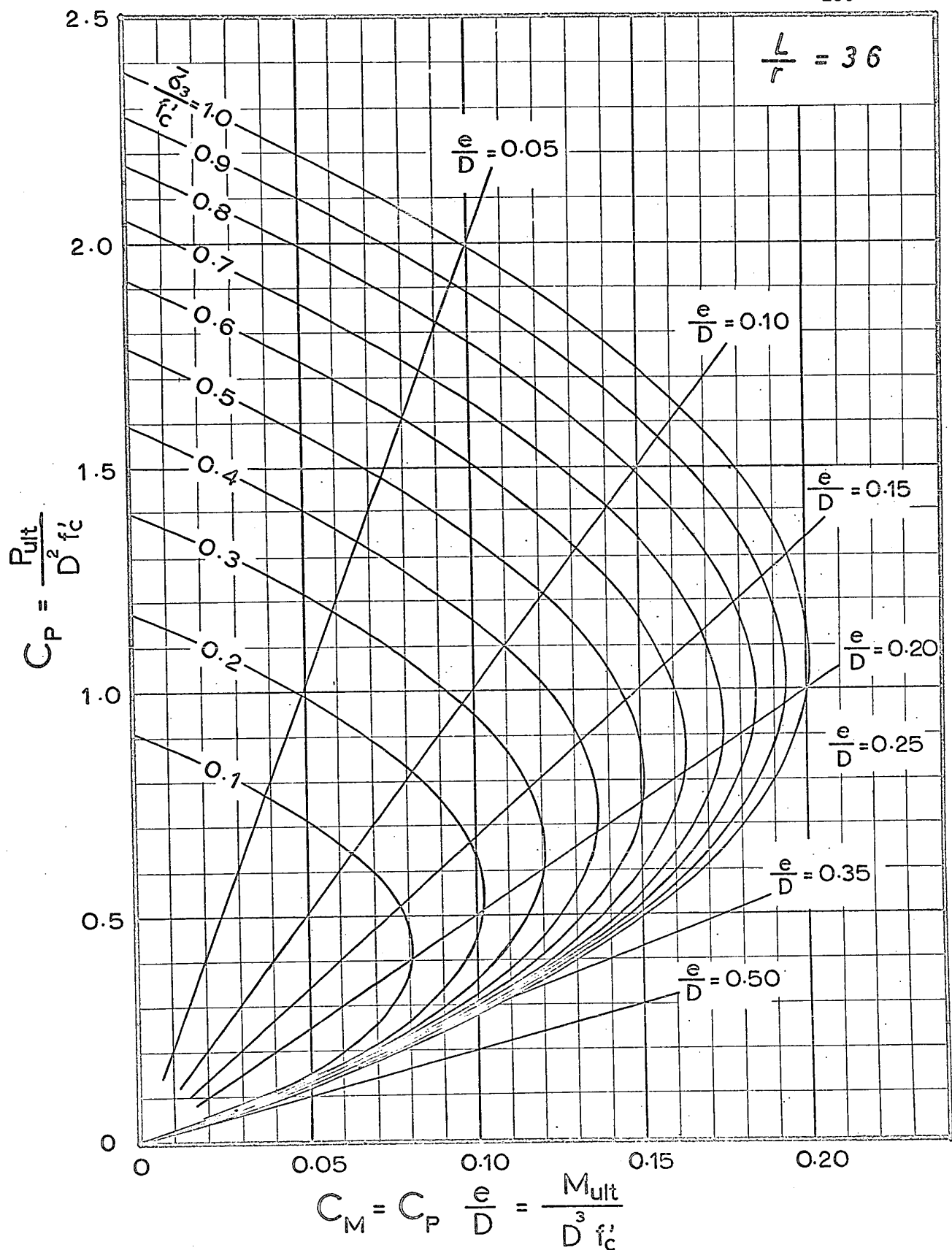


FIG. 6-23 INTERACTION DIAGRAM FOR LAT. PRESTRESSED CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 36$

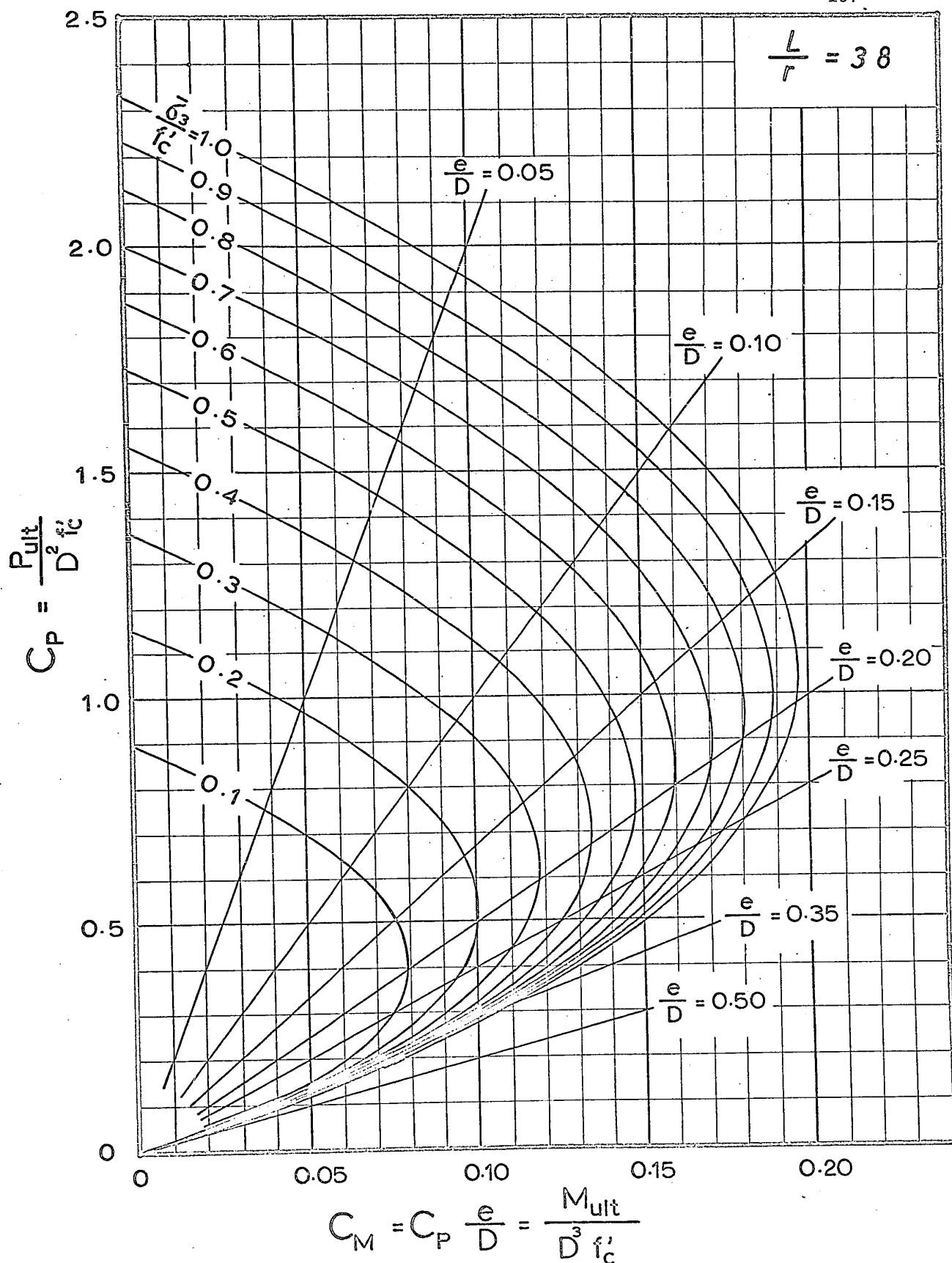


FIG. 6-24 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 38$

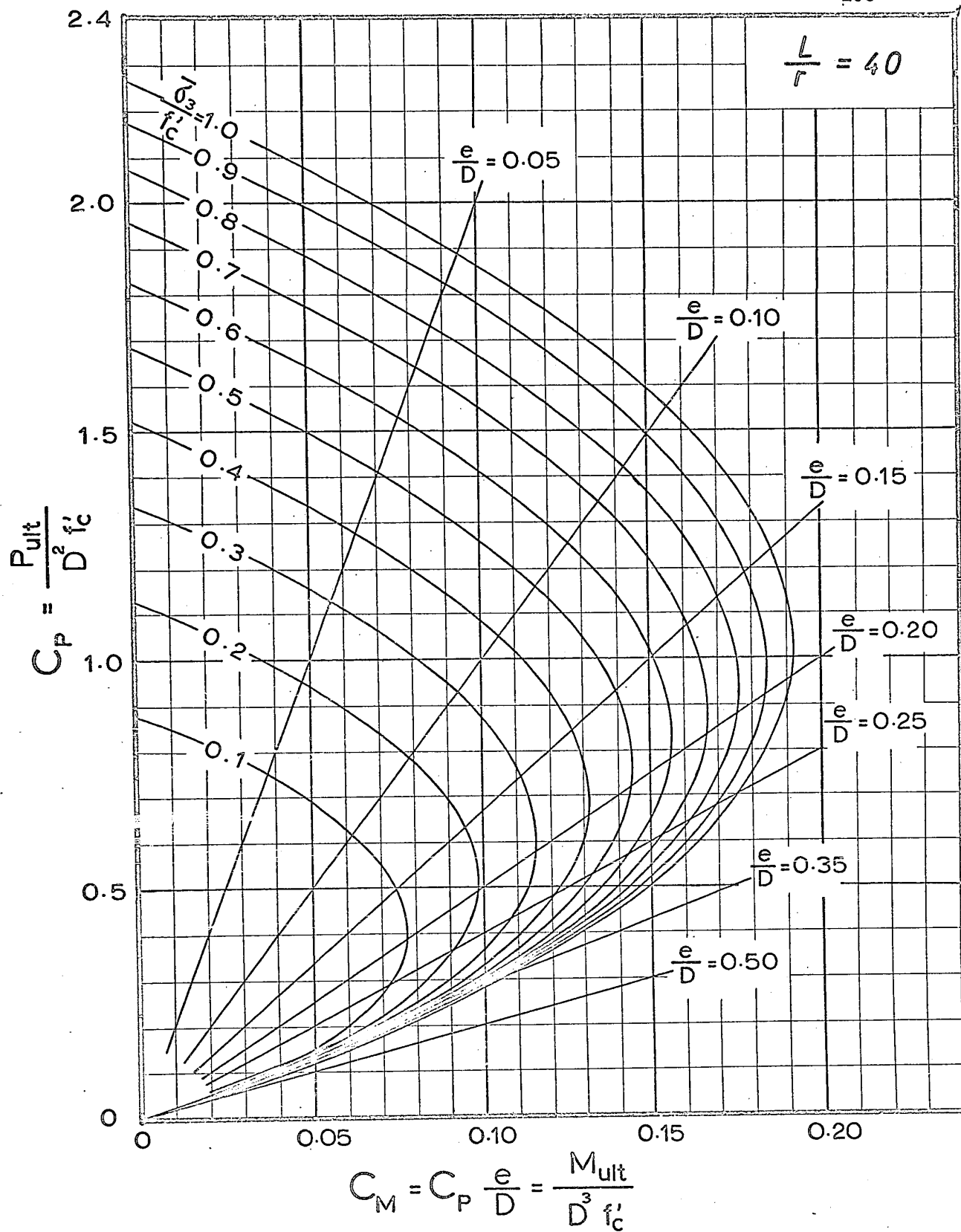


FIG. 6-25 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 40$

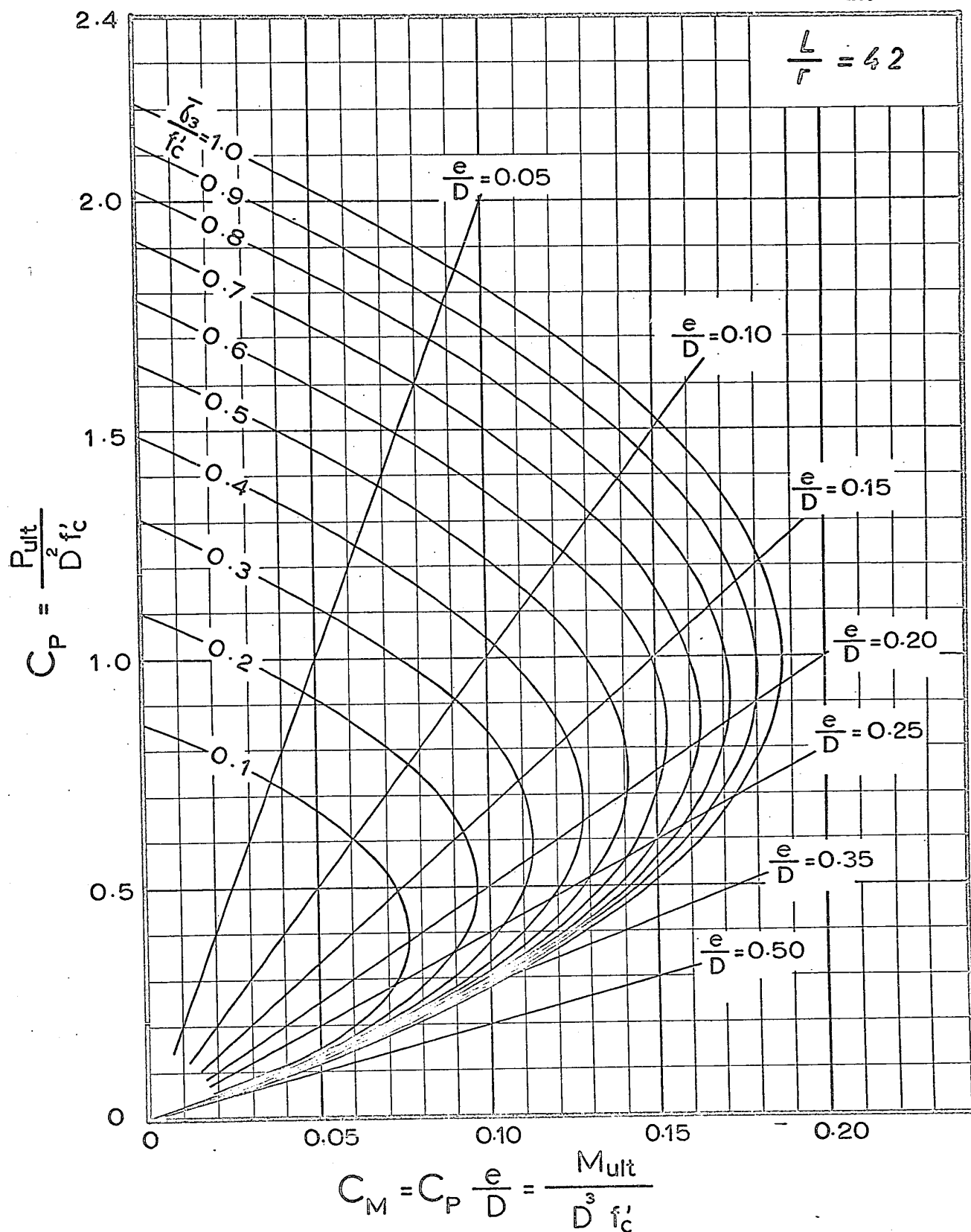


FIG. 6-26 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 42$

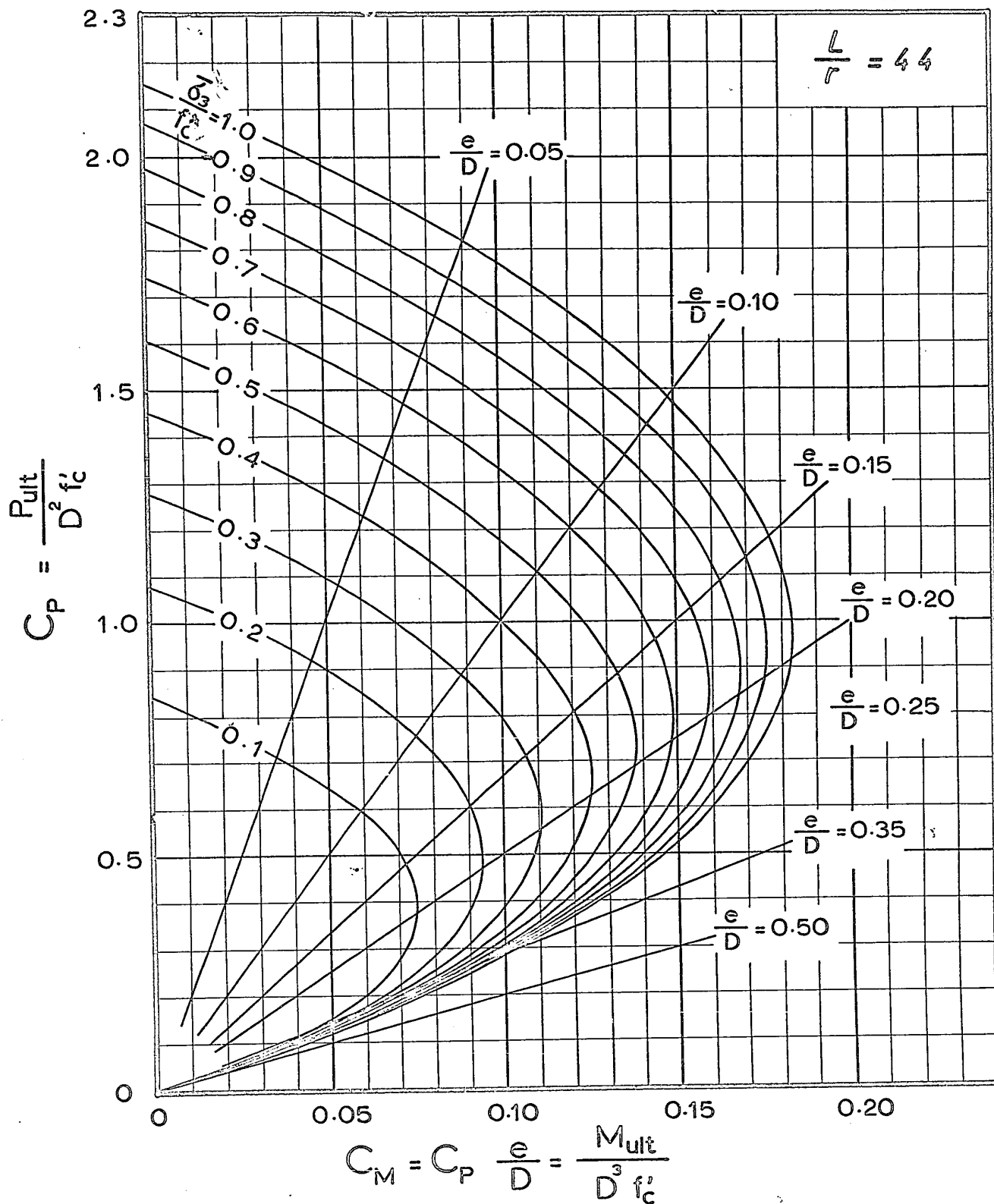


FIG. 6-27 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 44$

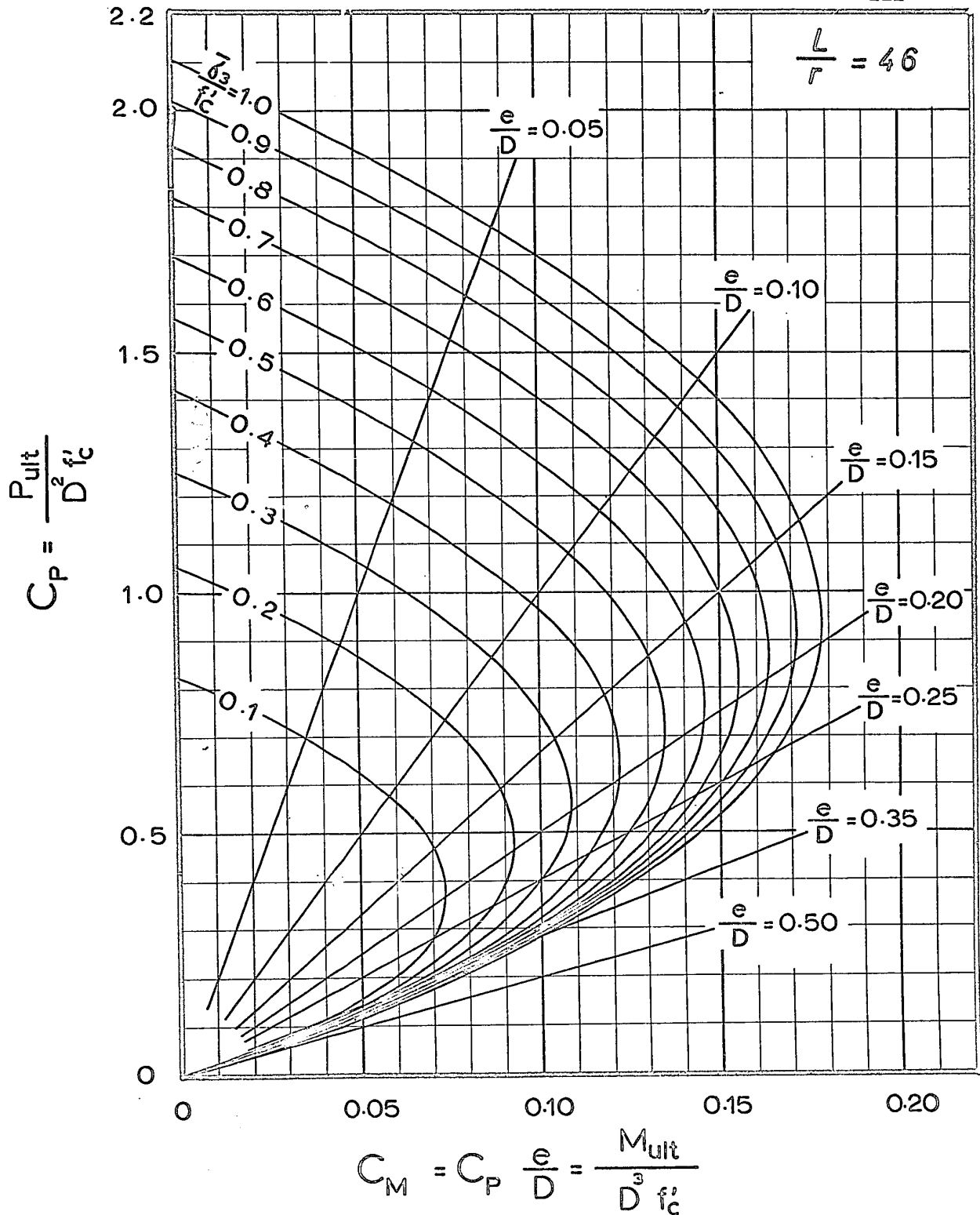


FIG. 6-28 INTERACTION DIAGRAM FOR LAT. PRESTRESSED
CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 46$

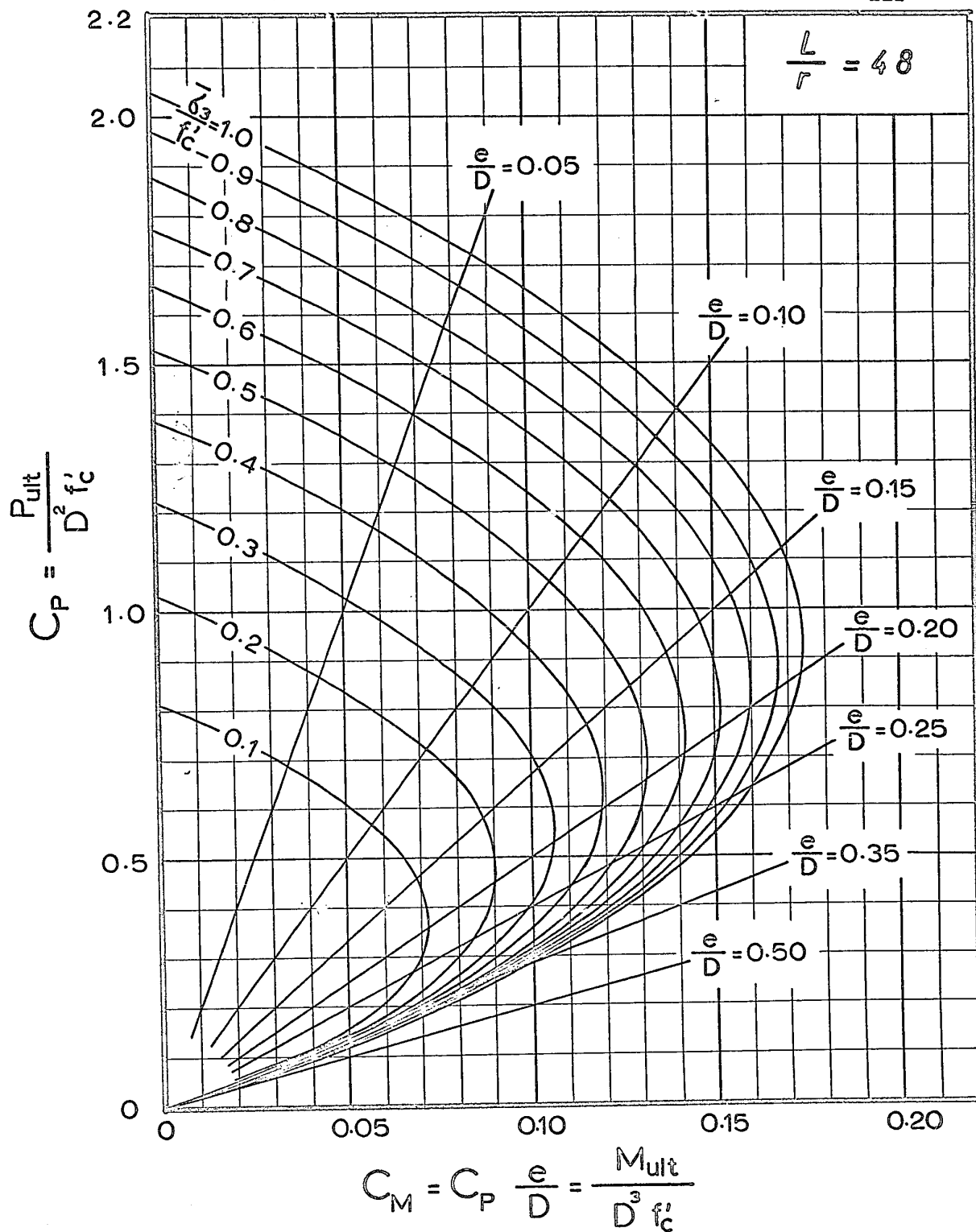


FIG. 6-29 INTERACTION DIAGRAM FOR LAT. PRESTRESSED CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 48$

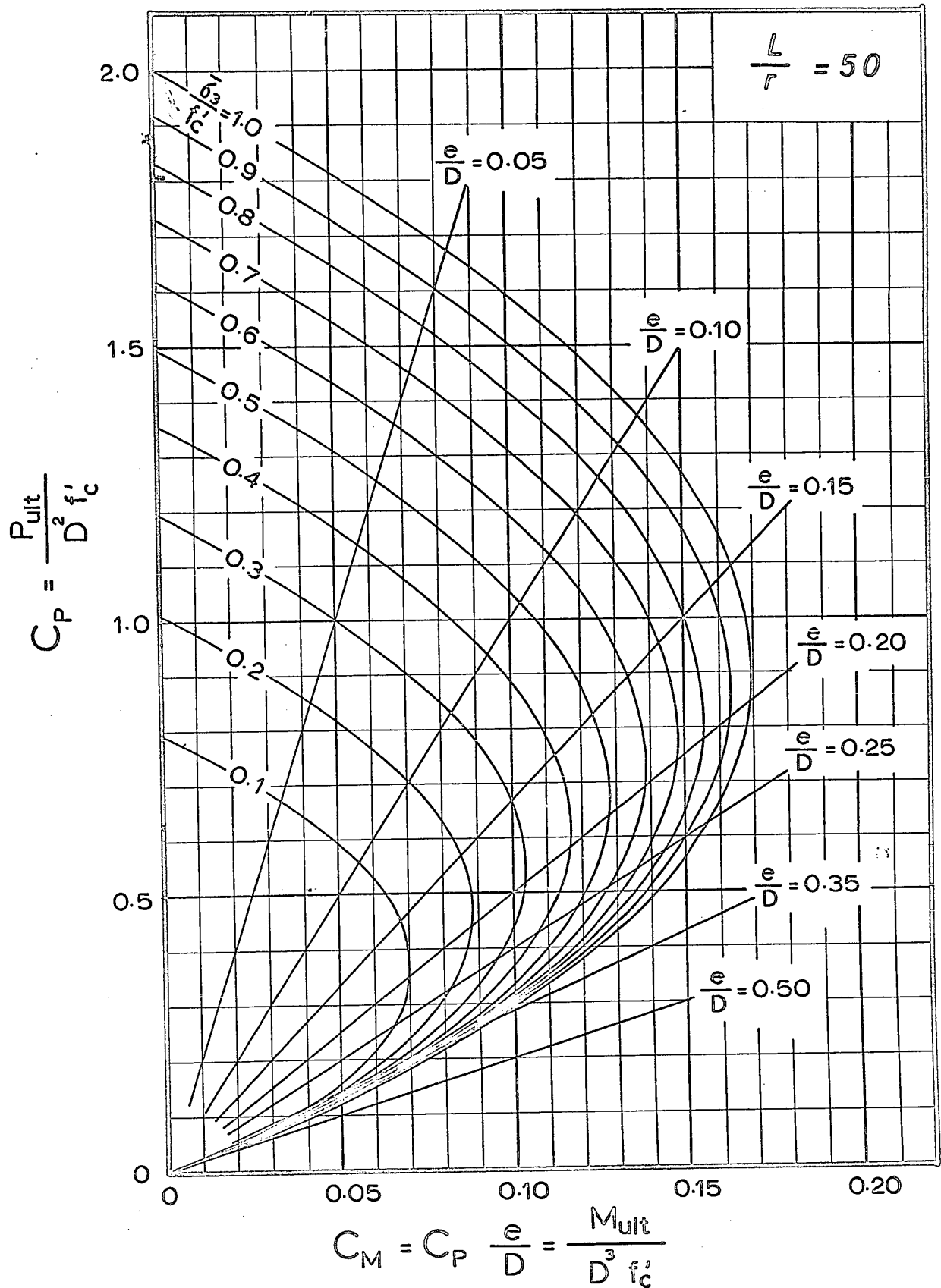


FIG. 6.30 INTERACTION DIAGRAM FOR LAT. PRESTRESSED CYLINDRICAL CONCRETE COLUMN $\frac{L}{r} = 50$

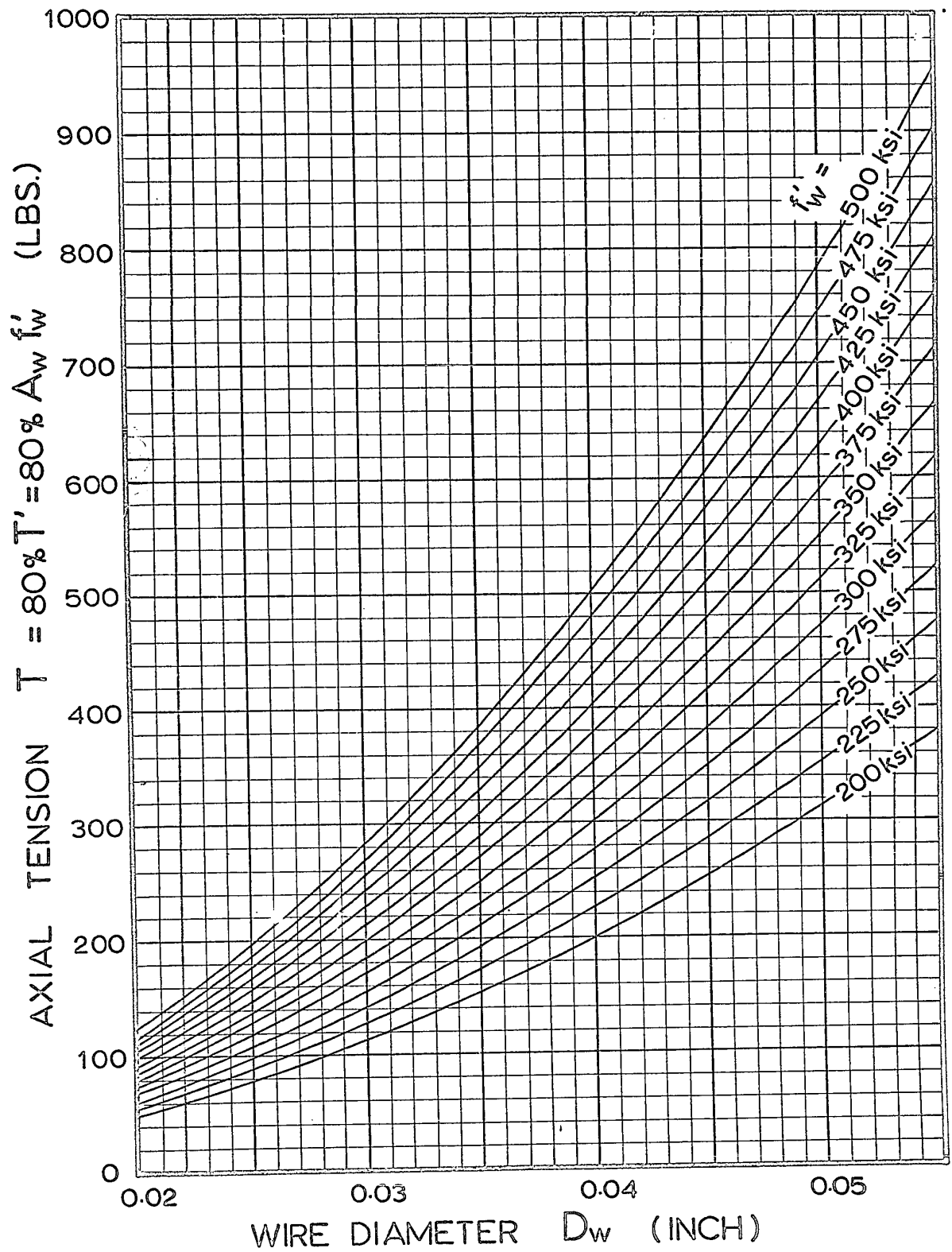


FIG. 6-31 80% ULTIMATE AXIAL FORCE
IN WRAPPING WIRE

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

From this study, several principal conclusions are made as follows :

- 1) Double (or more) wrapping is not recommended.
- 2) The load carrying capacity of concrete columns can be increased by prestressing the column by wrapping the column with wires under tension.
- 3) For cylindrical concrete columns, the relationship between the failure load, concrete crushing strength, the final lateral stress at the time of column failure, eccentricity $\frac{e}{D} < 0.50$ and slenderness ratio, can be expressed in the form of

$$P_{ult} = A'_c \left\{ 0.85f'_c + 11.6 \left[1 - 0.715 \left(\frac{\bar{\sigma}_3}{f'_c} \right)^{0.2} \right] \left[1.04 - 0.0056 \frac{L}{r} \right] \left[1 - 0.66 \frac{e}{D} \right] \bar{\sigma}_3 \right\} \\ \left\{ 1.042 - 0.0057 \frac{L}{r} \right\} \left\{ 1 - 0.35 \frac{e}{D} \right\}$$

$$\text{where } A'_c = \frac{\pi D^2}{4} - 2e \sqrt{\frac{D^2}{4} - e^2} - \frac{D^2}{2} \sin^{-1} \left(\frac{2e}{D} \right)$$

- 4) Failure of wrapped columns is governed by the maximum lateral stress $\bar{\sigma}_3$ developed in the wires just before the column fails. The magnitude of the maximum lateral $\bar{\sigma}_3$ is bounded between initial lateral prestress σ_3 and the ultimate lateral stress

σ'_3 , which disappears when the wire breaks.

- 5) For columns of small slenderness ratio and small eccentricity, columns will fail by both the crushing of the concrete and rupture of the wires simultaneously. The maximum lateral stress $\bar{\sigma}_3$ is then equal to the ultimate lateral stress σ'_3 .
- 6) For columns of large slenderness ratio or large eccentricity (for $\frac{e}{D} < 0.50$), columns will fail by the crushing of the concrete only, without the rupture of wires. The maximum lateral stress $\bar{\sigma}_3$ in between initial lateral prestress σ_3 and ultimate lateral stress σ'_3 is then unpredictable.
- 7) The lateral prestressing does not increase the tensile strength of concrete.

7.2 Recommendations for Future Work

- 1) To study analytically the effect of initial lateral prestress σ_3 and the ultimate lateral stress σ'_3 on the maximum lateral stress $\bar{\sigma}_3$ at the time of column failure, for the laterally prestressed concrete column under concentric and eccentric loads.
(i.e. To study the relationship of $\bar{\sigma}_3$, σ_3 , σ'_3 , $\frac{L}{r}$ and $\frac{e}{D}$, in order to express the unknown $\bar{\sigma}_3$ in terms of the known values of σ_3 , σ'_3 , $\frac{L}{r}$ and $\frac{e}{D}$ in the formula.)
- 2) To study the effect of lateral prestress on the reinforced concrete columns under eccentric loading.
- 3) Study of long-term loading and creep behaviour of laterally prestressed columns.
- 4) To study the effect of the size and the pitch of wrapping wires on the failure load of wrapped columns.

REFERENCES

Following abbreviations have been used:

- ACI : American Concrete Institute.
- ASCE : American Society of Civil Engineers.
- ASTM : American Society of Testing Materials.
- ENR : Engineering News Record.
- MCR : Magazine of Concrete Research.
- NRC : National Research Council of Canada.

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APPENDIXES

APPENDIX A Steel Properties and Calibration Curve

APPENDIX B Individual Graphs of Testing Data of Specimens

APPENDIX C Experimental Failure Loads of All Specimens

APPENDIX A

Steel Properties and Calibration Curve

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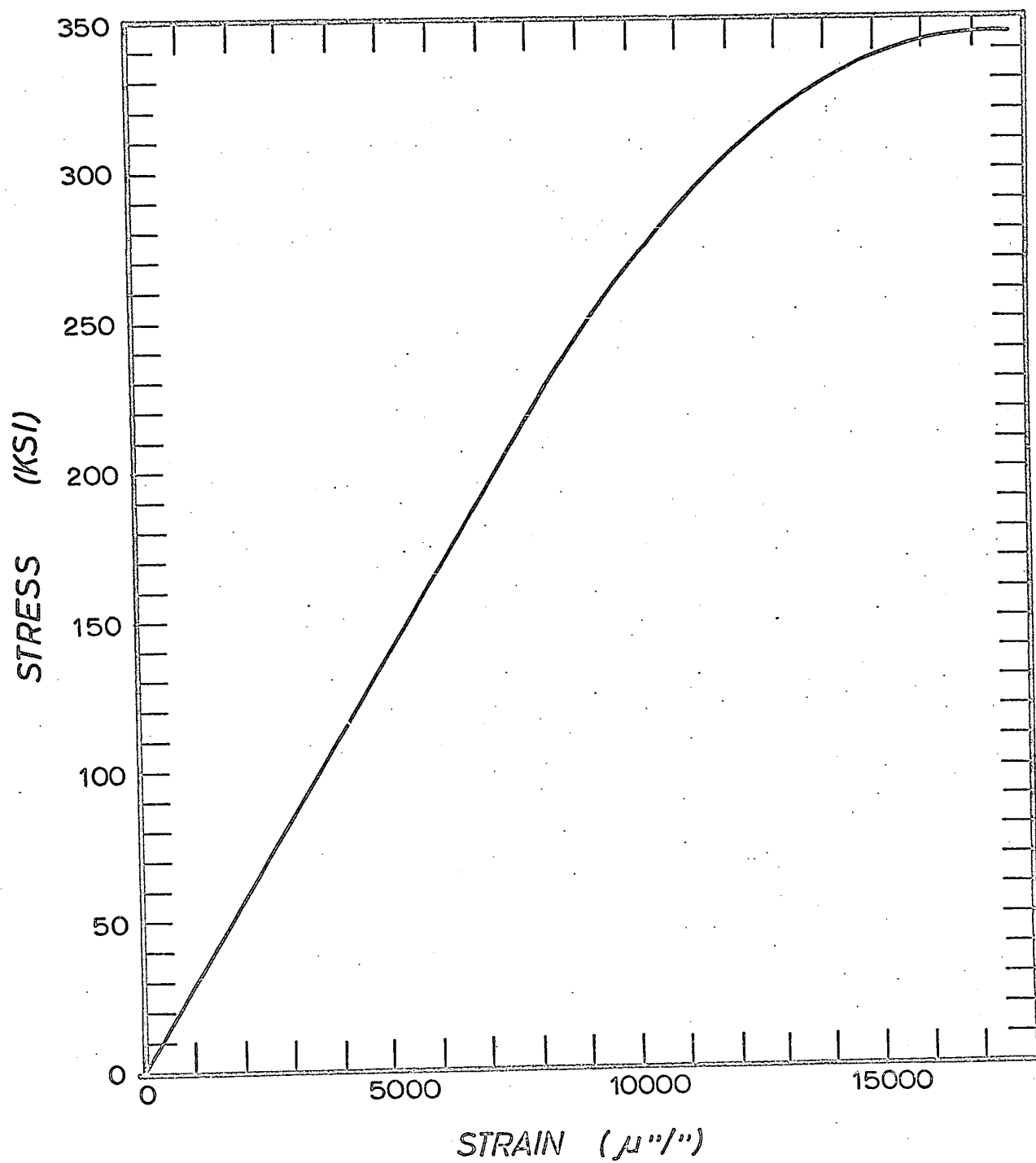


FIG. A-1 STRESS-STRAIN CHARACTERISTIC OF
0.031" DIAMETER STEEL WIRE

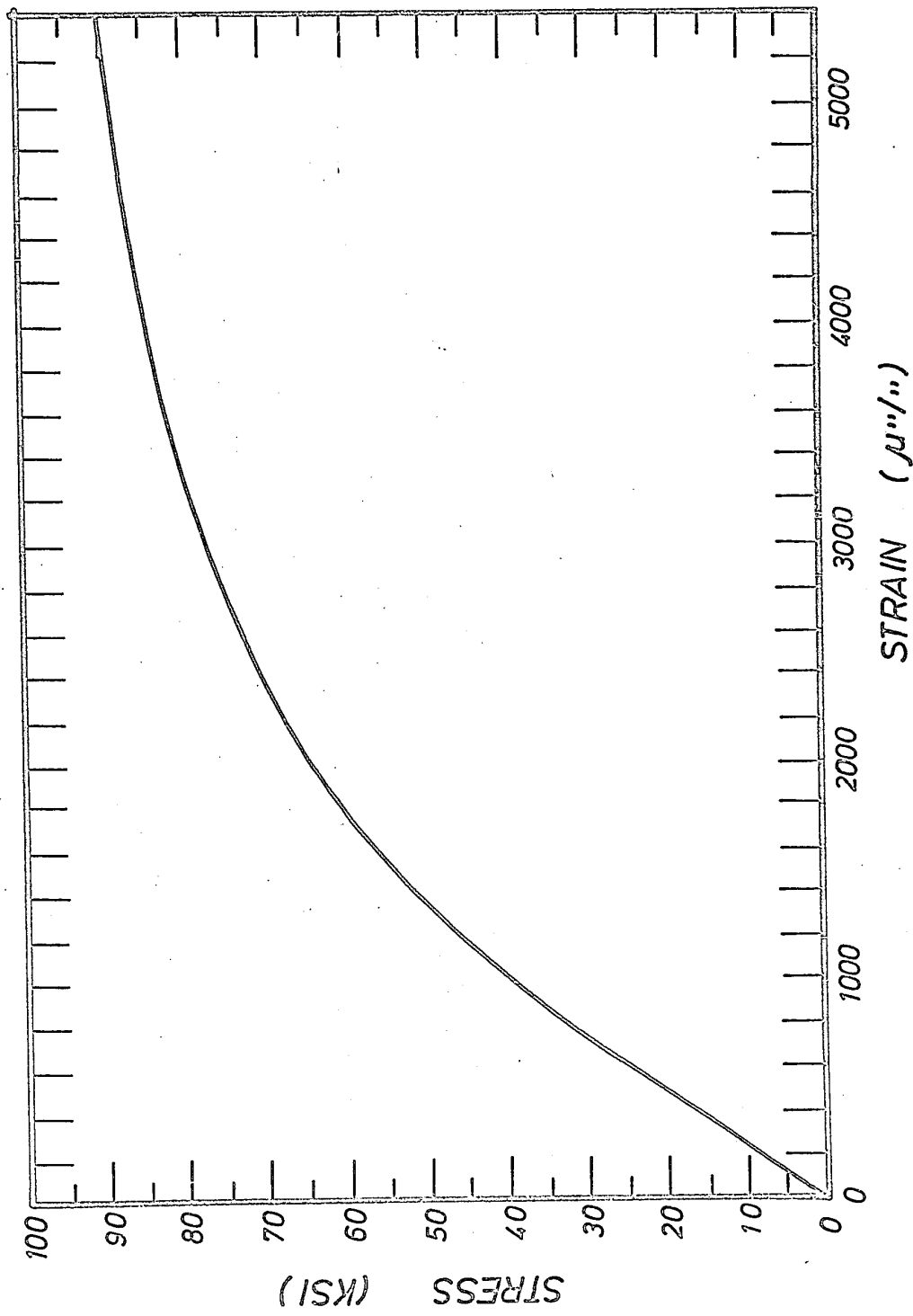


FIG. A-2 STRESS-STRAIN CHARACTERISTIC OF
LONGITUDINAL REINFORCEMENT (#3 BAR)

1

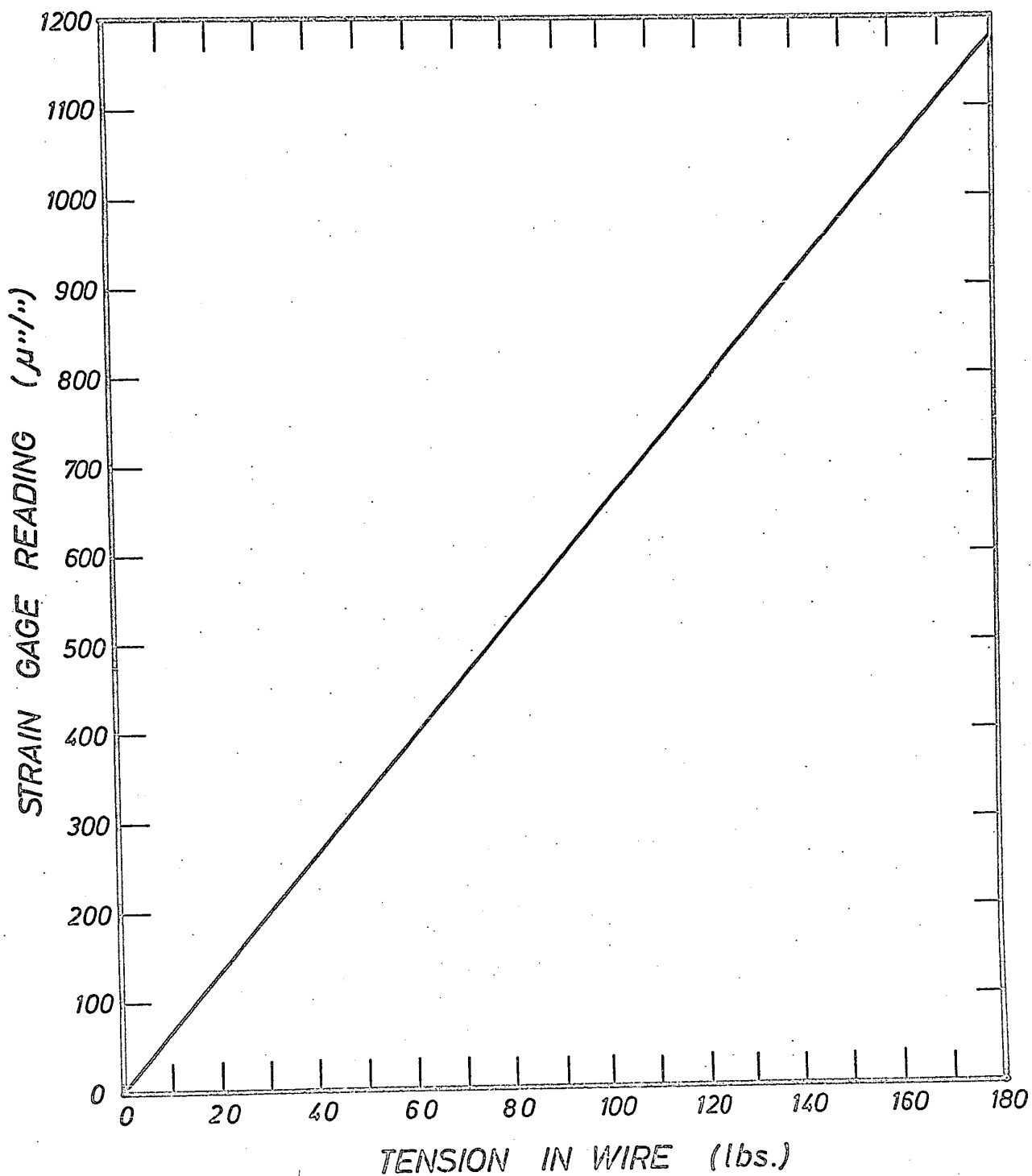


FIG. A-3 CALIBRATION CURVE OF
WRAPPING DEVICE

APPENDIX BIndividual Graphs of Testing Data of Specimens

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Fig.B.1 Load/Deformation Curves of Specimens	128
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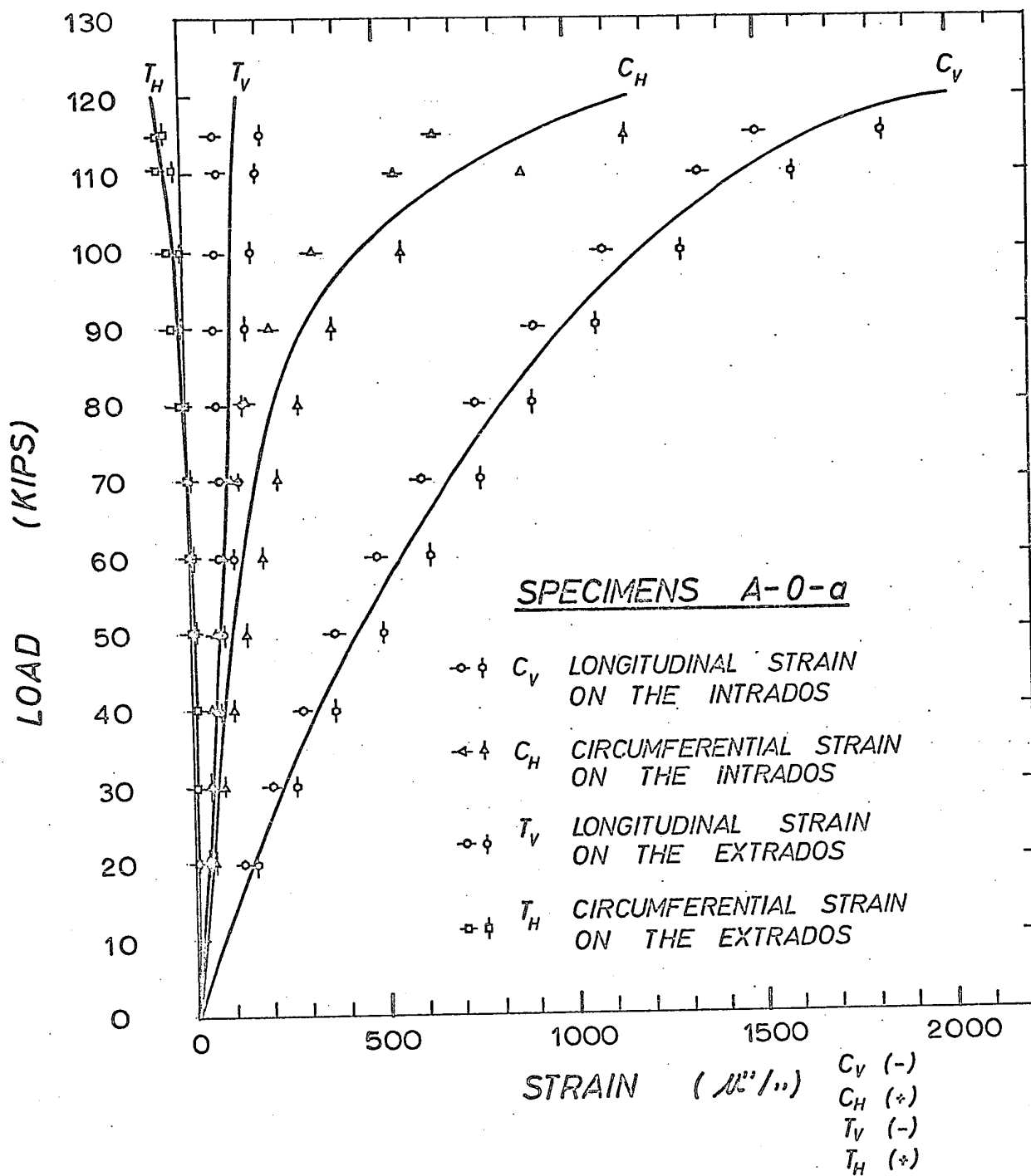


FIG. B-1 LOAD DEFORMATION CURVE
SPECIMENS A-0-a

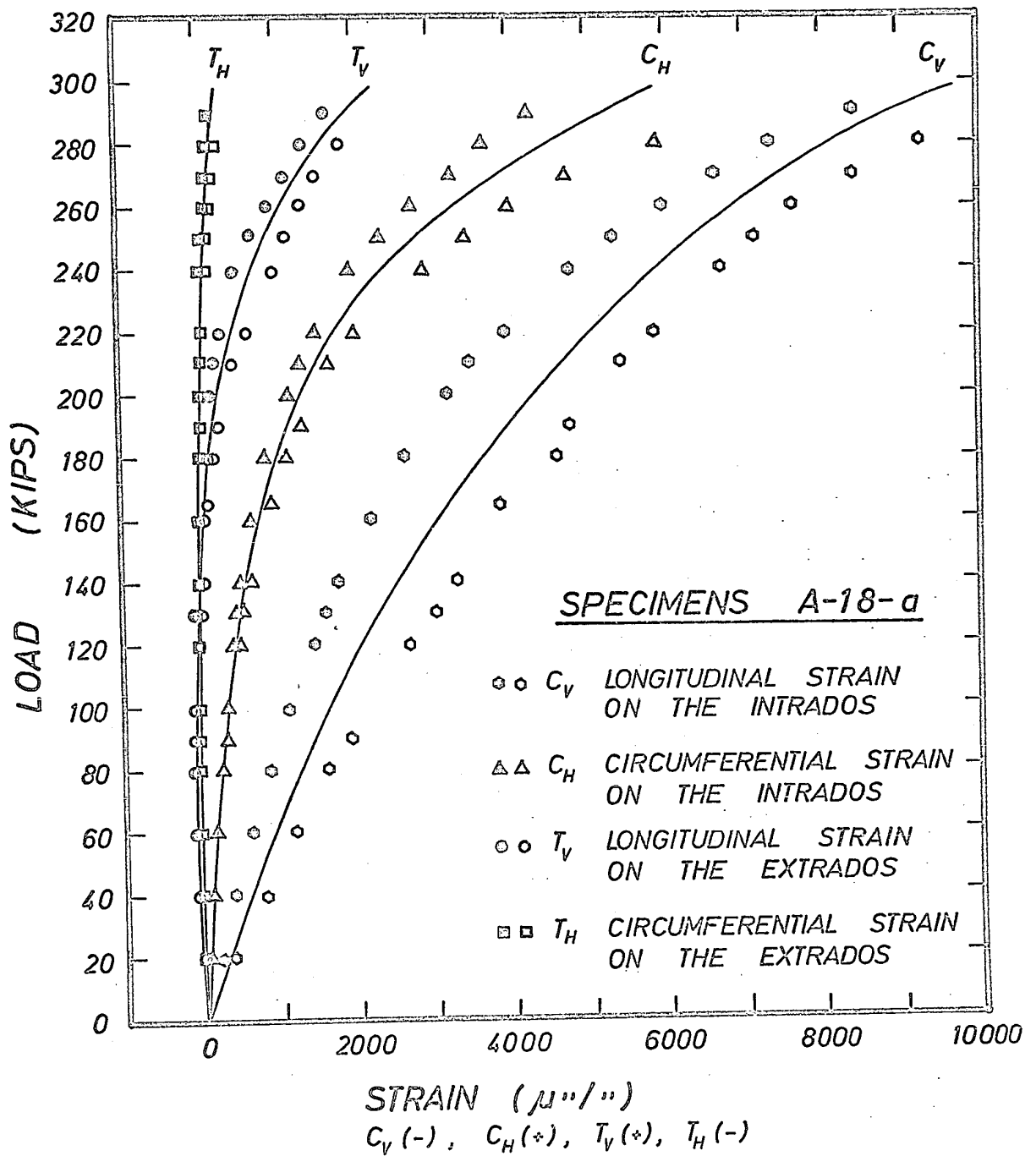


FIG. B.2 LOAD DEFORMATION CURVE OF SPECIMENS A-18-a

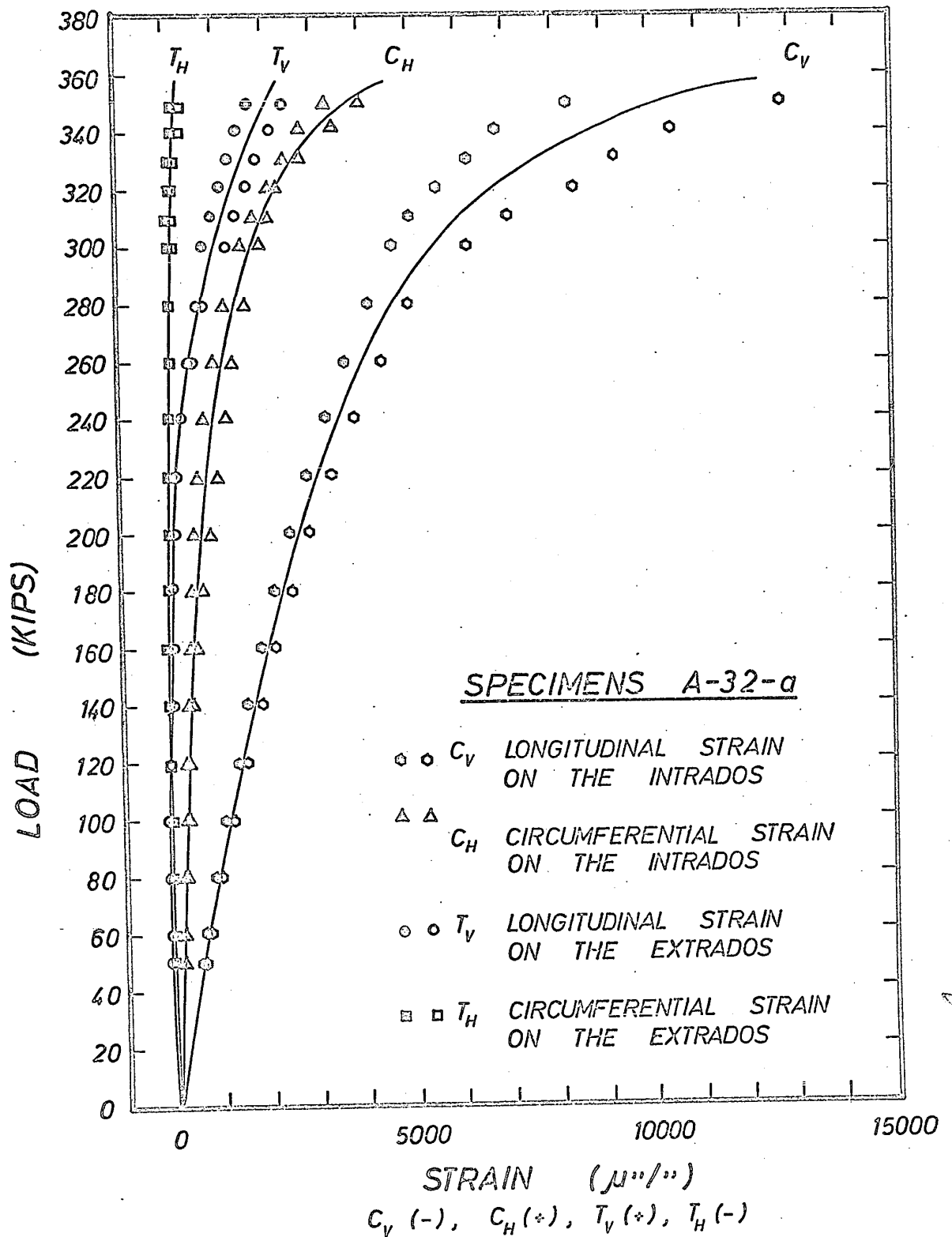


FIG. B-3

LOAD / DEFORMATION CURVE
OF SPECIMENS A-32-a

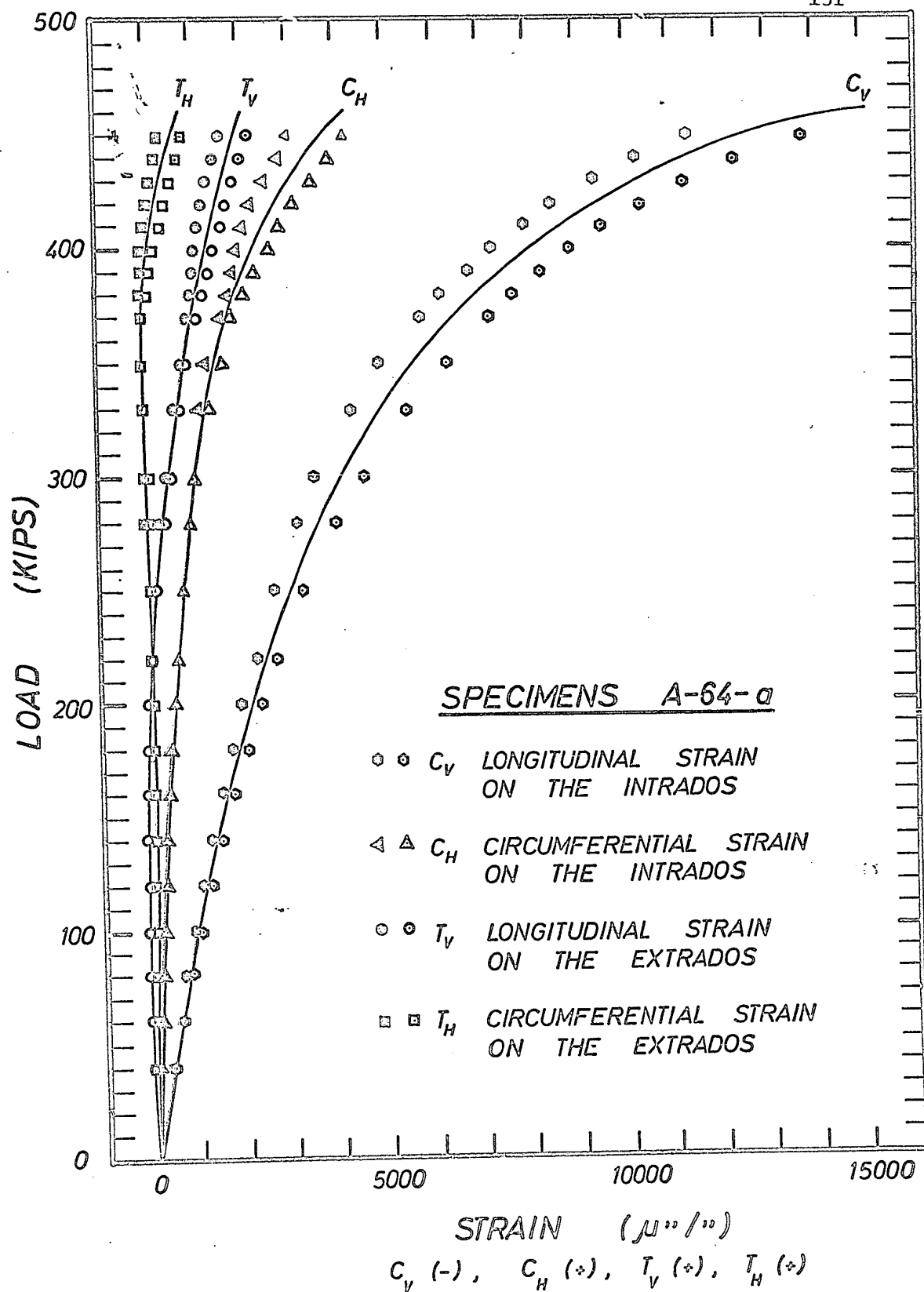


FIG. B-4 LOAD/DEFORMATION CURVE OF SPECIMENS A-64-a

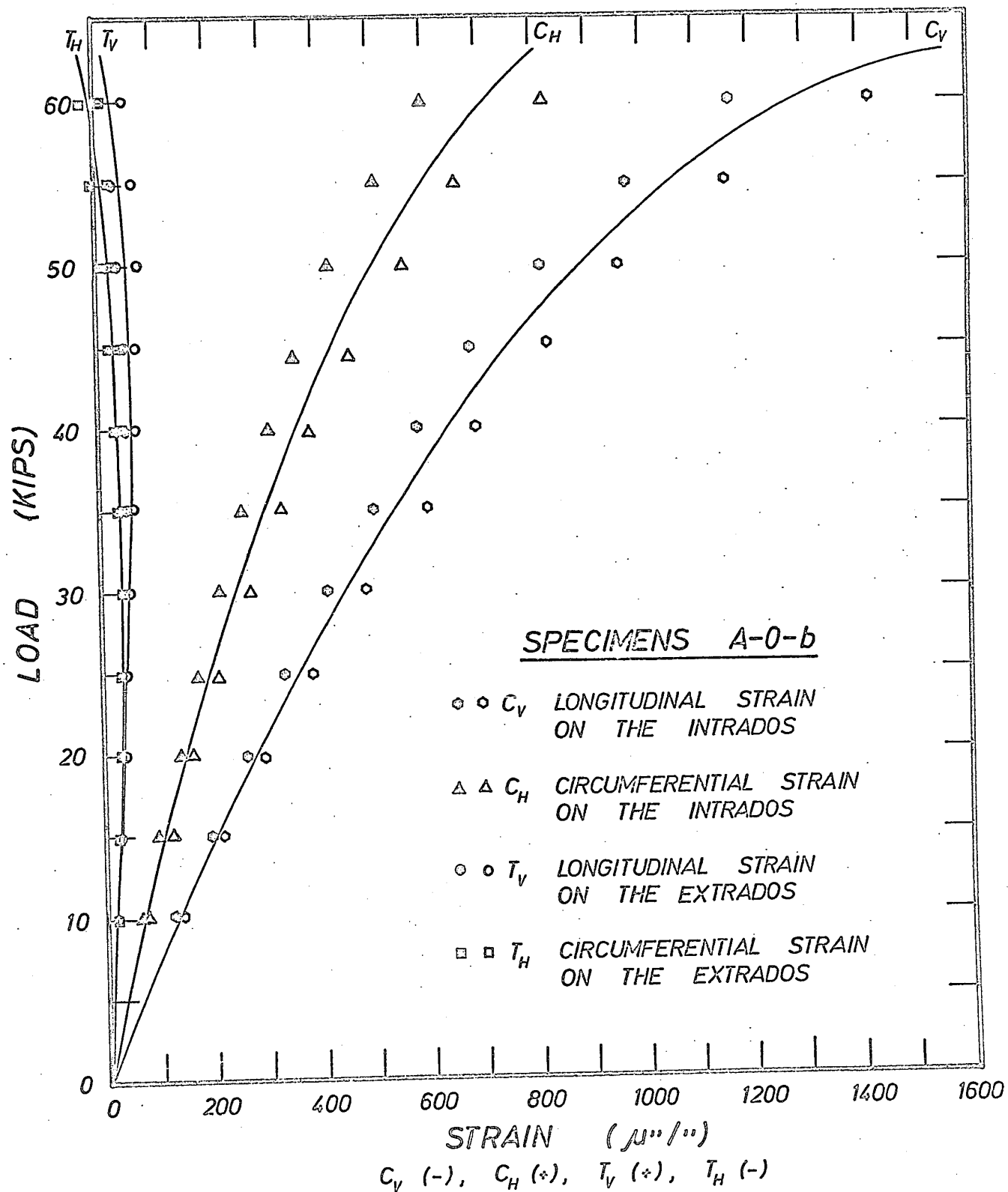


FIG. B-5 LOAD/DEFORMATION CURVE OF SPECIMENS A-0-b

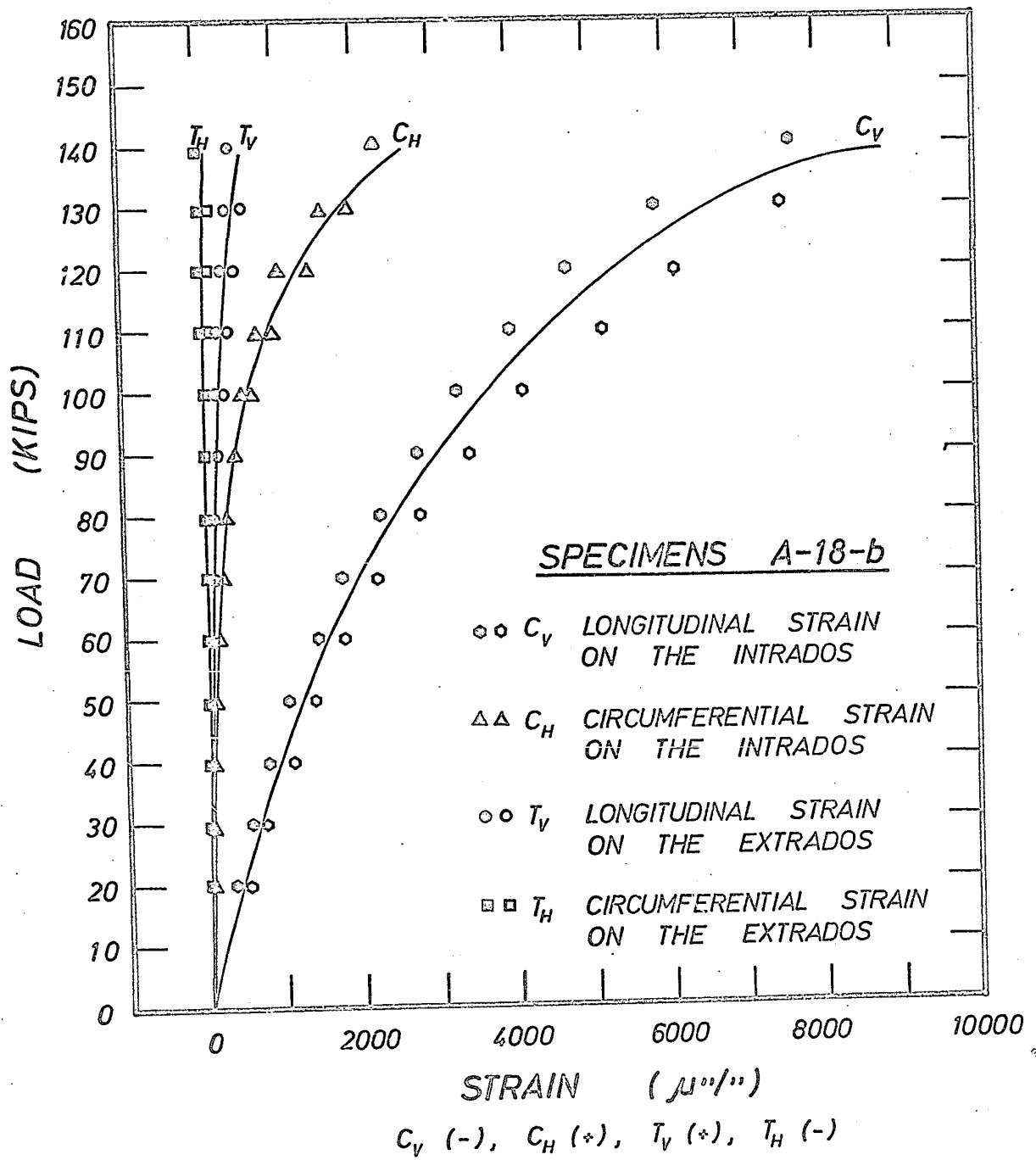


FIG. B-6 LOAD/DEFORMATION CURVE
OF SPECIMENS A-18-b

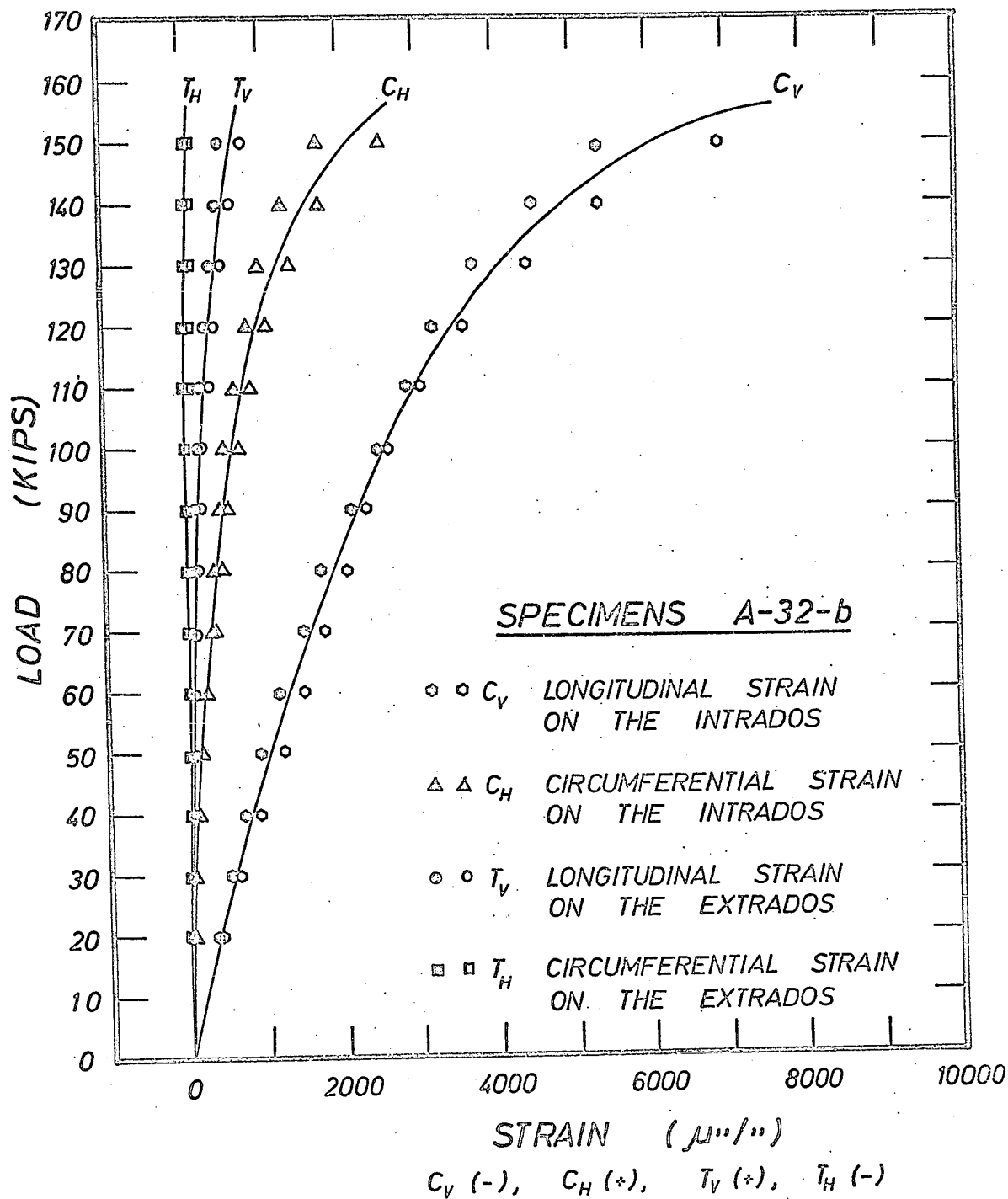


FIG. B-7 LOAD/DEFORMATION CURVE OF SPECIMENS A-32-b

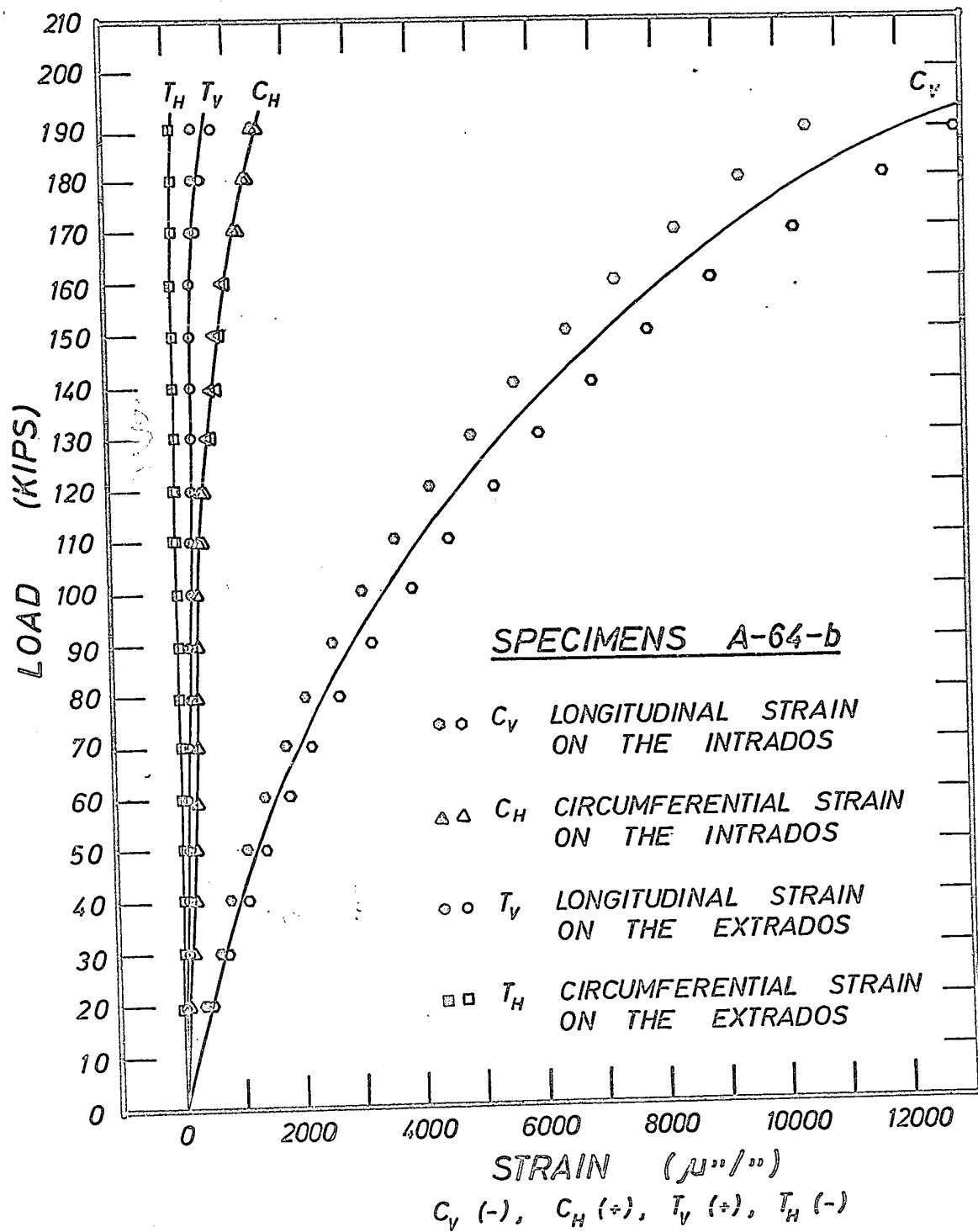


FIG. B-8 LOAD / DEFORMATION CURVE OF SPECIMENS A-64-b

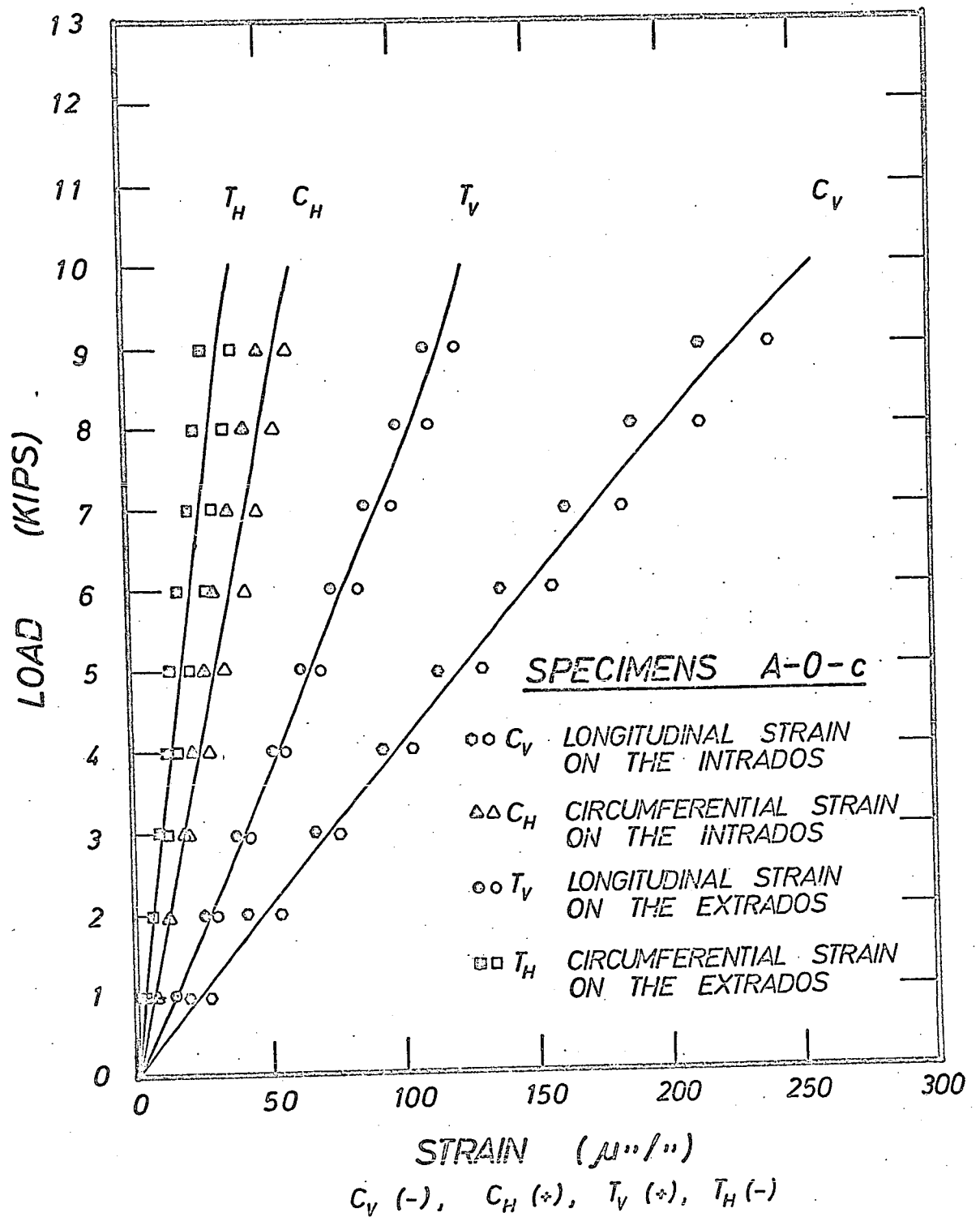


FIG. B-9 LOAD / DEFORMATION CURVE OF SPECIMENS A-0-c

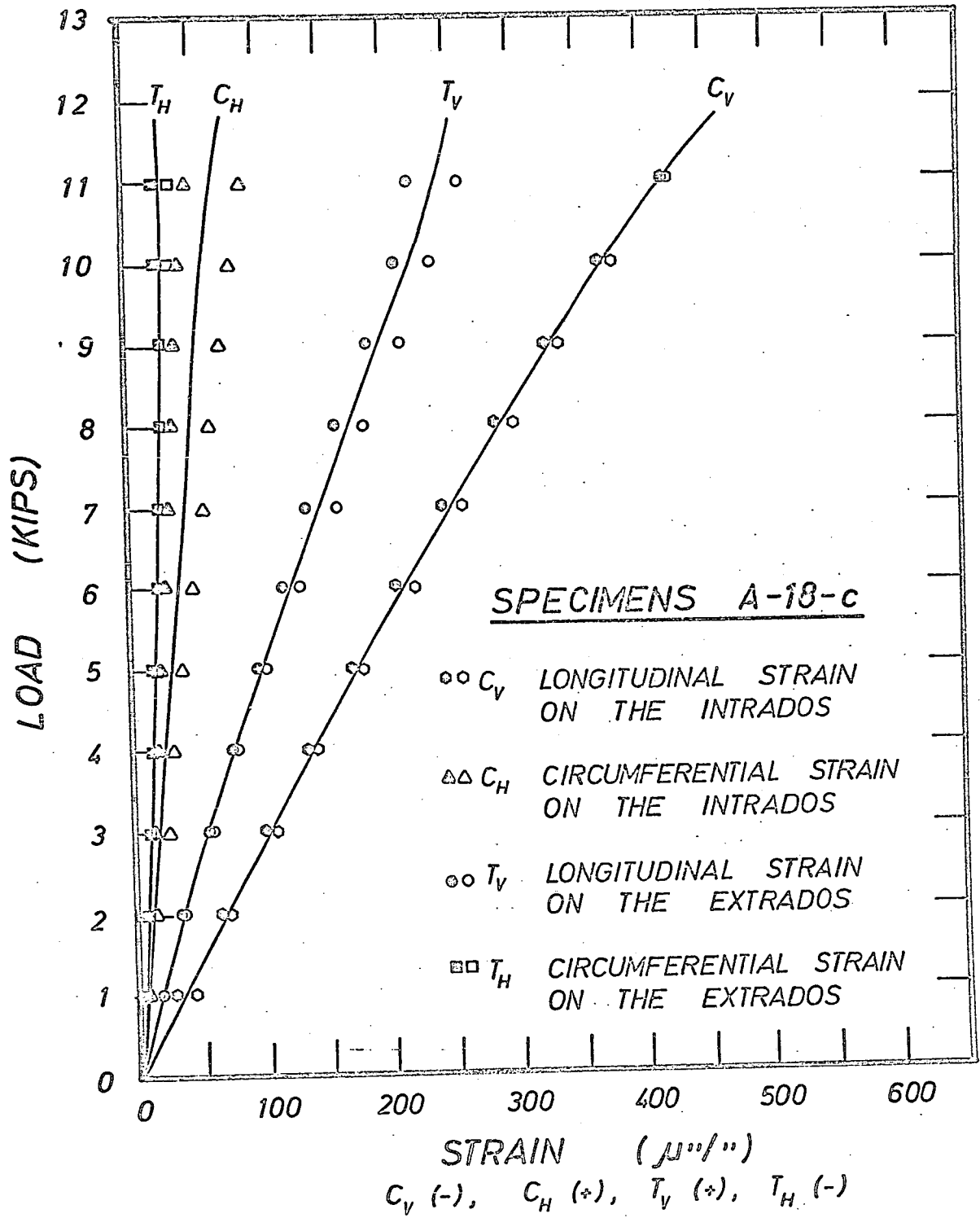


FIG. B-10 LOAD / DEFORMATION CURVE OF SPECIMENS A-18-c

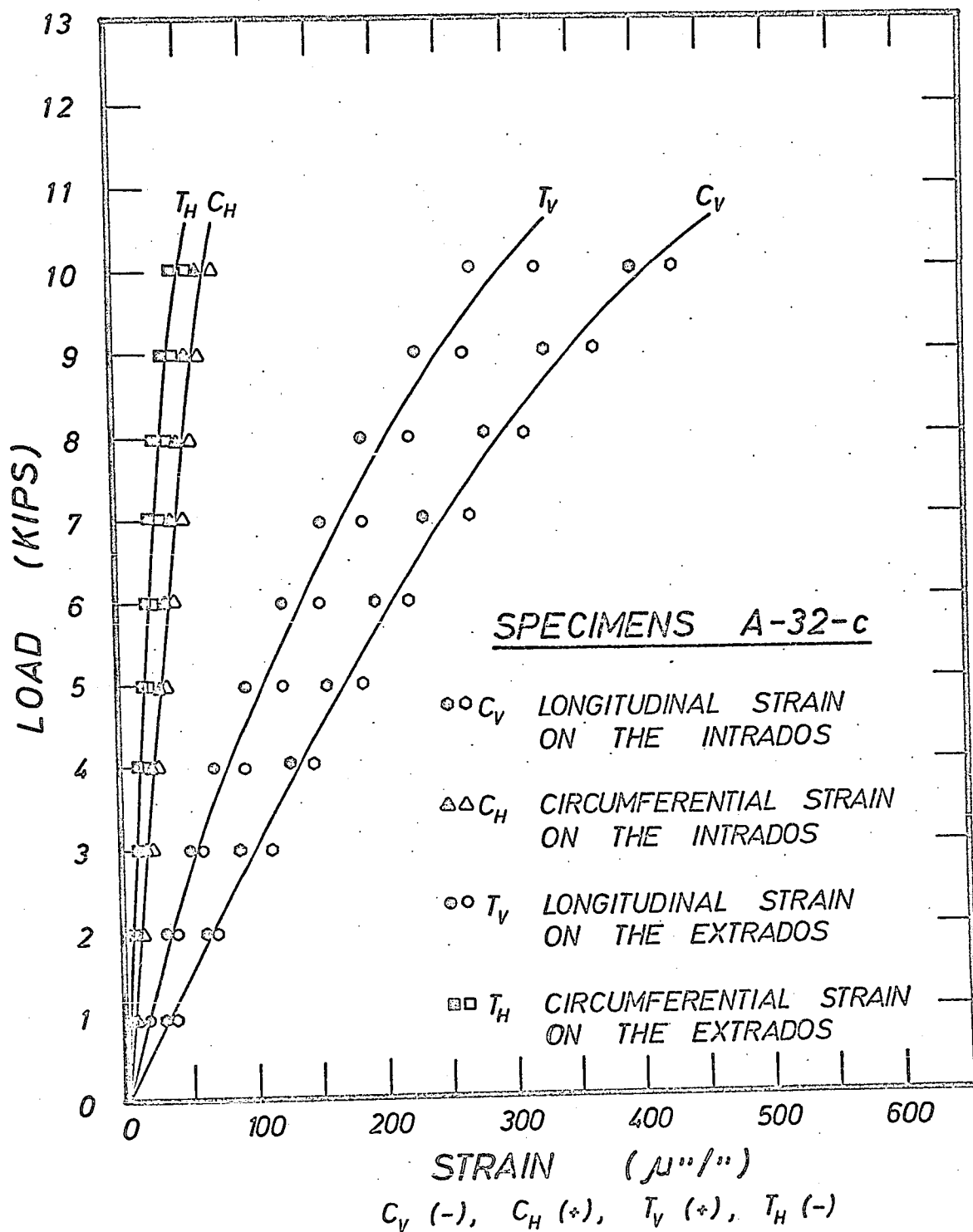


FIG. B-11 LOAD / DEFORMATION CURVE OF SPECIMENS A-32-c

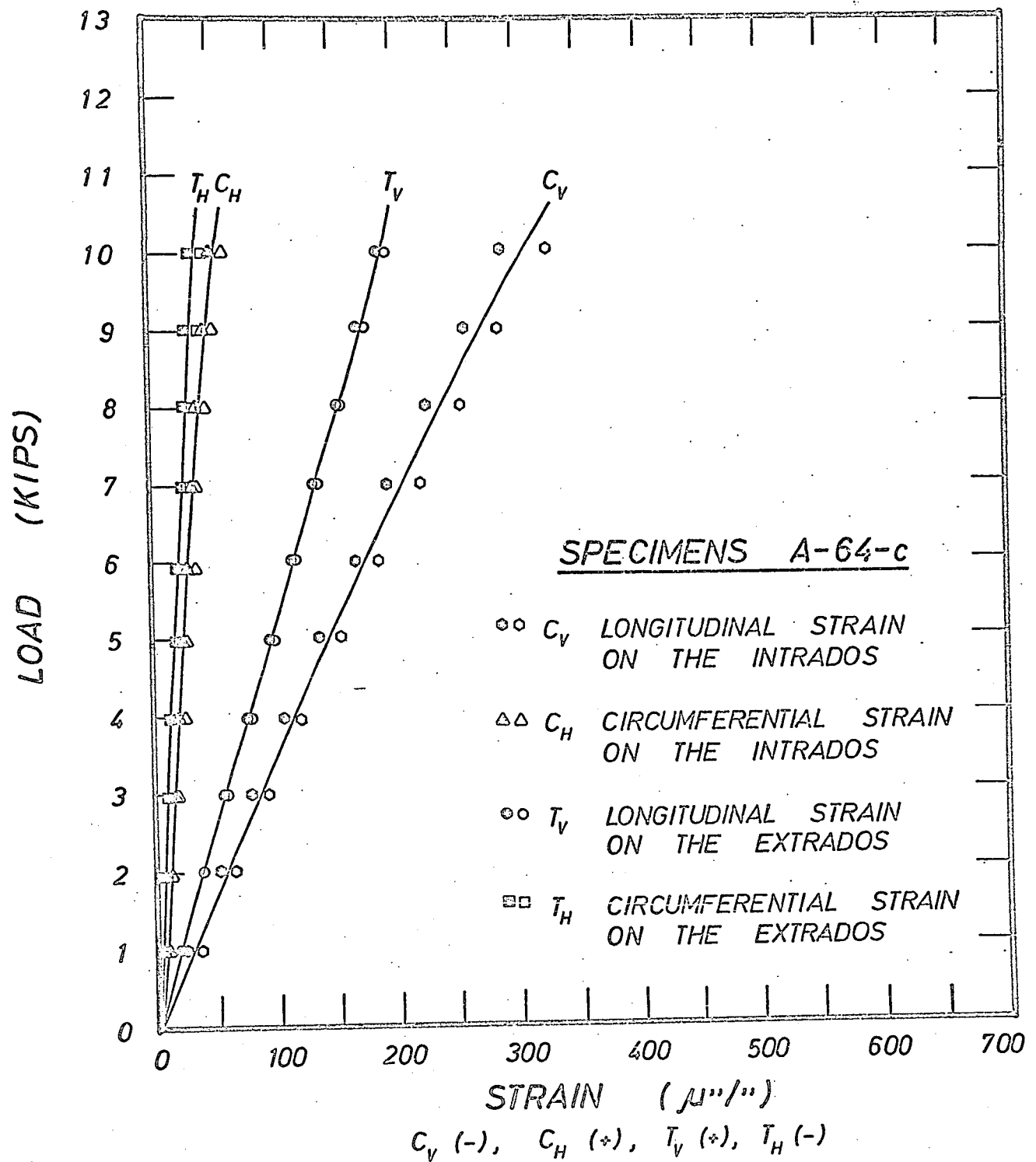


FIG. B-12 LOAD / DEFORMATION CURVE OF SPECIMENS A-64-c

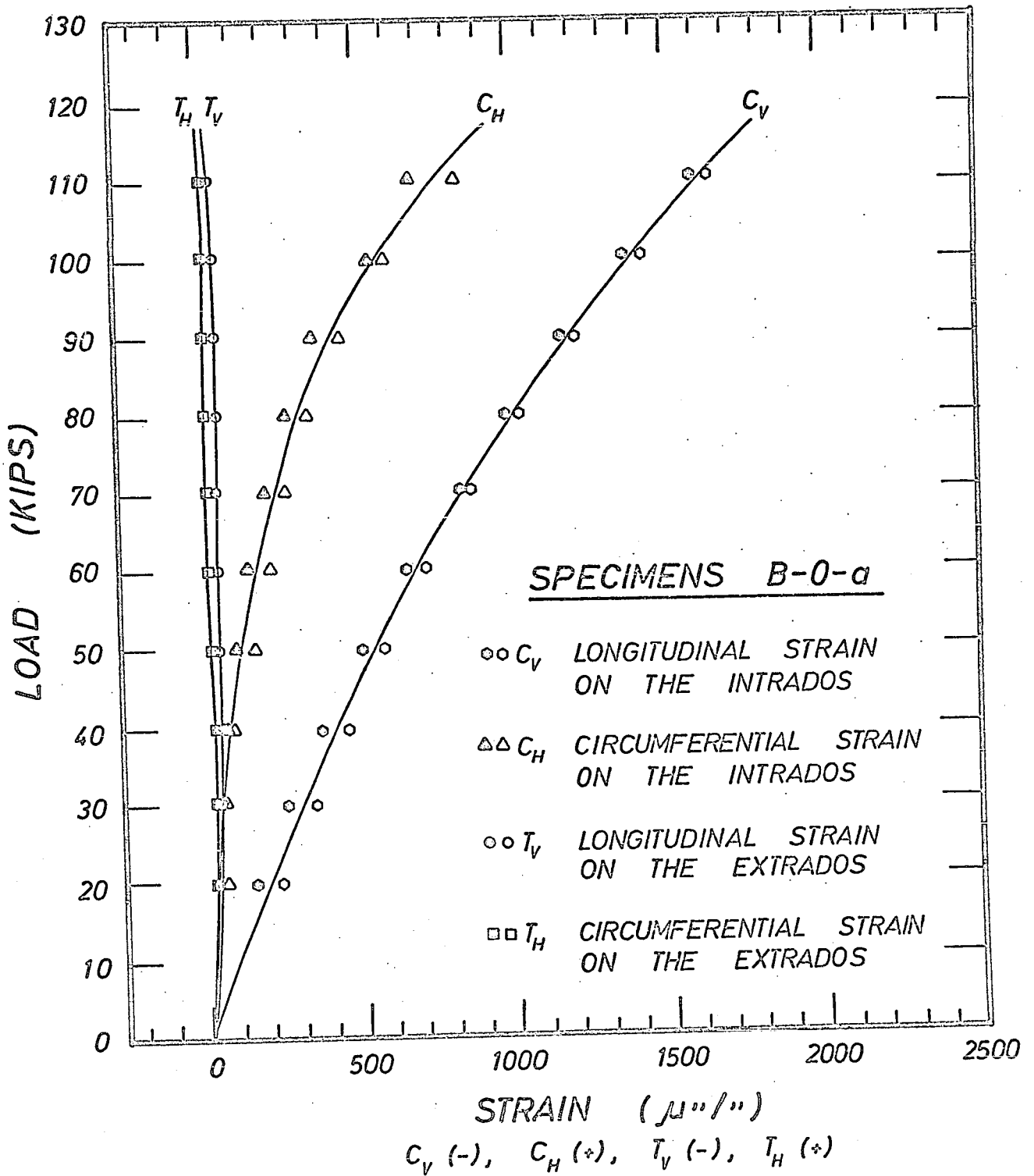


FIG. B-13 LOAD / DEFORMATION CURVE OF SPECIMENS B-0-a

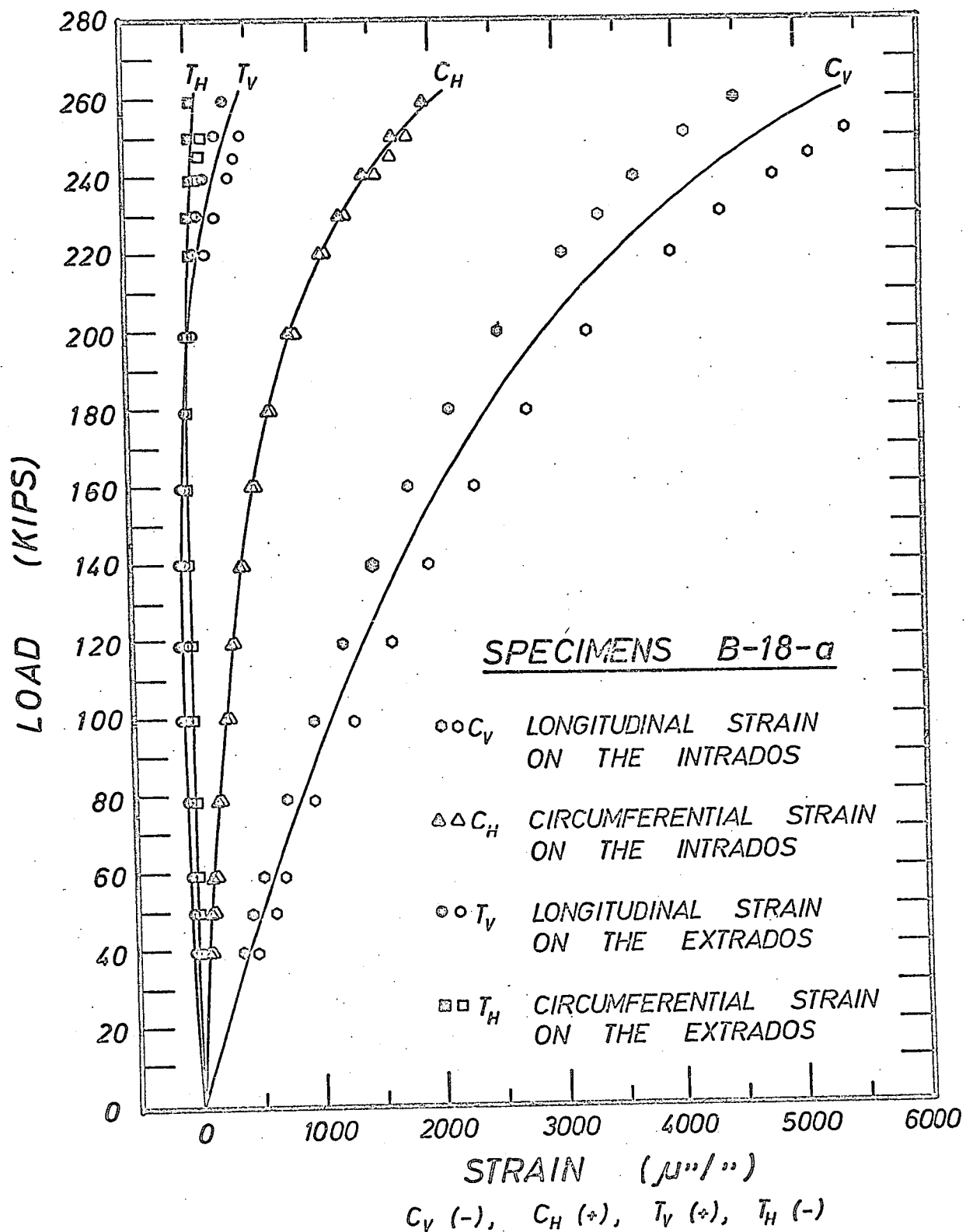


FIG. B-14 LOAD / DEFORMATION CURVE OF SPECIMENS B-18-a

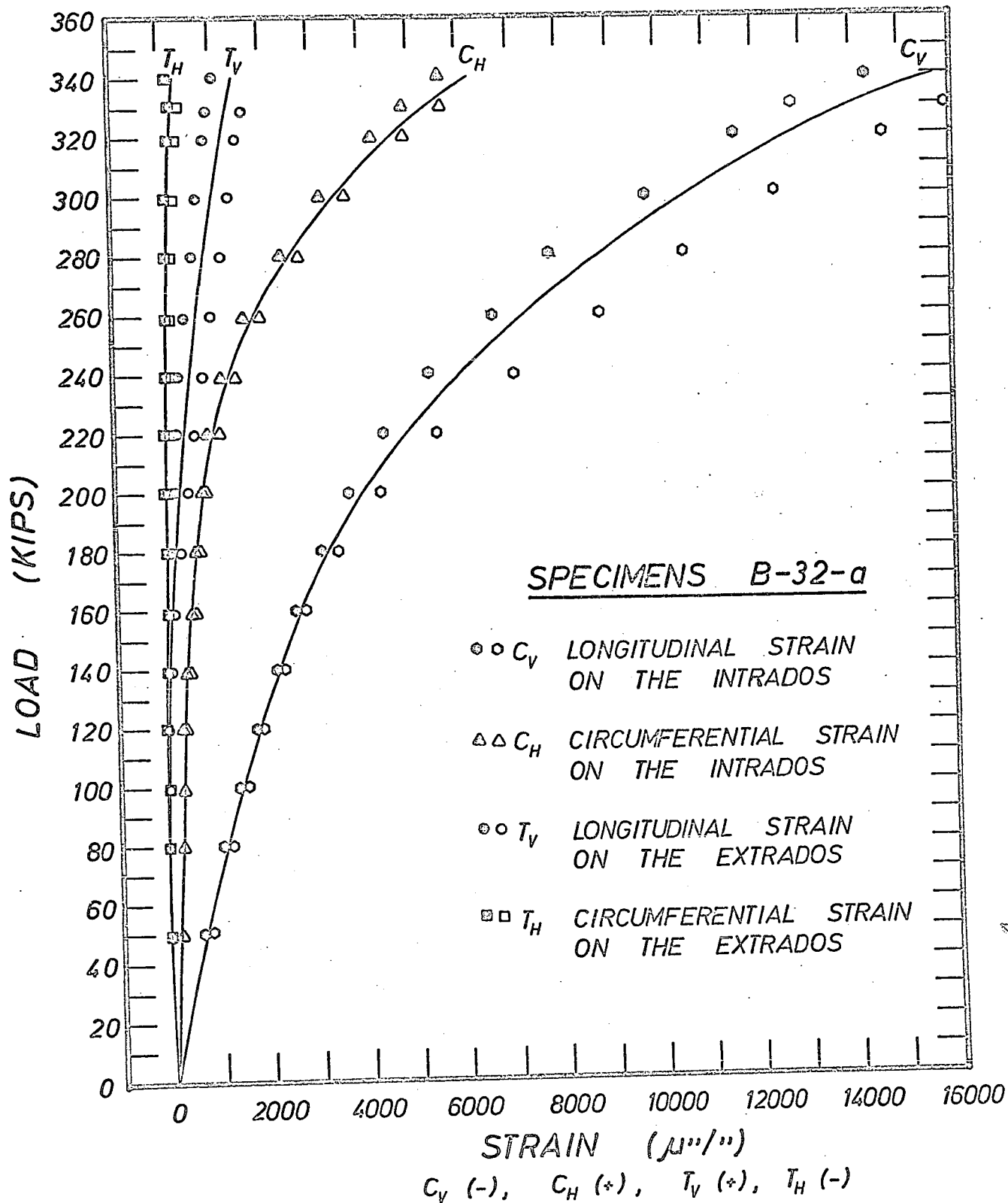


FIG. B-15 LOAD / DEFORMATION CURVE OF SPECIMENS B-32-a

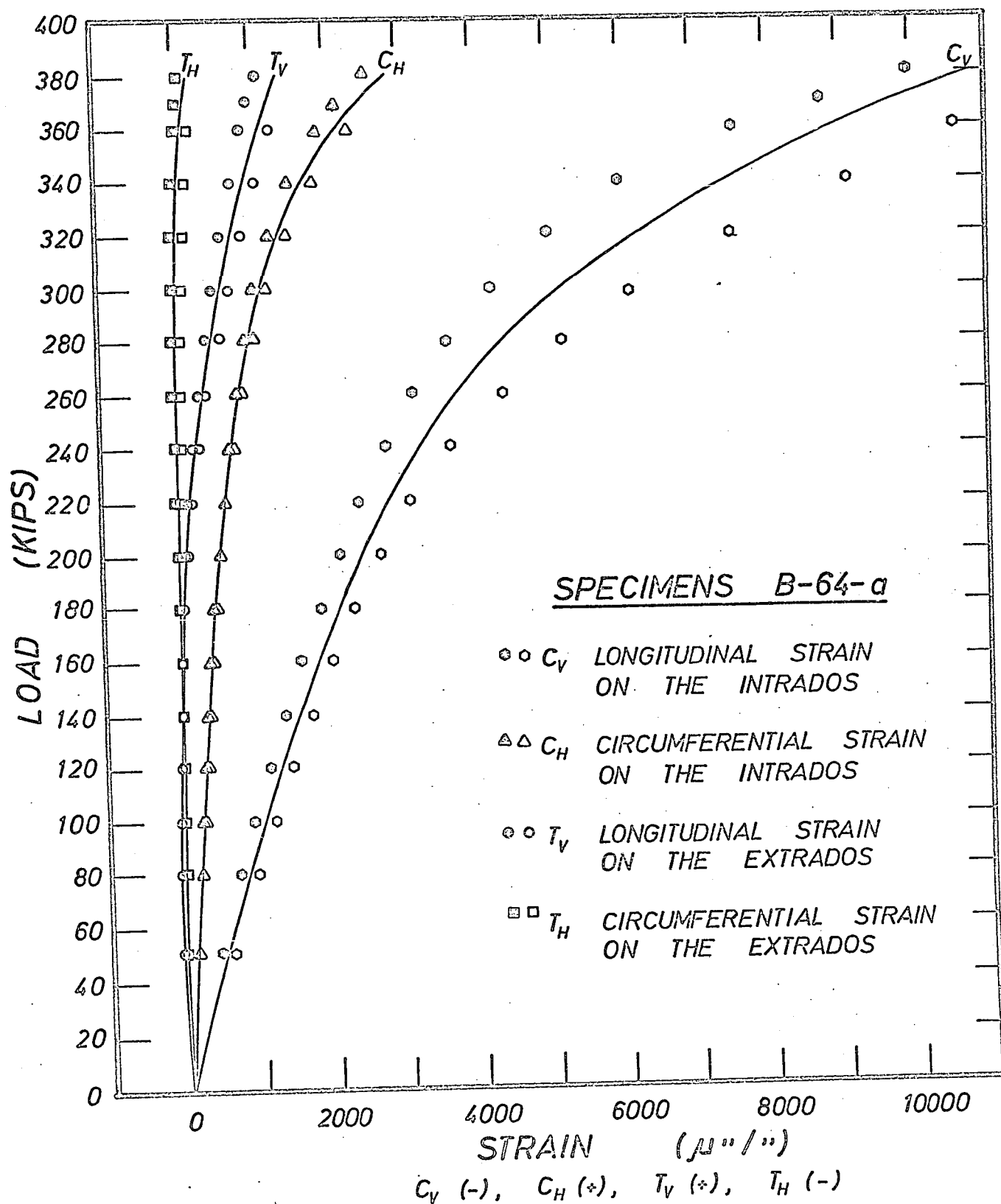


FIG. B-16 LOAD / DEFORMATION CURVE OF SPECIMENS B-64-a

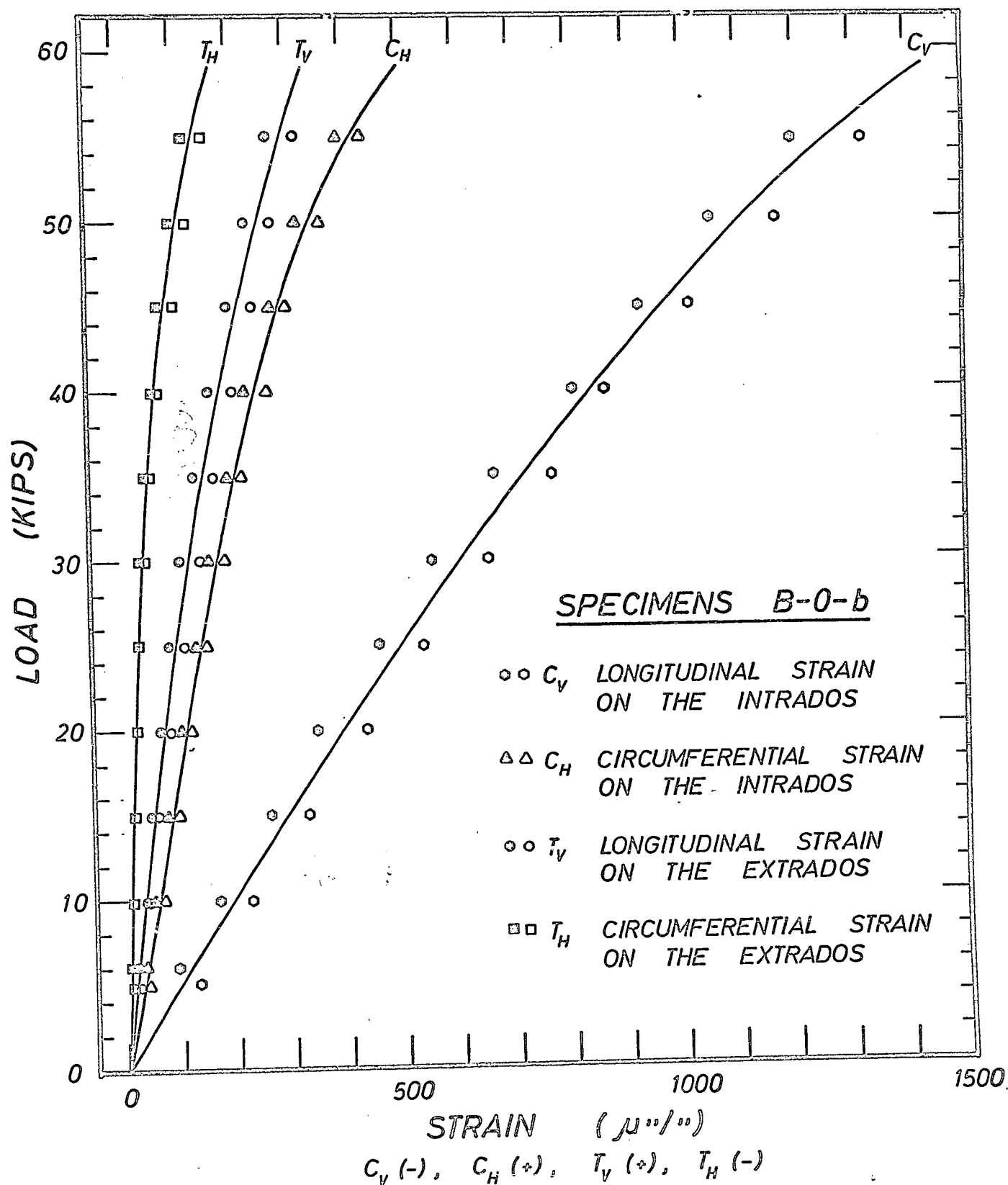


FIG. B-17

LOAD / DEFORMATION CURVE
OF SPECIMENS B-0-b

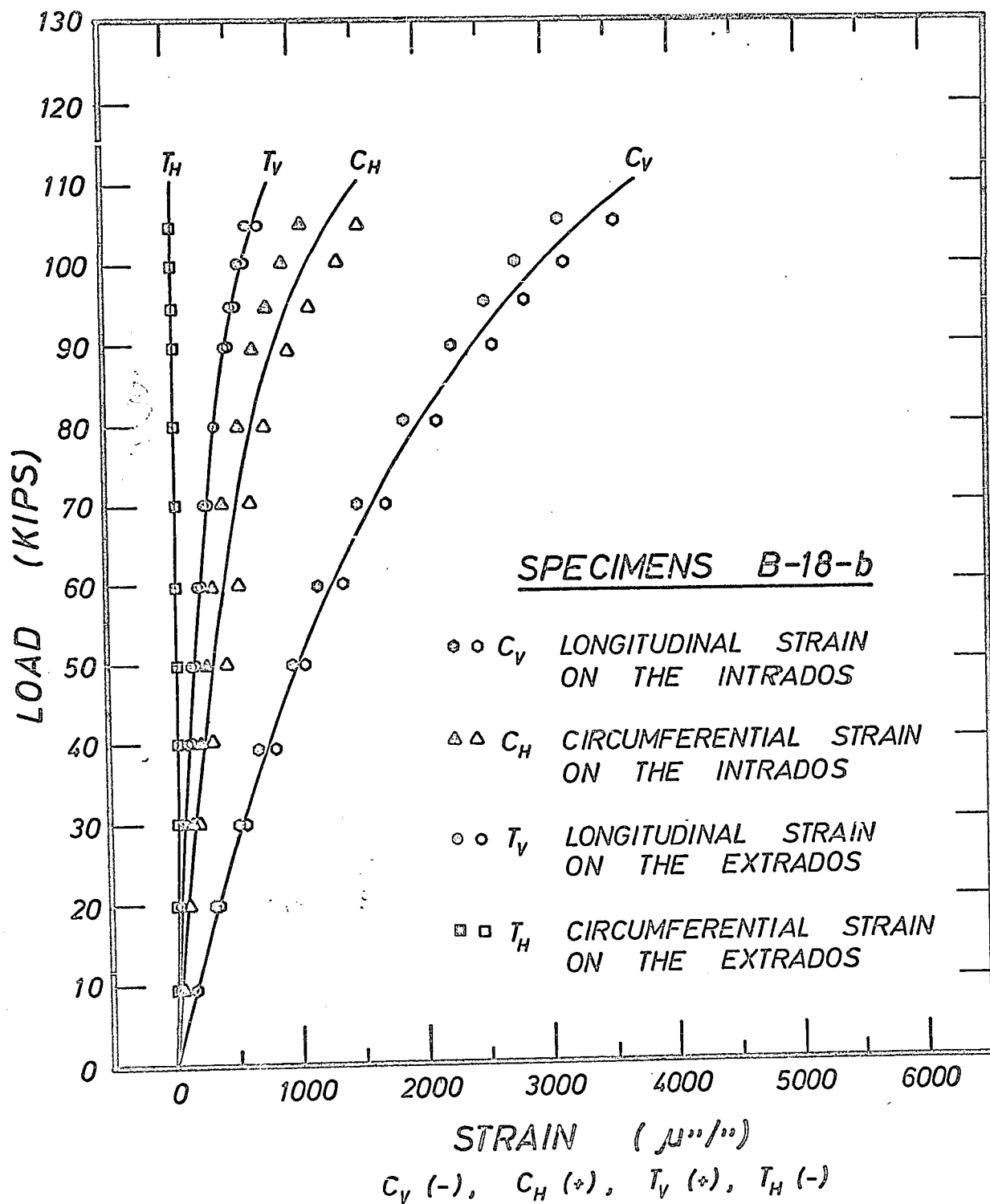


FIG. B-18 LOAD / DEFORMATION CURVE OF SPECIMENS B-18-b

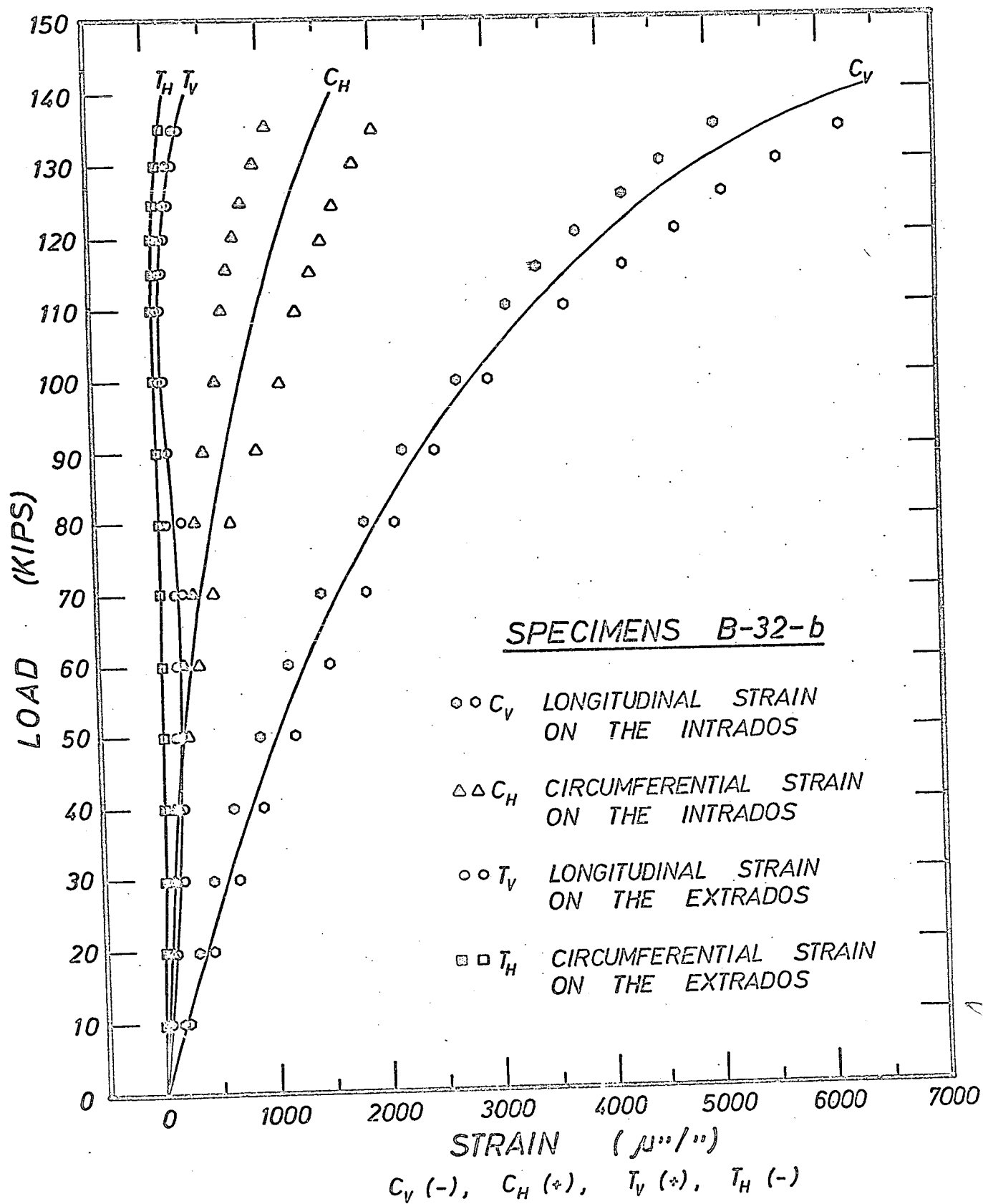


FIG. B-19 LOAD / DEFORMATION CURVE OF SPECIMENS B-32-b

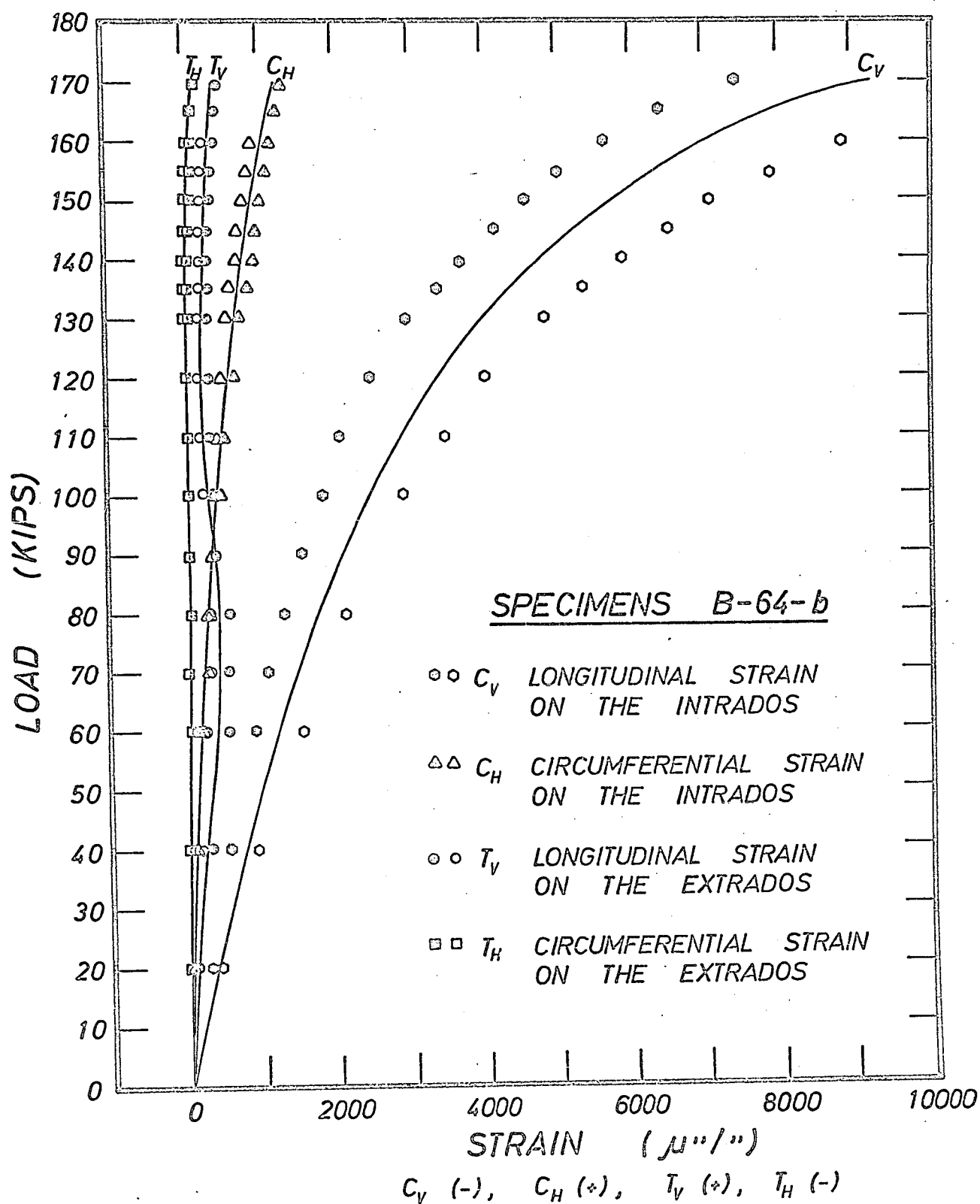


FIG. B-20 LOAD / DEFORMATION CURVE OF SPECIMENS B-64-b

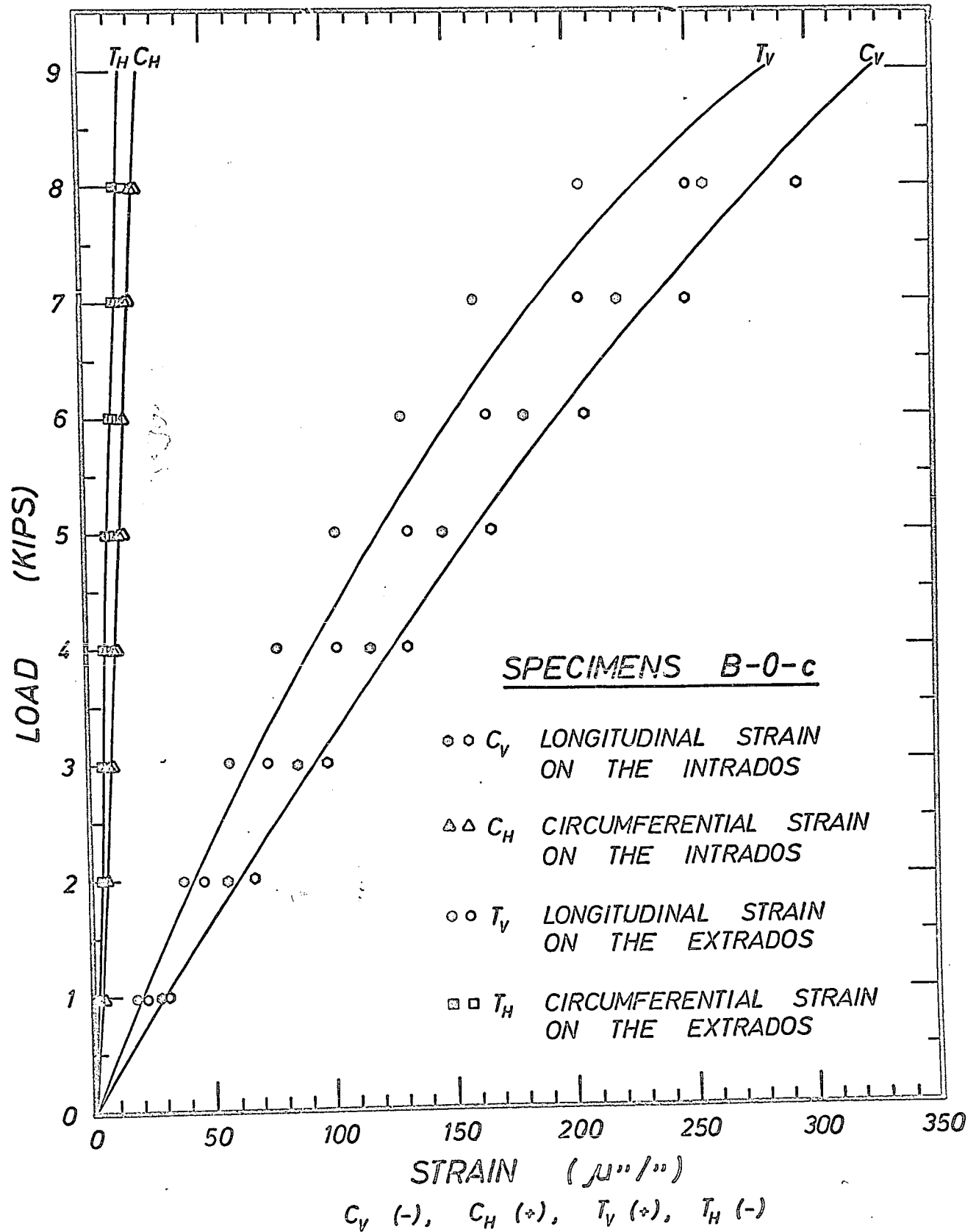


FIG. B-21 LOAD / DEFORMATION CURVE OF SPECIMENS B-0-c

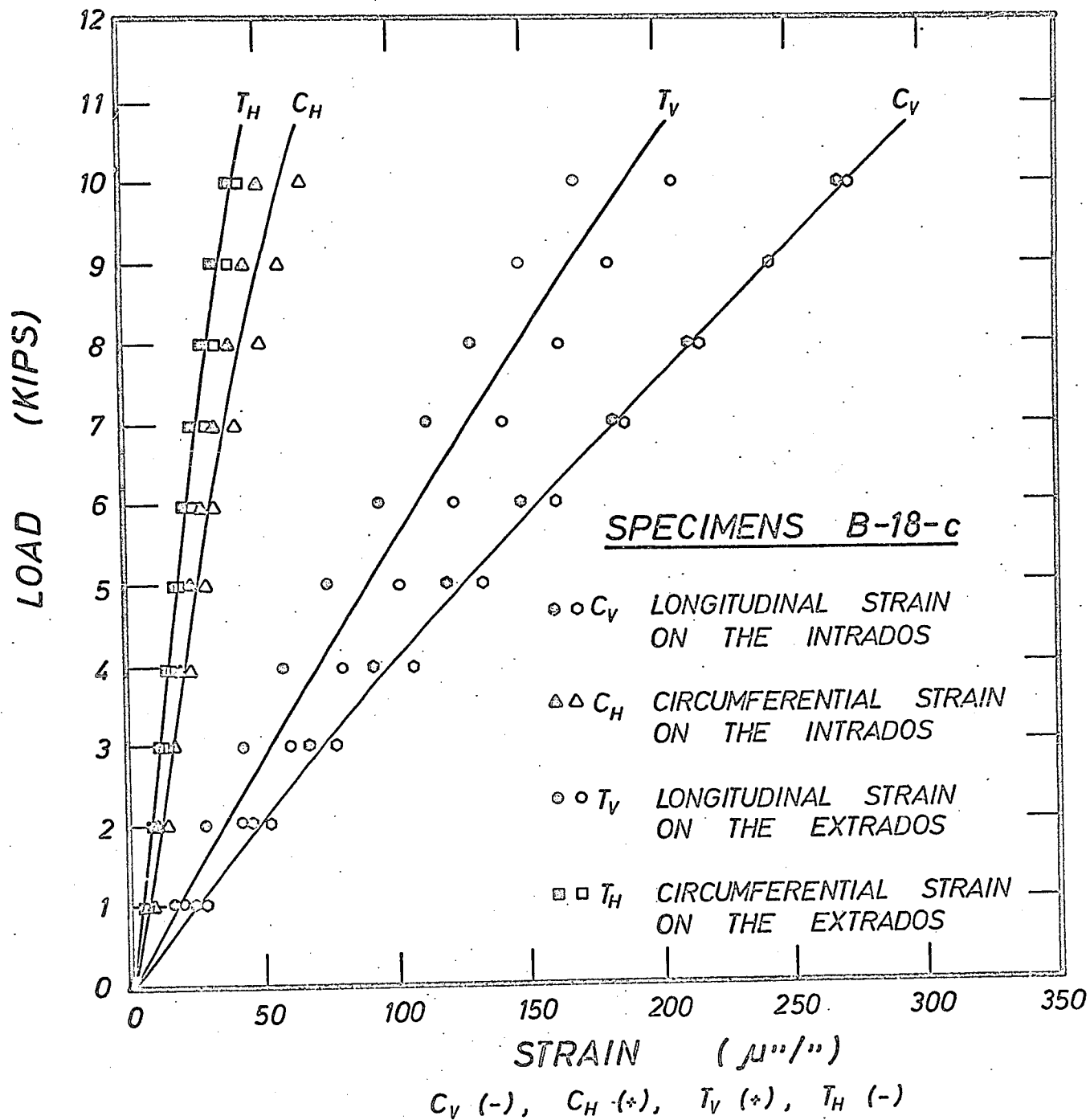


FIG. B-22 LOAD / DEFORMATION CURVE OF SPECIMENS B-18-c

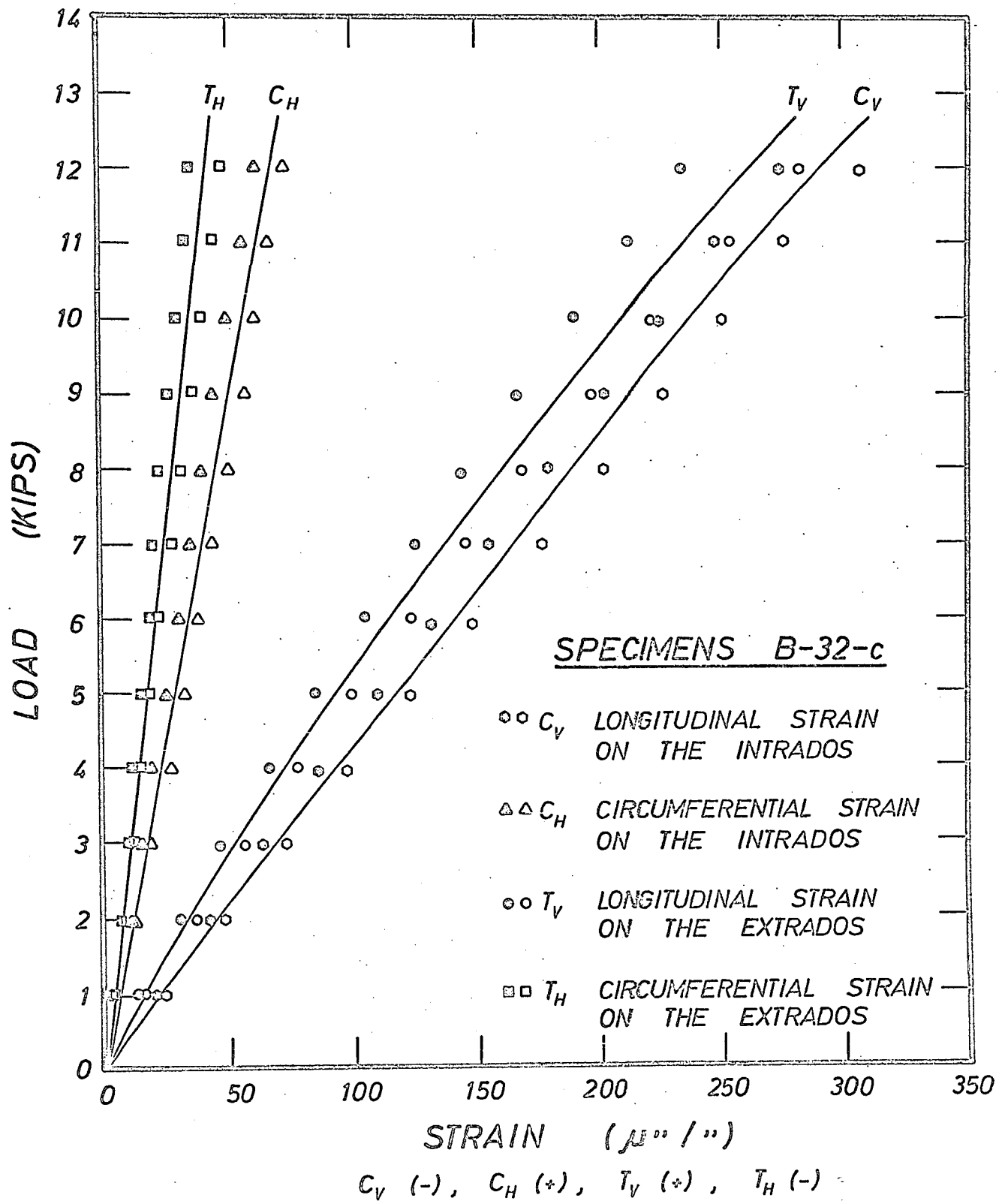


FIG. B-23 LOAD / DEFORMATION CURVE OF SPECIMENS B-32-c

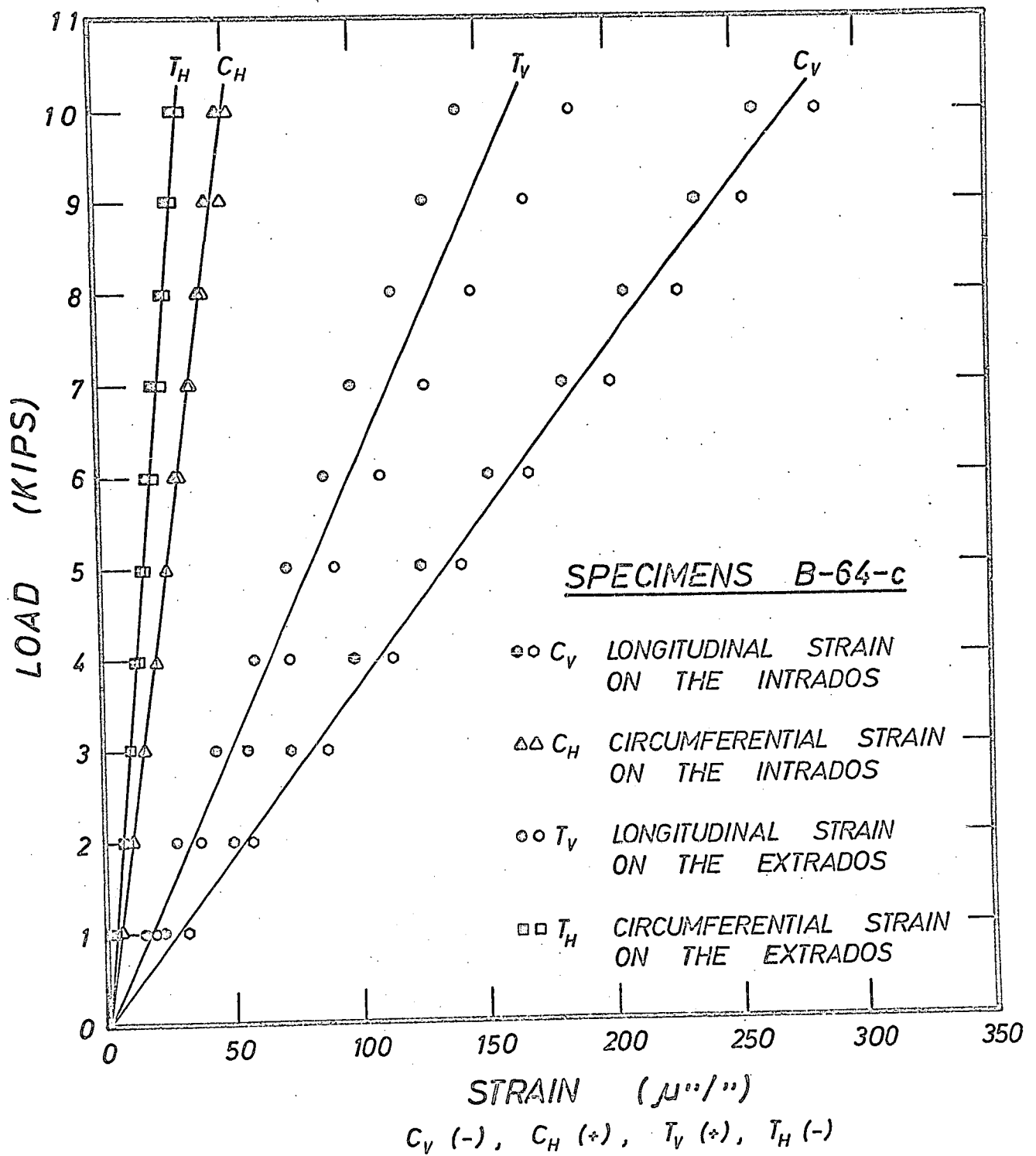


FIG. B.24 LOAD / DEFORMATION CURVE OF SPECIMENS B-64-c

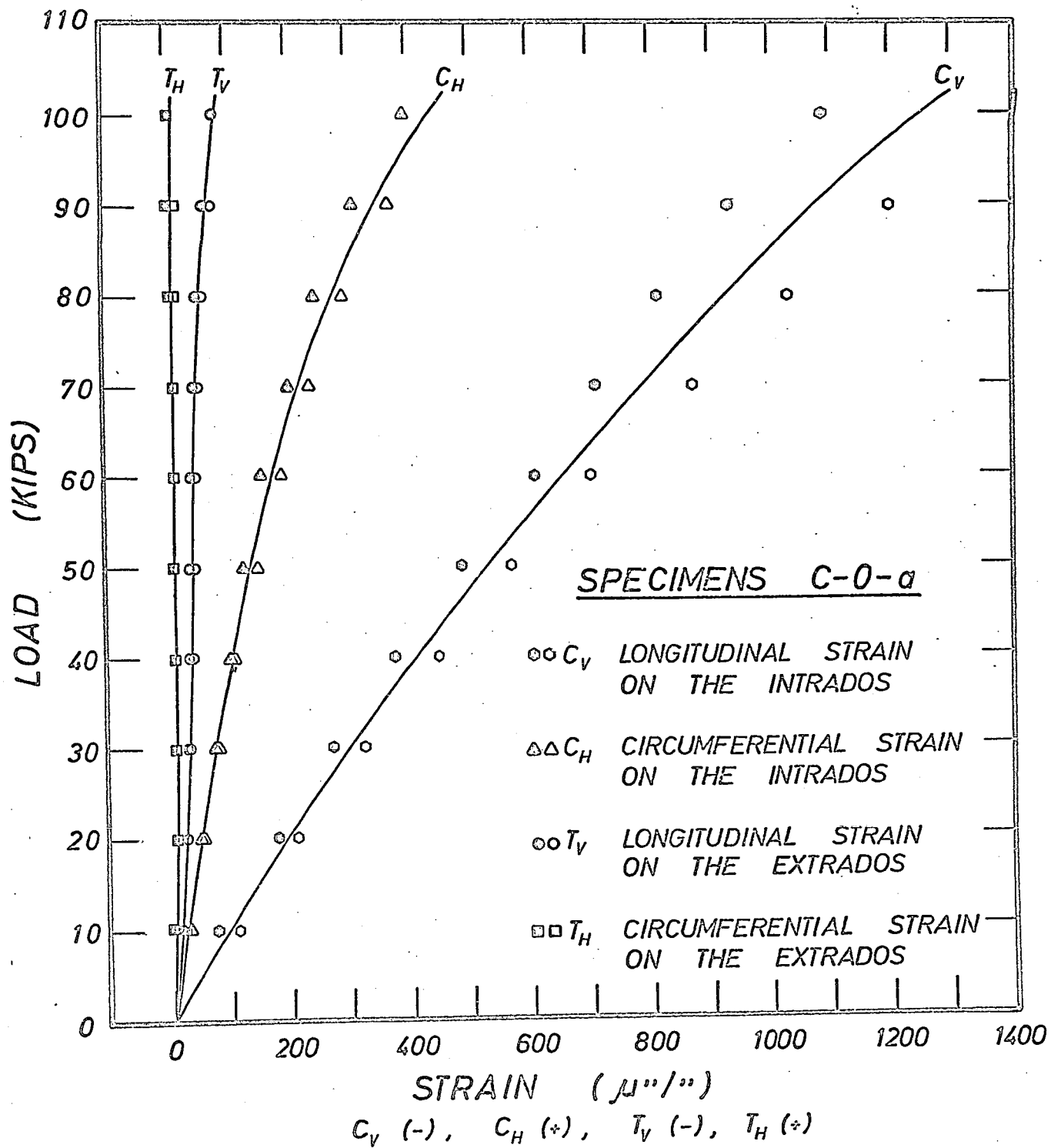


FIG. B-25 LOAD / DEFORMATION CURVE OF SPECIMENS C-0-a

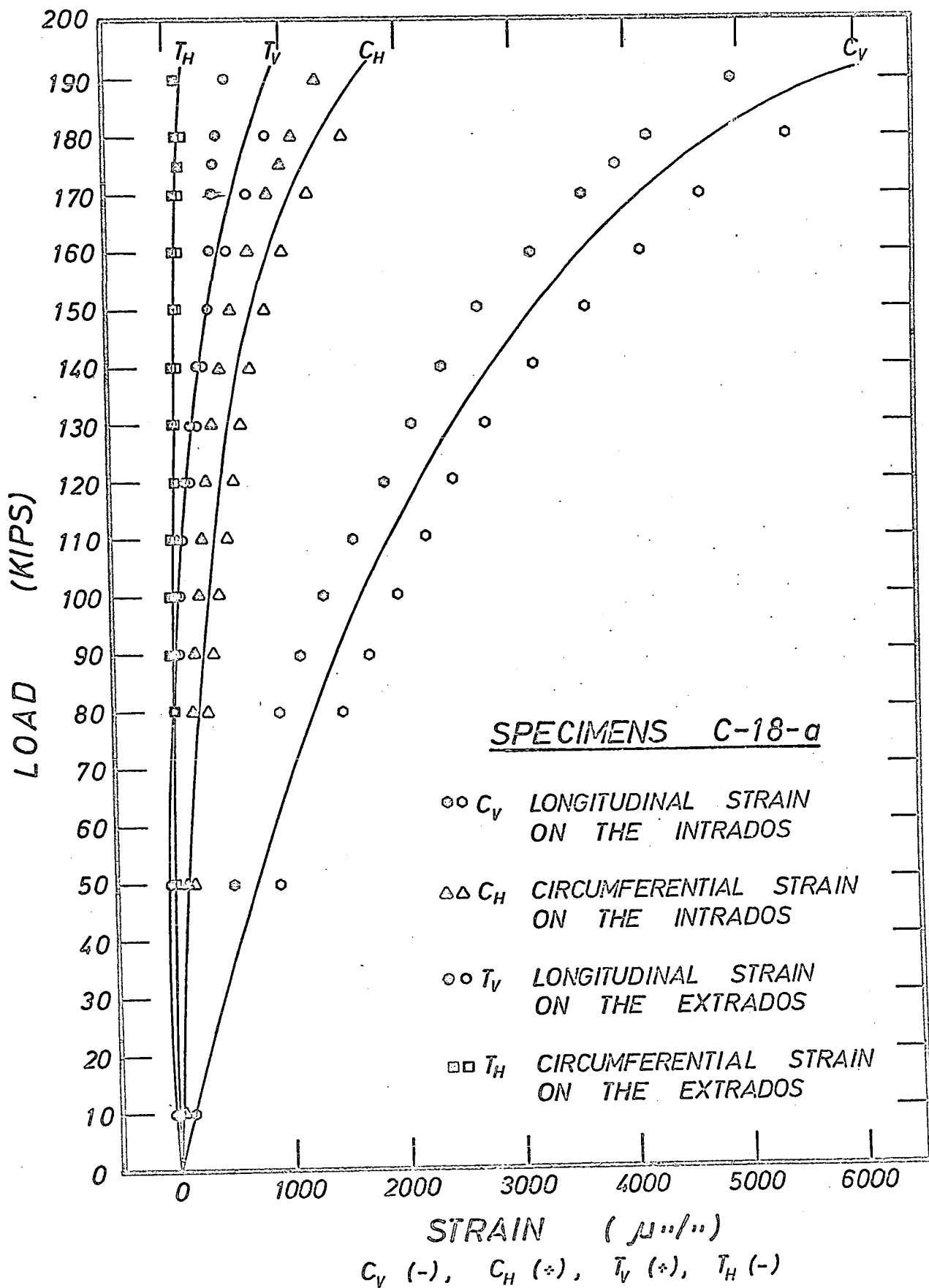


FIG. B-26 LOAD / DEFORMATION CURVE OF SPECIMENS C-18-a

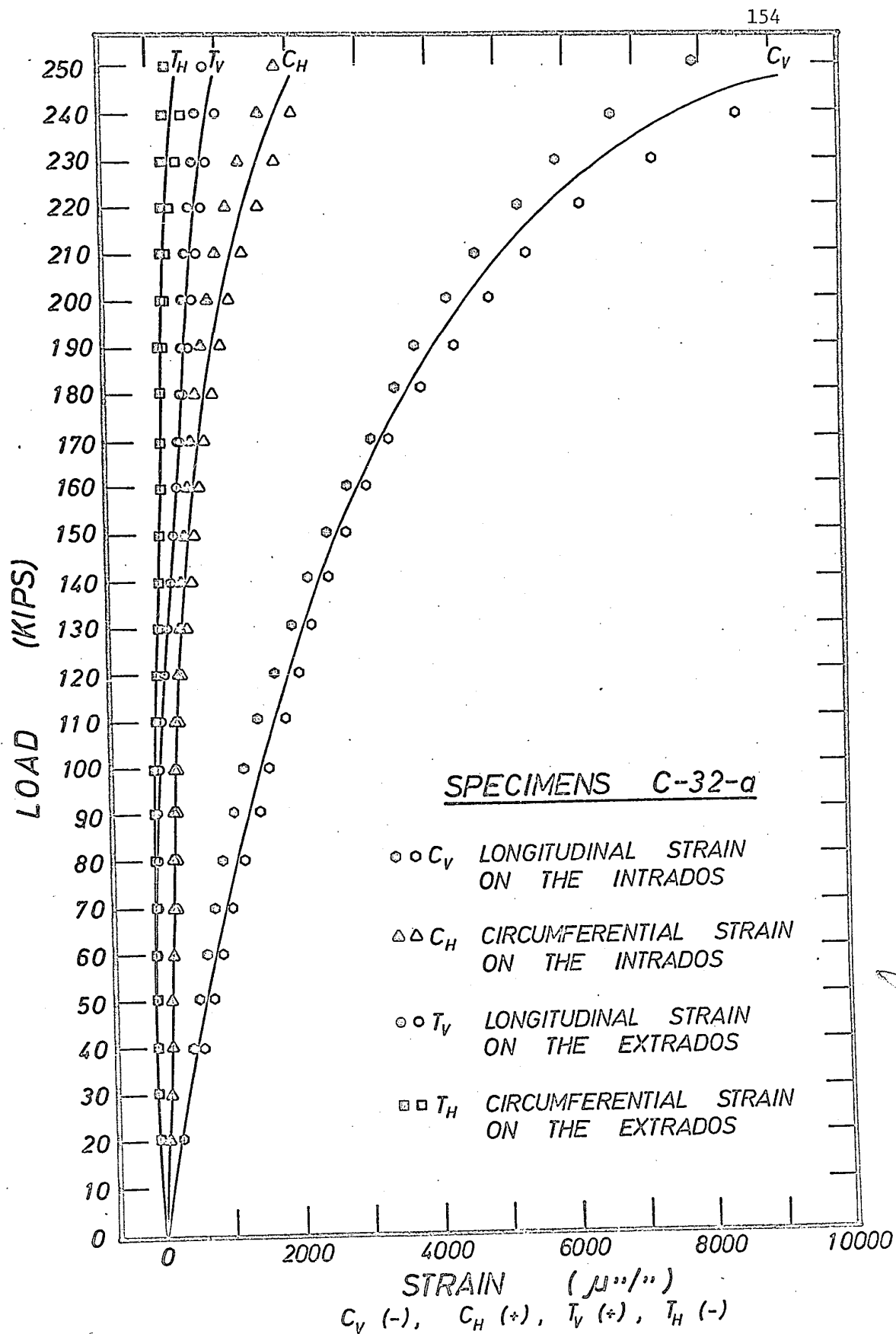


FIG. B-27 LOAD / DEFORMATION CURVE OF SPECIMENS C-32-a

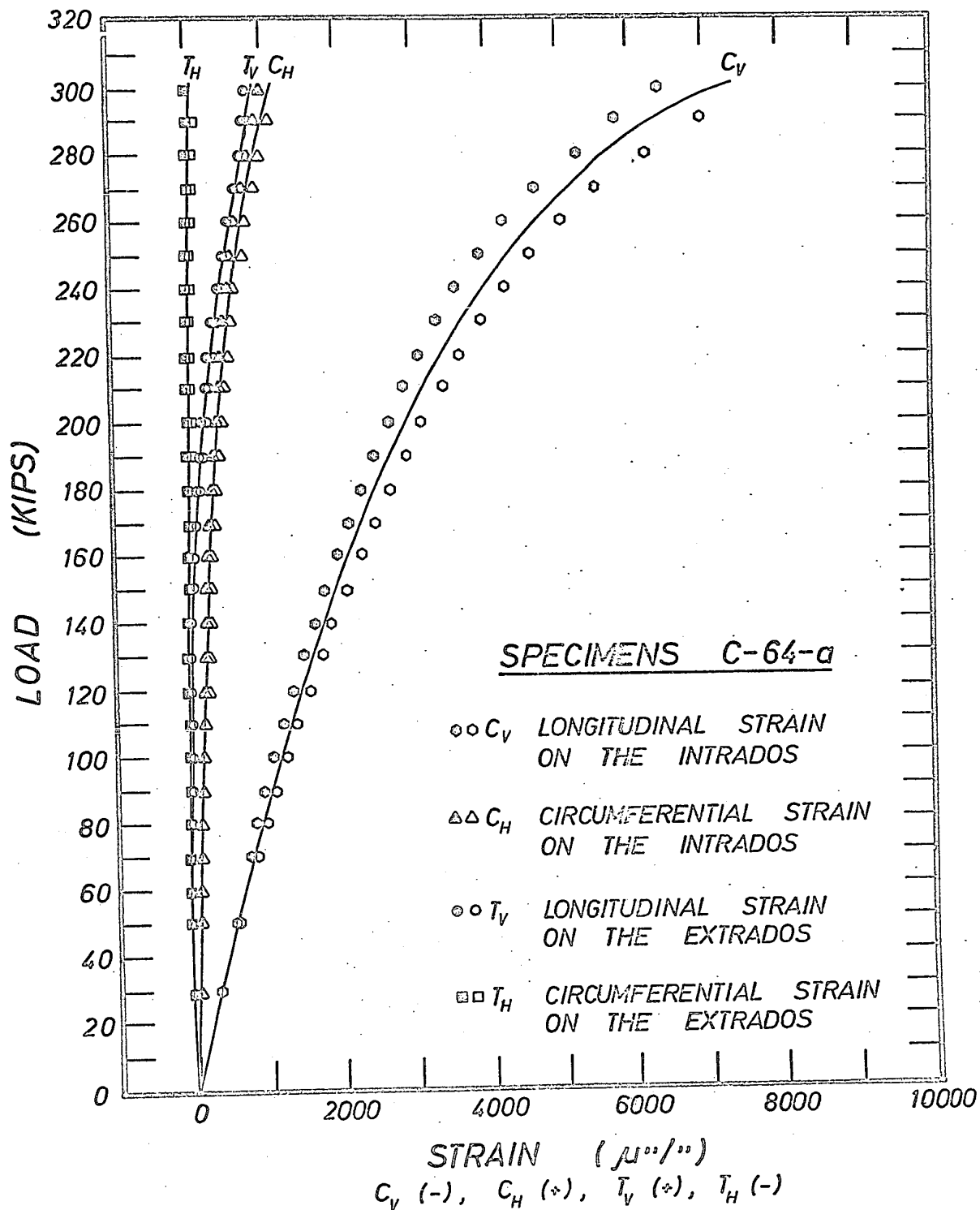


FIG. B-28 LOAD / DEFORMATION CURVE
OF SPECIMENS C-64-a

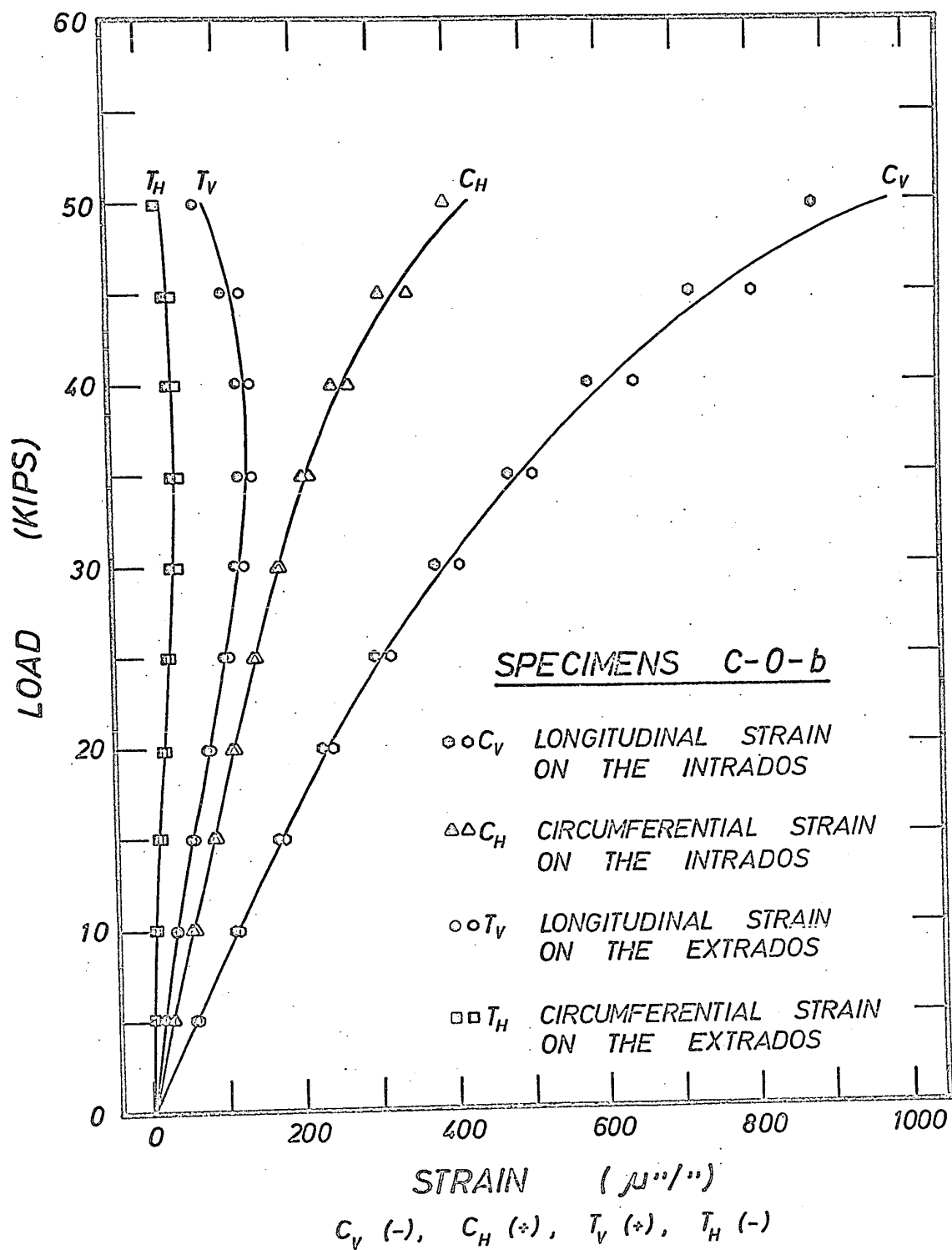


FIG. B-29 LOAD / DEFORMATION CURVE OF SPECIMENS C-0-b

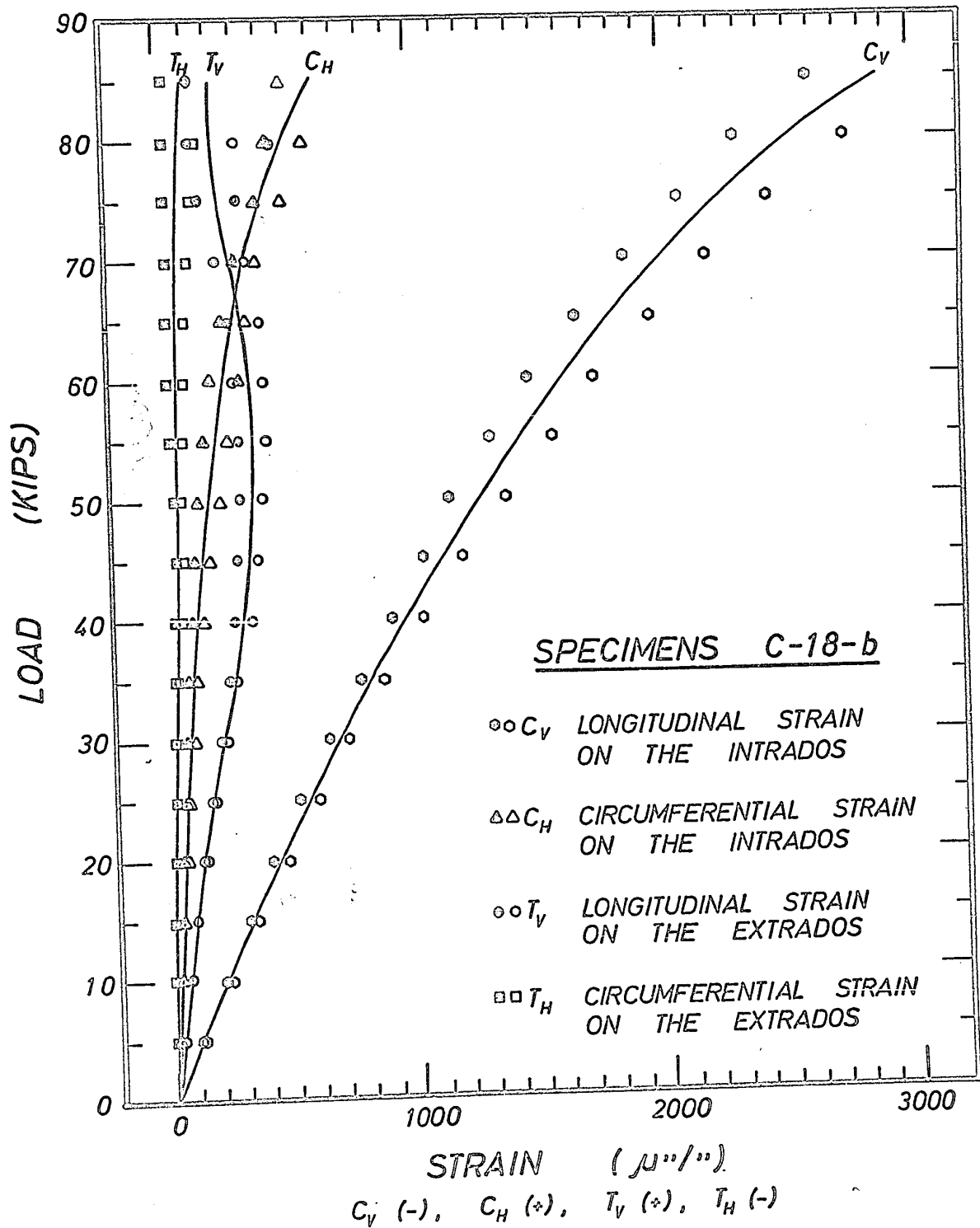


FIG. B-30 LOAD / DEFORMATION CURVE OF SPECIMENS C-18-b

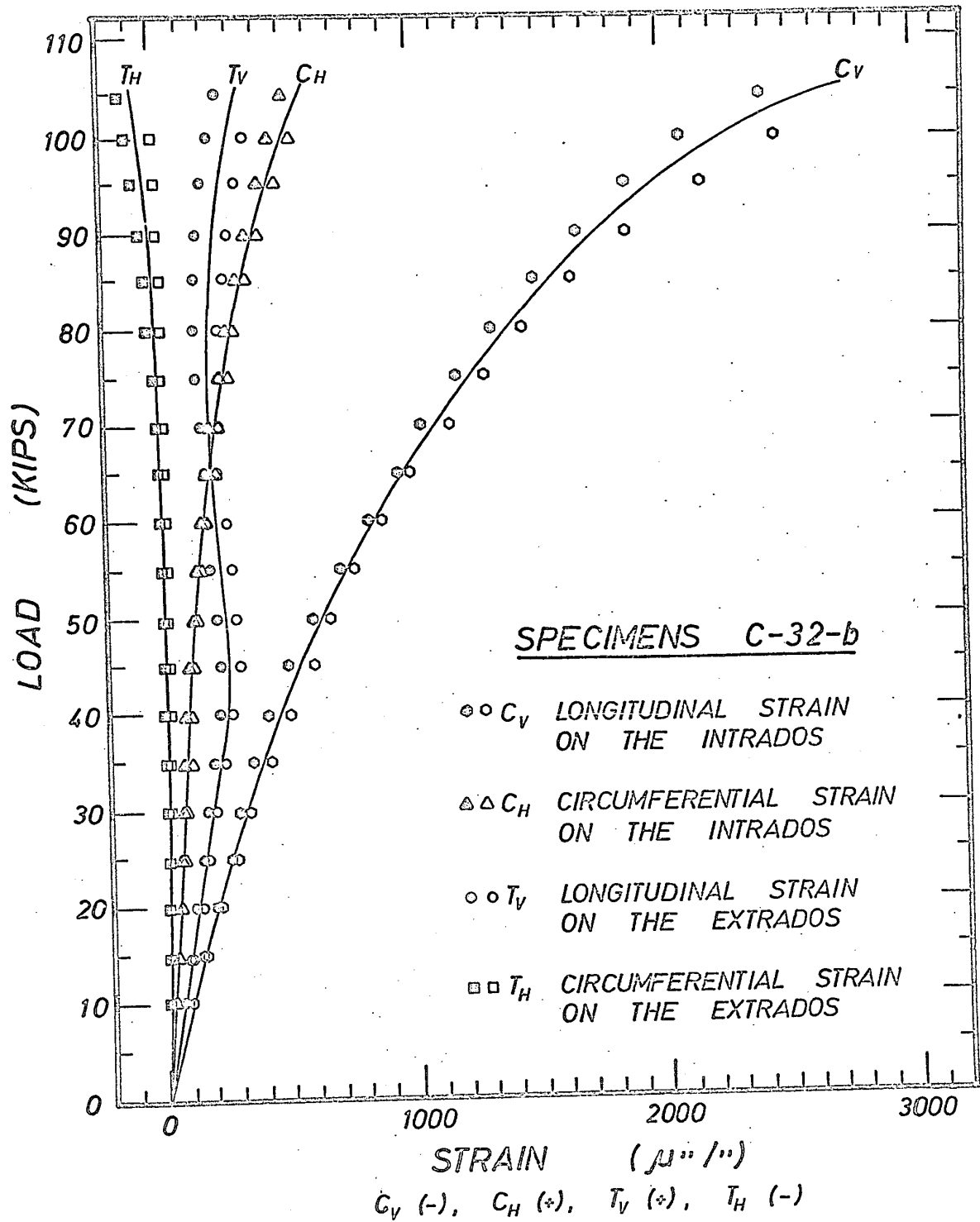


FIG. B-31 LOAD / DEFORMATION CURVE OF SPECIMENS C-32-b

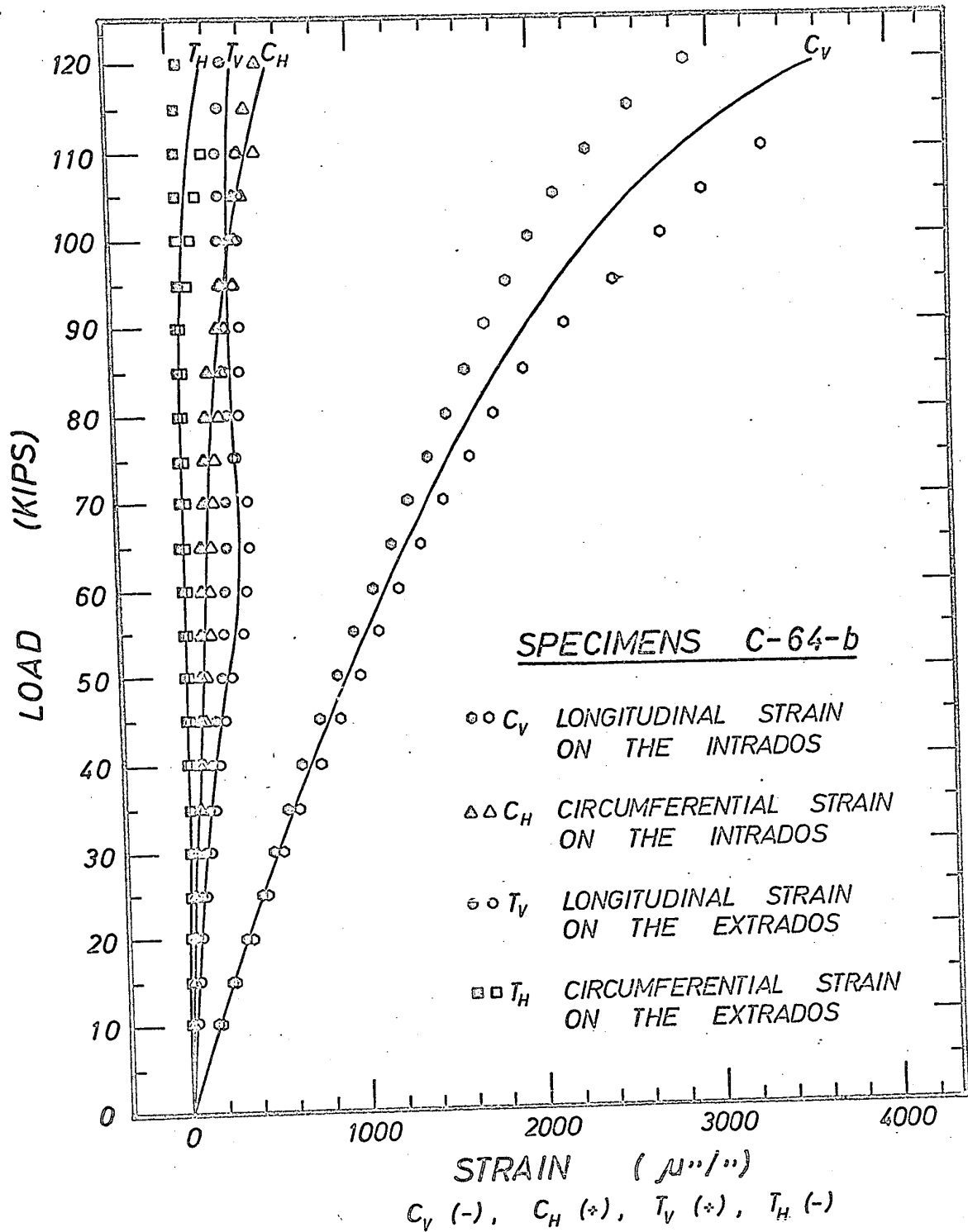


FIG. B-32 LOAD / DEFORMATION CURVE OF SPECIMENS C-64-b

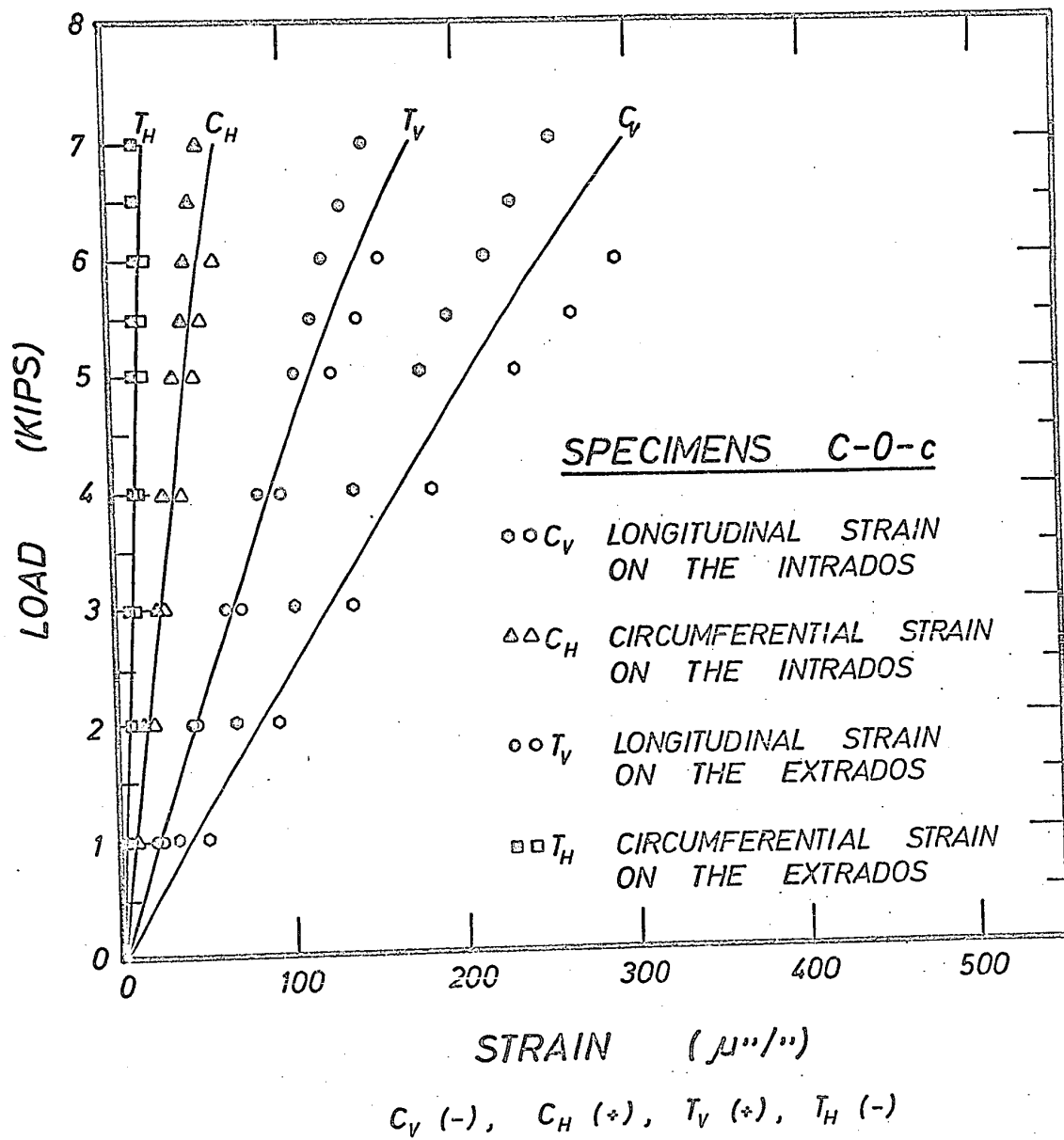


FIG. B-33 LOAD / DEFORMATION CURVE OF SPECIMENS C-0-c

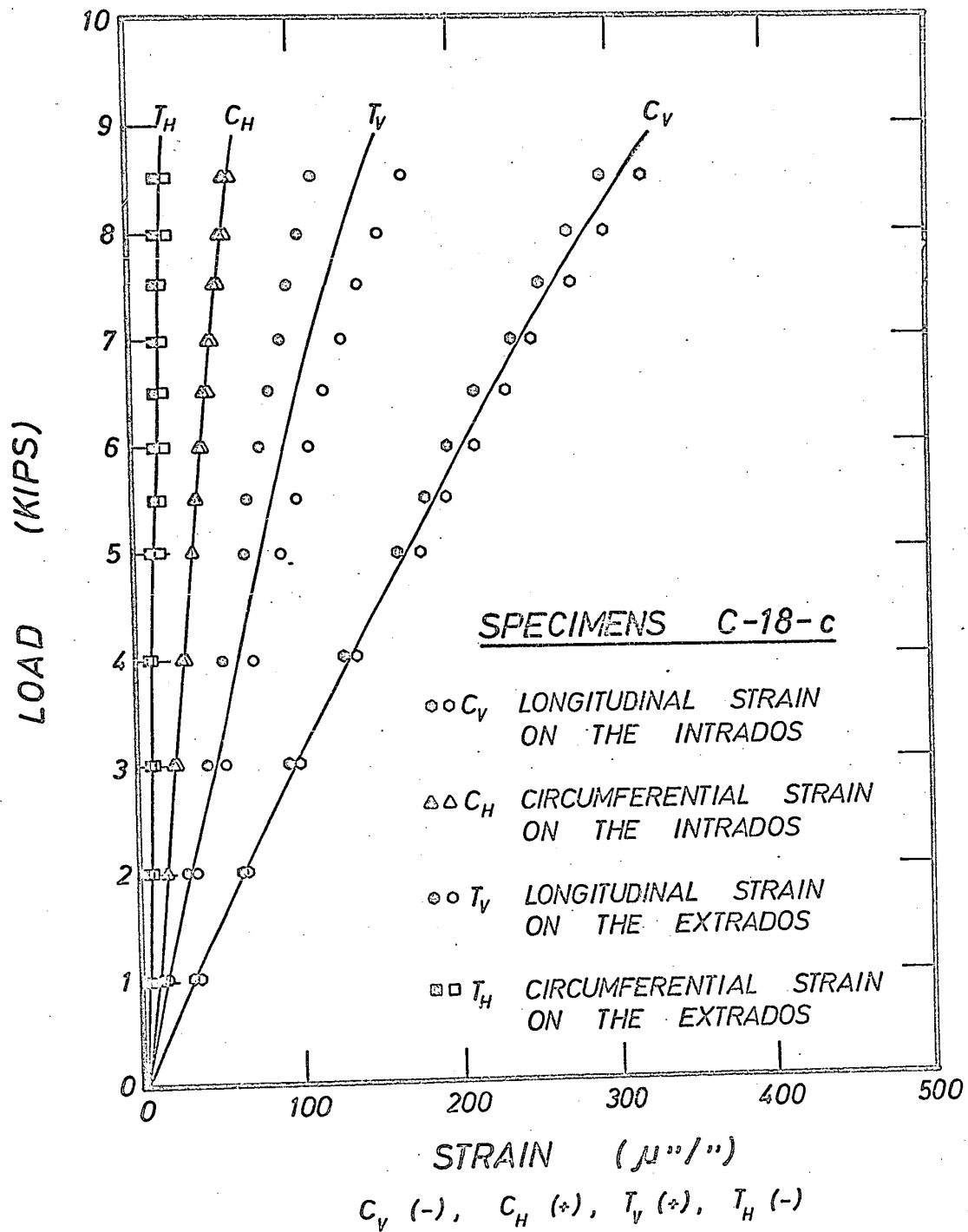


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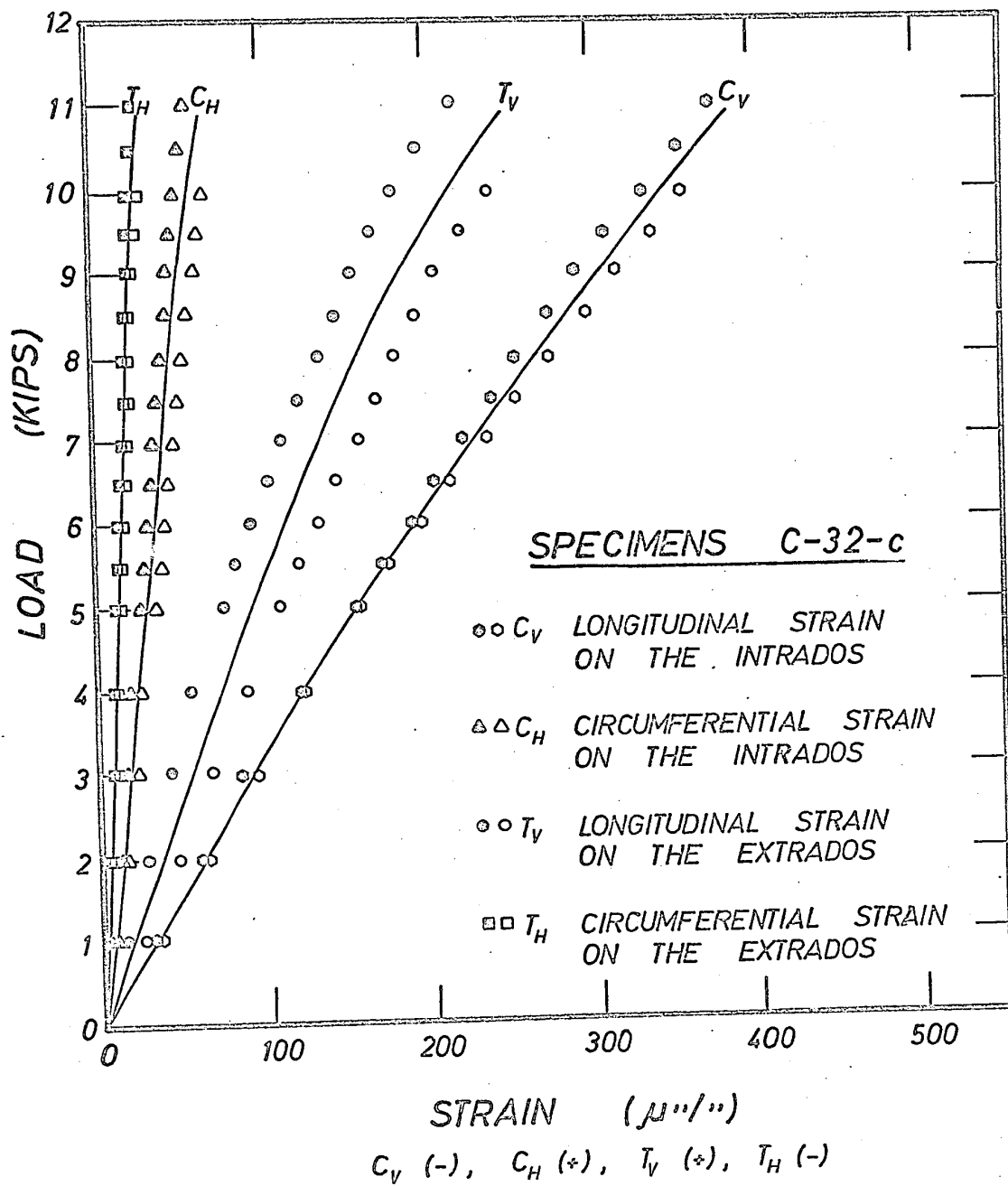


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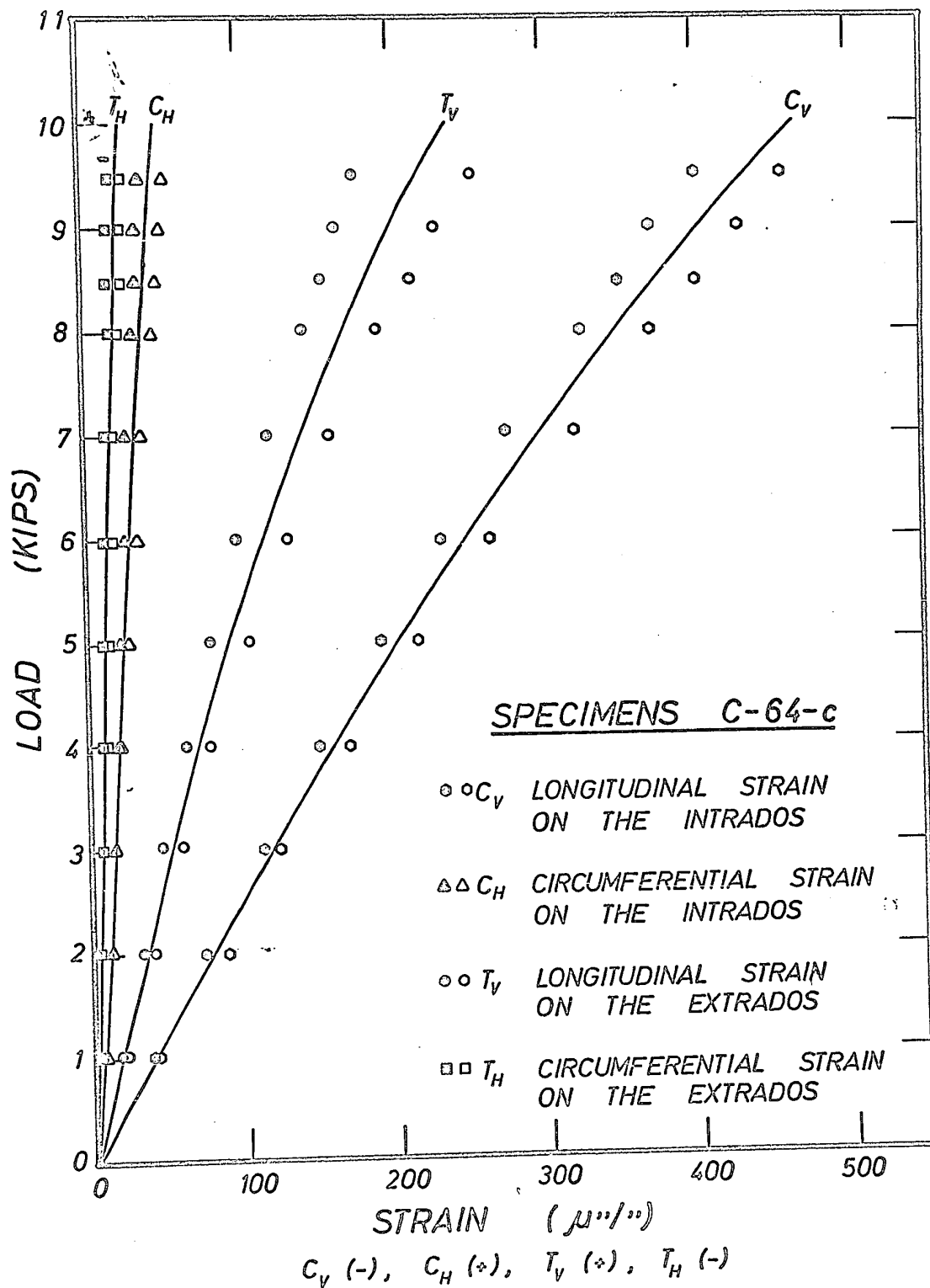


FIG. B-36 LOAD / DEFORMATION CURVE OF SPECIMENS C-64-c

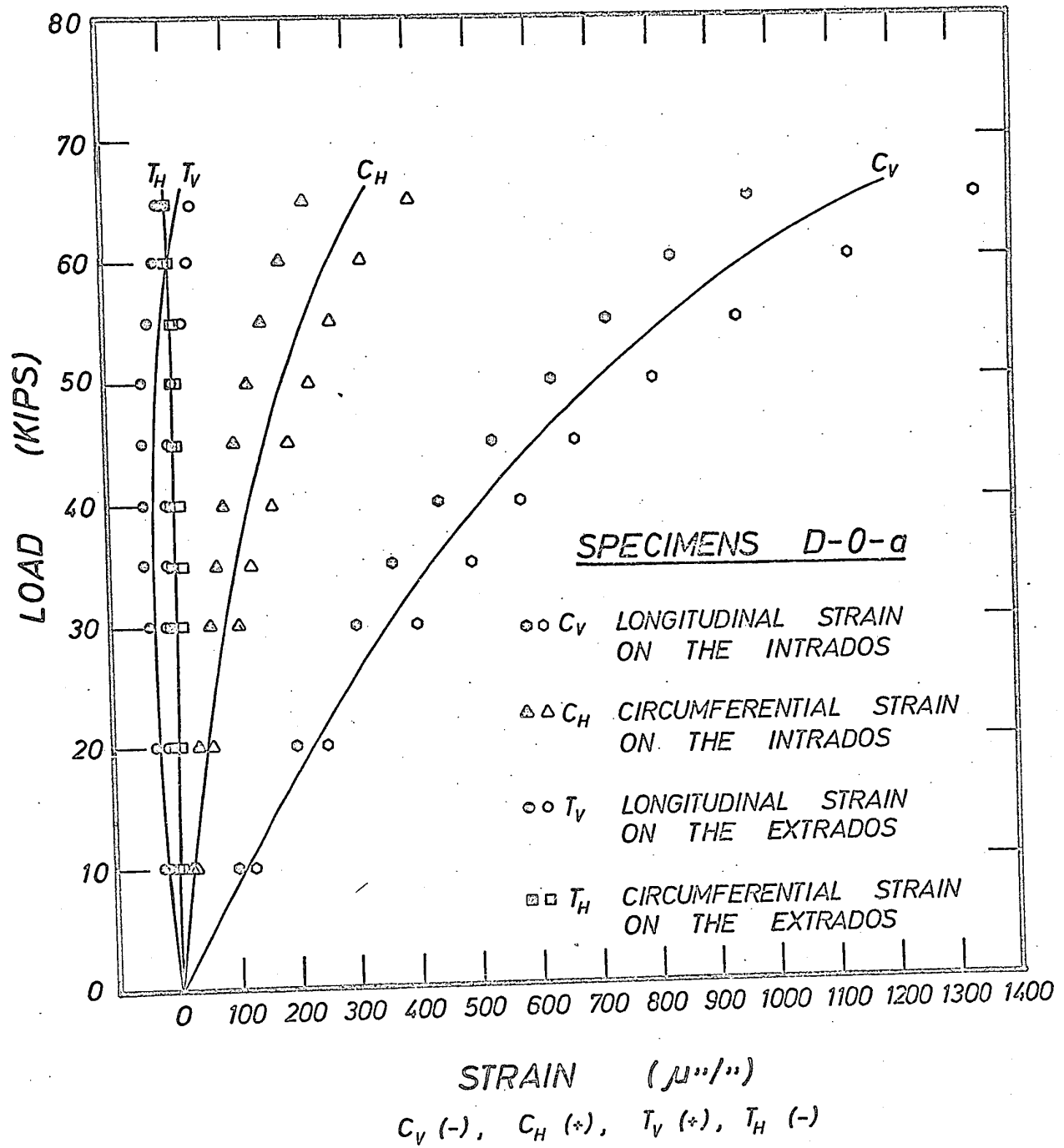


FIG. B-37 LOAD / DEFORMATION CURVE OF SPECIMENS D-0-a

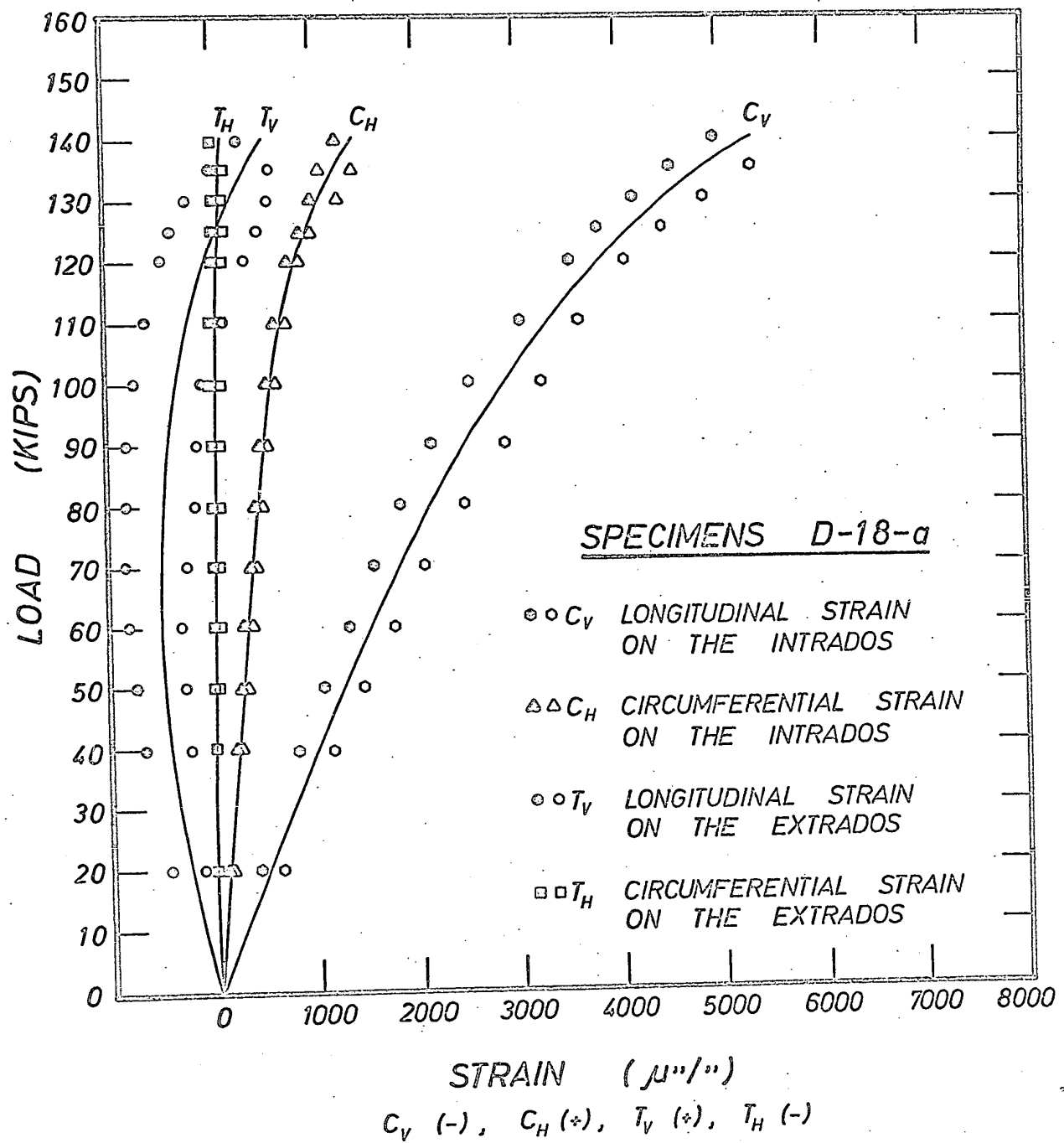


FIG. B-38 LOAD / DEFORMATION CURVE OF SPECIMENS D-18-a

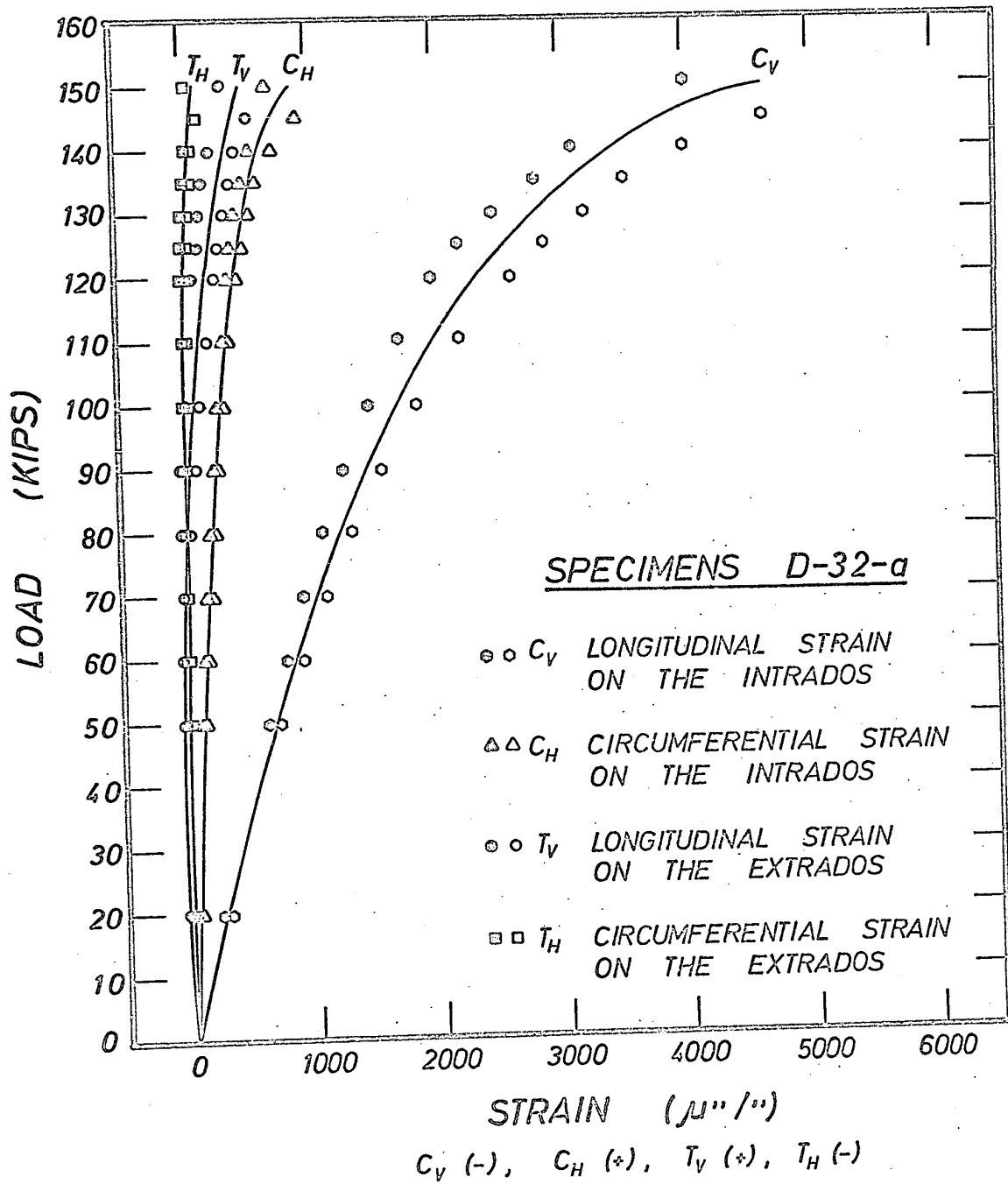


FIG. B-39 LOAD / DEFORMATION CURVE OF SPECIMENS D-32-a

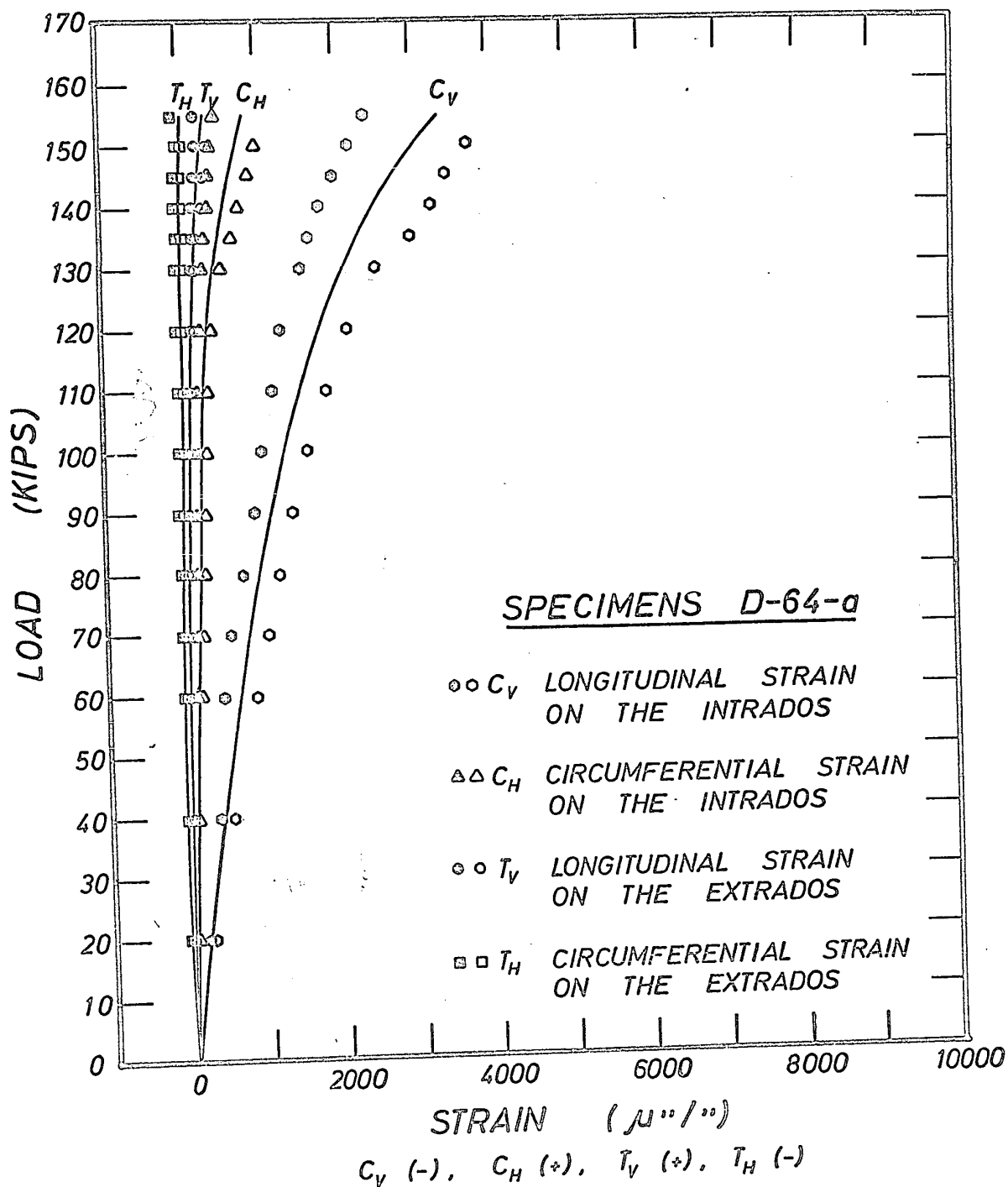


FIG. B-40 LOAD / DEFORMATION CURVE OF SPECIMENS D-64-a

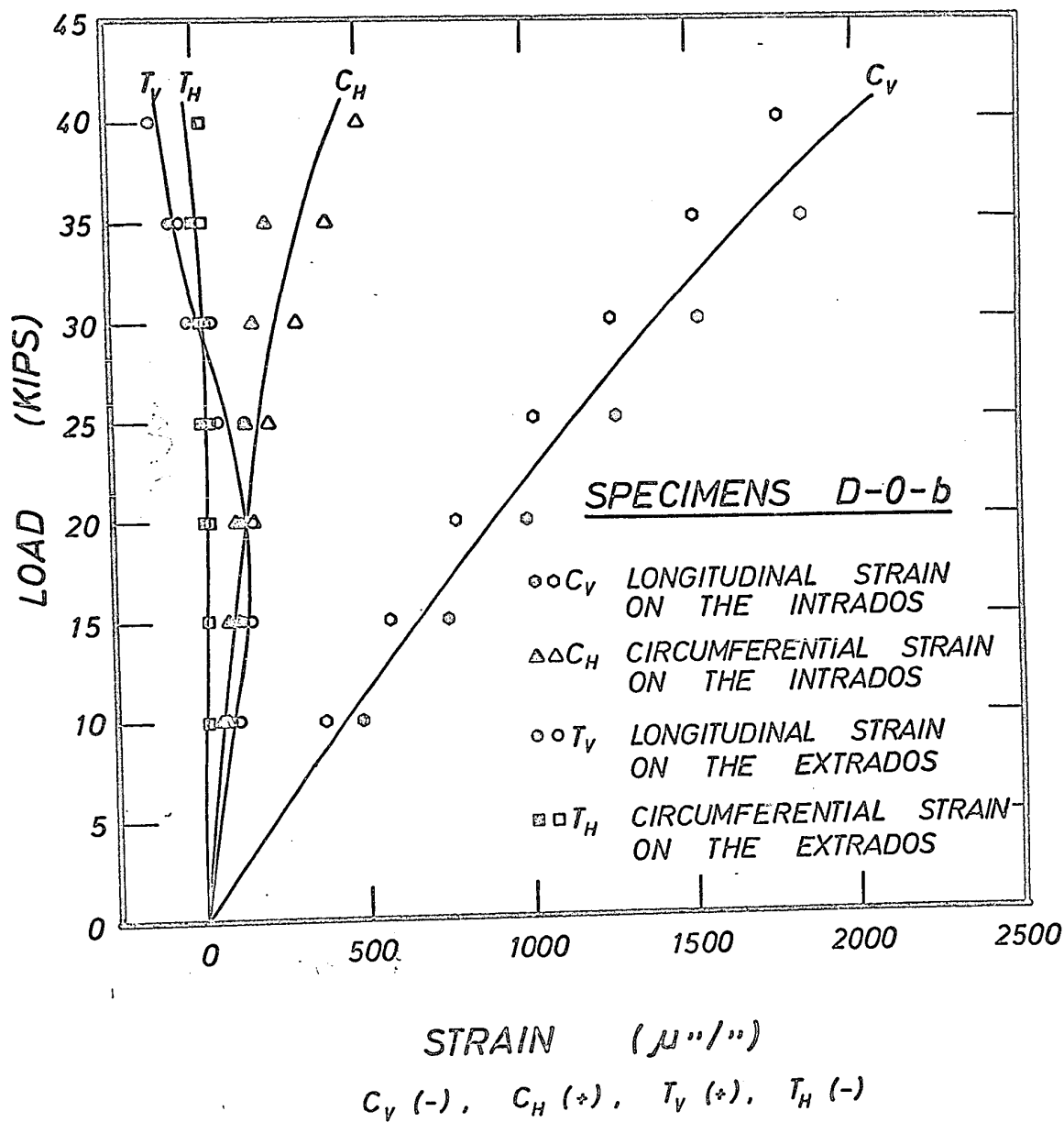


FIG. B-41 LOAD / DEFORMATION CURVE OF SPECIMENS D-0-b

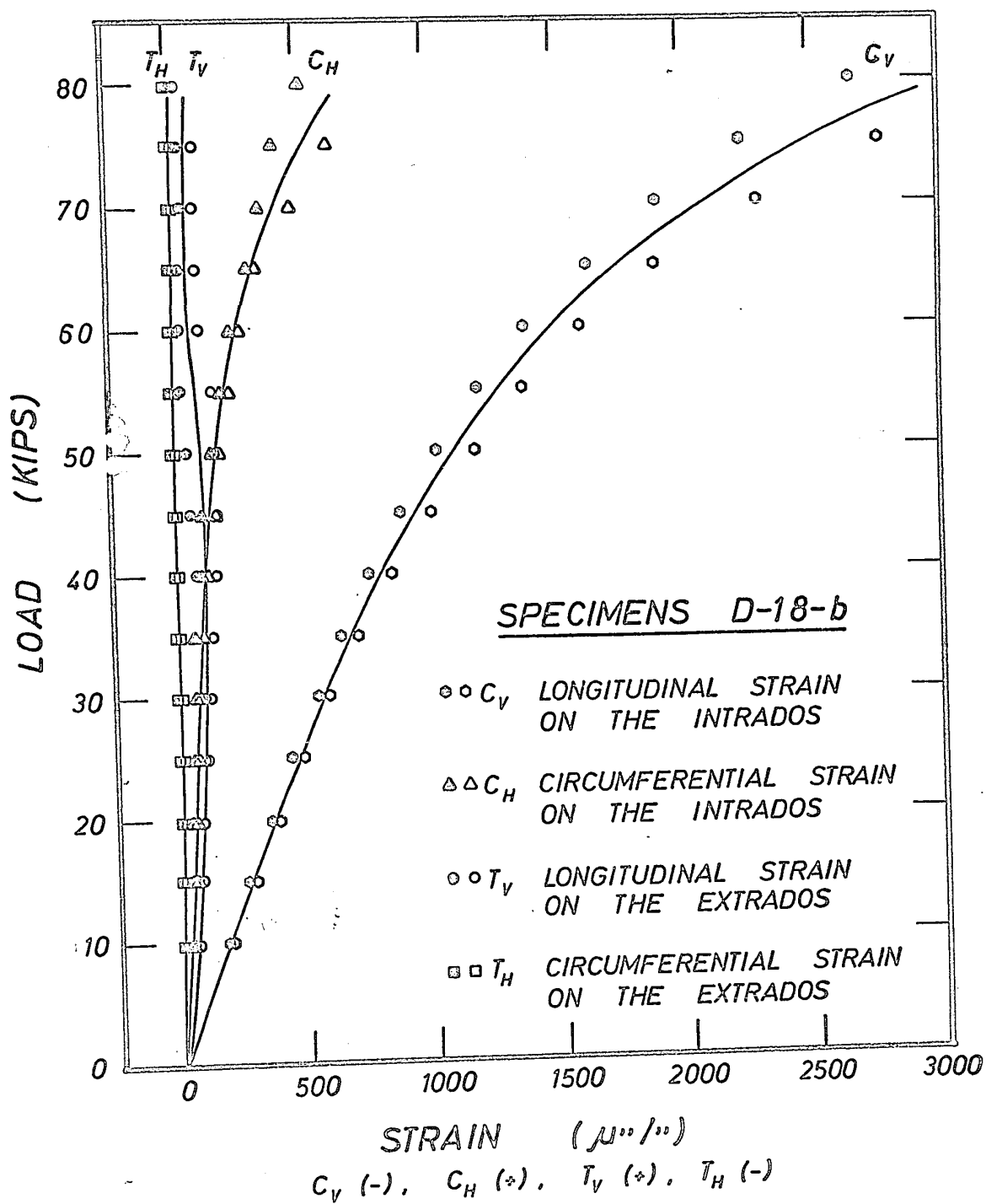


FIG. B-42 LOAD / DEFORMATION CURVE OF SPECIMENS D-18-b

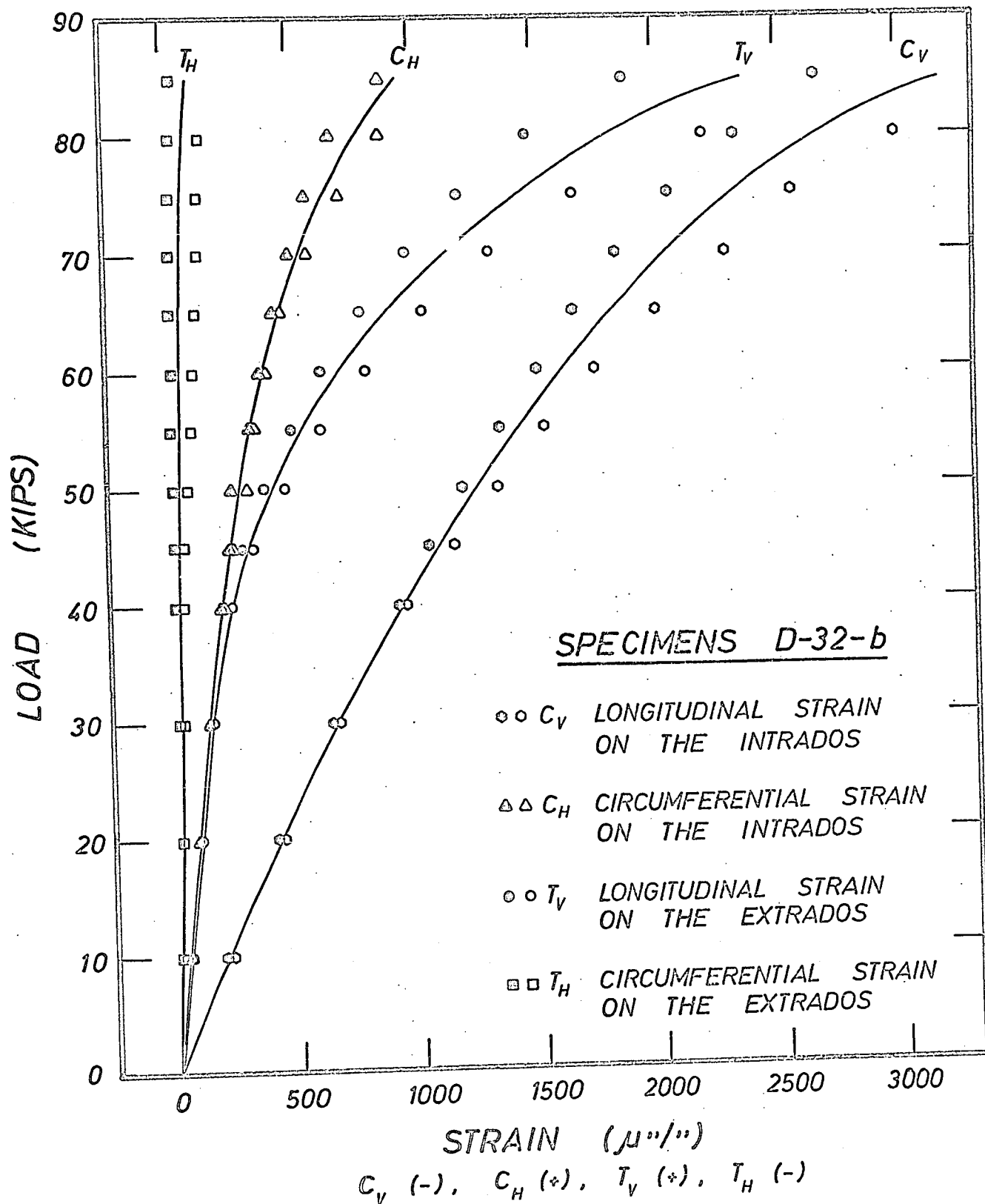


FIG. B.43 LOAD / DEFORMATION CURVE OF SPECIMENS D-32-b

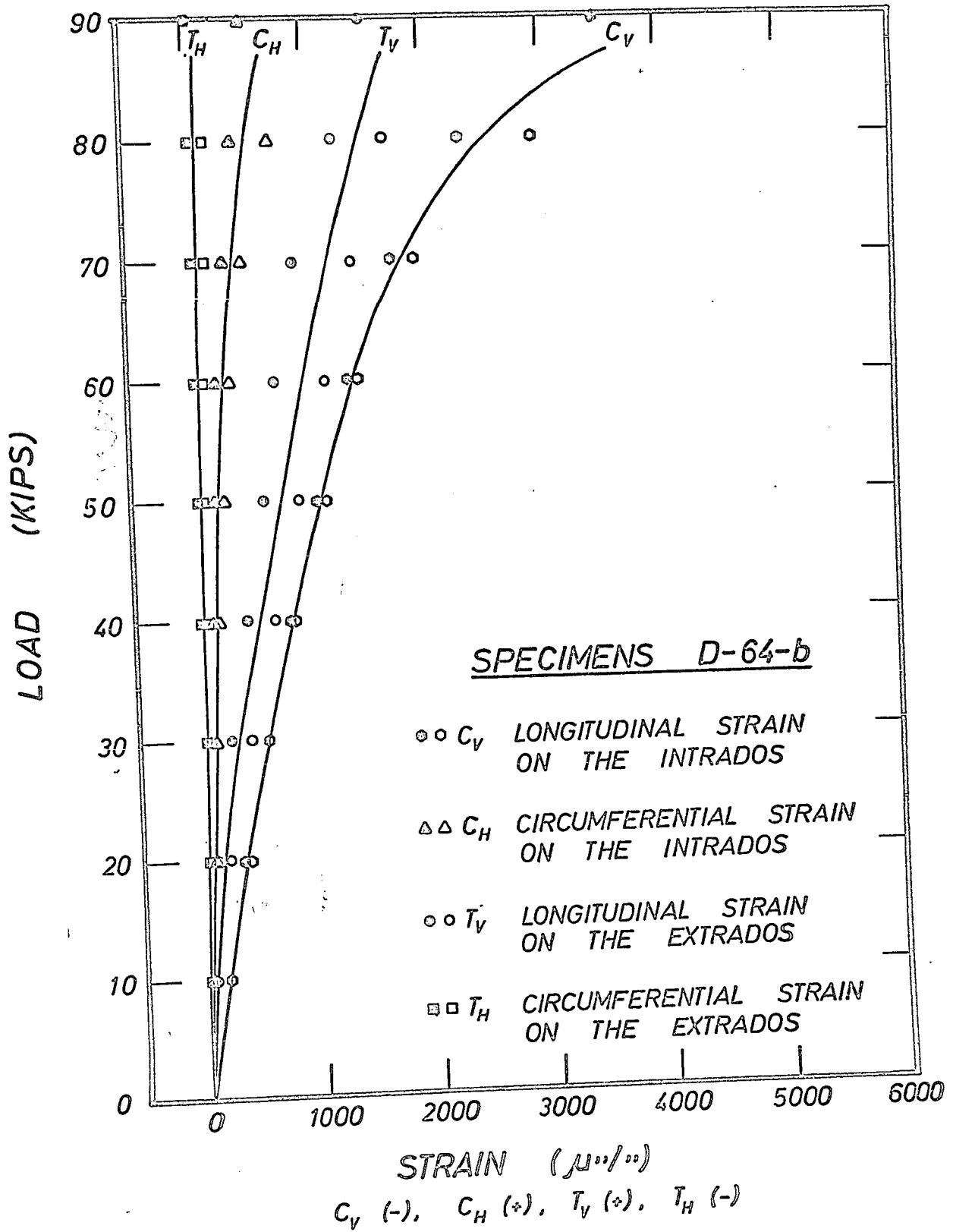


FIG. B-44 LOAD / DEFORMATION CURVE OF SPECIMENS D-64-b

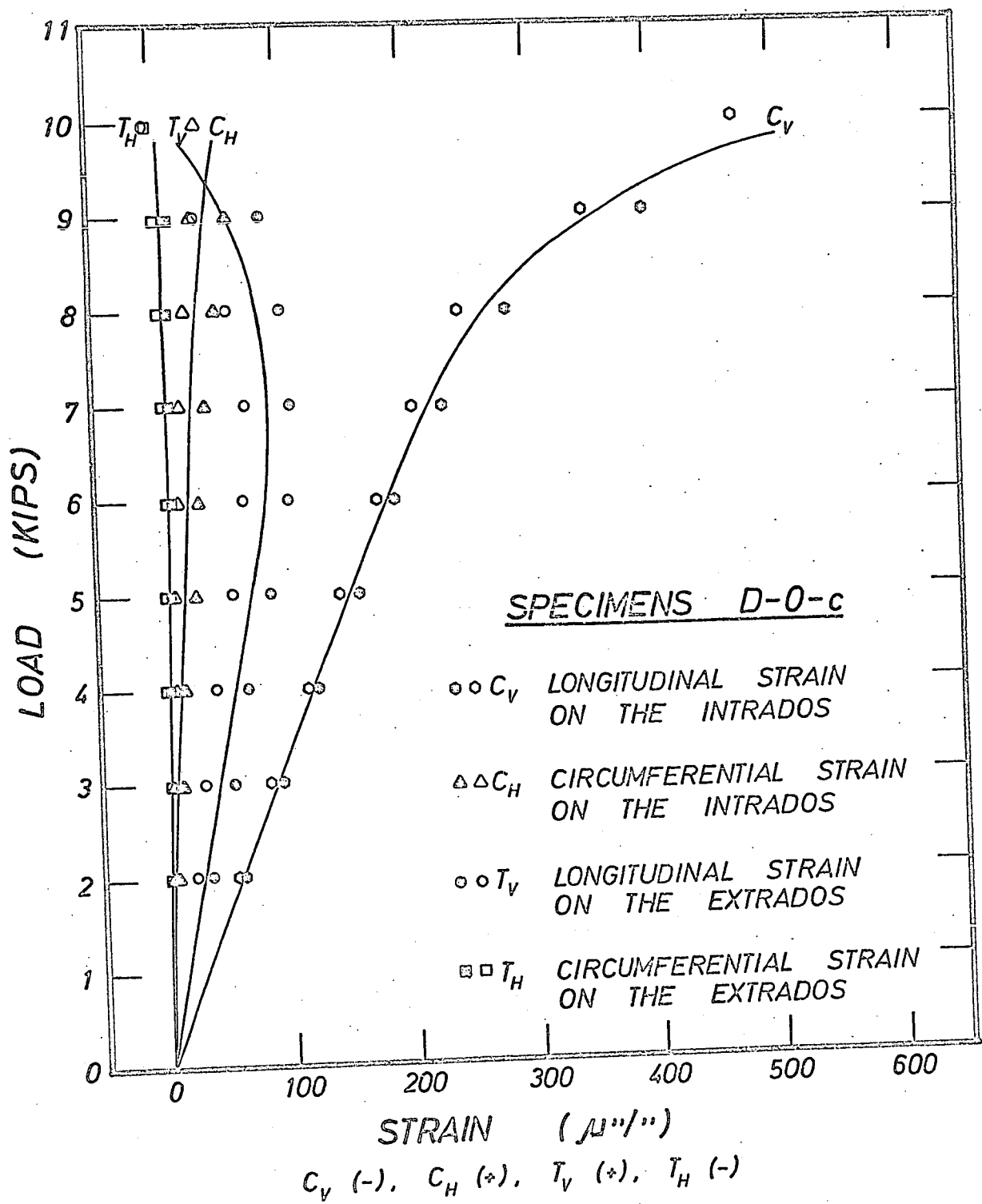


FIG. B.45 LOAD / DEFORMATION CURVE OF SPECIMENS D-0-c

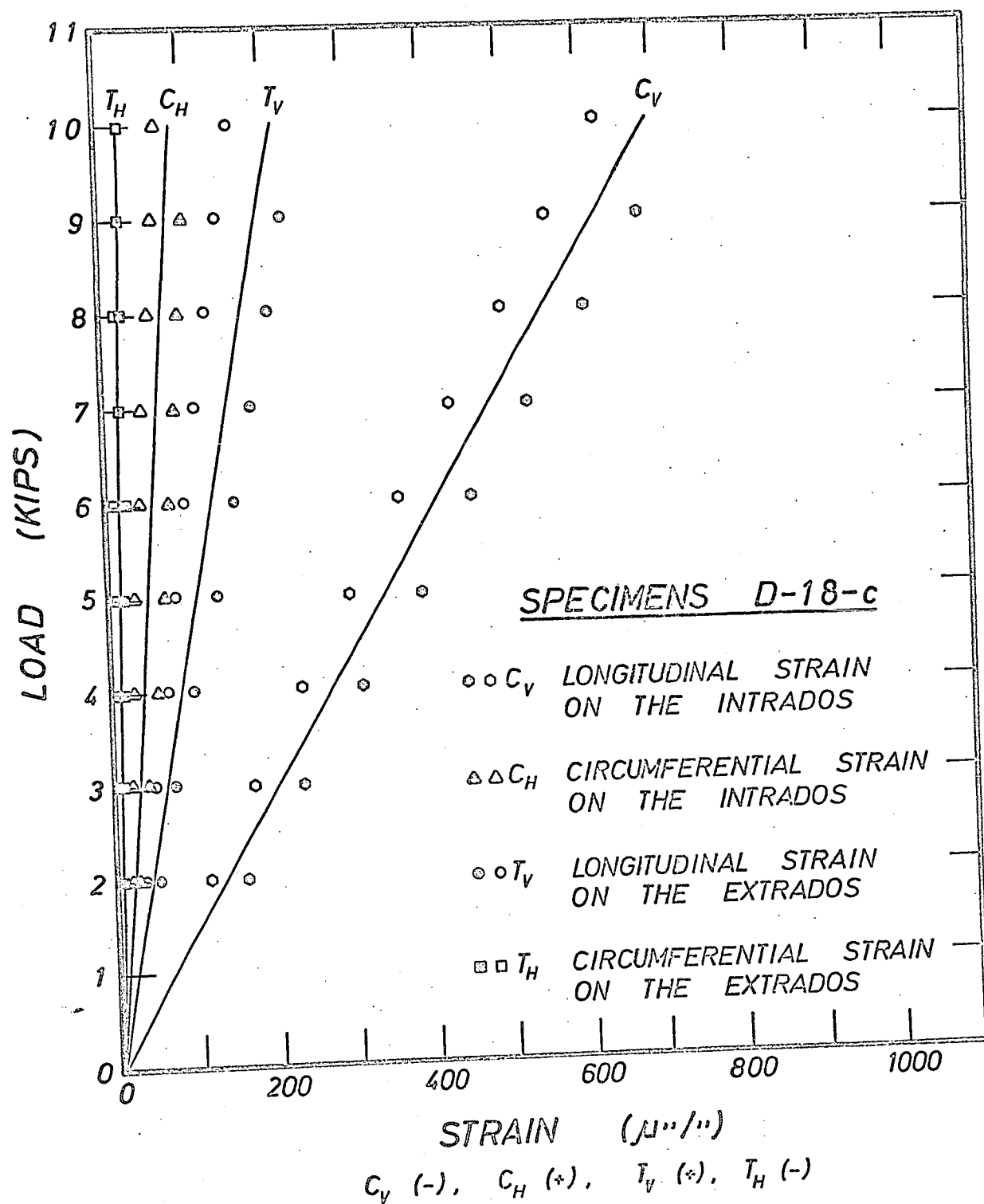


FIG. B.46 LOAD / DEFORMATION CURVE OF SPECIMENS D-18-c

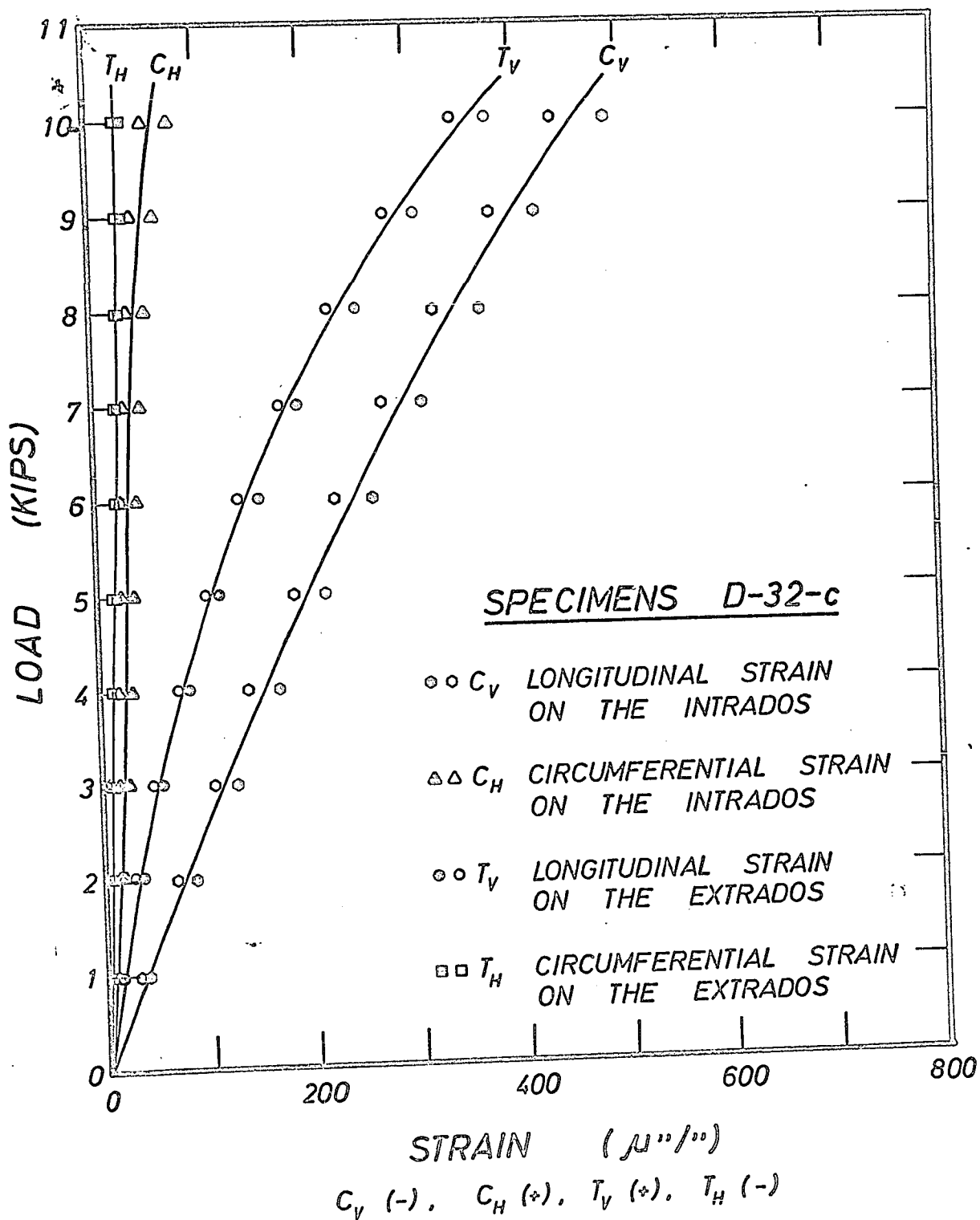


FIG. B.47 LOAD / DEFORMATION CURVE OF SPECIMENS D-32-c

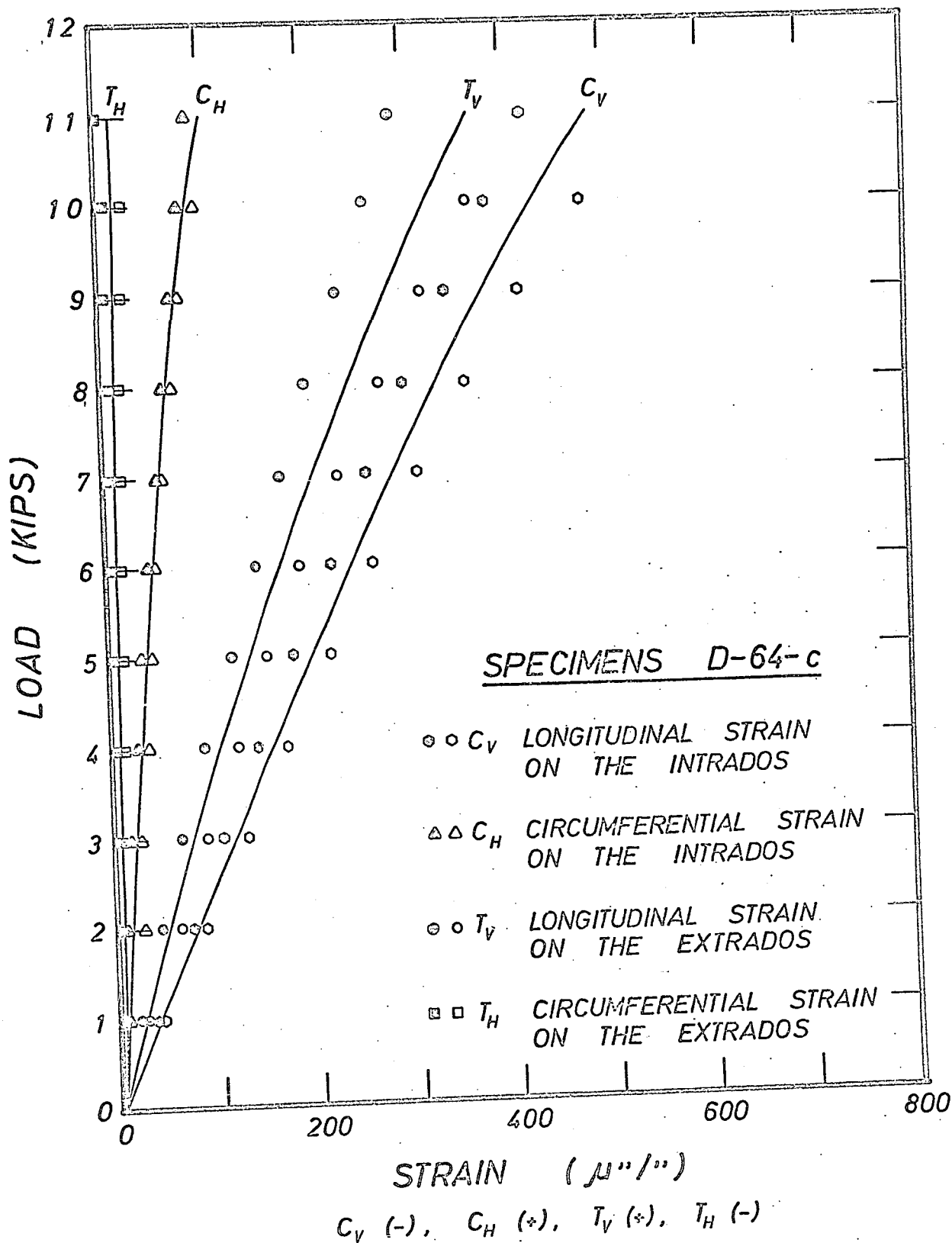


FIG. B-48 LOAD / DEFORMATION CURVE OF SPECIMENS D-64-c

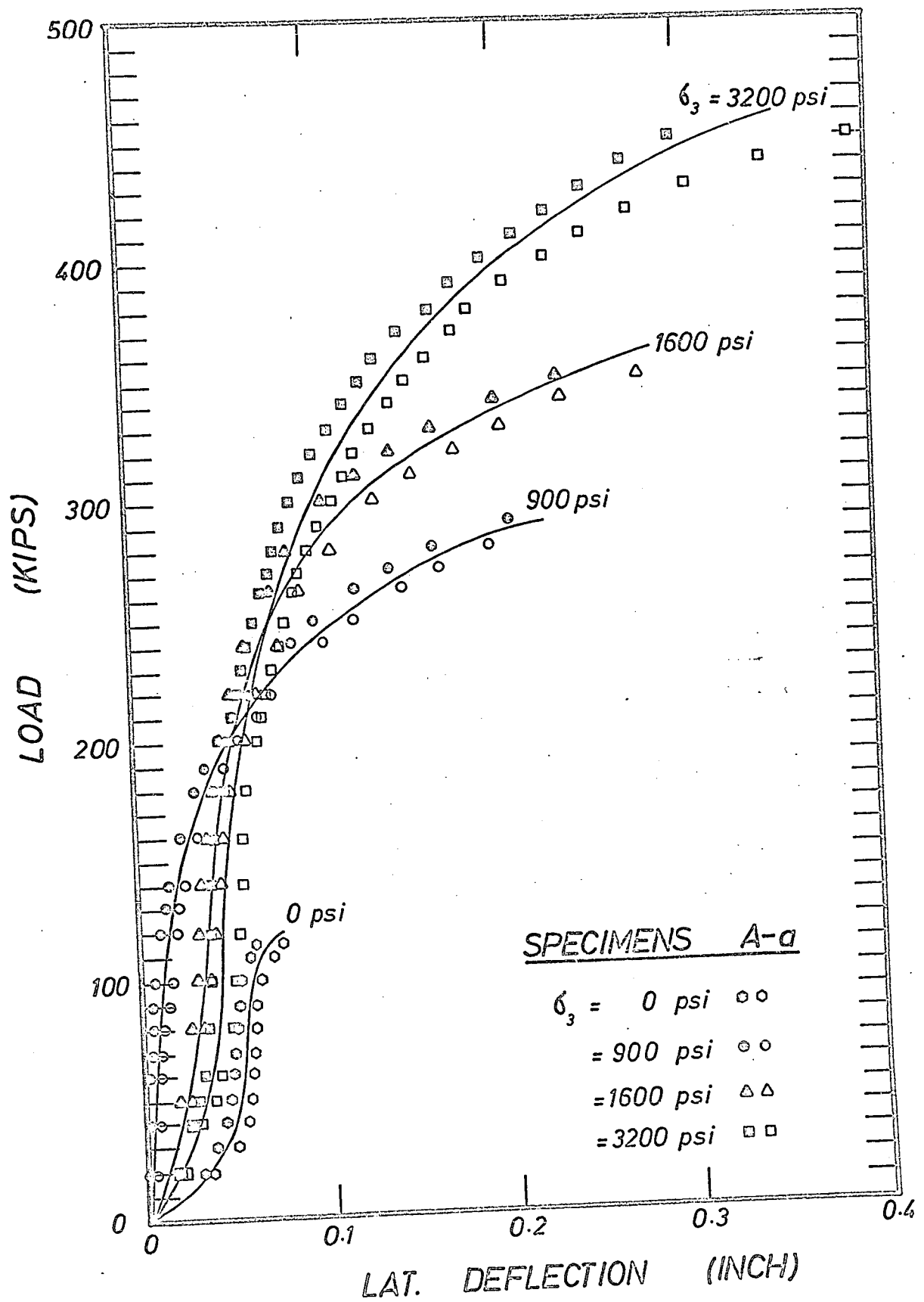


FIG. B-49 LOAD / LAT. DEFLECTION CURVE
OF SPECIMENS A-a

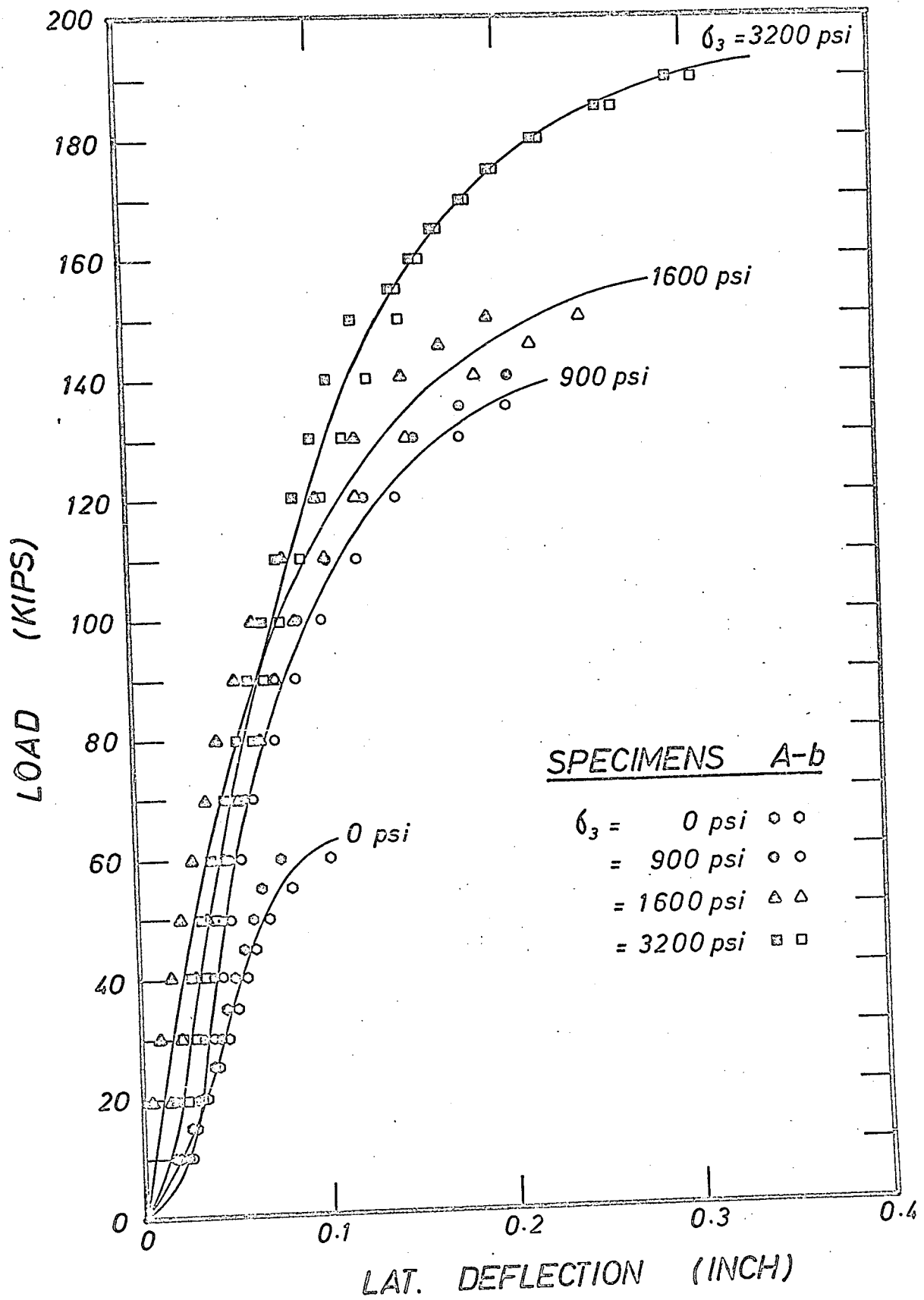


FIG. B-50 LOAD/LAT. DEFLECTION CURVE
OF SPECIMENS A-b

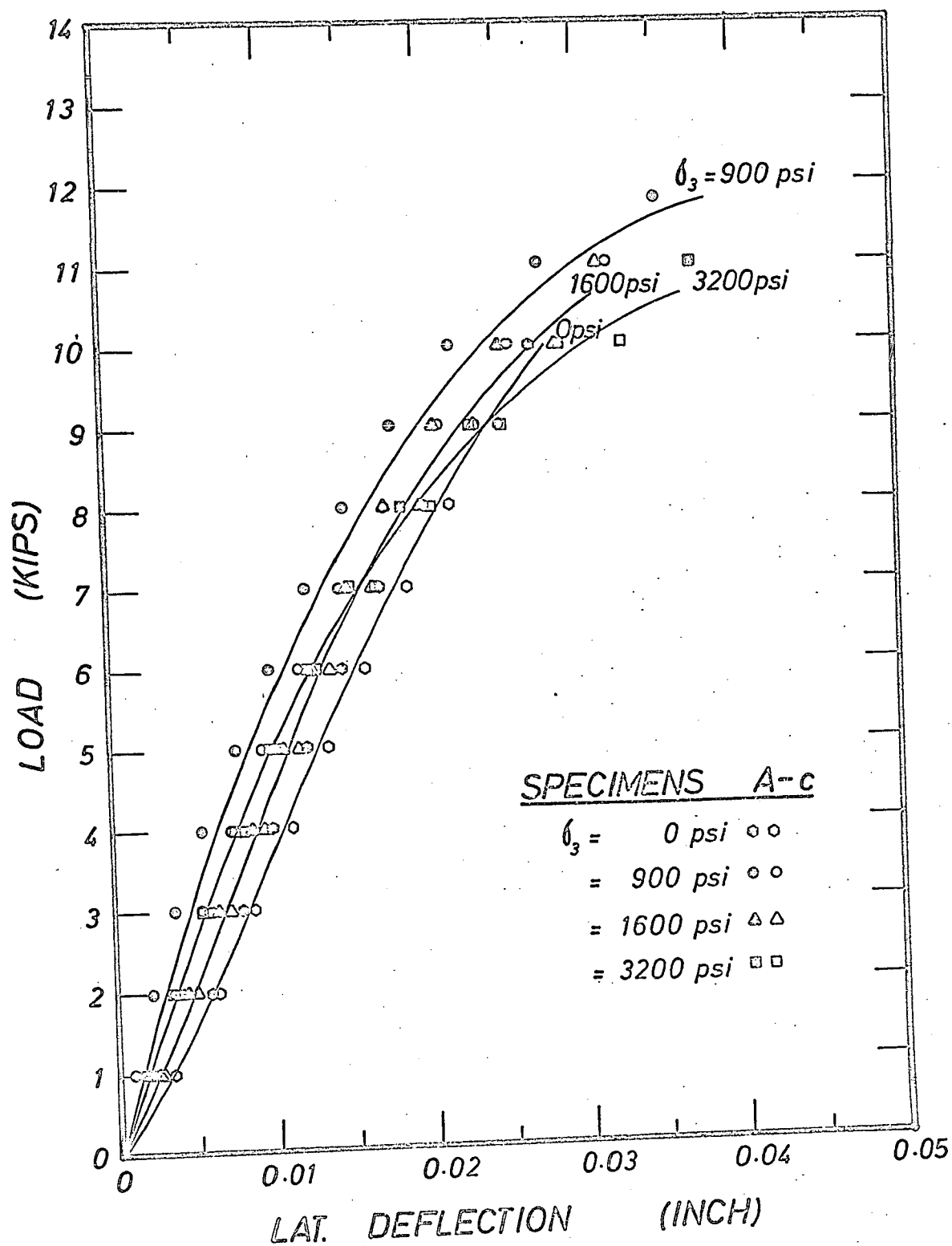


FIG. B-51 LOAD/LAT. DEFLECTION CURVE
OF SPECIMENS A-C

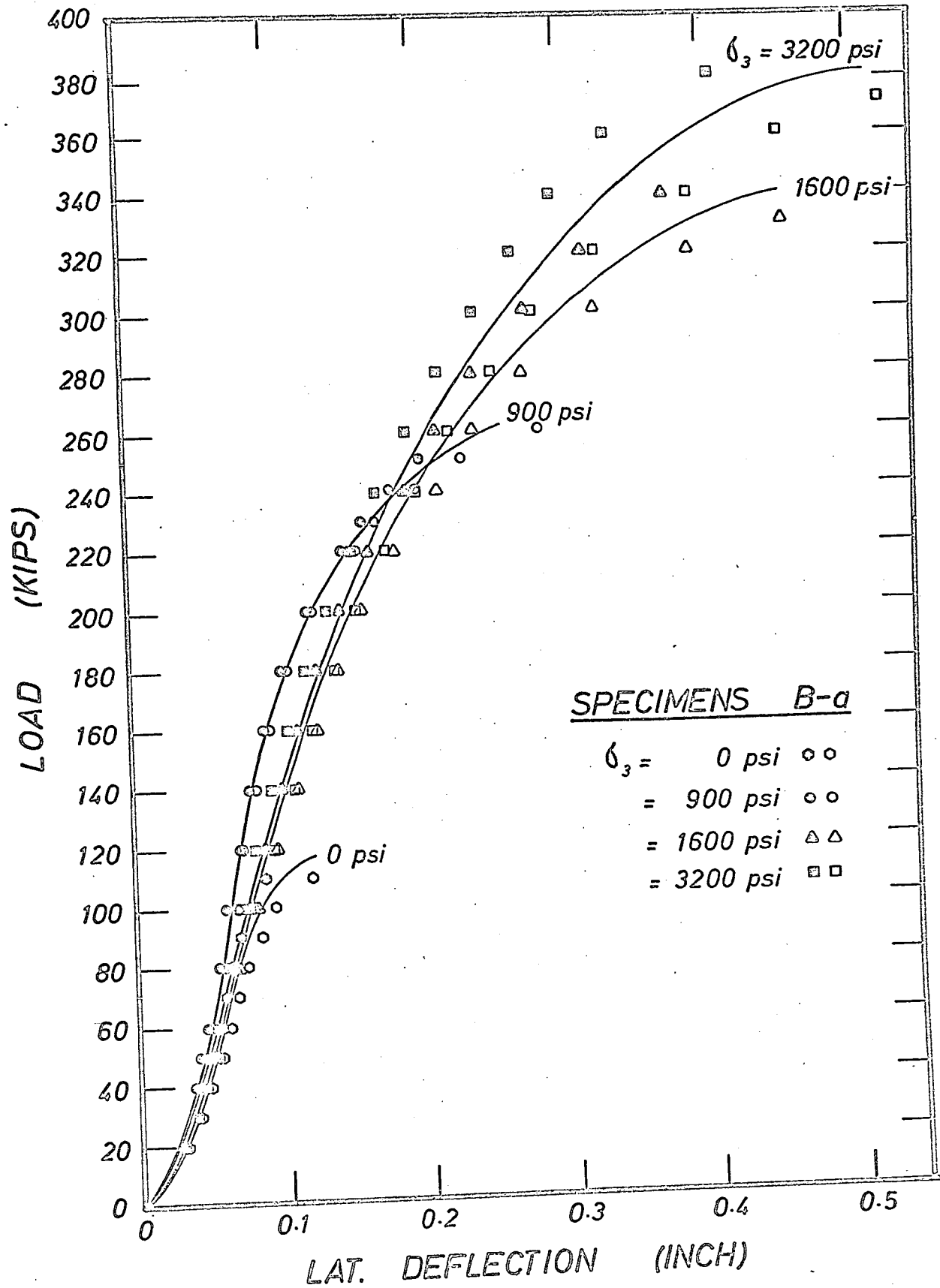


FIG. B-52 LOAD / LAT. DEFLECTION CURVE OF SPECIMENS B-a

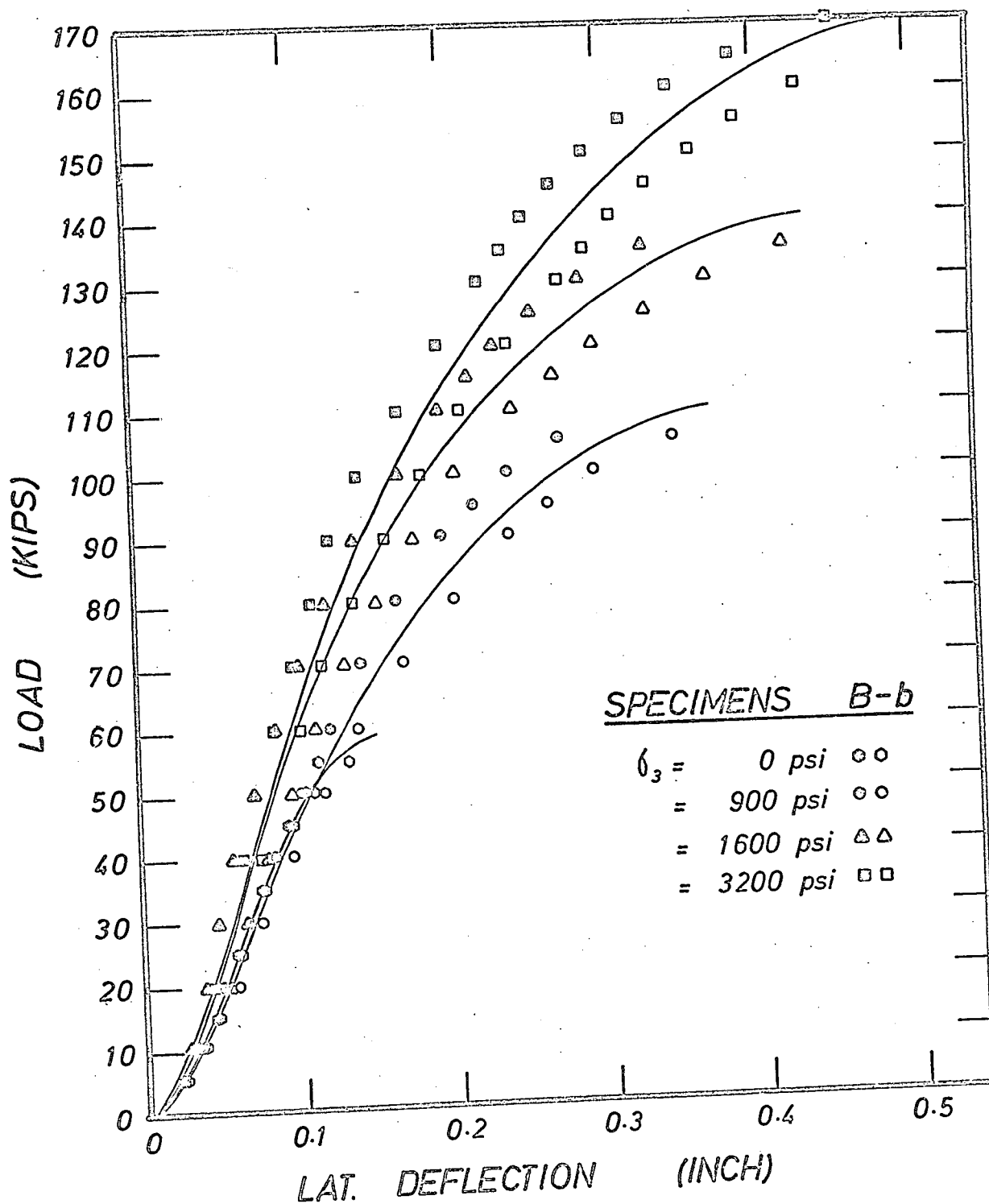


FIG. B-53 LOAD/LAT. DEFLECTION CURVE
 OF SPECIMENS B-b

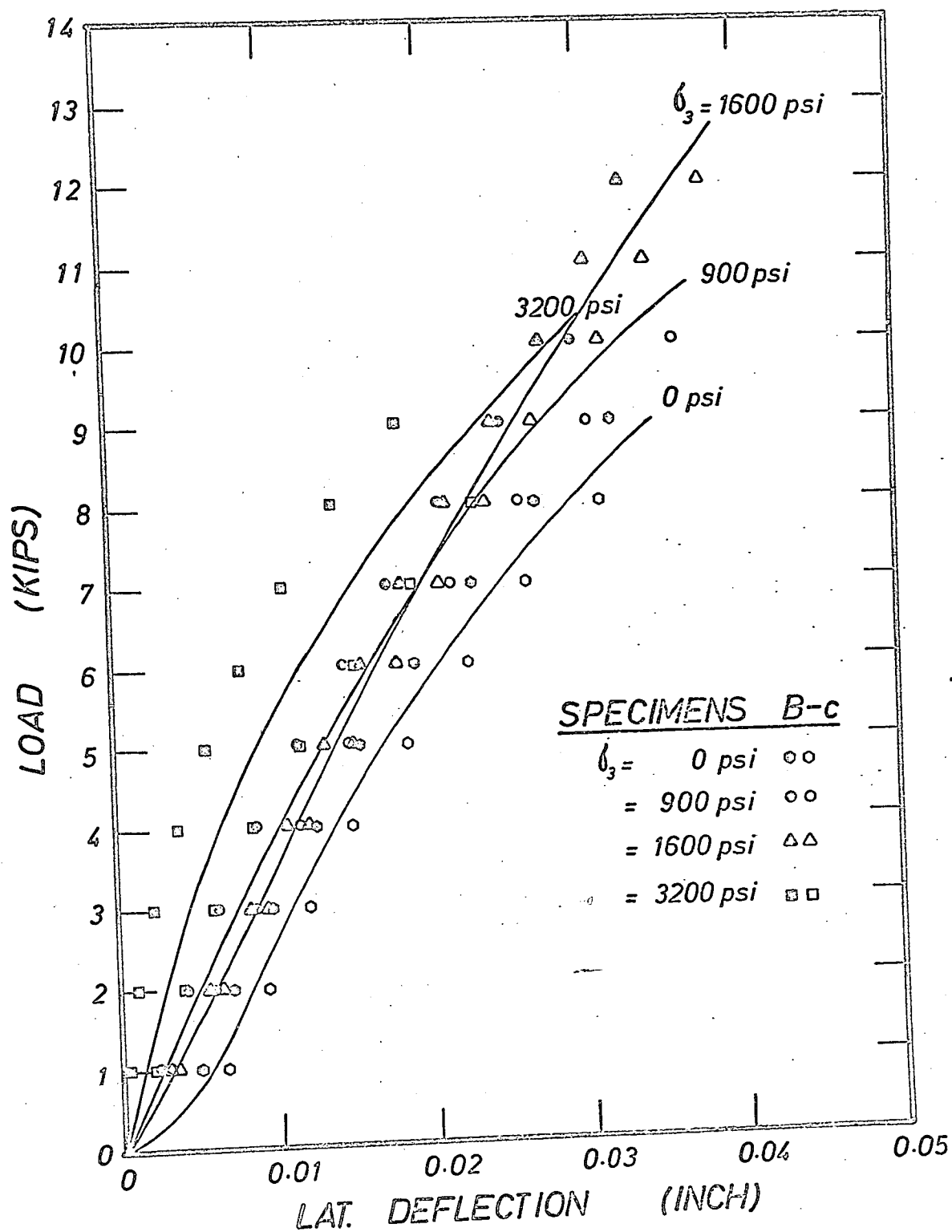


FIG. B-54 LOAD / LAT. DEFLECTION CURVE
OF SPECIMENS B-c

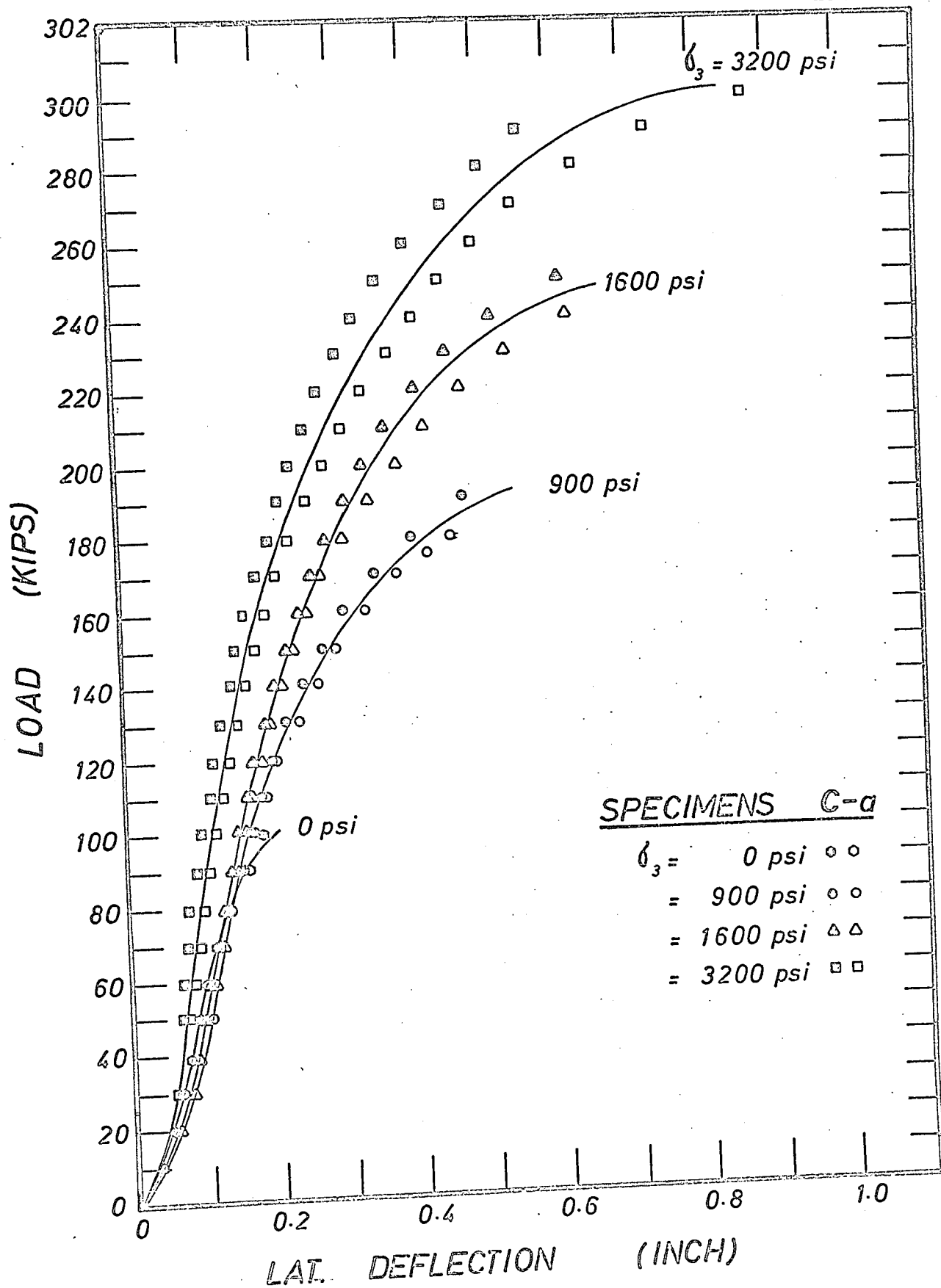


FIG. B-55 LOAD/LAT. DEFLECTION CURVE OF SPECIMENS C-a

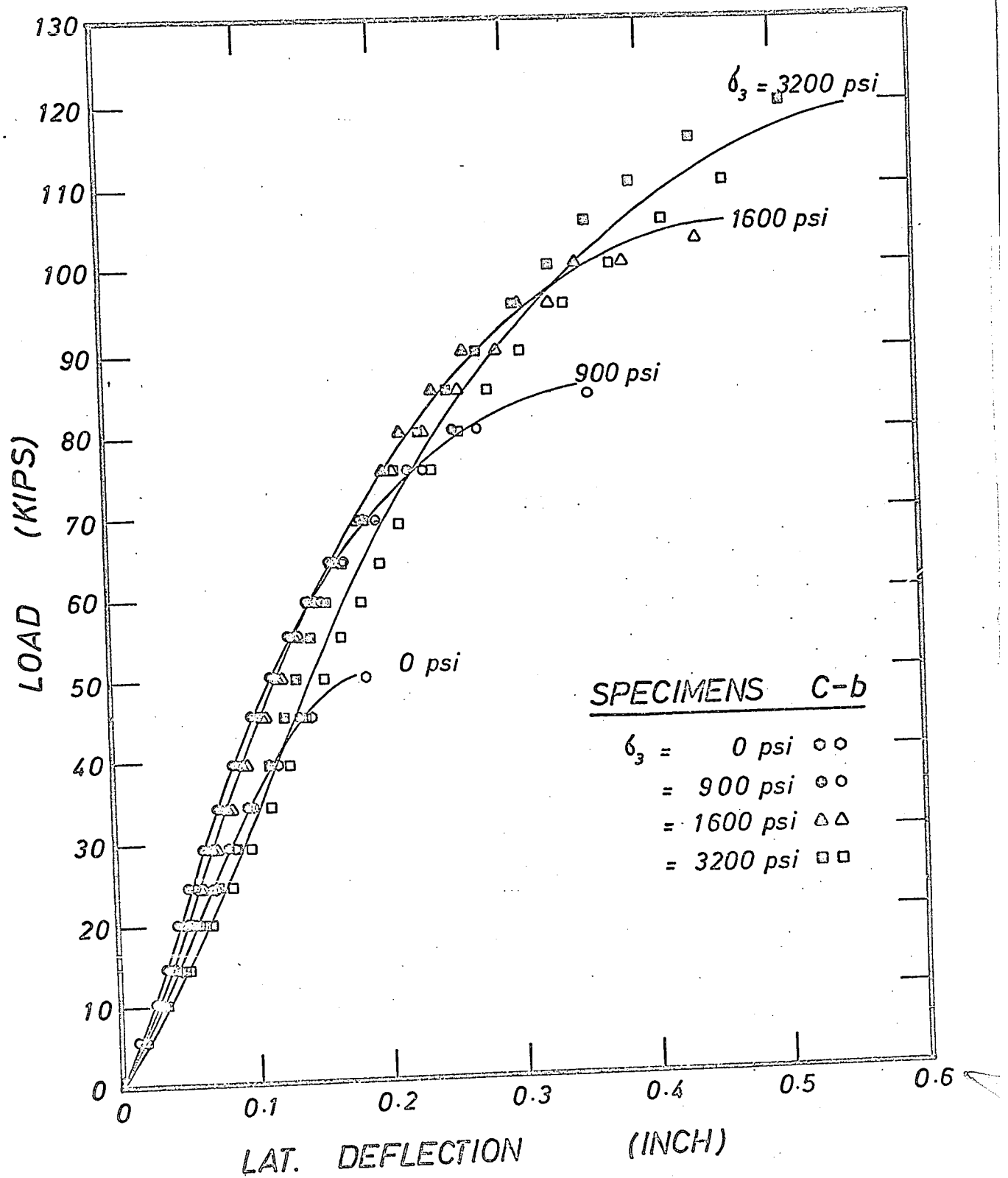


FIG. B-56 LOAD / LAT. DEFLECTION CURVE
OF SPECIMENS C-b

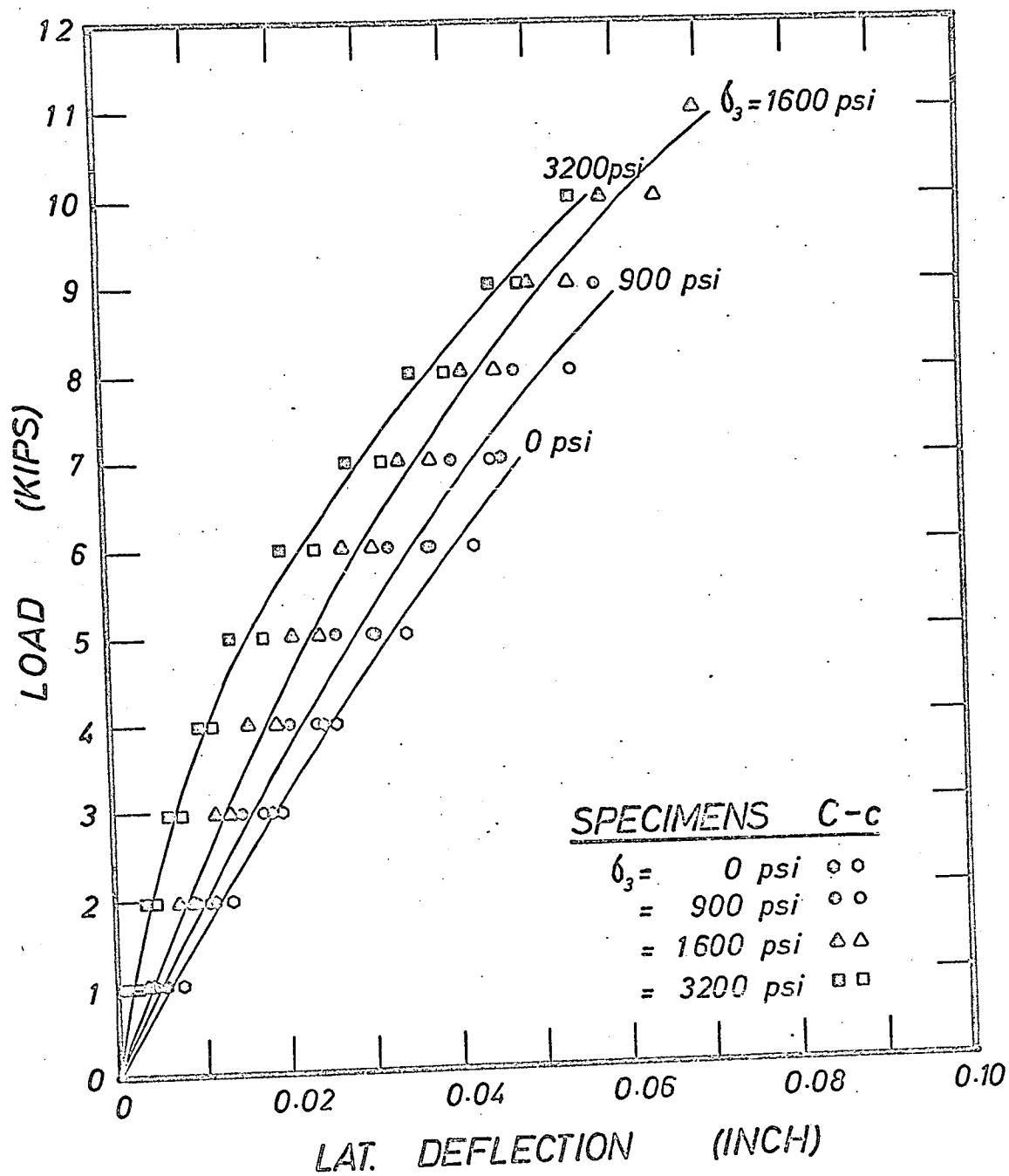


FIG. B-57 LOAD / LAT. DEFLECTION CURVE
OF SPECIMENS C-c

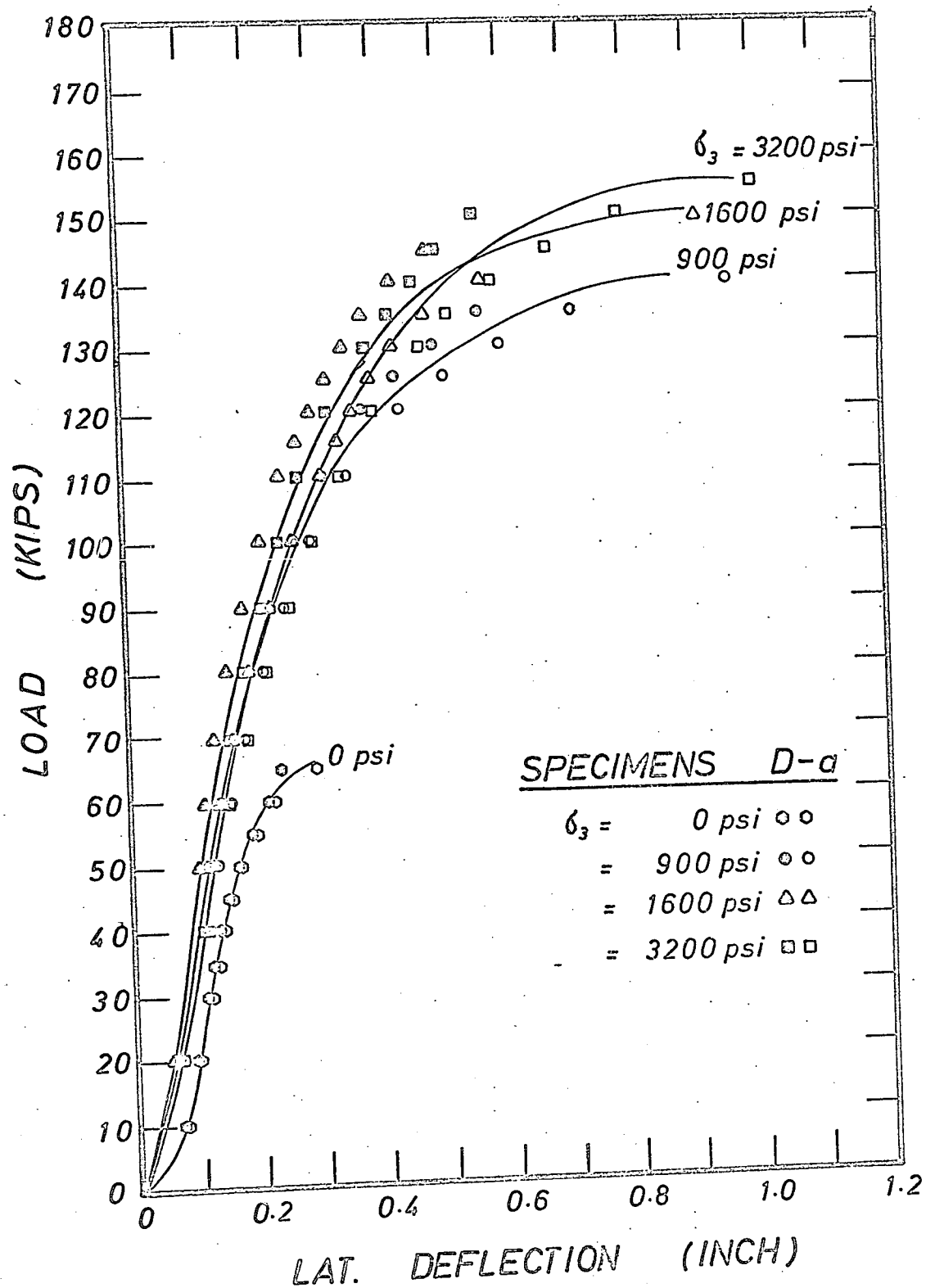


FIG. B-58 LOAD / LAT. DEFLECTION CURVE
OF SPECIMENS $D-\alpha$

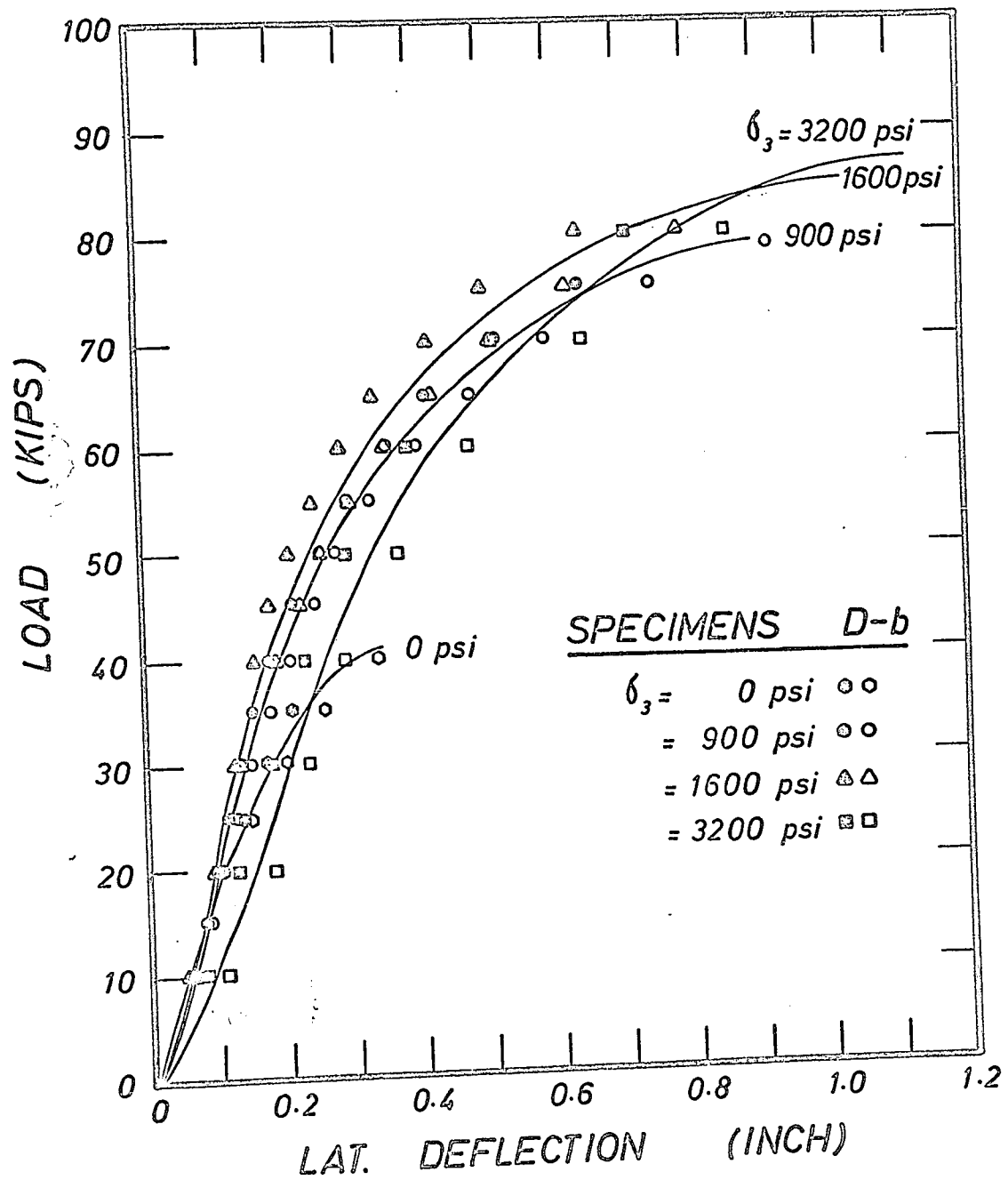


FIG. B-59 LOAD/LAT. DEFLECTION CURVE
 OF SPECIMENS D-b

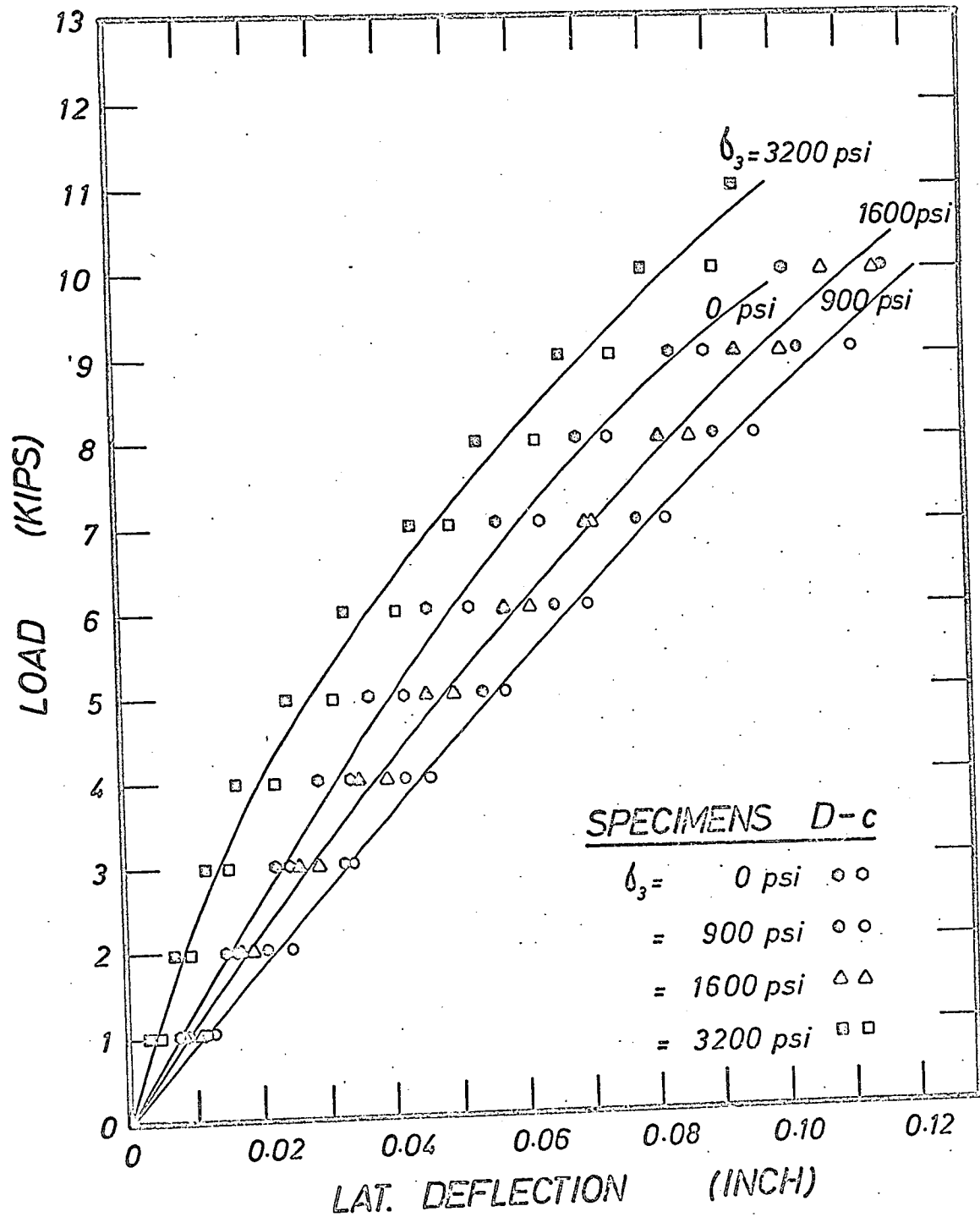


FIG. B-60 LOAD / LAT. DEFLECTION CURVE
OF SPECIMENS D-c

APPENDIX CExperimental Failure Loads of All Specimens

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Fig.C.1 Experimental Failure Loads of All Specimens 189

Fig.C.1 Experimental Failure Loads of All Specimens

Specimen No.	Failure Load (kips)		Specimen No.	Failure Load (kips)	
	Individual	Mean		Individual	Mean
A- 0-a(1)	123.0	120.0	C- 0-a(1)	107.0	101.0
A- 0-a(2)	117.0		C- 0-a(2)	93.0	
A- 0-b(1)	61.0	63.0	C- 0-b(1)	46.0	50.0
A- 0-b(2)	65.0		C- 0-b(2)	54.0	
A- 0-c(1)	10.0	10.5	C- 0-c(1)	6.0	7.0
A- 0-c(2)	11.0		C- 0-c(2)	8.0	
A-18-a(1)	281.0	290.0	C-18-a(1)	196.0	192.0
A-18-a(2)	299.0		C-18-a(2)	188.0	
A-18-b(1)	146.0	139.0	C-18-b(1)	81.0	85.0
A-18-b(2)	132.0		C-18-b(2)	89.0	
A-18-c(1)	12.0	11.8	C-18-c(1)	9.2	8.9
A-18-c(2)	11.6		C-18-c(2)	8.6	
A-32-a(1)	356.0	358.0	C-32-a(1)	241.0	248.0
A-32-a(2)	360.0		C-32-a(2)	255.0	
A-32-b(1)	157.0	156.0	C-32-b(1)	107.0	105.0
A-32-b(2)	155.0		C-32-b(2)	103.0	
A-32-c(1)	11.2	10.6	C-32-c(1)	10.3	10.9
A-32-c(2)	10.0		C-32-c(2)	11.5	
A-64-a(1)	468.0	460.0	C-64-a(1)	309.0	301.0
A-64-a(2)	452.0		C-64-a(2)	293.0	
A-64-b(1)	192.0	193.0	C-64-b(1)	114.0	119.0
A-64-b(2)	194.0		C-64-b(2)	124.0	
A-64-c(1)	10.8	10.6	C-64-c(1)	10.0	10.0
A-64-c(2)	10.4		C-64-c(2)	10.0	
B- 0-a(1)	120.0	117.0	D- 0-a(1)	65.0	66.0
B- 0-a(2)	114.0		D- 0-a(2)	67.0	
B- 0-b(1)	56.0	59.0	D- 0-b(1)	44.0	41.0
B- 0-b(2)	62.0		D- 0-b(2)	38.0	
B- 0-c(1)	8.9	9.0	D- 0-c(1)	10.4	9.8
B- 0-c(2)	9.0		D- 0-c(2)	9.2	
B-18-a(1)	258.0	262.0	D-18-a(1)	144.0	140.0
B-18-a(2)	266.0		D-18-a(2)	136.0	
B-18-b(1)	114.0	110.0	D-18-b(1)	76.0	79.0
B-18-b(2)	106.0		D-18-b(2)	82.0	
B-18-c(1)	10.3	10.7	D-18-c(1)	10.0	10.0
B-18-c(2)	11.3		D-18-c(2)	10.0	
B-32-a(1)	344.0	340.0	D-32-a(1)	153.0	150.0
B-32-a(2)	336.0		D-32-a(2)	147.0	
B-32-b(1)	135.0	140.0	D-32-b(1)	89.0	85.0
B-32-b(2)	145.0		D-32-b(2)	81.0	
B-32-c(1)	13.0	12.7	D-32-c(1)	10.7	10.4
B-32-c(2)	12.4		D-32-c(2)	10.1	
B-64-a(1)	377.0	380.0	D-64-a(1)	151.0	155.0
B-64-a(2)	383.0		D-64-a(2)	159.0	
B-64-b(1)	178.0	170.0	D-64-b(1)	92.0	87.0
B-64-b(2)	162.0		D-64-b(2)	82.0	
B-64-c(1)	10.1	10.3	D-64-c(1)	11.4	11.0
B-64-c(2)	10.5		D-64-c(2)	10.6	

END OF

REEL