SC

AN EXPERIMENTAL INVESTIGATION OF SPIRALLY PRESTRESSED CONCRETE COLUMNS

by

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ABSTRACT

This thesis is concerned with an experimental investigation of the effects of initial prestress and slenderness ratios on the ultimate load carrying capacity of spirally prestressed concrete columns.

The experimental investigation was carried out on 6 inch diameter concrete columns. The program was divided into series A, B, C and D with slenderness ratios 16.6, 25.4, 37.05 and 49.4 respectively. Series A and B were made of plain concrete, while series C and D had longitudinal reinforcing of 4 - #3 vertical and #2 ties @ 6 inches c/c. Lateral prestress, which varied from 20% to 60% of the compressive stress of the concrete was applied by means of a high-strength piano wire of 0.031 inch diameter. Details of fabrication, materials and testing are discussed in Chapter 3.

It was found that an increase in initial spiral prestress increases the load carrying capacity of columns, but the column capacity decreases with an increase in slenderness ratio of the column. Increase in initial spiral prestress showed considerable effect on the elastic behaviour and durability of the column.

Spirally prestressed concrete columns behave in a manner similar to plain concrete columns, and can be analyzed in the same way using the ultimate triaxial concrete strength in place of the cylinder strength.

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TABLE OF CONTENTS

ABSTRACT 1

ACKNOWLEDGEMENTS 2

TABLE OF CONTENTS 3

LIST OF PHOTOGRAPHIC PLATES 5

LIST OF TABLES 6

LIST OF FIGURES AND GRAPHS 7

NOMENCLATURE 13

CHAPTER 1. INTRODUCTION 15

CHAPTER 2. REVIEW OF PRIOR STUDIES 18

CHAPTER 3. MATERIALS AND TESTING PROCEDURE 30

3.1 Nomenclatures For Specimens

3.2 Materials

3.3 Experimental Technique

3.4 Fabrication and Curing

3.5 Testing Procedure

CHAPTER 4. THEORETICAL CONSIDERATIONS 47

4.1 Analysis Of Wire Wrapped Concrete Cylinders

4.2 Behaviour Of Concrete Under Triaxial Stress

4.3 Effect Of Slenderness Ratio On Laterally Prestressed Concrete

CHAPTER 5. DISCUSSION OF TEST RESULTS 51
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS 65
6.1 Conclusions
6.2 Recommendations For Future Work
BIBLIOGRAPHY 68
APPENDIX A STEEL PROPERTIES 96
APPENDIX B INDIVIDUAL GRAPHS OF SPECIMENS 100
VITA 125

LIST OF PHOTOGRAPHIC PLATES

Plate No.	Subject	Page No.
1.	Wrapping of Concrete Specimens	34
1a.	Details of Tensioning Device	38
2.	Wrapping of Specimens: (C & D Series)	40
3.	Casting of Specimens: (C & D Series)	42
4.	Details of Specimen Testing System	46
5.	Typical Failures of Specimens:	
	A-Series	52
6.	Typical Failures of Specimens:	
	B-Series	54
7.	Typical Failures of Specimens:	
	C-Series	56
8.	Typical Failures of Specimens:	
	D-Series	58

LIST OF TABLES

Table No.	Description	Page No.
1.	Specimen Details	31
2.	Mix Proportions by Weight	33
3.	Details of Tests	61
4.	Calculation of "Empirical Coefficients"........................	63

LIST OF FIGURES

Figure No.	Description	Page No.
1.	Fabrication Details of Specimen	36
2.	General Arrangement of Testing Specimen	44
3.	Analysis of Spirally Prestressed Concrete Specimen	48

LIST OF GRAPHS

Graph No.	Description	Page No.
1.	Longitudinal Stress-Strain Characteristic of A-Series	76
2.	Longitudinal Stress-Strain Characteristic of B-Series	77
3.	Longitudinal Stress-Strain Characteristic of C-Series	78
4.	Longitudinal Stress-Strain Characteristic of D-Series	79
5.	Load-Lateral Deflection of Specimens. B-Series	80
6.	Load-Lateral Deflection of Specimens. C-Series	81
7.	Load-Lateral Deflection of Specimens. D-Series	82
8.	Longitudinal Stress-Strain Characteristic With Initial Lateral Prestress = 960 psi..........	83

9. Longitudinal Stress-Strain Characteristic With Initial Lateral Prestress = 1,706 psi......... 84

10. Longitudinal Stress-Strain Characteristic With Initial Lateral Prestress = 3,412 psi........ 85

11. Relation Of Ultimate Load To Initial Lateral Prestress 86

12. Relation Of Axial Ultimate Stress To Lateral Ultimate Stress: A-Series 87

13. Relation Of Axial Ultimate Stress To Lateral Ultimate Stress: B-Series 88

14. Relation Of Axial Ultimate Stress To Lateral Ultimate Stress: C-Series 89

15. Relation Of Axial Ultimate Stress To Lateral Ultimate Stress: D-Series 90

16. Variation Of Empirical Coefficient βK With Slenderness Ratio 91

17. Variation Of Efficiency Ratio, M, With Slenderness Ratio 92

18. Variation Of Ratio Of Long Column Load To Short Column Load With Slenderness Ratio 93

19. Mohr's Circles For Specimens Of A-Series 94

20. Mohr's Circles For Specimens Of B-Series 95

APPENDIX A

A.1 Stress-Strain Characteristic Of 0.031 inch Diameter Steel Wire 97

A.2 Stress-Strain Characteristic Of Longitudinal Reinforcement (#3 Bars) 98

A.3 Calibration Curve For Tensioning Device 99

APPENDIX B

B.1 Load-Deformation Of Specimen A.O.O Series 102

B.2 Load-Deformation Of Specimen A.18.1 Series 103

B.3 Load-Deformation Of Specimen A.32.1 Series 104

B.4 Load-Deformation Of Specimen A.64.2 Series 105

B.5 Stress-Strain Characteristic Of Control Concrete (For Specimens Of A-Series) 106

B.6 Load-Deformation Of Specimen B.0.0 Series 108

B.7 Load-Deformation Of Specimen B.18.1 Series 109

B.8 Load-Deformation Of Specimen B.32.1 Series 110

B.9 Load-Deformation Of Specimen B.64.2 Series 111

B.10 Stress-Strain Characteristic Of Control Concrete (For Specimens Of B-Series) 112

B.11 Load-Deformation Of Specimen C.0.0 Series 114

B.12 Load-Deformation Of Specimen C.18.1 Series 115

B.13 Load-Deformation Of Specimen C.32.1 Series 116

B.14 Load-Deformation Of Specimen C.64.2 Series 117

B.15 Stress-Strain Characteristic Of Control Concrete (For Specimens Of C-Series 118

B.16 Load-Deformation Of Specimen D.0.0 Series 120

B.17 Load-Deformation Of Specimen D.18.1 Series 121

B.18 Load-Deformation Of Specimen D.32.1 Series 122

B.19 Load-Deformation Of Specimen D.64.2 Series 123

B.20 Stress-Strain Characteristic Of Control Concrete (For Specimen Of D-Series) 124

NOMENCLATURE

A_c = Area of concrete (Sq. In.)

A_s = Area of longitudinal steel reinforcement (Sq. In.)

D = Diameter of concrete column (In.)

f_c' = Ultimate (unrestrained) compressive stress of concrete (psi. or ksi.)

f_m' = Ultimate (unrestrained) compressive stress of motar (psi. or ksi.)

f_s' = Allowable stress in longitudinal steel reinforcement (psi. or ksi.)

K = Empirical coefficient; increase in ultimate load due to lateral prestress.

M = Efficiency Ratio, equal to increase in load due to spirally prestressed reinforcement divided by the load taken by the same amount of steel if used as longitudinal reinforcement at the same stress.

p = Ratio of area of longitudinal steel reinforcement to the gross area.

R' = Ratio $= \dfrac{\text{Long column load}}{\text{Short column load}}$

T = Tension in the wires per inch. (Number of wires per inch x tension (in lbs.) in each wire)

ν = Poisson's ratio.

β = Empirical coefficient dependent on slenderness ratio.

σ_1 = Maximum axial stress (psi. or ksi.)

$\sigma_{1\text{ultimate}}$ = Total ultimate stress of spirally prestressed concrete specimens (psi. or ksi.)

σ_3 = Lateral pressure on concrete. (psi. or ksi.)

$\sigma_{3\text{ultimate}}$ = Total ultimate lateral stress on specimen due to restraint of spiral prestress wire (psi. or ksi.)

in. = Inch, inches.

Kips. = One thousand pounds.

k.s.i. = One thousand pounds per square inch.

p.s.i. = Pounds per square inch.

μ = Micro (i.e. 10^{-6})

CHAPTER 1

INTRODUCTION

It has long been known that concrete laterally constrained in some manner can withstand larger stresses and deformations compared to the same concrete in its unrestrained state. This type of stress condition is often called a "Triaxial state of stress".

The practical use of lateral stress to increase the axial load carrying capacity of members has been wide spread in spirally tied columns and pipe columns. The lateral stress in these cases is created as the concrete is loaded and it tries to deform laterally due to a Poisson effect. This dilation is restrained by the spiral or pipe respectively and hoop stresses are developed in the steel, thus laterally stressing the concrete.

For both pipe columns and spirally tied columns, no lateral stress exists until lateral deformation occurs; thus this could well be defined as passive lateral prestressing.

Many investigations have been carried out into the triaxial behaviour of concrete using a constant lateral stress provided by a fluid under pressure. Nearly all these investigations were concerned with finding the failure strength of triaxially loaded concrete rather than the load deformation characteristics. All theoretical investigations have assumed some variant of the Coulomb Theory of slip occuring on a failure plane through the cylinder which gives an expression of the following type:

$$\sigma_{axial} = f_c' + K \cdot \sigma_{lateral} \quad \text{..................(1.1)}$$

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

As lateral stress dramatically increases the axial load capacity of concrete, it is logical to consider actively prestressing the concrete by wrapping the concrete with wire under tension.

Several investigators have reported experimental results on short columns (cylinders) wrapped with high strength wire under tension, which have shown the validity of this line of reasoning.

In practice, however, failure of any compression member is due to mixture of material crushing and instability. To date some work has been carried out on long spirally prestressed columns which has shown increased axial load capacity.

Currently (1969), there are no provisions for the design of spirally prestressed columns in either ACI (American Concrete Institute) or NBC (National Building Code of Canada

1965) codes. Unfortunately, not enough experimental data is available on which code recommendations could be based.

Practical application of a high-strength compressive unit to many important types of engineering structures is entirely feasible. One of the great handicaps of the reinforced concrete frame building has been the large diameter needed for the inside column of the lower stories. In many cases, a reinforced concrete frame building has been decided against in favour 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 using spirally prestressed concrete columns.

In many structures, compression members are common. Therefore, it is logical to conclude that sooner or later, with the need for economy, such a method of spirally prestressing must be developed in detail.

This investigation was to determine as accurately as possible, the effect of lateral prestressing and slenderness ratio on the load carrying capacity of concrete columns.

CHAPTER 2

REVIEW OF PRIOR STUDIES

Up-to-date published relevant research work is presented in this chapter in two sections:

A) Experiments Using Fluid Pressures.

B) Experiments Using Mechanical Restraints.

A) EXPERIMENTS USING FLUID PRESSURES

Considere (15), 1902, investigated the effects in triaxial loading on mortar. From the results of compression tests on mortar specimens, cylinders of 11.8 inches in diameter x 31.5 inches long, under pressure in a water medium, Considere formulated that,

$$\sigma_1 = a . f'_m + 4.8 \cdot \sigma_3 \qquad \text{.................(2.1)}$$

Where, σ_1 = Maximum axial stress.

a = Variable increasing with σ_3 from 1.0 to 1.5 in value.

f'_m = Ultimate (unrestrained) mortar compressive stress.

σ_3 = Lateral pressure.

Extensive studies of concrete under triaxial stress were made by Richart, Brandtzaeg and Brown (48) in 1928. In the first group of tests, the two principal stresses produced by the liquid pressure were smaller than the third principal stress. In this series, sixty-four, 4 inch diameter x 8 inch long concrete cylinders were tested in three-dimensional compression. The second group included the tests of forty-eight 4 inch diameter x 22 inch concrete cylinders in three-dimensional compression, the two principal stresses normal to the axis of the cylinders being larger than the axial stress. The concrete strength ranged from 605 psi. to 7,300 psi. The principal stresses normal to the axis of the cylinders varied from 3,200 psi. to 6,910 psi.

From these studies the authors concluded that the lateral pressure increases the load carrying capacity of the cylinders. This increase in strength was independent of the proportions of the concrete mixes.

Test results were presented in graphs showing a straight line relationship between the axial failure stress and lateral stresses.

The authors concluded that the following linear equation was a satisfactory approximation in predicting axial ultimate stress:

$$\sigma_1 = f_c' + 4.1\ \sigma_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.1A)$$

Where, σ_1 = Ultimate maximum axial stress.

f_c' = Unconfined compressive stress of concrete.

σ_3 = Lateral (fluid pressure) stress.

Balmer (5), in 1949, studied the failure of concrete under triaxial compression with lateral pressures in the range 0 to 25,000 psi. and with corresponding ultimate axial stresses from 3,000 to 86,000 psi. The lateral pressure was first applied and held constant while the axial stress increased until the concrete failed. From the test results, it was concluded that at low lateral pressures the increase in axial strength was about 6 to 7 times the lateral pressure but at high lateral pressure was only 3 times. The axial deformation, at failure of the specimens subject to large lateral pressures, was several times greater than that for specimens subject to small lateral pressures. All cylinders were 6 inches in diameter x 12 inches long.

Akroyd (1), in 1961, studied the failure of concrete under triaxial compression with lateral pressure up to 10,000 psi. using 3 inch diameter x 6 inch long cylindrical specimens. The testing procedure and conclusions were similar to those of Balmer (5). By conducting drained and undrained tests, Akroyd, determined the effect of pore pressure on the triaxial strength. These tests demonstrated that saturated concrete behaved like a material which did not possess the property of internal friction, when tested triaxially under conditions of no change of water contents.

Chinn and Zimmerman (14) (59), in 1965, extended the work of Balmer. All tests were performed on 6 inch diameter x 12 inch long concrete cylinders. The program was divided into wire to increase the pressure of the fluid that can be carried has been used for a long time. This technique was reported in the papers by Anon (2), Crepps (16), Crom (17), Doull (20) and others. All methods used mechanical means of wrapping the wire about the pipes.

Relatively little research has been carried out on the idea of wrapping a concrete column to increase its axial load carrying capacity. The work known to the author is described below.

In 1902, Considere (15) published the results of his researches on reinforced concrete. Considere carried out experiments on reinforced concrete, hoop reinforced and restrained concrete and restrained sand.

Considere prepared a number of concrete specimens with varying amounts of hoop reinforcement. The hooped specimens initially behaved in a similar manner to un-reinforced specimens. However, the hooped specimens carried, without crushing of the concrete, much higher loads than the unhooped specimens and only failed after extensive cracking had become apparent and large deformations had occurred. Furthermore, the hooped concrete showed no drop off in load with deformation, exhibiting a plastic type of behaviour.

Considere also noted that the ductility of concrete was much increased.

Richart, Bradtzaeg and Brown (49), in 1929, extended the study of Considere by doing research on spirally tied columns. The failure loads of spirally reinforced columns (cylinder) were compared with unreinforced cylinders. These results showed that the failure stress of hooped concrete could be predicted using the triaxial stress equation if an equivalent lateral stress due to the hooping was used.

$$\sigma_1 = f'_c + K\,\sigma_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.2)$$

The magnitude of K was 4.1.

Maney (43), in 1944, carried out an investigation on triaxial compression of concrete columns. Short cylindrical columns with diameters varying from 2 inches to 8 inches were used. These columns were confined laterally with combination of steel sheet liners of 0.037 inch, 0.044 inch and 0.285 inch thickness and spiral reinforcing of 0.165 inch and 0.500 inch diameter on the top of steel sheet liners. The prestressing in the spiral reinforcing was achieved by compacting the cylinders with high pressure thus straining the steel liners. All the concrete cylinders were tested to failure. The author concluded that by this method of carrying compressive load through the lateral action of a spiral in tension, 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 that of steel structures.

Johnson (35), in 1950 carried out an experimental investigation of spirally prestressed short columns, 3 inches in diameter and 12 inches high. Prestressing was achieved by wrapping wire around concrete columns by means of a lathe. The wire was wrapped in tension by feeding it through two sliding plates. The force in the wire was read by means of a spring-scale.

A total of 135 samples were cast, 27 were of plain concrete without any reinforcing, while 108 samples were spirally prestressed. The compressive stress of the concrete ranged from 3,470 to 4,870 psi. Three wire sizes, 0.0317 inch, 0.0475 inch and 0.0625 inch diameter were used with pitches of 3, 5, 7 and 9 wires per inch. Initial tension in the wires was 10,000, 50,000 and 100,000 psi. producing lateral stresses in the concrete of 300, 1,500 and 3,000 psi. respectively. All cylinders were tested to failure. The author concluded that all spirally prestressed specimens showed higher load carrying capacity than their unprestressed counterparts. The increase in load carrying capacity varied directly with the wrappings per inch of the spiral, and in proportion to the tension applied to the spiral.

The magnitude of K in the equation (2.2) was calculated using the experimental value of σ_1, f_c' and σ_3 and found to range from 3.60 to 5.77.

Johnson reported some difficulties in his experimental procedures. It was difficult to check the tension in the wire, when wrapping, because the speed of rotation of the lathe created vibration problems. There was failure of 'end grips' in testing the cylinders.

Gambarov (24), in 1961, reported a research program on columns prestressed by spirally wrapping with wire under tension.

His project was based on round columns, 5½ inches in diameter and 31½ inches long. The prestressing wire of 0.1 inch diameter was used with initial tensions in the wire of 7, 108 and 184 ksi. The author concluded that all the spirally prestressed columns had a higher load capacity than their unprestressed counterparts. This increase which was generally greater with a large percentage of spiral prestressing was influenced throughout by the slenderness of the specimen.

Feeser and Chinn (22) (23), in 1962, reported experiments on spirally prestressed concrete columns.

Prestressing was achieved by wrapping wire around concrete columns by means of a lathe. The wire was fed through a friction-plate device and the tension in the wire was measured by a strain-recorder. The friction-plate device was calibrated for various tensions in the wire. Thirty-four, 3 inch diameter x 12 inch long spirally prestressed cylinders were used in this study. The average 10 day unconfined compressive stress for the concrete mix was 5,230 psi. The variation of concrete compressive stress was ± 9%. Two sizes of wire were used, 0.0540 inch and 0.0595 inch diameter. The cylinders were wrapped with pitches of 1/16 inch, 1/10 inch, 1/8 inch, 1/6 inch and 1/4 inch with initial tensions of 10, 100, 200 and 300 lbs. per wire. The initial lateral pressure on the concrete attained with these wrappings ranged from 27 psi. to 2,133 psi. All the wrapped cylinders failed when the wire broke. The lateral pressure at ultimate ranged from 1,533 psi. to 6,120 psi.

The equation for predicting the ultimate stress of a wrapped cylinder is given by,

$$\sigma_1 = f'_c + 3.818\,\sigma_3 \qquad \text{..........................}(2.3)$$

It was also shown that the stiffness of the cylinders can be increased by increasing the initial tension in the spiral prestressing wire. However, the intial tangent modulus of all laterally prestressed columns was less than the initial tangent modulus of the plain concrete. It was also concluded that the ultimate stress of the cylinder was independent of the initial tension in the wire, and dependent only upon the lateral confining stress in the cylinder at failure.

Ben-Zvi, Muller, Rosenthal (9), in 1966, presented considerable experimental data on the effects of triaxial stress caused by circumferential prestressing.

Prestressing was achieved by wrapping wire around concrete columns by means of a lathe. The wire was wrapped under tension. Forty-four of 3 inch diameter x 6 inch long cylinders were spirally prestressed with intial tension in the wire of 7.10, 92.2 and 163.50 ksi. The wire diameter was

0.040 inch with a pitch of 0.040 inch. Concrete compressive stresses varied from 2,800 psi. to 7,450 psi. All the spirally prestressed cylinders failed by the rupture of the wire due to lateral pressure exerted on it through the expansion of the concrete. Six specimens of 10 inches in diameter and 100 inches long were spirally prestressed with a steel wire of 0.2 inch in diameter and pitch of 0.4 inch - 0.8 inch with initial tension in the wire of 99,400 psi. Concrete compressive stress for all cylinders was 4,260 psi. These columns were also prestressed longitudinally to take care of handling and transportation stresses. All the spirally prestressed columns in this series failed by buckling and rupture of the circumferential wire. Another set of 3 inches in diameter columns, 2 feet, 3 feet and 10 feet long were spirally prestressed in one and two layers. The wire used was 0.055 inch in diameter with a pitch of 0.067 inch and initial tensions of 7.1 and 163 ksi. All the wrapped columns in this series failed by buckling.

The authors concluded that column capacity was increased with an increase of initial spiral prestressing, but decreased with the increase of the slenderness ratio of the column. Longitudinal prestressing did not reduce ultimate strength of columns and had a beneficial effect on deformation and transportation. An outer shell, if used, might increase column stability and delay buckling. The shell would also protect the spiral prestressing wire against corrosion and fire. For short columns, the equation (2.2) was used and the value of

K found to vary from 2.25 to 3.91. For long columns, the equation (2.2) was transformed to account for slenderness effects to the following form:

$$\sigma_1 = (f_c' + p.f_s') + \beta K.\sigma_3 \ldots\ldots\ldots\ldots\ldots (2.4)$$

Where, β = Slenderness correction coefficient.

The value of βK varied from 0.15 to 3.52.

Martin (44), in 1968, reported experiments on 17 spirally prestressed concrete cylinders 4 inches in diameter and 8 inches long. The cylinders were wrapped with steel piano wire of 0.016 inch diameter in a lathe. The wire was spaced by passing it over a pulley attached to the power driven carriage. The pitch of the wire was 0.016 inch, 0.018 inch and 0.032 inch. Initial lateral stress varied from 140 to 1,481 psi. Concrete compressive stress was 6,490 psi. with a standard deviation of 357 psi. Tests were carried out on short-term loading and long-term loading. The author concluded that the large increase in axial ultimate stress and axial ultimate strain of cylinders caused by spiral steel depended essentially on the lateral ultimate stress that the spiral caused in the concrete. Very high strength steel may be effectively used in spiral reinforcement. Departure from elastic behaviour was distinctly delayed by prestressing of spiral wire. In axial compression tests of spirally prestressed cylinders, the value of Poisson's ratio varied from 0.16 at small stresses to more than 1.0 at large stresses during rapid loading.

Two basic behaviours can be observed from the previous work which can be reported as;

1. Active triaxial behaviour where the lateral stress is provided by the fluid in a triaxial cell or by wrapping the concrete with steel wire under tension. Active triaxial behaviour is also present in pipe columns with expansive concrete.

2. Long column effects due to buckling are going to affect, in some way, any attempts to practically exploit the advantages of traixial behaviour.

In order to study and to give a quantitative estimate of the effect of various lateral stresses and the slenderness ratios on the ultimate load carrying capacity of the columns, the present author carried out the experimental investigation reported on in this thesis.

This investigation was concerned with obtaining good experimental evidence of the variation of failure load of spirally prestressed concrete columns with the magnitude of lateral stress and slenderness ratio. Fifty-six columns were made and tested to failure. The columns were 6 inches in diameter and had lengths of 12 inches, 24 inches, 42 inches and 60 inches. Lateral prestresses of 0, 960, 1,706 and 3,412 psi. were used.

CHAPTER 3

MATERIALS AND TESTING PROCEDURES

General

The experimental program consisted of making a number of columns of different lengths with various amounts of lateral prestress and subsequent loading of these columns to failure. The columns were laterally prestressed by wrapping high-strength wire under tension around the columns.

Before commencement of the large scale production of columns, several pilot tests were carried out which verified the feasibility of the technique and allowed for unforeseen difficulties to be eliminated.

The lateral prestress was applied by turning the column slowly in a lathe, the rotation of the column drawing the wire through a pair of clamped friction plates. The friction plates were mounted upon a short column fixed to the carriage of the lathe and the force in the column was measured.

Concrete columns 6 inches in diameter were made with varying lengths of 12 inches, 24 inches, 42 inches and 60 inches, and were labelled as A, B, C and D series respectively. For each series, four levels of lateral prestress were employed, the lateral pressures being 0, 960, 1,706 and 3,412 psi. respectively. Table 1 gives details of the number and types of the test columns.

TABLE 1: SPECIMEN DETAILS

Mark	Overall length of specimen	Number of specimen of each	Casting Details		Layers of spirals	Pitch wires per inch	Longitudinal reinforcing details	Slenderness ratio *
			Age at wrapping	Age at testing				
A- 0-0	1'-0"	3	-	35 days	0	0	Nil	16.6
A-18-1	1'-0"	3	24 days	35 days	1	18		
A-32-1	1'-0"	3	25 days	36 days	1	32		
A-64-2	1'-0"	3	25 days	37 days	2	64		
B- 0-0	2'-1"	3	-	40 days	0	0	Nil	25.4
B-18-1	2'-1"	3	24 days	41 days	1	18		
B-32-1	2'-1"	3	25 days	41 days	1	32		
B-64-2	2'-1"	3	26 days	42 days	2	64		
C- 0-0	3'-7½"	3	-	41 days	0	0	4 - #3 vert. #2 ties at 6" c/c	37.05
C-18-1	3'-7½"	3	27 days	41 days	1	18		
C-32-1	3'-7½"	3	28 days	42 days	1	32		
C-64-2	3'-7½"	3	29 days	43 days	2	64		
D- 0-0	5'-1"	2	-	57 days	0	0	4 - #3 vert. #2 ties at 6" c/c	49.4
D-18-1	5'-1"	2	49 days	57 days	1	18		
D-32-1	5'-1"	2	50 days	58 days	1	32		
D-64-2	5'-1"	2	51 days	59 days	2	64		

*See Fig. 2

3.1 NOMENCLATURES FOR SPECIMENS

Each specimen was designated for length, wires per inch i.e. lateral stress, number of layers of wire wrapping and a number denoting the specimen in that particular series. This was the only way to distinguish all the specimens from each other. Thus as an example;

B. 18. 1. 2 gives,

B = Columns 2 feet long.
18 = 18 wires per inch.
1 = One layer of wire wrapping.
2 = Specimen number 2 out of three of the same kind.

3.2 MATERIALS

Concrete

The concrete used in all tests was obtained from the Francon-Concrete Company, a local ready-mixed manufacturing plant. The concrete was proportioned automatically by weight and machine mixed. The mix proportion of the concrete is presented in Table 2.

TABLE 2: MIX PROPORTIONS BY WEIGHT

Slump = 3 inches

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

Reinforcing

Longitudinal reinforcing was provided for columns in series C and D to take care of stresses due to the self weight of the columns plus the lateral load of the prestressing wire while the column was supported in the lathe.

Nominal reinforcing of 4 - #3 bars in the longitudinal direction and #2 ties at 6 inches centre to centre (to comply with the National Building Code of Canada, 1965, requirement section 4.5.3.67 (2); p. 22) was provided.

The stress-strain characteristic of one of these reinforcing bars is presented in the Appendix A.

PLATE I: WRAPPING OF SPECIMEN.

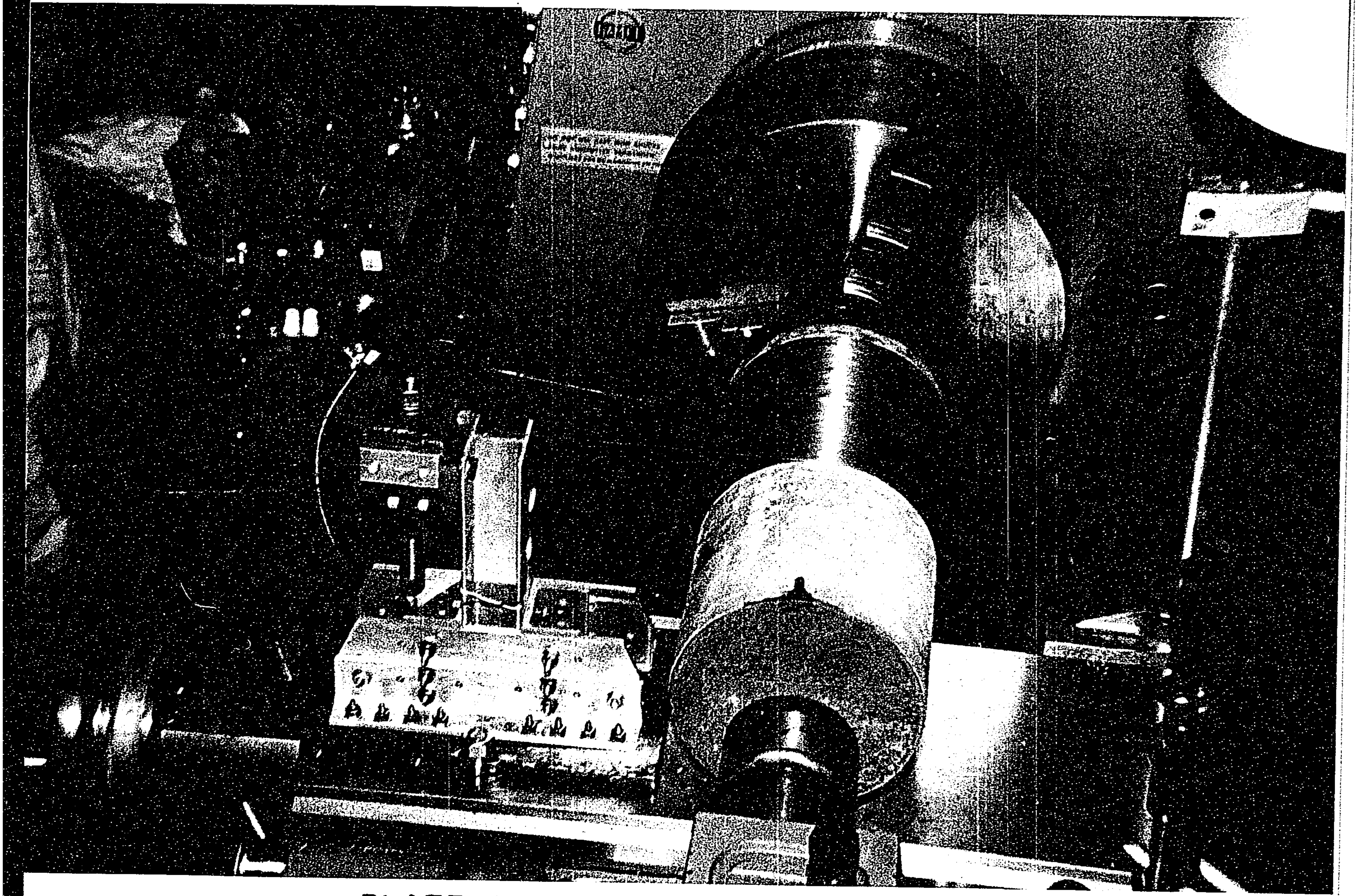


PLATE : I : WRAPPING OF SPECIMEN

Wire For Wrapping

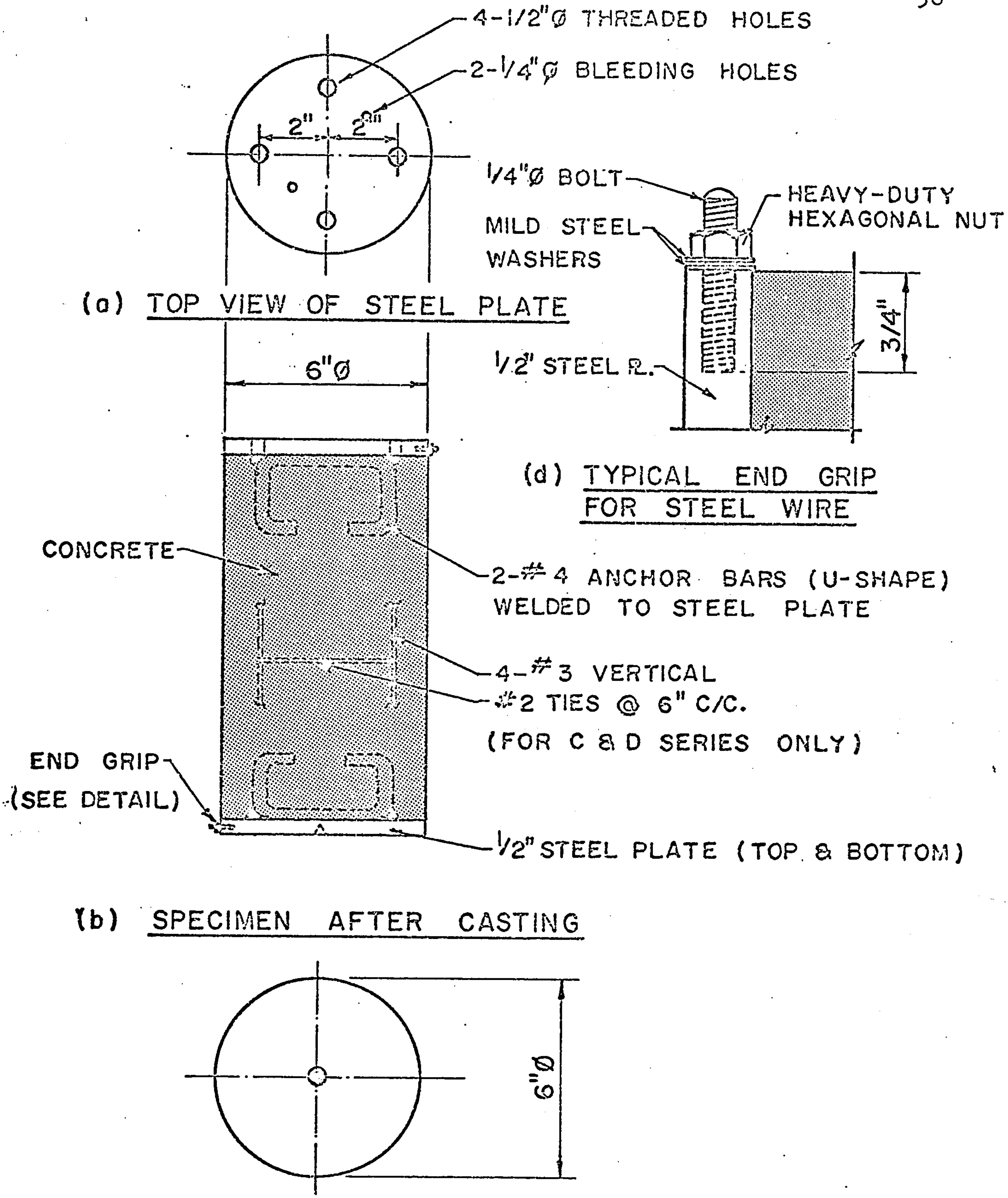
High strength piano wire of 0.031 inch diameter was used for spiral prestressing. The average ultimate stress of the wire was 342,000 psi. The wire was tested in an 10,000 lbs. capacity "Instron" testing machine. The stress-strain characteristic of the wire is presented in the Appendix A.

3.3 EXPERIMENTAL TECHNIQUE

Wire Anchorage

However simple is the concept of wrapping of wire around a cylinder, the practical details are troublesome. It was essential to have a device to anchor the wire at the ends of the specimens. This was achieved by passing the wire between two mild steel washers and clamping them with a nut. Details of end plates and the general arrangement of these bolts are given in Figure 1. Anchorage of the wire at the beginning of wrapping was no problem. The wire was looped around the bolt and was clamped between two washers by tightening the nut. The same operation was repeated at the other end of the cylinders with at least two windings on each side of the bolt before it was securely held by the washers and a nut.

Preliminary tests on the intended gripping device showed no indication of slippage. It is also interesting to note that there was no anchorage failures in any samples during the tests.



(a) TOP VIEW OF STEEL PLATE

(d) TYPICAL END GRIP FOR STEEL WIRE

(b) SPECIMEN AFTER CASTING

(c) BOTTOM VIEW OF STEEL PLATE

FIGURE 1: FABRICATION DETAILS OF SPECIMEN

Wrapping Of Cylinder

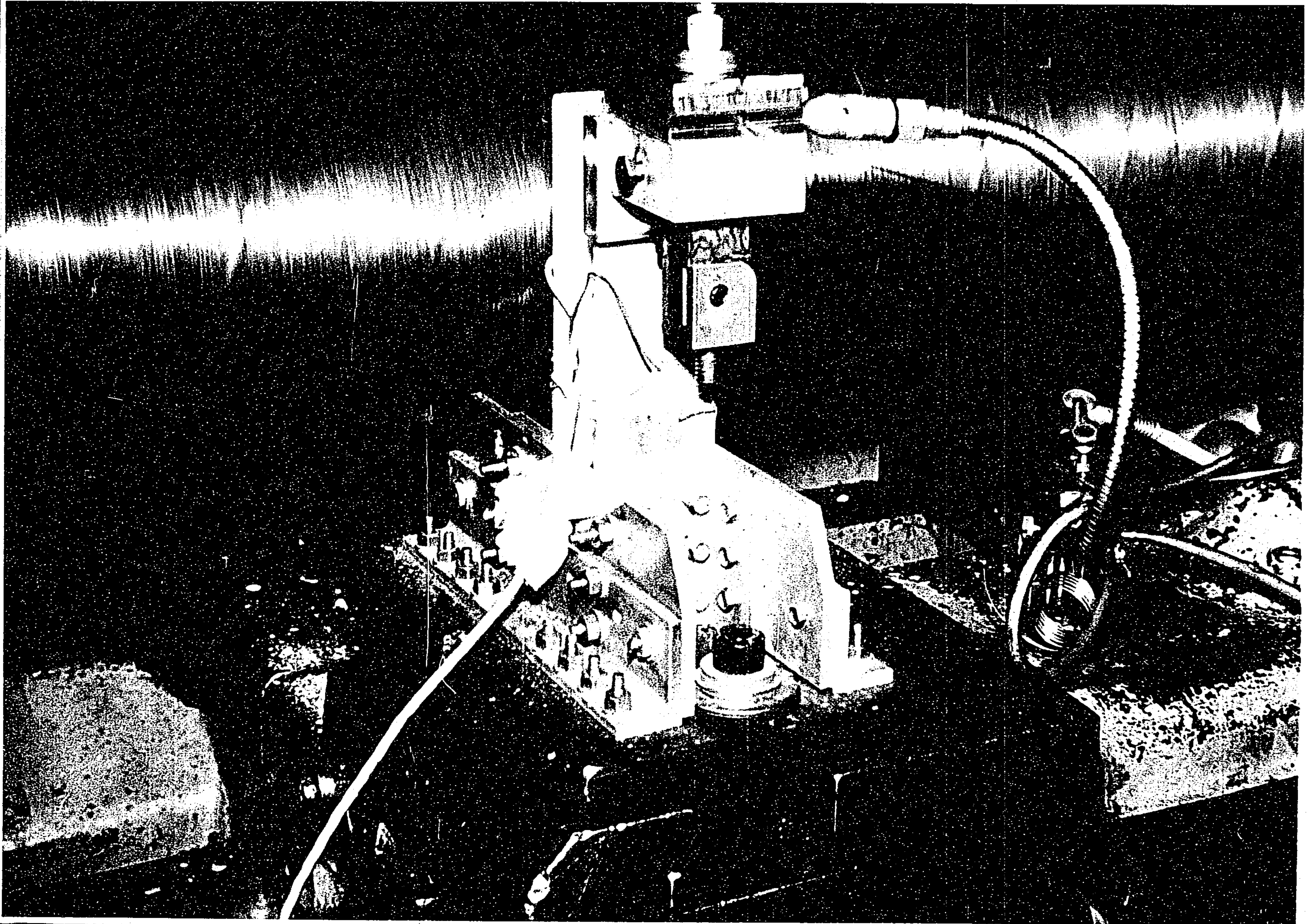
All the wrapping of the cylinders was done by means of a lathe. The pitch of the wire was chosen to be similar to the cutting threads of a lathe, and the tool carriage could be driven at any given pitch by adjusting the set up of the lathe. For the present experiment, the set up for 18 threads per inch and 32 threads per inch was used. A wrapping of 64 wires per inch was accomplished by a double layer of 32 wires per inch wrap.

Application Of Tension To The Wire

Application of uniform tension in the wire was accomplished by means of the device shown in Plate 1a. The device consisted of an Aluminum I - section, 2 WF 3.8 which was fixed at the base on ½ inch thick steel plate. An aluminum block, 2 inches x 2 inches x 1 inch was fixed on the top of the I - section to support a bolt and stainless steel plates. The height of the I - section was such that the wire coming through the plates was tangent to and in the same plane as that of the top of the concrete cylinders. At the base of the I - section, SR-4, FAE-25-12S6 type strain gages were mounted with full-bridge connection.

The strains were continuously read by means of a Budd's-Strain-Gage Indicator.

PLATE Ia: DETAILS OF TENSIONING DEVICE



The tensioning device was calibrated, using the wire from experiments, at the National Research Council Machine Shop on a "Bertram" Vertical Boring Machine, Dundas, Ontario.

The wire was supplied on wooden spools which were mounted in a dexion stand giving a smooth and continuous feed of the wire to the tensioning device.

To avoid excessive heating of the friction plates and thereby damage or change the structural properties of the wire, a constant air flow was used to cool the friction plates.

Because of the slow speed of the lathe (i.e. cylinders), no vibration was experienced. A previous investigator (35) reported a vibration problem during the wrapping process. Slow speed and smooth plates allowed uniform tension in the wire throughout the operation of wrapping of cylinders.

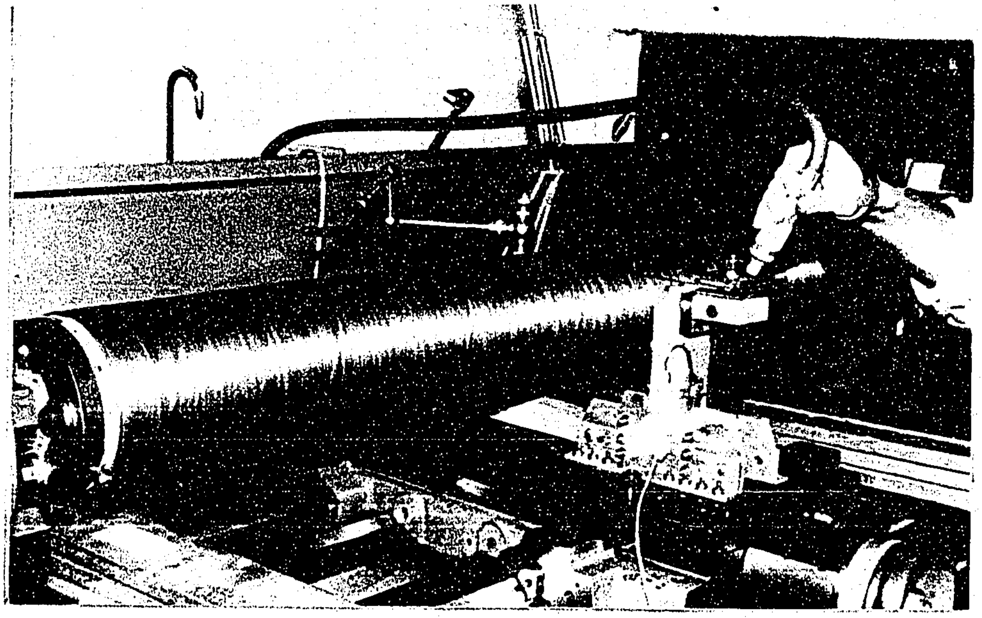
The tensioning device was bolted to the tool-carriage.

3.4 FABRICATION AND CURING

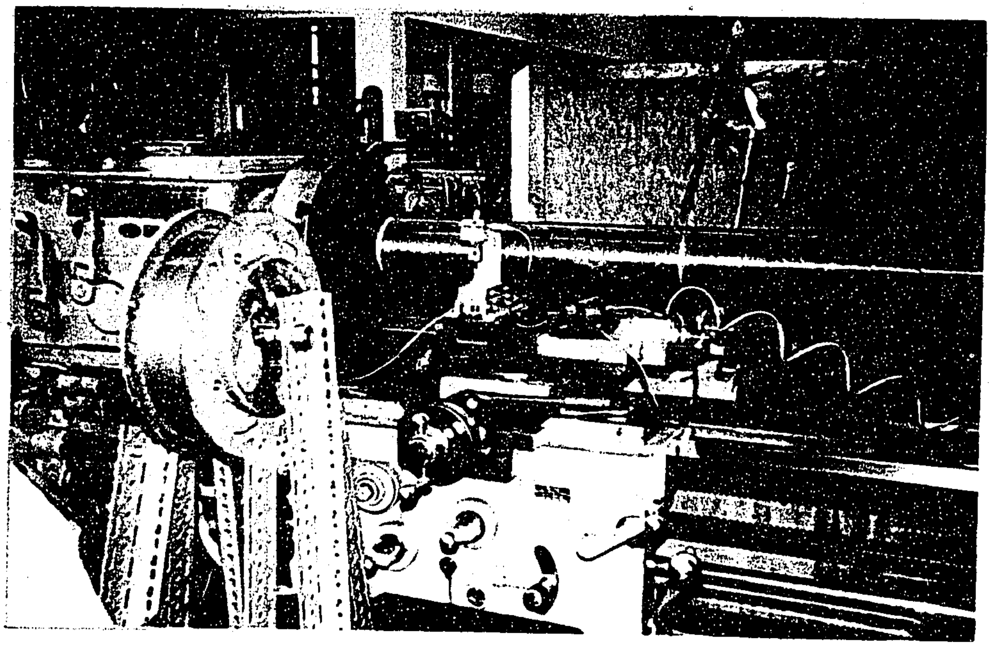
Fabrication

All the columns were cast in sono-tube-moulds. Sono-tubes were cut to the required lengths from pieces 8 feet long. Good care was taken to make sure that the ends of the tubes were parallel and plane.

A steel stand was designed to accommodate 12 moulds at a time and hold them vertical while casting.



a) WRAPPING SPECIMENS OF C-SERIES



b) WRAPPING SPECIMENS OF D-SERIES

PLATE 2 : WRAPPING OF SPECIMENS

(C & D SERIES)

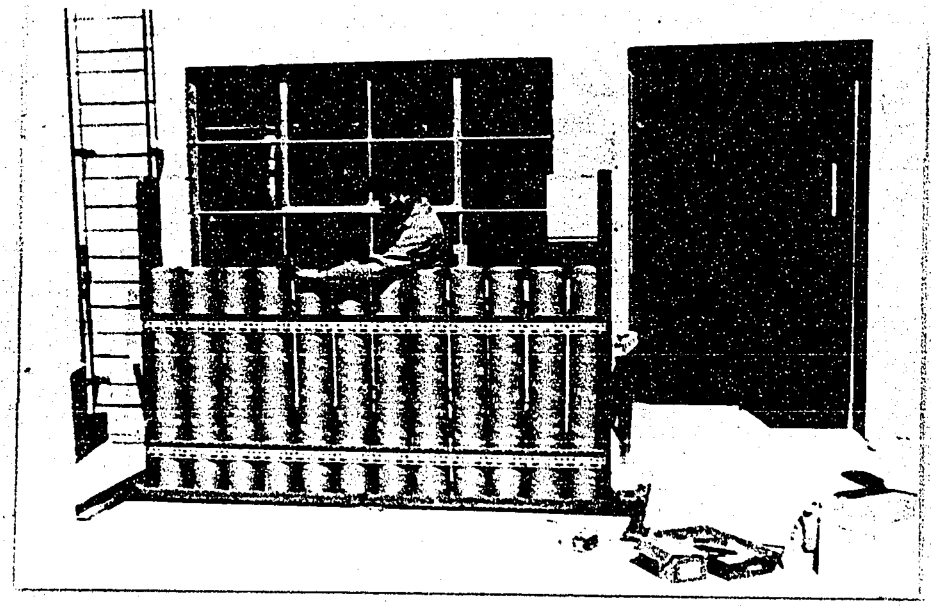
To enable the columns to be held in the lathe while wrapping and to provide end plates to load the specimen uniformly, steel plates were cast into the column at each end. Great care was taken to have the centres of the end plates in the centre of columns. It is obvious that any discrepancy between two end centres would have created a problem in turning as well as in wrapping of the columns because of wobbling.

Particular care was taken in attempting to compact the concrete uniformly and to provide equally complete compaction for 6 inch x 12 inch control cylinders. Compaction was achieved by rod-tamping according to the procedure outlined by C.S.A. standard A.23.2.17. All the columns of a particular series and the associated control cylinders in a series were cast at the same time and from the same batch of concrete mix to ensure uniformity.

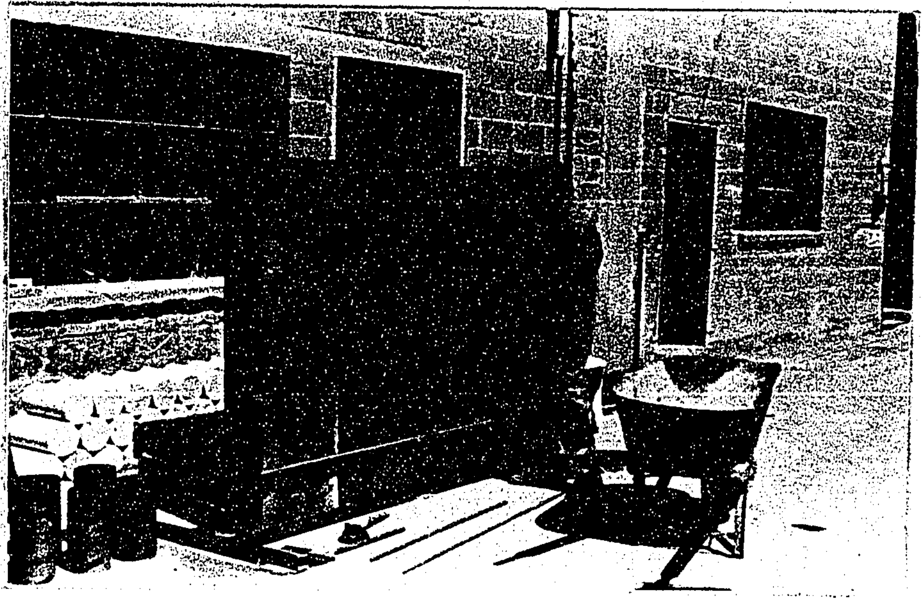
Curing

After casting, specimens were covered and left in the form for 12 to 18 hours. After stripping the forms, all the specimens were cured vertically in the curing room at the Francon Concrete Manufacturing Plant for a minimum of 15 days.

The specimens were taken out of the curing room and air dried for at least 7 days before wrapping.



a) CASTING SPECIMENS OF C-SERIES



b) CASTING SPECIMENS OF D-SERIES

PLATE 3 : CASTING OF SPECIMENS (C & D SERIES

3.5 TESTING PROCEDURE

Instrumentation

Longitudinal and circumferential strains were measured by electrical resistance foil type strain gages, 5 mm. gage length, type KF-5-C-11, Kyowa. Three longitudinal and three circumferential gages were bonded to the outside surface of each wrapped specimen. An epoxy material was applied over the piano wire to provide a base for the strain gages. The gages were located 120 degrees apart at the mid-height of the specimen.

The lateral deflection of the columns was measured at mid-height. The pilot tests showed that when the wire ruptured, the broken ends of the wire whipped about. In this type of failure, it was dangerous to try to read the deflection by means of dial gages. Three 8 inch long cantilevers, spaced at 120 degrees apart at mid-height were used to measure the lateral deflection. Any deflection was measured by strain gages at the base of the cantilevers.

These cantilevers were calibrated by applying a known displacement to the cantilevers and measuring the strain.

Testing

Each test specimen, spirally prestressed or otherwise, was loaded axially and concentrically to failure.

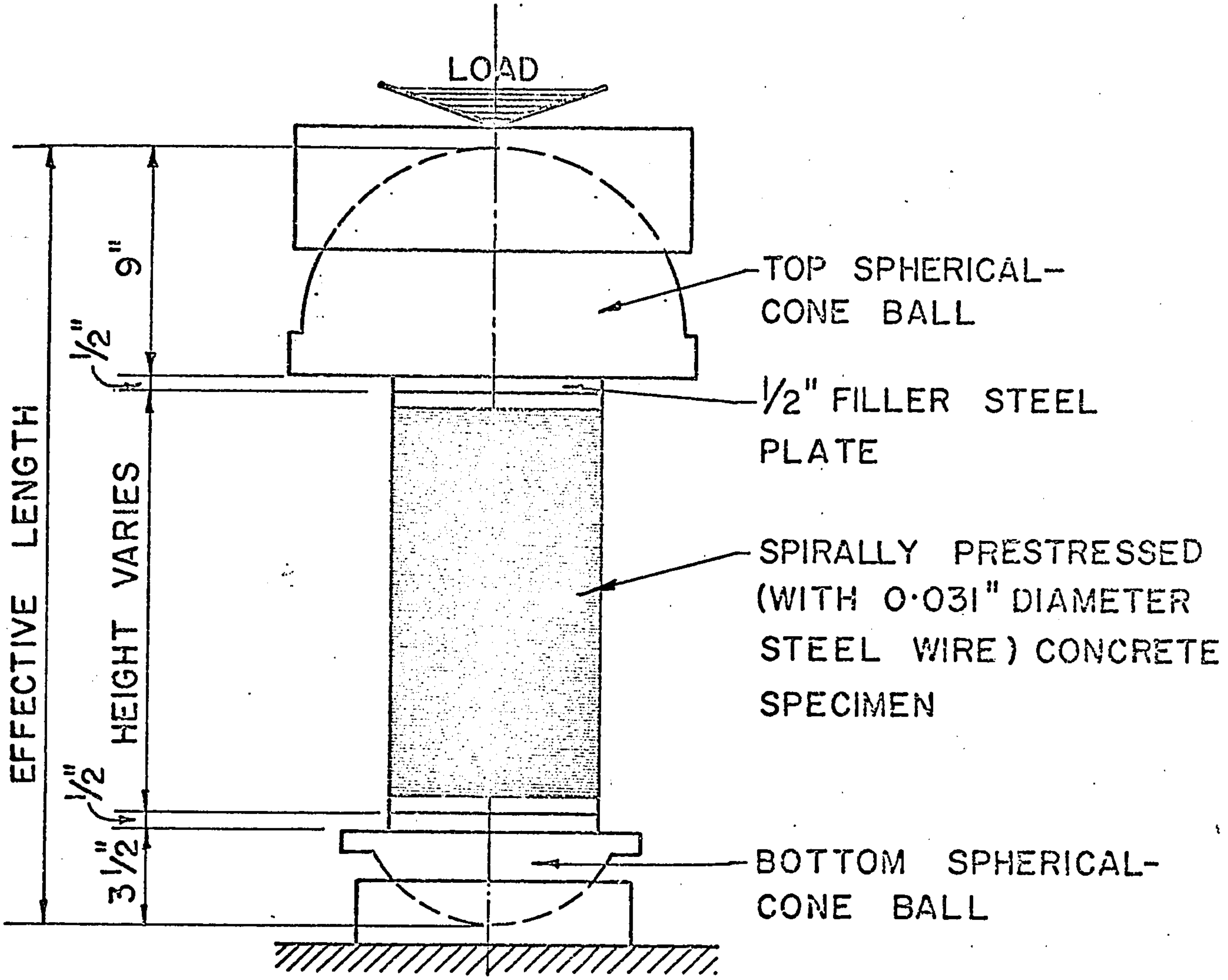


FIGURE 2: GENERAL ARRANGEMENT OF TESTING SPECIMEN

All specimens were tested in a hydraulically operated 600,000 lbs. capacity Badwin-Sowthwark Tate-Emery Testing Machine. Loads were applied through spherically-seated bearing blocks at the top and bottom of the columns. Initially, the specimens were loaded to approximately 5,000 lbs. to facilitate levelling and centering operations.

All deformation measurements were made at essentially equal load increments of 10,000 lbs. for wrapped specimens and 5,000 lbs. for the unwrapped specimens.

Tests on most specimens were run until the specimen failed by bursting of the wire wrapping.

Strain readings were recorded by a "Multichannel Digital Strain Indicator, 161-Mini-system, Programmer P1237, B & C Instruments Inc." with automatic strain Printer. The use of automatic strain recorder allowed readings of all the strain gages to be completed within a few seconds.

PLATE 4 : DETAILS OF SPECIMEN TESTING SYSTEM

PLATE 4: DETAILS OF SPECIMEN TESTING SYSTEM

CHAPTER 4

THEORETICAL CONSIDERATIONS

4.1 Analysis Of Wire Wrapped Concrete Cylinders

Figure 3, (a), (b) shows free body diagrams for the concrete and steel wire.

By equilibrium considerations across a diameter over a unit length,

$$\sigma_3 \times D = 2T$$

$$\therefore \quad \sigma_3 = \frac{2T}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.1)$$

Thus if the tension in the wire is known, then the equivalent lateral stress on the concrete is known.

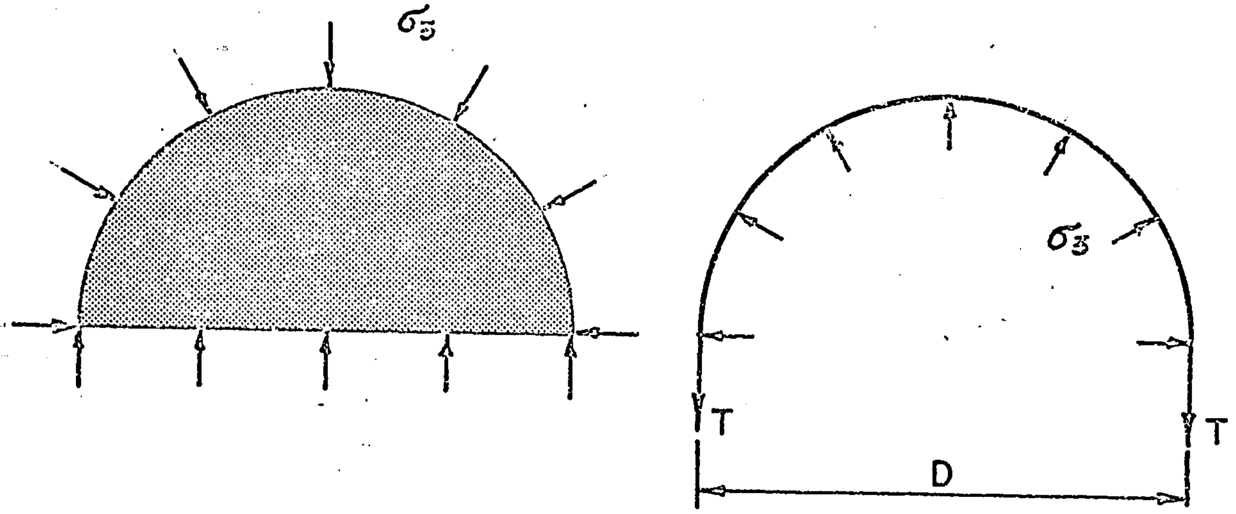
4.2 Behaviour Of Concrete Under Triaxial Stress

Figure 3, (d) shows concrete under triaxial compression.

It is generally accepted that the failure condition of triaxially compressed concrete can be represented by a Coulomb type equation as expressed by Richart, Brandtzneg and Brown (48), Balmer (5) etc.

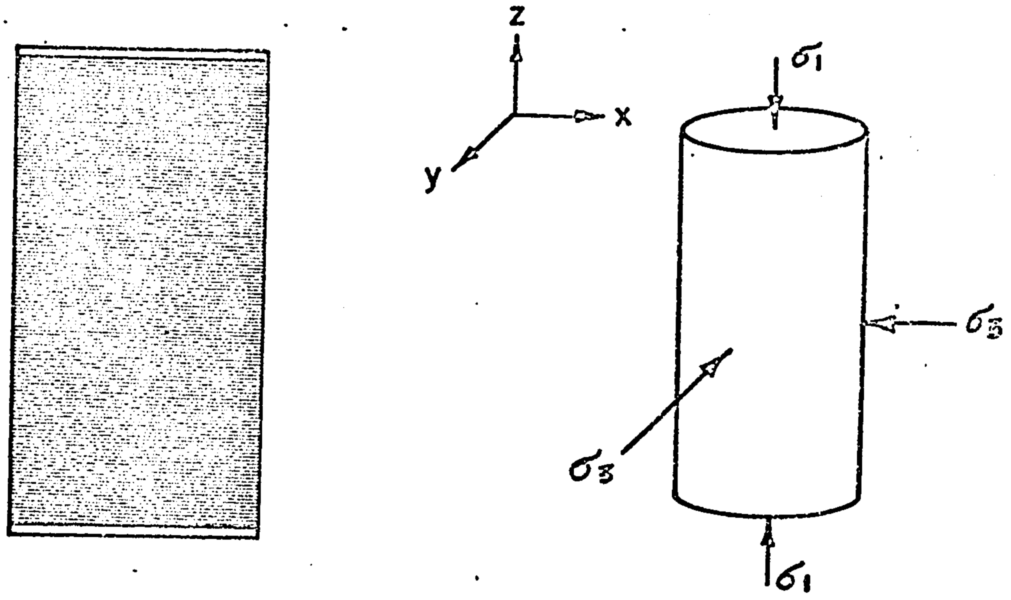
$$\sigma_1 = f_c' + K.\sigma_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.2)$$

The load deformation behaviour of triaxially compressed concrete has been hypothesized by Gardner (26) to be of the following form;



(a) CONCRETE (b) STEEL WIRE

FREE BODY DIAGRAMS



(c) SPIRALLY PRESTRESSED CONCRETE SPECIMEN

(d) SPECIMEN UNDER TRIAXIAL COMPRESSION

FIGURE 3: ANALYSIS OF SPIRALLY PRESTRESSED CONCRETE SPECIMEN

$$\frac{\sigma_1}{\nu} = f_c' + K.\ \sigma_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.3)$$

Where, ν = Instantaneous Poisson's Ratio.

Thus if the variation of Poisson's Ratio with axial strain can be estimated, the state of stress existing in the concrete can be estimated. Unfortunately, many problems are associated in trying to equate equilibrium and compatibility requirements between the inelastic steel wire and the inelastic triaxially compressed concrete. This analysis was not attempted in this study.

4.3 Effect Of Slenderness Ratio On Laterally Prestressed Concrete

Exact Solution

If the inelastic behaviour of a short spirally prestressed concrete cylinder is known, then it is relatively easy to use the tangent modulus approach to predict the behaviour of a long column.

Empirical Relationships

Ben-Zvi, Muller and Rosenthal (9) have proposed that the Richart, Brandtzaeg and Brown (48) equation can be modified to include the effect of slenderness ratio by multiplying K by a factor β giving the following equation;

$$\sigma_1 = (f_c' + p.f_s') + \beta K.\ \sigma_3 \ldots\ldots\ldots\ldots (4.4)$$

This formulae is logically wrong because it does not take into account the variation of concrete compressive stress, area of steel and area of concrete, but is included for the sake of completeness.

A possible correct form of the equation (4.4) with further modification may be;

$$\sigma_1 = \left\{\left(f_c' + \frac{As}{Ac}\cdot f_s'\right) + K.\sigma_3\right\}\beta \quad \ldots\ldots\ldots\ldots\ldots(4.5)$$

<u>ACI and NBC CODE REQUIREMENTS</u>

No specific requirements for spirally prestressed columns are given in either the ACI (American Concrete Institute) or NBC (National Building Code of Canada, 1965) building codes. However, general requirements relating to the effect of slenderness ratio on the strength of long columns are given by both the ACI and NBC codes.

The NBC strength reductions for length of compression members hinged at both ends (Section 4.5.4.17.(1) A (ii)) is;

$$R = 1.07 - 0.0075\,\frac{kh}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots(4.6)$$

The ACI strength reduction for length of compression members hinged at both ends (Section 318-916) is;

$$R = 1.07 - 0.008\,\frac{h}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots(4.7)$$

Where, $\frac{h}{r}$ = Slenderness ratio.

R = Reduction factor.

k = Effective length factor.

Graph 18 compares the above requirements with the findings of the present investigations.

CHAPTER 5

DISCUSSION OF TEST RESULTS

The results are presented in the form of graphs of longitudinal and circumferential strain against axial load for each column. These graphs are presented in Appendix B, from B-1 to B-20.

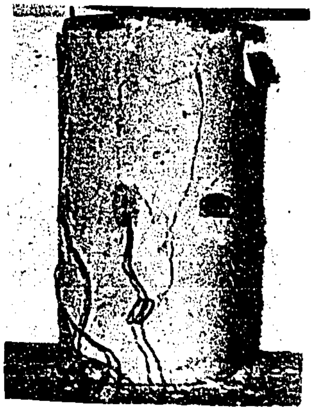
The average time of testing for all specimens varied between 20 and 30 minutes. The use of automatic strain recording equipment enabled a considerable saving in time.

A-SERIES

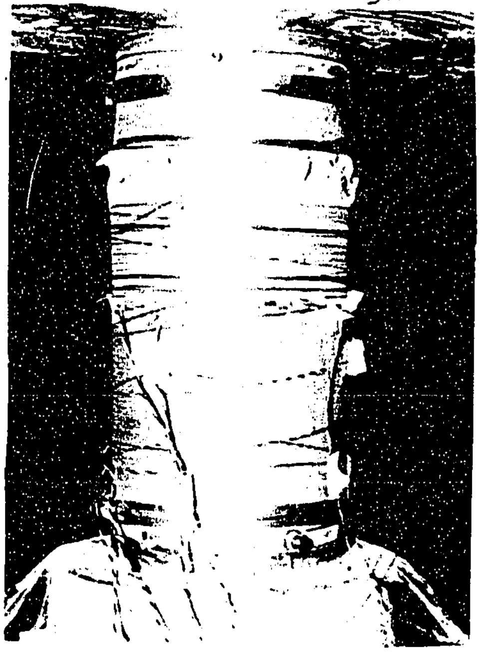
The variation between the behaviour of the nominally identical specimens of each group of columns can be seen to be small from Graphs B-1 to B-4.

Graph 1 summarizes the behaviour of all the columns in the A-series of tests. It can be seen that there is an increase in the ultimate load carrying capacity of columns which varies directly with the initial intensity of prestress. An increase of the magnitude of the prestress also increased the stiffness of the column.

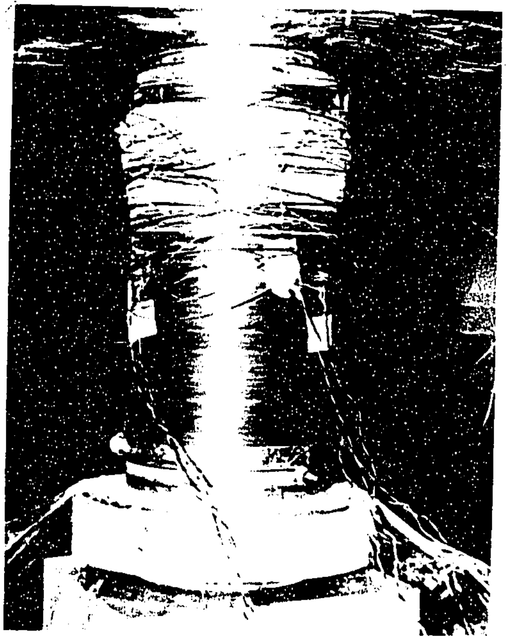
All specimens failed by the rupture of the wire causing a loss of lateral stress and consequent catastrophic disintegration of the specimen. As the specimens were loaded,



A.O.O SERIES



A.18.1 SERIES



A.32.1 SERIES

A.64.2 SERIES

PLATE: 5: TYPICAL FAILURES OF SPECIMENS — A-SERIES the concrete deformed laterally, due to a Poisson's effect, increasing the stress in the prestressing wire.

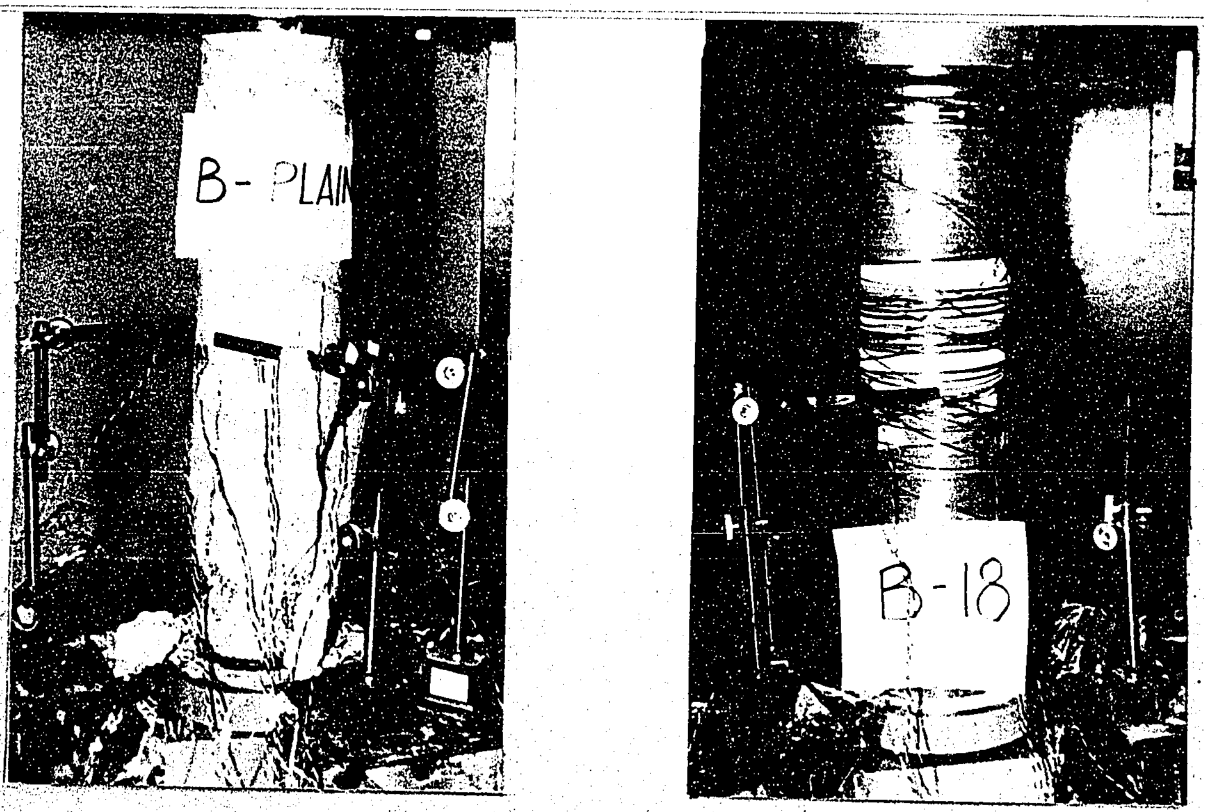
Rupture of the wire occurred on several strands of wire, the loose ends flailing about, and the concrete completely fractured across the section. Typical failures of the specimens in A-series are shown in Plate 5.

Most of the spirally prestressed specimens failed at or close to mid-height. Plain concrete specimens failed by crushing the concrete at mid-height.

Excluding the plain concrete specimens, the axial stiffness of columns increased as the prestress increased. No quantitative reason has been found for the drop in stiffness between the plain concrete and the prestressed concrete but this is probably due to the initial prestress changing the microstructure of the concrete. The same effect can be seen in the results presented by Feeser (23).

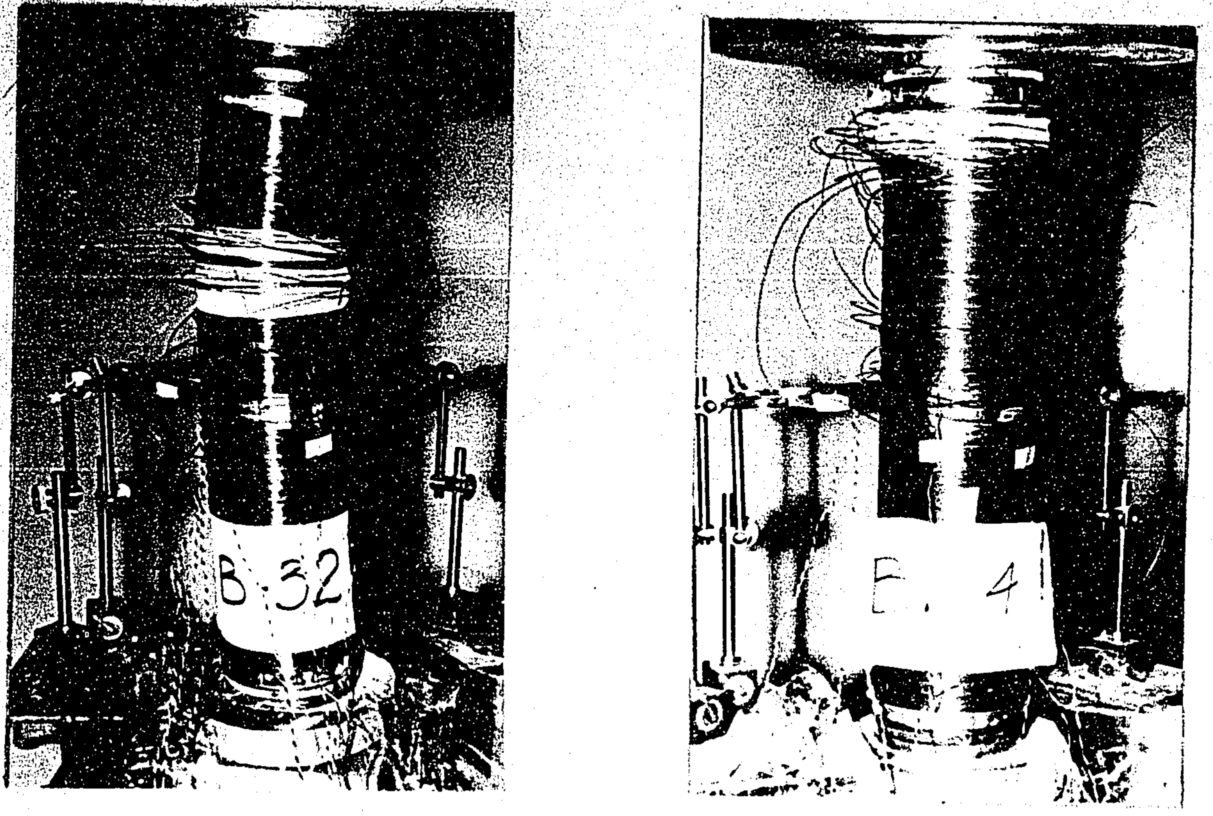
B-SERIES

The individual results for each column of B-series are presented in Graphs B-6 to B-9 in Appendix B. The results are summarized in Graph 2 showing the variation of behaviour with the magnitude of the initial prestress. The behaviour is similar to the A-series with increase in initial prestress giving increased load carrying capacity and stiffness with the reservation detailed in the description of the A-series results.



B.0.0 SERIES

B.18.1 SERIES



B.32.1 SERIES

B.64.2 SERIES

PLATE 6: TYPICAL FAILURES OF SPECIMENS — B·SERIES
(NOTE "CANTILEVERS" FOR LATERAL DEFLECTION)

Graph 5 summarizes the load-lateral deflection behaviour of the series. The increase in initial prestress delays the buckling of every specimen.

Most of the specimens failed at or close to mid-height. Plain concrete specimens failed by crushing of the concrete at the ends of the specimens. Typical failures of specimens in B-series are shown in Plate 6.

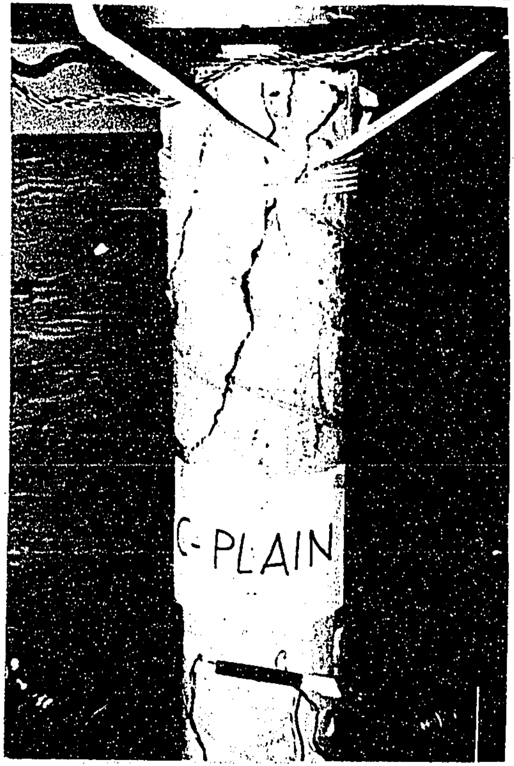
C-SERIES

As noted previously, this series of specimens had 4 - #3 bars as a longitudinal reinforcement and #2 ties at 6 inches centre to centre to take care of handling and transportation loads.

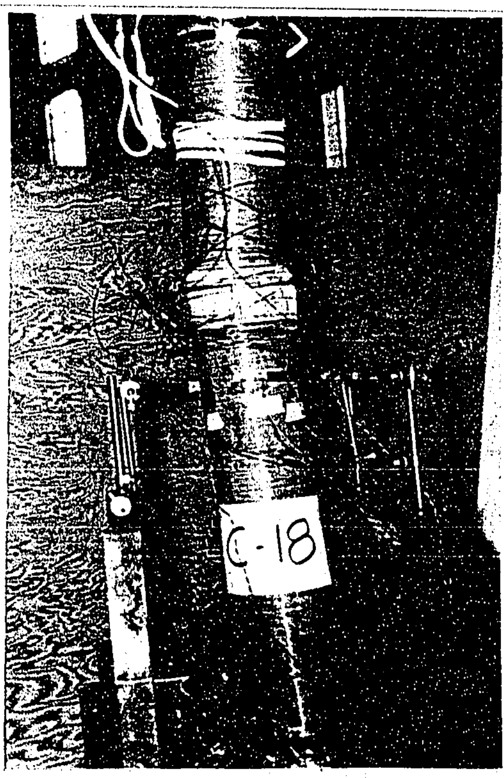
The individual column results are presented in Graphs B-11 to B-14 in Appendix B. The summary of the results is shown in Graph 3. Graph 6 summarizes the load-lateral deflection behaviour of the series. It is interesting to note that in the case of C.64.2 series, the buckling becomes the main criterion for failure of the specimens.

In analyzing the results of this series, the load contributed by the longitudinal reinforcing is deducted in calculating empirical factors like K, β, R', and M, which are tabulated in Table 3 and Table 4.

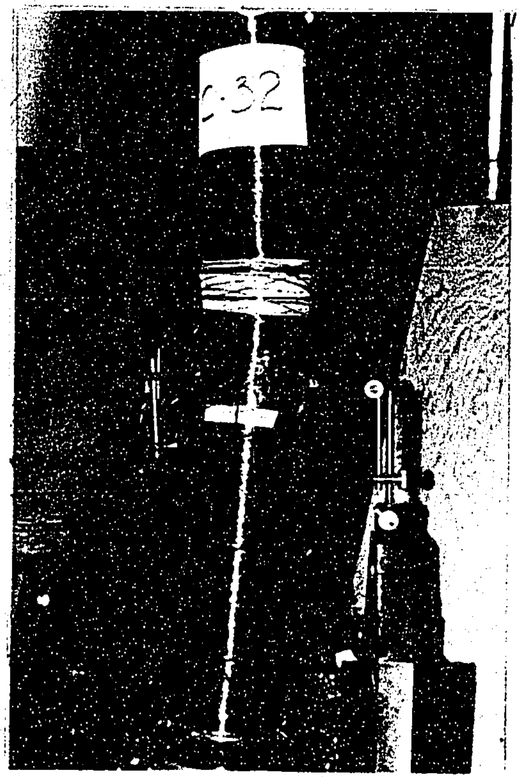
In this series, specimens with initial prestress of 990 psi. and 1,706 psi. (i.e. C.18.1 and C.32.1 series)



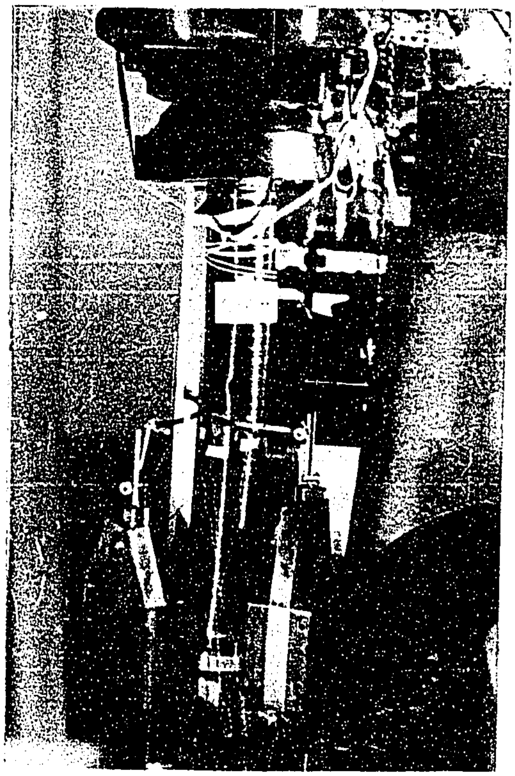
C.O.O SERIES



C.18.I SERIES



C.32.I SERIES



C.64.2 SERIES

PLATE 7: TYPICAL FAILURES OF
SPECIMENS — C·SERIES

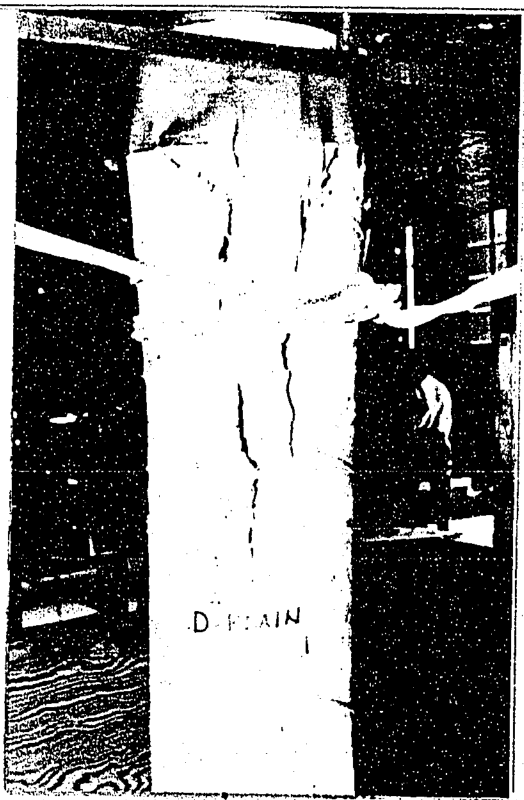
failed by buckling and the rupture of the wrapped wire. It was most unfortunate that the rupture of the spirally prestressed wire at ultimate load caused a sudden dynamic shock to the top spherical-ball attachment. This shock was so severe that it broke the holding bolt of the top spherical-ball, which then dropped, damaging various parts of the testing machine. To avoid this danger, the remaining specimens in this series, i.e. C.64.2 series, were not loaded to the actual failure, but only to an "apparent" yield load. Therefore, the actual failure load of the subsequent specimens may be somewhat higher than recorded. Plain concrete specimens failed by crushing at the top.

Typical failure of specimens in this series are shown in Plate 7.

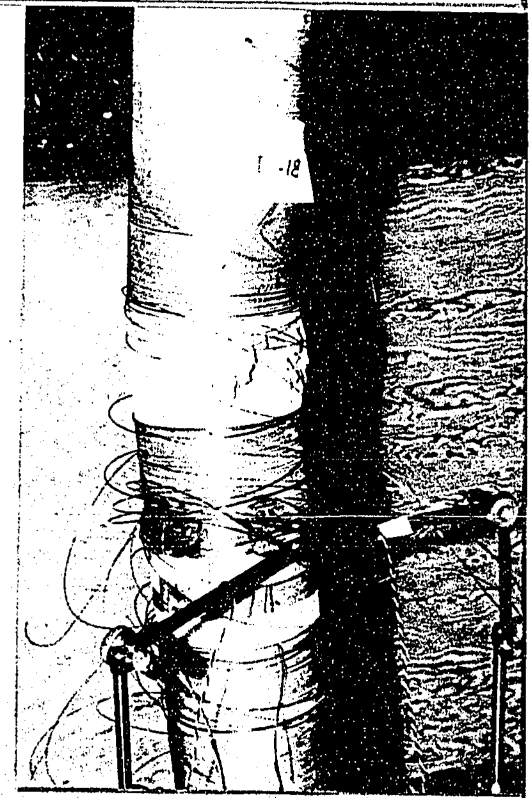
D-SERIES

Specimens in this series were also reinforced longitudinally with 4 - #3 bars and had #2 ties at 6 inches centre to centre.

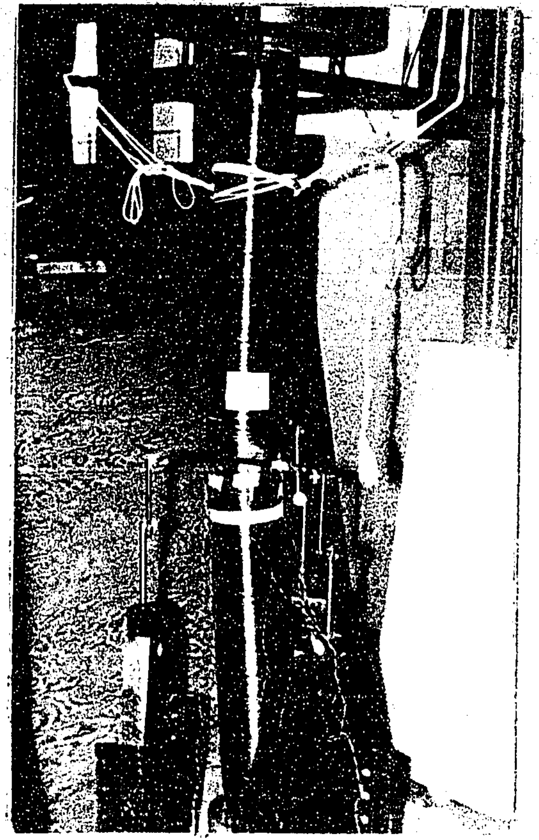
In wrapping the specimens in this series, it was difficult to maintain the same initial lateral stress because the wire supplied had many kinks and was badly wound on the spool. Therefore, to continue the rest of the project, the initial lateral stress was reduced for all the specimens. Thus D-series had initial lateral prestresses of 900, 1,270 and 2,540 psi. respectively.



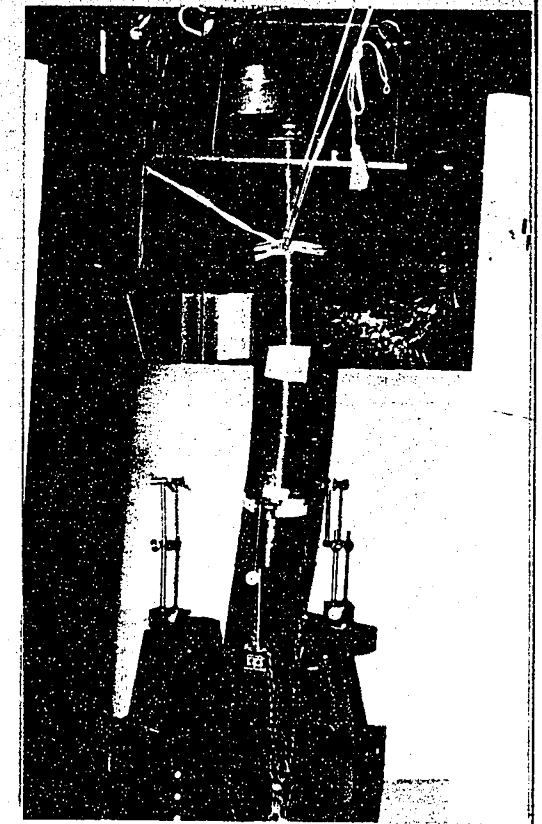
D.O.O SERIES



D. 18.1 SERIES



D. 32.1 SERIES



D.64.2 SERIES

PLATE 8: TYPICAL FAILURES OF SPECIMENS — D·SERIES

Individual results for each column are presented in Graphs B-16 to B-19 in Appendix B. Graph 4 summarizes the results for the complete series which exhibit a behaviour similar to the previous series. Graph 7 summarizes the load-lateral deflection behaviour of the D-series. Similar to C-series, D.64.2 specimens showed buckling as a governing factor for failure.

In analyzing the results of this series, the load contributed by the longitudinal reinforcing is deducted in calculating empirical factors like K, β, R', and M.

In this series, specimens with initial lateral prestresses of 900 psi. and 1,270 psi. (i.e. D.18.1 and D.32.1 series) failed by buckling and the rupture of the wrapped wire. Specimens with an initial lateral prestress of 2,540 psi. (i.e. D.64.2 series) failed by buckling at "apparent" yield load. Plain concrete specimens failed by crushing of the concrete at the top.

Typical failures of specimen in this series are presented in Plate 8.

Comparison Of Results With Those Of Other Investigators

Values of the empirical coefficient K have been back calculated from the experimental results and are presented in Table 4. These values were calculated using the ultimate value of the lateral prestress,not the initial value. The K values decrease with increase in lateral prestress. The results from this study agree with Balmer's (5) results for tests at high lateral stresses where K ranged between 2.0 and 3.0 and with Ben-Zvi's (9) results of K between 2.72 and 3.91 for short columns. The relation of axial ultimate stress to lateral ultimate stress is presented in Graphs 12 to 15.

The effect of the percentage of lateral reinforcing on the load carrying capacity of columns of various lengths is shown in Table 4. The efficiency of the spiral is also presented in Table 4. The efficiency ratio, M , is defined as the increase in load due to spirally prestressed reinforcement divided by the load taken by the same amount of steel at the same stress if used as longitudinal reinforcement.

The variation of the efficiency ratio, M, with the slenderness ratio is plotted in Graph 17. The present results are in close agreement with those of Ben-Zvi, Muller and Rosenthal (9), Gamborov (24) and Johnson (35).

TABLE 3 - DETAILS OF TESTS

Mark	Compressive Stress at the time of testing, f_c'	Initial lateral stress σ_3 initial	Average failure load	Load taken by Longitudinal steel P_3	Net ave. failure load (4-5)	Ave. axial failure stress $\sigma_1 = \frac{\text{load}}{\text{area}}$	Increase in load due to spirals	Lateral stress at failure* $\sigma_{3\,\text{ultimate}}$	Axial stress $\sigma_1 - f_c'$ (7-2)	Coefficien R' **
1	2	3	4	5	6	7	8	9	10	11
Units	ksi.	ksi.	kips.	kips.	kips.	ksi.	kips.	ksi.	ksi.	-
A--0-0	4.600	0.00	112	-	112	3.96	0	0	0	1
A-18-1	4.600	0.960	290	-	290	10.25	178	1.452	5.65	1
A-32-1	4.600	1.706	374	-	374	13.200	262	2.581	8.60	1
A-64-2	4.600	3.412	573	-	573	20.200	461	5.162	15.60	1
B--0-0	5.100	0.00	111	-	111	3.930	0	0	0	0.990
B-18-1	5.100	0.960	284	-	284	10.04	173	1.452	4.94	0.978
B-32-1	5.100	1.706	364	-	364	12.90	253	2.581	7.80	0.965
B-64-2	5.100	3.412	545	-	545	19.25	434	5.162	14.15	0.952
C--0-0	5.000	0.00	130	28.3	101.7	3.60	0	0	0	0.905
C-18-1	5.000	0.960	290	28.3	261.7	9.20	160	1.452	4.20	0.900
C-32-1	5.000	1.706	352	28.3	323.7	11.40	222	2.581	6.40	0.865
C-64-2	5.000	3.412	407	28.3	378.7	13.40	277	5.162	8.40	0.660
D--0-0	4.950	0.00	126	28.3	97.7	3.46	0	0	0	0.872
D-18-1	4.950	0.900	268	28.3	239.7	8.45	142	1.452	3.50	0.825
D-32-1	4.950	1.270	310	28.3	281.7	9.90	184	2.581	4.95	0.751
D-64-2	4.950	2.540	348	28.3	319.7	11.30	222	5.162	6.35	0.556

* Rupture stress of wire = 342 ksi.

** Coefficient R' = Ultimate load of column/ultimate load of corresponding short column.

Ben-Zvi proposed a modification of the Richart, Brandtzaeg and Brown (48) formulae to include the effect of slenderness.

$$\sigma_1 = (f_c' + p.f_s') + \beta K.\sigma_3 \ldots\ldots\ldots\ldots\ldots (5.1)$$

Values of the empirical coefficient βK are presented in Table 4, and in Graph 16. These values are compared to the results from Ben-zvi, Muller and Rosenthal (9) experiments. All three sets of results are in close agreement.

Effect Of Slenderness Ratio On Load Carrying Capacity

Graphs 1-4 and 8-10 show the effect of slenderness ratio on the load-deformation characteristics for each of the four levels of lateral prestress. The plain concrete specimens show an unusual tendency in that the long columns (C and D series) are stiffer and stronger than the short columns (A and B series). This is, of course, due to the presence of longitudinal reinforcing bars.

Graphs 5 to 7 show that the length of the column increases the lateral deflection. The variation of ratio, R', of the long column load to short column load with slenderness ratio is given in Graph 18. Also shown are the ACI and NBC requirements for strength reduction for lengths of compression

TABLE 4: CALCULATION OF "EMPERICAL COEFFICIENTS"

Mark	"βK" (10/9)	"βK" £ as per Ben-Zvi et.al.(9)	Weight of wire/ sample	Equivalent area of steel	Load taken by steel in (15) if used as longitudinal rfg.	Efficiency of spiral "M" (8)/(16)	"M" # as per Ben-Zvi	Emperical Coefficients $\sigma_{1\,ult.} = f_c' + \beta K.\sigma_{3\,ult.}$			% of spiral rfg. by vol. of conc.	% of longitudinal rfg. by vol. of conc.
								K	β 12/19	'β' by Ben-Zvi		
1	12	13	14	15	16	17	18	19	20	21	22	23
	-	-	lbs.	sq.inch	kips.	-	-	-	-	-	-	-
A--0-0	-	-	-	-	-	-	-	-	-	-	-	-
A-18-1	3.90	3.55	0.835	0.254	87.0	2.02		3.90	1	1	1.59	-
A-32-1	3.32		1.48	0.450	154.0	1.70	1.90	3.32	1	1	2.83	-
A-64-2	3.02	2.61	2.96	0.900	308.0	1.50	1.32	3.00	1	1	5.65	-
B--0-0	-	-	-	-	-	-	-	-	-	-	-	-
B-18-1	3.40	3.52	1.77	0.258	88.40	1.96		3.90	0.87		0.95	-
B-32-1	3.02		3.16	0.460	157.00	1.61	1.65	3.32	0.91	0.98	1.70	-
B-64-2	2.74	2.45	6.32	0.920	315.00	1.37	1.25	3.00	0.910	0.87	3.40	-
C--0-0	-	-	-	-	-	-	-	-	-	-	-	
C-18-1	2.90	3.10	3.08	0.258	88.40	1.81		3.90	0.744		0.985	1.49
C-32-1	2.48		5.49	0.460	157.00	1.41	1.57	3.32	0.747	0.86	1.75	1.49
C-64-2	1.63	1.90	10.98	0.920	315.00	0.88	0.96	3.00	0.544	0.70	3.50	1.49
D--0-0	-	-	-	-	-	-	-	-	-	-	-	2.12
D-18-1	2.40	2.00	4.87	0.258	88.40	1.72		3.90	0.615		1.09	2.12
D-32-1	1.94		7.80	0.460	157.00	1.33	1.0	3.32	0.585	0.58	1.74	2.12
D-64-2	1.23	0.90	15.60	0.920	315.00	0.76	0.50	3.00	0.410	0.36	3.48	2.12

£ Calculated from graphs.
Calculated by Godse using the ultimate strength of wire in the experiments.

members (pin ended). It is not as effective to use two layers of wire to achieve a given value of lateral prestress as it is to use a single layer.

If single layers of wire wrapping only are considered then the results presented in Graph 18 show that laterally prestressed concrete follows the same trend as unreinforced plain concrete, albeit, with a very high crushing strength.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The principal conclusions to be drawn from this study are:

1. It is possible to increase the load carrying capacity of concrete columns by prestressing the column by wrapping the column with wire under tension.

2. The load carrying capacity of a short column of given area increases as the lateral prestress increases as defined by the following equation;

$$\sigma_{1_{\text{ultimate}}} = f_c' + K \cdot \sigma_3 \ldots\ldots\ldots\ldots\ldots\ldots (6.1)$$

   K varied from 3.02 to 3.90.

3. As the spirally wrapped concrete is loaded, the concrete compresses axially and expands laterally, thus increasing the strain in the wire and the lateral stress on the concrete. Failure occurs when the concrete starts expanding laterally faster than the axial strain; the steel wire keeps straining until it fractures. As soon as the steel fractures, the lateral stress on the concrete suddenly goes and the concrete loses its high load carrying capacity and catastrophically disintegrates.

4. The catastrophic effect of loss of lateral prestress means great care must be taken to avoid overloading spirally prestressed concrete columns. (The external prestressing wire must also be protected against fire hazards.)

5. Spirally prestressed concrete columns behave in a manner similar to plain concrete columns, and can be analyzed in the same way using the ultimate triaxial concrete strength in place of the cylinder strength.

6.2 Recommendations For Future Work

1. To study the behaviour of spirally prestressed concrete under varying eccentric loading. The variables should be lateral prestress and slenderness ratio.

2. Study of long-term loading and creep behaviour of spirally prestressed concrete columns.

3. Determine the behaviour of spirally prestressed columns under repeated loading.

4. Response of spirally prestressed columns to blast loading. This study will be of particular interest to earthquake design as well as bomb-shelters.

5. Design of a machine, similar to a merry-go-round, for wrapping columns vertically, i.e. columns in situ, to solve problems of handling and transportation, and thereby, to reduce the cost of fabrication.

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ASCE = American Society of Civil Engineers
ASTM = American Society of Testing Materials
ENR = Engineering News-Record
MCR = Magazine of Concrete Research

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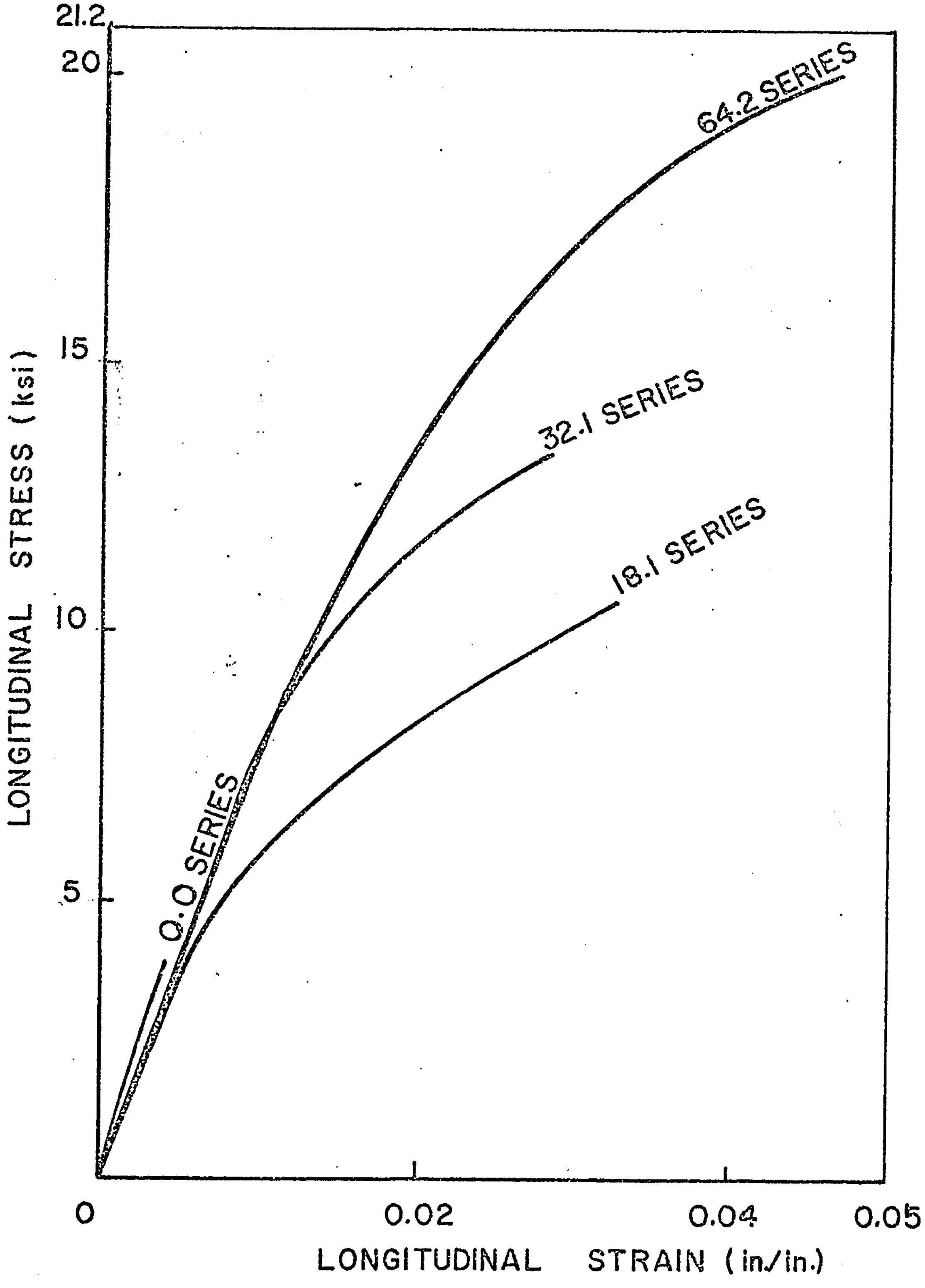
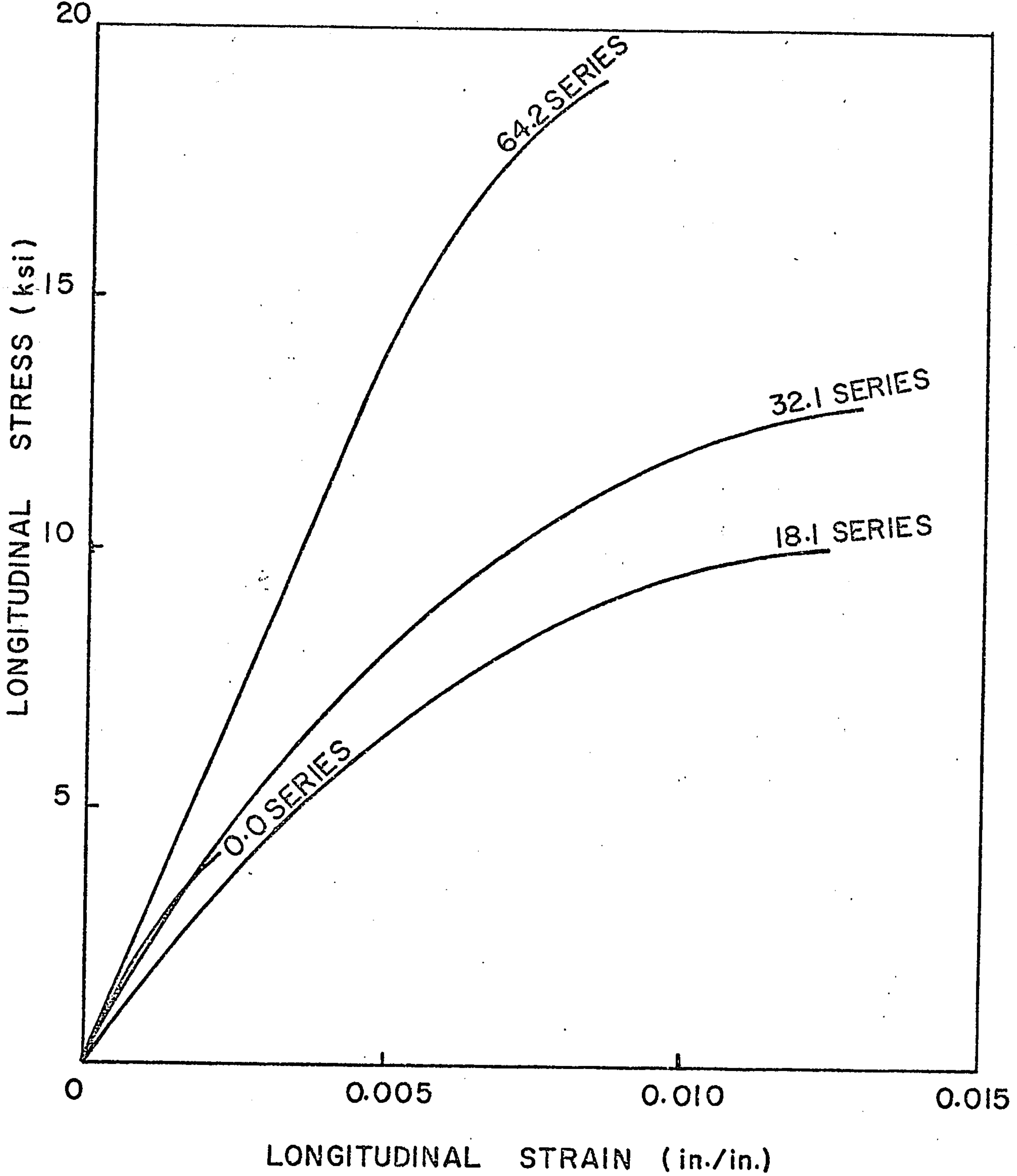
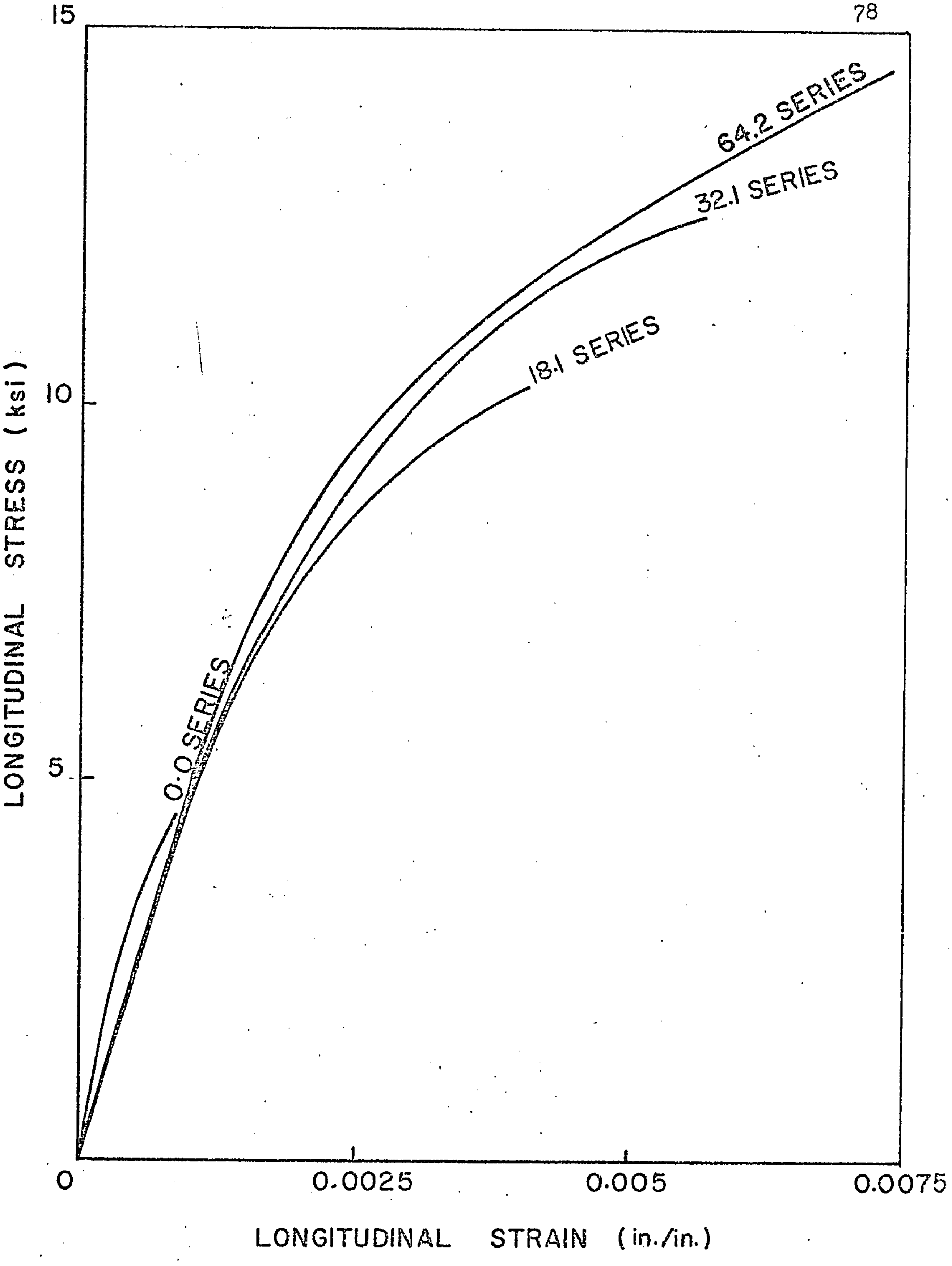


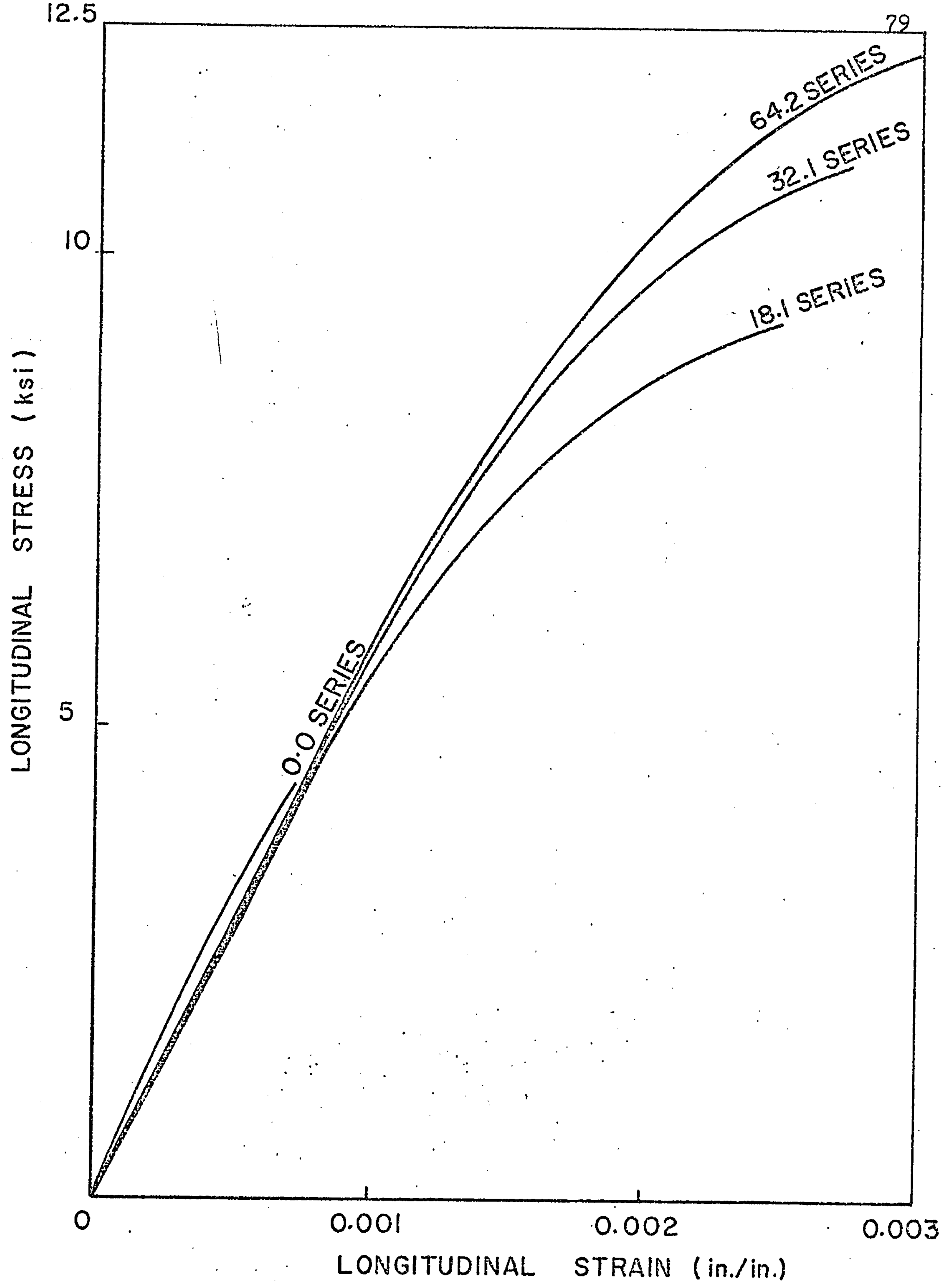
GRAPH 1: LONGITUDINAL STRESS-STRAIN CHARACTERISTIC OF A-SERIES



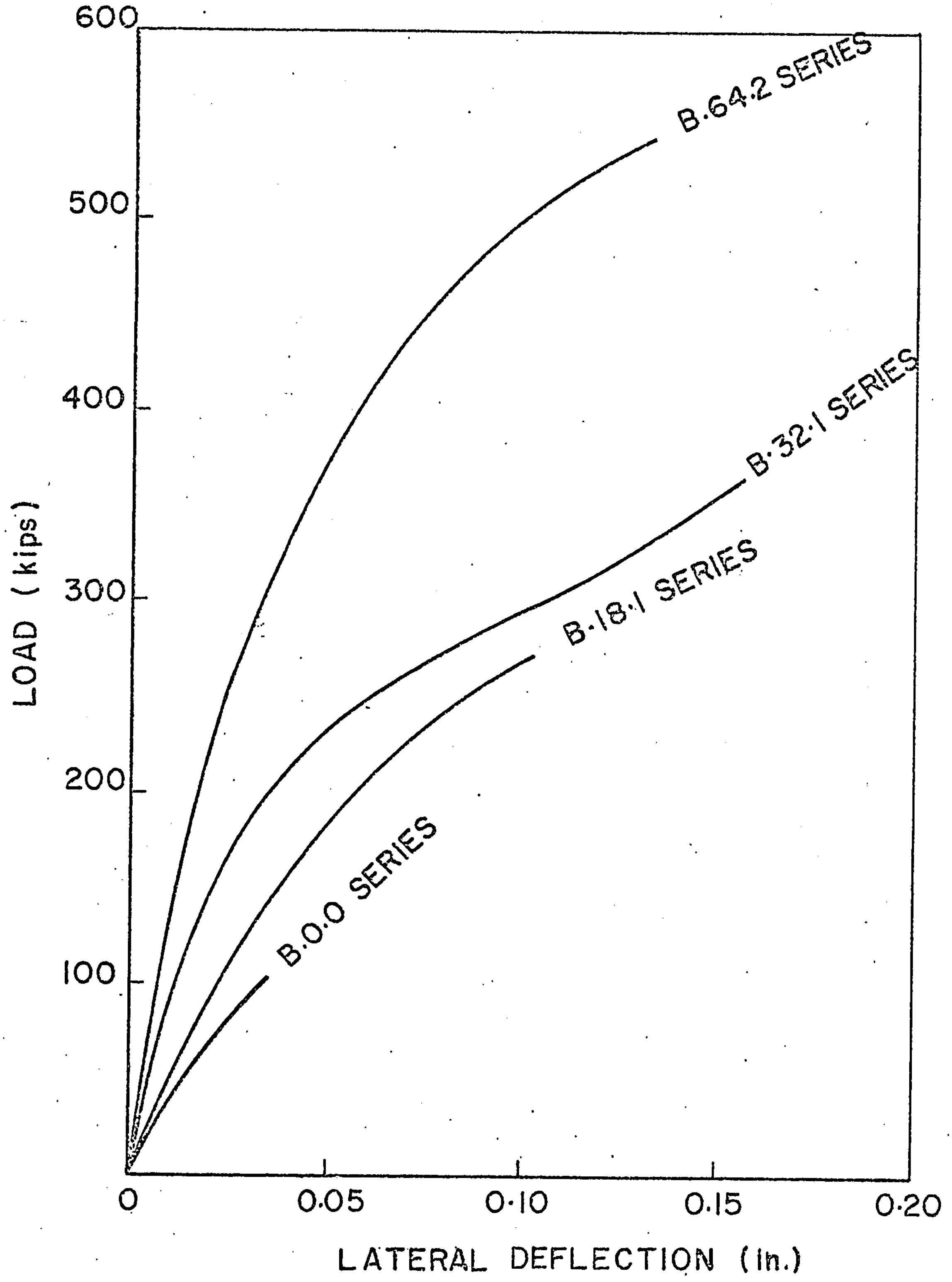
GRAPH 2 : LONGITUDINAL STRESS-STRAIN CHARACTERISTIC OF B-SERIES



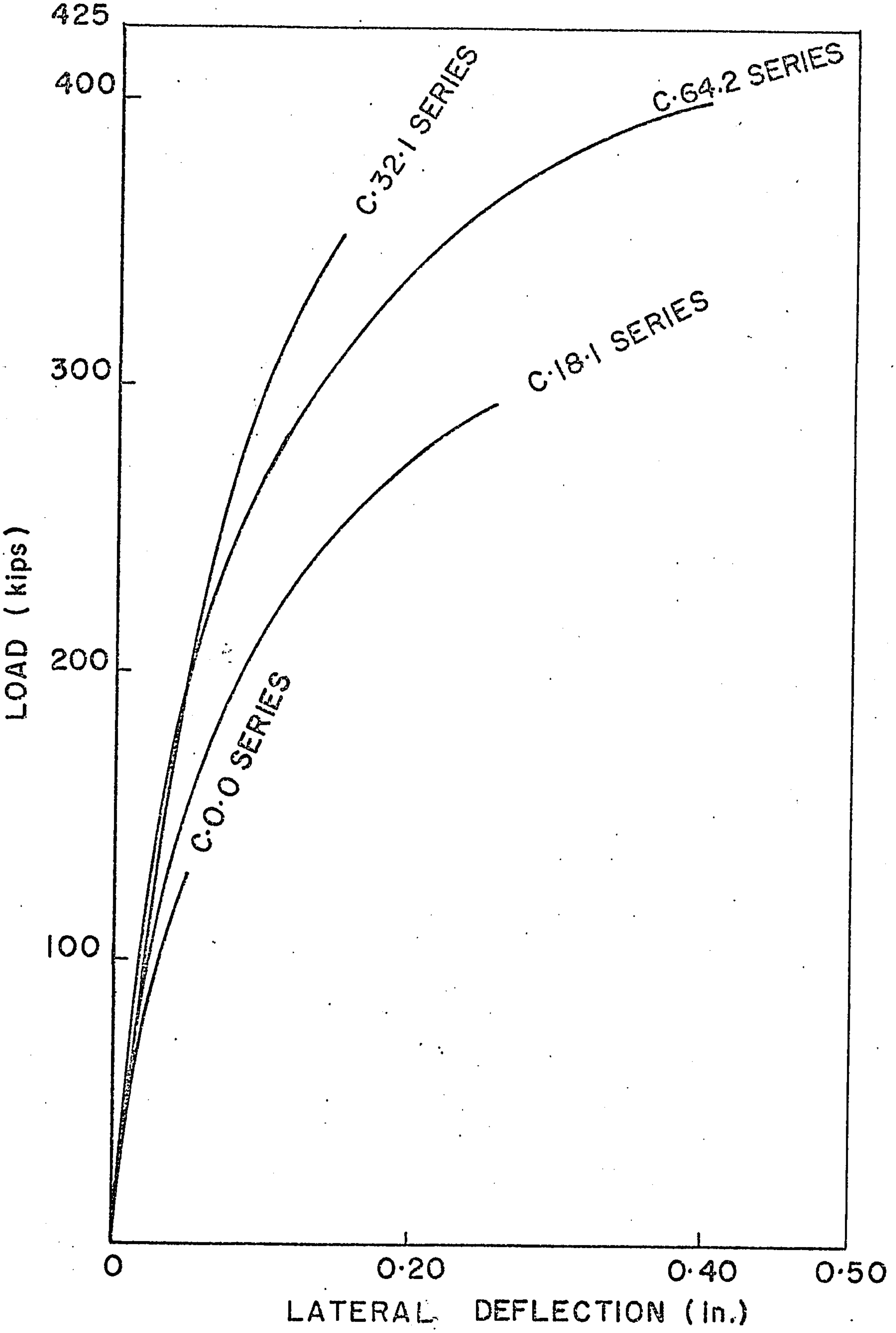
GRAPH 3 : LONGITUDINAL STRESS-STRAIN CHARACTERISTIC OF C-SERIES



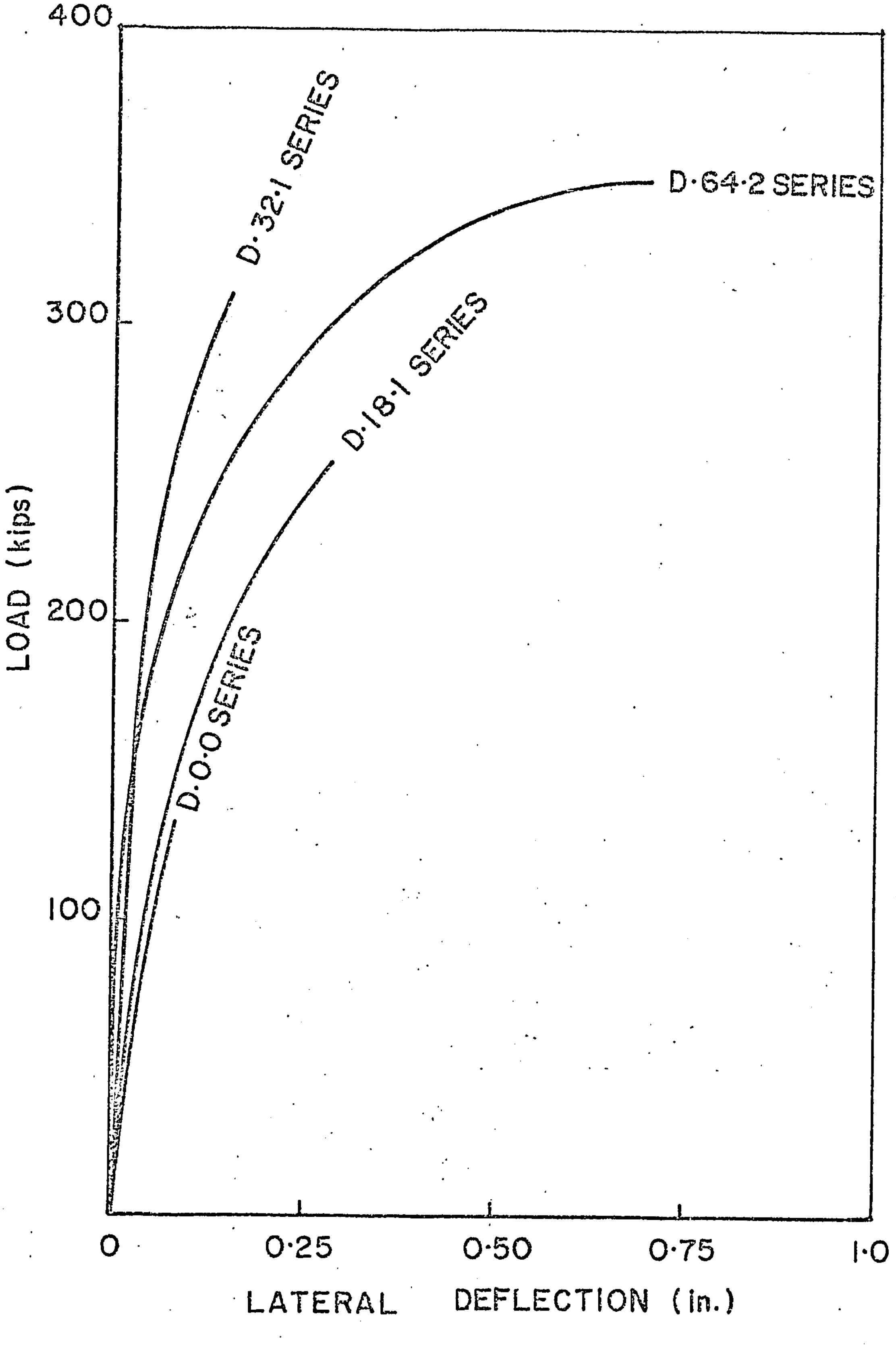
GRAPH 4 : LONGITUDINAL STRESS-STRAIN CHARACTERISTIC OF D-SERIES



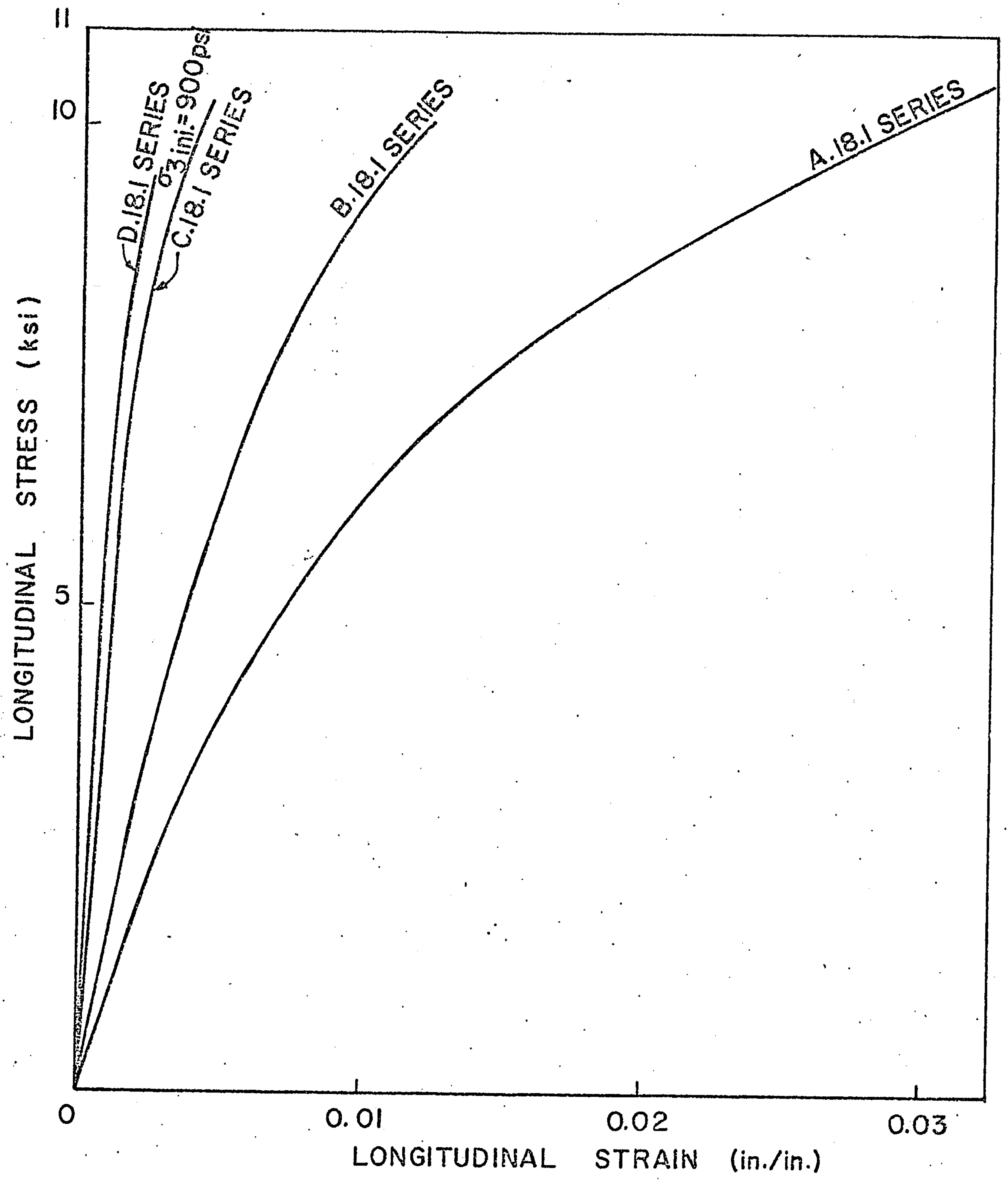
GRAPH 5 : LOAD-LATERAL DEFLECTION OF SPECIMENS : B—SERIES



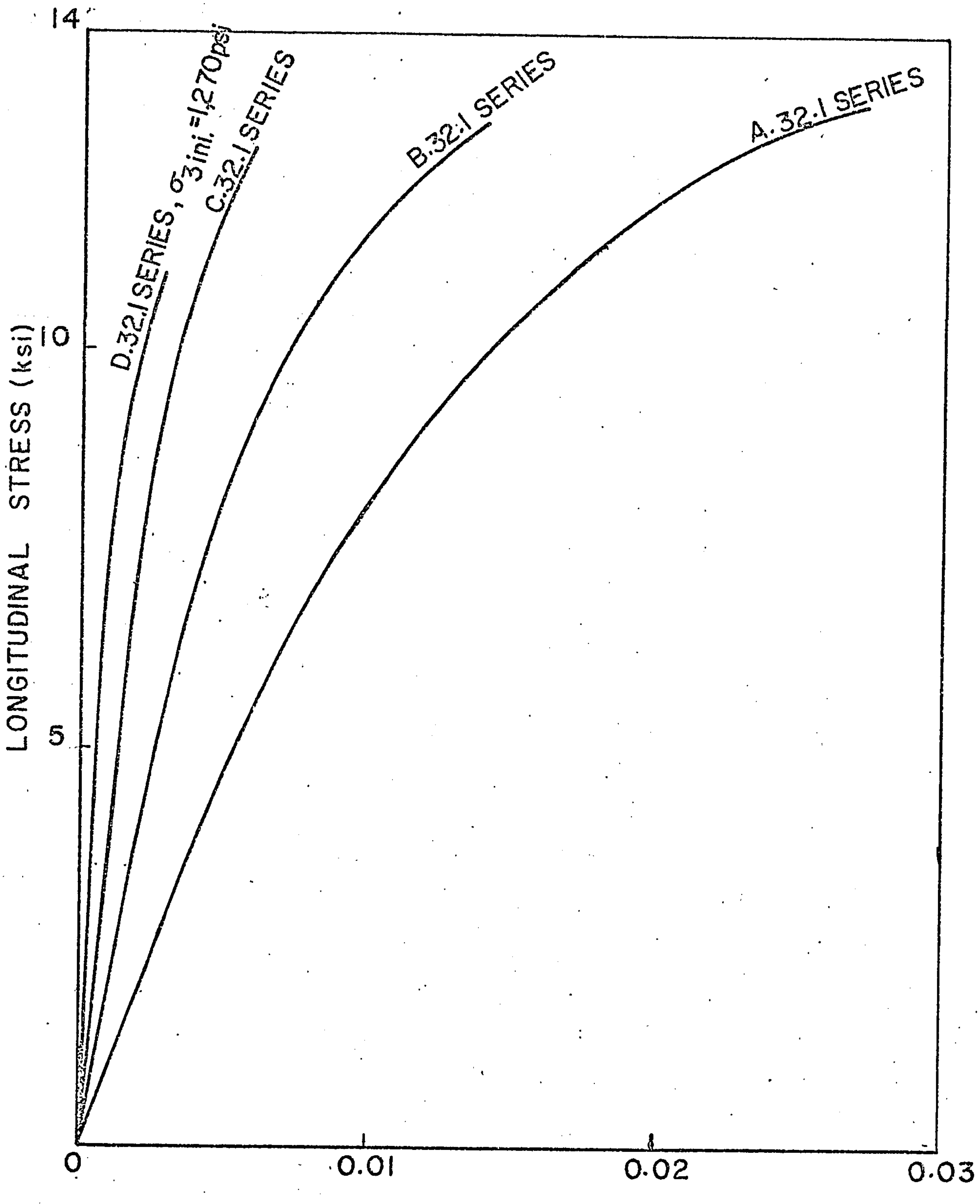
GRAPH 6: LOAD–LATERAL DEFLECTION OF SPECIMENS : C–SERIES



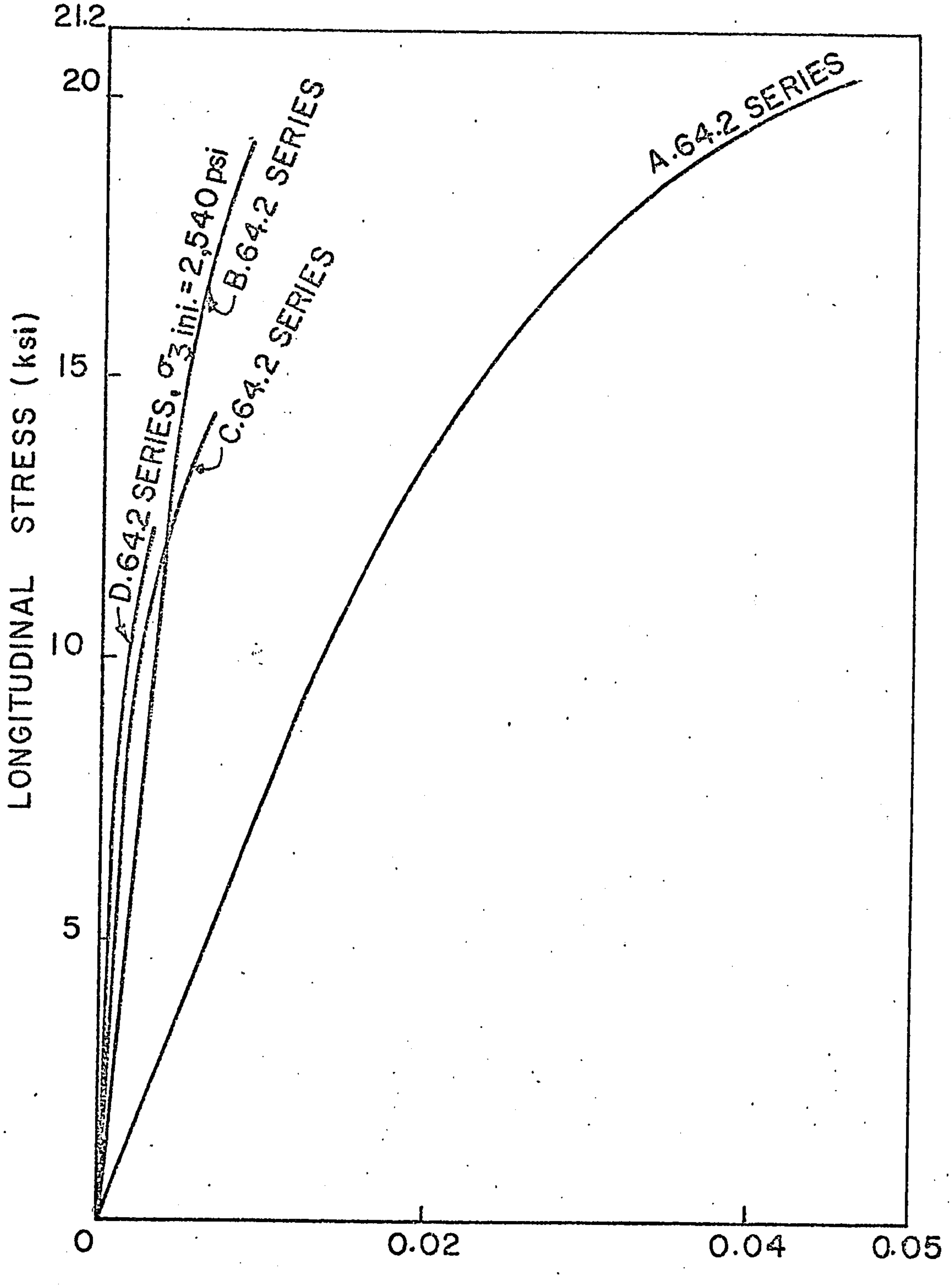
GRAPH 7 : LOAD-LATERAL DEFLECTION OF SPECIMENS : D-SERIES



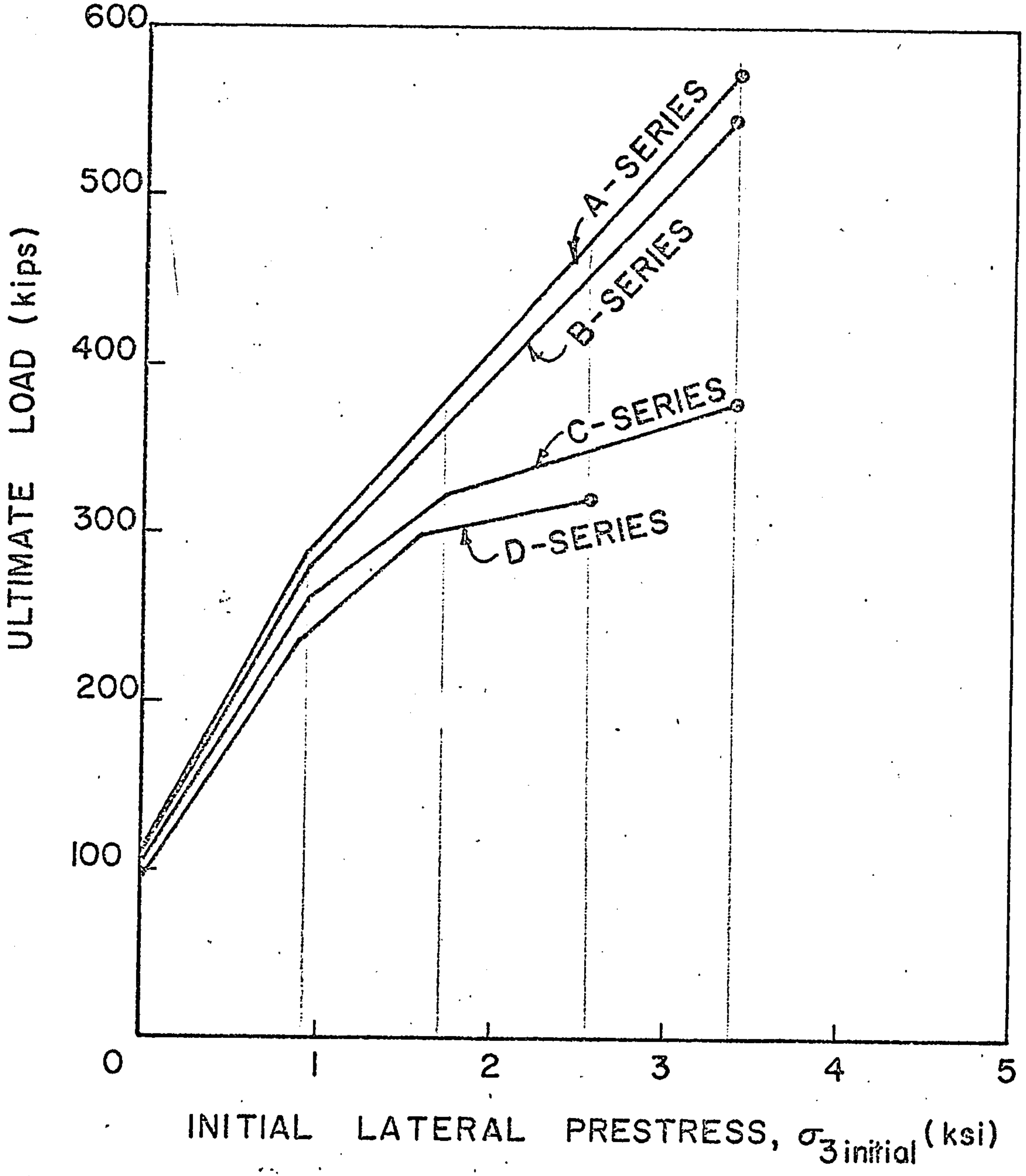
GRAPH 8 : LONGITUDINAL STRESS-STRAIN CHARACTERISTIC WITH INITIAL LATERAL PRESTRESS, $\sigma_{3\ \text{initial}}$, = 960 p.s.i. (unless noted)



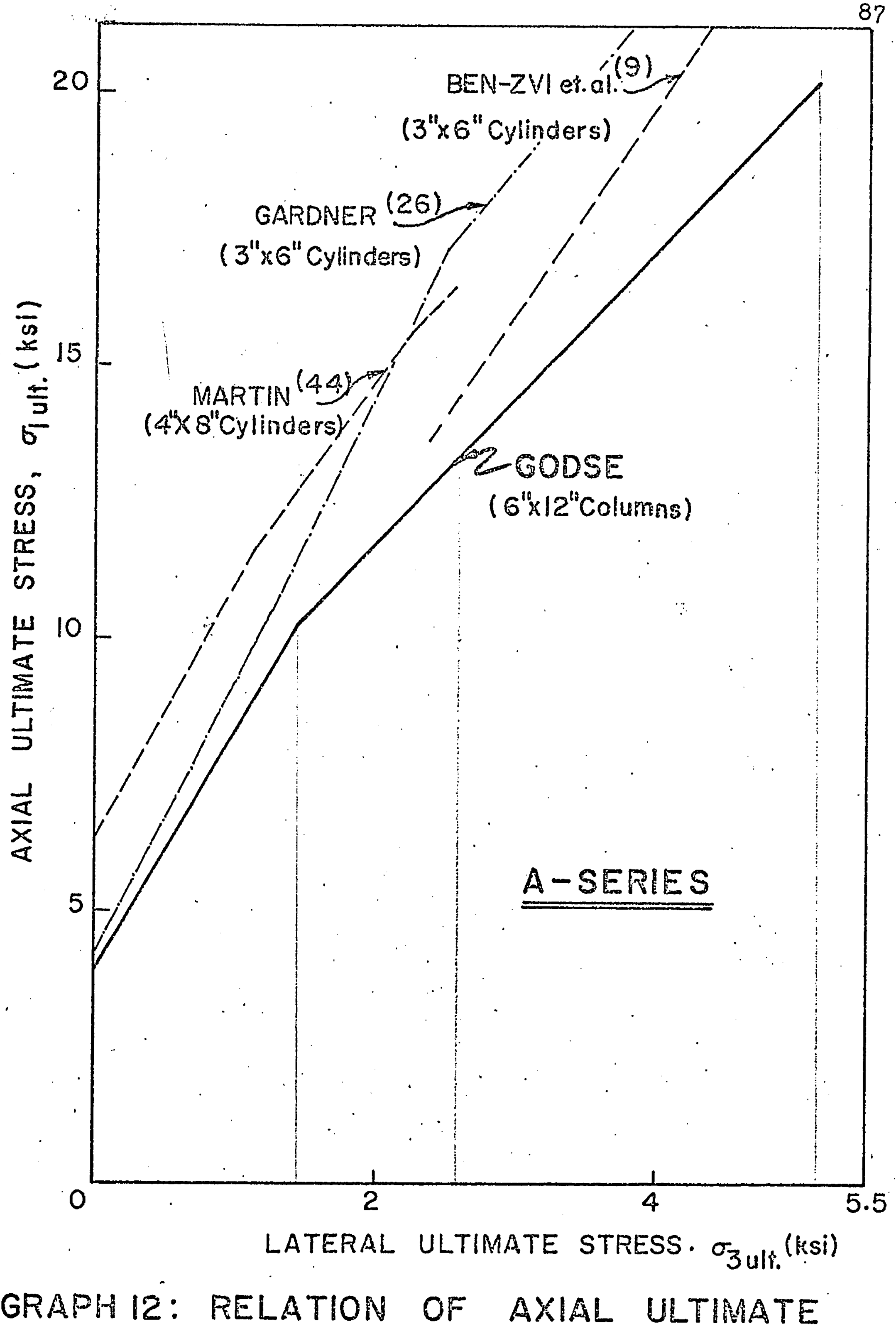
GRAPH 9 : LONGITUDINAL STRESS-STRAIN CHARACTERISTIC WITH INITIAL LATERAL PRESTRESS, $\sigma_{3ini.}$=1,706 p.s.i. (u/n)



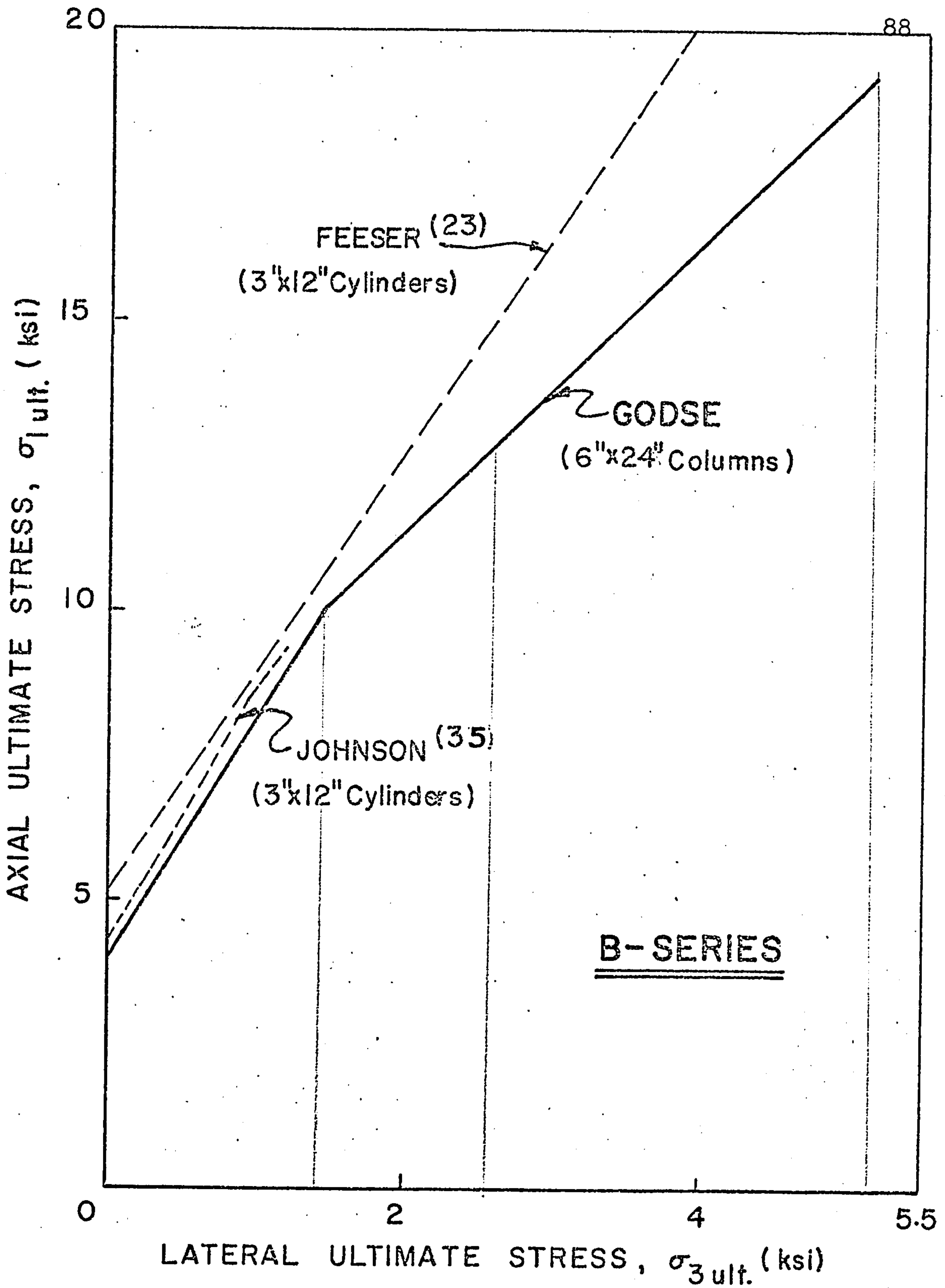
GRAPH 10: LONGITUDINAL STRESS–STRAIN CHARACTERISTIC WITH INITIAL LATERAL PRESTRESS, $\sigma_{3\,initial}$, = 3,412 p.s.i. (unless noted)



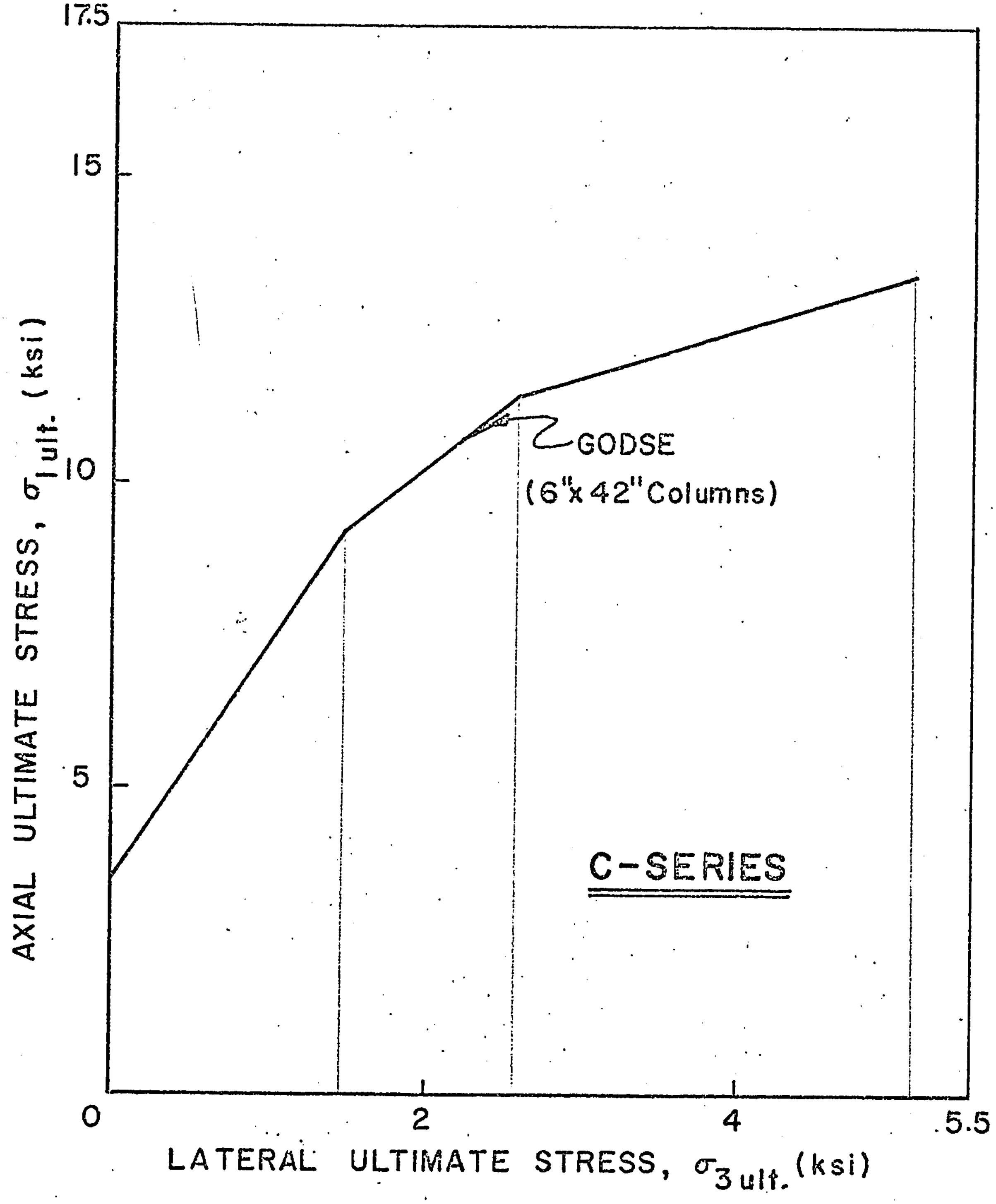
GRAPH II : RELATION OF ULTIMATE LOAD TO INITIAL LATERAL PRESTRESS



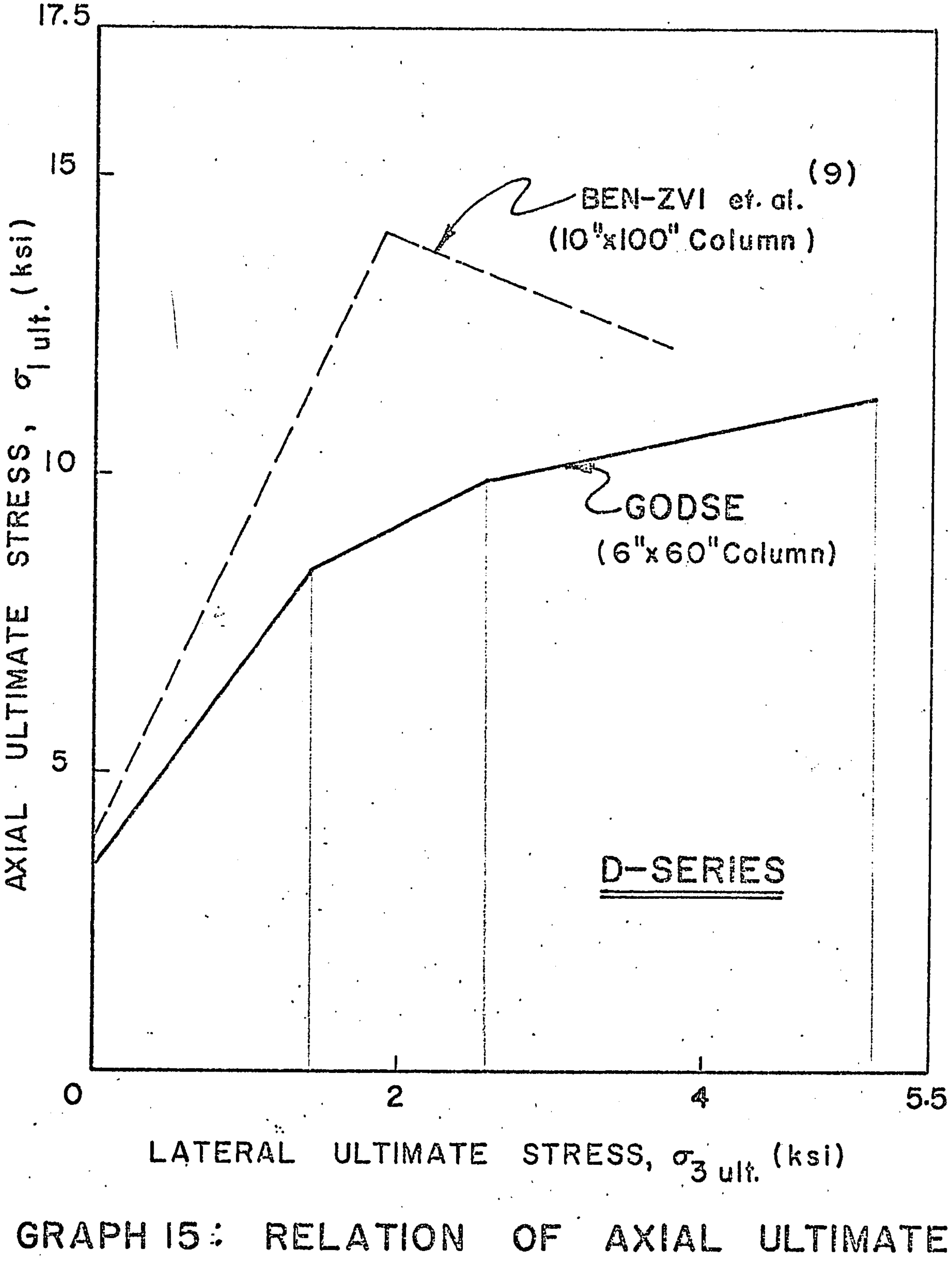
GRAPH 12: RELATION OF AXIAL ULTIMATE STRESS TO LATERAL ULT. STRESS



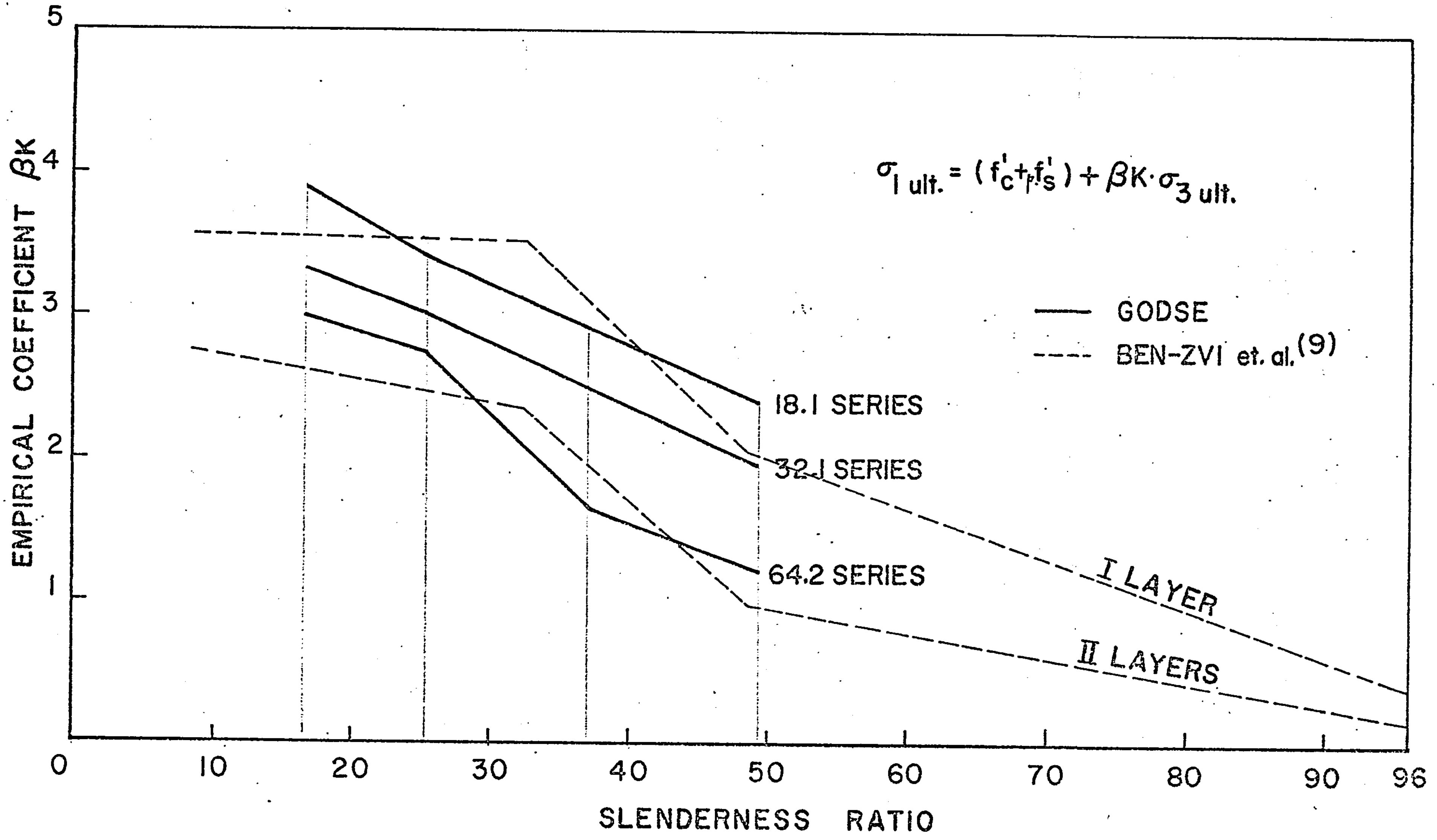
GRAPH 13: RELATION OF AXIAL ULTIMATE STRESS TO LATERAL ULT. STRESS



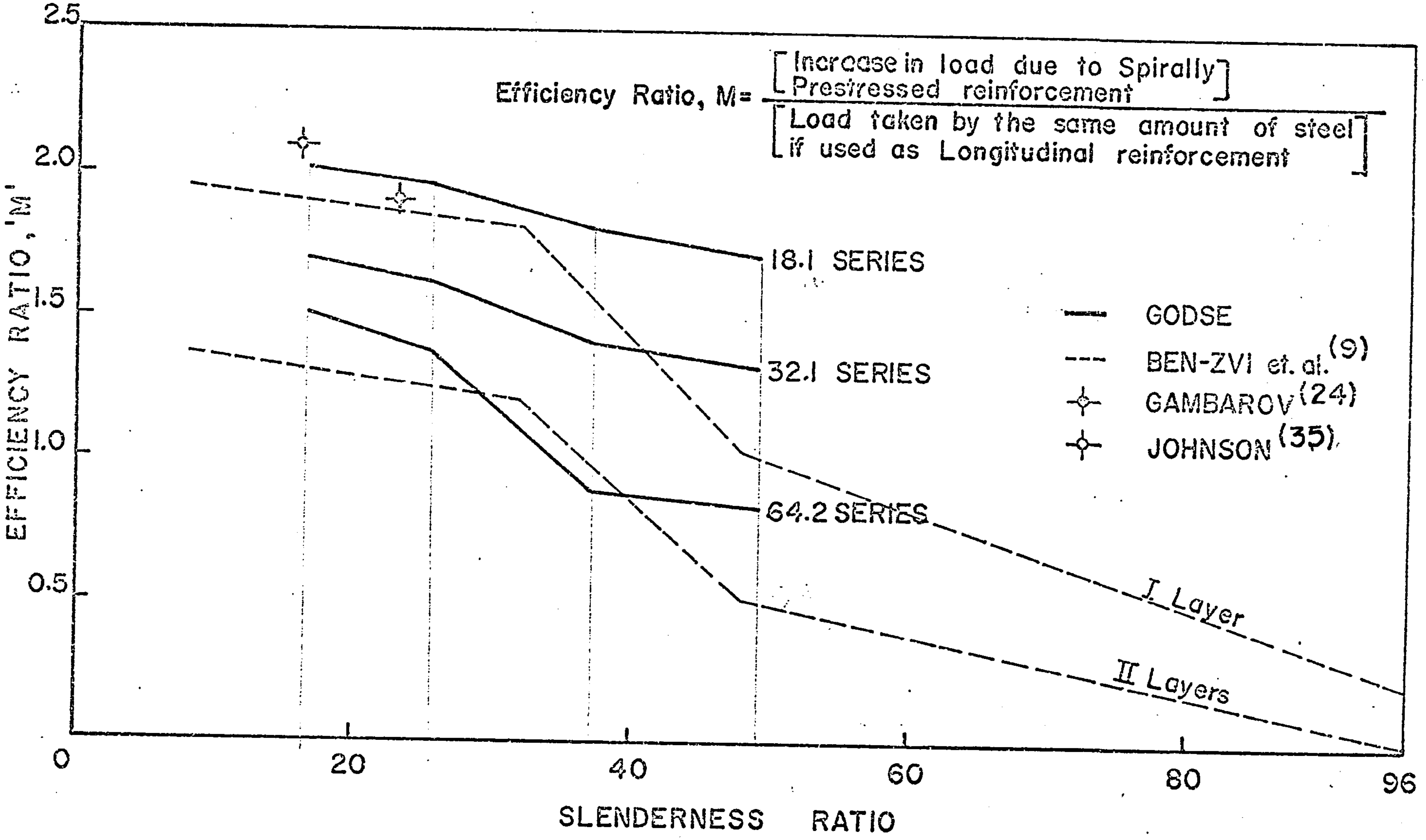
GRAPH 14: RELATION OF AXIAL ULTIMATE STRESS TO LATERAL ULTIMATE STRESS



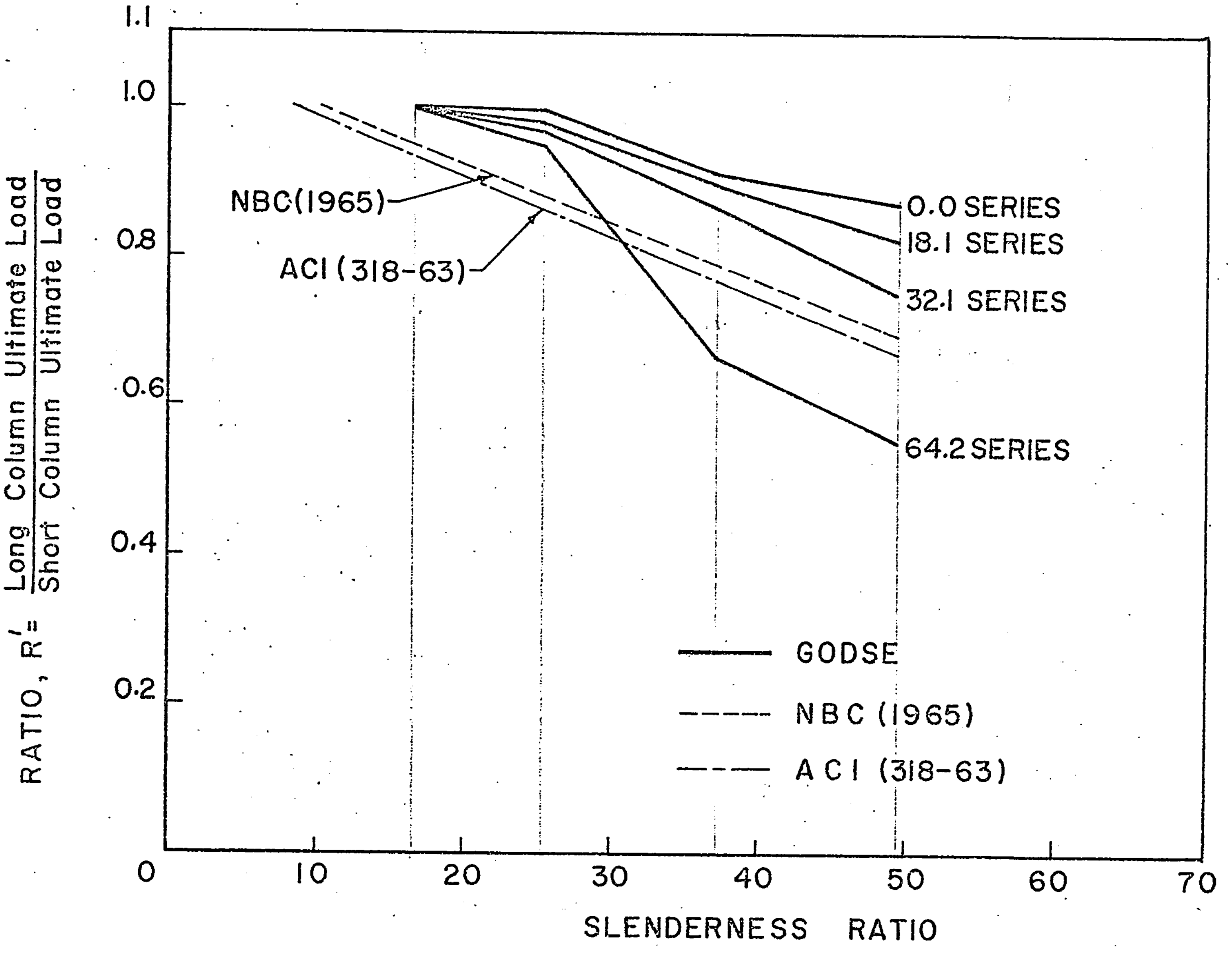
GRAPH 15: RELATION OF AXIAL ULTIMATE STRESS TO LATERAL ULTIMATE STRESS



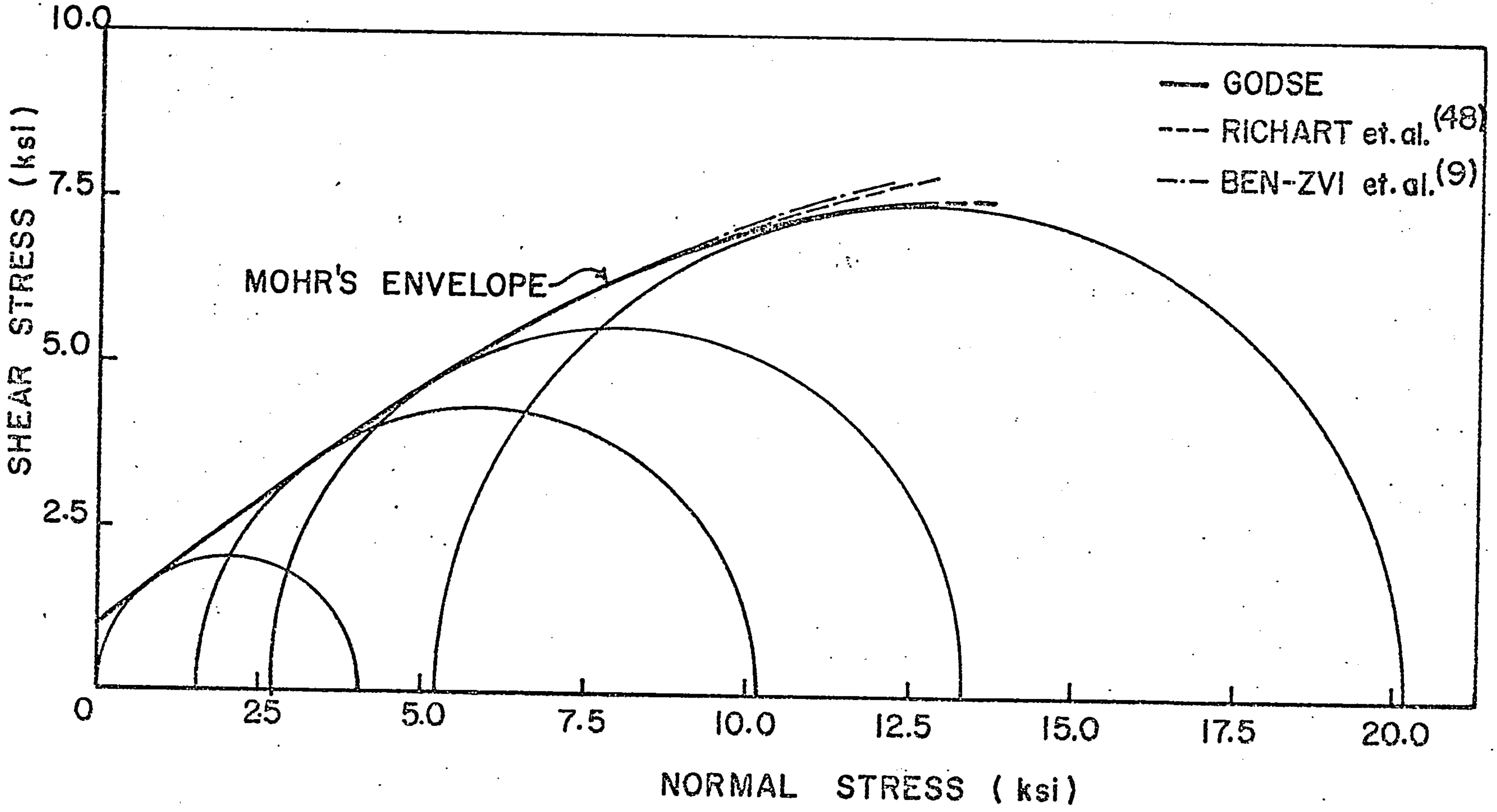
GRAPH 16: VARIATION OF EMPIRICAL COEFFICIENT βK WITH SLENDERNESS RATIO



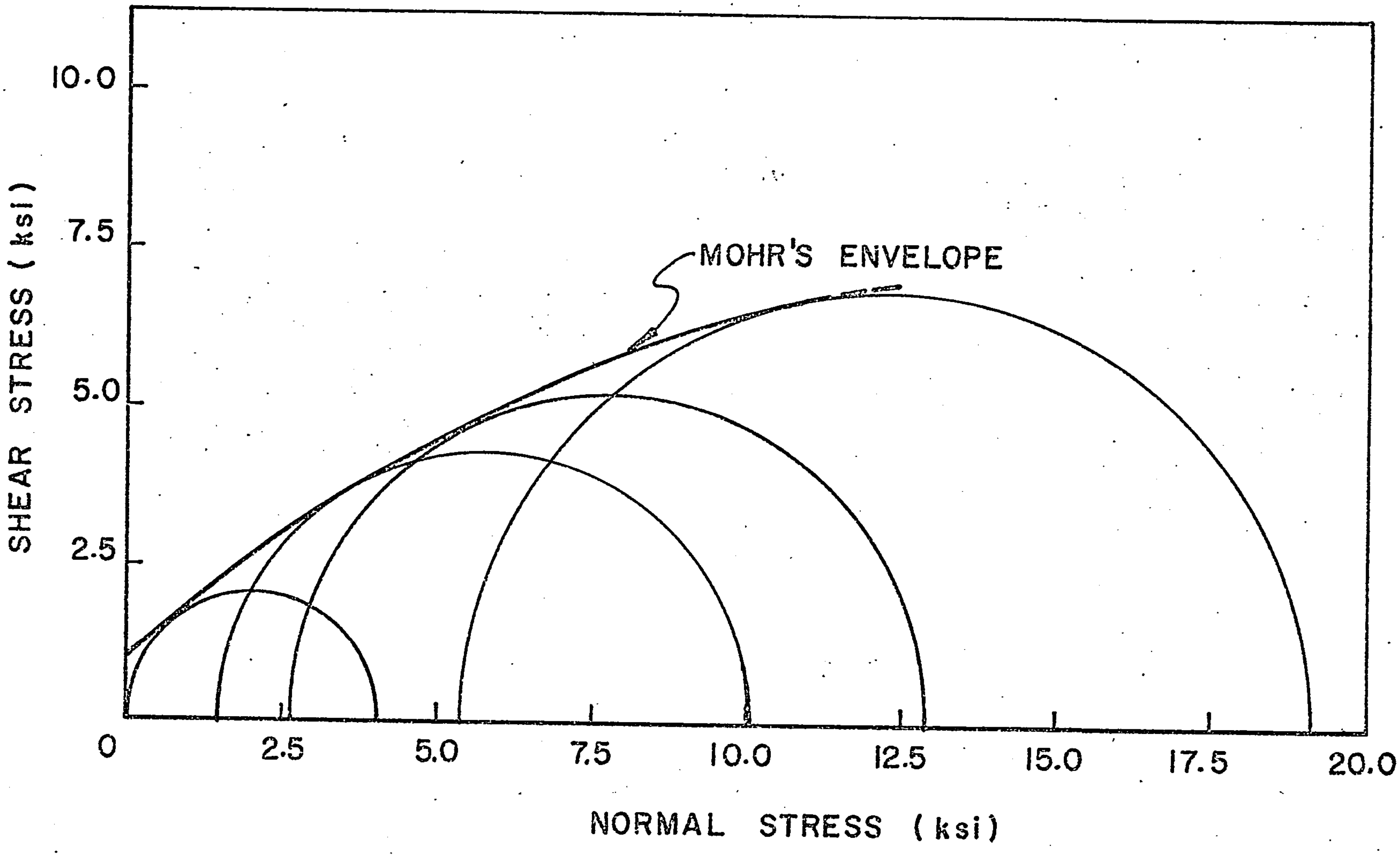
GRAPH 17: VARIATION OF EFFICIENCY RATIO, M, WITH SLENDERNESS RATIO



GRAPH 18: VARIATION OF RATIO OF LONG COLUMN LOAD TO SHORT COLUMN LOAD WITH SLENDERNESS RATIO

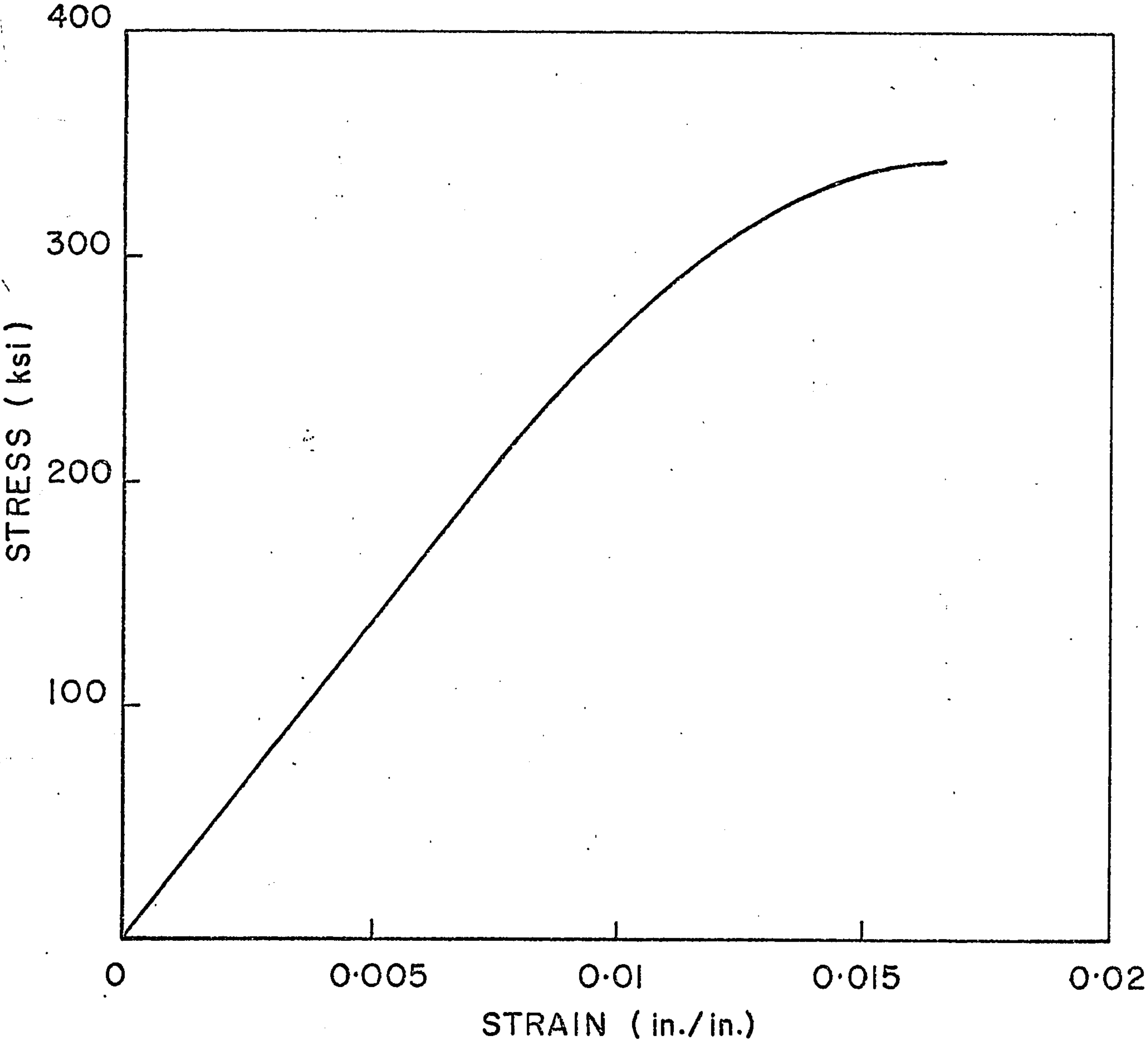


GRAPH 19: MOHR'S CIRCLES FOR SPECIMENS OF A-SERIES

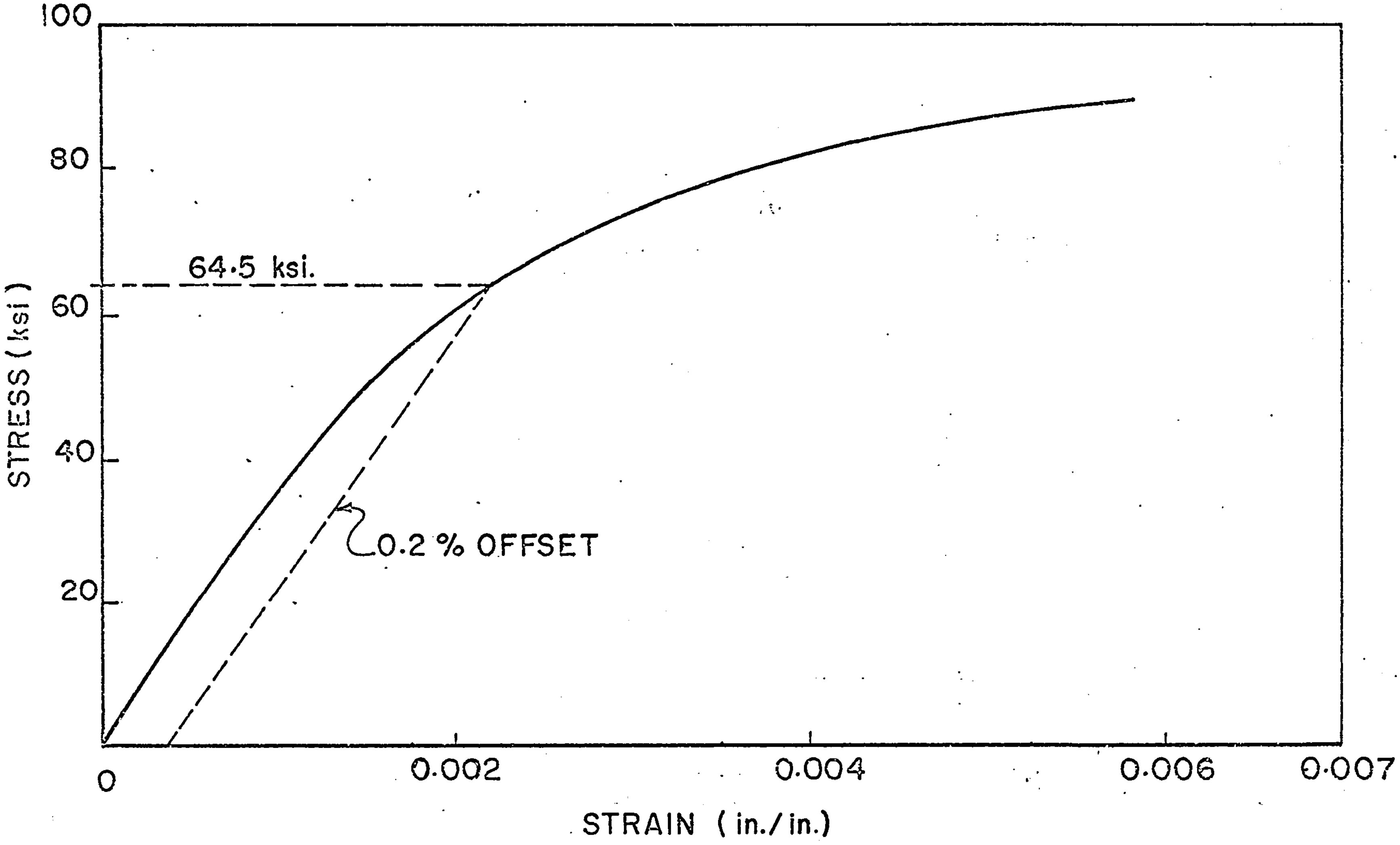


GRAPH 20: MOHR'S CIRCLES FOR SPECIMENS OF B-SERIES

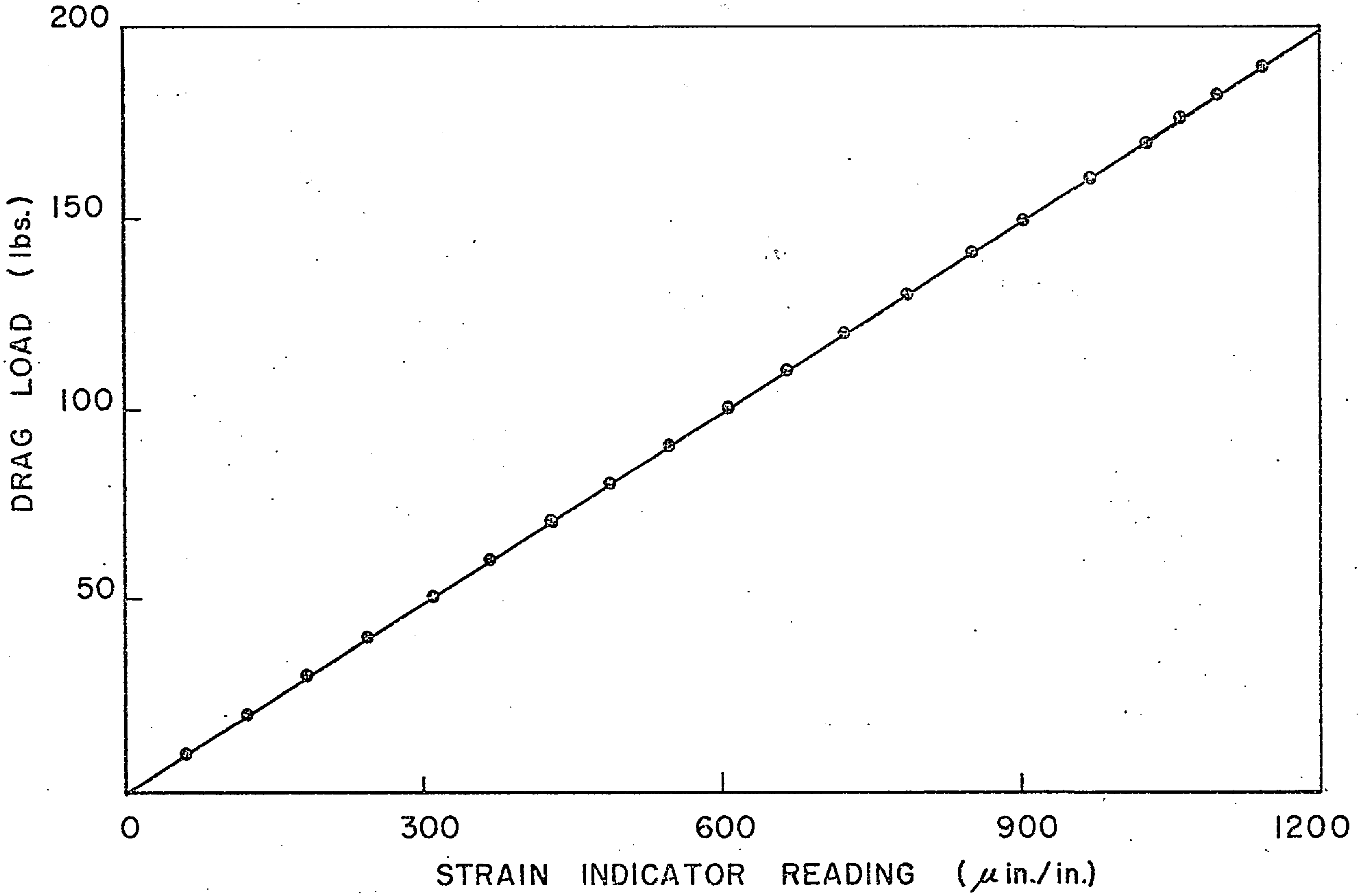
APPENDIX - A



GRAPH A.1: STRESS-STRAIN CHARACTERISTIC OF 0·031" DIA. STEEL WIRE



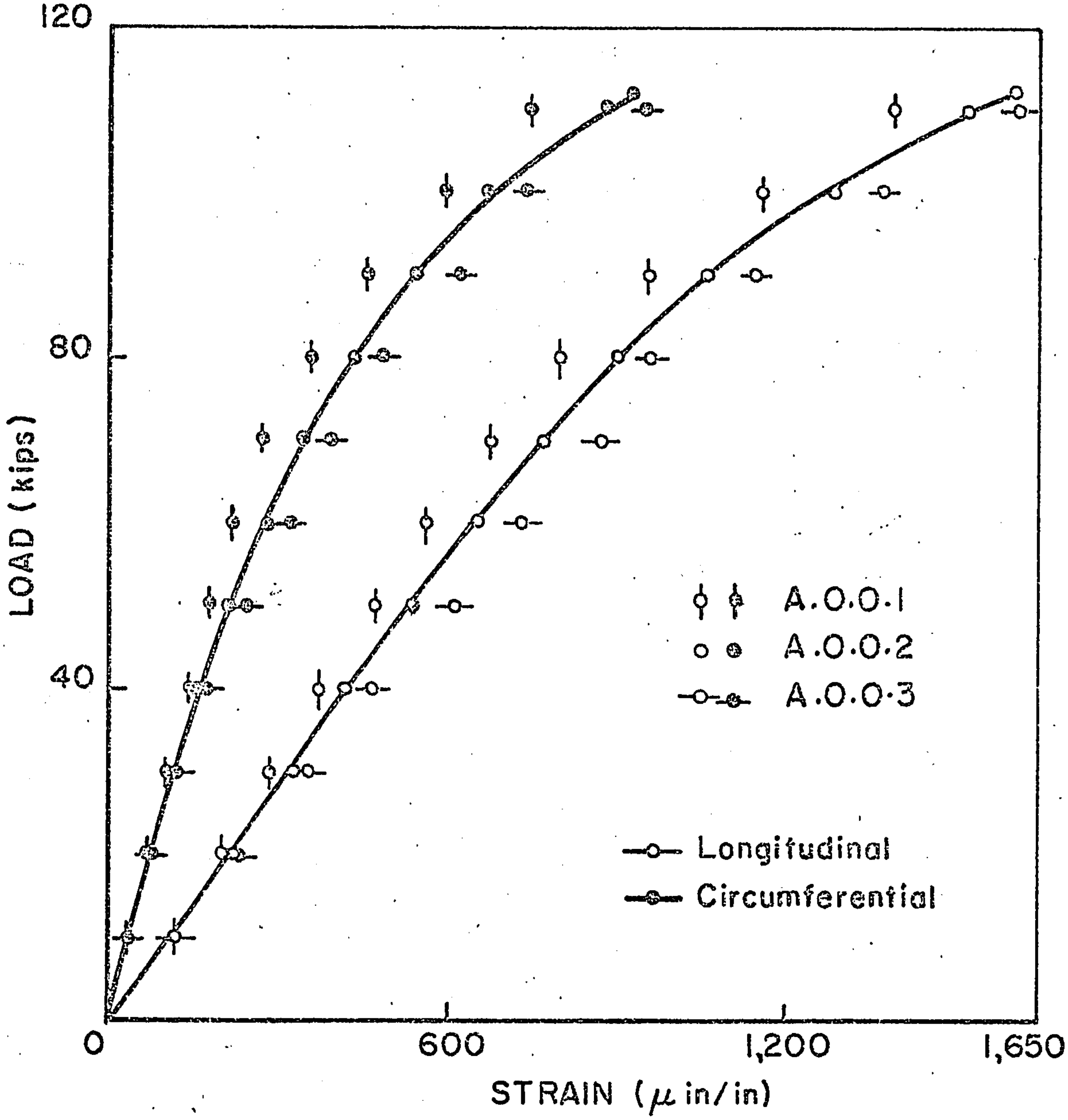
GRAPH A.2: STRESS-STRAIN CHARACTERISTIC OF LONGITUDINAL REINFORCEMENT (# 3 BAR)



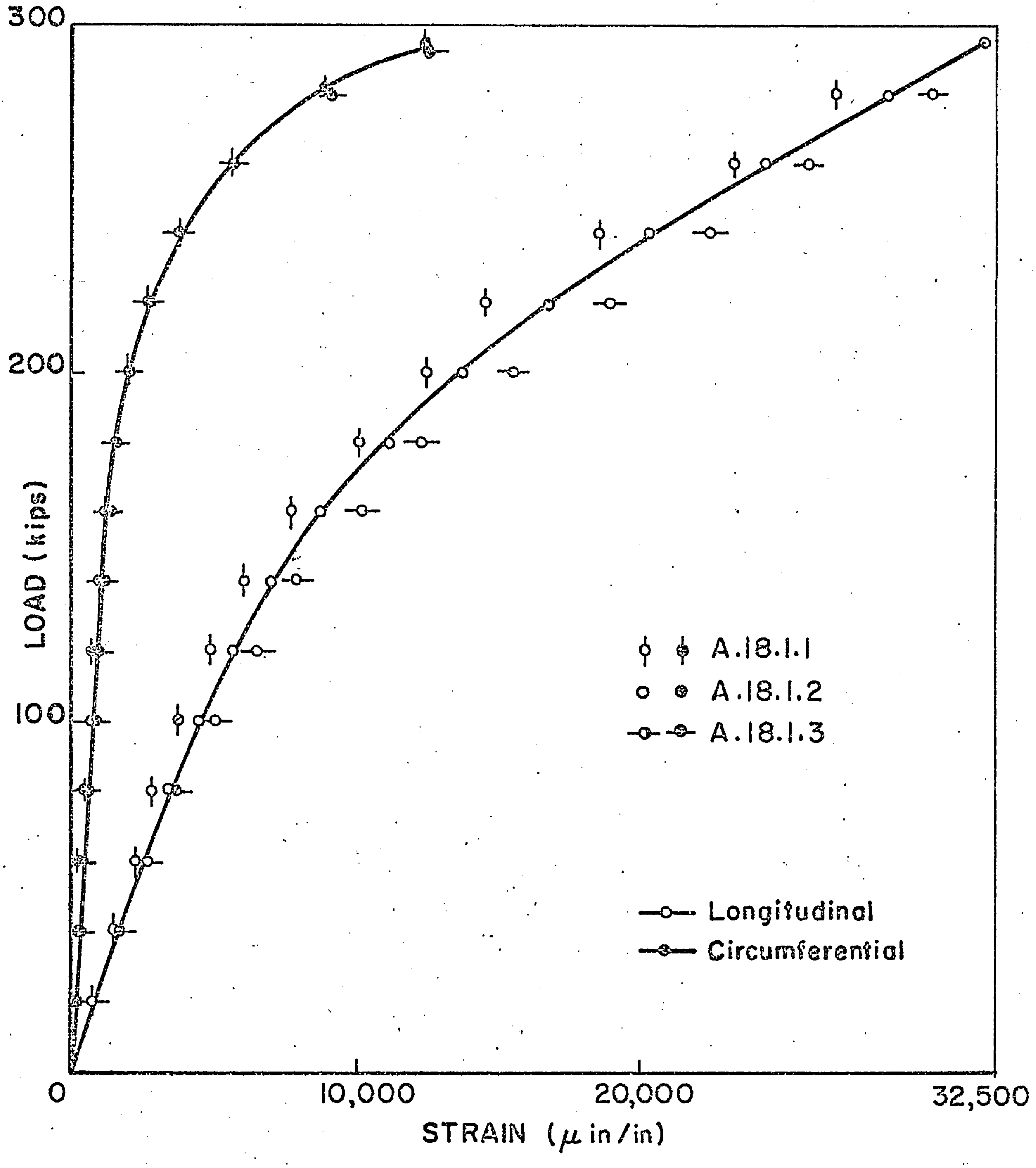
GRAPH A.3: CALIBRATION CURVE FOR TENSIONING DEVICE

APPENDIX — B

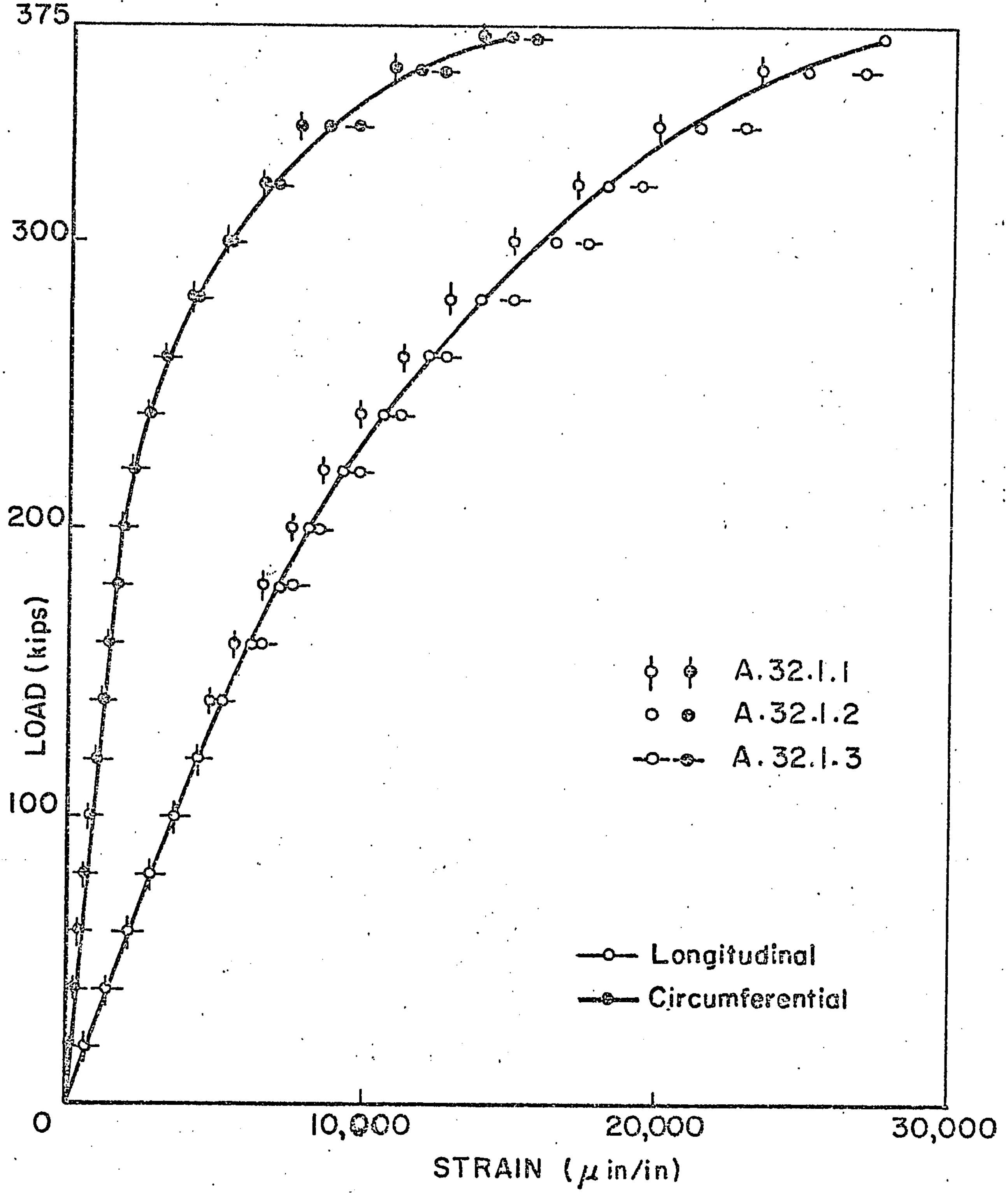
TEST RESULTS OF
6" DIAMETER, SPIRALLY PRESTRESSED
CONCRETE SPECIMENTS
A -SERIES (1'-0" LONG)



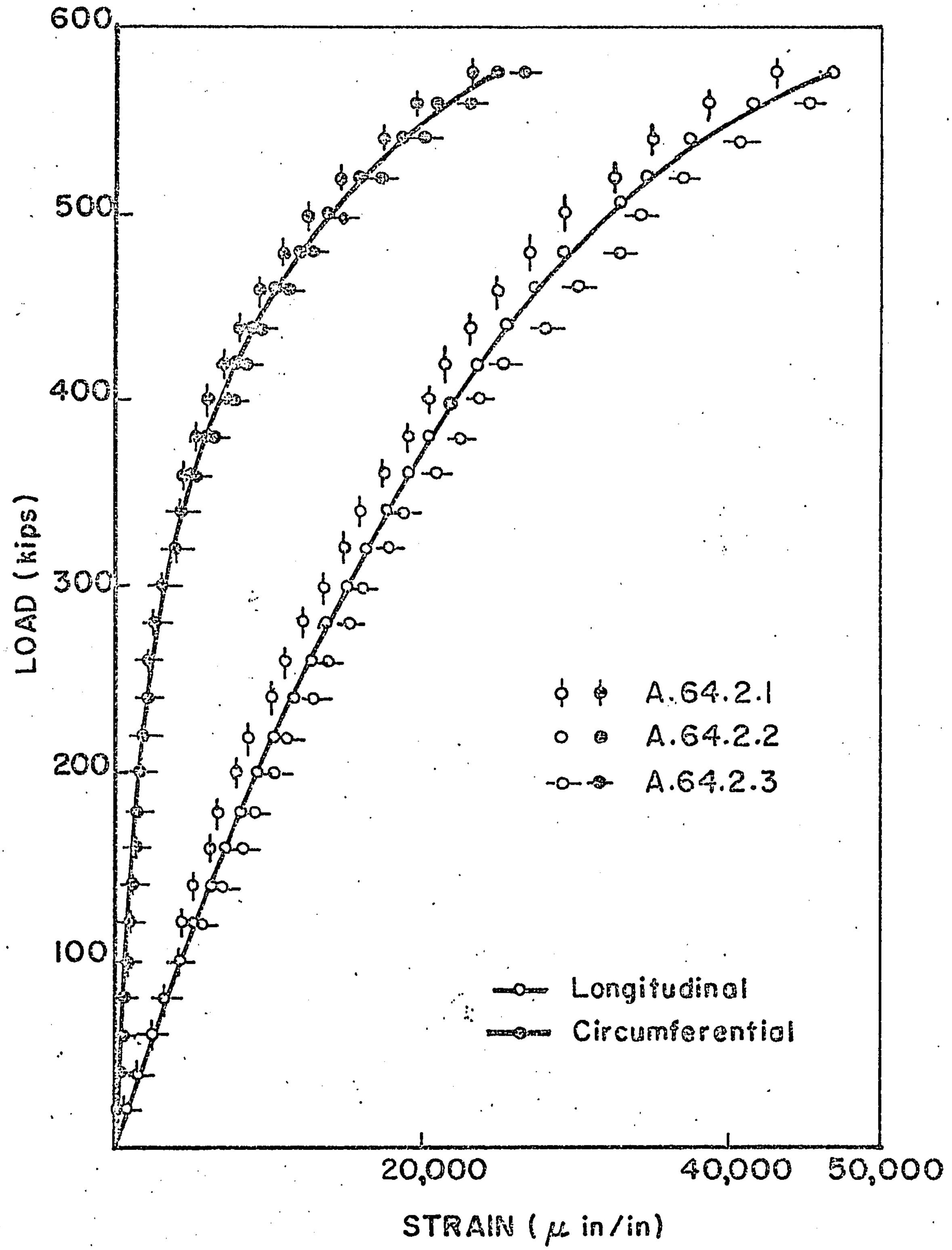
GRAPH B·I: LOAD-DEFORMATION OF SPECIMEN A-O-O SERIES



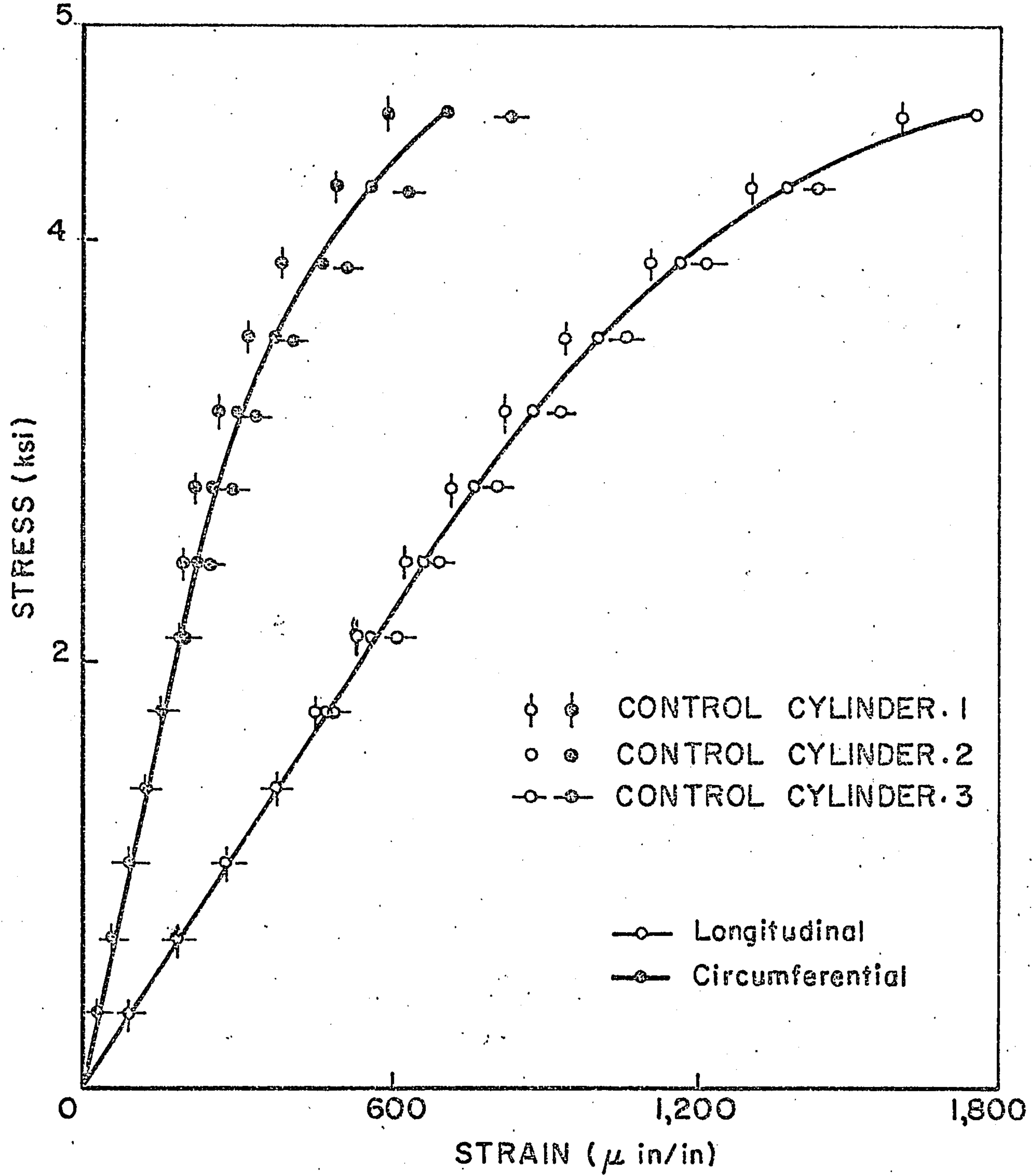
GRAPH B.2: LOAD-DEFORMATION OF SPECIMEN A-18-1 SERIES



GRAPH B.3: LOAD-DEFORMATION OF SPECIMEN A-32-1 SERIES



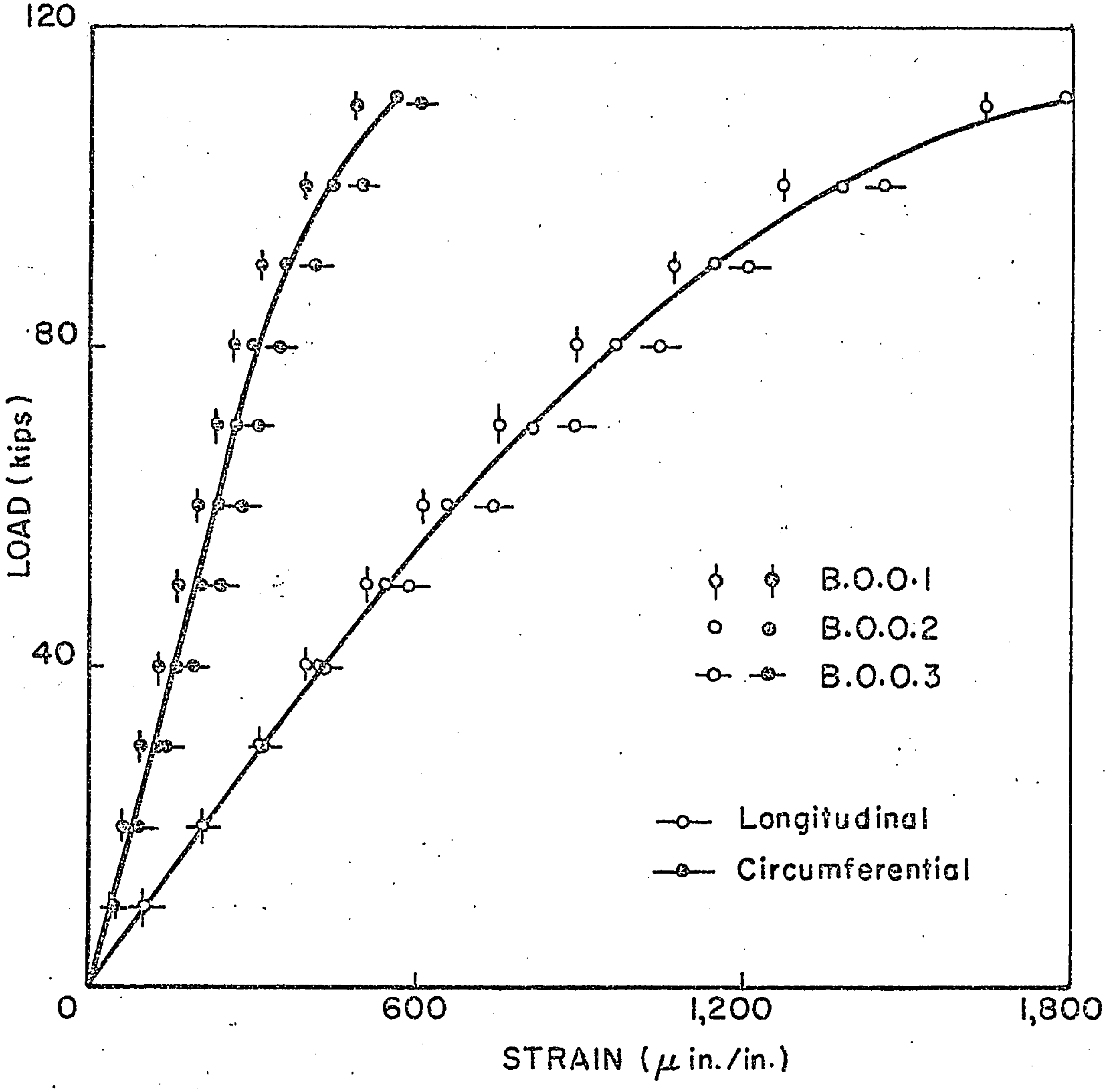
GRAPH B.4:LOAD-DEFORMATION OF SPECIMEN A-64-2 SERIES



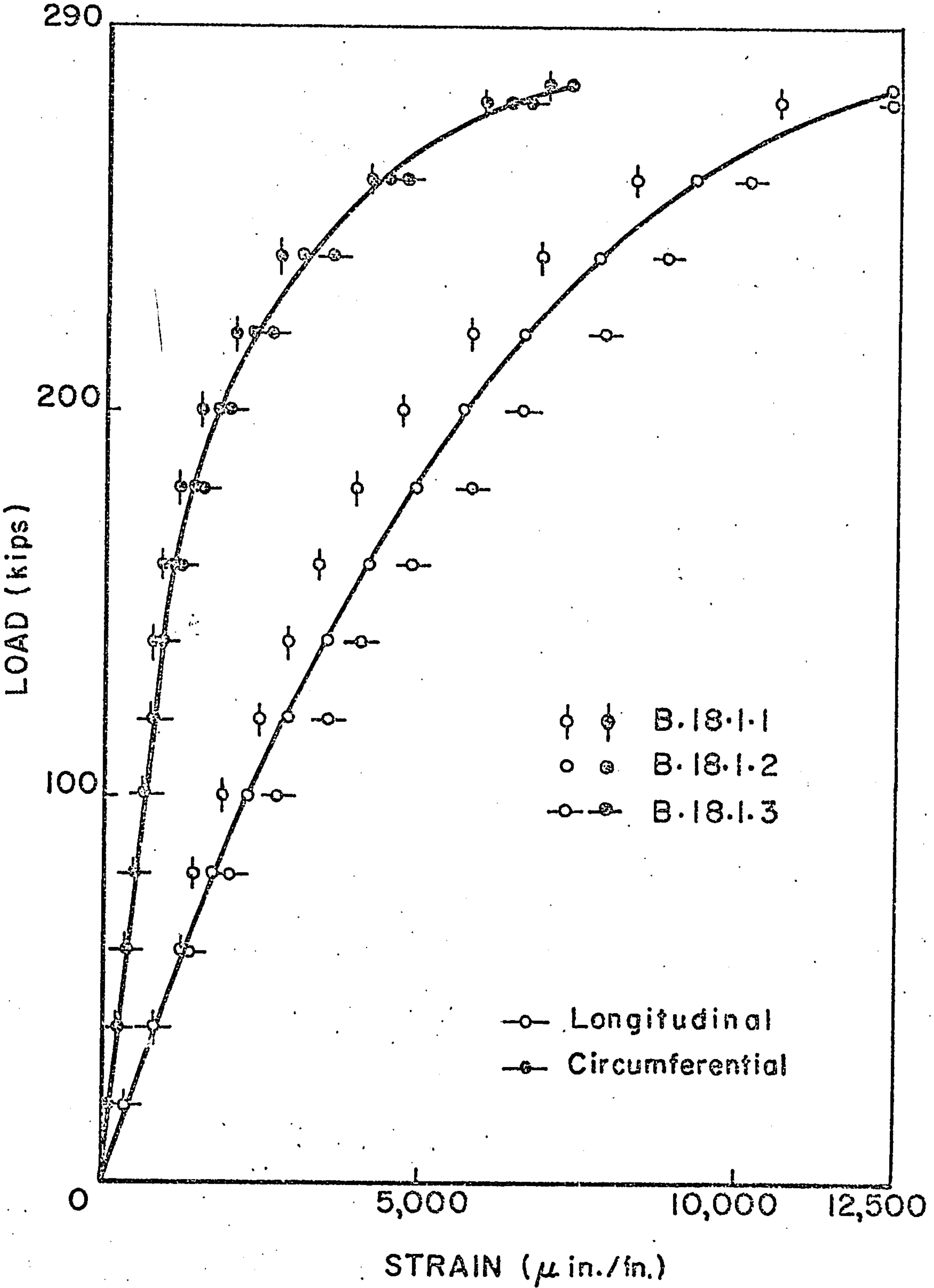
GRAPH B.5: STRESS-STRAIN CHARACTERISTIC OF CONTROL CONCRETE

(FOR SPECIMENS OF A-SERIES)

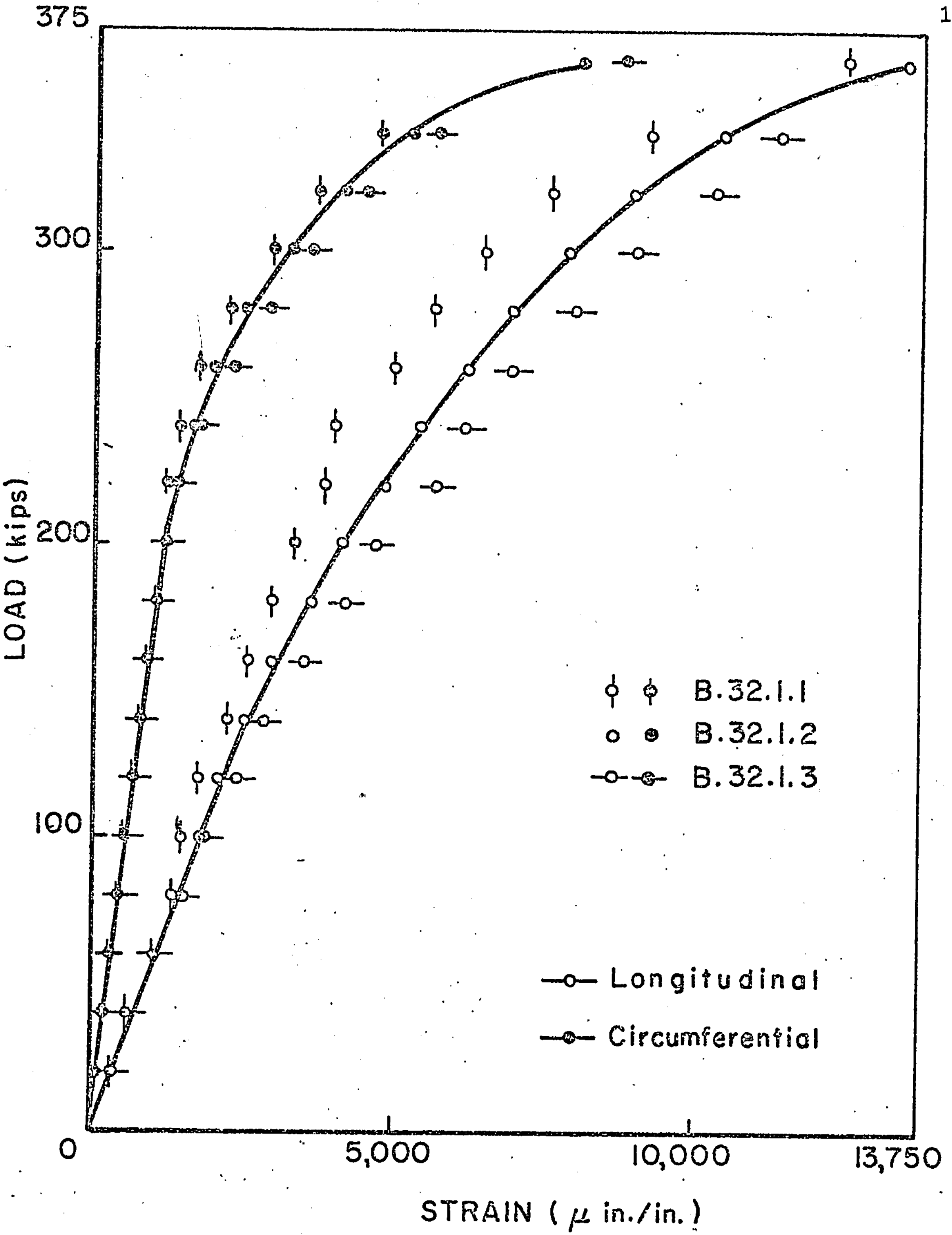
TEST RESULTS OF
6" DIAMETER, SPIRALLY PRESTRESSED
CONCRETE SPECIMENS
B -SERIES (2'-0" LONG)



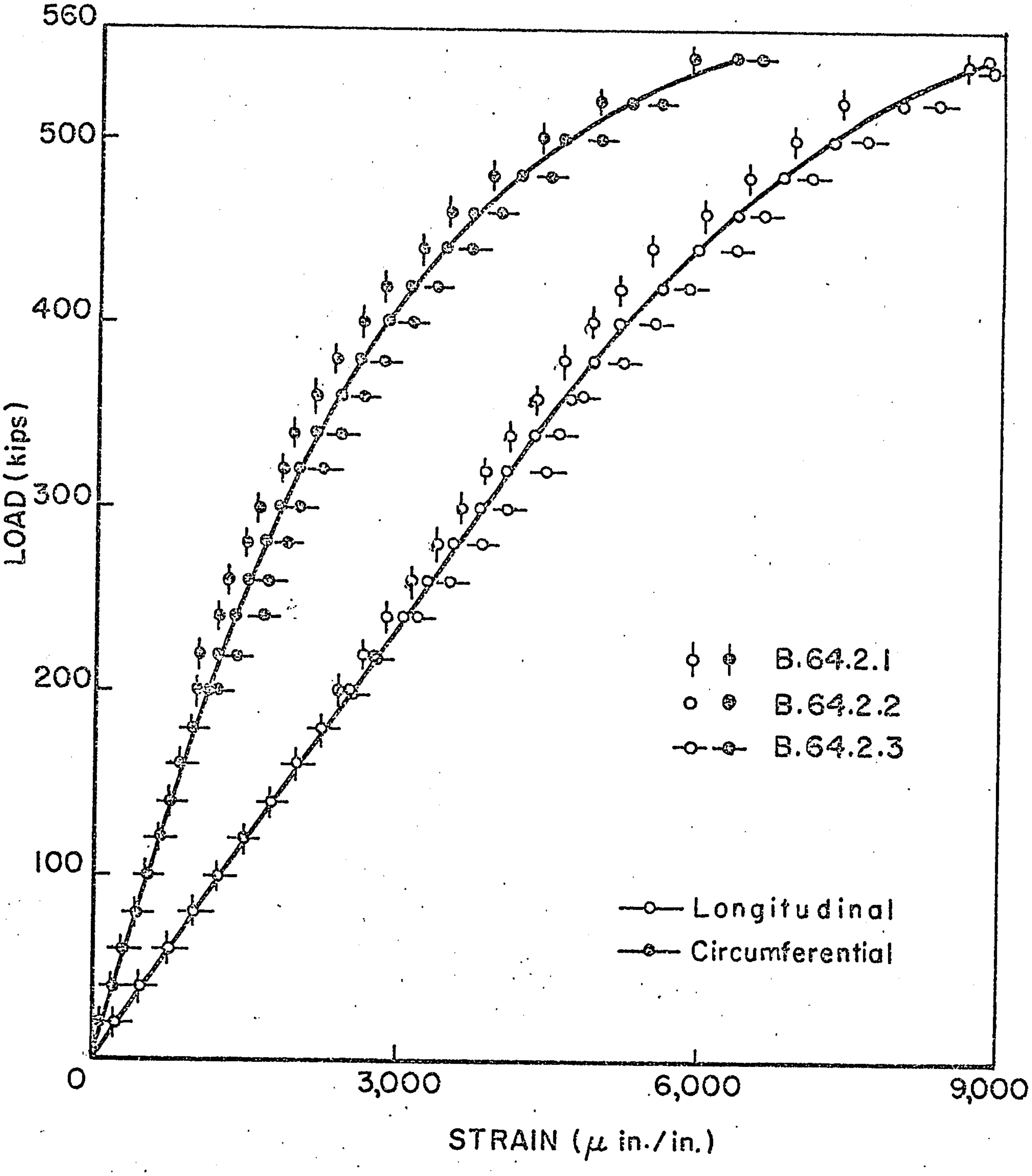
GRAPH B.6: LOAD-DEFORMATION OF SPECIMEN B-O-O SERIES



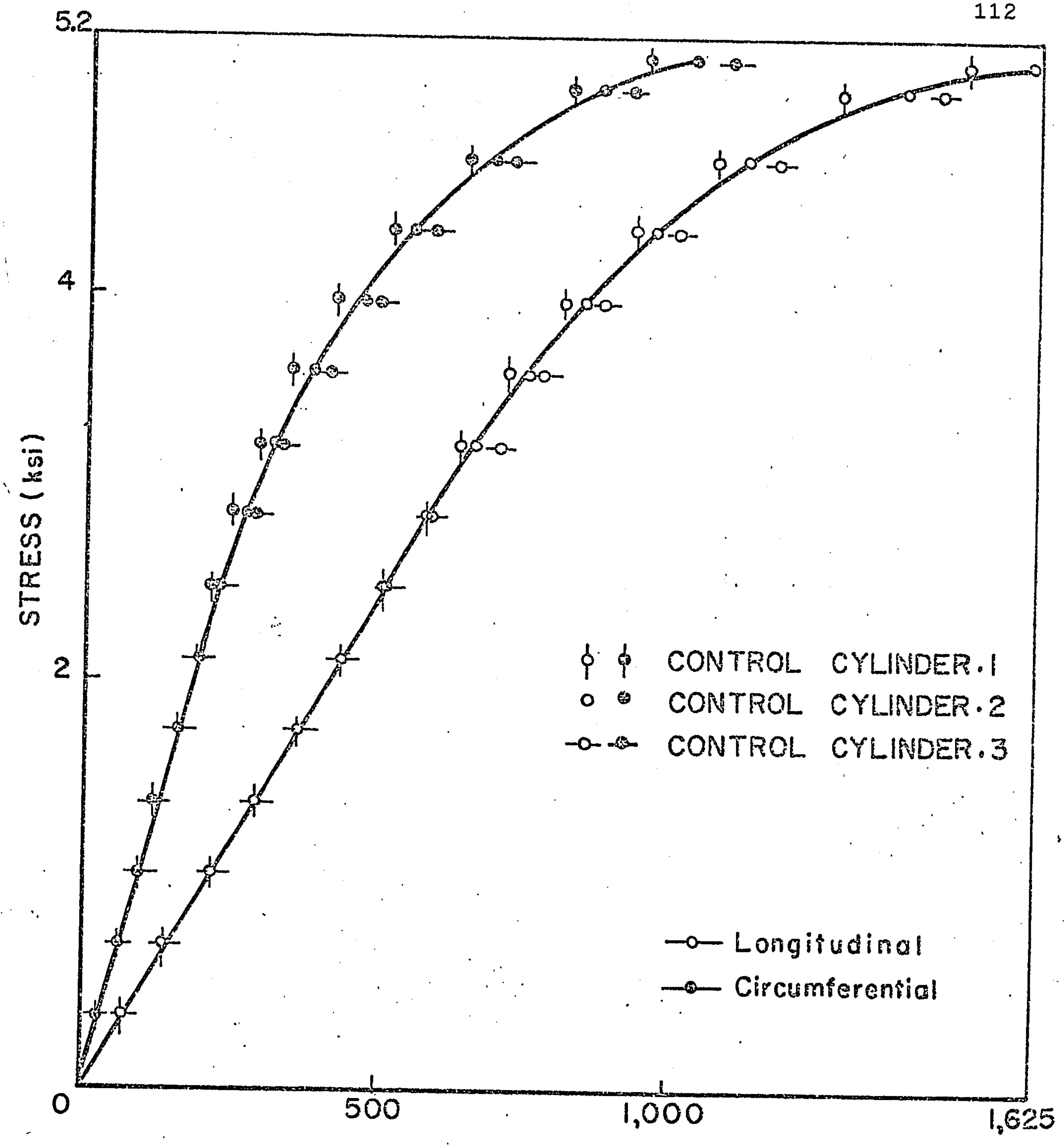
GRAPH B.7: LOAD-DEFORMATION OF SPECIMEN B-18-1 SERIES



GRAPH B.8: LOAD-DEFORMATION OF SPECIMEN B-32-1 SERIES



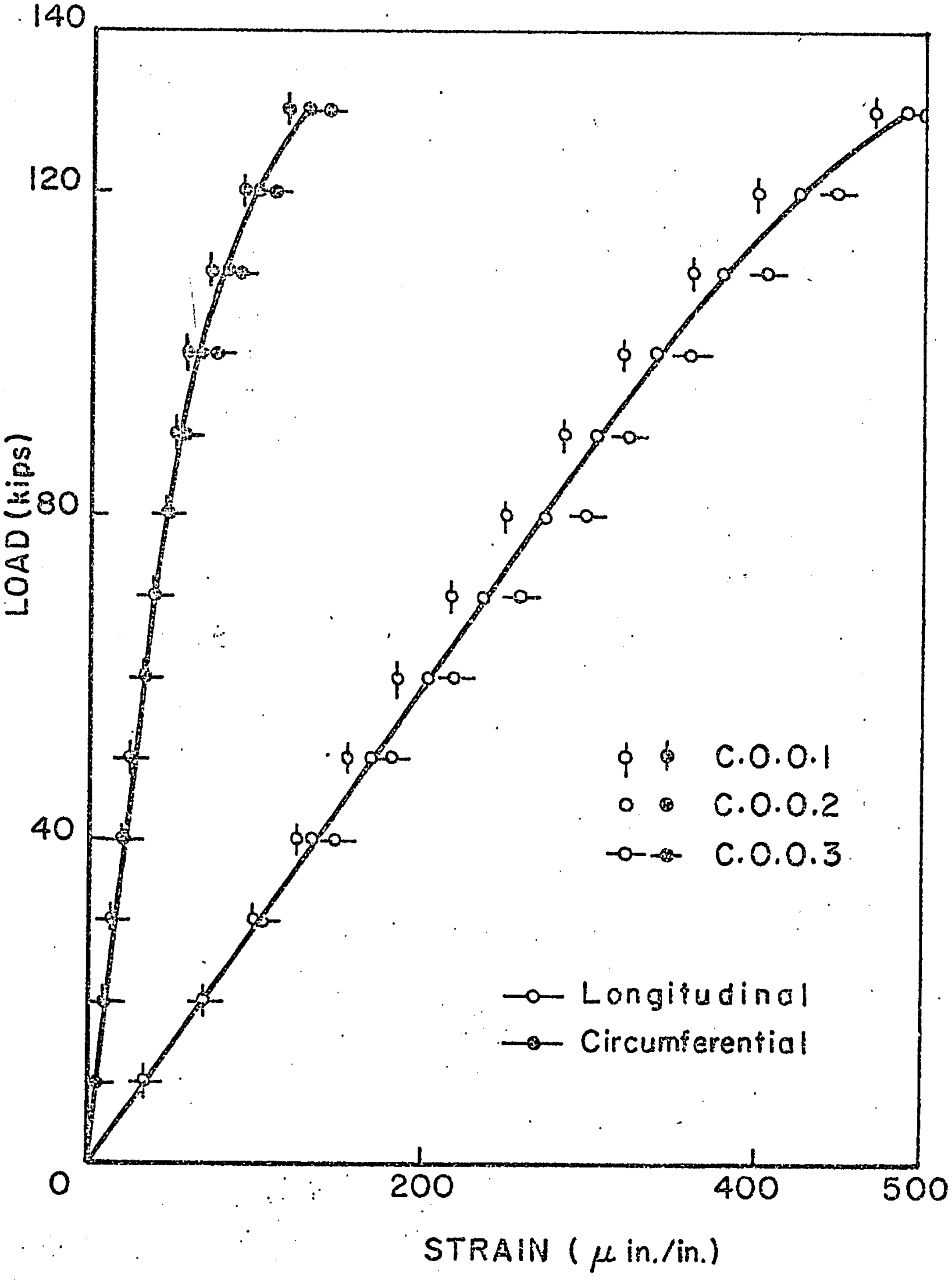
GRAPH B.9: LOAD-DEFORMATION OF SPECIMEN B-64-2 SERIES



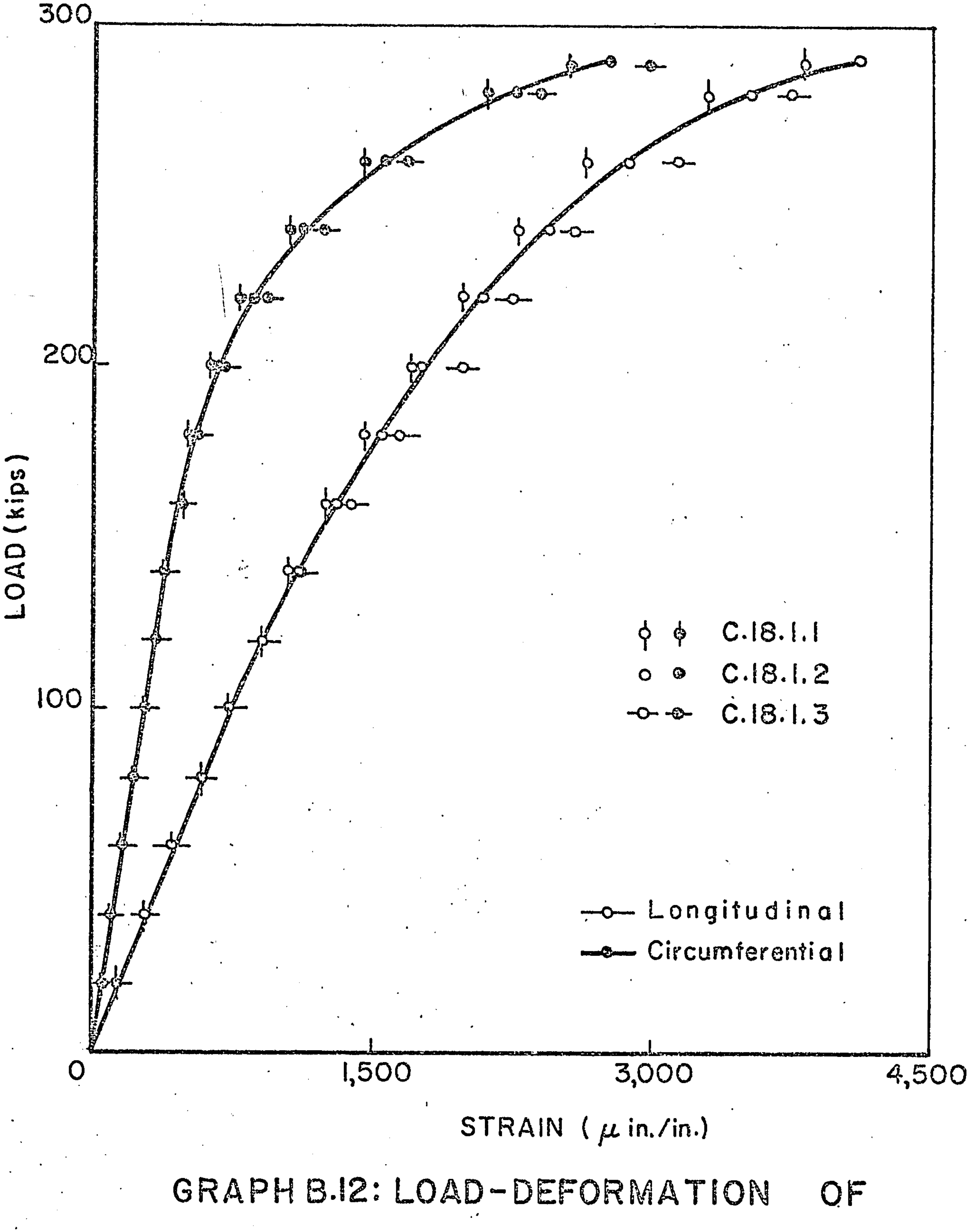
GRAPH B·10: STRESS-STRAIN CHARACTERISTIC OF CONTROL CONCRETE

(FOR SPECIMENS OF B-SERIES)

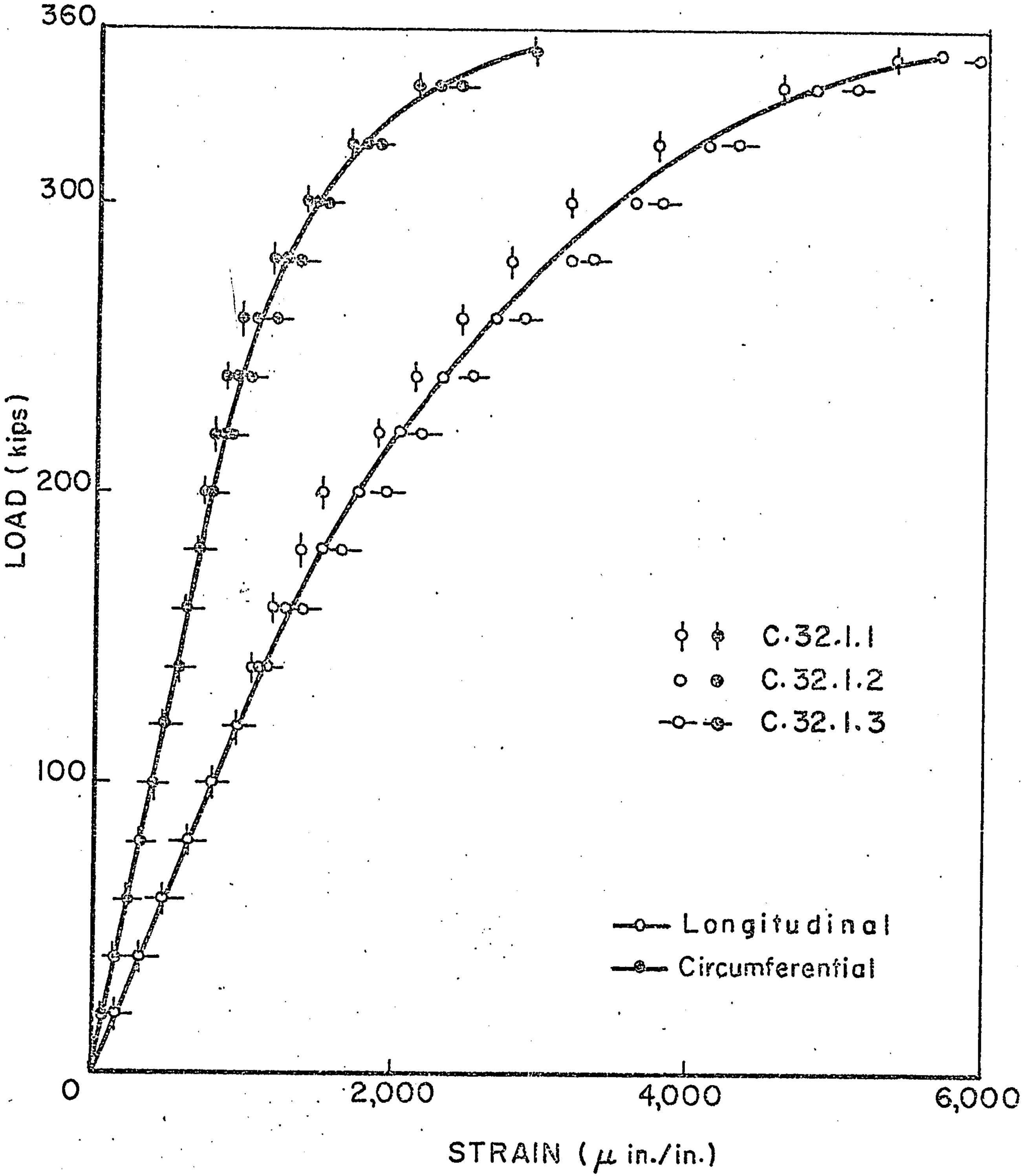
TEST RESULTS OF
6" DIAMETER, SPIRALLY PRESTRESSED
CONCRETE SPECIMENS
C -SERIES (3'-6" LONG)



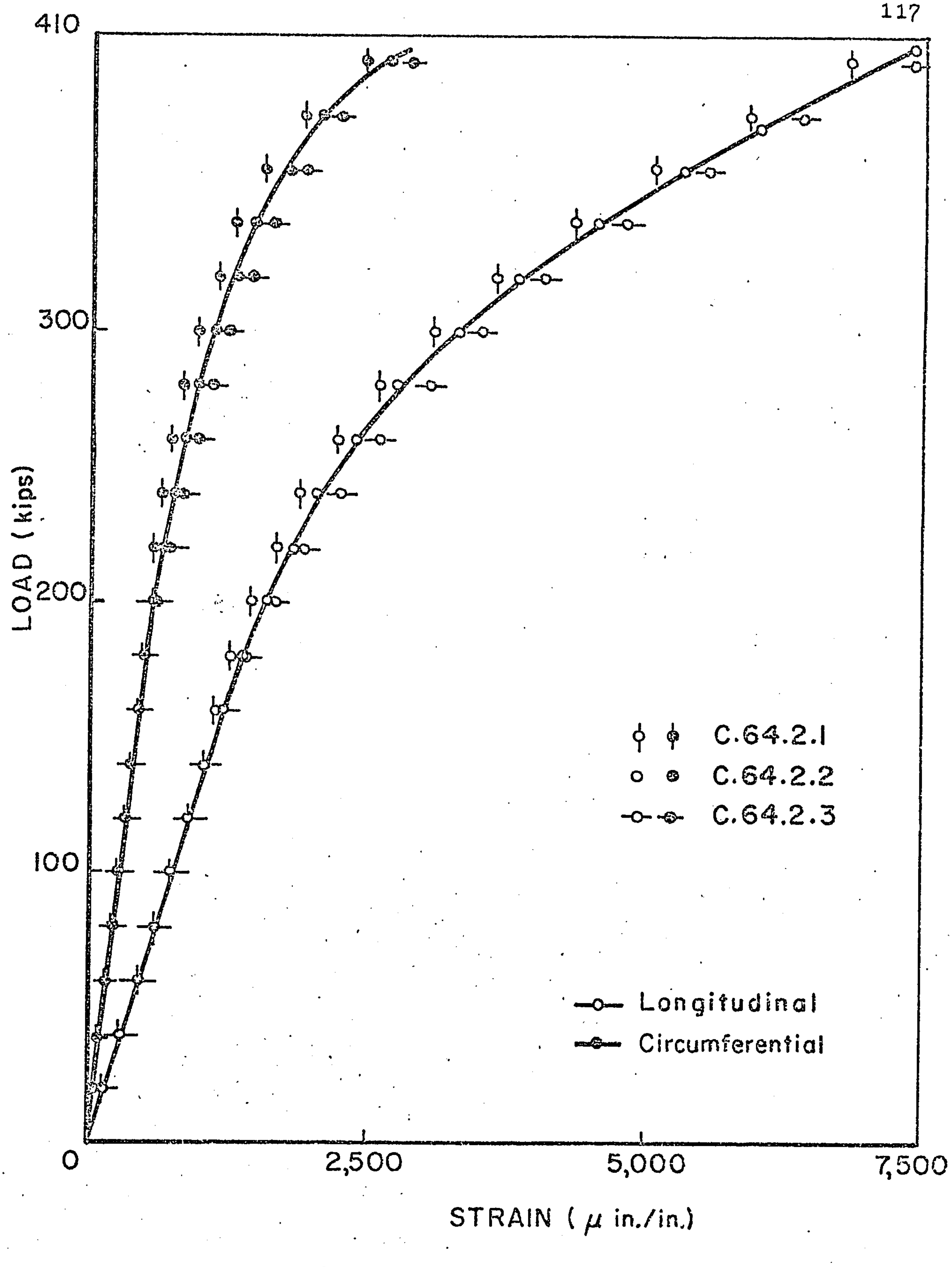
GRAPH B.II: LOAD-DEFORMATION OF SPECIMEN C-O-O SERIES



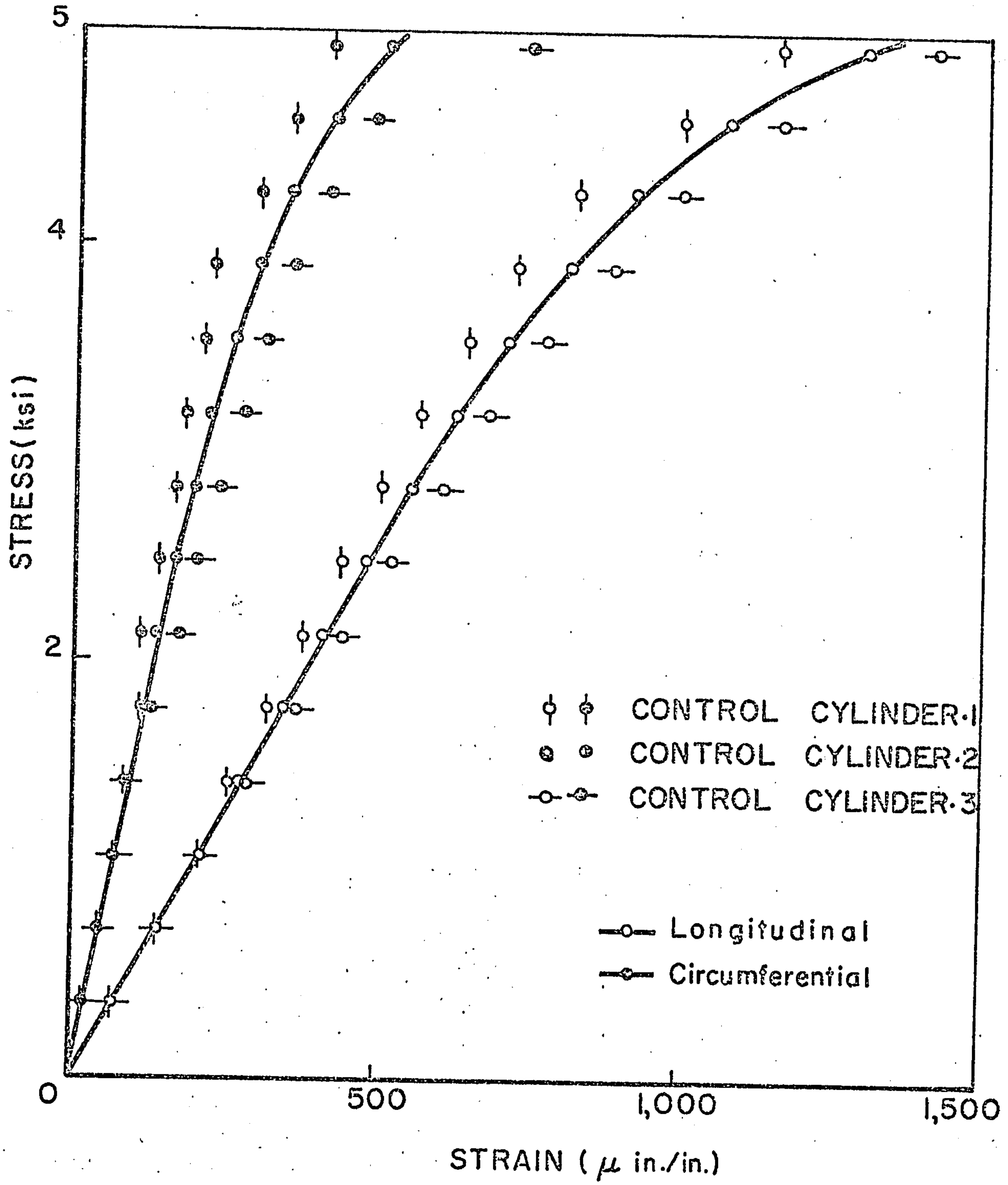
GRAPH B.12: LOAD-DEFORMATION OF SPECIMEN C-18-1 SERIES



GRAPH B.13: LOAD-DEFORMATION OF SPECIMEN C-32-1 SERIES



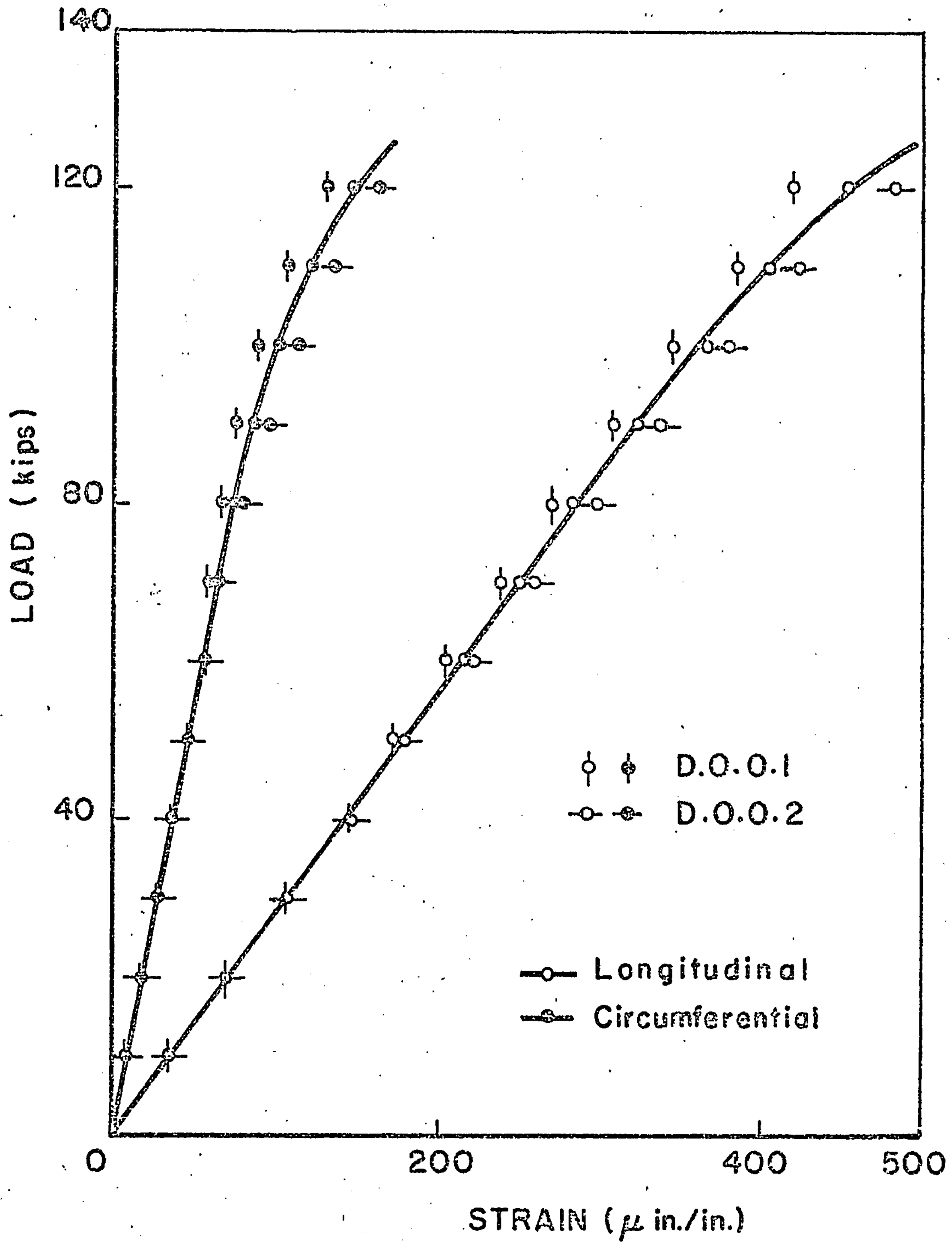
GRAPH B.14: LOAD-DEFORMATION OF SPECIMEN C-64-2 SERIES



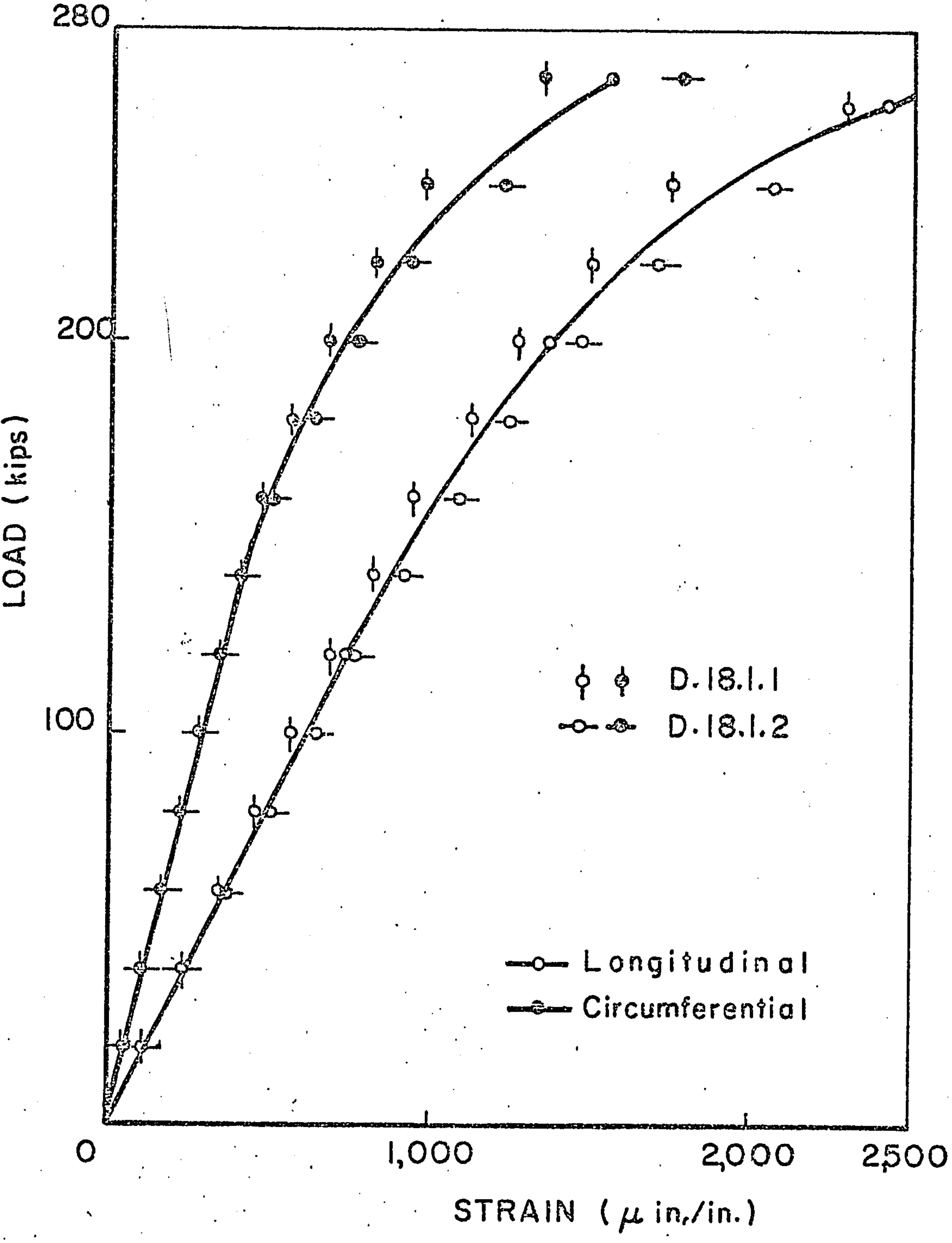
GRAPH B.15: STRESS–STRAIN CHARACTERISTIC OF CONTROL CONCRETE

(FOR SPECIMENS OF C–SERIES)

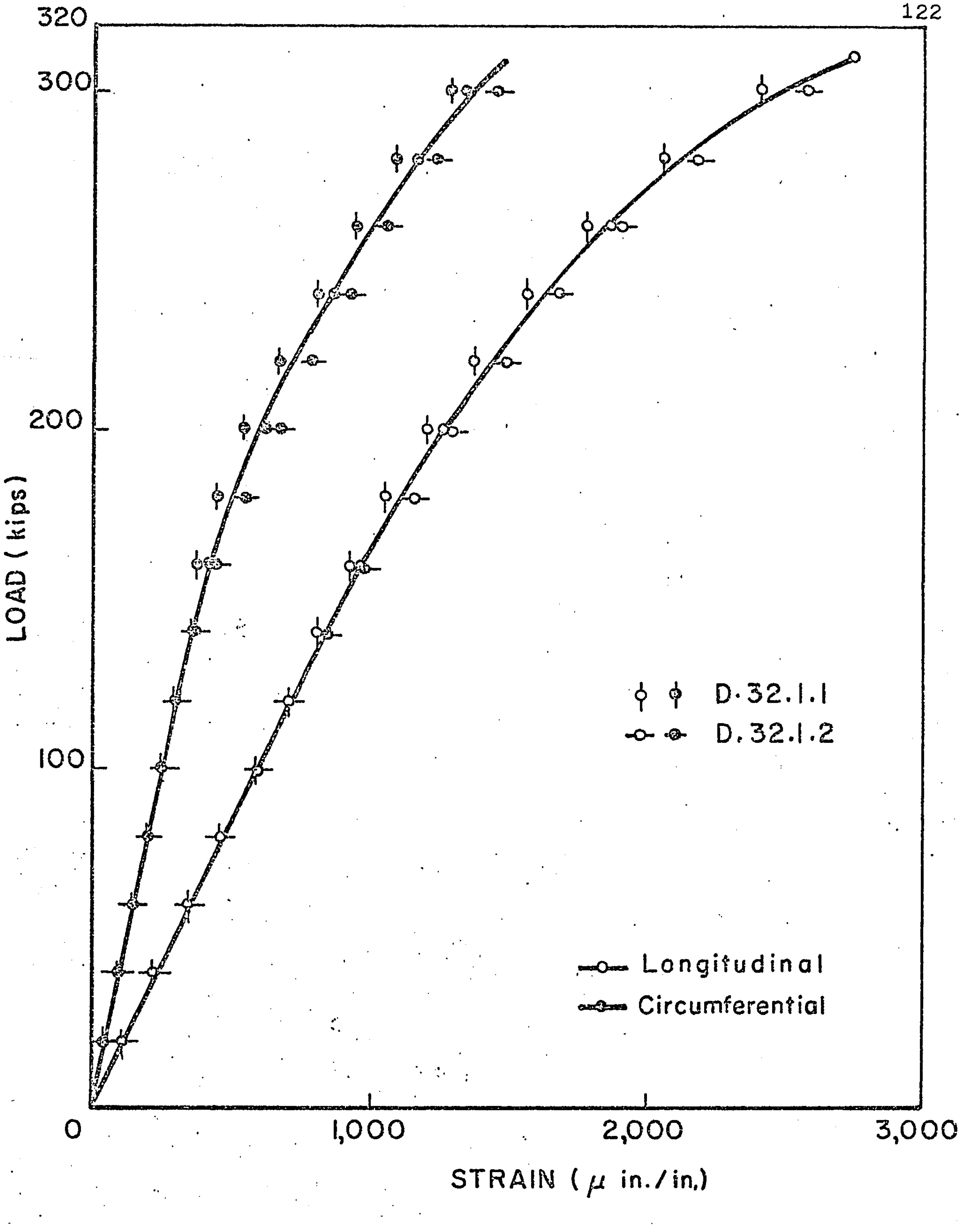
TEST RESULTS OF
6" DIAMETER, SPIRALLY PRESTRESSED
CONCRETE SPECIMENS
D -SERIES (5'-0" LONG)



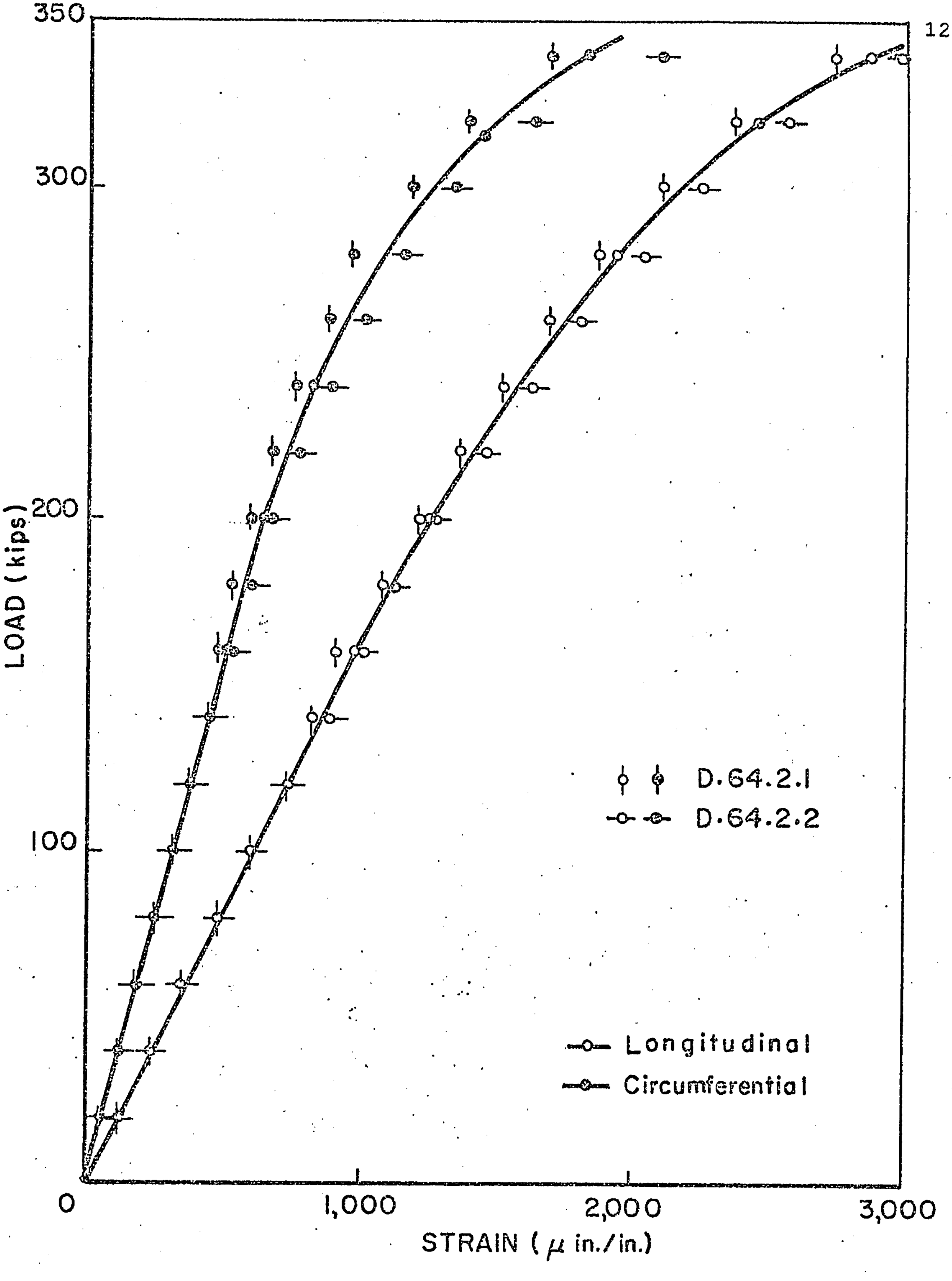
GRAPH B.16: LOAD-DEFORMATION OF SPECIMEN D-O-O SERIES



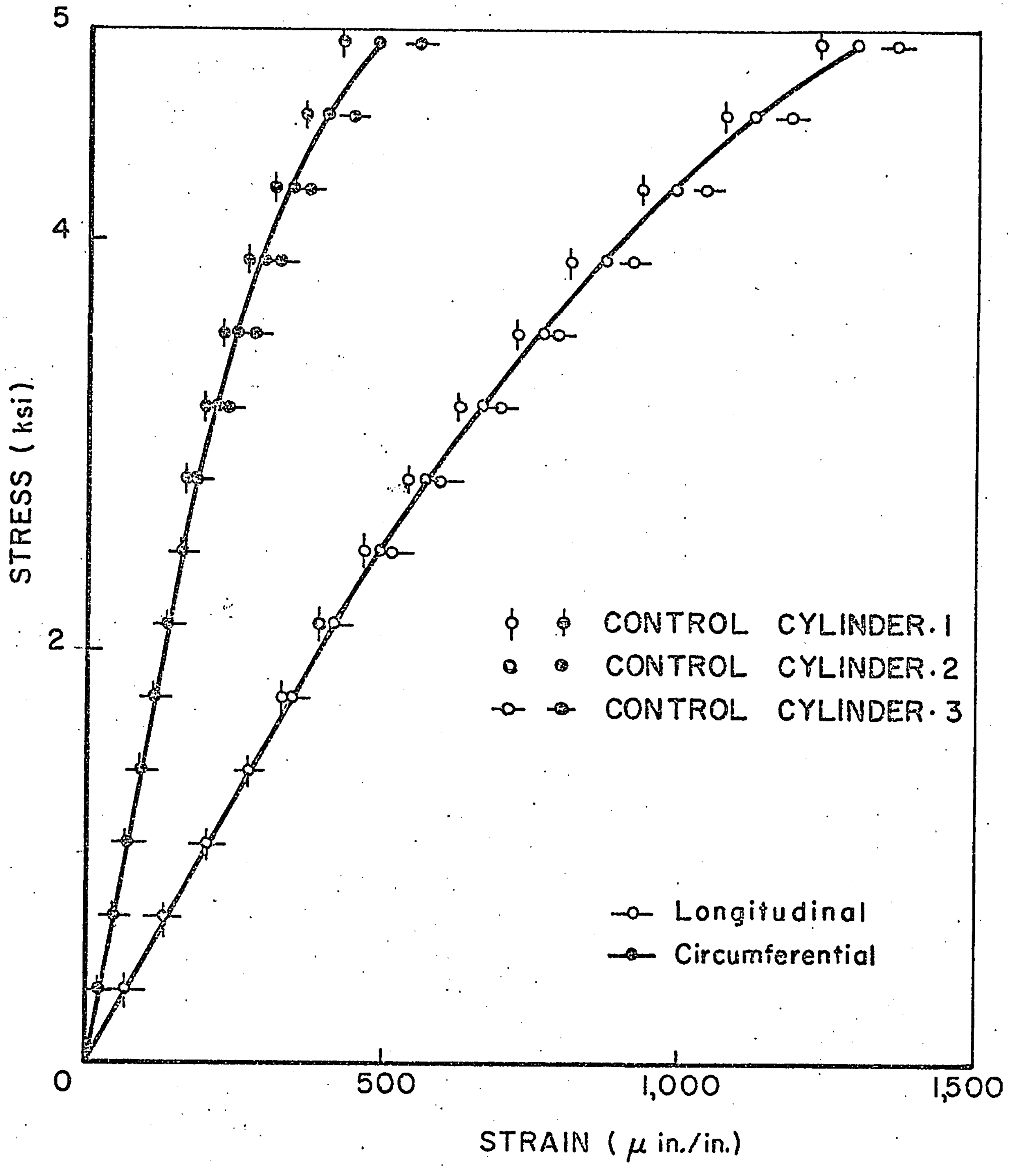
GRAPH B.17: LOAD-DEFORMATION OF SPECIMEN D-18-1 SERIES



GRAPH B.18: LOAD-DEFORMATION OF SPECIMEN D-32-1 SERIES



GRAPH B.19: LOAD-DEFORMATION OF SPECIMEN D-64-2 SERIES



GRAPH B.20:STRESS—STRAIN CHARACTERISTIC OF CONTROL CONCRETE

(FOR SPECIMENS OF D-SERIES)

VITA

The author graduated with a degree in Civil Engineering from Government Polytechnic, Poona, India, in 1962. After graduation, the author worked as an Assistant Engineer for two years in India with a Construction Engineer's firm, two years as a Design Engineer with a Consulting Engineer's firm in England and two years as a Design Engineer with Architects and Consulting Engineer's firm in Canada prior to studying for the Master's Degree in Civil Engineering at the University of Ottawa. The author is a Member of The Institution of Structural Engineers, London, England; an Associate Member of The Engineering Institute of Canada, and an Associate Member of The American Society of Civil Engineers.