University of Alberta
Department of Civil Engineering

Structural Engineering Report No. 76

Effects of Reinforcement Detailing for Concrete Masonry Columns

by

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and

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May, 1979

EFFECTS OF REINFORCEMENT DETAILING FOR CONCRETE MASONRY COLUMNS

by

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May 1979

ABSTRACT

Thirty-seven concrete masonry columns were tested in order to establish the effects of reinforcement detailing on their strength and behavior. All columns were 64 inches long, had a 16 inch square cross-section and were loaded concentrically through pinned ends. Detailing variables included size and location of tie reinforcement as well as size and amount of vertical reinforcement.

Test results obtained included failure loads, horizontal and vertical deformations and strains in both masonry and reinforcement. Auxiliary tests were conducted to establish the constituent material properties. Test results were analysed to show the effects of the reinforcement details investigated on column strength and behavior.

The ultimate strength of the masonry was increased by increasing the tie diameter but the contribution of the masonry to the strength of the columns decreased as the vertical bar diameter increased.

Further study is recommended to establish more flexible tie detailing requirements. Additional testing is required for eccentrically loaded columns to establish strength design procedures for reinforced concrete masonry columns.

ACKNOWLEDGEMENTS

This investigation was made possible through funds from the Natural Sciences and Engineering Research Council of Canada, the Alberta Masonry Institute and the Masonry Research Foundation of Canada with facilities provided by the Department of Civil Engineering. Additionally, Edcon Block kindly donated the masonry units used in the fabrication of these specimens.

This report is based on an M.Sc. thesis written by C. Feeg.

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LIST OF SYMBOLS

A = cross-sectional area

A_g = gross cross-sectional area

A_m = cross-sectional area of masonry

A_n = net cross-sectional area

A_s = cross-sectional area of vertical reinforcement

C_e = eccentricity coefficient

C_s = slenderness coefficient

d = diameter of tie reinforcement

E = modulus of elasticity

E_m = modulus of elasticity of masonry

E_s = modulus of elasticity of reinforcement

e = virtual eccentricity

e_1 = the smaller virtual eccentricity

e_2 = the larger virtual eccentricity

F_a = maximum allowable stress for axial load

F_b = maximum allowable flexural stress

f_a = actual stress due to axial load

f_b = actual flexural stress

f_m = compressive stress in masonry

f'_m = ultimate compressive strength of masonry

f_s = allowable stress in reinforcement

f_y = yield strength of reinforcement

h = height of column

I = moment of inertia of tie cross section

L = length of one side of square tie

P = allowable axial load for column

p = percentage of vertical reinforcement

p_g = percentage of vertical reinforcement based on A_g

P_m = vertical load carried by masonry

p_n = percentage of vertical reinforcement based on A_n

P_s = vertical load carried by vertical reinforcement

P_u = ultimate load carried by column

t = least thickness of column

ε_m = measured strain on surface of column

ε_s = average measured strain in vertical reinforcement

CHAPTER I

INTRODUCTION

1.1 General Remarks

From the beginning of recorded history masonry has been used as a construction material. Many of the ancient stone masonry structures are still in existence today in both mild and harsh climatic regions.

Today masonry is a major construction material and is widely used. Flexibility in the design and layout of buildings has made masonry very popular with designers. Its ability to simultaneously serve an architectural and a structural function is an added advantage of masonry construction.

Recent research in structural masonry has provided greater knowledge of material properties and structural behavior. Material properties and construction practices are controlled through specifications and building codes. Greater control of material properties through specifications has resulted in a reliable product which may be confidently used in designs based on engineering principles.

Concrete masonry units are manufactured products which are produced in accordance with standards which ensure strength, durability and dimensional tolerance. Since their inception they have been used economically in both non-loadbearing and loadbearing walls which have more recently been reinforced for additional strength and ductility. Tensile stress is carried by the reinforcement, resulting in more slender walls.

Reinforced concrete masonry columns are common structural elements of buildings in which concrete masonry walls are used. When these columns are built contiguous with the walls they serve as stiffening elements as well as compression members. It is often advantageous and economical to design reinforced concrete masonry columns using standard size wall units although special units are frequently used. The study presented herein provides additional information to aid in the design of reinforced concrete masonry columns.

Objectives and Scope

An experimental study was undertaken in order to meet the following objectives:

a) to determine the effects of reinforcement detailing on the strength and behavior of reinforced concrete masonry columns.

b) to provide further data for the establishment of more rational design procedures for reinforced concrete masonry columns.

c) to provide comparative data on the ductility of concrete masonry columns.

The investigation was restricted to the study of short concentrically loaded columns to eliminate effects of slenderness and bending.

CHAPTER II

REVIEW OF PREVIOUS WORK AND CODE DESIGN PROCEDURES

2.1 Introduction

The design of reinforced masonry columns and the detailing of reinforcement for concrete columns have been the subjects of various investigations. A brief review of this research work is presented in this chapter. Current building code requirements for the design of reinforced masonry columns are also examined.

2.2 Review of Previous Work

2.2.1 Brick Masonry Columns

Tests of brick piers were reported as early as 1882.[1] In 1933 the first tests of reinforced brick masonry columns were conducted. Lyse[1] found the strength of reinforced brick columns to be made up of the effective strength of the masonry plus the yield strength of the longitudinal reinforcement. Results of tests by Withey[2] indicated that column strength varied directly with the strength of the masonry and the percentage of longitudinal reinforcement, and was increased by the use of lateral reinforcement. Results of eccentric load tests by Davey and Thomas[3] were used by Brettle[4] to substantiate a proposed ultimate strength design procedure for reinforced brick columns. Another ultimate strength design procedure was proposed by Anderson and Hoffman[5] based on a method used for the design of reinforced concrete columns.

2.2.2 Concrete Masonry Columns

Plain concrete block pilasters were tested by Shank and Foster[6] to establish ultimate strength and elastic properties when subjected to eccentric loading. To the writer's knowledge, there have been no tests conducted on reinforced concrete masonry columns.

2.2.3 Reinforced Concrete Columns

Numerous studies related to protective shells or tie reinforcement have been carried out for reinforced concrete columns. Richart[7] et al have shown that the concrete cover outside the reinforcement of tied concrete columns is less effective than the protective shell of spiral columns.

Small tie spacings were shown to increase the ultimate strength of concrete columns by King.[8] Hudson[9] found ties had no effect on the ultimate strength but had a strong influence on the mode of failure of axially loaded columns. Results of a study by Pfister[10] indicated that the primary function of ties was to provide lateral restraint for the concrete. Experimental results obtained by Bunni[11] showed that tie reinforcement has a significant effect on the behavior and ultimate load carrying capacity of tied reinforced concrete columns.

2.3 Building Code Requirements

Design procedures and reinforcement detailing requirements for masonry columns are examined here for four codes of practice. These building codes are:

a) "Masonry Design and Construction for Buildings", Canadian Standards Association S-304, 1977.[12]

b) "Uniform Building Code", International Conference of Building Officials, 1976.[13]

c) "British Standard Code of Practice", CP-111-1970.[14]

d) "Plain and Reinforced Masonry Structures Design Standards 1972", Building Standards and Regulations (U.S.S.R.)[15]

Table 2.1 provides a comparison of the four code requirements relating to column dimensions and reinforcement detailing.

2.3.1 Reinforced Masonry Column Design According to CSA S304-1977[12]

In this code the vertical load carrying capacity of a reinforced masonry column is based on allowable stresses. The allowable stress for the masonry, f_m, is dependent on the masonry strength, f'_m. This compressive

TABLE 2.1 COMPARISON OF BUILDING CODE REQUIREMENTS

Code Requirement	CSA S304-1977	Uniform Building Code 1976	British...CP111-1970	U.S.S.R. 1972
Maximum Slenderness	25[1]	20	24	18.75[2]
Size of Cross-Section	N/A	Minimum Dimension 12 in. or 8 in.[3]	For cross-section area less than or equal 139.5 sq.in. reduce allowable stress by	
			$0.75 + \frac{A}{1.3}$	0.8
Minimum Vertical Steel Ratio	0.5% or 0.27%[3]	0.5% or 0.25%[3]	0.4%	Compressive -0.1% Tensile-0.05%
Maximum Vertical Steel Ratio	4.0%	4.0%	4.0%	N/A
Minimum Number of Vertical Bars	N/A	4	4	N/A
Minimum Diameter of Vertical Bars	N/A	0.375 in.	N/A	0.315 in.
Maximum Diameter of Vertical Bars	N/A	1.25 in.	N/A	N/A

<table>
<tr><th>Code Requirement</th><th>CSA S304-1977</th><th>Uniform Building Code 1976</th><th>British...CP111-1970</th><th>U.S.S.R. 1972</th></tr>
<tr><td>Minimum Tie Diameter</td><td>0.1483 in.</td><td>0.25 in. or 0.375 in.[4,5]</td><td>Not less than 1/4 of vertical bar diameter but greater than 0.24 in.</td><td>0.118 in.[6]</td></tr>
<tr><td rowspan="4">Tie Spacing</td><td colspan="3">Not exceeding the smallest of</td><td rowspan="4">To provide a minimum volumetric percentage of 0.1% mesh[7] max. - 1.0%</td></tr>
<tr><td colspan="2">16-bar dia., 48-tie diameters.</td><td>12-bar diameters</td></tr>
<tr><td colspan="3">nor the least dimension of the column</td></tr>
<tr><td>N/A</td><td>nor 18 in.[8]</td><td>nor 11.8 in.</td></tr>
<tr><td rowspan="2">Tie Location</td><td colspan="2">In horizontal mortar joint[9] or in contact with vertical steel</td><td rowspan="2">N/A</td><td rowspan="2">N/A</td></tr>
<tr><td>See footnote 10</td><td>≮ 1½" nor > 5" from surface of column[11]</td></tr>
<tr><td>Clearance Between Vertical Bars</td><td>N/A</td><td>2½ bar diameters center to center</td><td>N/A</td><td>N/A</td></tr>
</table>

Footnotes to Table 2.1

1. Dependent on e_1/e_2.
2. Dependent on strength and type of masonry, strength of mortar, and least dimension of column.
3. Where designed for half the allowable stress.
4. For vertical reinforcement 1.0 in. diameter or larger.
5. May use 0.25 in. diameter ties if they are spaced to provide the same area and are placed in horizontal bed joints of at least 1/2-inch thickness.
6. The maximum diameters for reinforcement in horizontal joints is 0.315 in. or 0.197 in. if it intersects. Joint thickness must be at least 0.16 in. greater than reinforcement diameter.
7. Distance between mesh wires not more than 4.7 in. nor less than 1.2 in.
8. Smaller spacings depending on seismic zone.
9. Minimum cover of 5/8 in. for 0.25 in. diameter or smaller bars in horizontal mortar joint.
10. Allows 0.25 in. diameter bars in 3/8 in. thick mortar joints.
11. Special anchorage requirements are specified depending on seismic zone and if tie is placed in horizontal mortar joint.

strength may be determined by testing or it may be assumed on the basis of the compressive strength of the masonry unit.

Two design procedures are employed depending on the magnitude of the vertical load eccentricity, e, in relation to the thickness of the section, t. For a column with tied vertical reinforcement, subject to bending about one principal axis, the procedures are as follows:

a) for $e \leq t/3$,

$$P = C_e C_s (f_m + 0.8\ p_n f_s)\ A_n$$

where;

C_e = eccentricity coefficient

$= 1.0$ for $e \leq t/20$

$$= \frac{1.3}{1 + 6\frac{e}{t}} + \frac{1}{2}\left(\frac{e}{t} - \frac{1}{20}\right)\left(1 - \frac{e_1}{e_2}\right)$$

for $t/20 < e \leq t/6$

$$= 1.95\left(\frac{1}{2} - \frac{e}{t}\right) + \frac{1}{2}\left(\frac{e}{t} - \frac{1}{20}\right)\left(1 - \frac{e_1}{e_2}\right)$$

for $t/6 < e \leq t/3$

e_1 = the smaller virtual eccentricity

e_2 = the larger virtual eccentricity

C_s = slenderness coefficient

$$= 1.2 - \frac{h/t}{300}\left[5.75 + \left(1.5 + \frac{e_1}{e_2}\right)^2\right] \leq 1.0$$

h = effective height of column

f_m = allowable compressive stress for masonry

= $0.20\ f'_m$

p_n = ratio of the area of reinforcement to the net cross-sectional area

f_s = allowable compressive stress for reinforcement

= 40% of the yield stress, but less than 24,000 psi

A_n = net cross-sectional area.

b) For $e > t/3$ or a value producing tension in the reinforcement, an elastic analysis is used to determine the allowable load on a cross-section. Slenderness effects are taken into account by modifying this load by means of the slenderness coefficient, C_s. For this case the allowable masonry stresses are:

f_m = $0.32\ f'_m$ for brick masonry columns

= $0.28\ f'_m$ for concrete masonry columns

The allowable tensile or compressive stresses for the reinforcement range from 18,000 psi to 24,000 psi depending on the grade of the reinforcement. The modulus of elasticity of steel is assumed to be 29,000,000 psi while the modulus of elasticity of masonry is taken as 1000 f'_m, but not greater than 3,000,000 psi.

2.3.2 Reinforced Masonry Column Design According to the Uniform Building Code, 1976 [13]

In this code the design of masonry columns is based on specified allowable stresses. If inspection during construction is not provided a special provision requires that the allowable stresses for masonry be reduced by 50%.

The allowable axial load for a column is given by:

$$P = A_g \; (0.18 \; f'_m + 0.65 \; p_g f_s) \left[1 - \left(\frac{h}{40t}\right)^3\right]$$

where;

A_g = the gross area of the column.

f'_m = ultimate compressive masonry strength as determined by test or assumed base on unit strength.

$\leq$ 6000 psi

p_g = ratio of the area of vertical reinforcement to the gross area, A_g.

f_s = allowable stress in reinforcement.

= 40% of the yield stress, but no greater than 24,000 psi.

h = unsupported height of column.

t = least thickness of column.

For bending in combination with axial load the following criterion must be met:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1$$

where;

f_a = actual stress due to axial load.

F_a = maximum allowable stress for axial load.

= $0.18\ f'_m$

f_b = actual flexural stress.

F_b = maximum allowable flexural stress.

= $0.33\ f'_m$, but not greater than 900 psi.

The value of 1 in the above equation may be increased by one third in the case of temporary loads due to wind and earthquake. The resulting design, however, must not be less than the design determined using only dead and live loads.

2.3.3 Reinforced Masonry Column Design According to British Standard Code of Practice CP-111-1970[14]

This code of practice employs an elastic analysis based on allowable stresses. The allowable stress for masonry is dependent on the type of masonry unit and its strength. In design, these allowable stresses are reduced by a factor accounting for slenderness and eccentricity. The modular ratio to be used in the elastic analysis is specified for ranges of the allowable stress.

2.3.4 Reinforced Masonry Column Design According to Building Standards and Regulations (U.S.S.R.), 1972[15]

The limit states of load-bearing capacity, formation and opening of cracks and deformation are included in this building code. Design resistances for different types and strengths of masonry units are tabulated for assemblages of different course heights and mortar strengths. The design resistance of vertical reinforcement restrained by ties is evaluated for restraint in one direction and for restraint in two directions. For deformed bars these are 22,760 psi and 34,140 psi respectively. The design of a mesh reinforced element incorporates factors accounting for:

- slenderness
- modulus of elasticity
- type of masonry
- mortar strength
- percentage of longitudinal reinforcement
- percentage of mesh reinforcement
- long term load effects

The use of mesh reinforcement is prohibited for eccentricities outside the kern limits of the section and for slenderness above a specified value.

CHAPTER III

RESEARCH PROGRAM

3.1 Introduction

The research program described in this chapter was undertaken in order to determine the effects of reinforcement detailing on the strength and behavior of reinforced concrete masonry columns. First, the program variables are defined and then incorporated into the design of the test specimens. Next, the construction of the columns is described. Finally, an outline of the testing procedure and the auxiliary tests is given.

3.2 Program Variables

Three major reinforcement detailing variables were investigated.

1. The diameter and location within the cross section of tie reinforcement.
2. The distribution of the cross sectional area of vertical reinforcement.
3. The amount of vertical reinforcement.

Other variables studied were the yield strength of vertical reinforcement and the type of concrete masonry unit used in the construction of columns.

Other factors which could affect the strength and behavior of reinforced concrete masonry columns were controlled. These included loading, column dimensions and construction, strength of masonry materials and location of reinforcement.

For this investigation vertical load was applied concentrically to the columns through pinned ends. All columns were 64 inches high and all had a nominal 16 inch square cross section. Only face shell mortar was used and a mortar joint thickness of 3/8 inch was maintained. Cross sections composed of two wall units were constructed using running bond. Experienced masons were employed to construct the specimens and all concrete units were supplied in one lot by one manufacturer. The proportioning, mixing and curing of mortar and grout were controlled in the laboratory. An 8 inch vertical spacing was maintained for all tie reinforcement since placement was restricted to the space between courses. The location in the cross section of vertical reinforcement was controlled by maintaining the centroid of the vertical reinforcement within a cell coincident with the centroid of the cell.

3.3 Design of Test Specimens

Two series of test specimens were designed to assess the contribution of tie reinforcement to the strength and behavior of reinforced concrete masonry columns. The first series had tie diameter as the variable while tie location in the cross section was the variable in the second series.

Three tie diameters; #9 wire, 3/16 in and 0.25 in. were chosen for the first series. The smallest diameter tie is the minimum allowed in CSA S-304, 1977. The largest size, 0.25 inch diameter, was selected as the maximum size which could be satisfactorily placed in the 3/8 inch thick mortar joint. All three tie diameters were readily available as either masonry wall reinforcement or concrete reinforcement. For this series all tie reinforcement was located within the mortar joint of the cross section. For comparison, columns were also constructed without ties. All vertical reinforcement parameters were maintained constant for this series with the exception that columns without vertical reinforcement were constructed with either minimum size ties or without tie reinforcement.

Two tie locations within the cross section were investigated in the second series. Tie reinforcement was located either in the horizontal mortar joint or in contact with the vertical reinforcement. Ties of 0.25 inch diameter were placed in the horizontal joints as blocks were laid. In the case of ties located in contact with vertical reinforcement an exception was made for the column constructed of special units where the ties were wired to the vertical reinforcement and all reinforcement was then placed as a unit.

Another series of test specimens was designed to assess the contribution of vertical reinforcement to the strength and behavior of reinforced concrete masonry columns. The effect of the distribution of vertical reinforcement area was investigated by testing columns with large reinforcing bars and companion columns reinforced with smaller diameter bars. Both column types contained the same cross sectional area of vertical reinforcement. The number of small bars used in the companion columns was either 2, 3 or 4 times the number used in columns containing large bars as shown in Table 3.1. All vertical reinforcement percentages are within the code requirements of 0.5% to 4.0%. Larger percentages than 1.29% were not feasible for this cross section due to placement difficulties. The reinforcement percentages provided by large bars were used to study the effect of the amount of vertical reinforcement provided. The ability of vertical reinforcement to reach its specified yield strength was determined by measuring the strains on both grade 40 and grade 60 reinforcement. Outer shells of columns were tested to establish their strength and behavior compared to reinforced concrete masonry columns.

The above variables and their effects on column strength and behavior were investigated using the fifteen details described in Table 3.2

TABLE 3.1 VERTICAL REINFORCEMENT DETAILS

No. of Small Bars / No. of Large Bars	Small Bars			Large Bars		
	Detail	A_s (in.^{2})	p(%)	Detail	A_s (in.^{2})	p(%)
2:1	4-#5 & 4-#6	3.00	1.229	4-#8	3.16	1.294
3:1	12-#4	2.40	0.983	4-#7	2.40	0.983
4:1	16-#3	1.76	0.721	4-#6	1.76	0.721

TABLE 3.2 COLUMN DETAILS

Column Mark	Vertical Reinforcement		Tie Reinforcement	
	Detail	Grade (ksi)	Diameter (in.)	Location[1]
A1	4-#6	40	N/A	N/A
A2	4-#6	40	0.25	1
A3	4-#6	40	0.25	2
A4	4-#6	40	0.1875	1
B1	N/A	N/A	0.1483	1
B2	4-#6	40	0.1483	1
B3	16-#3	40	0.1483	1
B4	4-#7	40	0.1483	1
B5	12-#4	40	0.1483	1
B6	4-#8	40	0.1483	1
B7	4-#5 & 4-#6	60	0.1483	1
C1	N/A	N/A	N/A	N/A
C2[2]	N/A	N/A	N/A	N/A
D1[3]	4-#6	60	0.25	1
D2[3]	4-#6	60	0.25	2

Footnotes to Table 3.2

1. Location 1 is in mortar joint. Location 2 - in contact with vertical reinforcement.
2. Ungrouted column.
3. Column built using special units instead of wall units.

3.4 Construction of Test Specimens

Masonry wall reinforcement was used to fabricate the #9 wire and 3/16 inch diameter ties. The rungs of the ladder type wall reinforcement were removed and the remaining straight wires were bent to form a 13.5 inch square. As shown in Figure 3.1, the bends had a 1 inch radius and the ends were lapped 32 bar diameters.

The 0.25 inch diameter ties were cut from #2 reinforcing bars which were supplied in 40 foot lengths. A 3/16 inch radius bend was used in fabricating the 13.5 inch and 9.5 inch square ties and laps of 32 and 16 bar diameters respectively, were provided. Figure 3.2 shows the dimensions and placement within a typical cross section. Also shown are locations of electrical resistance strain gages. The method used in instrumenting reinforcement is described in Appendix A. All ties required slight straightening for ease of placement in the 3/8 inch thick mortar joint. For most 0.25 inch diameter ties this necessitated small tack welds as shown in Figure 3.2.

Vertical reinforcement was supplied in 6 foot lengths and instrumented with strain gages at locations corresponding to the mid-height of the columns. Prior to placement of vertical reinforcement, 1/4 inch diameter holes were drilled through the outside shell of the columns to accommodate the connection of the strain gages. The vertical reinforcement was then lowered into position

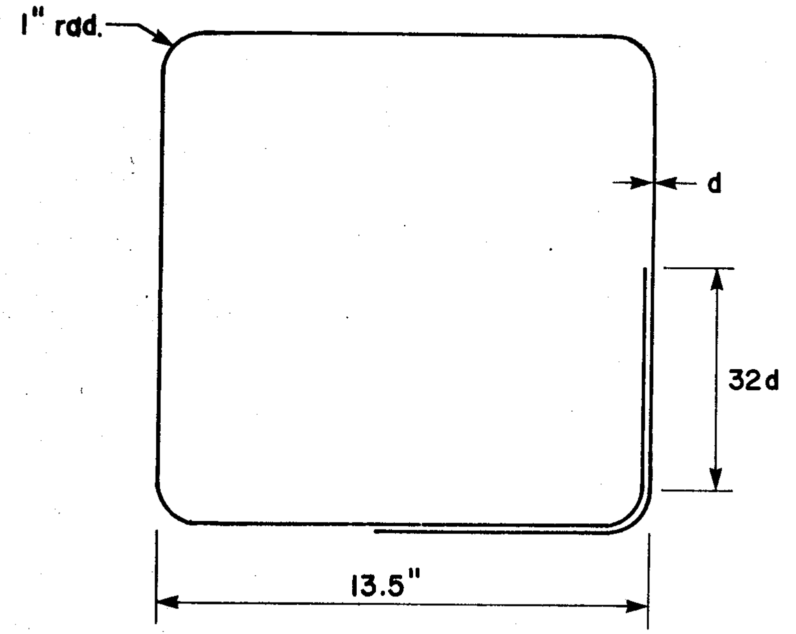


Figure 3.1 ' Tie Details for #9 Wire and 3/16 in. Diameter Ties

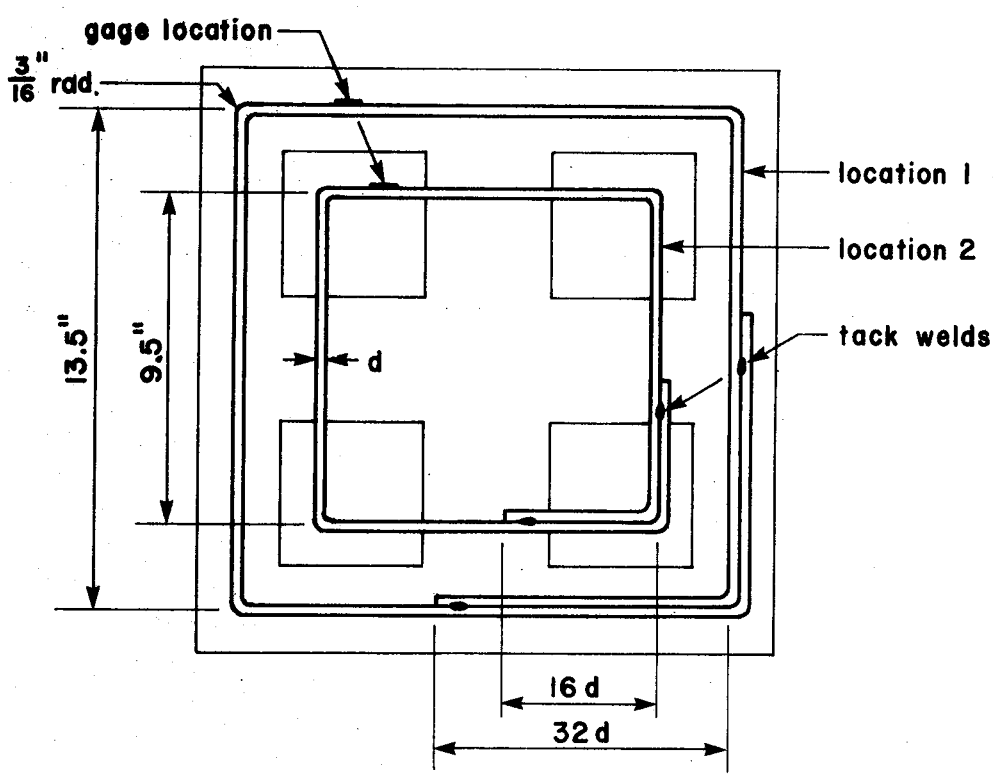


Figure 3.2 Tie Detail for 0.25 in. Diameter Ties

from the top of the columns and centered in the cell openings.

Type S mortar was used in the construction of all columns. Volume proportions of 1:1/2:4 of normal portland cement, lime and aggregate were used. A 1-1/4 cubic foot batch was dry mixed. Then water was added to obtain a consistency desirable to the mason. Ties, where required, were positioned by the mason with the tie lap consistently located at one corner of the cross section. Face shell mortar was then applied and the dry concrete masonry unit was placed, plumbed and leveled. For columns in which cleanout openings were provided, mortar droppings were removed from the bottom of the columns and the openings were then blocked off with plywood forms.

Coarse grout with proportions of 1:3:2 of normal portland cement, sand and pea gravel was batched by volume and mixed in a flat bottom paddle type mixer. Approximately 100 pounds of water was used in each 6 cubic foot batch to produce a very fluid mix. A hopper was charged with half of the batch and lifted by the laboratory crane into position for discharge. The fluid grout was discharged, vibrated and the top surface was trowelled. The columns were then allowed to cure at the temperature and humidity of the laboratory.

3.5 Testing

An MTS Systems closed-loop electro-hydraulic testing machine was used to apply the concentric vertical load to the columns. The 1.4 million pound capacity testing machine was calibrated to within ± 0.3% error in the ranges used for these tests. Automated data collection and processing was employed using the Nova 210/E digital computer. This unit received input from 13 channels used during testing.

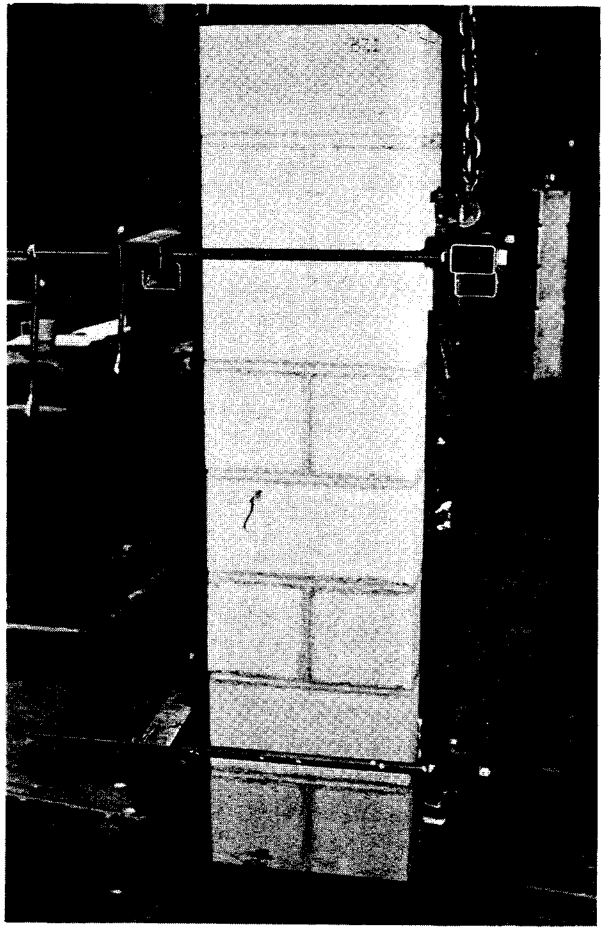
The procedure used in testing the columns was as follows:

a) The test specimen was prepared by cutting off the protruding vertical reinforcement. It was then transported to the testing machine by the laboratory crane while supported by the clamping device shown in Plate 3.1(a).

b) The specimen was positioned in the testing machine as shown in Plate 3.1(b). After the specimen was plumbed, the ends were capped with plaster as a small preload was applied. Column ends and end fixtures are shown in Plate 3.2(a) and (b).

c) Transducers (LVDT's) were positioned and surface strain gages applied as required and all electrical connections were completed.

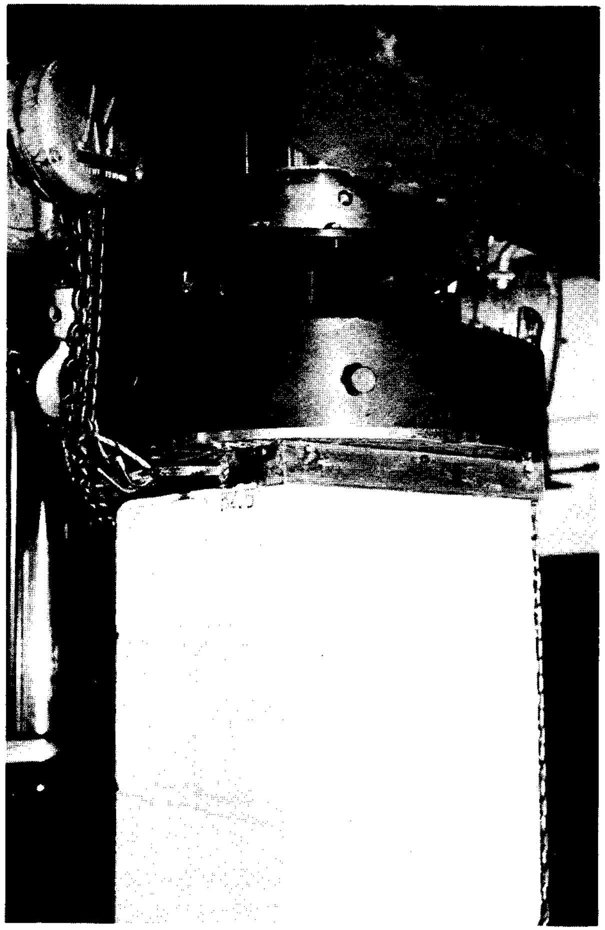
d) The preload was increased to 100 kips and



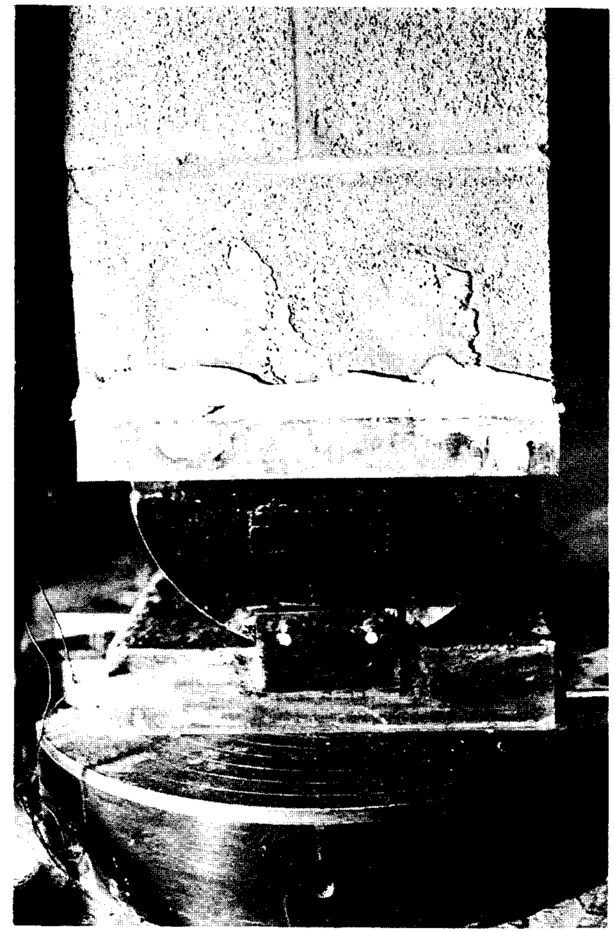
(a) Column being lifted using clamping device

(b) Column being positioned in testing machine

PLATE 3.1 TRANSPORTING AND POSITIONING TEST SPECIMEN



(a) Top of column



(b) Bottom of column

PLATE 3.2 COLUMN ENDS AND FIXTURES

then decreased to 5 kips. Initial instrumentation readings were then taken.

e) Load was applied until failure of the specimen occurred. Data readings were taken at each 40 kip increment of load.

After failure each specimen was sketched and photographed. Collected data was printed out and transferred to the Amdahl 470 computer. A Fortran program was used to plot this data using a CalComp plotter.

Auxiliary tests were conducted in order to establish important material properties. These consisted of compression tests of concrete masonry units, mortar, prisms and grout, and tensile tests of the reinforcement. These auxiliary tests are described in Appendix A.

CHAPTER IV

TEST RESULTS

4.1 Introduction

The results of 37 concrete masonry column tests are presented in this chapter in tabular, graphical and photographic form. Data used in plotting graphs are those read from electrical resistance strain gages and LVDT's through use of a program implemented in the Nova data collection system. Typical data plots are included in this chapter while Appendix B contains all the plots. Table 4.1 gives a description of the various columns tested and their failure loads.

4.2 Load-Deformation Relationships

Figure 4.1 shows the relationship between load and vertical deformation for column group B3 which is typical of the reinforced columns tested. Horizontal deflections are used to plot the deflected shape of columns under various loads. One such plot is presented in Figure 4.2 for column B1.2 which was instrumented with LVDT's.

4.3 Load-Strain Relationships

Strain readings from gages mounted at mid-height on the vertical reinforcement are plotted against load for either 2 or 4 gages per column. A typical plot is shown in Figure 4.3 for column B2.1 which had a gage mounted on each vertical bar.

Strains in the two 0.25 inch diameter ties per column are plotted against load in Figure 4.4 for column A2.1.

Figure 4.5 presents vertical and horizontal surface concrete strains plotted against load for a typical grouted unreinforced column.

4.4 Failure Modes and Auxiliary Test Results

Photographs of failed specimens are shown in Plates 4.1 to 4.15 for each of the fifteen column groups.

Tables 4.2 to 4.5 present the ultimate load and stress values for the auxiliary tests of masonry units, mortar, prisms and grout specimens. Results of reinforcement tensile tests are tabulated in Table 4.6.

Plots of load versus vertical deformation for eight prism tests are shown in Figures 4.6 and 4.7.

Stress versus strain data was obtained for three grout tests and sixteen reinforcement tests. This data is presented in Figure 4.8 and 4.9 respectively.

TABLE 4.1

SUMMARY OF TEST RESULTS

Column Group	Column No.	Vertical Reinf. Detail	Vertical Reinf. Grade	Tie Reinf. Dia. (in.)	Tie Reinf. Location[1]	Unit Type[2]	Failure Load (kips)
A1	.1	4-#6	40	N/A	N/A	W	506.0[5]
	.2	4-#6	40	N/A	N/A	W	525.1[5]
	.3	4-#6	40	N/A	N/A	W	538.4[5]
A2	.1	4-#6	40	0.25	1	W	556.3
	.2	4-#6	40	0.25	1	W	626.1
A3	.1	4-#6	40	0.25	2	W	610.2
	.2	4-#6	40	0.25	2	W	599.6
A4	.1	4-#6	40	0.1875	1	W	589.3
	.2	4-#6	40	0.1875	1	W	575.8
B1	.1	N/A	N/A	0.1483	1	W	470.4[5]
	.2	N/A	N/A	0.1483	1	W	471.0[5]
	.3	N/A	N/A	0.1483	1	W	454.1[5]
B2	.1	4-#6	40	0.1483	1	W	566.5[5]
	.2	4-#6	40	0.1483	1	W	527.7[5]
	.3	4-#6	40	0.1483	1	W	505.3[5]
	.4	4-#6	40	0.1483	1	W	589.2
	.5	4-#6	40	0.1483	1	W	545.2
B3	.1	16-#3[6]	40	0.1483	1	W	630.0[5]
	.2	16-#3[6]	40	0.1483	1	W	625.0[5]
	.3	16-#3[6]	40	0.1483	1	W	629.4[5]
B4	.1	4-#7	40	0.1483	1	W	600.0
	.2	4-#7	40	0.1483	1	W	518.1
B5	.1	12-#4[7]	40	0.1483	1	W	680.2
	.2	12-#4[7]	40	0.1483	1	W	620.6
B6	.1	4-#8	40	0.1483	1	W	520.5
	.2	4-#8	40	0.1483	1	W	666.7

TABLE 4.1 (Cont'd.)

Column Group	Column No.	Vertical Reinf.		Tie Reinf.		Unit Type[2]	Failure Load (kips)
		Detail	Grade	Dia. (in.)	Location[1]		
B7	.1	4-#5 & 4-#6[7]	60	0.1483	1	W	592.8
	.2	4-#5 & 4-#6[7]	60	0.1483	1	W	605.2
C1	.1	N/A	N/A	N/A	N/A	W[3]	468.3
	.2	N/A	N/A	N/A	N/A	W[3]	412.4
	.3	N/A	N/A	N/A	N/A	W[3]	450.3
C2	.1	N/A	N/A	N/A	N/A	W[4]	111.0
	.2	N/A	N/A	N/A	N/A	W[4]	138.9
	.3	N/A	N/A	N/A	N/A	W[4]	120.5
D1	.1	4-#6	60	0.25	1	S	601.1
	.2	4-#6	60	0.25	1	S	560.2
D2	.1	4-#6	60	0.25	2	S	570.2

Footnotes to Table 4.1.

1. Location 1 is in mortar joint, Location 2 is in contact with vertical reinforcement.
2. W - Wall Unit, S - Special Concrete Masonry Unit.
3. Grouted
4. Plain
5. No clean-out openings provided.
6. Evenly spaced bars in cell.
7. Bundled bars in cell.

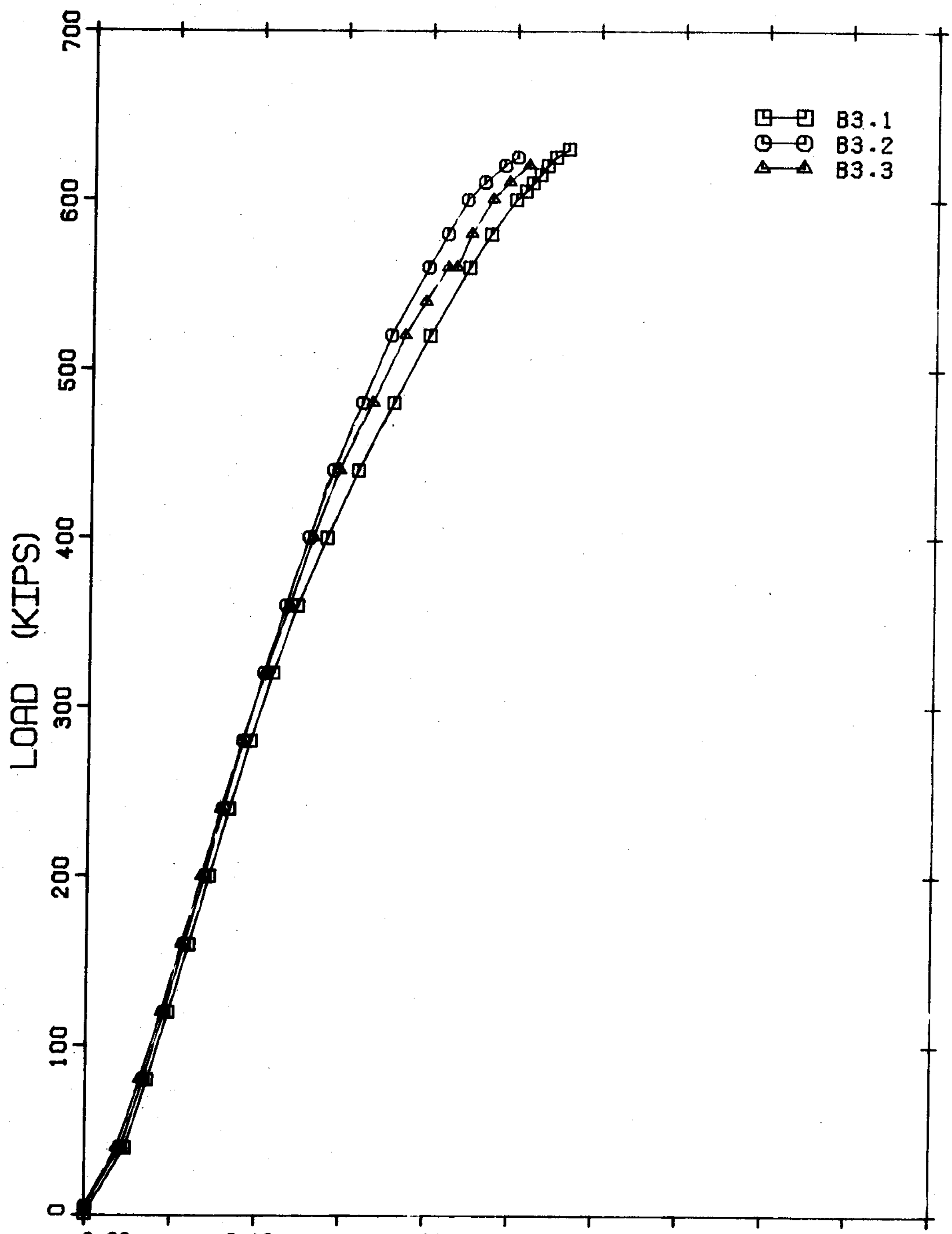


FIGURE 4.1 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP B3

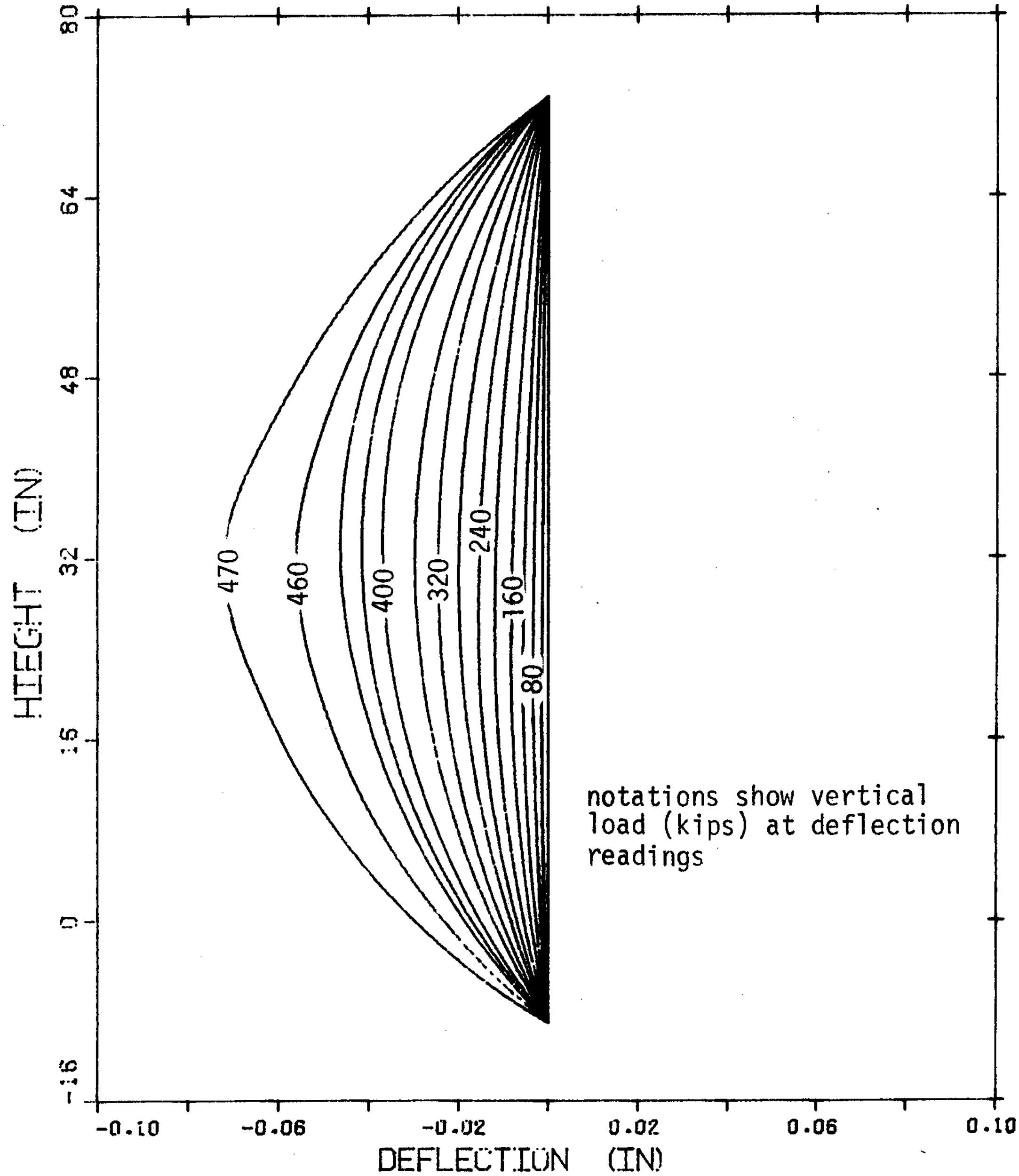


FIGURE 4.2 DEFLECTED SHAPE OF COLUMN B1.2

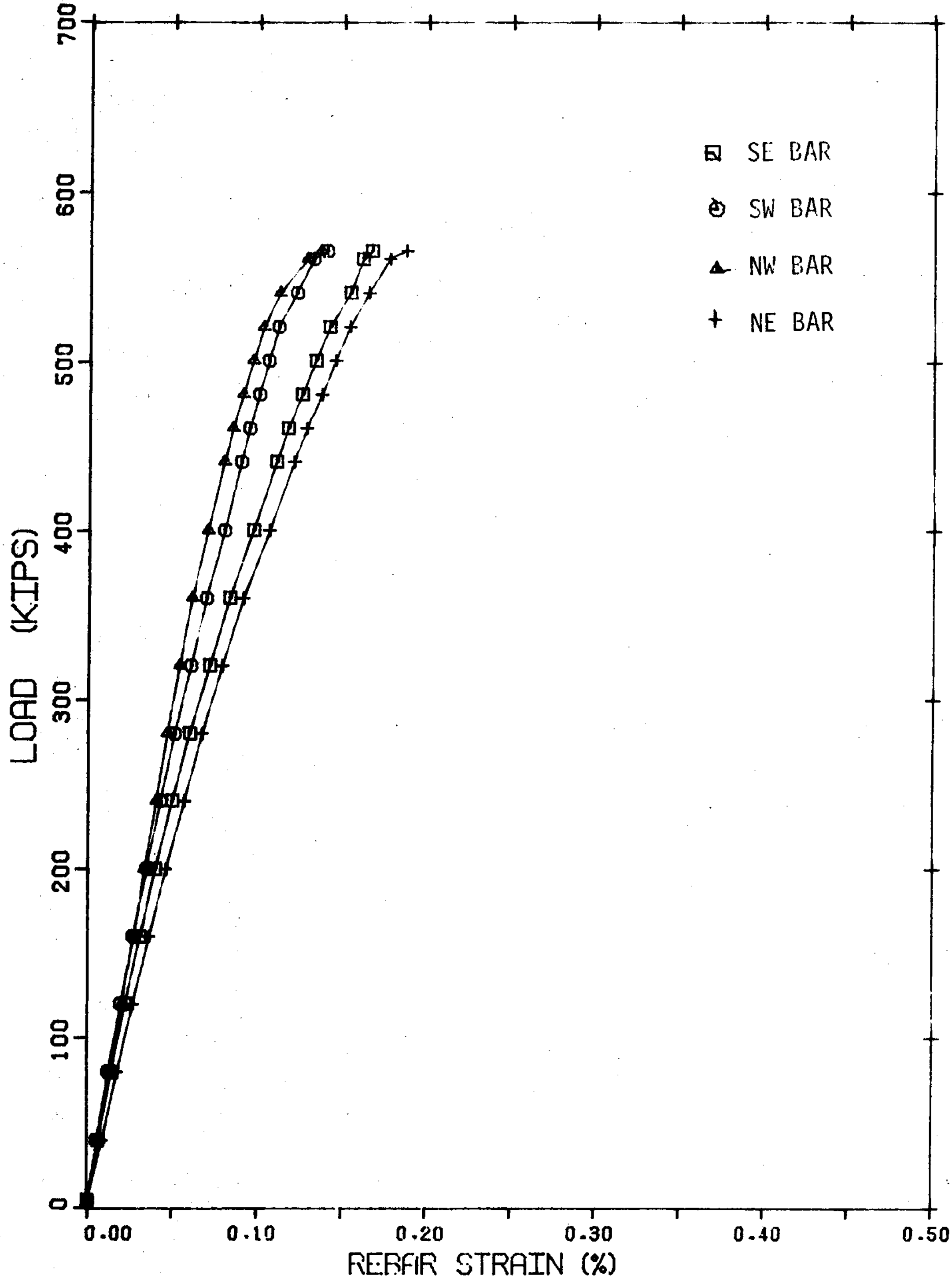


FIGURE 4.3 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B2.1

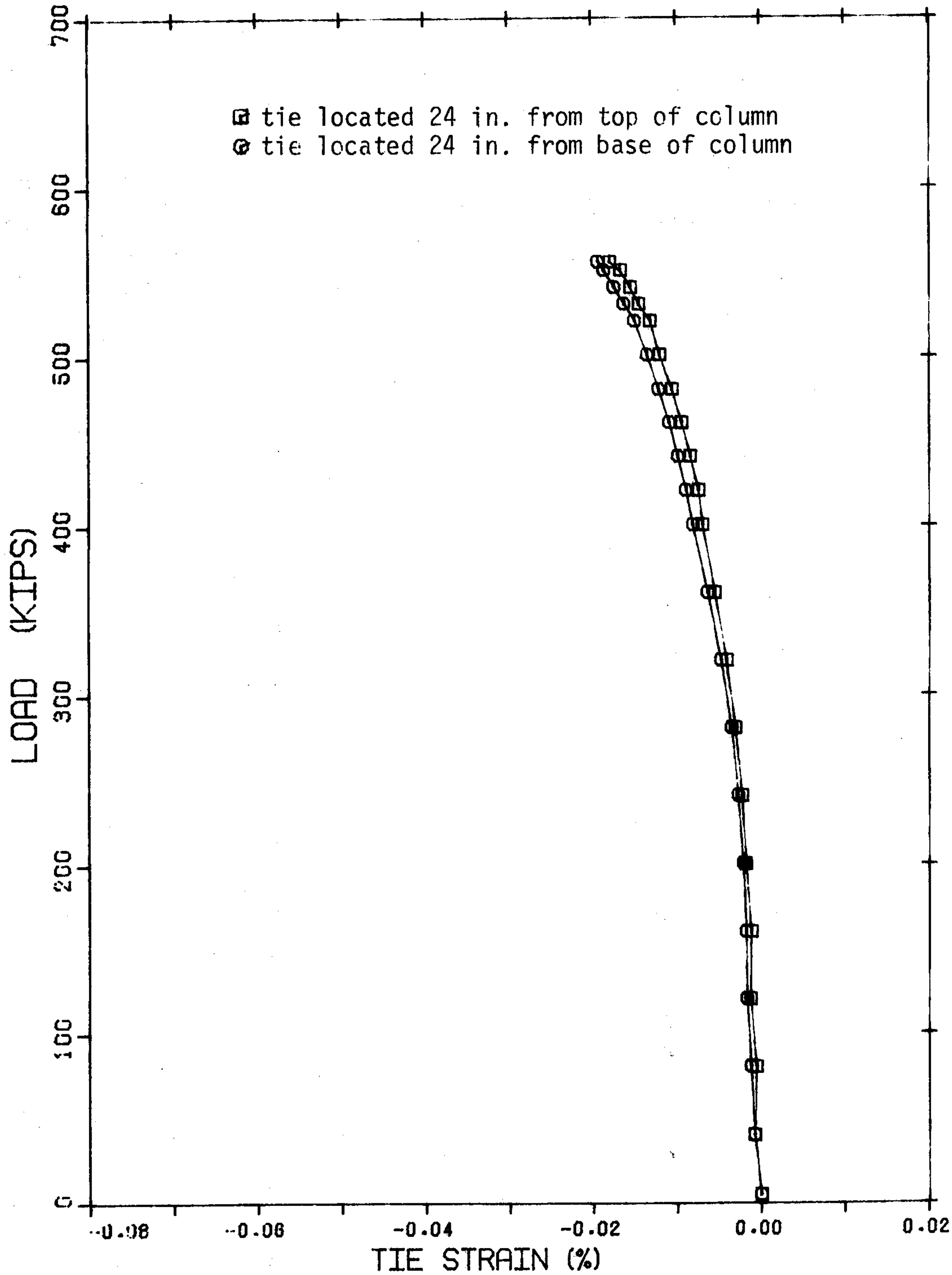


FIGURE 4.4 LOAD VS TIE STRAIN
FOR COLUMN A2.1

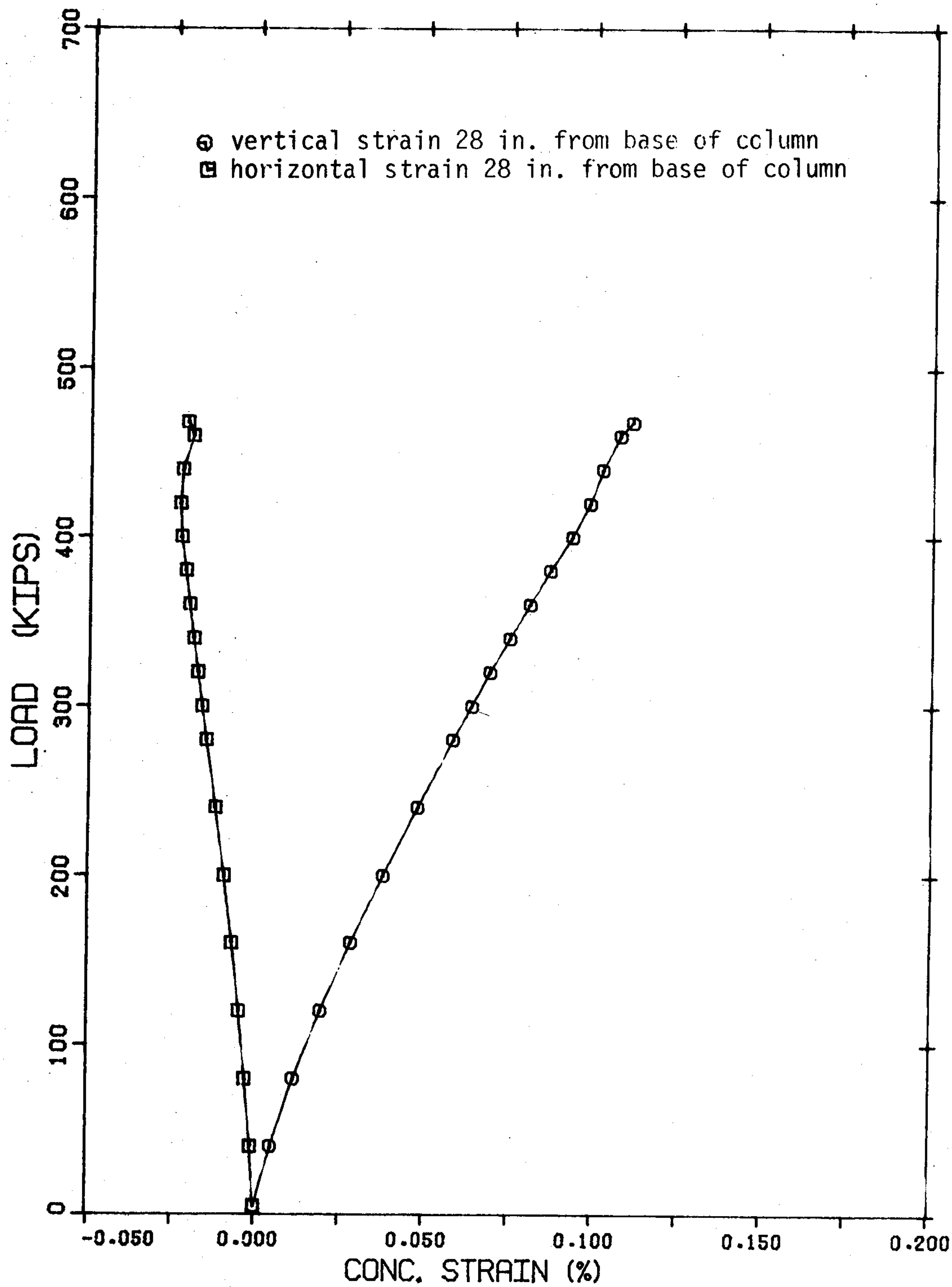


FIGURE 4.5 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN C1.1

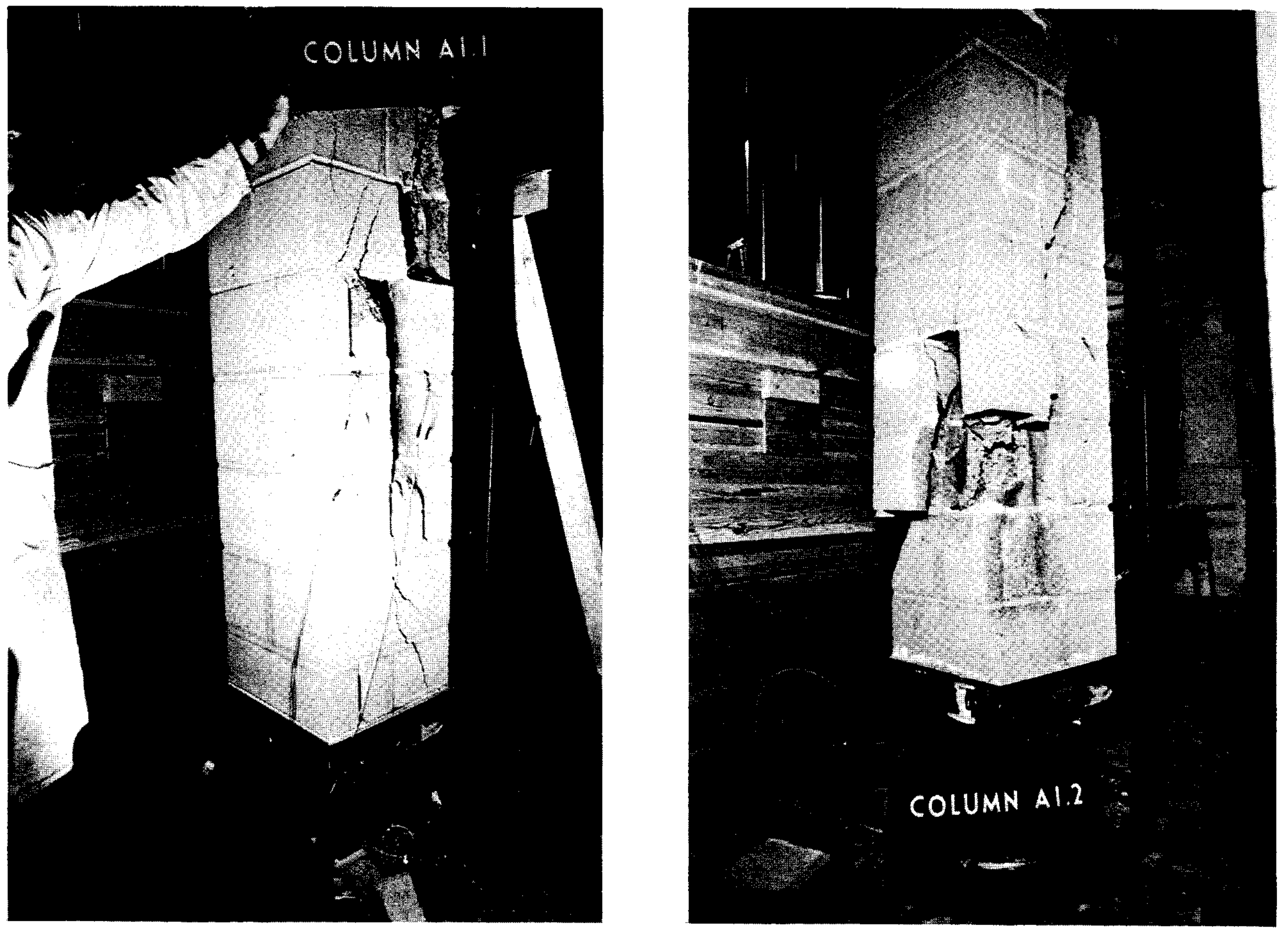


PLATE 4.1 COLUMN GROUP A1 FAILURES

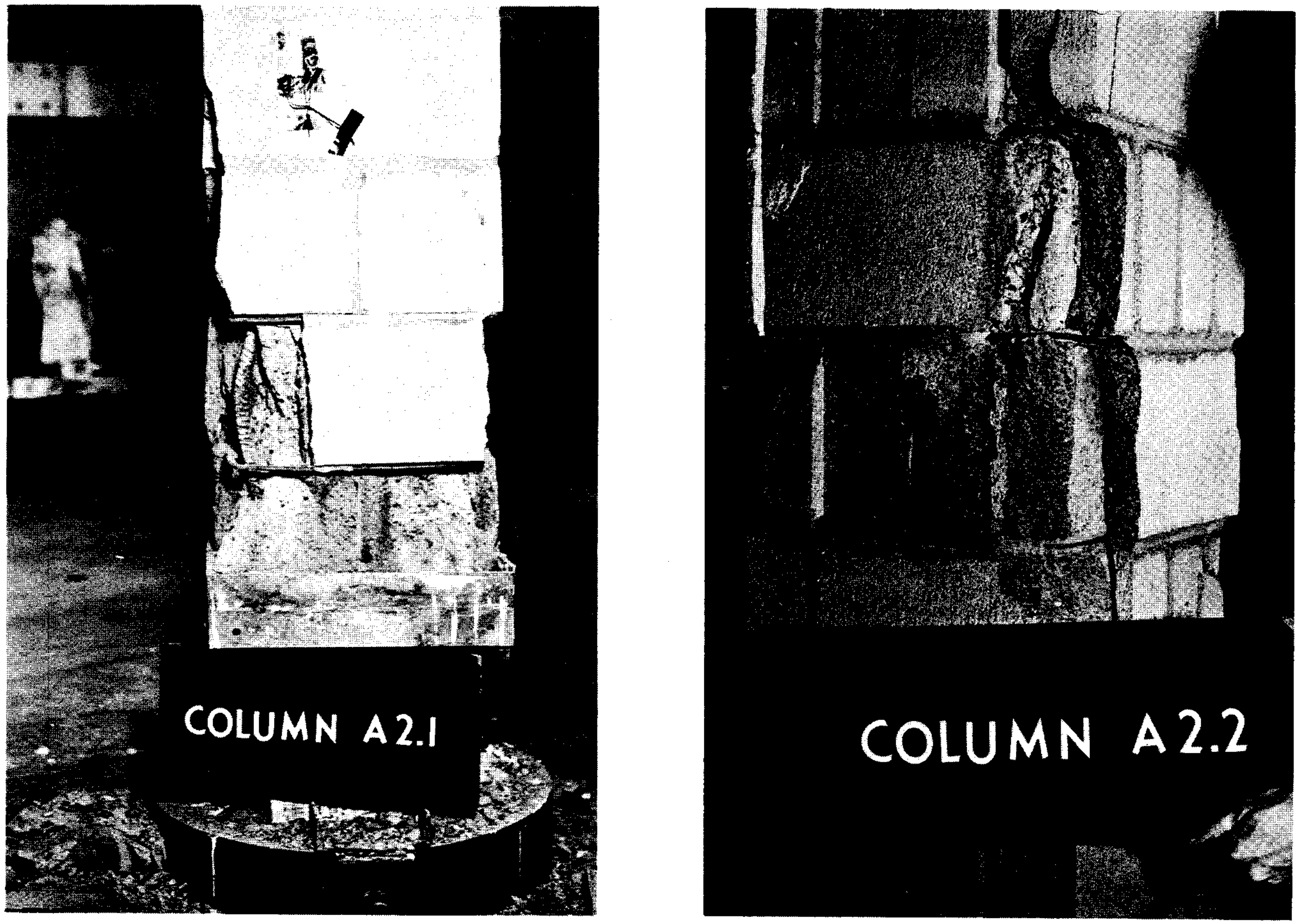


PLATE 4.2 COLUMN GROUP A2 FAILURES

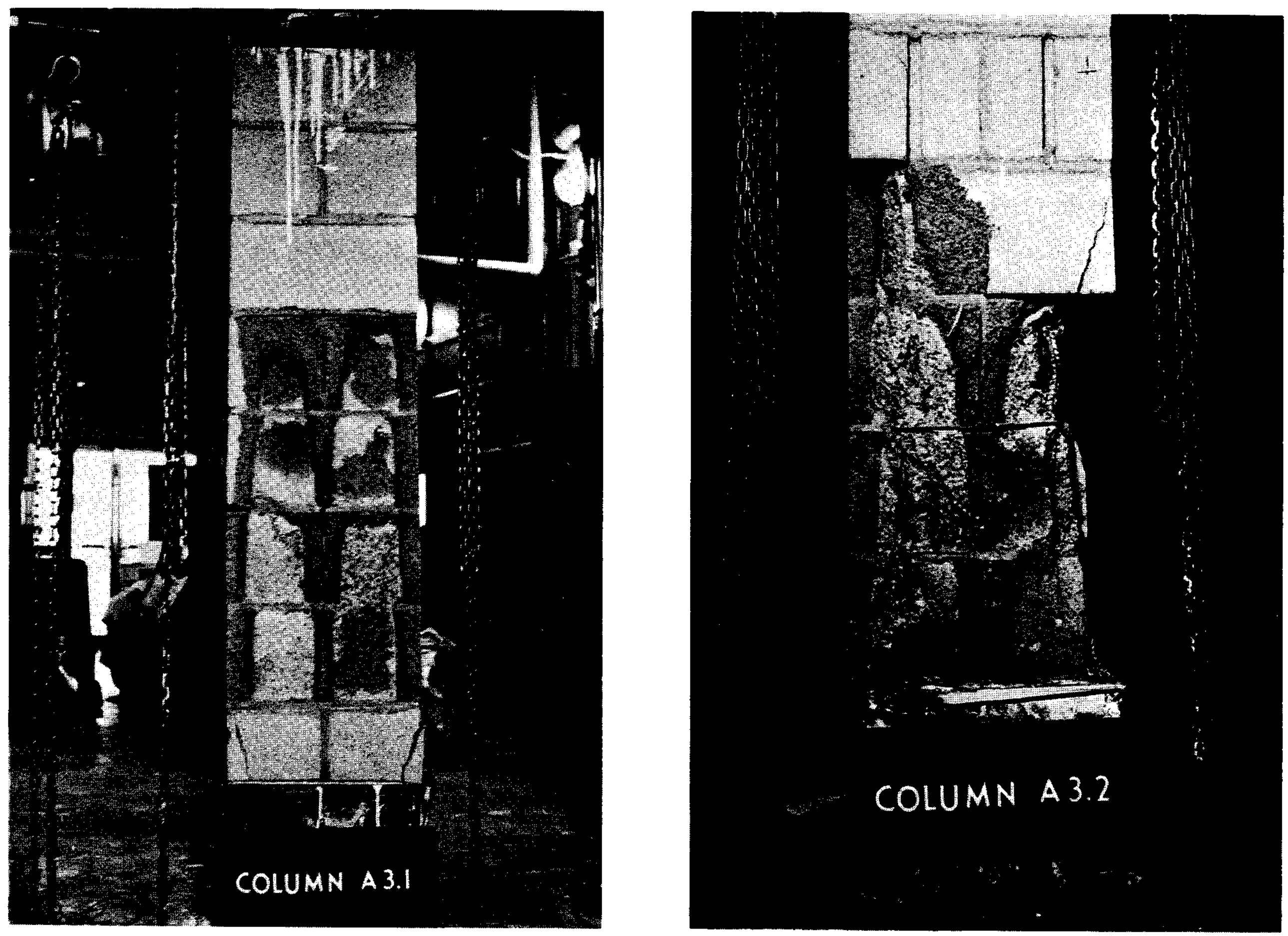


PLATE 4.3 COLUMN GROUP A3 FAILURES

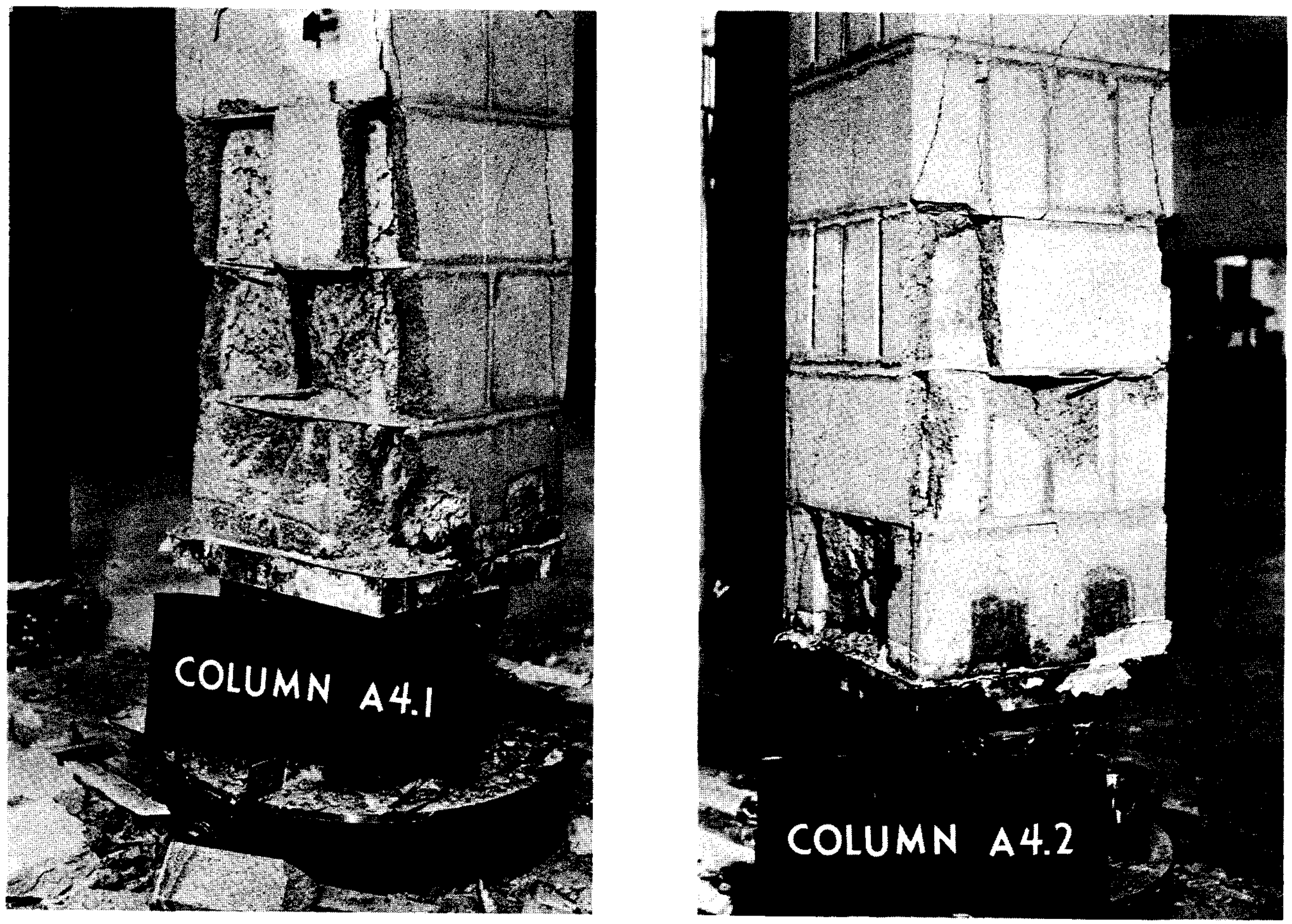
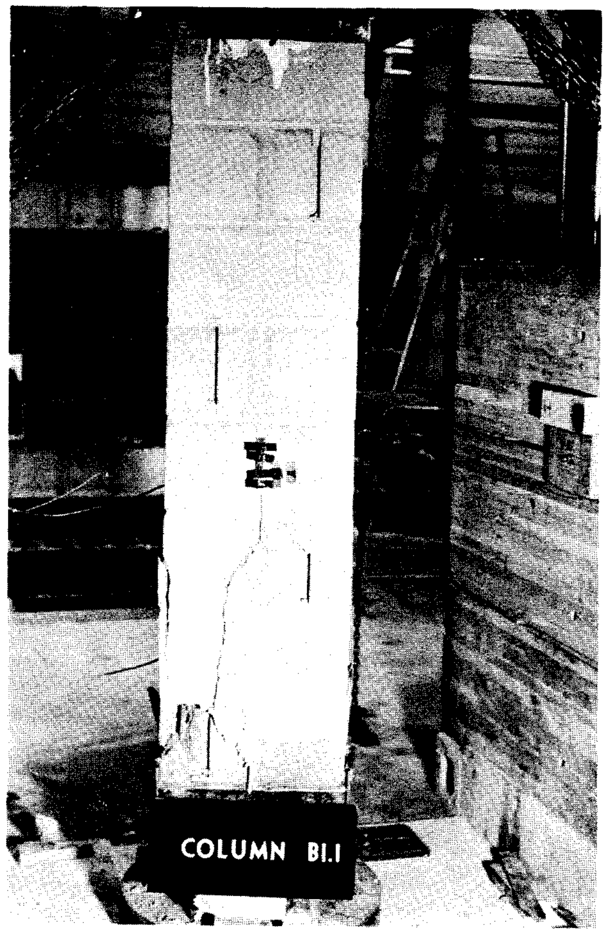


PLATE 4.4 COLUMN GROUP A4 FAILURES



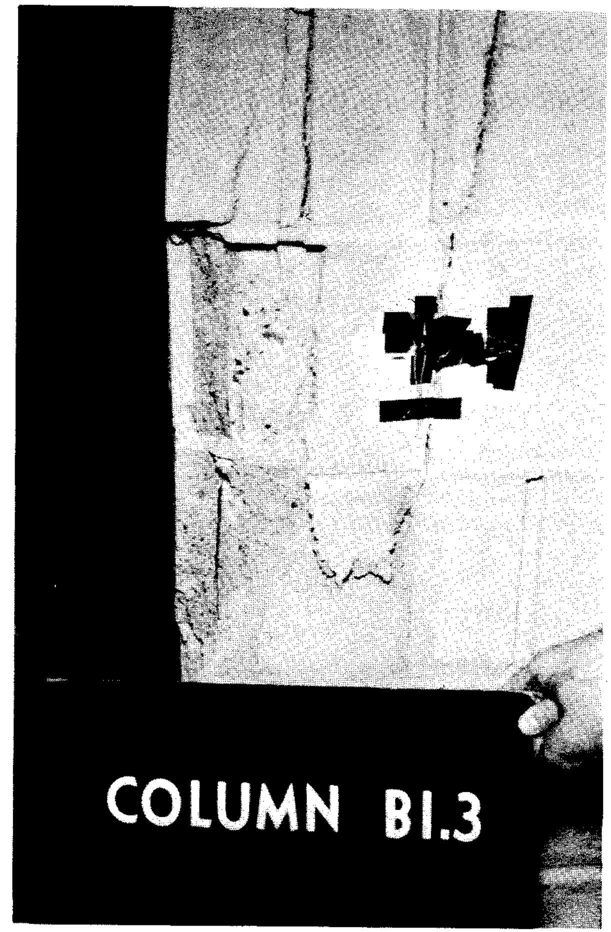
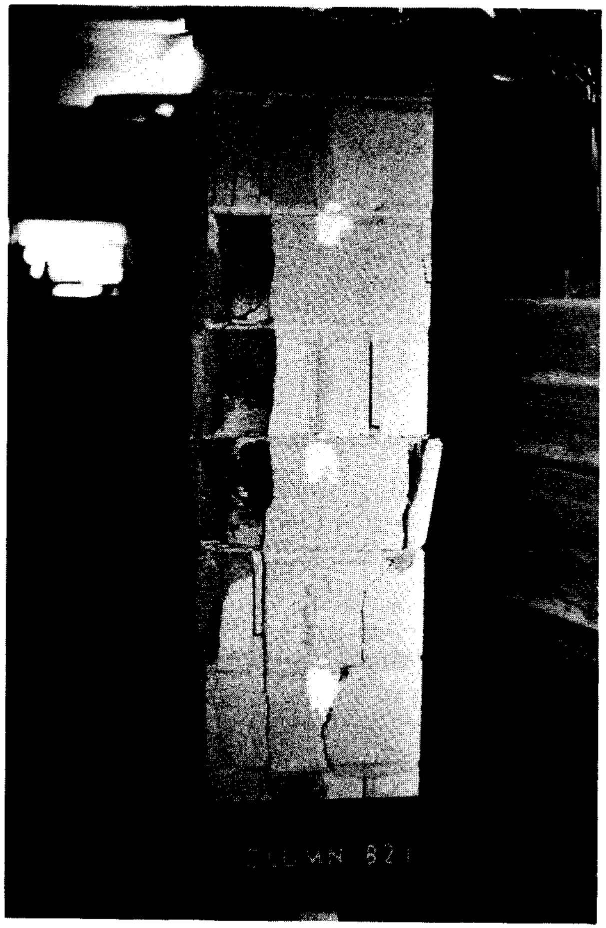


PLATE 4.5 COLUMN GROUP B1 FAILURES



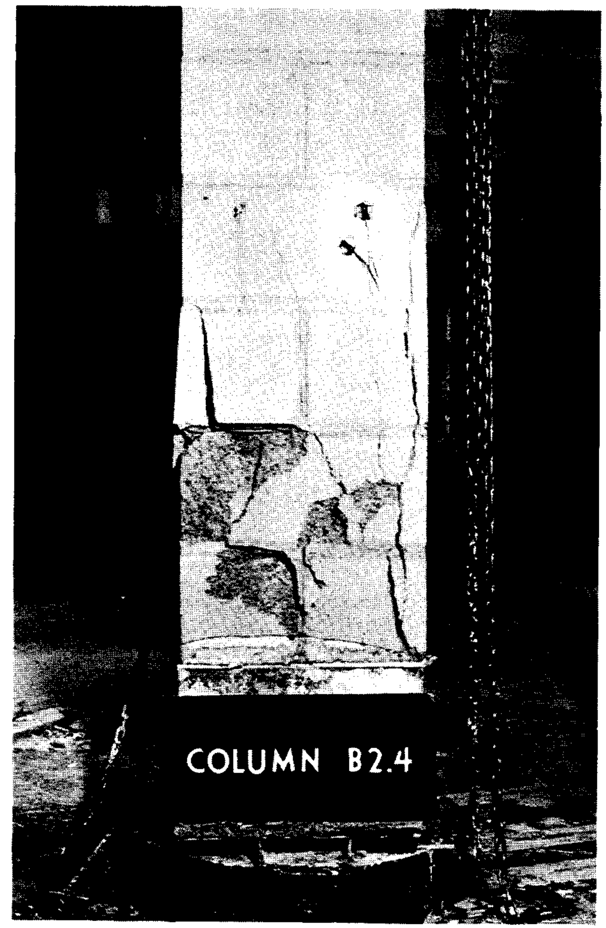
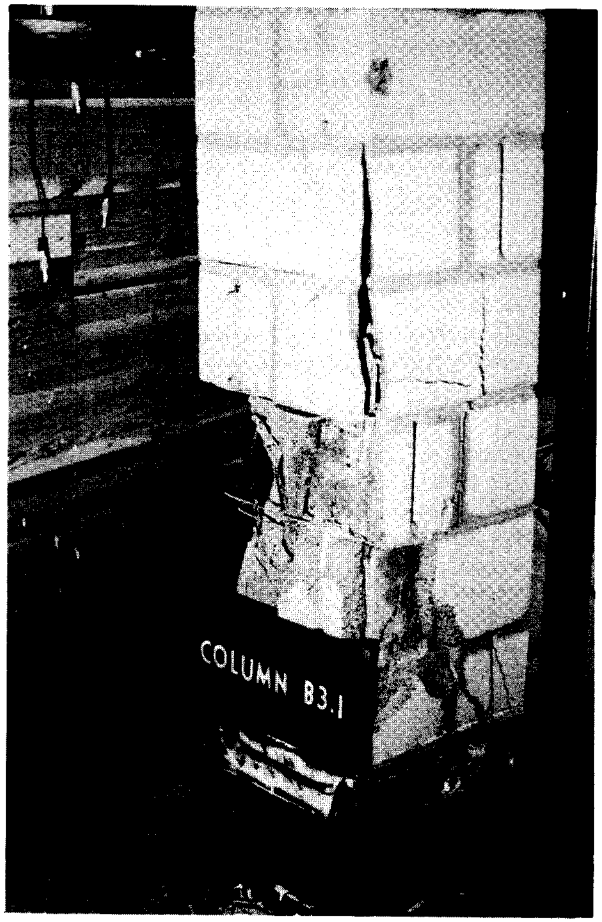


PLATE 4.6 COLUMN GROUP B2 FAILURES



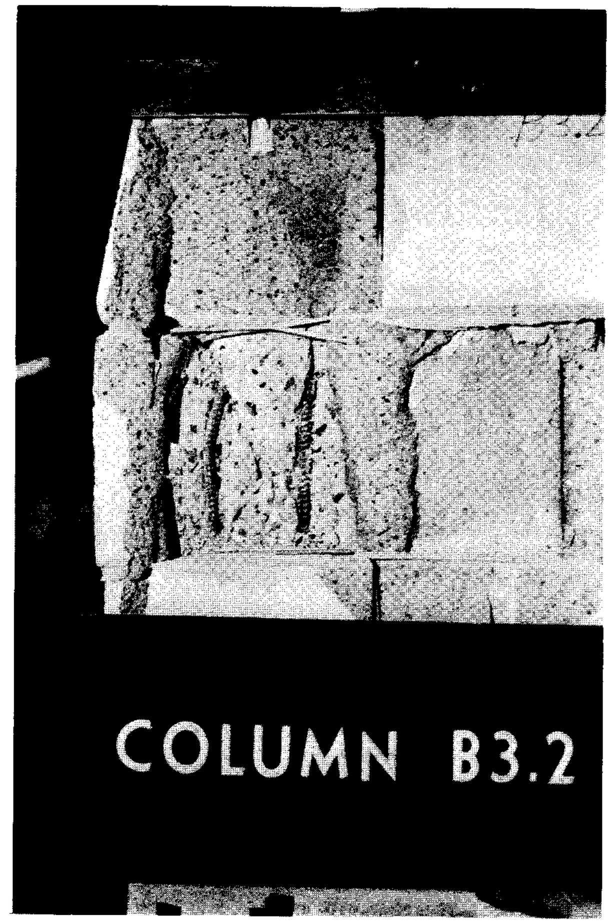
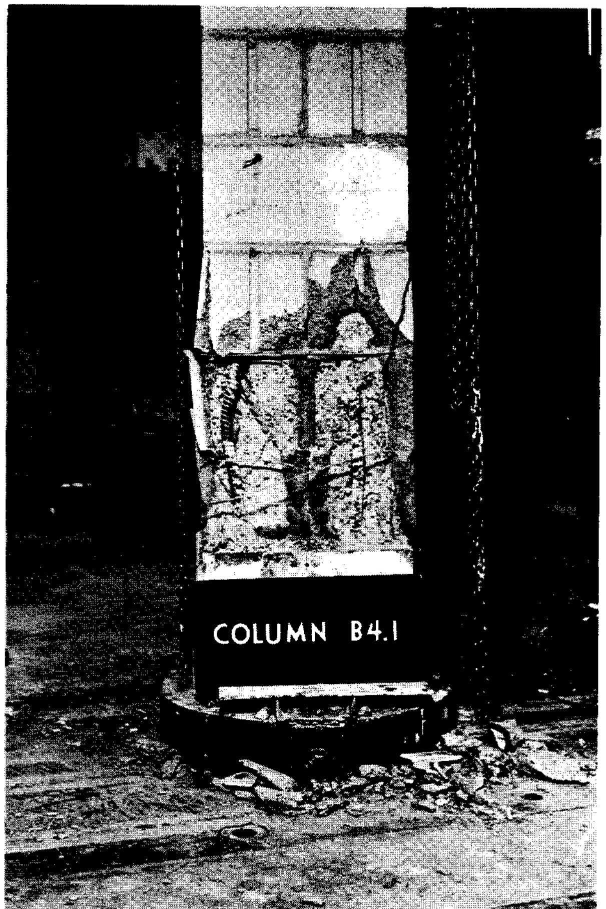


PLATE 4.7 COLUMN GROUP B3 FAILURES



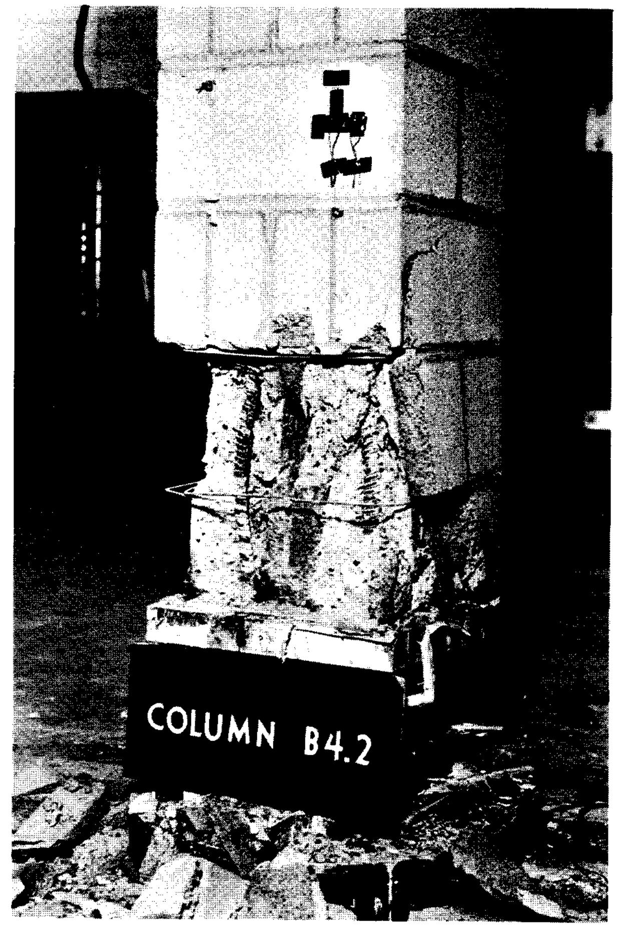


PLATE 4.8 COLUMN GROUP B4 FAILURES

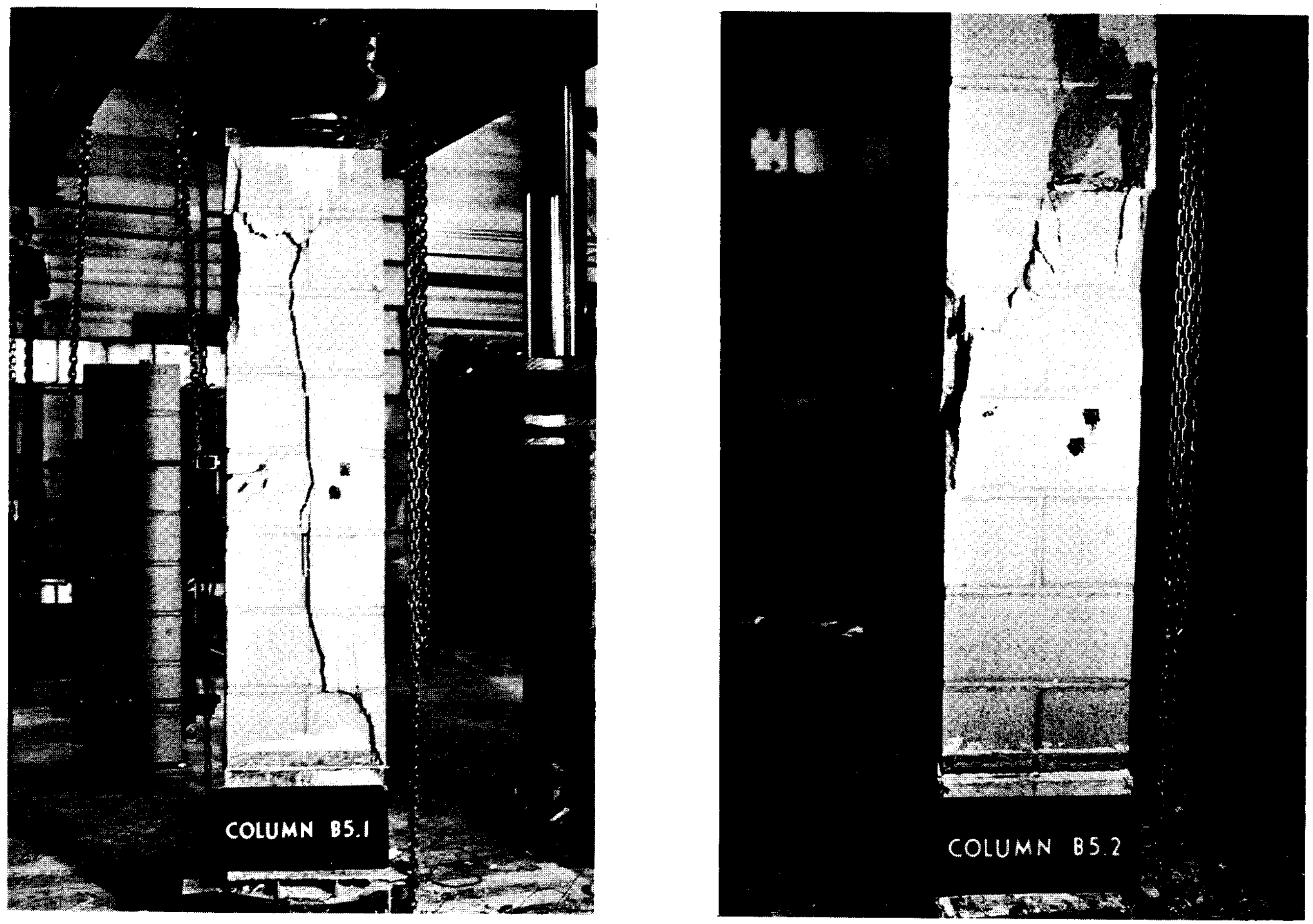


PLATE 4.9 COLUMN GROUP B5 FAILURES

PLATE 4.10 COLUMN GROUP B6 FAILURES

PLATE 4.11 COLUMN GROUP B7 FAILURES

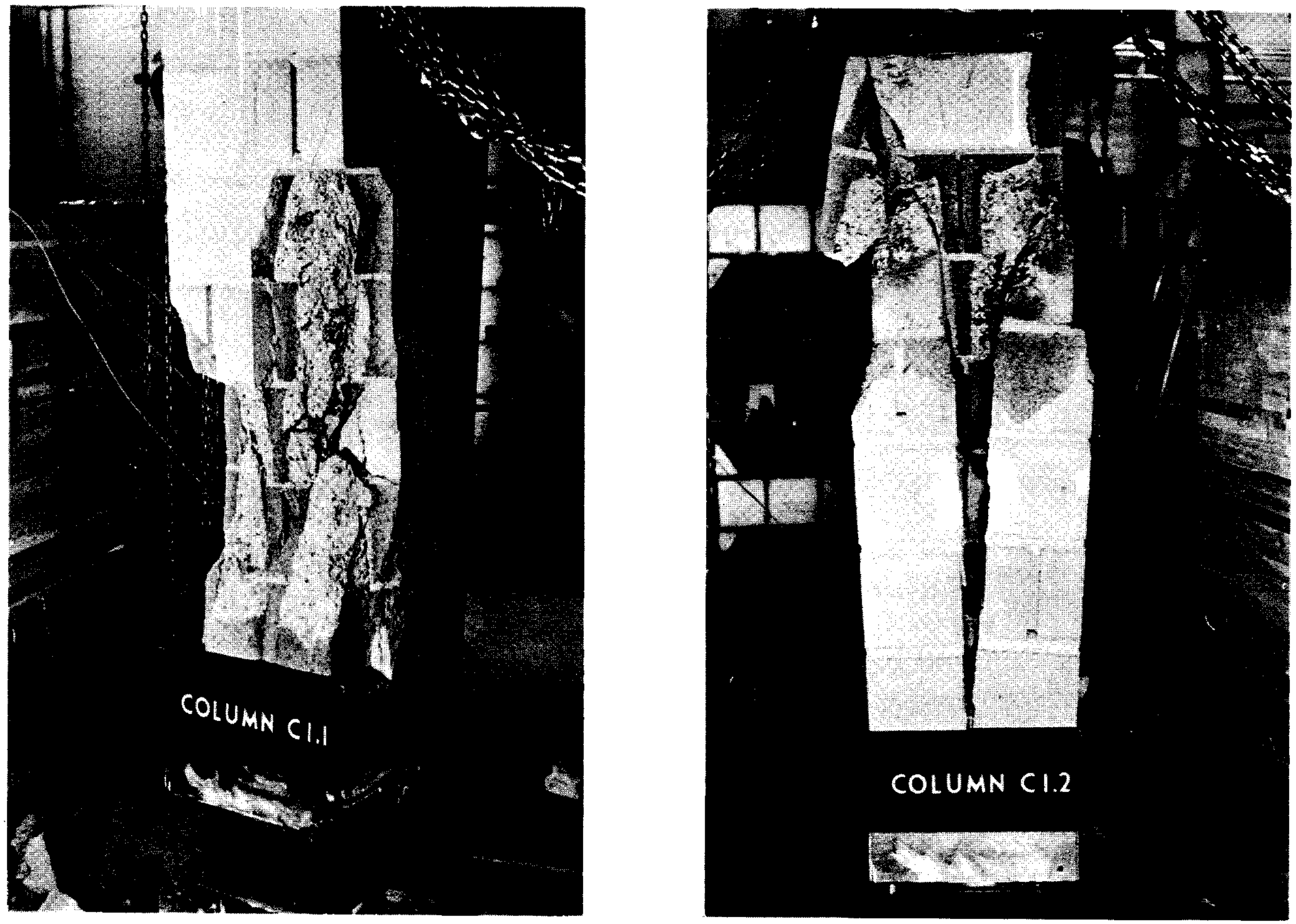
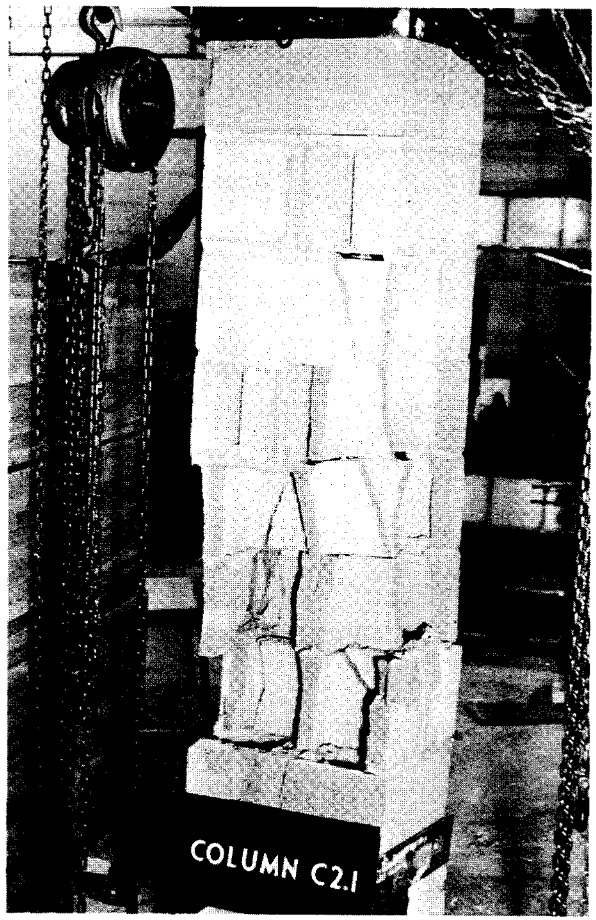


PLATE 4.12 COLUMN GROUP C1 FAILURES



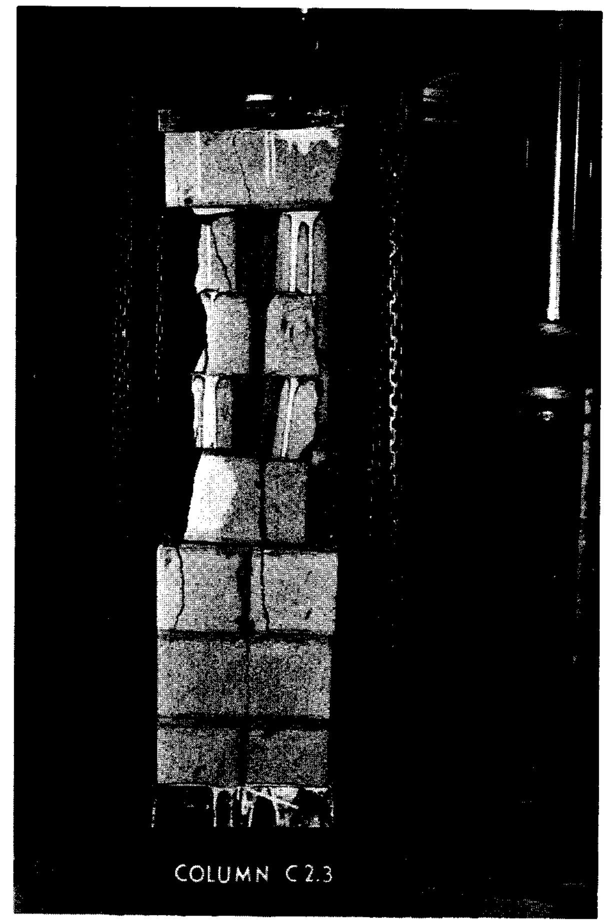


PLATE 4.13 COLUMN GROUP C2 FAILURES

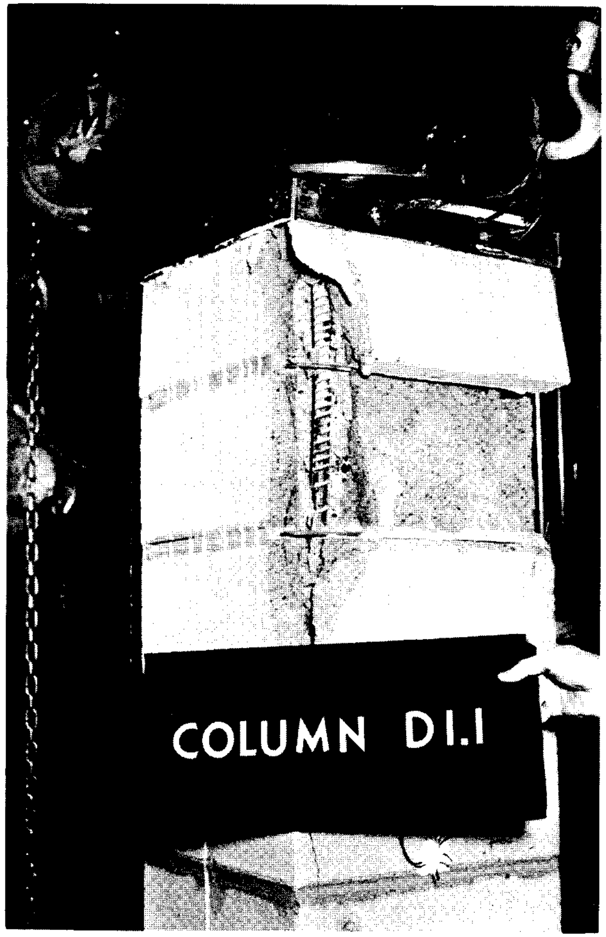


PLATE 4.14 COLUMN GROUP D1 FAILURES

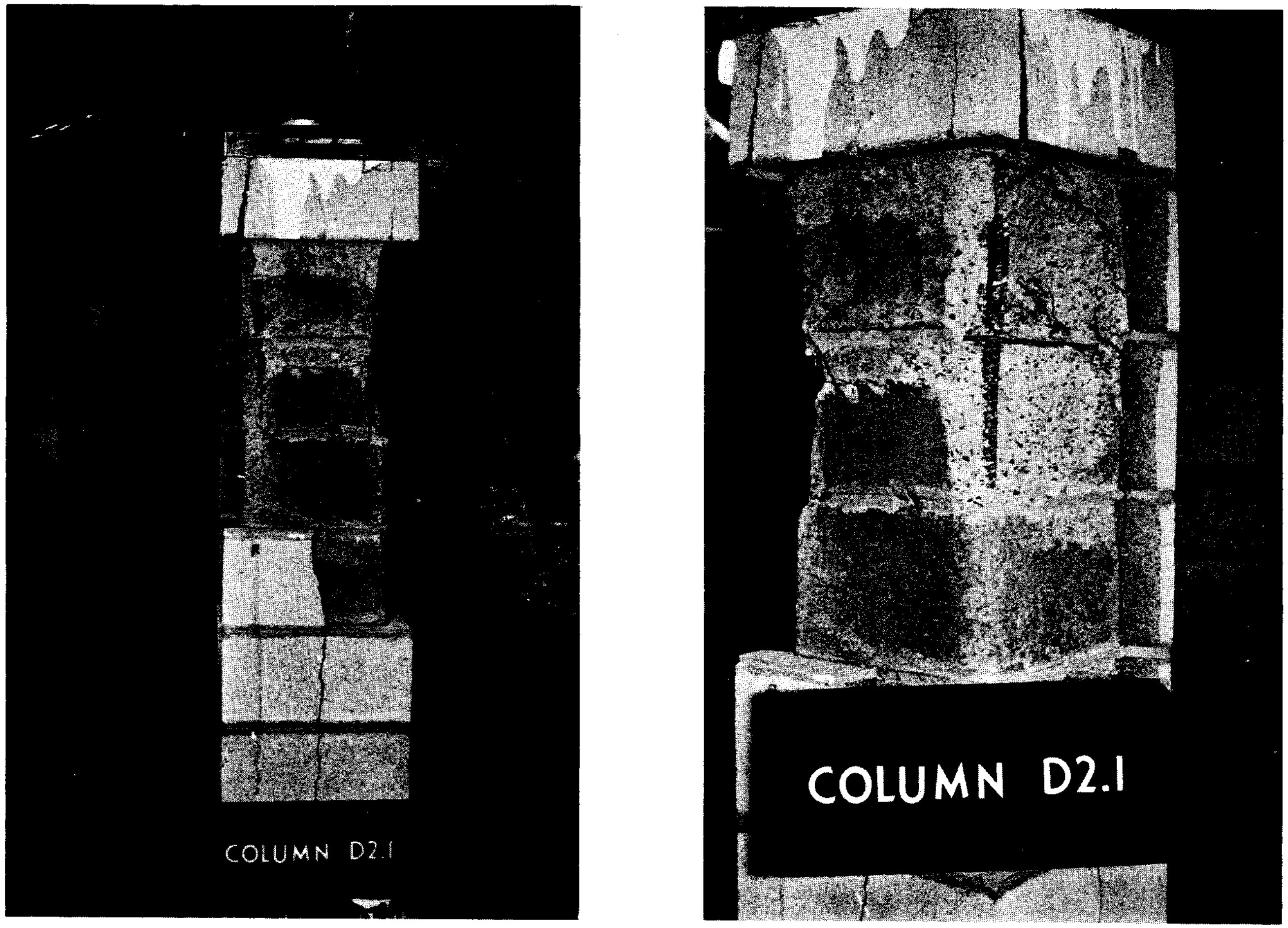


PLATE 4.15 COLUMN D2.1 FAILURE

TABLE 4.2

RESULTS OF WALL UNIT TESTS

Specimen No.	Ult. Load (Kips)	Stress[1] (psi)
1	153.0	2468
2	153.2	2471

1. Based on area of 62 $in.^2$

TABLE 4.3

MORTAR TEST RESULTS

Specimen No.	Ult. Load (Kips)	Stress (psi)
1	2.1	525
2	3.8	950
3	3.6	900
4	3.6	900
5	2.9	725
6	1.8	450
7	1.8	450
8	2.0	500
9	1.6	400
10	2.9	725
11	2.2	550
12[1]	5.9	1475
13[1]	8.2	2050
14[1]	9.5	2375
15[1]	6.2	1550
16[1]	5.8	1450
17[1]	9.1	2275

1. Cured under wet burlap.

TABLE 4.4

MASONRY PRISM TEST RESULTS

Specimen No.	Dimensions (in.)	Ultimate Load (kips)	Stress (psi)
1[2]	16 x 8 x 16	122.4	2246
2[2]	16 x 8 x 16	134.6	2470
3[2]	16 x 8 x 16	150.1	2754
4[2]	16 x 8 x 16	131.0	2404
5[2,4]	16 x 8 x 16	117.1	2149
Aux 1[3]	16 x 16 x 16	236.2	3281
Aux 2[3]	16 x 16 x 16	246.9	3429
Aux 3[1] [3]	16 x 16 x 16	494.4	2025
Aux 4[2]	16 x 8 x 16	153.2	2811
Aux 5[2]	16 x 8 x 16	157.7	2894
Aux 6[1,2]	16 x 8 x 16	229.6	1921
Aux 7[1,2]	16 x 8 x 16	229.6	1921
Aux 8[1,2]	16 x 8 x 16	199.8	1672

1. grouted
2. mortar bed area = 54.5 in.^{2}
3. mortar bed area = 72.0 in.^{2}
4. mortar joint broken

TABLE 4.5

GROUT TEST RESULTS

Specimen No.	Dimensions	Ult. Load (kips)	Stress (psi)
1	4.25 in. sq.	20.5	1135
2	4.25 in. sq.	22.2	1229
3	4.25 in. sq.	19.25	1066
4	4.25 in. sq.	25.6	1417
5	4.0 in. sq.	17.7[1]	2213
6	4.0 in. sq.	20.4[1]	2550
7	4.0 in. sq.	19.9[1]	2488
8	4.0 in. sq.	15.6[1]	1950
9	3.75 in. sq.	18.5	1316
10	3.625 x 3.875 in.	17.6	1253
11	3.75 in. sq.	15.0	1067
12	3.75 in. sq.	14.6	1038
13	3.75 in. sq.	20.6	1465
14	4.0 in. sq.	38.0[1]	2375
15	6 x 12 cylinder	60.5[1]	2140
16	6 x 12 cylinder	85.0[1]	3006
17	6 x 12 cylinder	81.5[1]	2882
18	6 x 12 cylinder	33.8[1,2]	299
19	6 x 12 cylinder	23.0[1,2]	203
Aux. 7-1	23 sq. in.	41.5[3]	1804
Aux. 7-2	23 sq. in.	48.0[3]	2087

1. cured under wet burlap
2. tensile splitting test
3. cores of failed Aux 7 prism

TABLE 4.6

REINFORCEMENT TEST RESULTS

Specimen No.	Grade Mark	Area (in.^{2})	Yield Load (Kips)	Ultimate Load (Kips)	Yield Stress (ksi)	E (ksi) x 10^3
1-2	N/A	0.028	2.39	2.40*	85.36	34.4
1-4	N/A	0.028	2.40	2.66	85.71	30.0
2-1	N/A	0.049	3.60	4.33	73.47	30.6
2-2	N/A	0.049	3.50	4.39	71.43	30.6
3-1	40	0.11	6.10	9.15	55.45	27.3
3-2	40	0.11	6.08	9.20	55.27	29.6
4-2	40	0.20	10.25	15.65	51.25	27.4
5-2	40	0.31	16.75	25.25	54.03	31.9
5-5	60	0.31	19.40	32.45	62.58	28.2
6-2	60	0.465	33.25	55.40	71.50	30.7
6-5	40	0.44	23.00	36.50	52.27	28.7
7-2	40	0.60	32.20	49.20	53.67	29.4
7-5	40	0.60	31.40	48.80	52.33	31.1
8-6	40	0.79	38.10	61.50	48.23	27.0
9-1	N/A	0.017	1.10	1.36	64.71	24.7
9-3	N/A	0.017	1.16	1.34	68.24	28.7

* broke at spot weld.

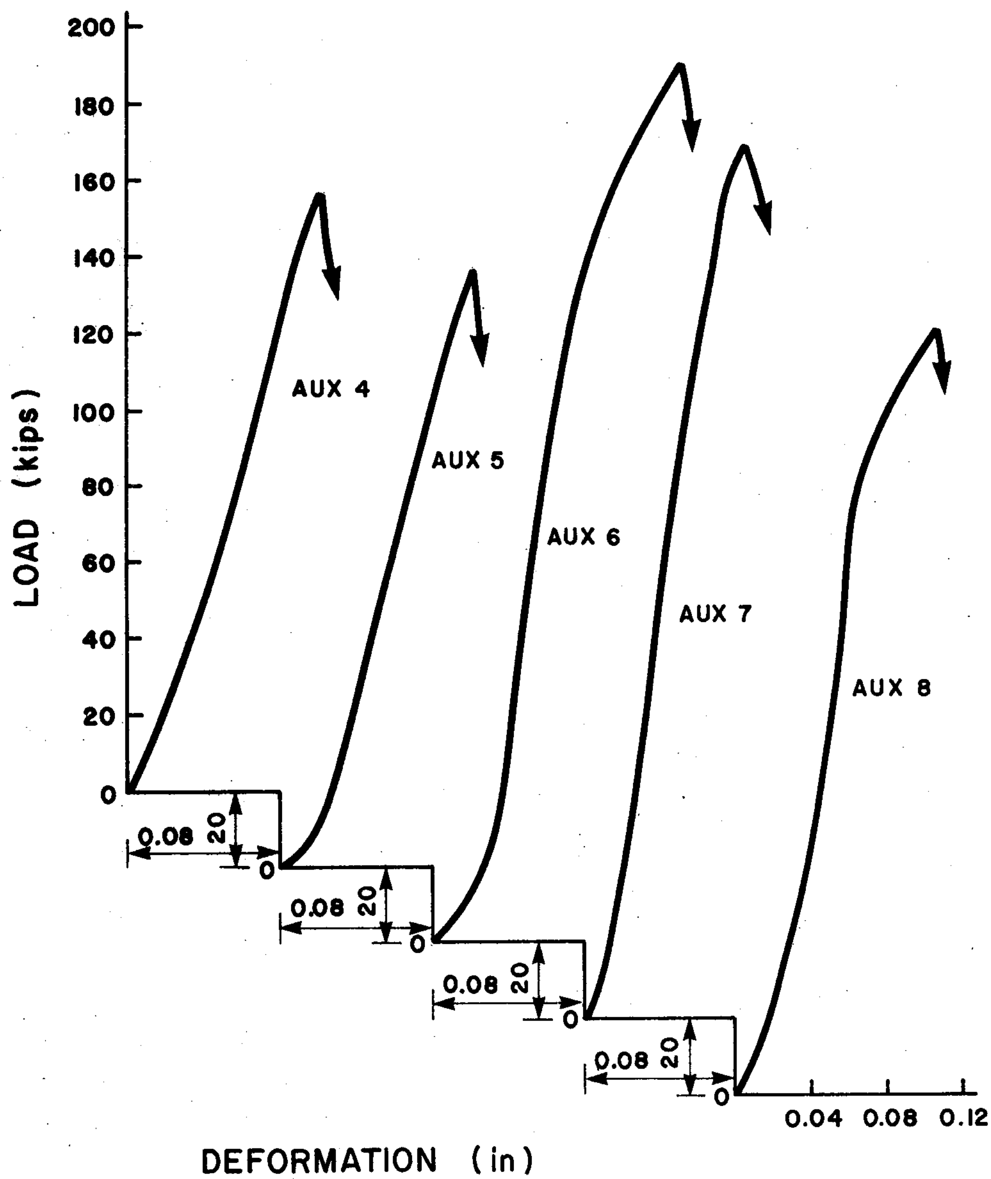


FIGURE 4.6 PRISM TESTS OF WALL UNITS
LOAD VS VERTICAL DEFORMATION

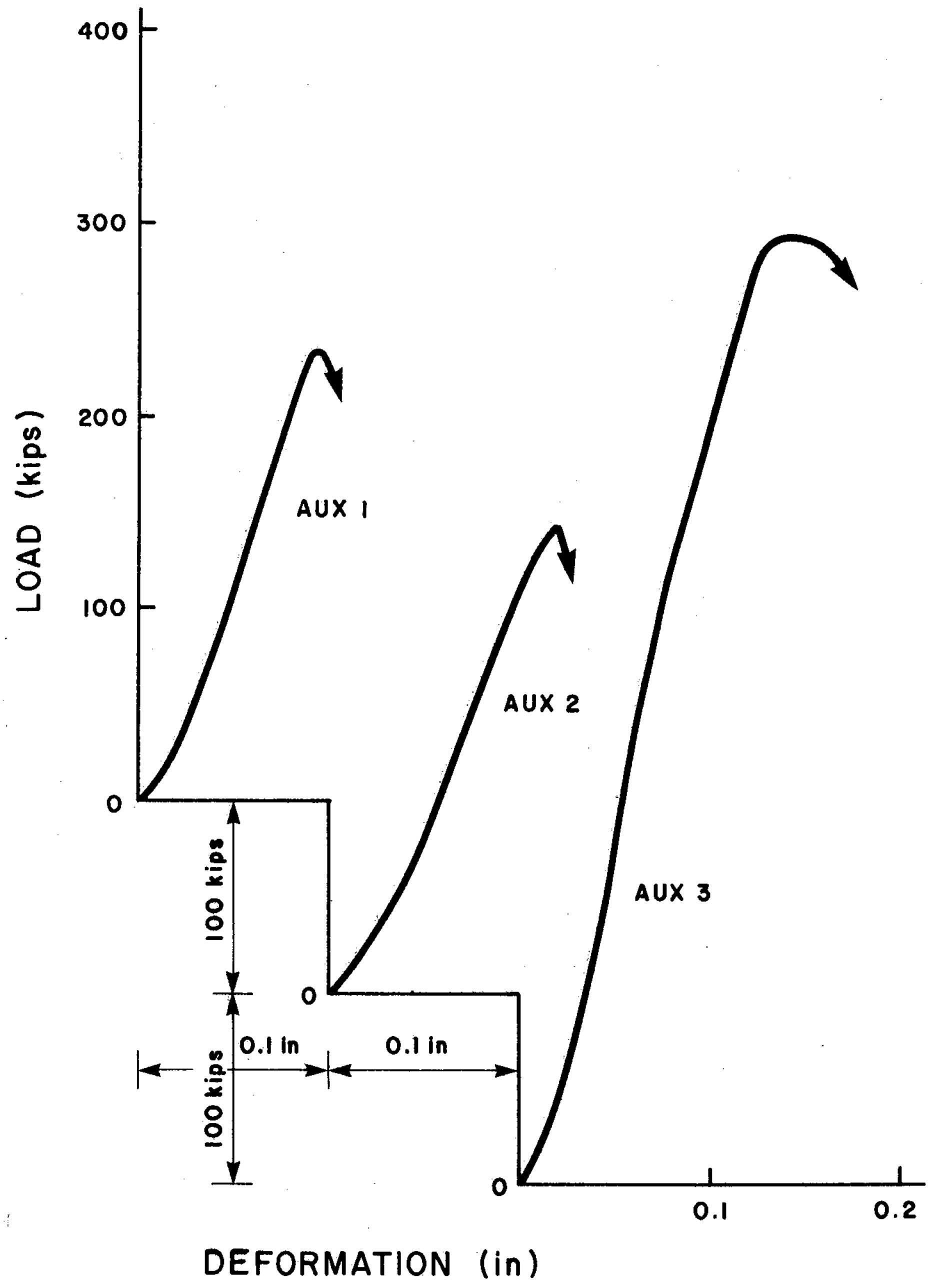


FIGURE 4.7 PRISM TESTS OF SPECIAL UNITS
LOAD VS VERTICAL DEFORMATION

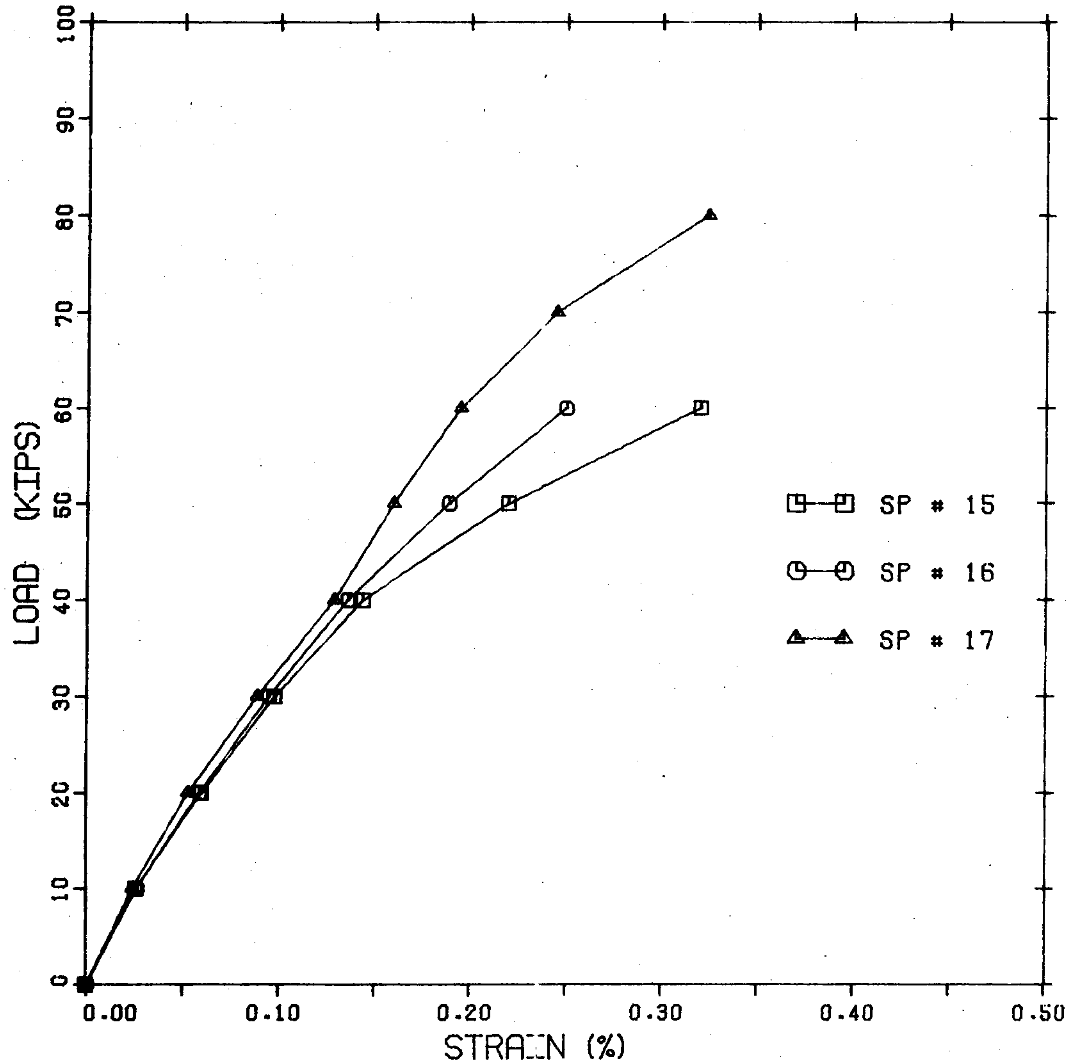


FIGURE 4.8 GROUT CYLINDER TESTS
LOAD VS STRAIN

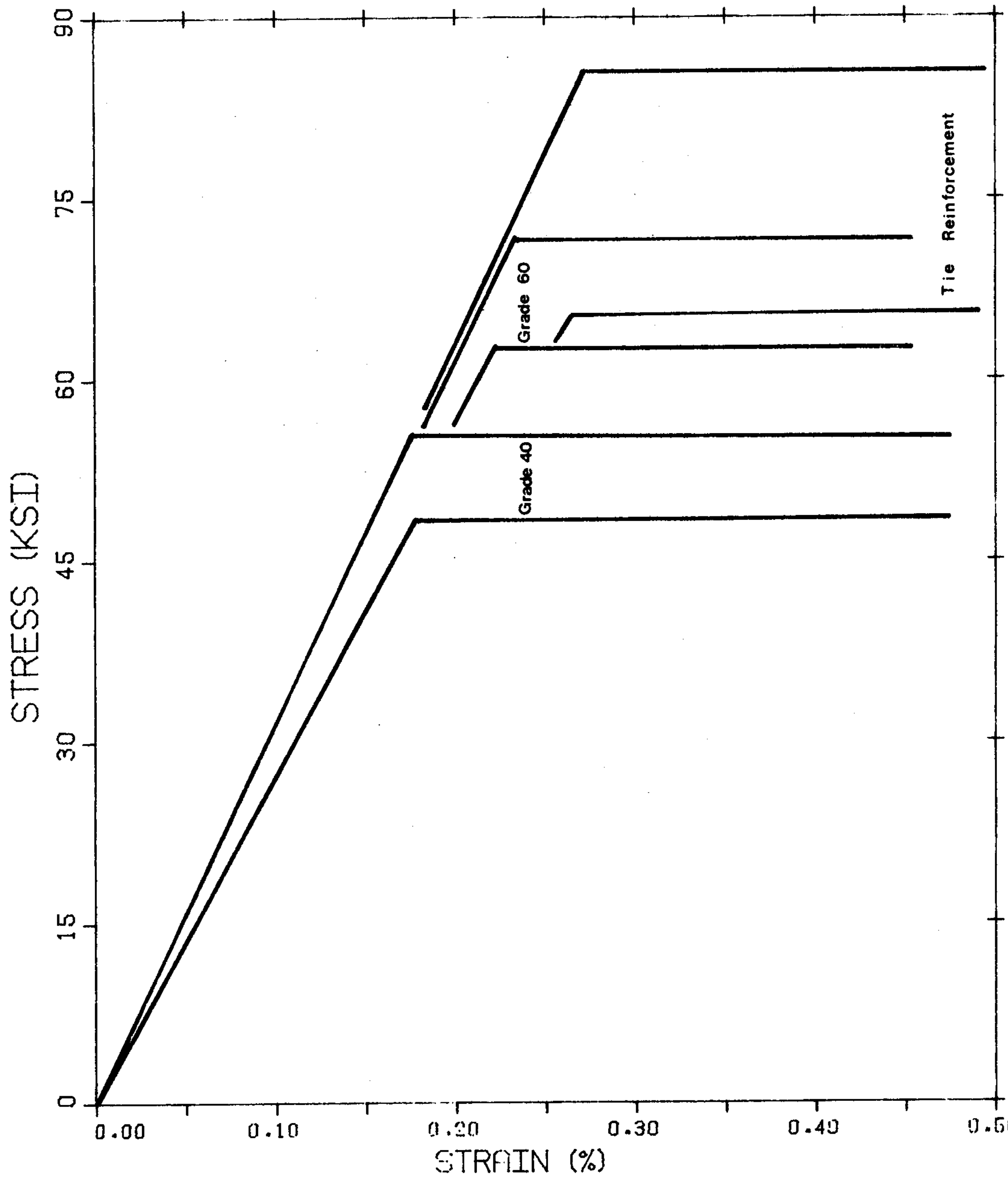


FIGURE 4.9 REINFORCEMENT TESTS
STRESS VS STRAIN

CHAPTER V

DISCUSSION

5.1 Introduction

A discussion of test results to show the effects of both tie and vertical reinforcement detailing on the strength and behavior of reinforced concrete masonry columns is presented in this chapter. Construction procedures which influenced the performance of these columns are also examined.

5.2 General Behavior

Behavior of all the columns was essentially elastic for loads up to approximately 75% of the ultimate load. Small amounts of plastic deformation occurred thereafter due to creep of the masonry.

Although the columns were assumed to be loaded concentrically, small amounts of bending were present. Bending stresses of up to 13% of the axial stresses were detected in the vertical reinforcement of one column while smaller amounts occurred in other columns.

Strains measured in the vertical reinforcement exhibited load-strain relationships similar to the load-deformation relationships of columns. These strains, which were measured at mid-height of the columns, showed slight deviations for different bar locations within the cross section. These small differences were largely due to column out-of-plumbness which averaged less than 1% of the column thickness.

The magnitude of the measured horizontal deflections of the columns also showed a small amount of bending was present. The deflected shape of the column, plotted using these horizontal deflections, showed little or no restraint at the column ends which validated the assumption of pinned ends for the columns.

The amount of bending and end restraint was, in general, small enough to be ignored in this investigation. Therefore, columns were assumed to be loaded concentrically through pinned ends.

Column failure occurred in one of the following three modes:

a) overall vertical splitting of the column,
b) simultaneous crushing of the masonry and buckling of the vertical reinforcement within the tie spacing,
c) same as b) but buckling was not confined to within the tie spacing.

Failures of most columns with ties in the mortar joint began when vertical cracks were initiated above and below the tie reinforcement. These cracks then propagated along the interface of the grout and the masonry unit causing spalling of the outside masonry shell. This localized failure was followed immediately by ultimate failure of the column.

Horizontal and vertical strains on the surface of a concrete masonry unit, located at mid-height of the column, were monitored during testing of most columns. Plots of this data show the vertical load-strain relationship to be similar to the load-deformation relationship of the column. The horizontal strains, however, reached a maximum and then decreased slightly just before the ultimate load was reached. This behavior may be attributed to the breaking of the bond between the mortar and the masonry unit which caused a relaxation in lateral stress at the mid-height of the block where the strain measurements were taken.

The modulus of elasticity of the masonry is computed in Table 5.1 for 26 reinforced concrete masonry columns. To exclude creep strains from the analysis a load approximately equal to 75% of the ultimate load was used in the computation. The average modulus of elasticity was found to be approximately 800 times the masonry prism strength which agrees with previous findings.[16]

The vertical deformation at ultimate load was an indication of the ductility of the columns. By

TABLE 5.1 MODULUS OF ELASTICITY COMPUTATION

Column	Reinforcement Vertical	Reinforcement Tie Type[1]	0.75 p_u (kips)	ε_s (x 10^{-3})	$p_s = \varepsilon_s E_s A_s$ (kips)	$p_m = 0.75\ p_u - p_s$ (kips)	$f_m = p_m/A_m$ (ksi)	ε_m (x 10^{-3})	$\varepsilon_s/\varepsilon_m$ at 0.75 p_u	$\varepsilon_s/\varepsilon_m$ at p_u	$E_m = f_m/\varepsilon_s$ (ksi)
A1.1	4-#6	N/A	381.6	0.9737	49.2	332.4	1.37	N/A	N/A	N/A	1409
A1.2	4-#6	N/A	400.0	1.0704	54.1	345.9	1.43	N/A	N/A	N/A	1333
A1.3	4-#6	N/A	400.3	1.0199	51.5	348.8	1.44	N/A	N/A	N/A	1411
A2.1	4-#6	3	421.1	0.9605	48.5	372.6	1.54	0.8697	1.10	1.13	1600
A2.2	4-#6	3	480.8	1.2960	65.5	415.3	1.71	0.8901	1.46	1.27	1322
A3.1	4-#6	4	440.3	1.0825	54.7	385.6	1.59	0.8500	1.27	1.27	1470
A3.2	4-#6	4	440.7	1.0108	51.1	389.6	1.61	0.7634	1.32	1.25	1590
A4.1	4-#6	2	440.4	1.2525	63.3	377.1	1.56	0.9560	1.31	1.30	1242
A4.2	4-#6	2	440.6	1.1755	59.4	381.2	1.57	1.1040	1.06	1.02	1338
B2.1	4-#6	1	441.2	1.0032	50.7	390.5	1.61	N/A	N/A	N/A	1606
B2.4	4-#6	1	440.4	1.0810	54.6	385.8	1.59	1.4020	0.77	1.00	1472
B2.5	4-#6	1	400.4	1.1235	56.8	343.6	1.42	1.1030	1.02	0.91	1262
B3.1	16-#3	1	480.4	1.0316	51.7	428.7	1.77	N/A	N/A	N/A	1714
B3.2	16-#3	1	480.2	1.0306	51.7	428.5	1.77	N/A	N/A	N/A	1715
B3.3	16-#3	1	480.4	1.1256	56.5	423.9	1.75	N/A	N/A	N/A	1554
B4.1	4-#7	1	441.0	1.0249	75.3	365.7	1.51	0.8366	1.23	1.22	1476
B4.2	4-#7	1	400.2	0.7489	55.0	345.2	1.43	0.7599	0.99	0.93	1907
B5.1	12-#4	1	520.6	1.0990	72.3	448.3	1.85	0.9847	1.12	0.97	1688
B5.2	12-#4	1	480.6	1.1595	76.2	404.4	1.67	0.9089	1.28	1.11	1443
B6.1	4-#8	1	400.5	0.8631	73.6	326.9	1.35	0.8425	1.02	1.02	1572
B6.2	4-#8	1	480.1	1.0925	93.2	386.9	1.61	0.7888	1.39	1.43	1470
B7.1	4(#5+#6)[2]	1	440.6	0.9405	86.6	354.0	1.47	1.0720	0.88	0.83	1562
B7.2	4(#5+#6)[2]	1	440.7	1.0082	92.8	347.9	1.44	0.8654	1.17	1.06	1432
D1.1	4-#6[2]	3	441.0	0.9852	56.3	384.7	1.59	N/A	N/A	N/A	1612
D1.2	4-#6[2]	3	441.0	1.2071	68.9	372.1	1.54	N/A	N/A	N/A	1272
D2.1	4-#6[2]	4	440.7	1.2896	73.6	367.1	1.52	1.0880	1.19	1.27	1175

Footnotes to Table 5.1

1. 1 - 0.1483 in. diameter; 2 - 0.1875 in. diameter; 3 & 40.25 in. diameter; 1 - 3 in. mortar joint; 4 - in contact with vertical reinforcement.
2. Grade 60 vertical reinforcement; all others are Grade 40

Mean E_m = 1486 ksi
Std. Dev. = 172 ksi

comparing the amount of vertical deformation in columns with different reinforcement details, a relative measure of the ductility of the columns was obtained.

Plain columns had the lowest vertical deformation at ultimate load which averaged 0.112 inches. This deformation was essentially doubled, to an average of 0.220 inches, with the addition of grout in the columns. The inclusion of 0.1483 inch diameter ties in the mortar joint increased this by an additional 3.6% to an average deformation of 0.228 inches. When only vertical reinforcement was added to the grouted column, i.e. 4-#6 bars, the average ultimate vertical deformation increased by 15.9% to 0.255 inches. When the #9 wire, 3/16 inch and 0.25 inch diameter ties were added the increase in average vertical deformation at ultimate was 4.0%, 15.5% and 22.1%, respectively.

There was no definite trend in the values of the ultimate vertical deformation when larger amounts of vertical reinforcement were used in the columns reinforced with the #9 wire ties. The average of these values for all 14 such columns was 0.282 inches with a standard deviation of 0.032 inches.

5.3 Effects of Tie Reinforcement Detailing

The effect of tie diameter on the ultimate masonry strength is shown in Table 5.2 for columns with ties

TABLE 5.2 EFFECT OF TIE DIAMETER ON MASONRY STRENGTH

Column Group	Tie Reinforcement Diameter (in.)	Tie Reinforcement Stiffness (EI/L kip-in)	Column	P_s (kips)	P_m (kips)	f_m (ksi)	Avg. f_m (ksi)
C1	N/A	N/A	C1.1 C1.2 C1.3	N/A	468.3 412.4 450.3	1.92 1.69 1.84	1.82
B1	0.1483	0.047	B1.1 B1.2 B1.3	N/A	470.4 471.0 454.1	1.93 1.93 1.86	1.91
A1	N/A	N/A	A1.1 A1.2 A1.3	73.0 92.0 91.5	433.0 433.0 446.9	1.79 1.79 1.84	1.81
B2	0.1483	0.047	B2.1 B2.4 B2.5	79.6 92.0 87.0	486.9 497.2 458.2	2.01 2.05 1.89	1.98
A4	0.1875	0.138	A4.1 A4.2	92.0 85.0	497.3 490.8	2.05 2.02	2.04
A2	0.25	0.435	A2.1 A2.2	72.2 92.0	484.1 534.1	2.00 2.20	2.10

located in the mortar joint. Columns with no tie reinforcement and/or columns with no vertical reinforcement are included for comparison. The computational data from Table 5.2 is presented graphically in Figure 5.1 which shows an increase in ultimate masonry strength with increased tie diameter. The increase in ultimate masonry strength for tied columns over columns with no ties averaged 7.4%, 12.6% and 15.9% for tie diameters of #9 wire, 3/16 and 0.25 inches respectively.

The failure mode of columns with no tie reinforcement was Type (a). Plates 4.1 and 4.12 illustrate the extensive vertical cracking, typical of this failure mode. The amount of vertical cracking diminished for columns reinforced by the #9 wire ties as shown in Plate 4.5. This decrease became more significant with the addition of vertical reinforcement as shown in Plate 4.6. As larger diameter tie reinforcement was used the amount of cracking decreased proportionately as illustrated in Plates 4.4 and 4.2 for columns with 3/16 inch and 0.25 inch diameter ties respectively. Plate 4.2 shows a Type (b) failure mode which was typical for the three column groups which had 4-#6 bars as vertical reinforcement.

The amount of confinement afforded by the 0.25 inch diameter tie reinforcement is quantitatively shown in the plots of Vertical Load vs. Tie Strain of which

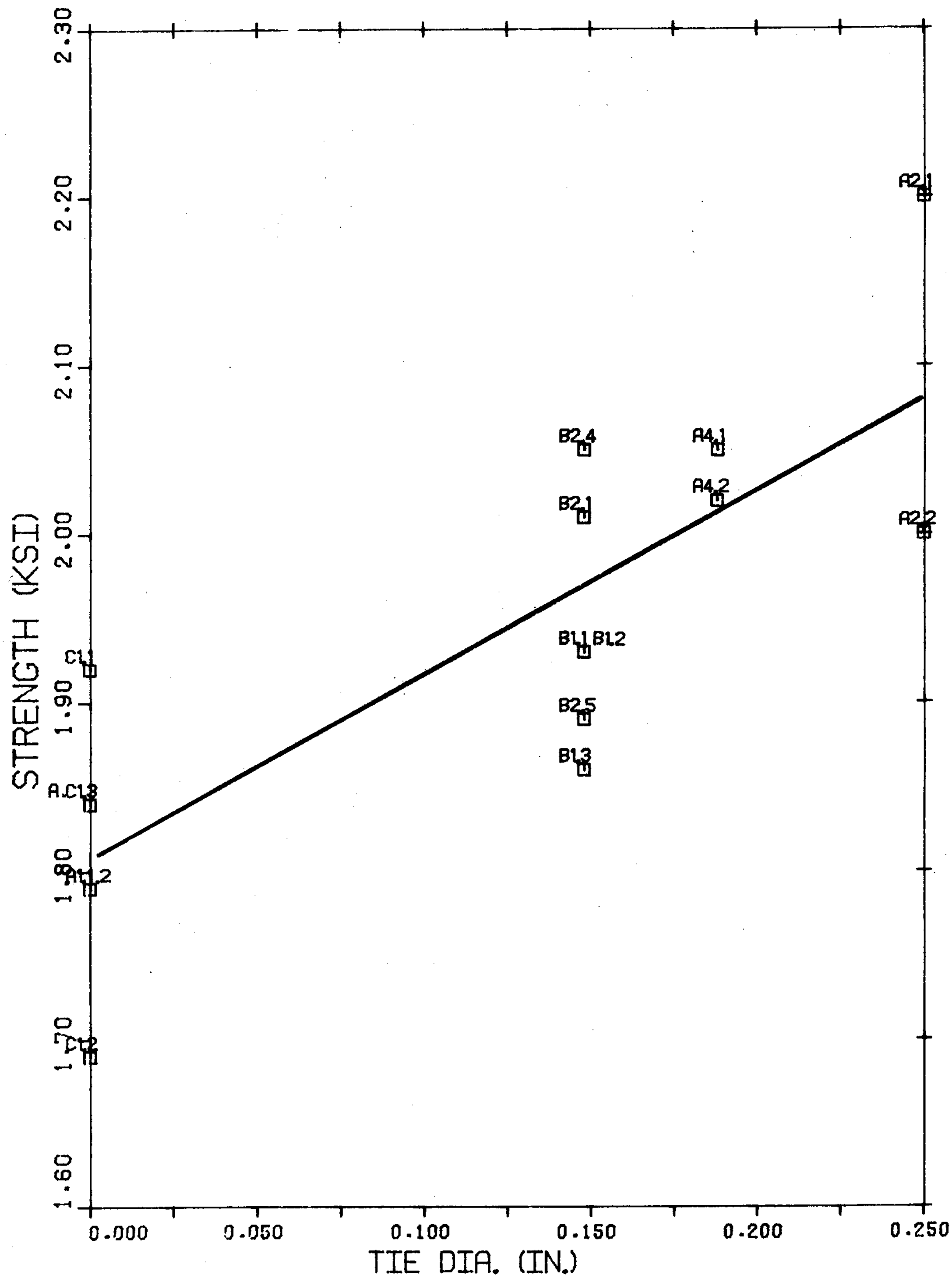


FIGURE 5.1 ULTIMATE MASONRY STRENGTH VS TIE DIAMETER

Figure 4.4 is typical. The maximum recorded tie strain, which occurred in column D2.1, indicated a tensile stress of 24 ksi in the tie at failure. The small tack welds broke just prior to failure of this column.

A small initial compressive strain is apparent for a 0.25 inch tie used in column D1.1. This was due to the location of the strain gage on the tie as shown in Figure 3.2. As the masonry inside the tie expanded, the sides of the tie were bent outward. Such action would initially cause small compressive stresses near the corners of the tie on the outside surface where the gage was located. The constraining effect of the tie was then dependent, to a degree, on the bending stiffness of the tie.

The average masonry stress at ultimate load is plotted against tie stiffness in Figure 5.2 for values taken from Table 5.2 for columns with vertical reinforcement. The curve drawn through the data points show a non-linear relationship between ultimate masonry stress and tie stiffness. The curve of Figure 5.3 represents the average percentage increase in ultimate masonry strength for tie stiffness defined as AE/L. The analysis of this complex relationship is beyond the scope of this study and, in any case, would not be truly representative with the limited data available.

The effects of tie location within the cross-section are shown in Table 5.3. There is no significant difference

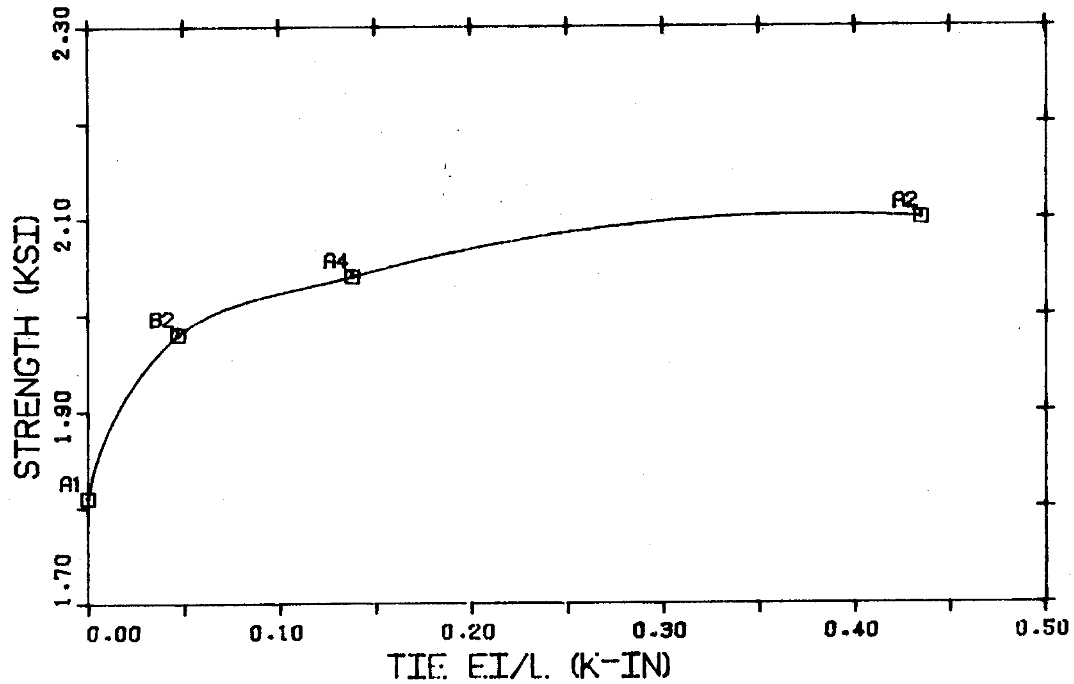


FIGURE 5.2 ULTIMATE MASONRY STRESS VS TIE STIFFNESS

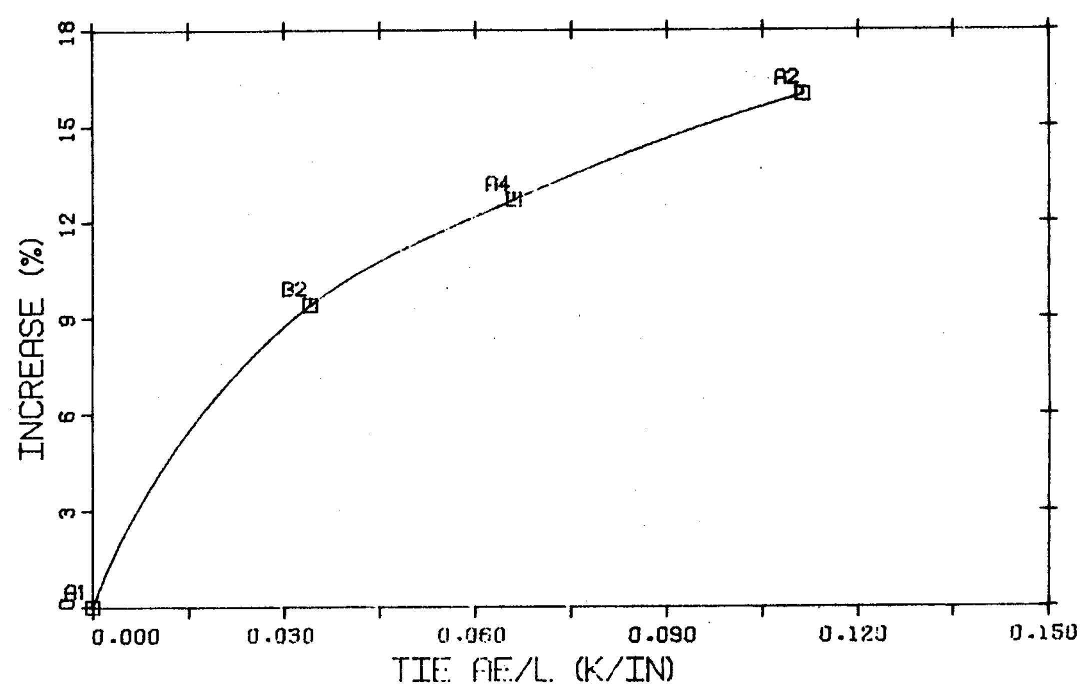


FIGURE 5.3 INCREASE IN MASONRY STRENGTH VS TIE STIFFNESS

TABLE 5.3 EFFECT OF TIE LOCATION

Column	Tie Location[1]	P_s (kips)	P_m (kips)	f_m (ksi)	Avg. f_m (ksi)	Avg. Deformation (in)	Avg. Tie Strain
A2.1	1	72.2	484.1	2.00	2.10	0.3138	0.02933
A2.2		92.0	534.1	2.20			
A3.1	2	86.5	523.7	2.16	2.16	0.3079	0.01459
A3.2		76.6	523.0	2.16			
D1.1	1	87.8	513.3	2.12	2.01	0.2685	0.04345
D1.2		101.0	459.2	1.89			
D2.1	2	133.0	437.2	1.80	1.80	0.2700	0.03036

1. Location 1 in mortar joint; 2 - in contact with vertical reinforcement.

between values related to the two tie locations although tie strains were, on the average, larger for the ties located in the mortar joint. The general trend for the average masonry strength to increase for increasing tie stiffness is exhibited for columns A2 and A3. The ties in column D2.1 were not tack welded which allowed them to slip an amount such that the lap bend was reduced from 90° to approximately 60°. This accounts for the lower masonry strength at ultimate load for this column. The masonry strength is based on the gross area of the masonry which was approximately 242 square inches. Although the 13.5 inch square ties enclosed an area twice as great as the 9.5 inch square ties, both enclosed virtually the same grouted area, therefore, the same gross area for both ties was used in the strength computations.

The difference between the vertical strain in the masonry at the location of the vertical reinforcement and the strain on the surface of the columns may also be used as an indication of the relative amount of constraint afforded by different diameters and locations of ties. This difference in the strains at ultimate is shown by the ratio $\varepsilon_s/\varepsilon_m$ in Table 5.1. For columns reinforced with the #9 wire ties, located in the mortar joint, this ratio had an average value of 1.05. The use of 3/16 inch and 0.25 inch diameter ties, in the same location, showed average increases in $\varepsilon_s/\varepsilon_m$ of 10.7% and

14.5% respectively, over the value of 1.05. For column A3, which had 0.25 inch diameter ties located in contact with the vertical reinforcement, an average increase of 20.2% was found.

5.4 Effects of Vertical Reinforcement Detailing

The relationship between the amount of vertical reinforcement provided and the ultimate load is shown in Figure 5.4 for columns with 4 vertical bars. The regression analysis performed on this data indicates that the vertical reinforcement contributed its full yield strength to the strength of the column. In Table 5.4 a comparison is made between the computed ultimate stresses in the masonry and reinforcement and their prism or yield strengths respectively for columns constructed of wall units. This comparison shows the average stress in vertical reinforcement for details using #7 and #8 size bars and Grade 60 reinforcement did not reach the specified yield stress. The columns in which the vertical reinforcement did not reach the specified yield strength failed in mode (c), i.e. buckling of the vertical reinforcement was unrestrained by the ties.

An attempt was made to determine the effect on the ultimate masonry strength of the size and spacing of tie reinforcement relative to the vertical bar diameter.

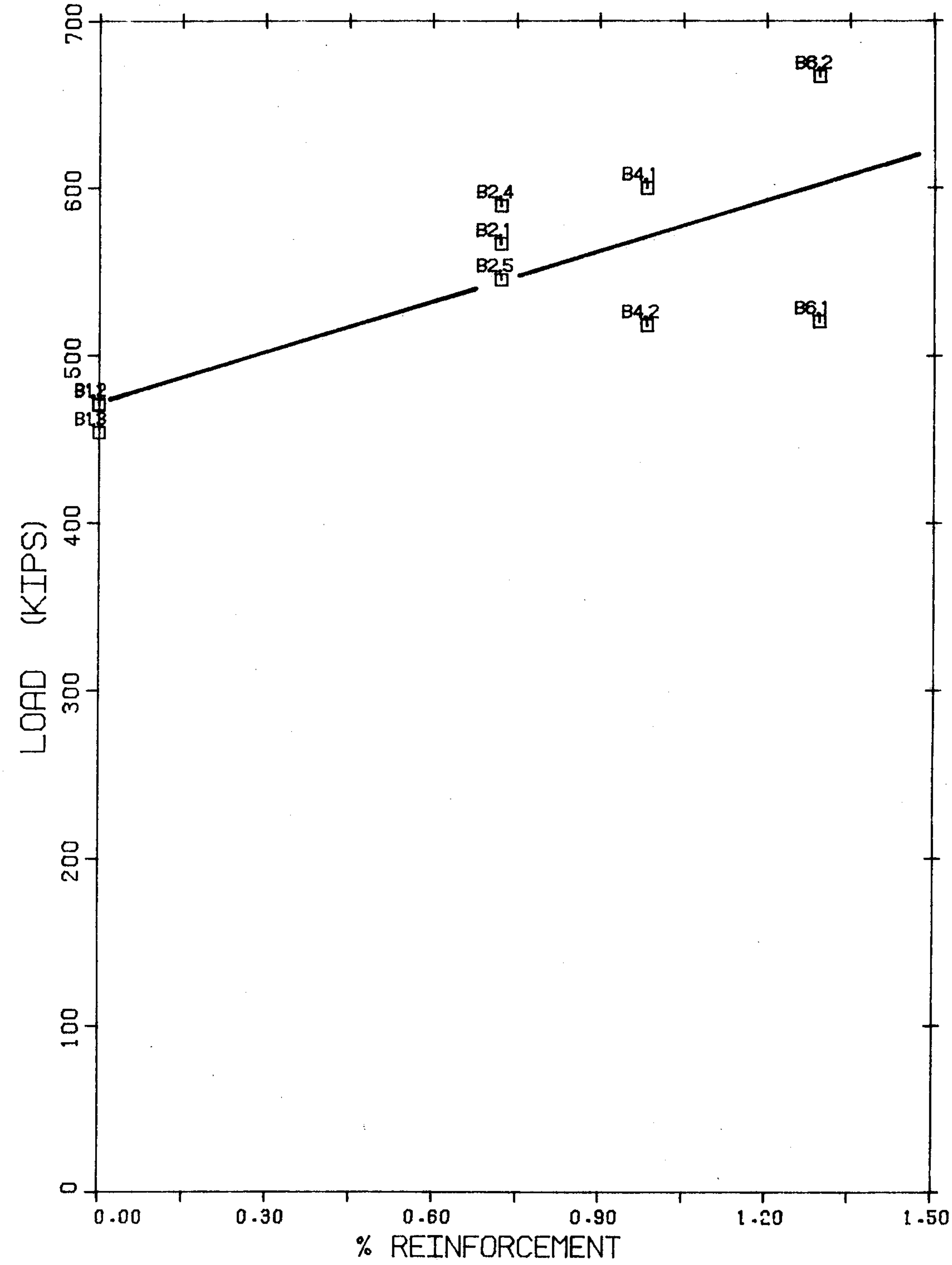


FIGURE 5.4 REINFORCEMENT PERCENTAGE VS

ULTIMATE LOAD

TABLE 5.4 COMPARISON OF COMPUTED AND SPECIFIED STRESSES

Column	Detail	Tie Sp./Bar Dia.	Tie Dia./Bar Dia.	P_s (kips)	f_s/f_y	Avg. f_s/f_y	P_m (kips)	f_m/f'_m
B2.1 B2.4 B2.5	4-#6	10.7	0.20	78.9 83.8 86.0	1.12 1.19 1.22	1.18	487.6 505.4 459.2	1.09 1.13 1.02
B3.1 B3.2 B3.3	16-#3	21.3	0.40	81.9 81.4 87.4	1.16 1.16 1.24	1.19	548.1 543.6 542.0	1.22 1.21 1.21
B4.1 B4.2	4-#7	9.1	0.17	100.4 69.1	1.05 0.72	0.89	499.6 449.0	1.12 1.00
B5.1 B5.2	12-#4	16.0	0.30	105.8 107.6	1.10 1.12	1.11	574.4 513.0	1.28 1.15
B6.1 B6.2	4-#8	8.0	0.15	103.9 144.8	0.82 1.15	0.99	416.6 521.9	0.94 1.17
B7.1 B7.2	4(#5+#6) Grade 60	12.8	0.24	129.8 133.9	0.69 0.76	0.73	463.0 471.3	1.04 1.06

However, the results of this analysis did not explain the large differences in ultimate masonry strength which occurred for different vertical bar sizes.

The average ratio of ultimate masonry strength to prism strength is plotted against bar diameter in Figure 5.5. The curve fitted through the data points suggests an optimum vertical reinforcing bar diameter for Grade 40 bars with #9 ASWG wire ties spaced at 8 inches.

5.5 Other Considerations

This investigation was carried out mainly for columns constructed of wall units. Face shell mortar was used for ease of comparison with columns constructed of special units. As shown in Plate 5.1 the grout was able to penetrate and fully fill the horizontal space between courses but was unable to fill the vertical space between masonry units. These inherent vertical planes of weakness contributed to the vertical splitting of the columns when loaded.

When cleanout openings were not provided at the base of columns, mortar droppings could not be removed. The effect of not removing this mortar can be seen in Figure 5.6 for columns B2.2 and B2.3 where a decrease in ultimate strength of 14% is noted.

The 0.25 inch diameter ties were not galvanized as were the ties made from masonry wall reinforcement.

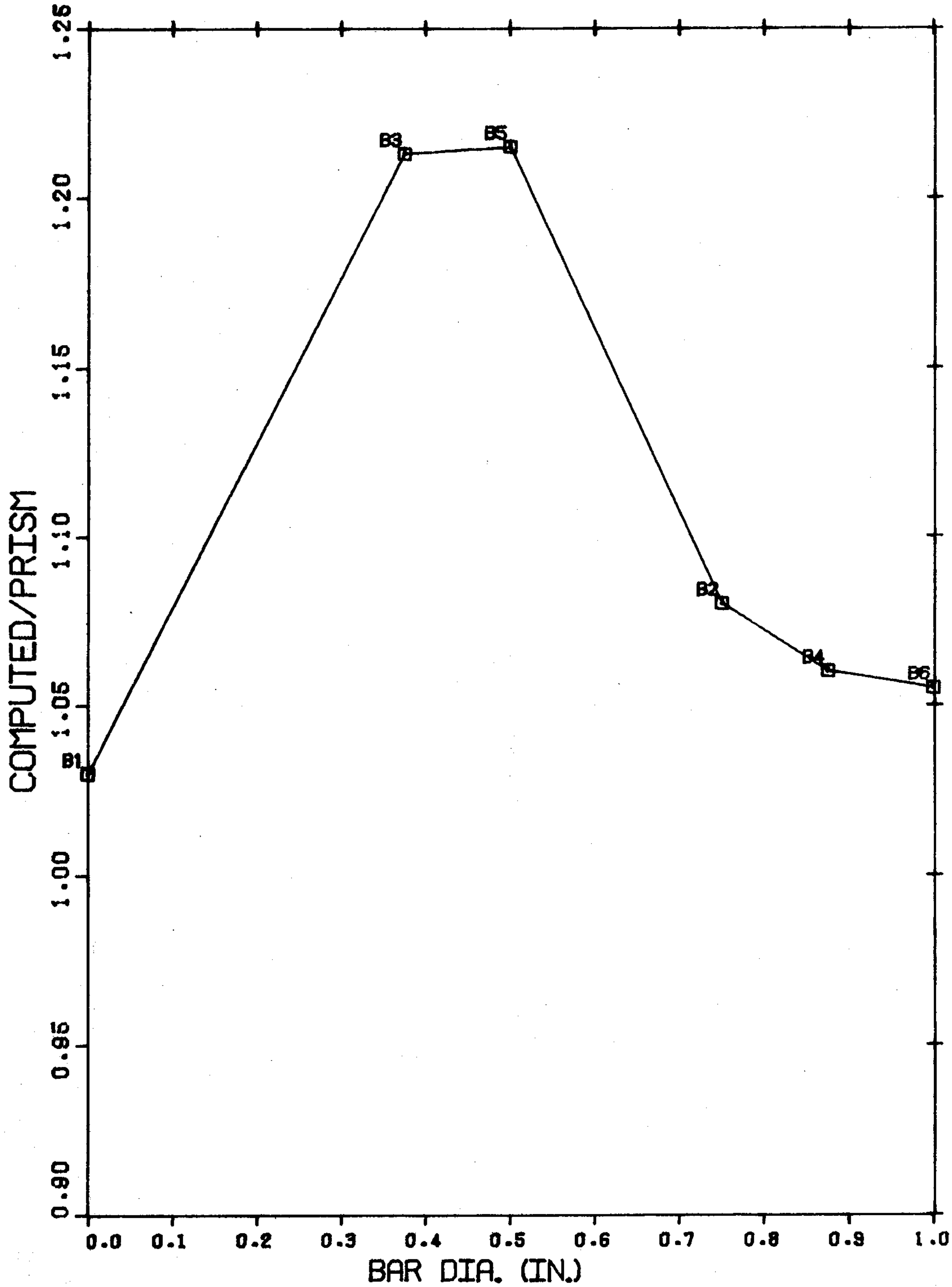


FIGURE 5.5 RATIO OF ULTIMATE MASONRY STRENGTH TO PRISM STRENGTH VS BAR DIAMETER

PLATE 5.1 FAILURE OF COLUMN BY VERTICAL SPLITTING

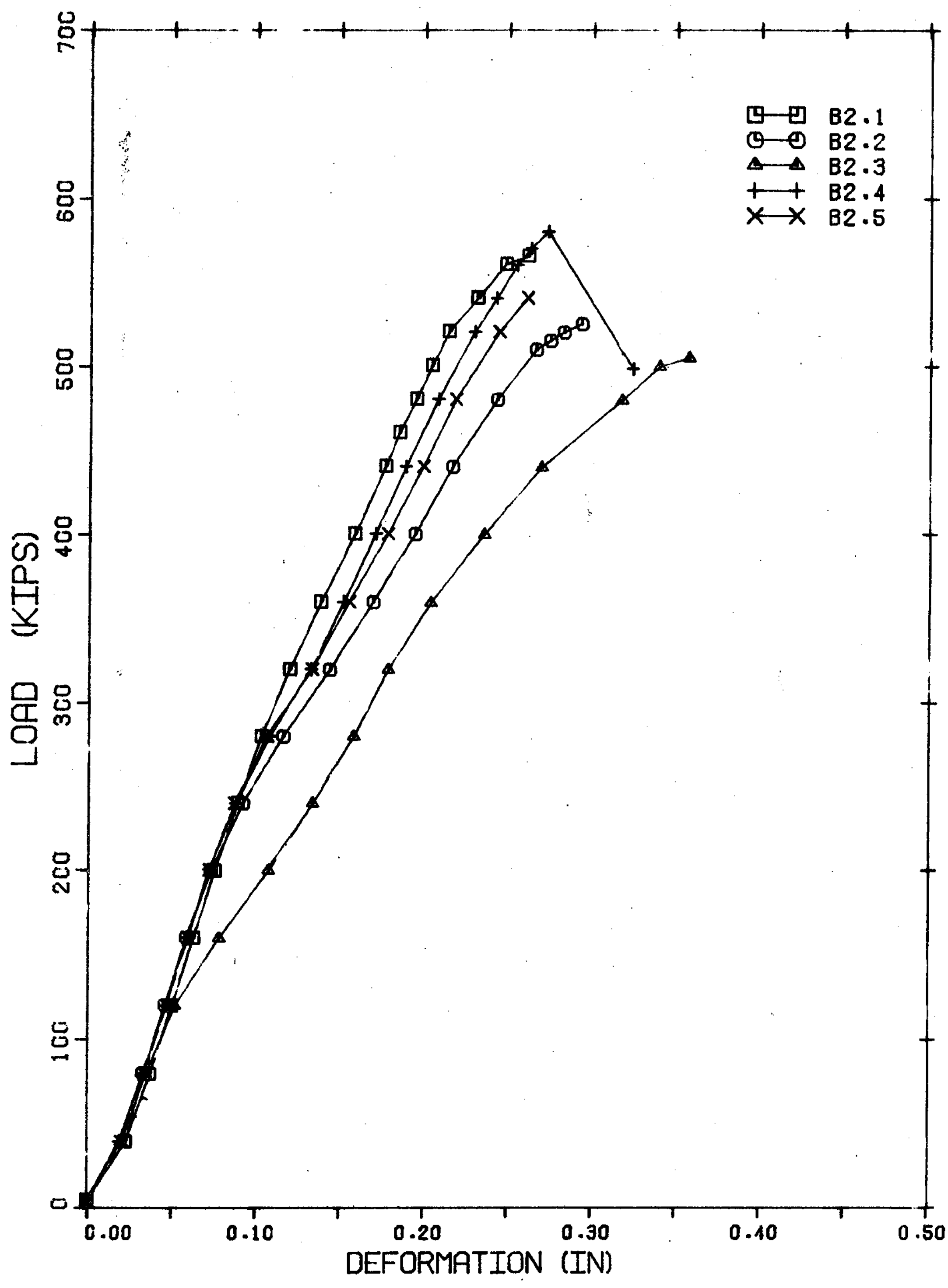


FIGURE 5.6 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP B2

Rust was noted on these ungalvanized ties after failures of columns where they had been placed in the mortar joint. The mortar, placed on top of the tie, did not completely surround the tie.

The same grout mix was used for columns constructed of wall units and of special units. The fluidity of the grout mix was needed to place the grout in the small openings of the cells formed by the wall units. Here, small amounts of shrinkage occurred, while in the columns made of special units, large amounts of shrinkage were noted. As a result it was found that very little bond existed between the grout and the masonry unit in these columns.

CHAPTER VI

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

Thirty-seven concrete masonry columns were tested in order to establish the effects of reinforcement detailing on their strength and behavior. All columns were 64 inches long, had a 16 inch square cross-section and were loaded concentrically through pinned ends. Detailing variables included size and location of tie reinforcement as well as size and amount of vertical reinforcement. Test results obtained included failure loads, horizontal deflections, vertical deformations and strains in both masonry and reinforcement. Auxiliary tests were conducted to establish the constituent material properties. Test results were analysed to show the effects of the reinforcement details investigated on column strength and behavior.

6.2 Conclusions

Based on the analysis of the data and observations obtained from the 37 column tests the following conclusions are presented:

a) Disregarding creep strains in the masonry, which occurred during testing, the behavior of the columns was elastic.

b) The modulus of elasticity of the masonry was 800 times the masonry prism strength.

c) Reinforcement detailing was a major factor affecting the strength and behavior of the columns.

d) The strength and ductility of the columns increased with increasing tie diameter.

e) Ties located in contact with the vertical reinforcement were more effective in providing restraint against buckling of the vertical reinforcement than the ties located in the mortar joint.

f) The ability of the vertical reinforcement to reach its specified yield strength was dependent on the strength developed by the masonry.

g) The contribution of the masonry to the strength of columns decreased as vertical bar diameter increased.

h) The presence of mortar droppings at the base of columns decreased the ultimate strength significantly.

i) The use of a fluid grout mixture resulted in large amounts of shrinkage which was detrimental to the strength and behavior of columns constructed from special units.

6.3 Recommendations

a) All cells which are to contain vertical reinforcement should be provided with cleanout openings for the removal of mortar droppings. After the placement of the reinforcement, special inspection may be necessary to ensure that mortar droppings have been removed.

b) Columns constructed from special units should be filled with normal slump concrete instead of the high slump grout used in this study. An investigation of this type of column would show reduced shrinkage of the column core. The contribution to load carrying capacity of different thicknesses of shells constructed from special concrete masonry units could be evaluated in such an investigation.

c) Reinforcement placed in mortar joints should be galvanized to prevent deterioration due to rust.

d) A study should be undertaken to show the effect of bar size on masonry strength based on tangential tensile stresses being developed in the masonry surrounding the reinforcing bar. The magnitude of these tensile stresses depend on the size of the bar and its deformations and are related to the difference in Poisson's ratio between the bar material and the masonry material. The study should include the evaluation of the resistance to these tensile stresses by the tensile strength of the masonry and by the confining effects of the tie reinforcement.

e) A study should be made to establish the minimum amount of lateral reinforcement for columns with both tie and mesh types of reinforcement. For the same lateral reinforcement diameter, the mesh reinforcement will provide more confinement since its bending stiffness is greater than that of the tie reinforcement. It is recommended that the reduction in masonry strength, due to the use of large diameter vertical reinforcing bars, could best be recognized through tie detailing requirements. The resulting requirements would be based on providing a minimum volumetric percentage of lateral reinforcement depending on vertical bar size. They would also give the needed design flexibility particularly when size and spacing of lateral reinforcement is restricted.

f) Tests of eccentrically loaded columns should be conducted. The results of such tests would be valuable in providing additional information for use in strength design procedures.

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APPENDIX A

Reinforcement Instrumentation and Auxiliary Test Methods

A.1 Reinforcement Instrumentation

Electrical resistance strain gages, having a gage factor of 2.095 ± 0.5%, were used in the instrumentation of reinforcement. The following procedure was used:

1) The surface of the reinforcing bar was prepared by removing deformations and filing until a smooth surface was obtained.
2) This surface was treated with an acid based solution and then neutralized.
3) The strain gage was cemented in position with a special adhesive by applying pressure.
4) Waterproofing was applied to the gage.
5) Electrical connections were made by soldering.
6) Electrical tape was applied to the gage and further waterproofing provided by coating with epoxy or silicone.

A.2 Auxiliary Tests

With the exception of 2 grout tests, the auxiliary tests described herein were performed in accordance with the appropriate standards.

A.2.a Masonry Units

Concrete masonry wall units were capped with plaster and tested in axial compression in the MTS Systems testing machine.

A.2.b Mortar

Mortar cubes were prepared, and cured either in air or under wet burlap. They were tested at 28 days in a Baldwin Universal testing machine.

A.2.c Prisms

Concrete masonry wall units or special units, as depicted in Figures A1 and A2 respectively, were used in the construction of prisms. All prisms were 16 inches high and were constructed using face shell mortar. Both grouted and plain prisms were cured under the same conditions as the columns and tested at the same age. The prisms were capped with plaster and tested in axial compression in the MTS Systems testing machine.

A.2.d Grout

Grout prisms and cylinders were prepared and tested in axial compression in the Baldwin testing machine. In addition, the grouted cores of a failed masonry prism were tested. Deformation data was obtained from the grout cylinder compression tests through extensometer measurements over an 8 inch gage length.

A.2.e Reinforcement

Two 20 inch long specimens of each bar size used were tested in axial tension in the Baldwin testing machine. The

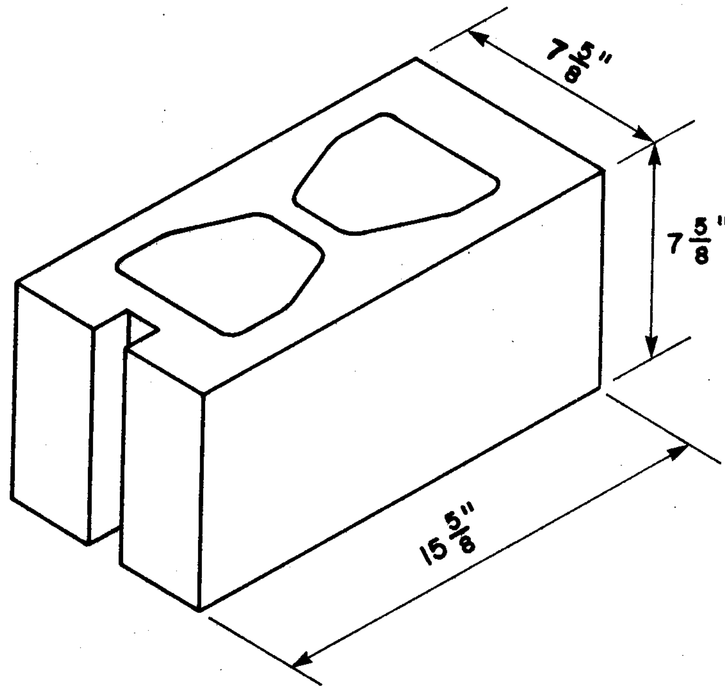

FIGURE A1 CONCRETE MASONRY WALL UNIT

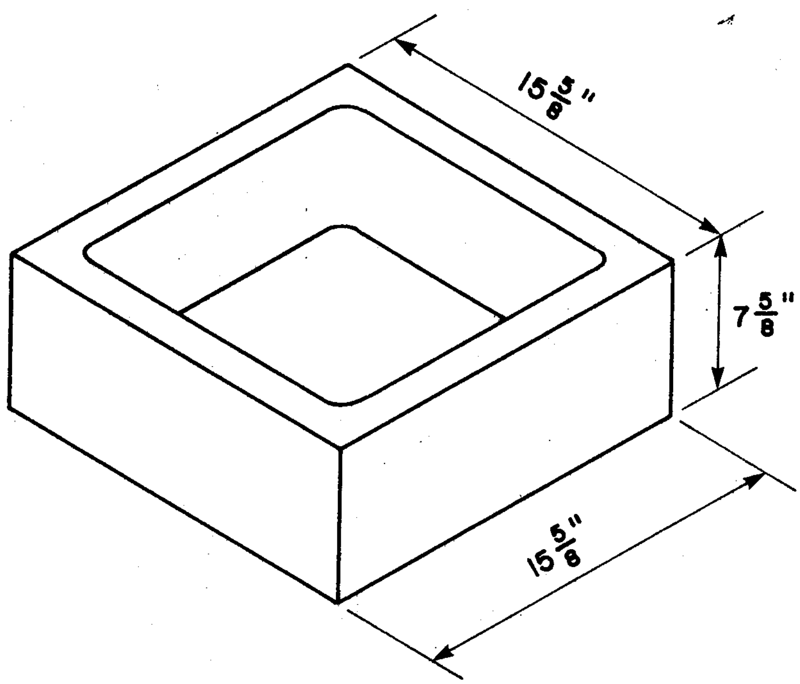

FIGURE A2 SPECIAL CONCRETE MASONRY UNIT

use of an extensometer during testing provided plots of load vs. strain for these tests.

APPENDIX B

Plotted Test Results

DISCLAIMER

In some of the plots showing the deflected shape of columns, the transducers were inoperative. This, combined with the curve fitting technique used to plot these graphs, resulted in deflected shapes which are incorrect.

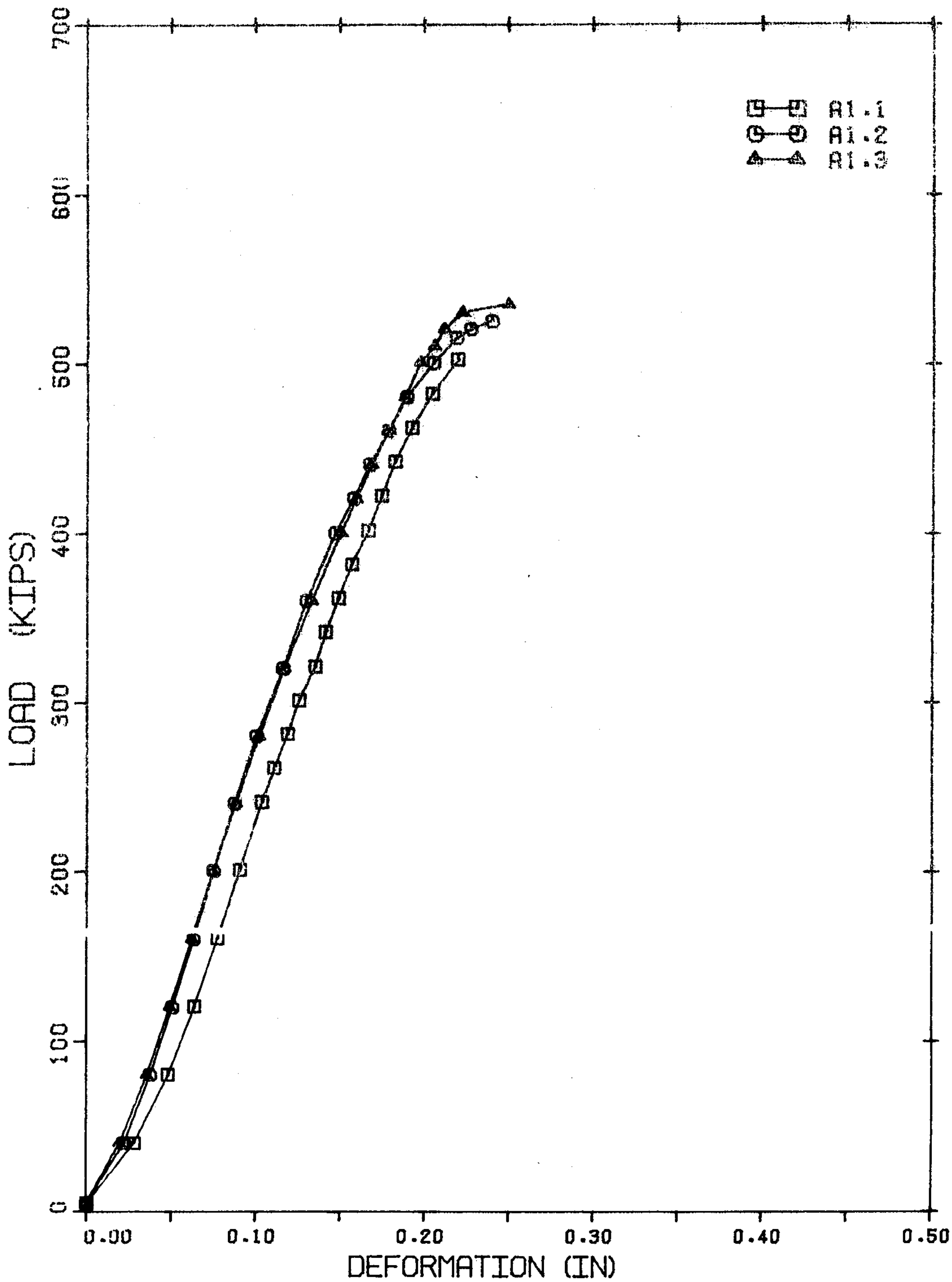

FIGURE B1.1 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP A1

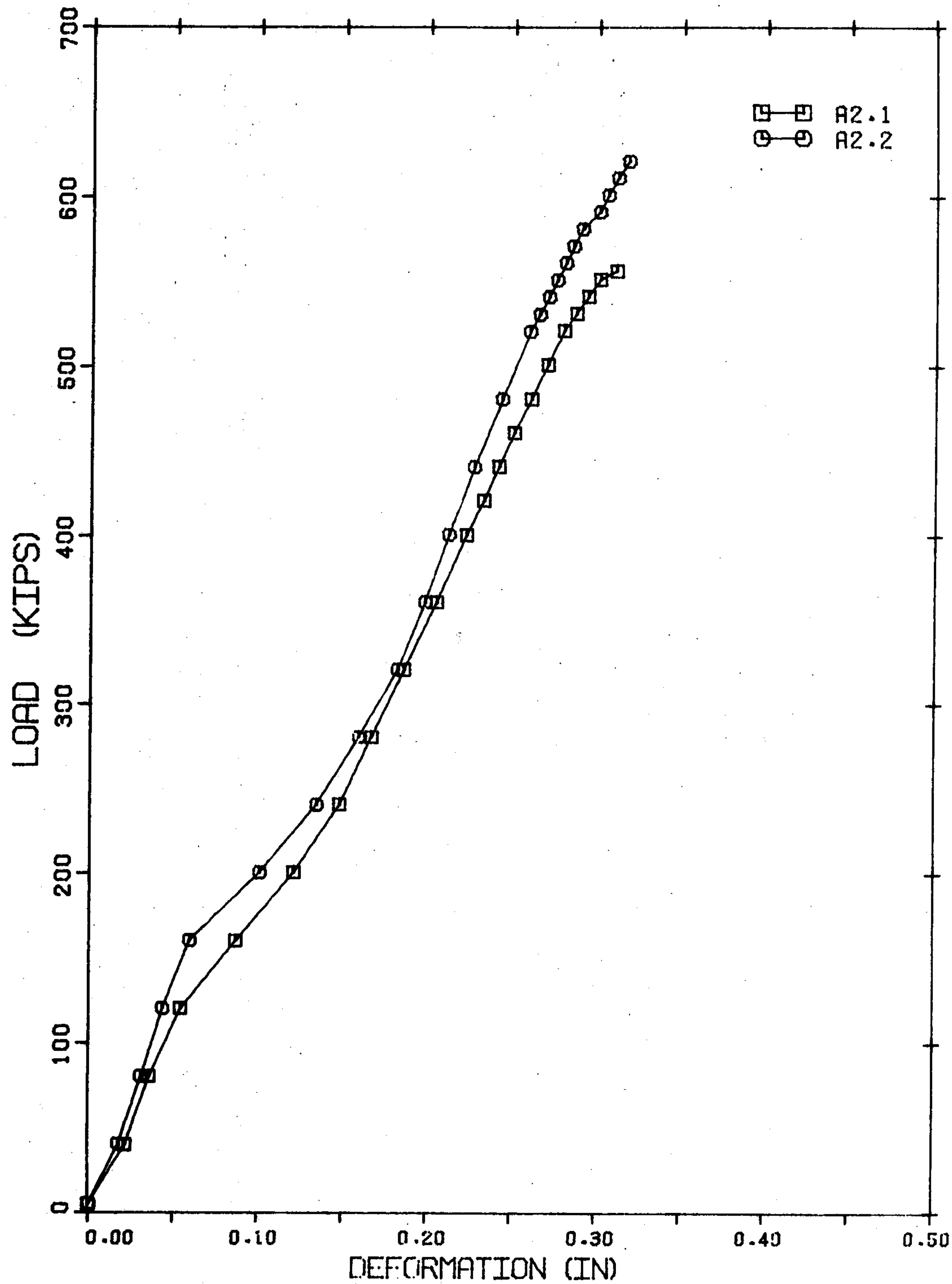

FIGURE B1.2 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP A2

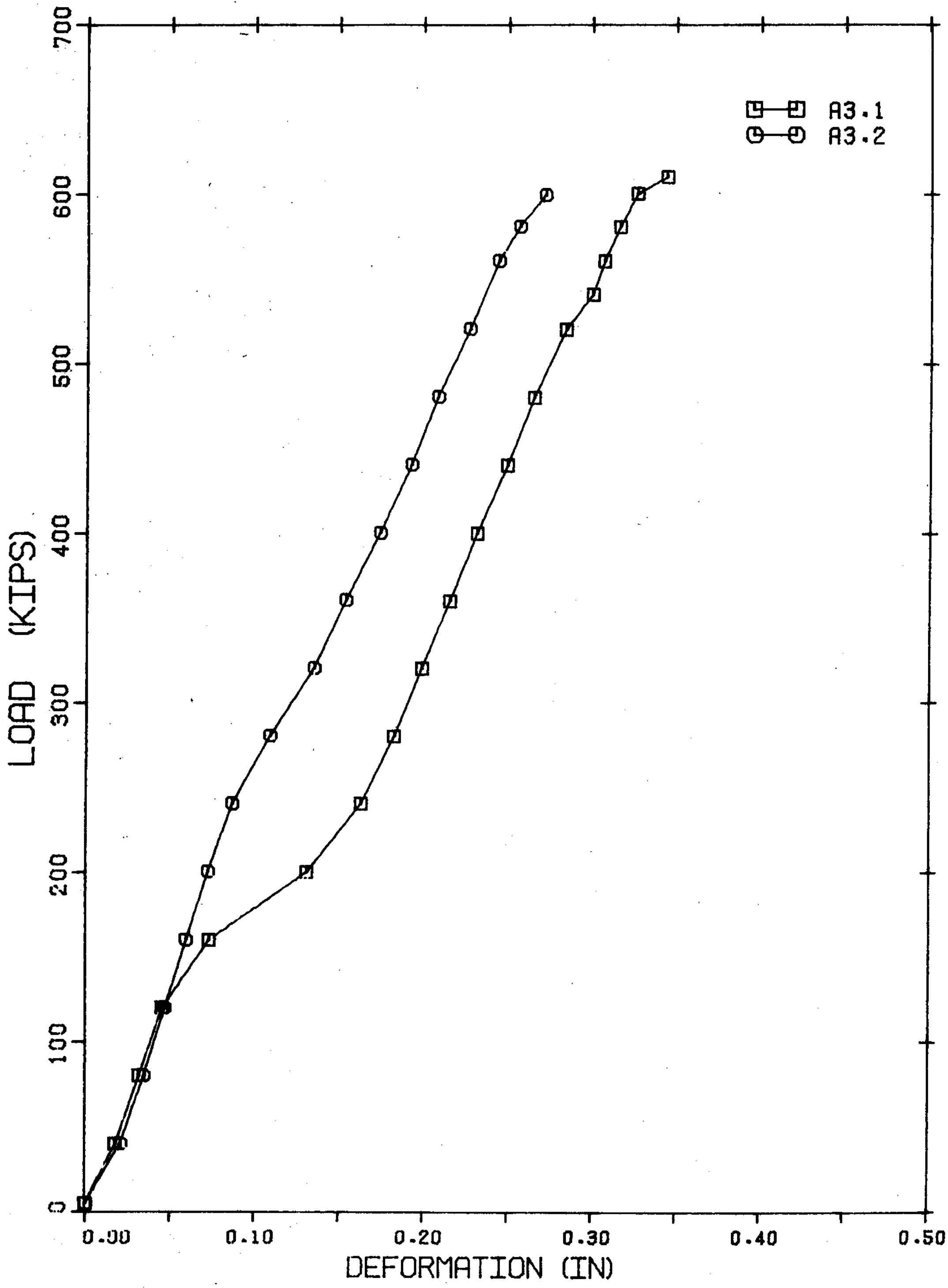

FIGURE B1.3 LOAD VS VERTICAL DEFORMATION

FOR COLUMN GROUP A3

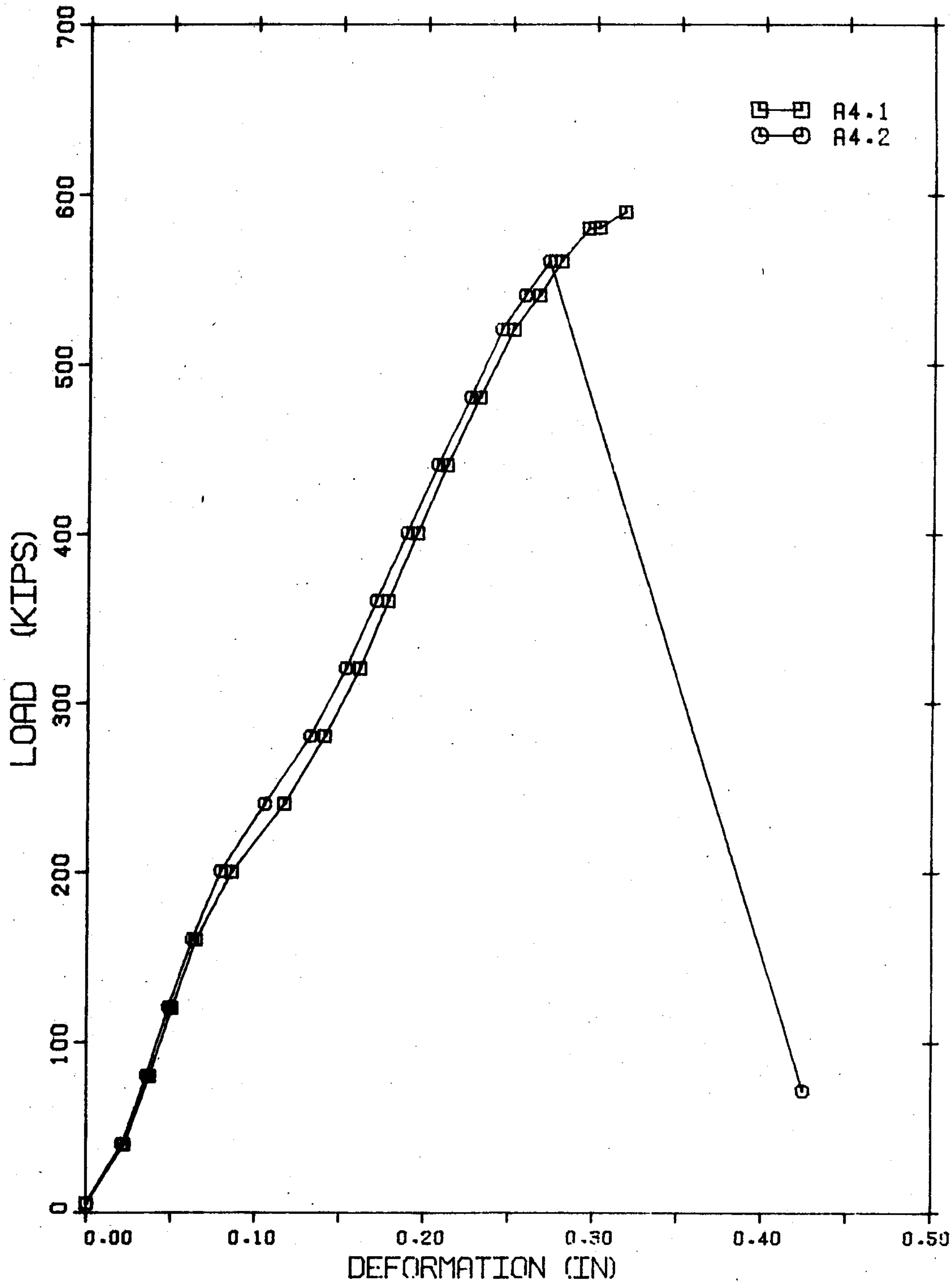

FIGURE B1.4 LOAD VS VERTICAL DEFORMATION FOR

COLUMN GROUP A4

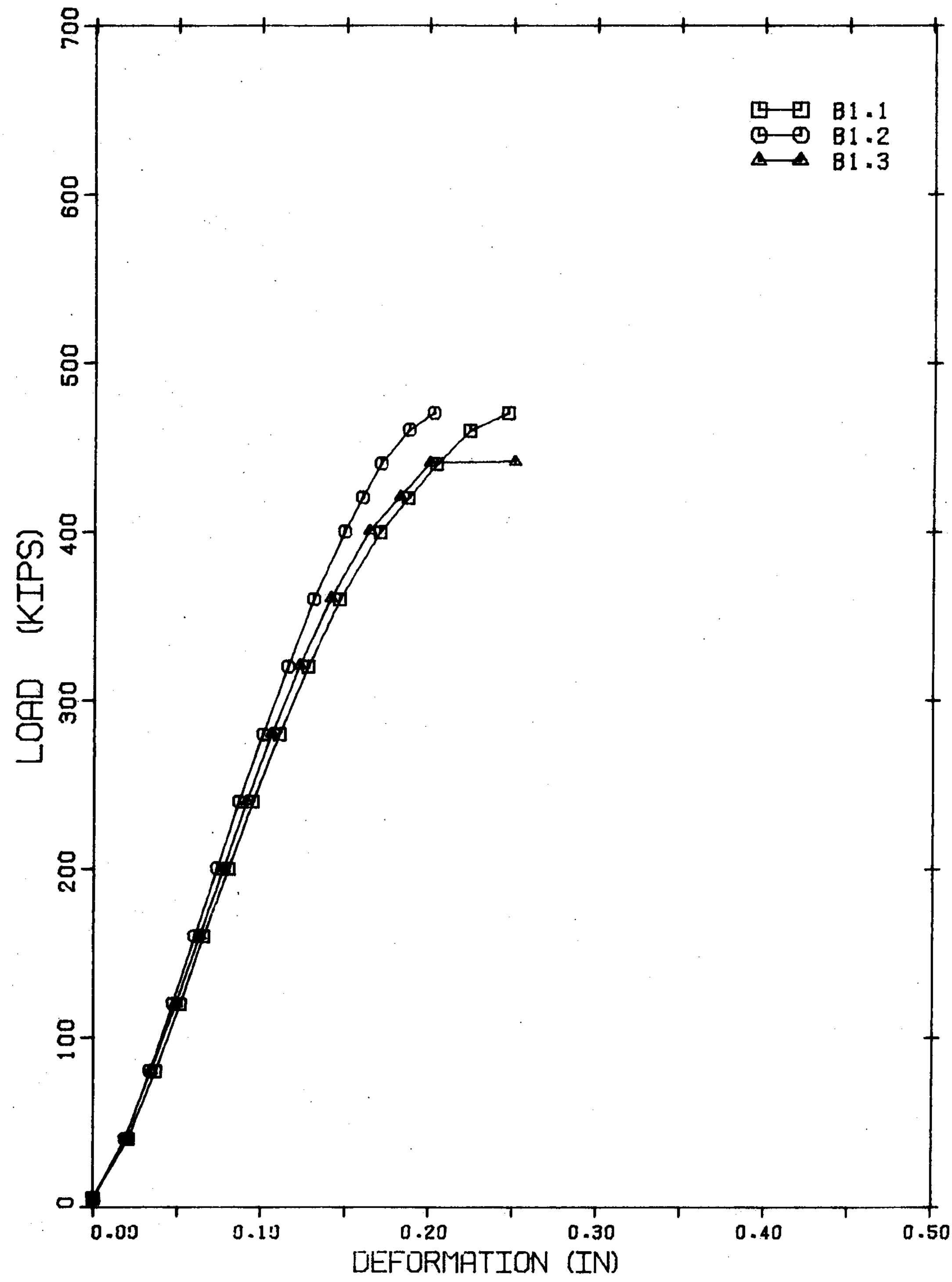

FIGURE B1.5 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP B1

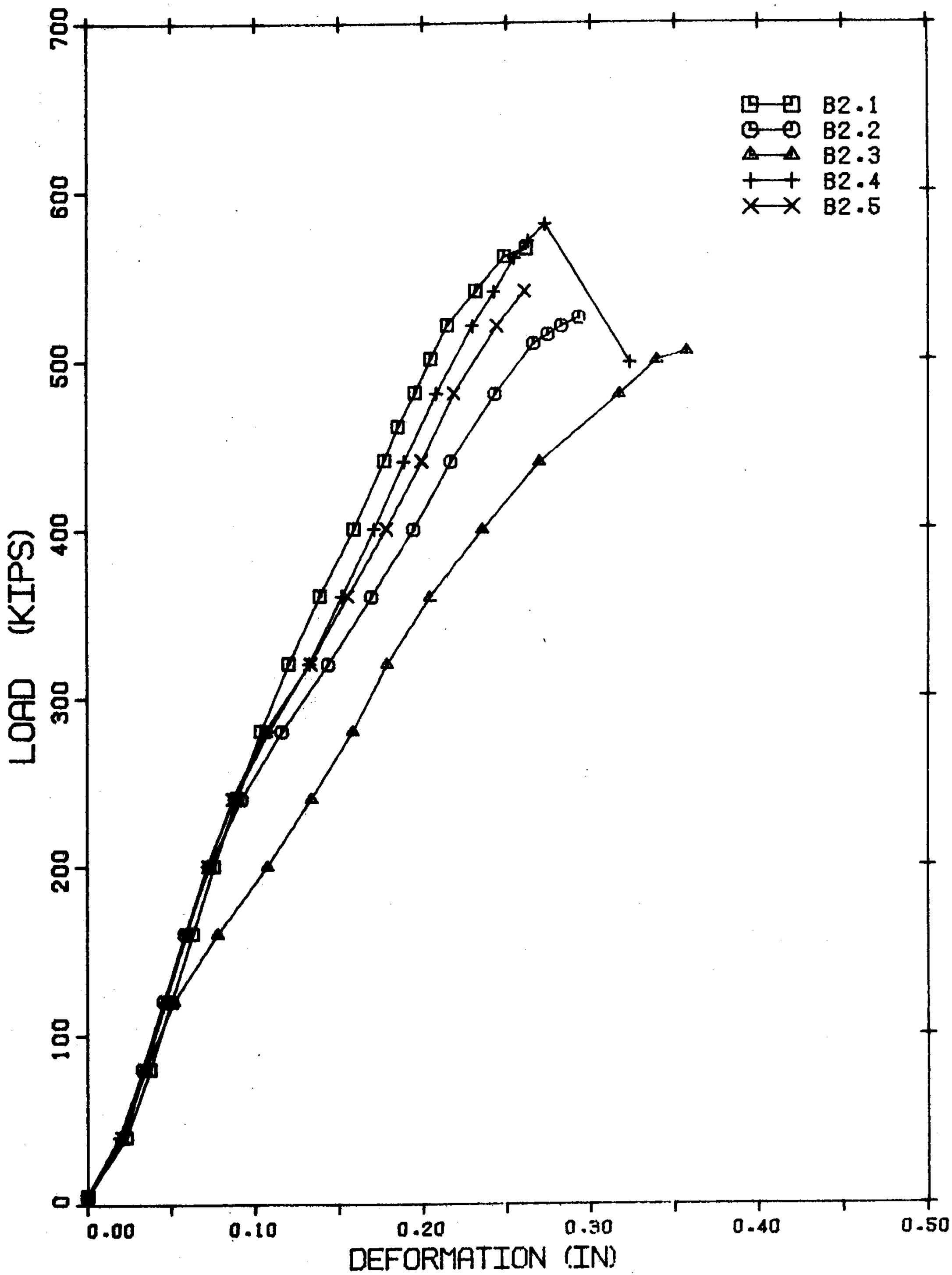

FIGURE B1.6 LOAD VS VERTICAL DEFORMATION

FOR COLUMN GROUP B2

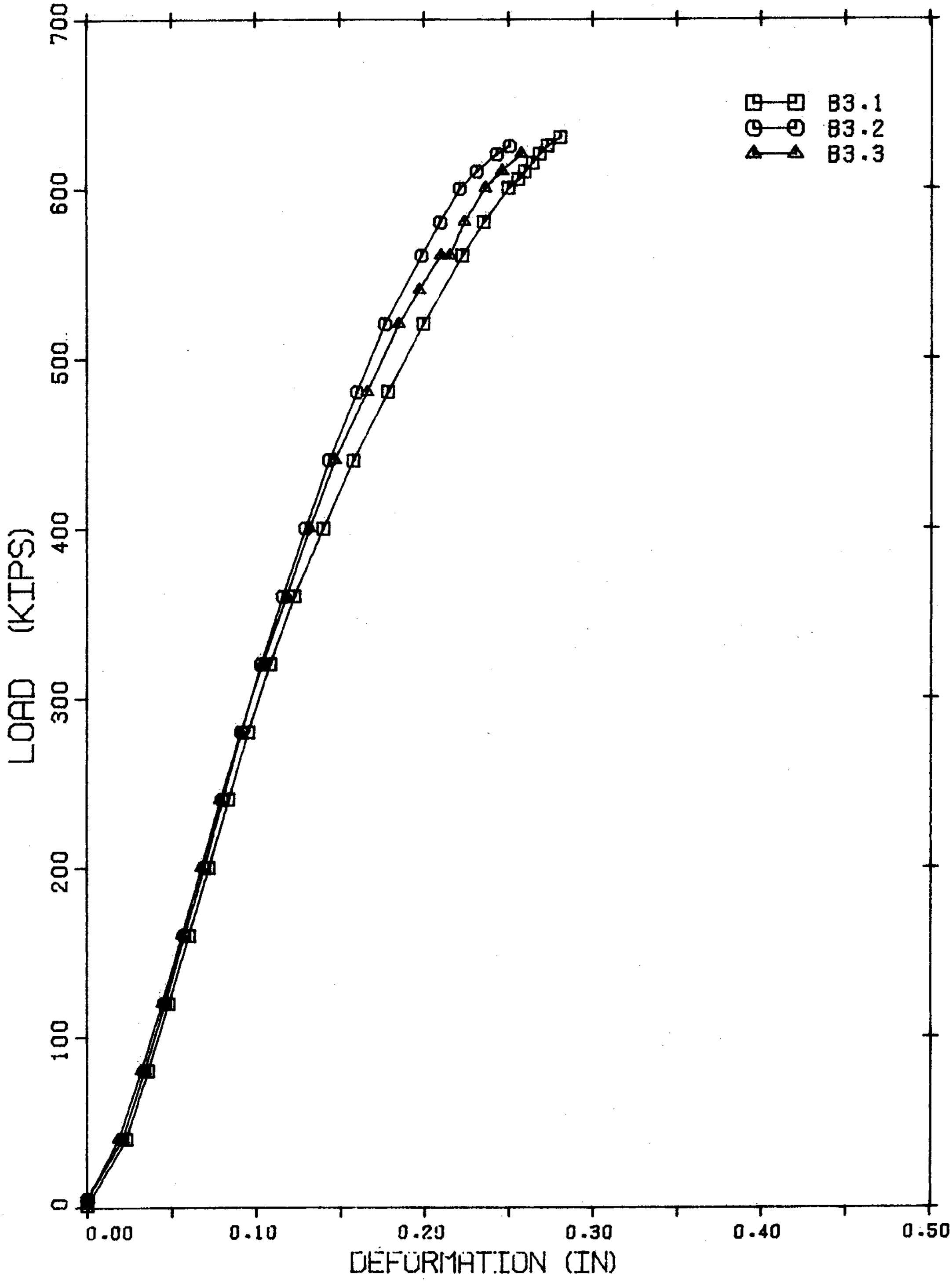

FIGURE B1.7 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP B3

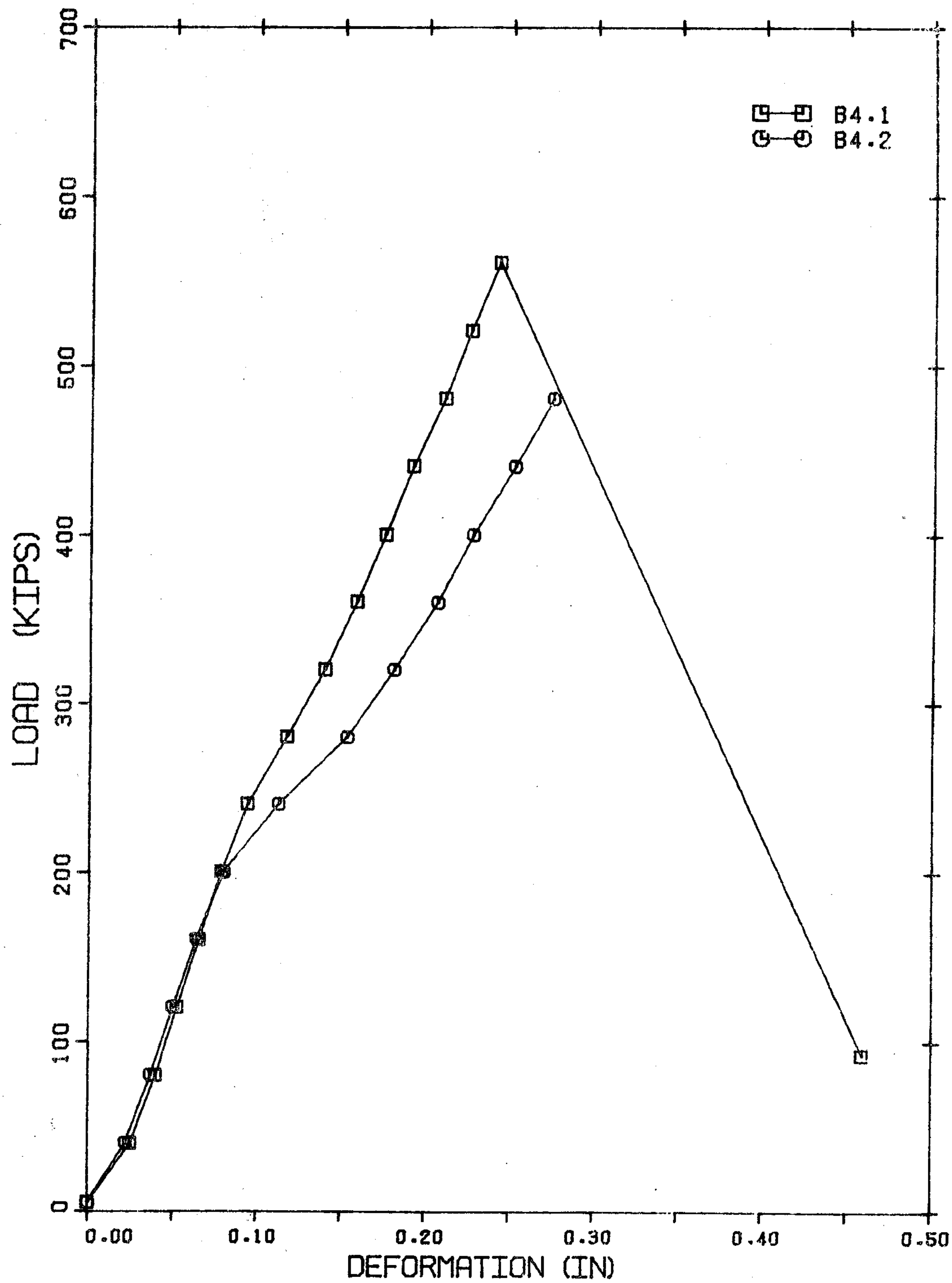

FIGURE B1.8 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP B4

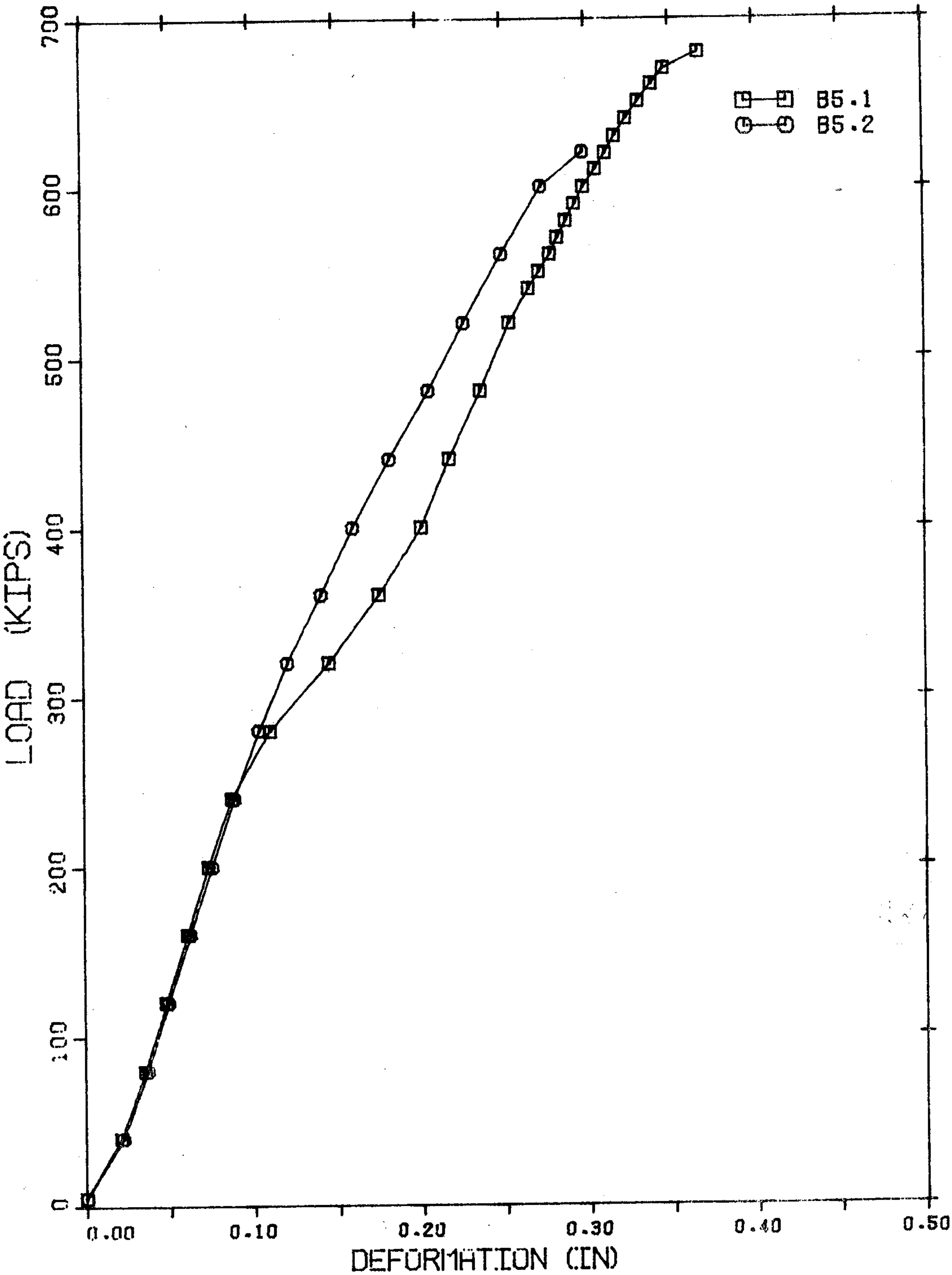

FIGURE B1.9 LOAD VS VERTICAL DEFORMATION

FOR COLUMN GROUP B5

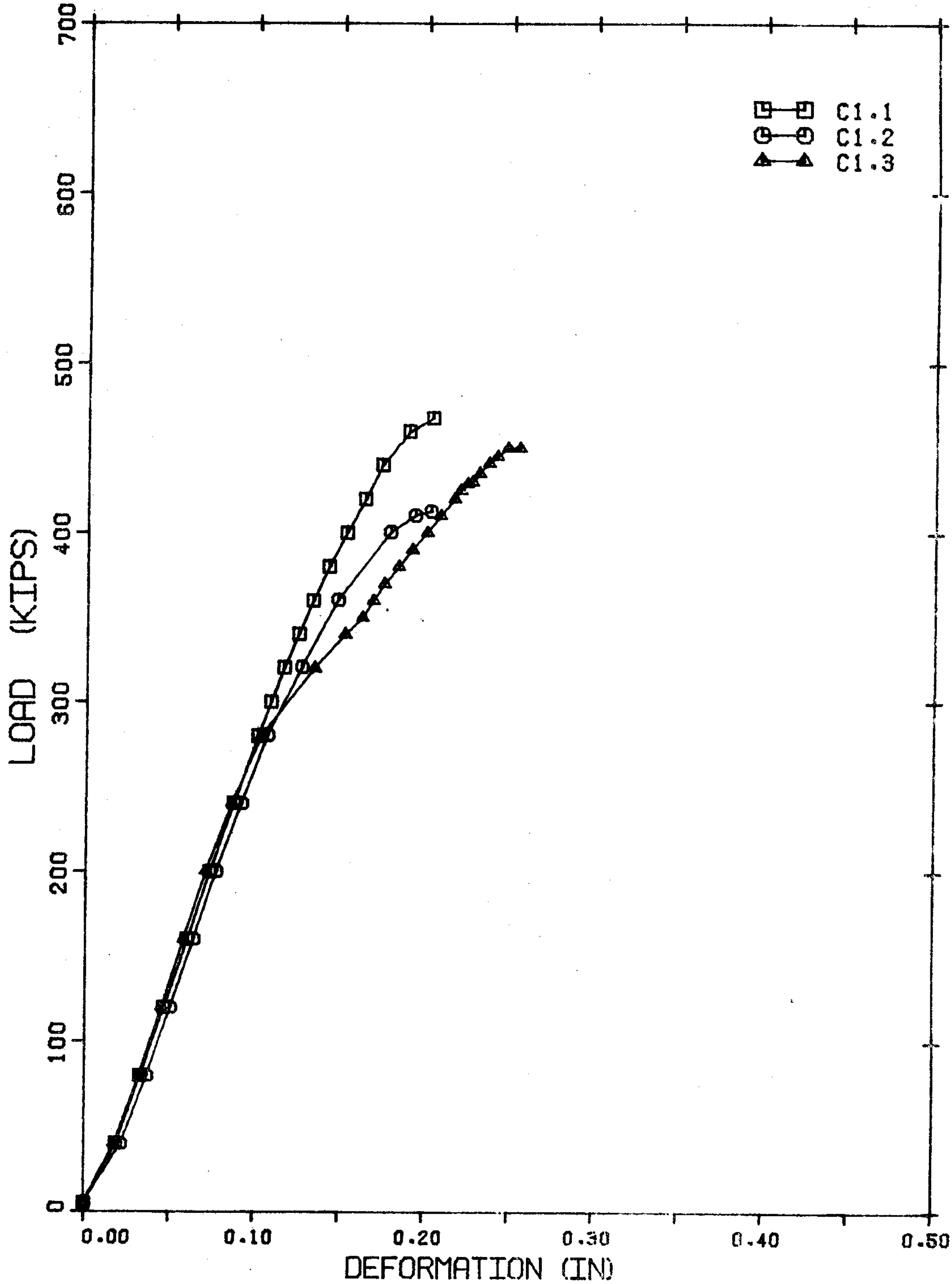

FIGURE B1.12 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP C1

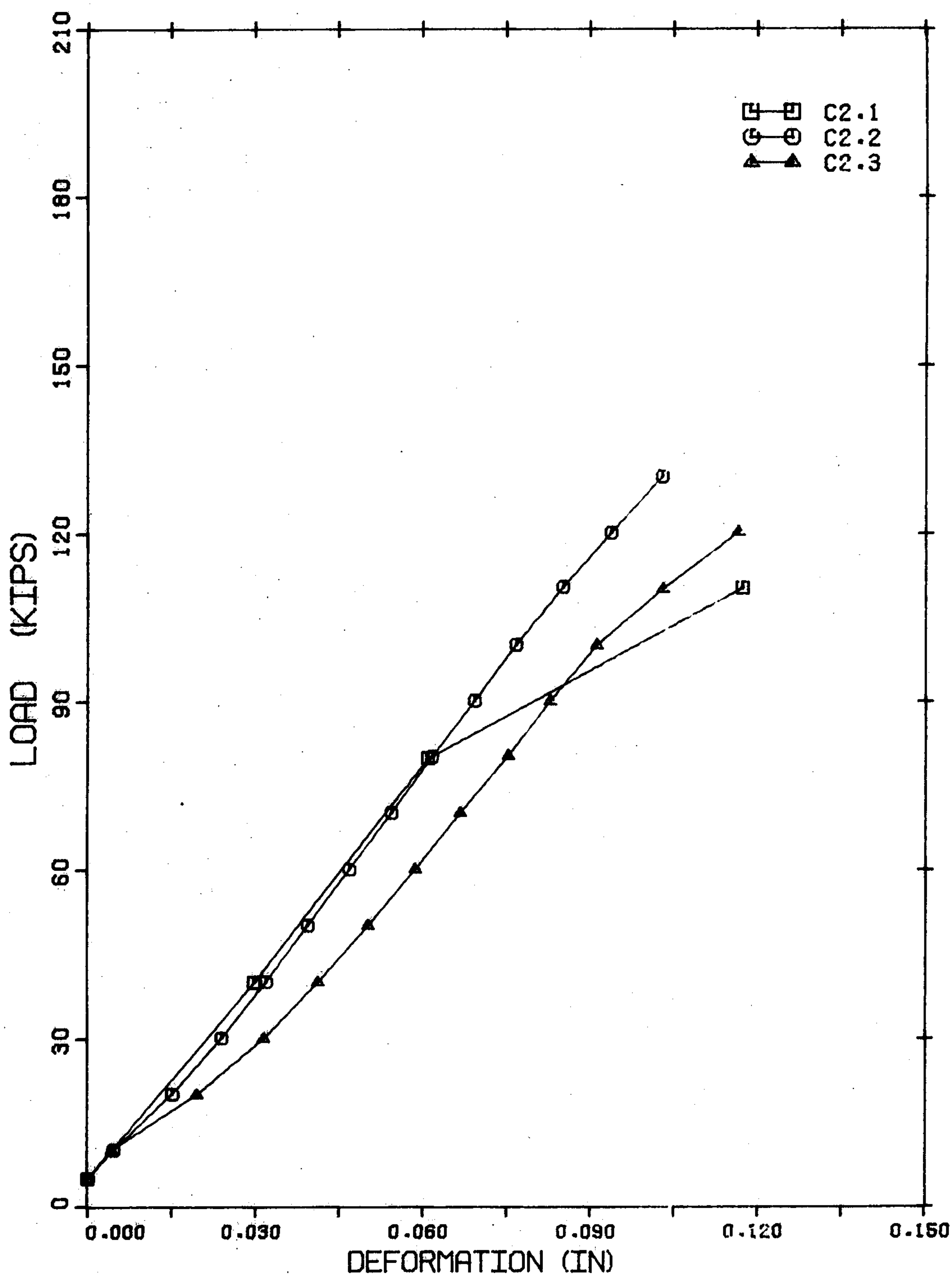

FIGURE B1.13 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP C2

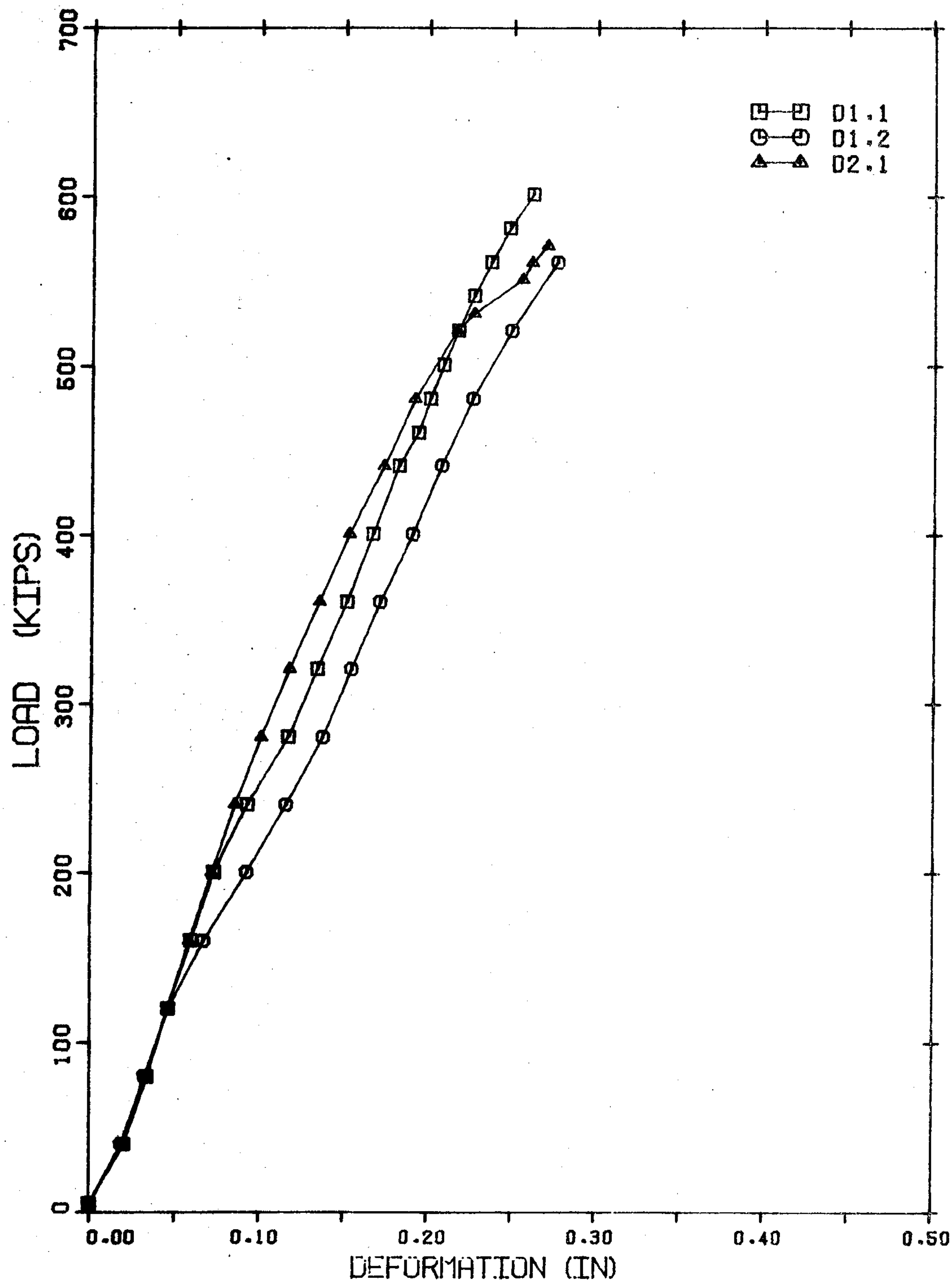

FIGURE B1.14 LOAD VS VERTICAL DEFORMATION FOR COLUMN GROUP D

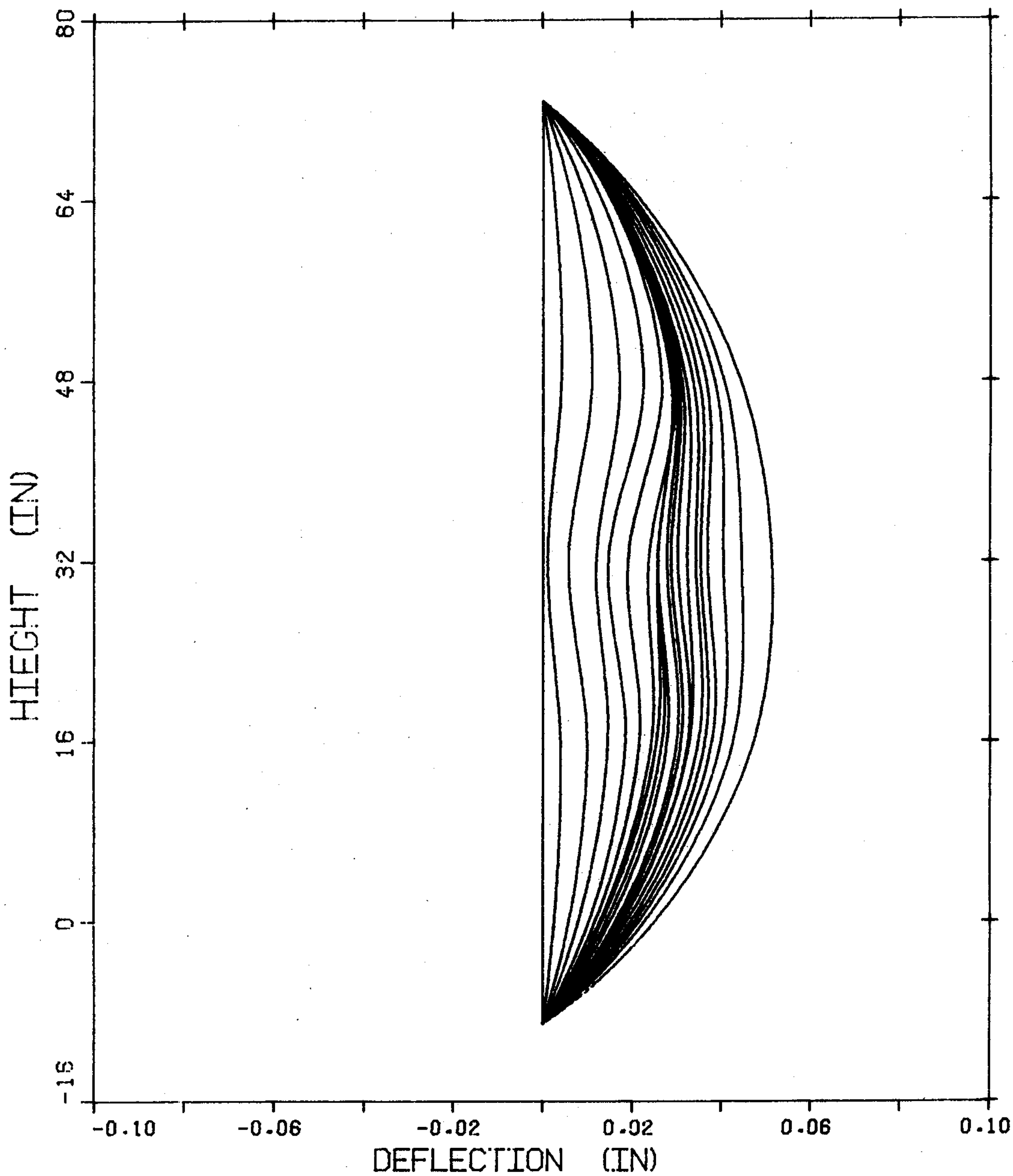

FIGURE B2.1 DEFLECTED SHAPE OF COLUMN A1.1

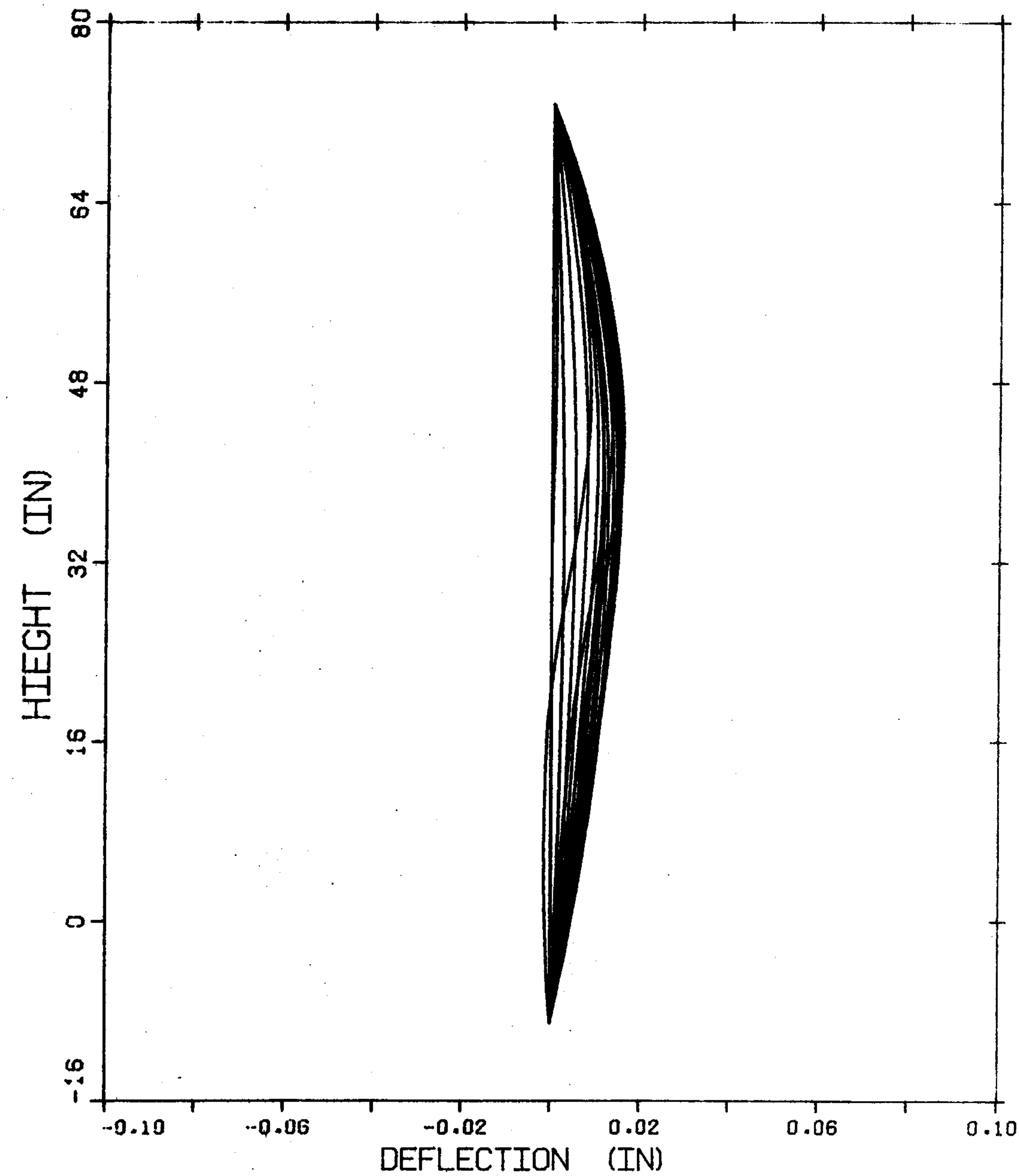

FIGURE B2.2 DEFLECTED SHAPE OF COLUMN A1.2

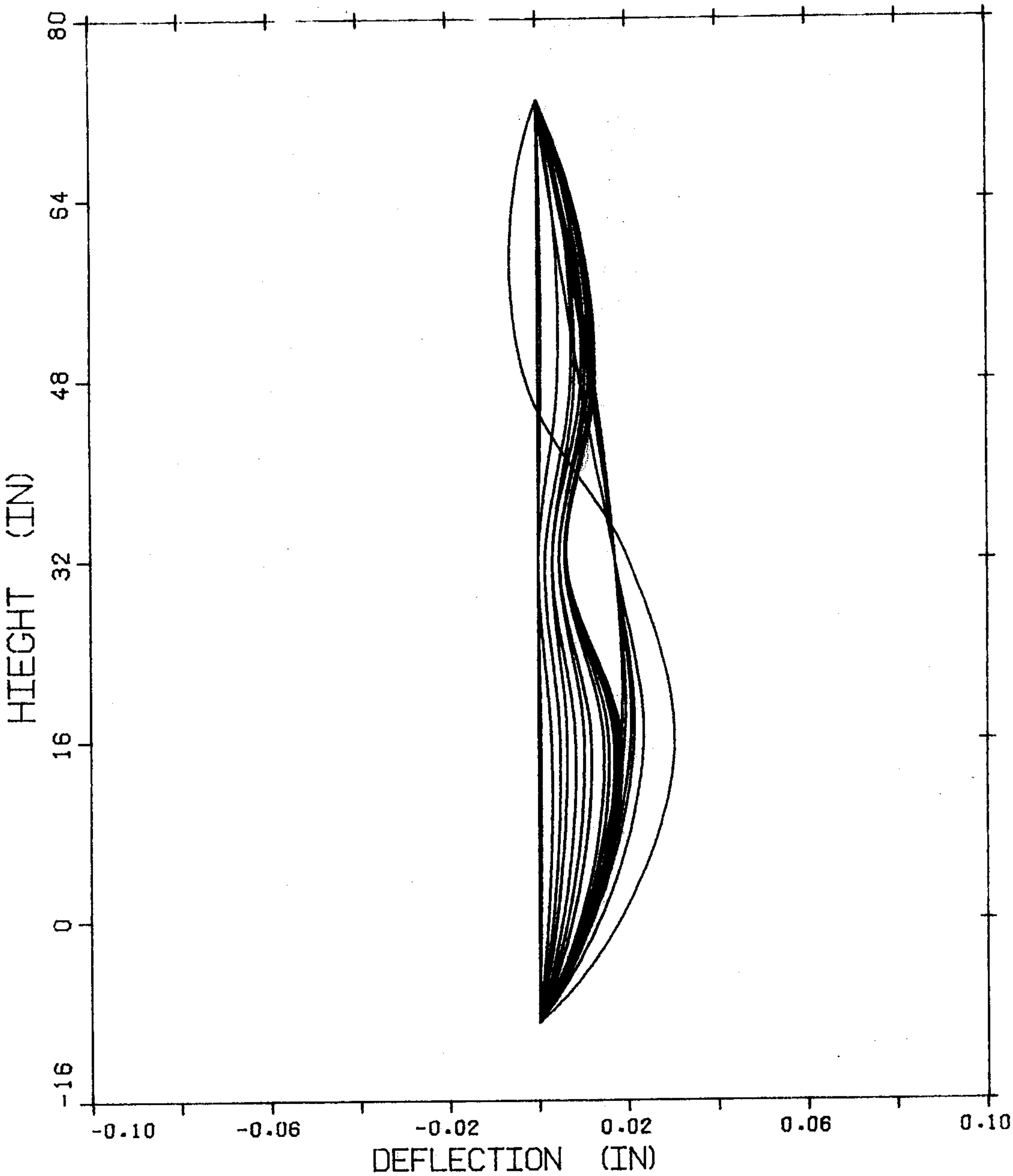

FIGURE B2.3 DEFLECTED SHAPE OF COLUMN A1.3

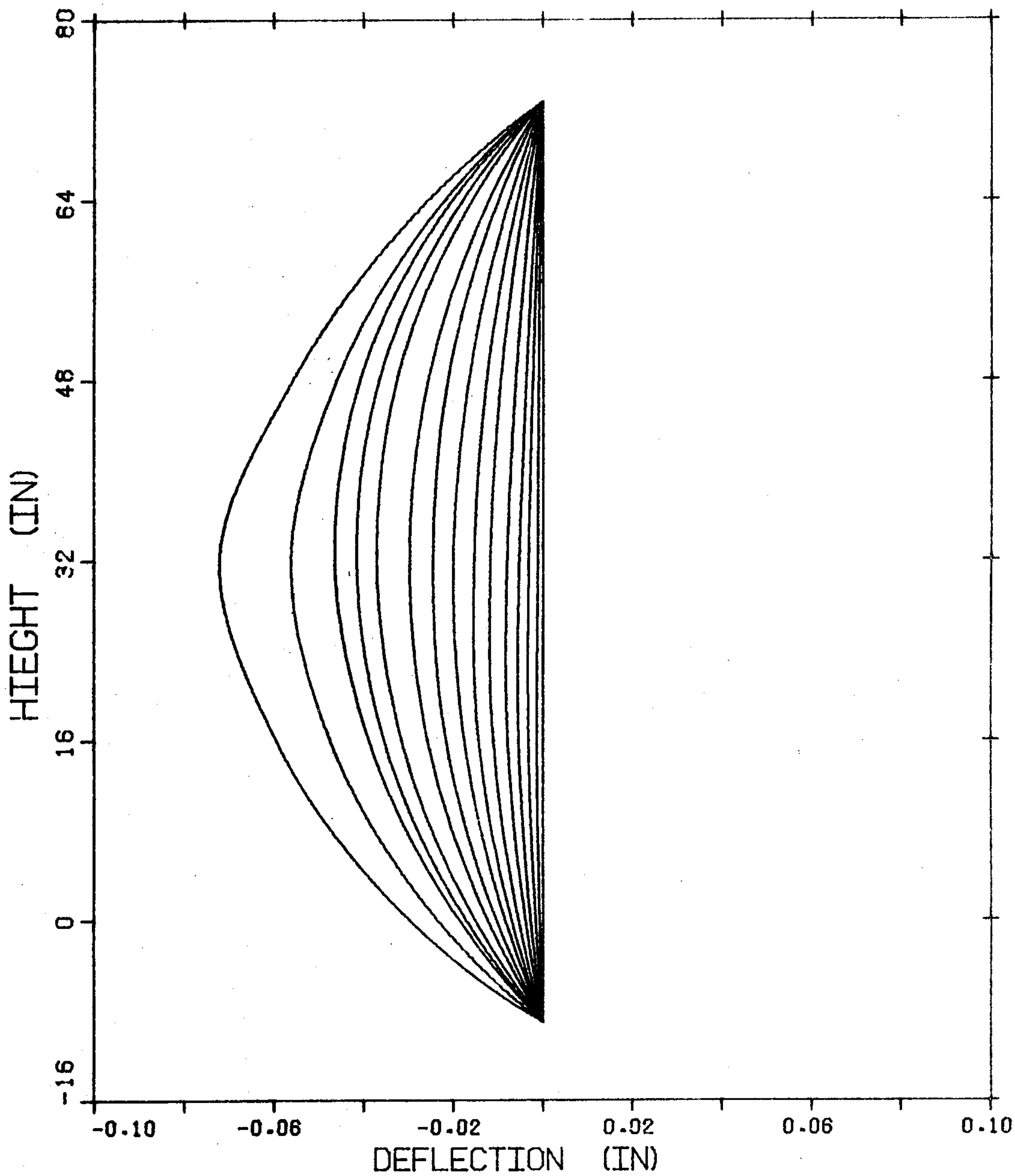

FIGURE B2.5 DEFLECTED SHAPE OF COLUMN B1.2

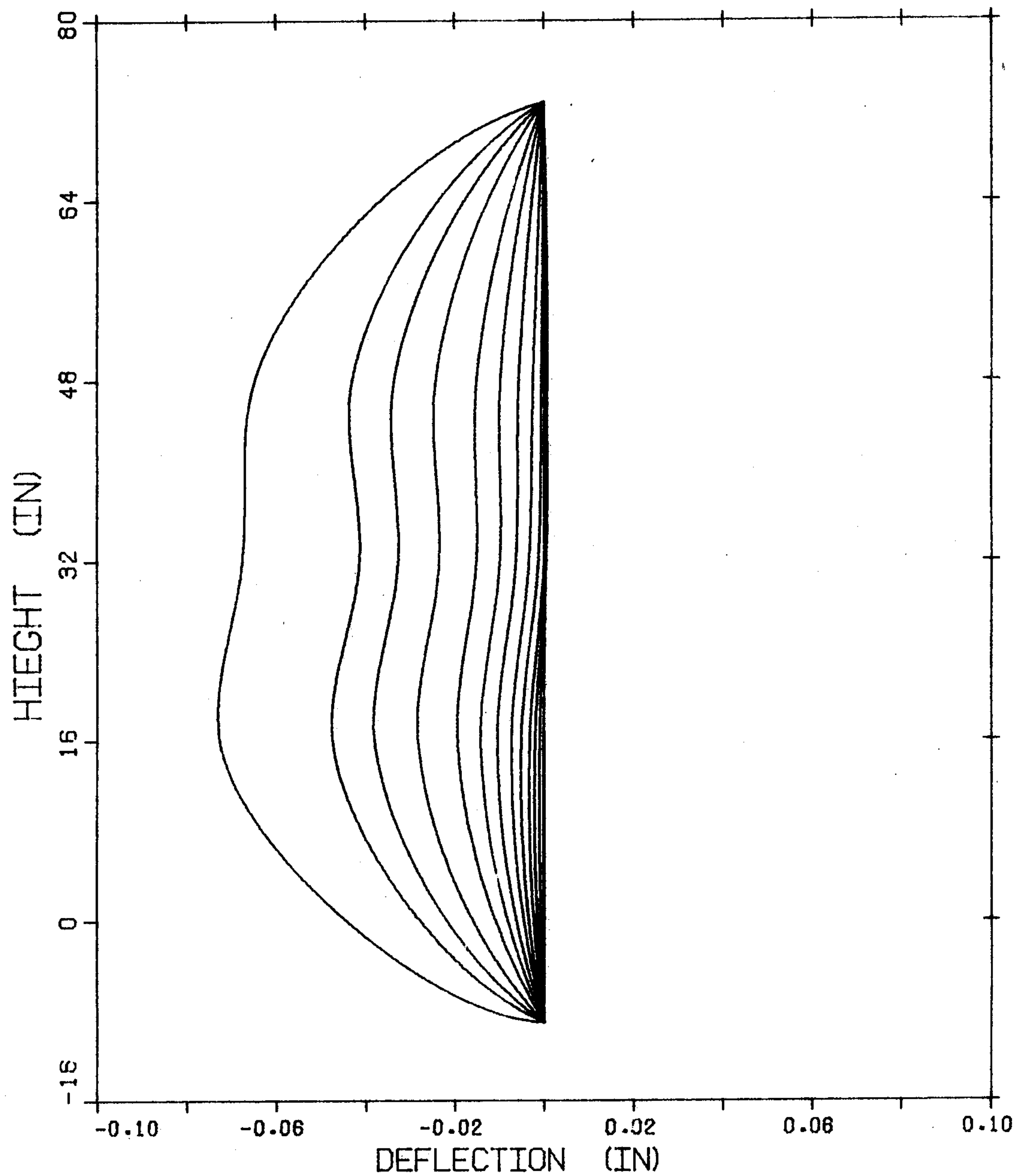

FIGURE B2.6 DEFLECTED SHAPE OF COLUMN B1.3

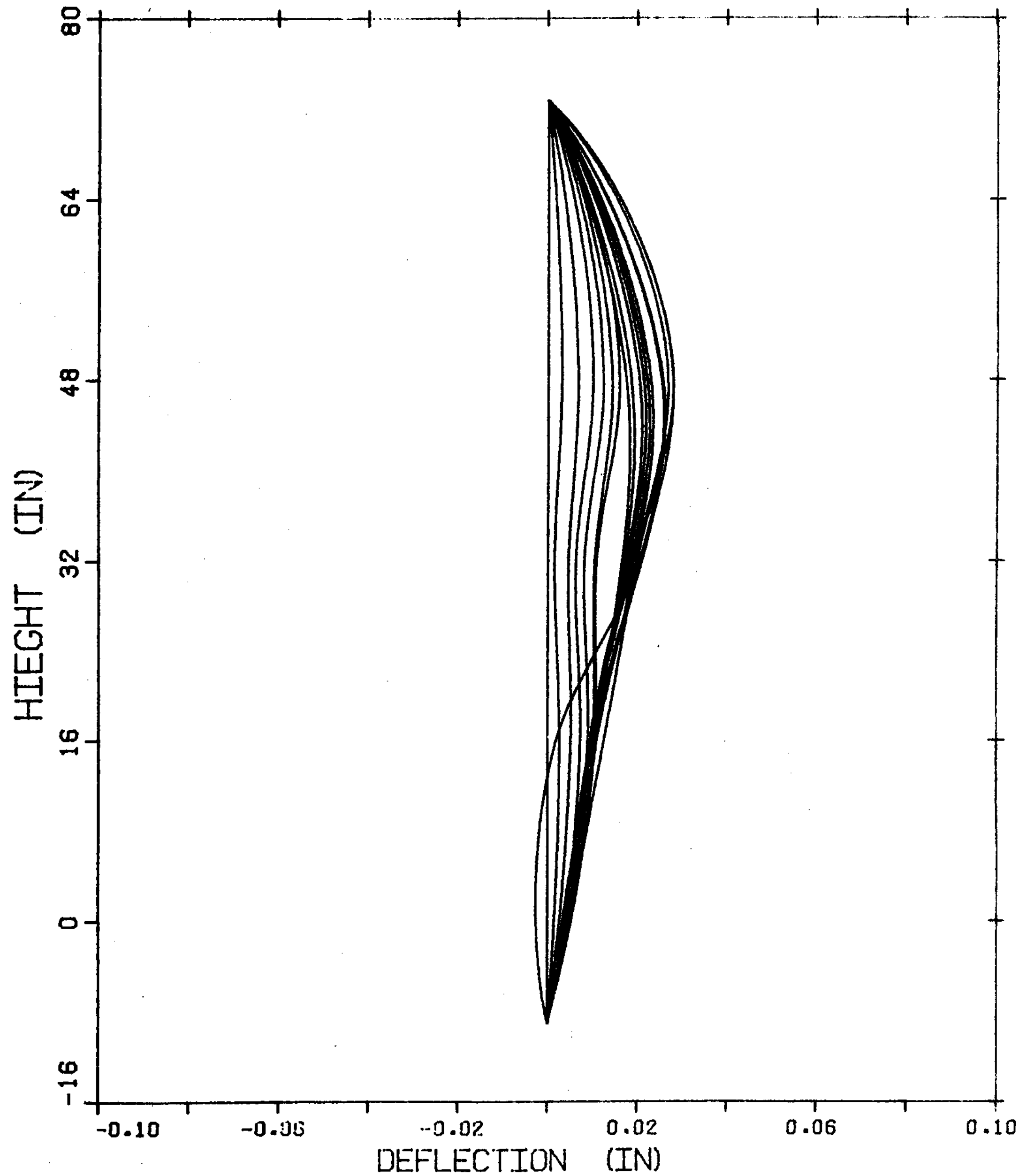

FIGURE B2.7 DEFLECTED SHAPE OF COLUMN B2.1

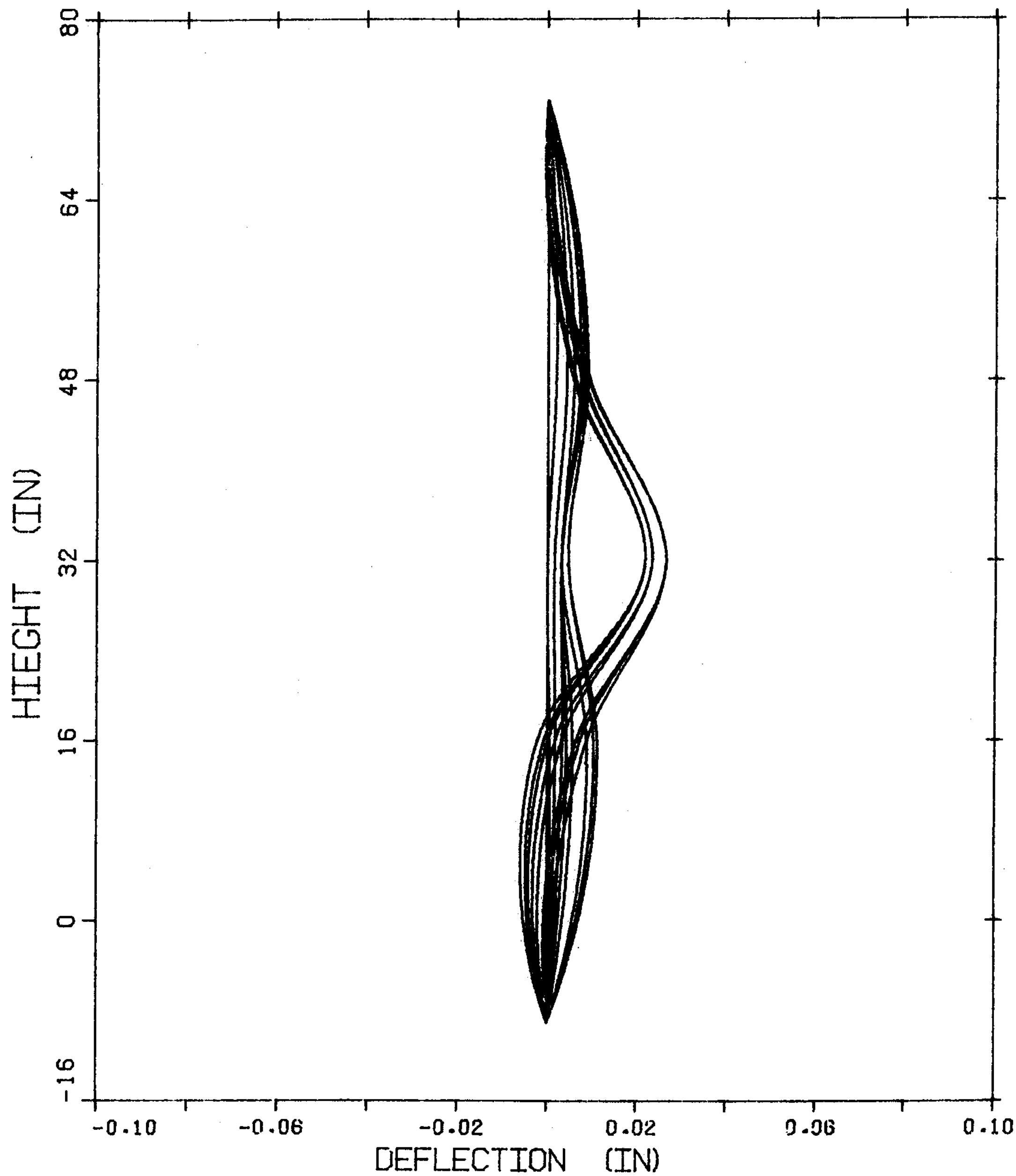

FIGURE B2.8 DEFLECTED SHAPE OF COLUMN B2.2

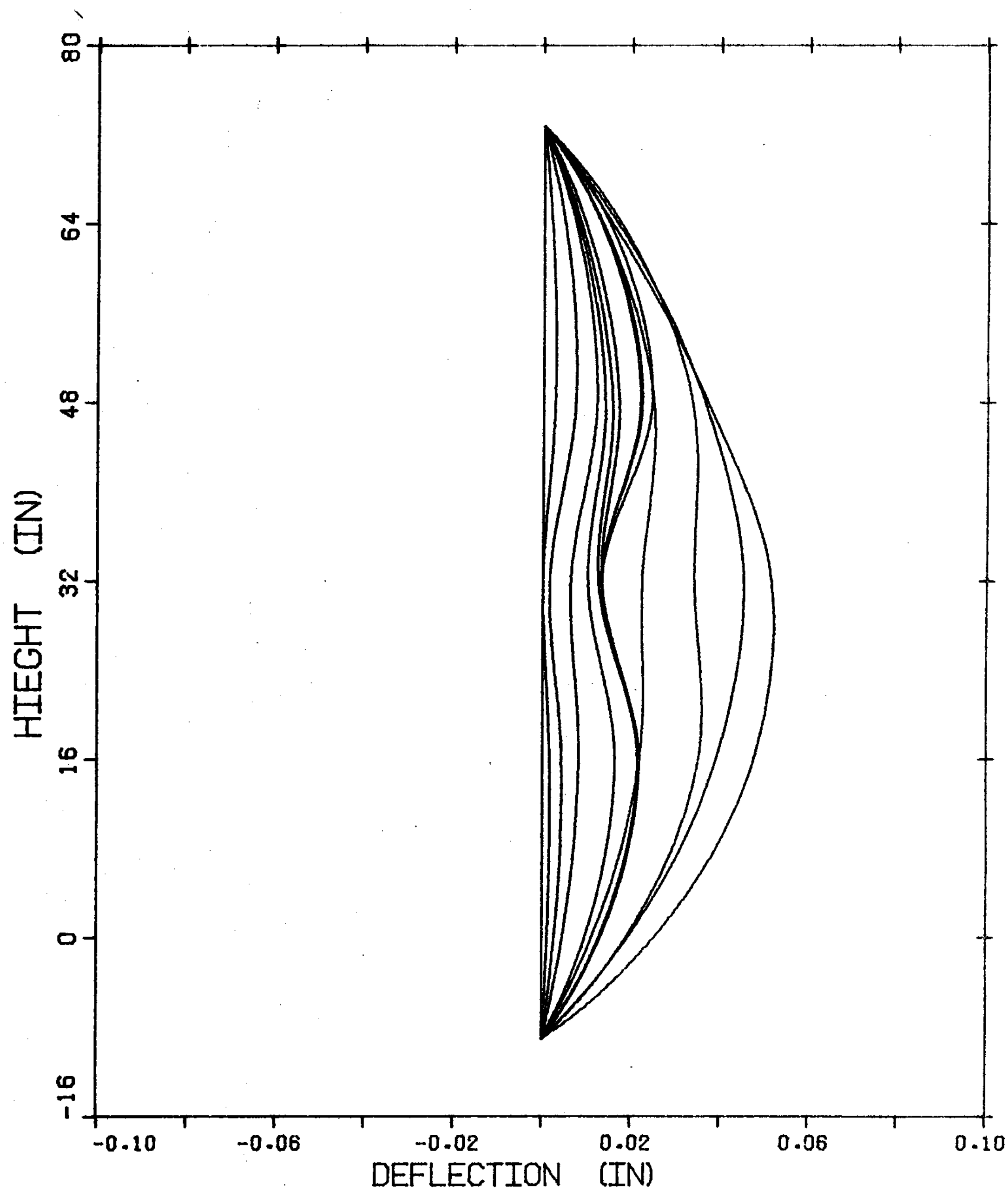

FIGURE B2.9 DEFLECTED SHAPE OF COLUMN B2.3

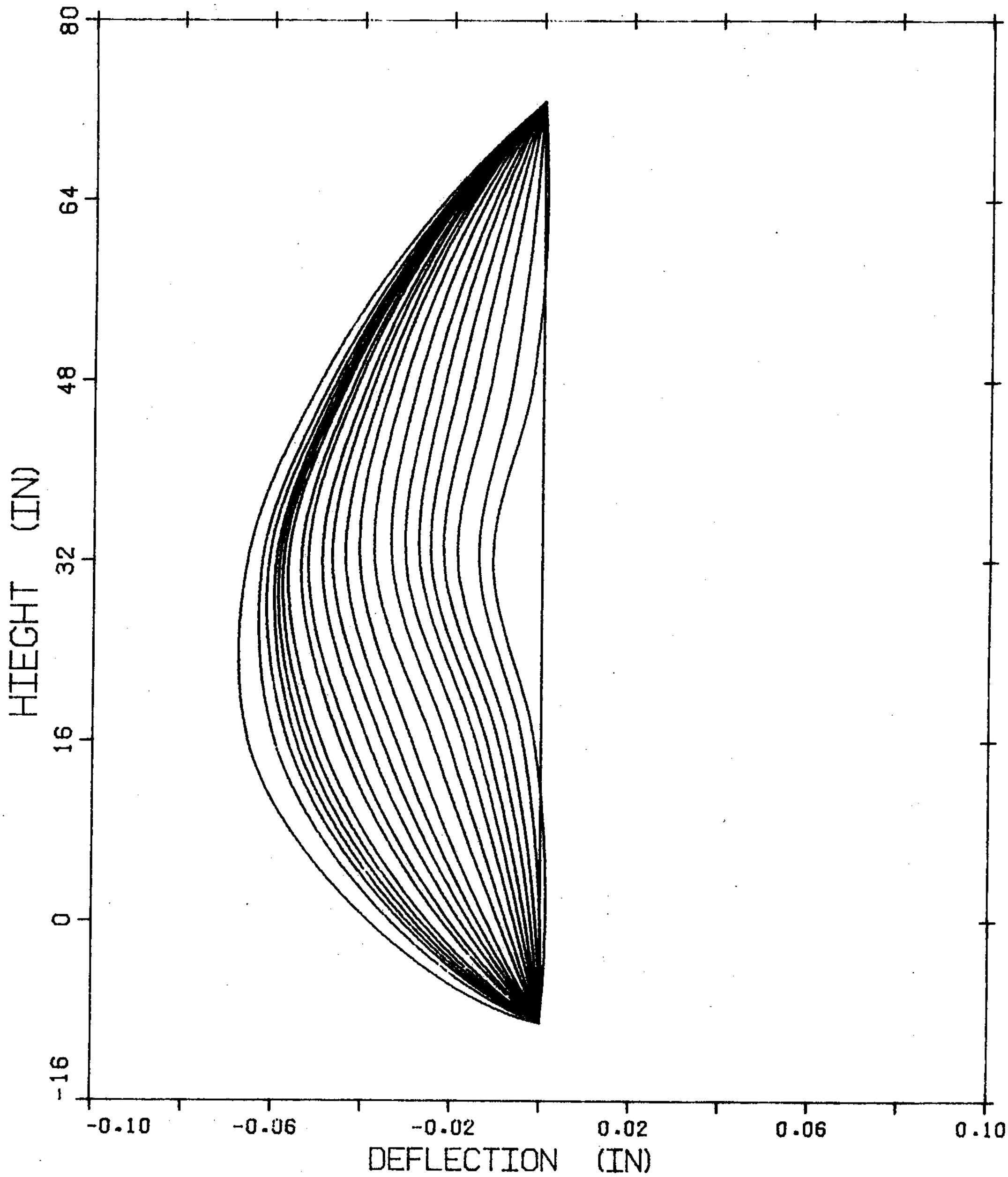

FIGURE B2.10 DEFLECTED SHAPE OF COLUMN B3.1

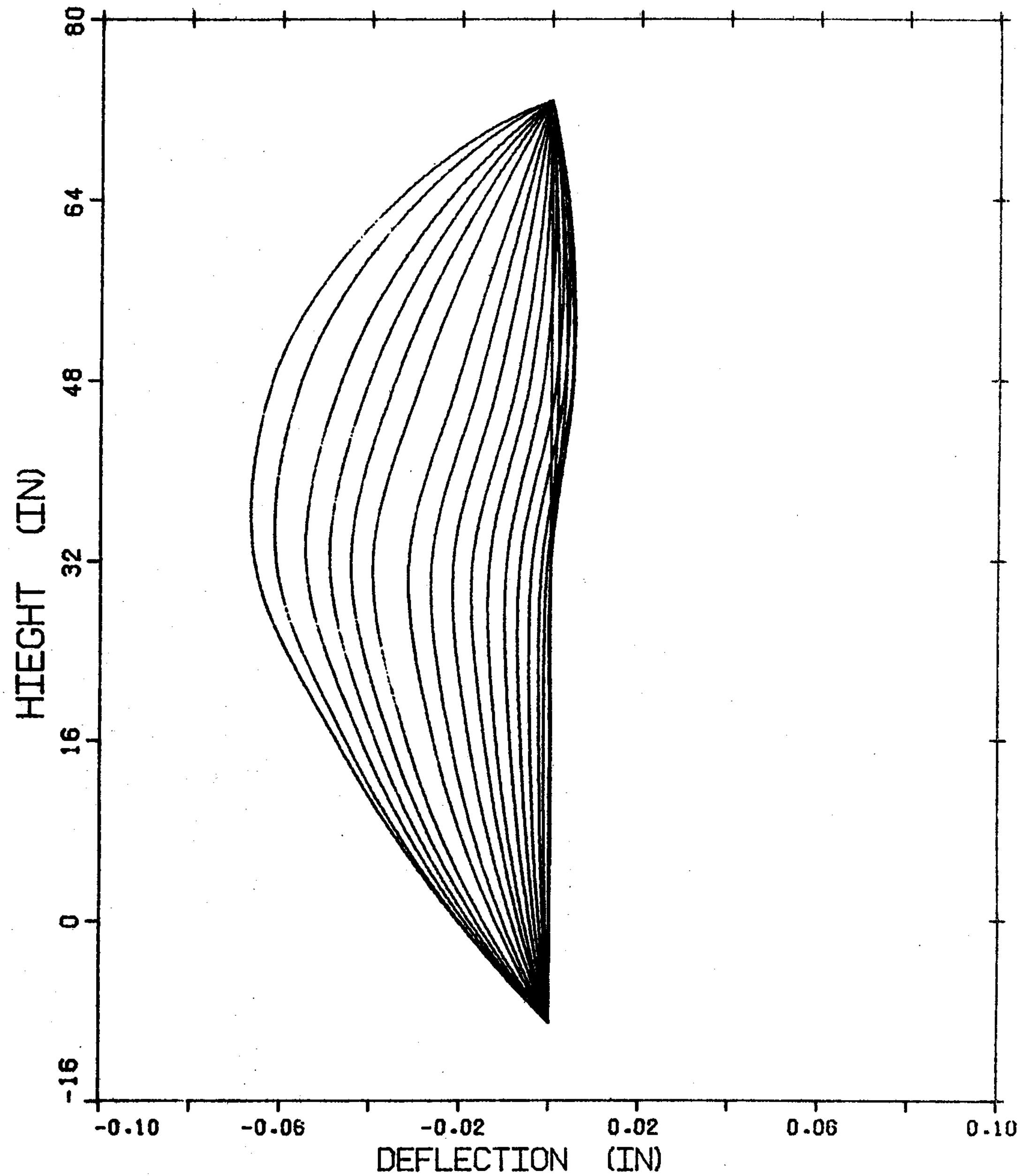

FIGURE B2.11 DEFLECTED SHAPE OF COLUMN B3.2

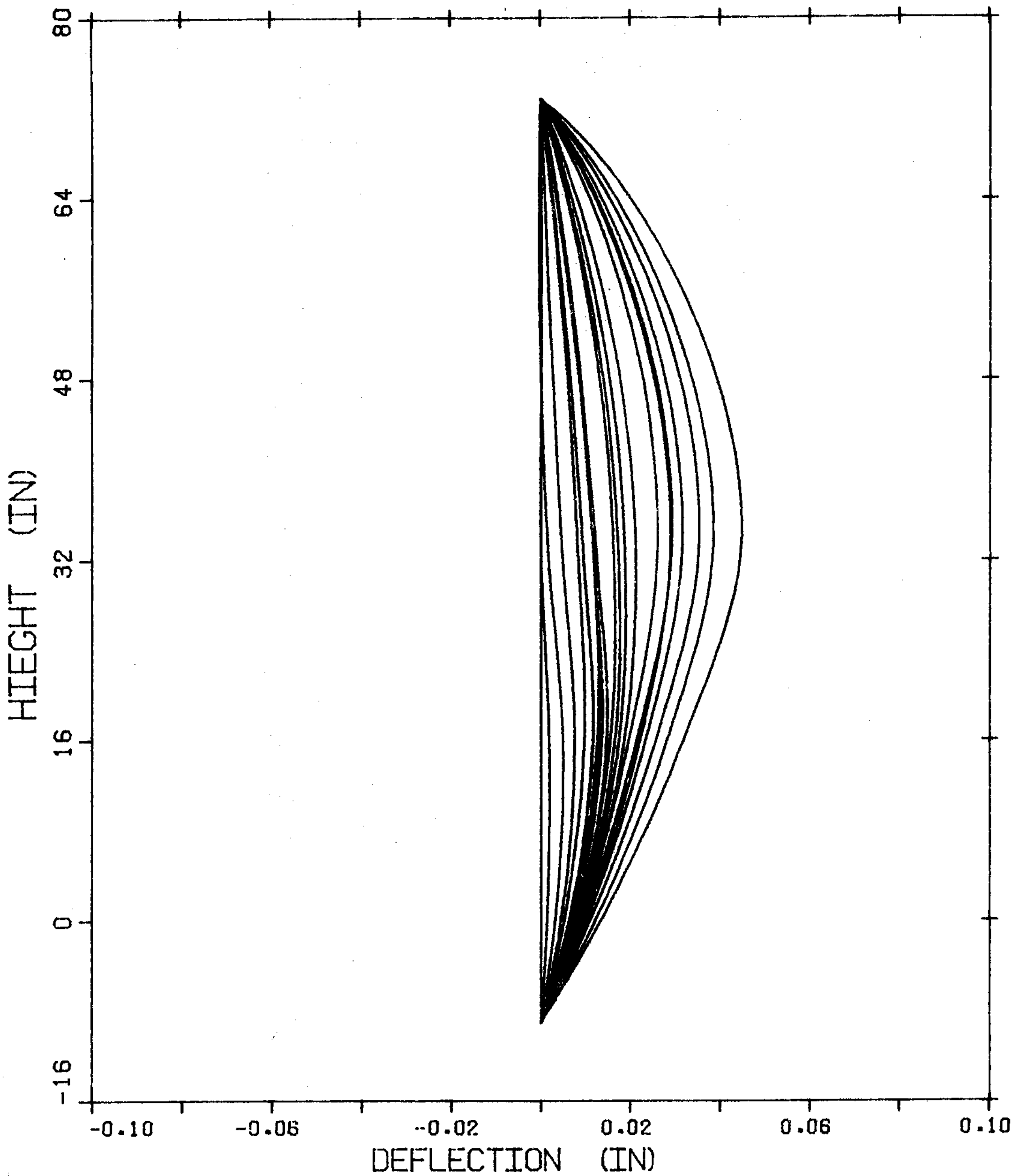

FIGURE B2.12 DEFLECTED SHAPE OF COLUMN B3.3

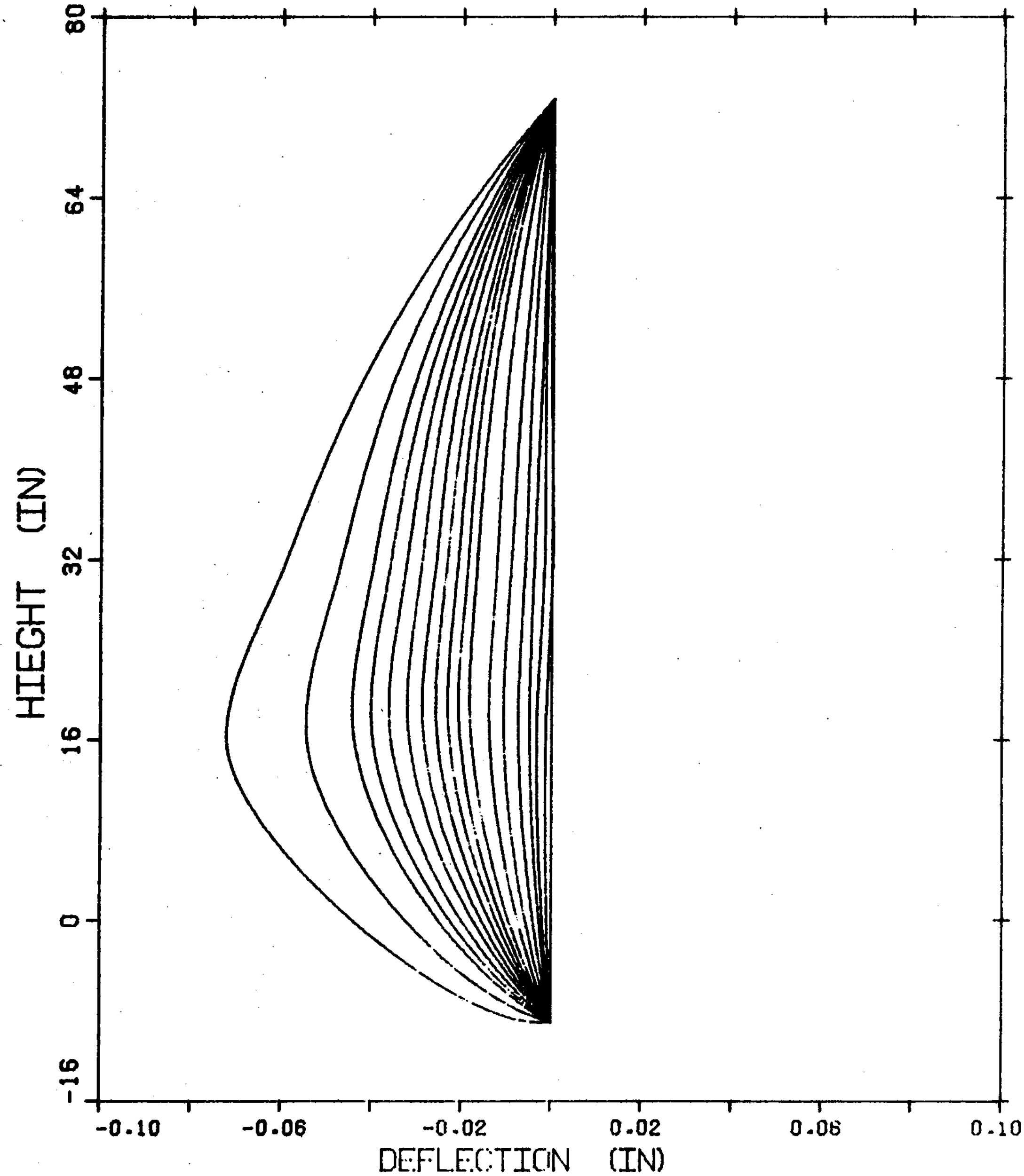

FIGURE B2.13 DEFLECTED SHAPE OF COLUMN C1.1

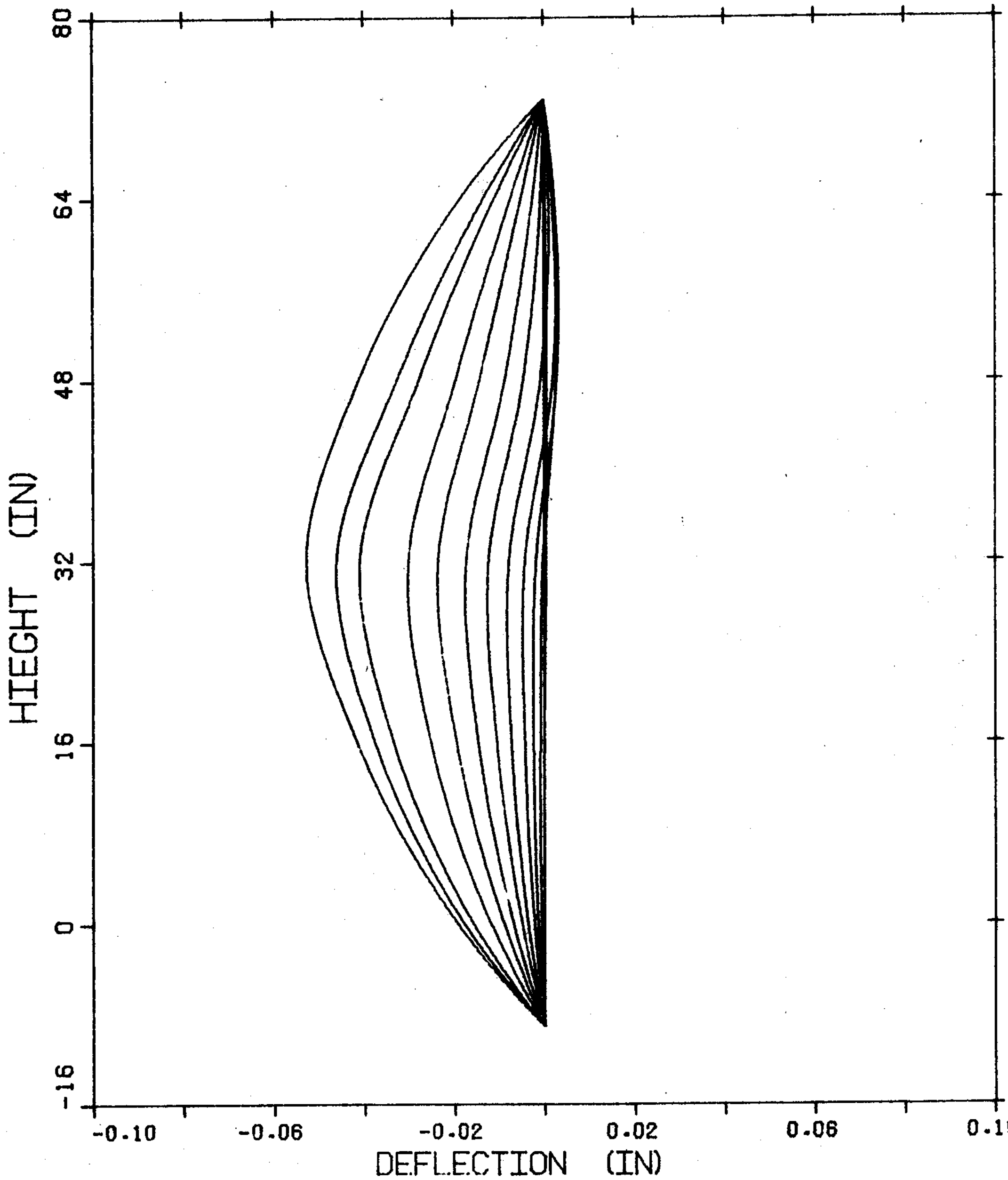

FIGURE B2.14 DEFLECTED SHAPE OF COLUMN C1.2

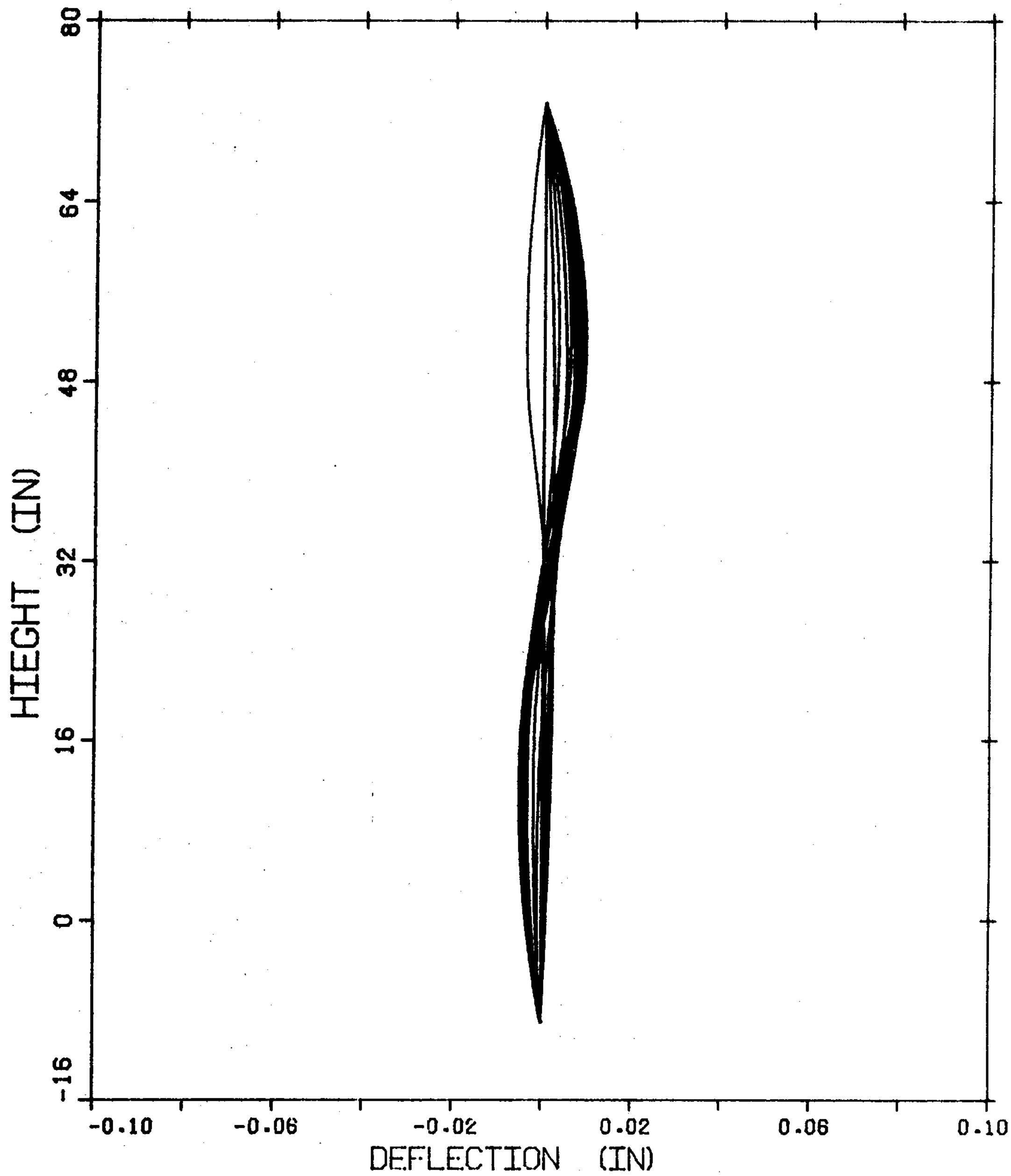

FIGURE B2.15 DEFLECTED SHAPE OF COLUMN C1.3

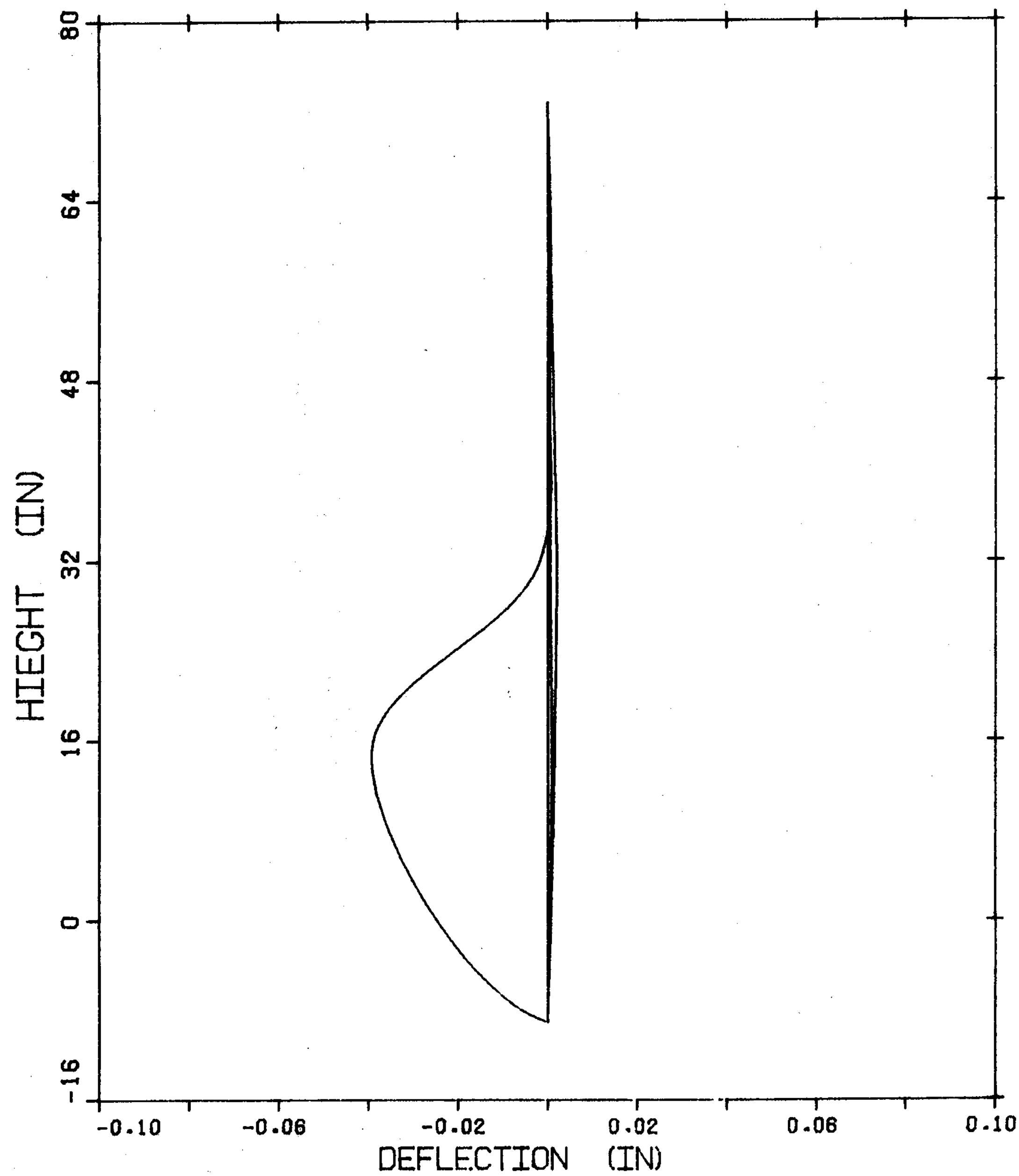

FIGURE B2.16 DEFLECTED SHAPE OF COLUMN C2.1

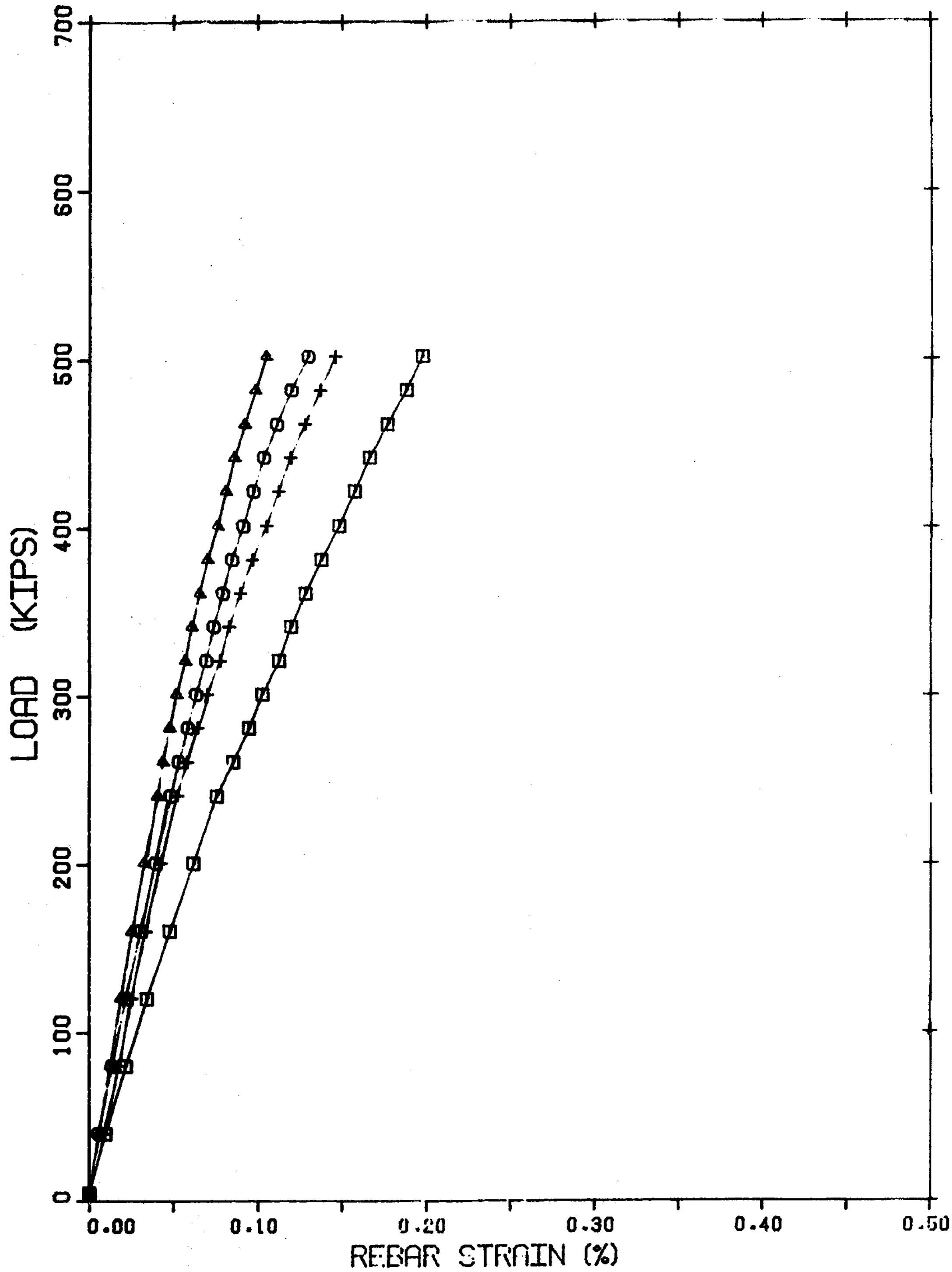

FIGURE B3.1 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A1.1

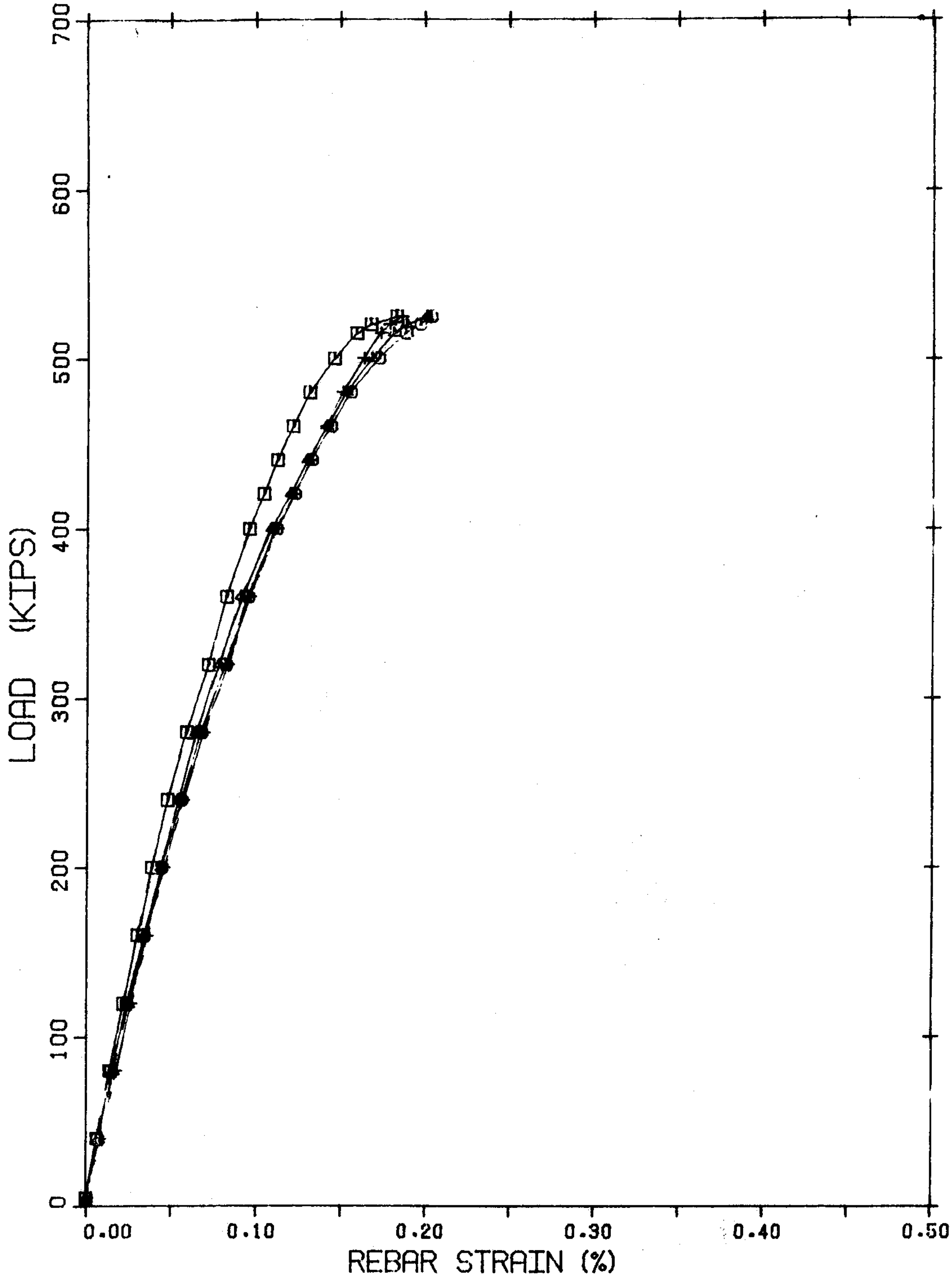

FIGURE B3.2 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A1.2

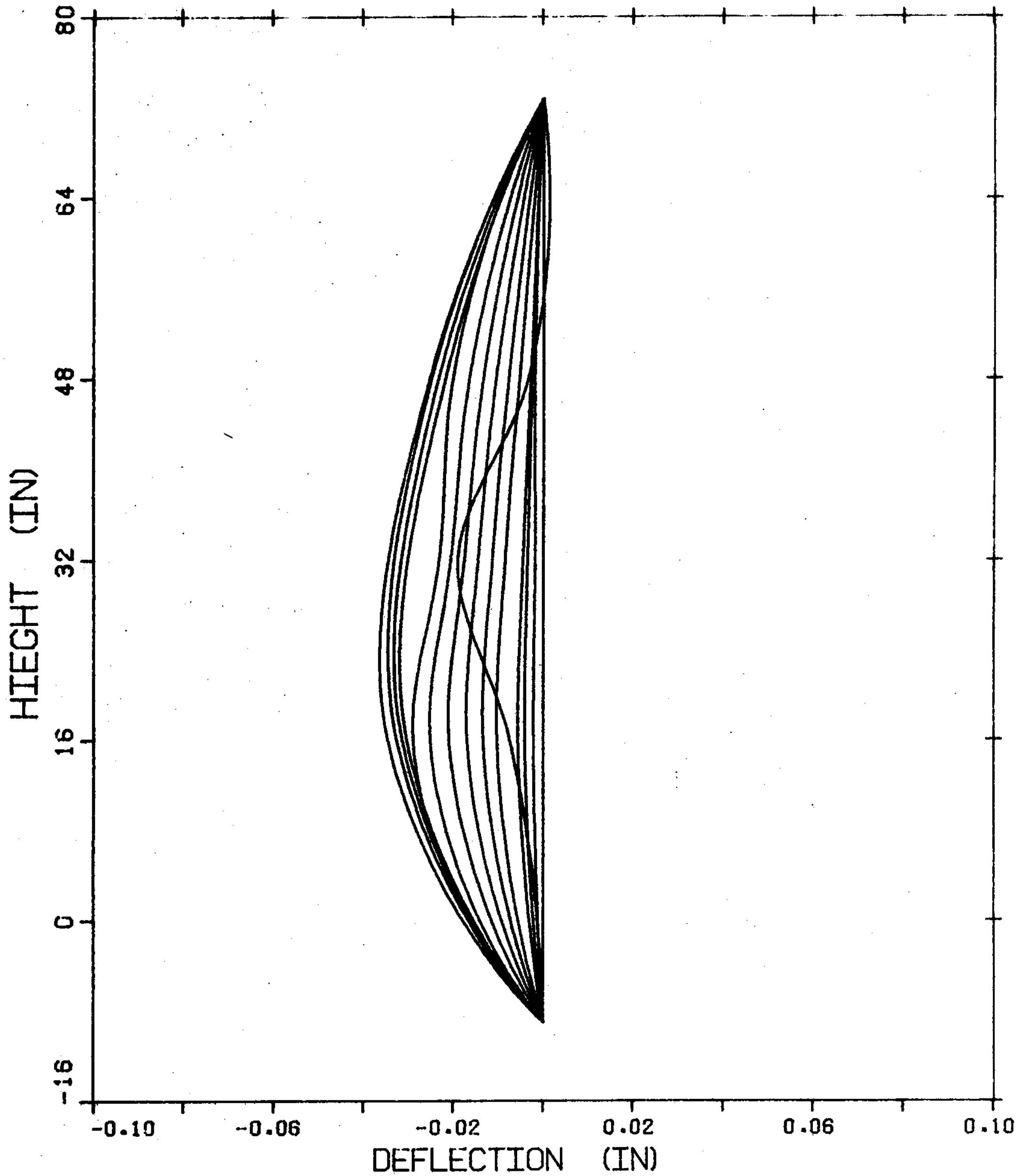

FIGURE B2.4 DEFLECTED SHAPE OF COLUMN B1.1

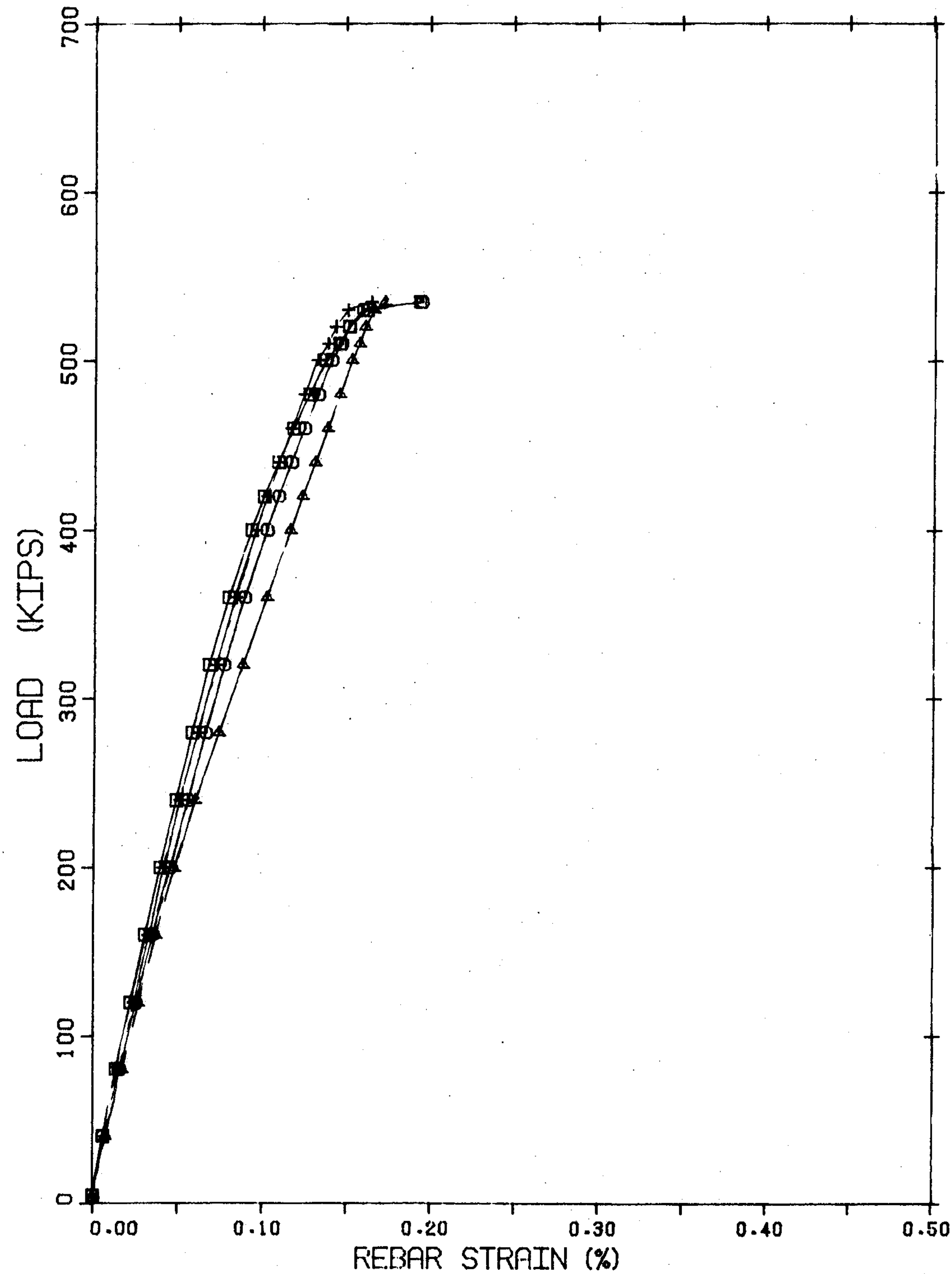

FIGURE B3.3 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A1.3

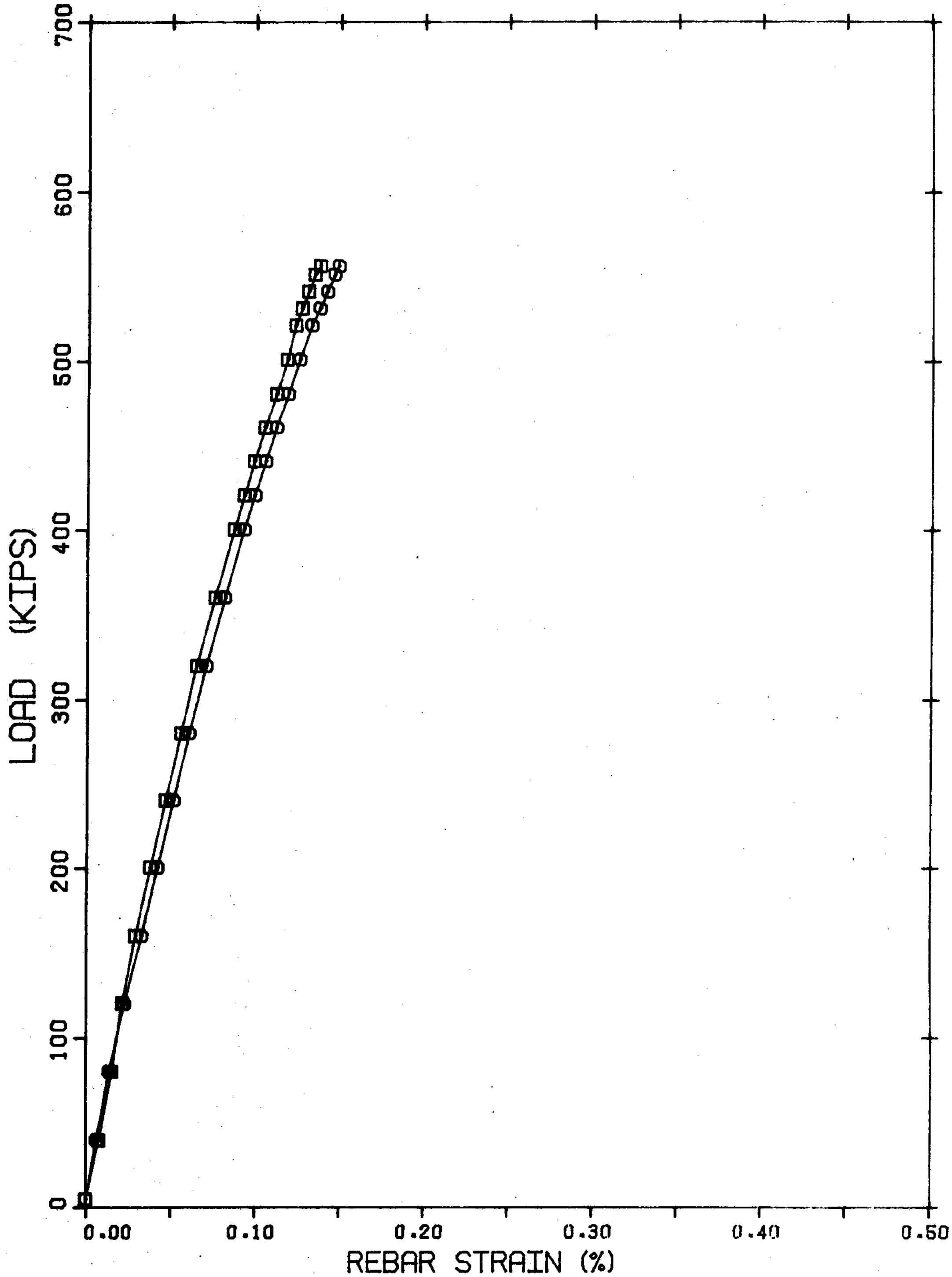

FIGURE B3.4 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A2.1

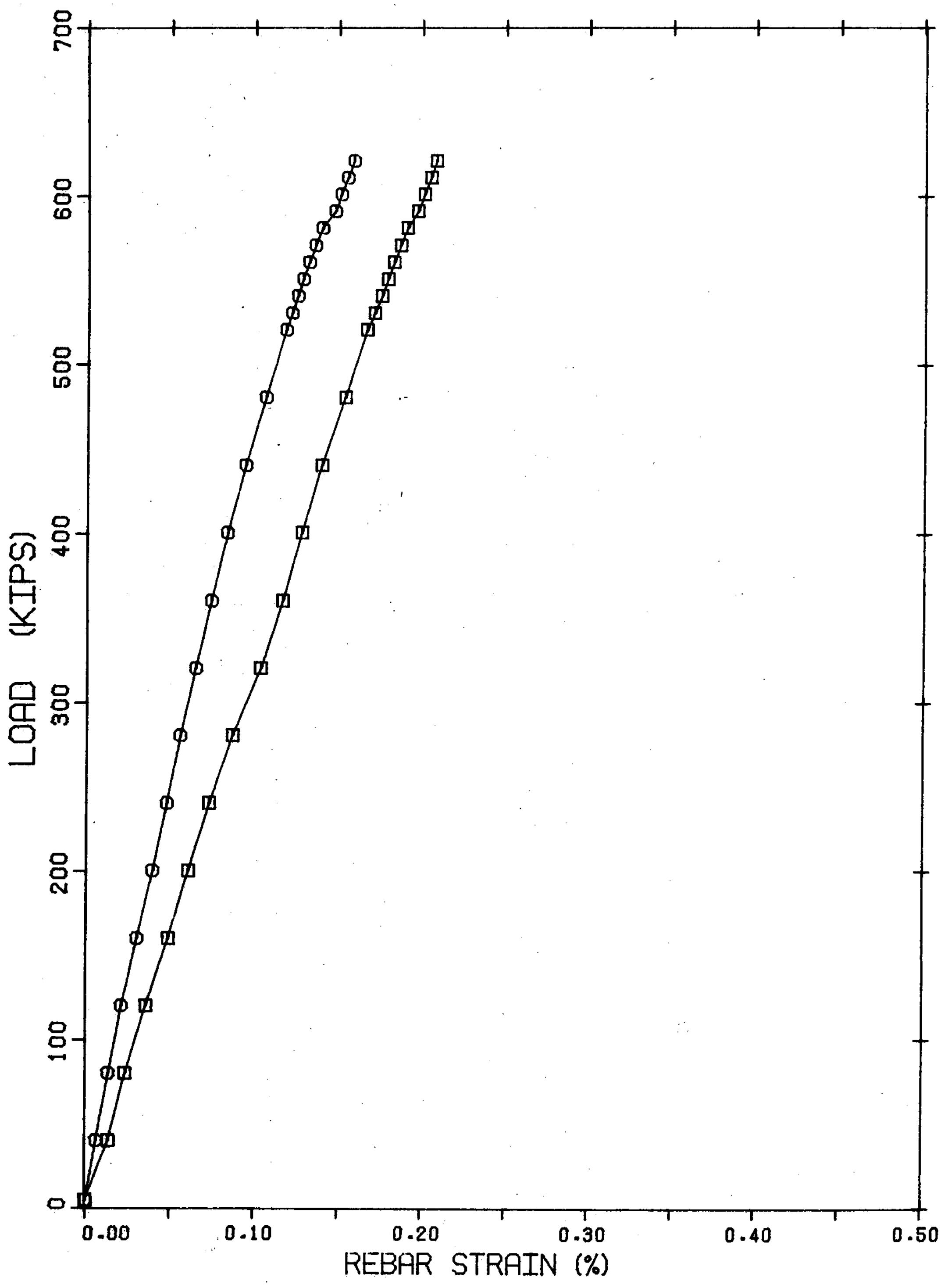

FIGURE B3.5 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A2.2

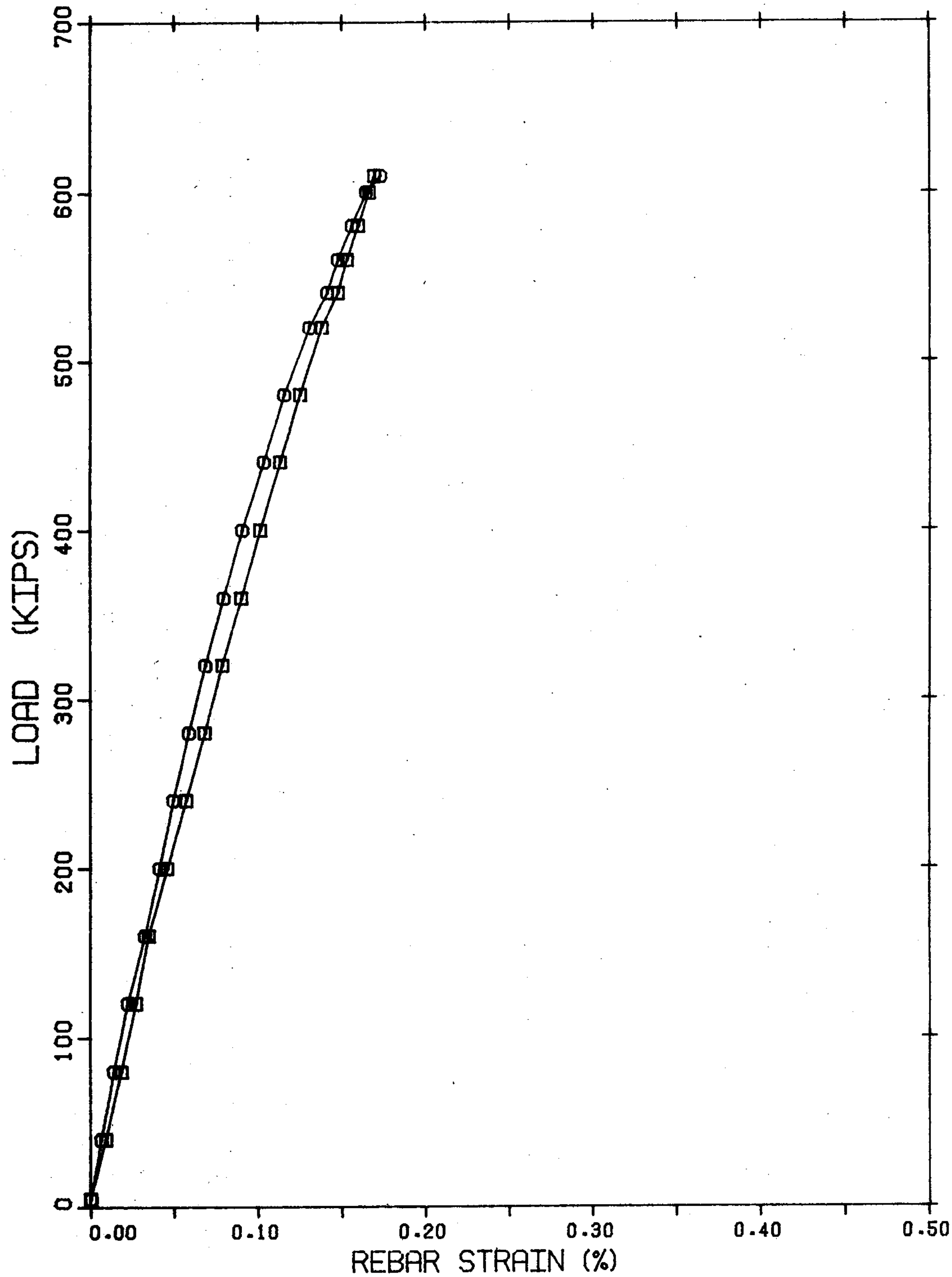

FIGURE B3.6 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A3.1

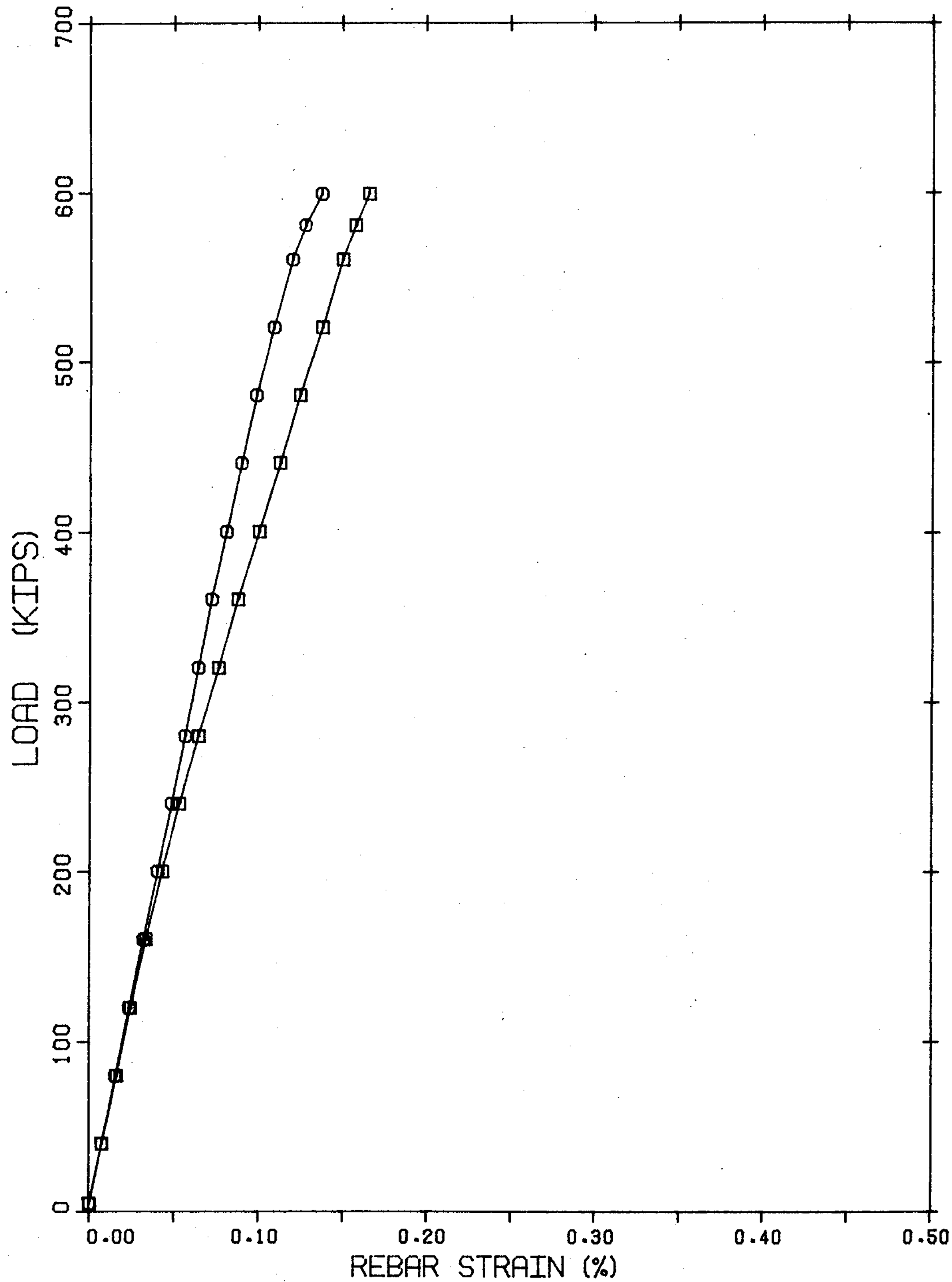

FIGURE B3.7 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A3.2

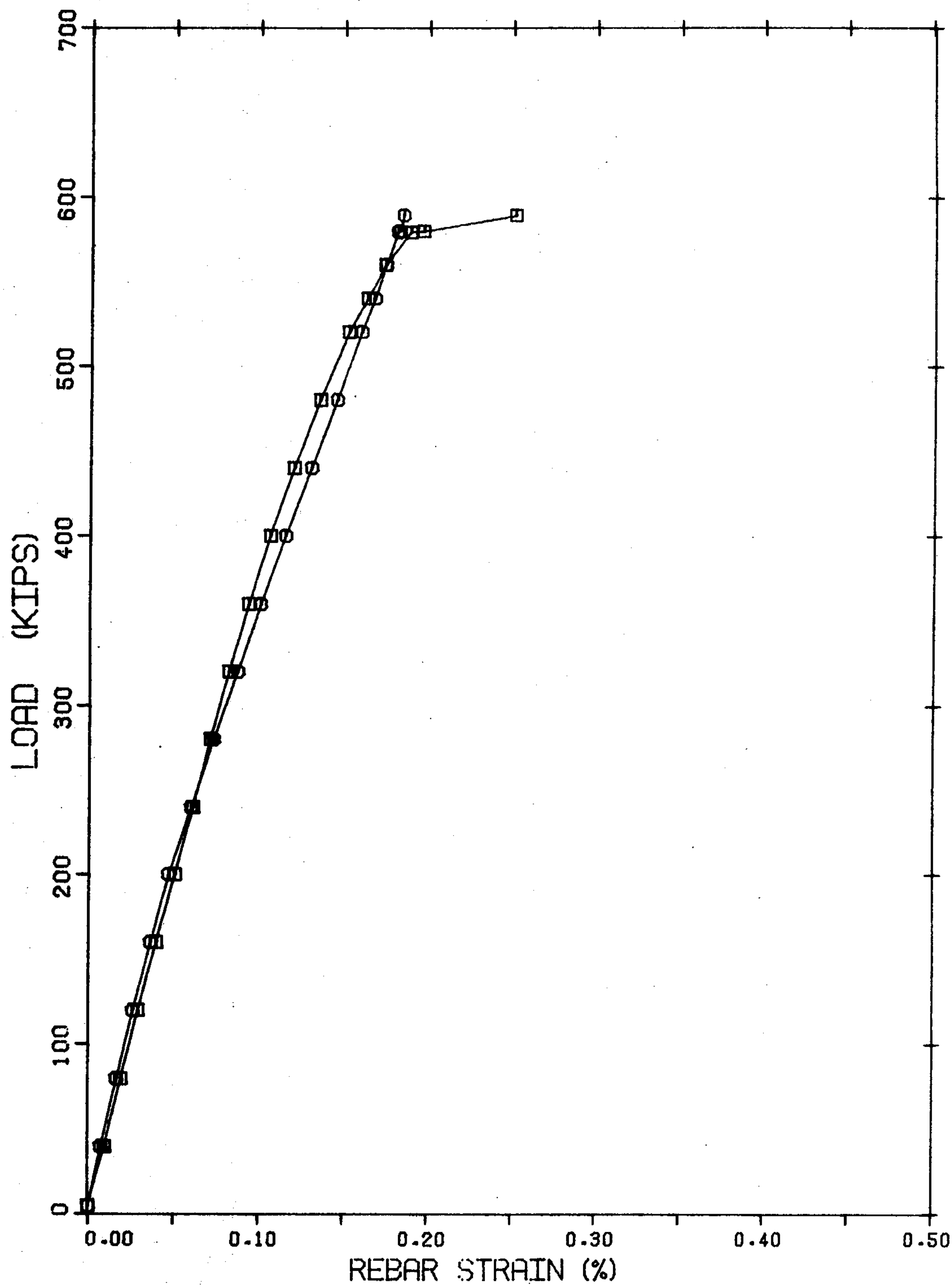

FIGURE B3.8 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A4.1

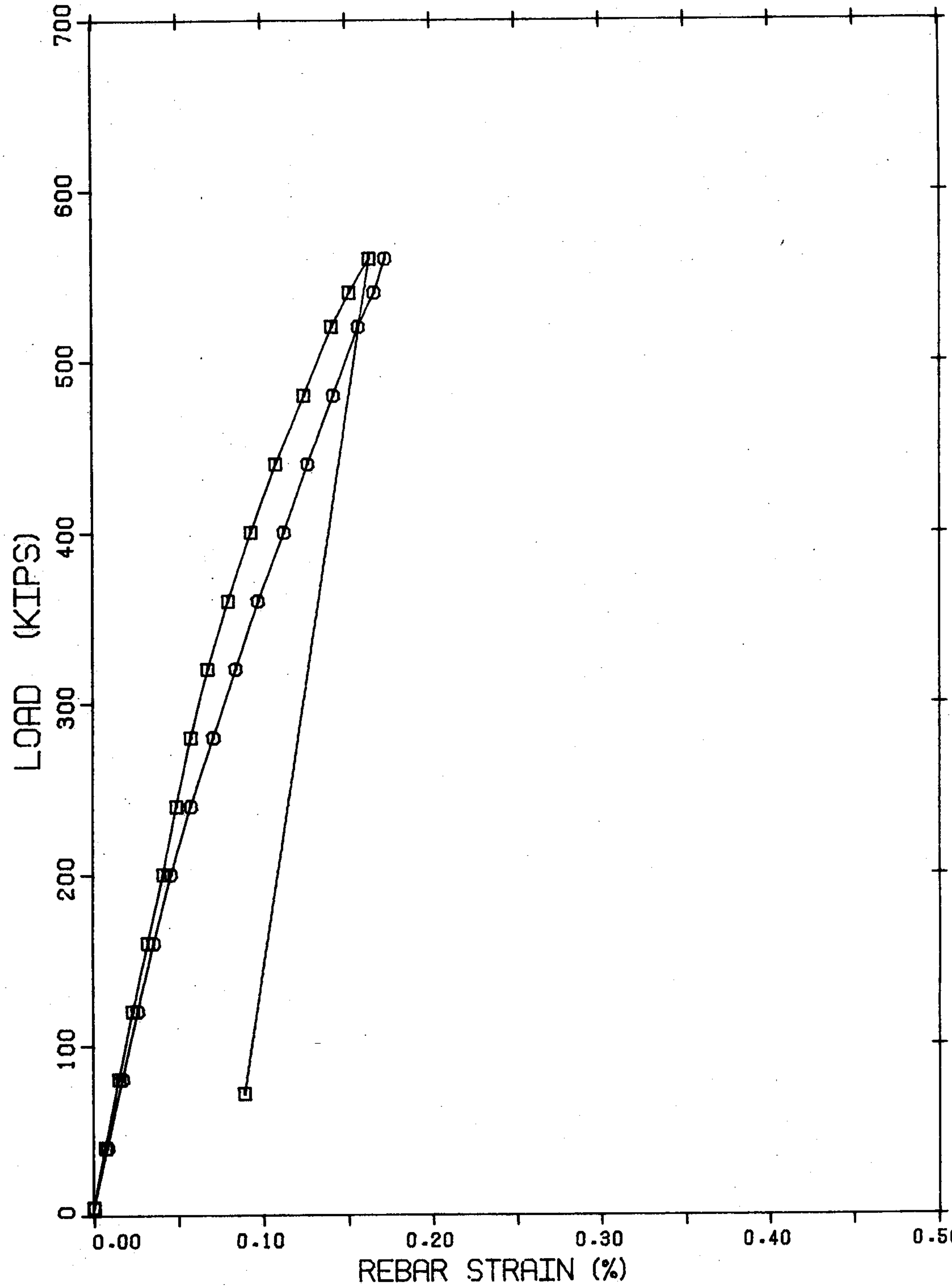

FIGURE B3.9 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN A4.2

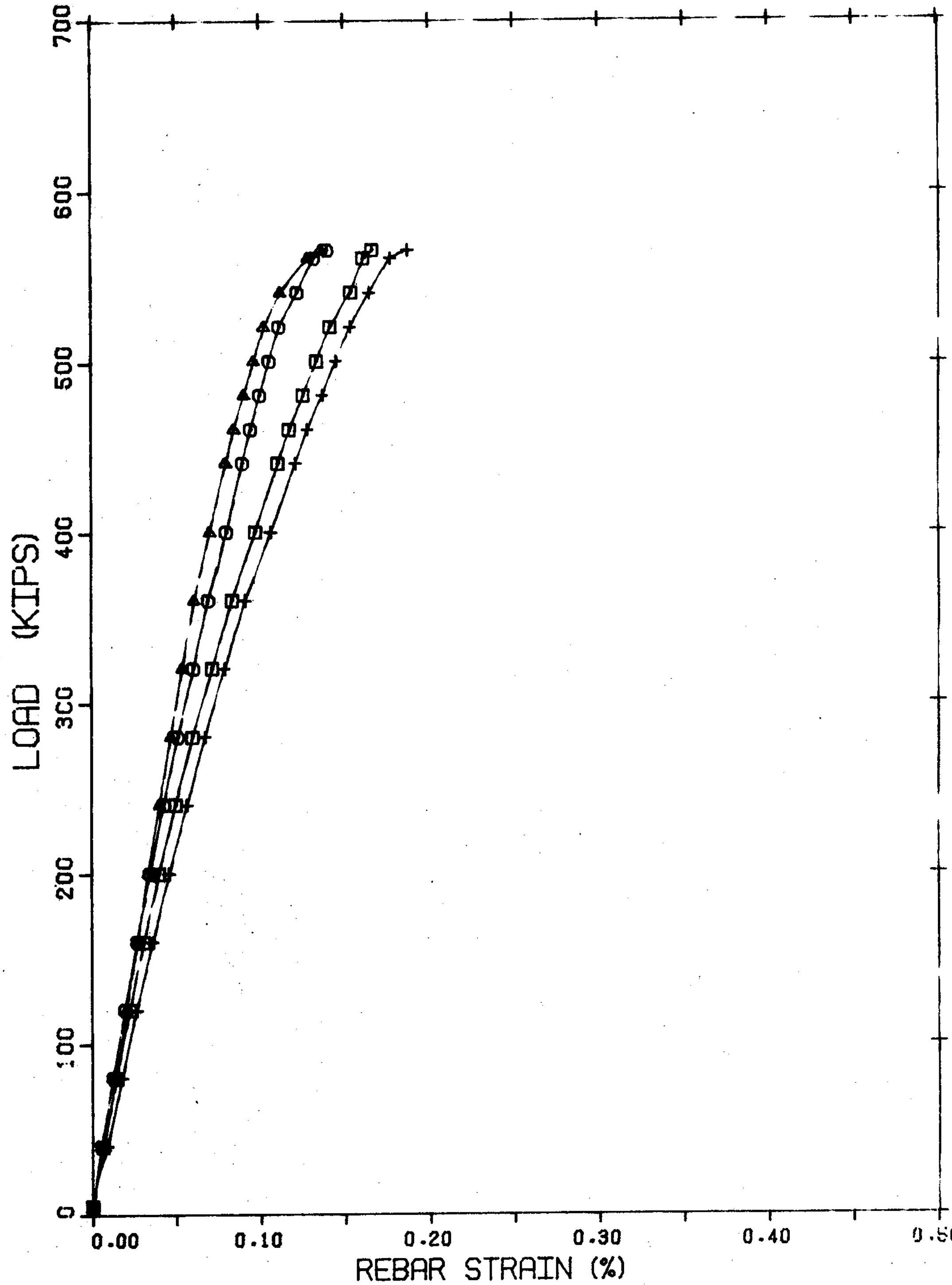

FIGURE B3.10 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B2.1

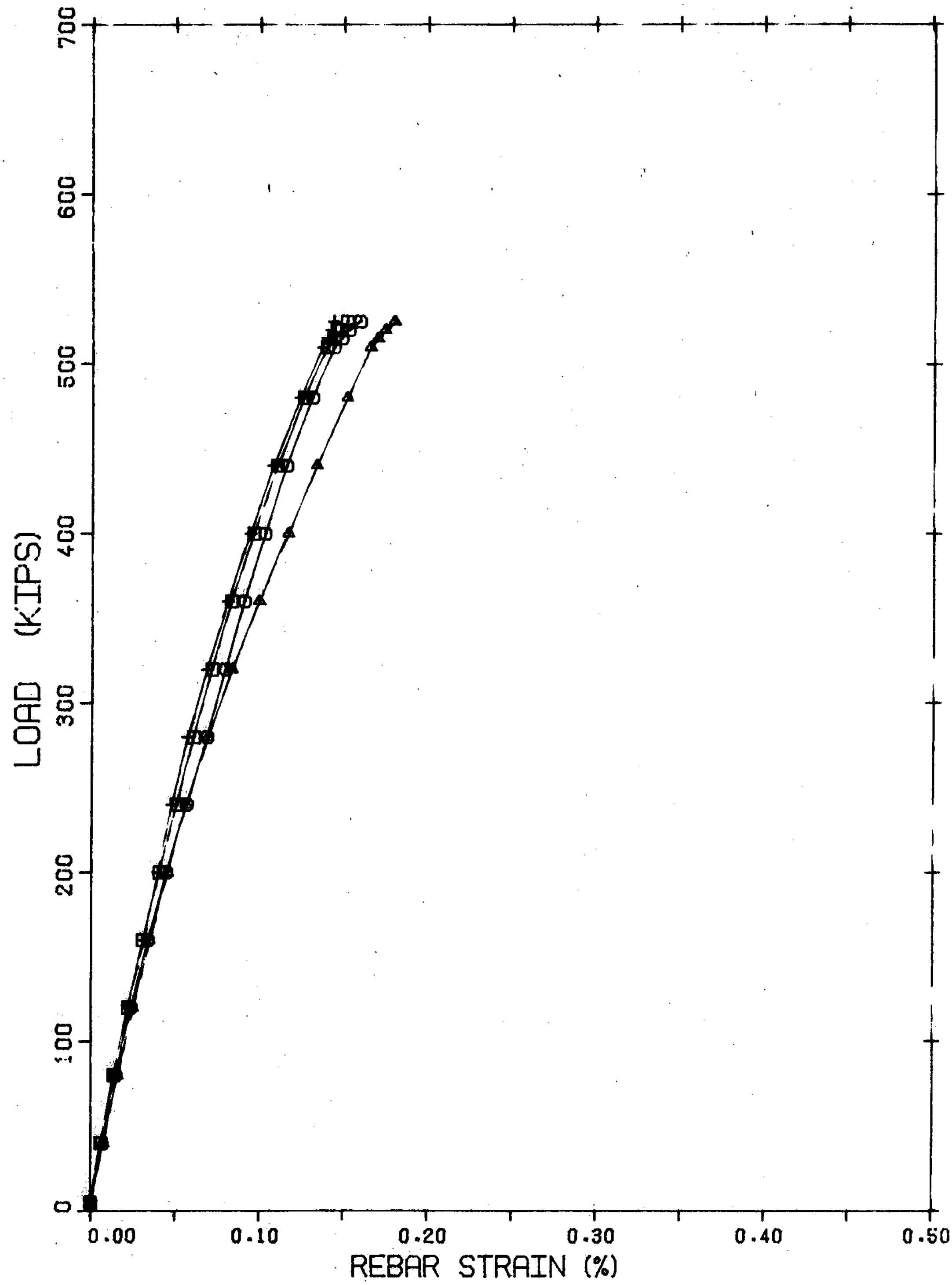

FIGURE B3.11 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B2.2

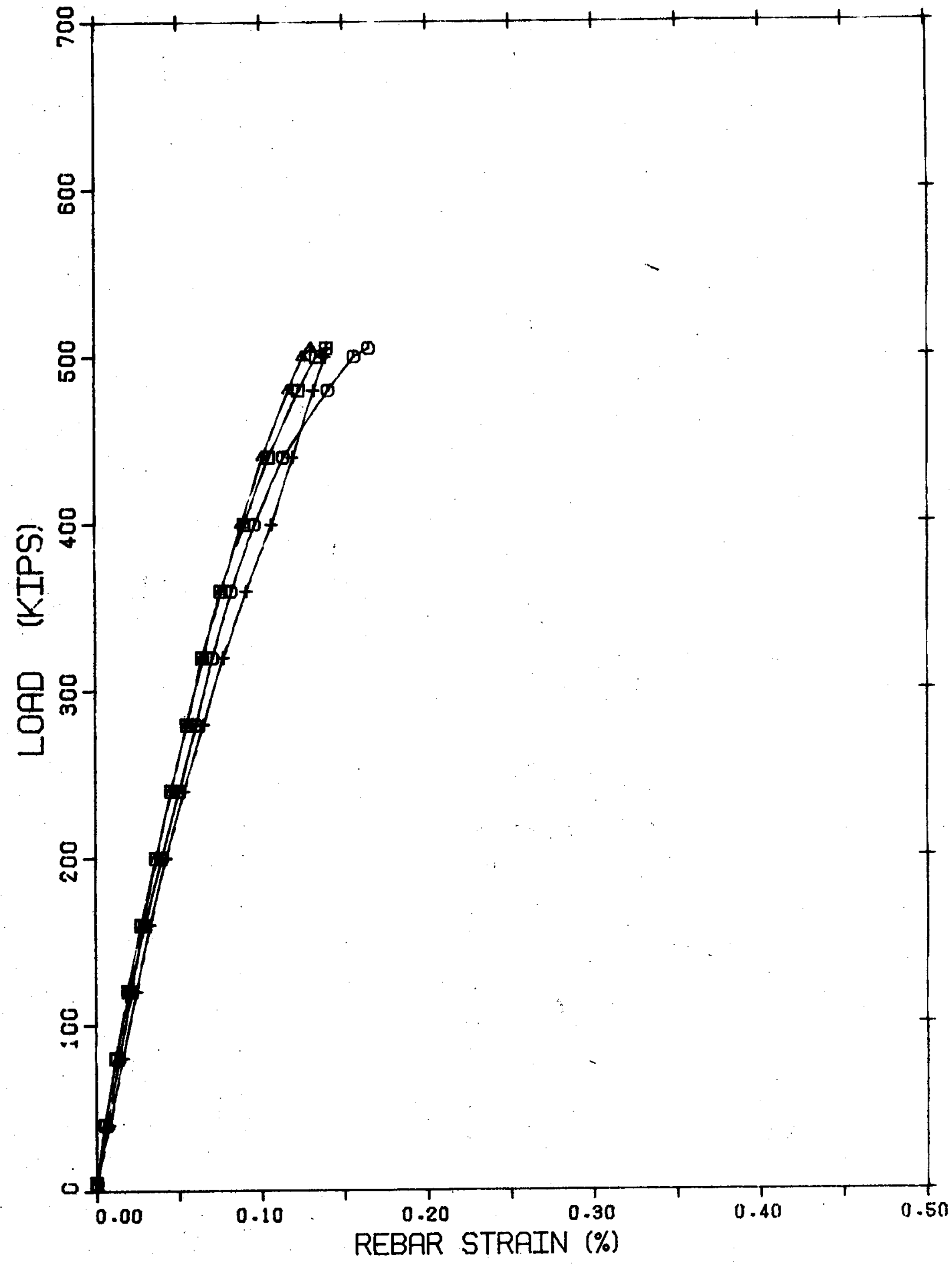

FIGURE B3.12 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B2.3

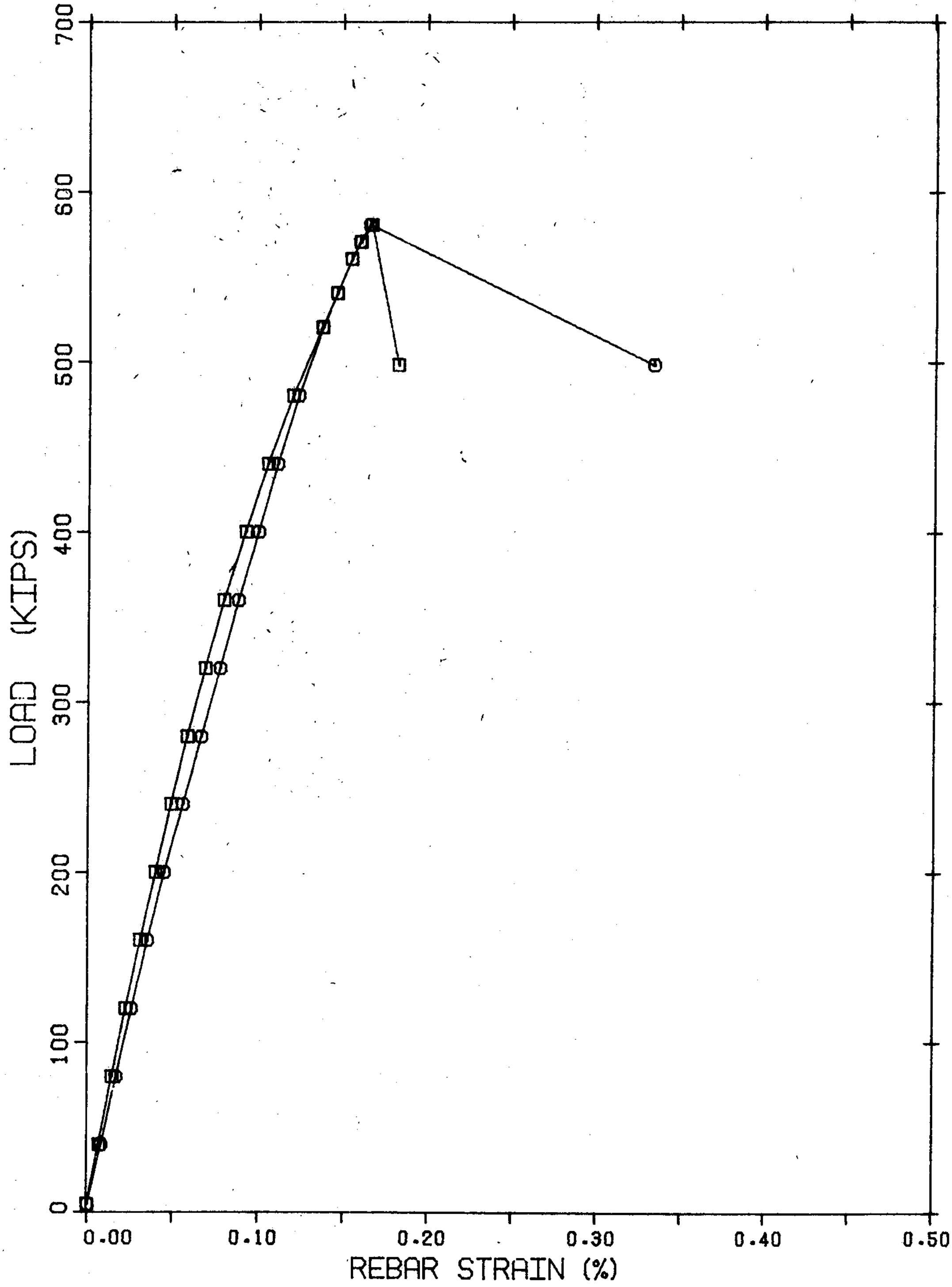

FIGURE B3.13 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B2.4

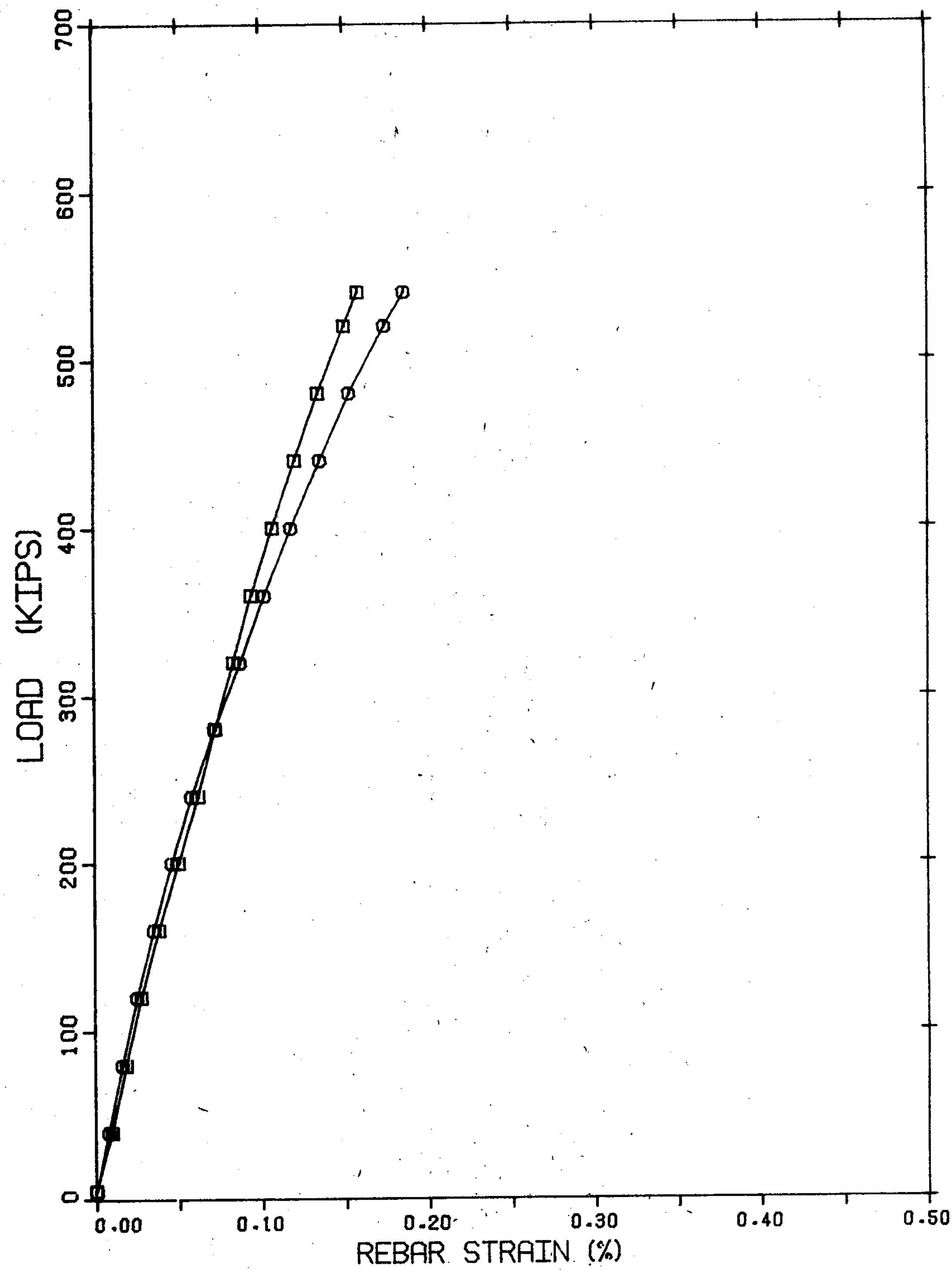

FIGURE B3.14 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B2.5

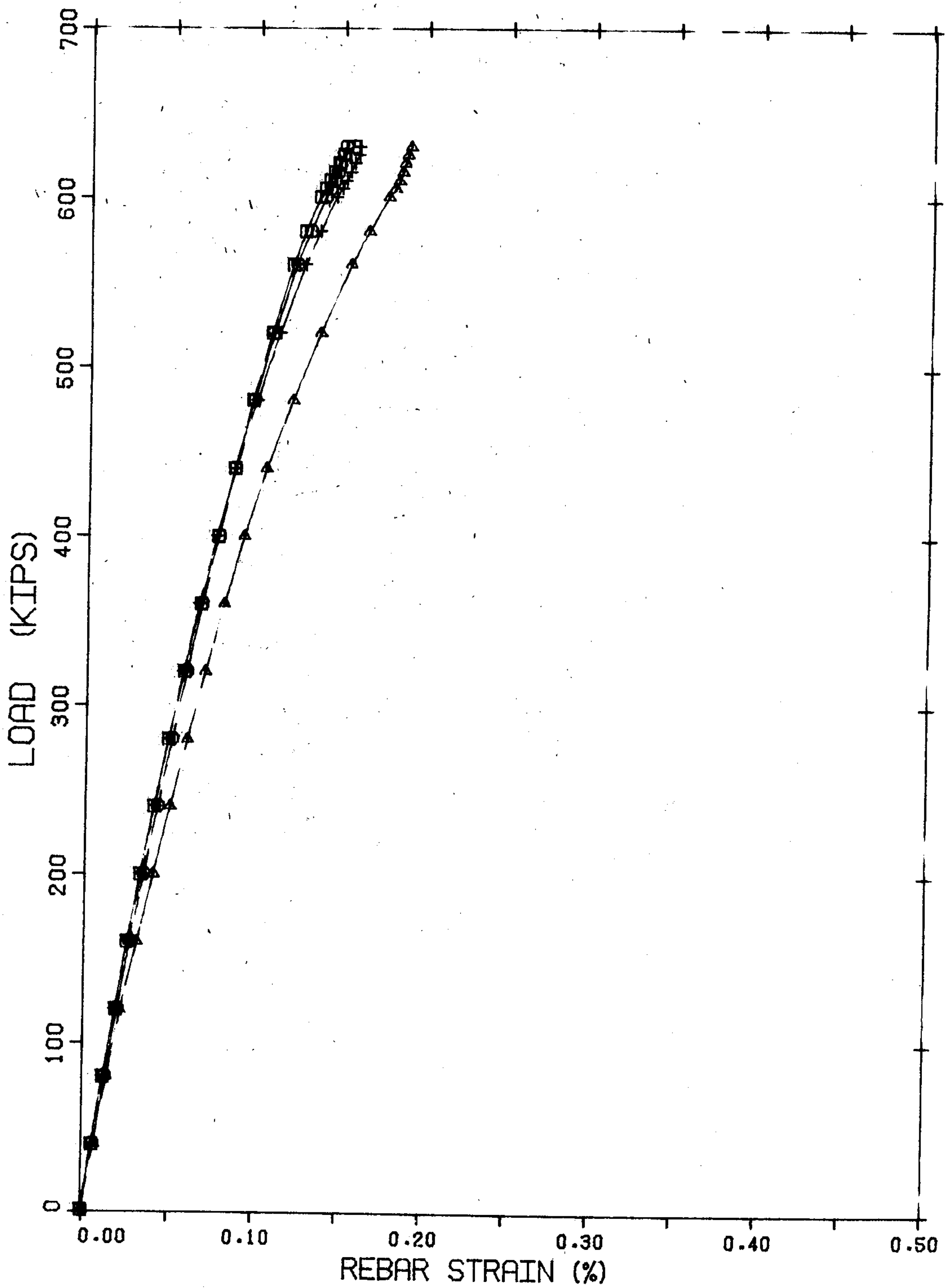

FIGURE B3.15 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B3.1

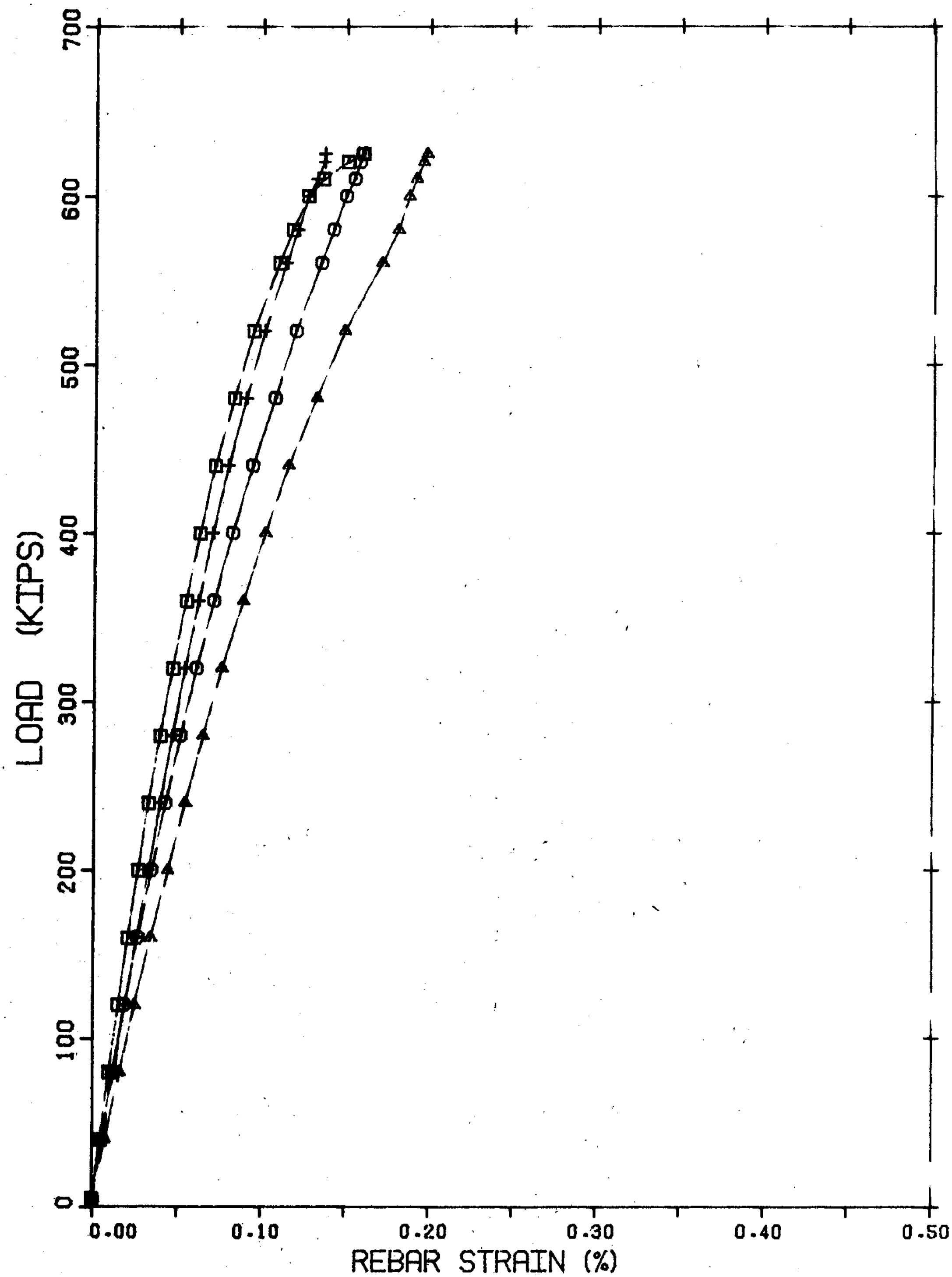

FIGURE B3.16 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B3.2

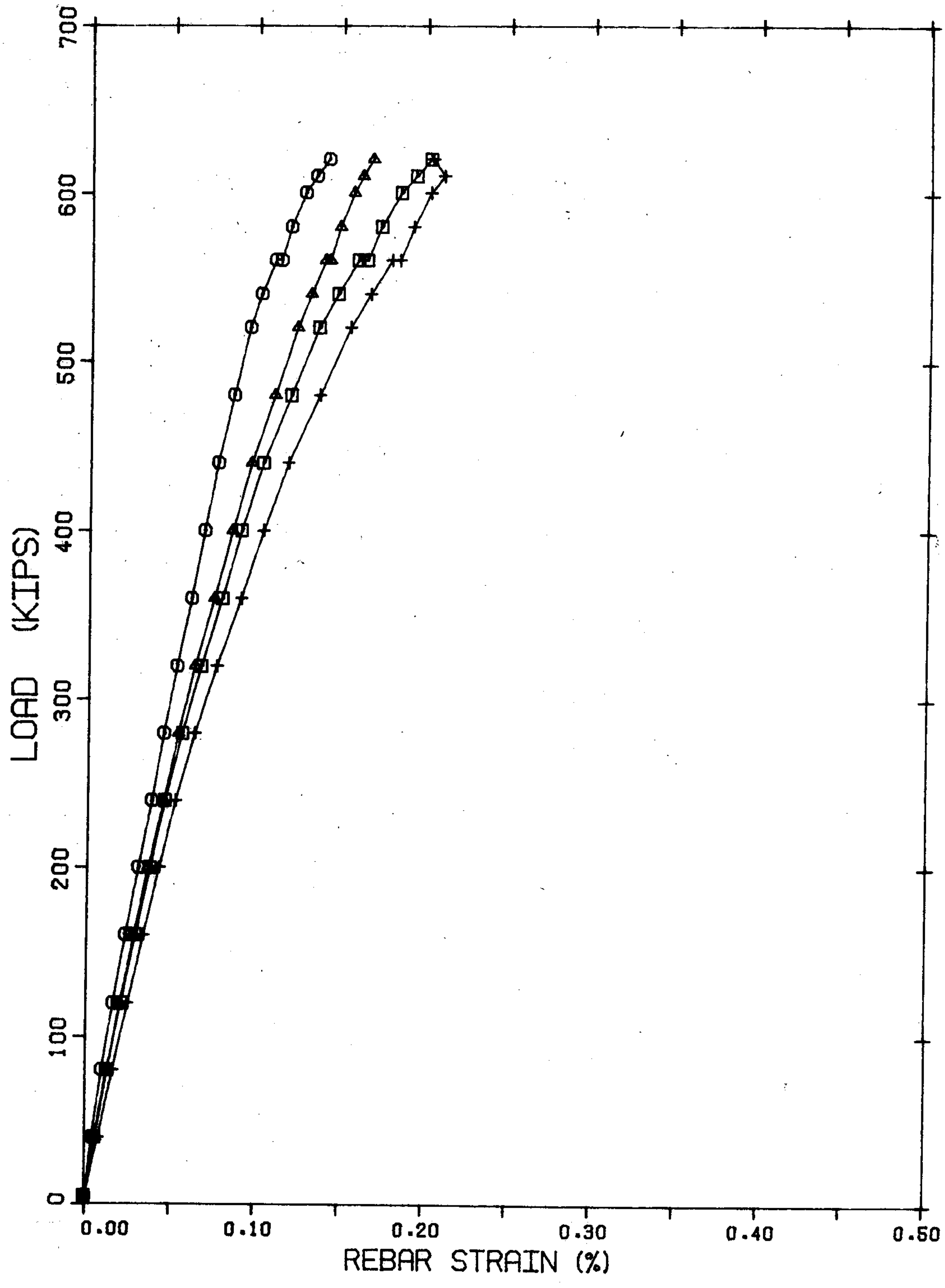

FIGURE B3.17 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B3.3

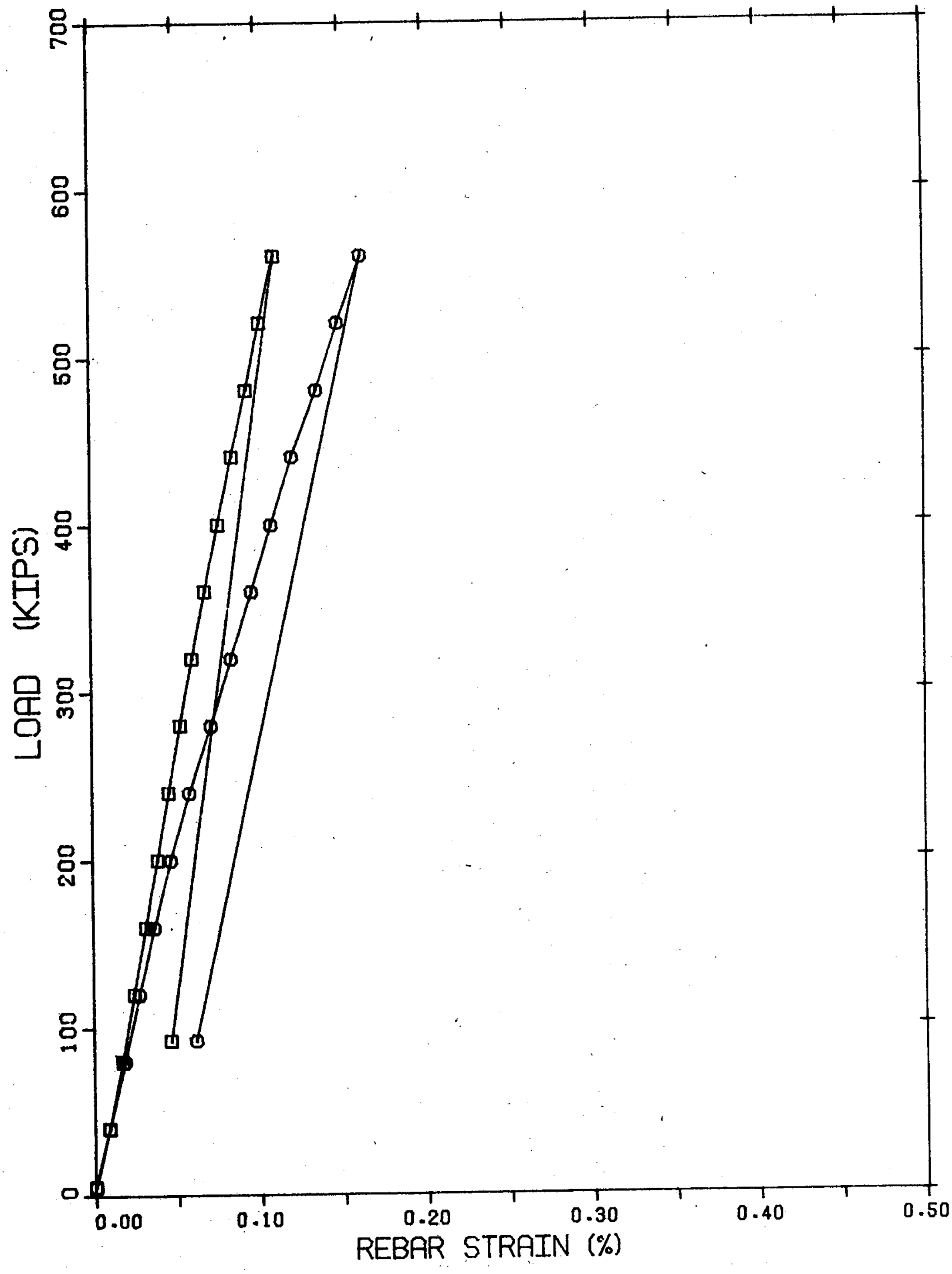

FIGURE B3.18 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B4.1

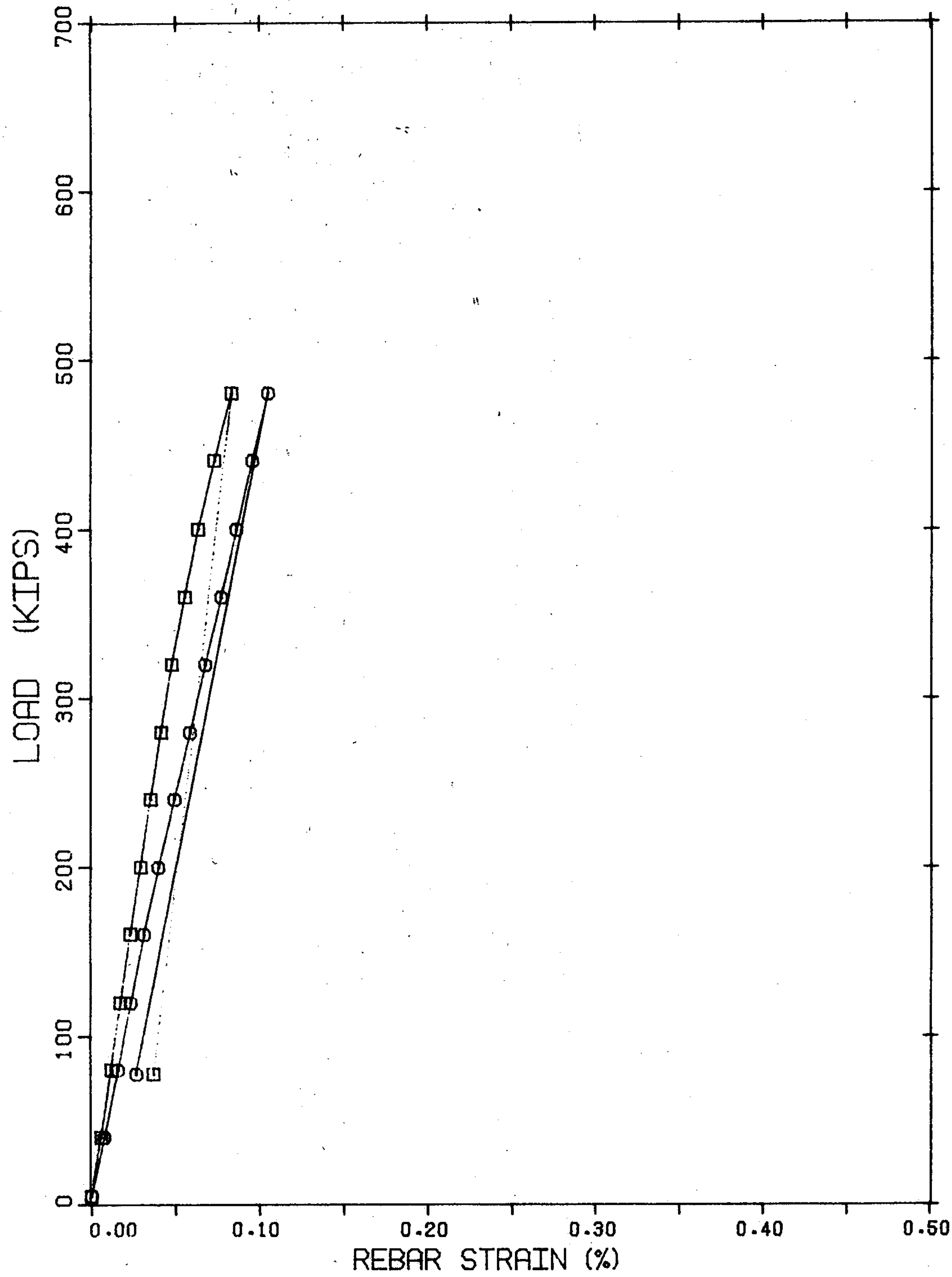

FIGURE B3.19 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B4.2

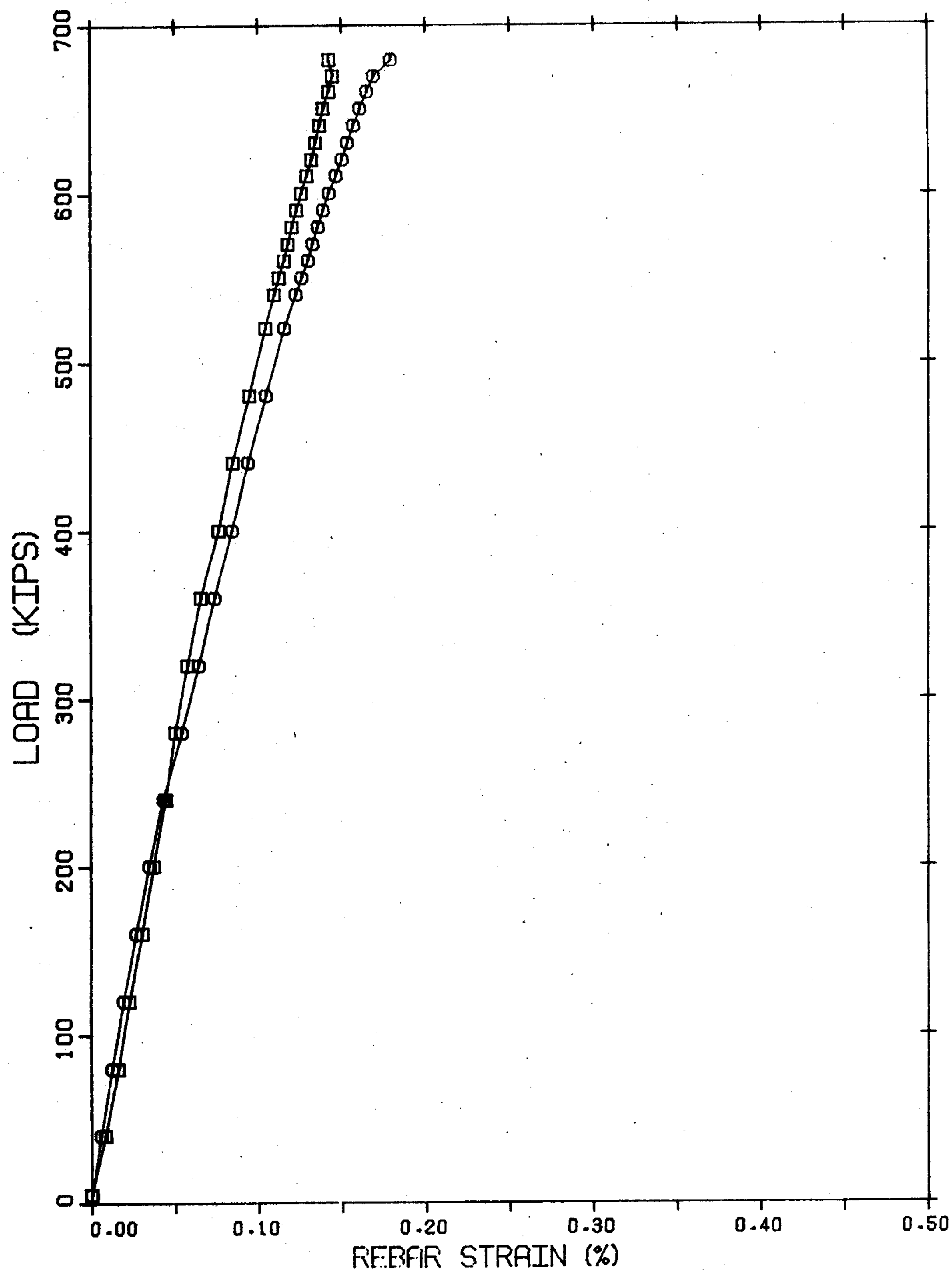

FIGURE B3.20 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B5.1

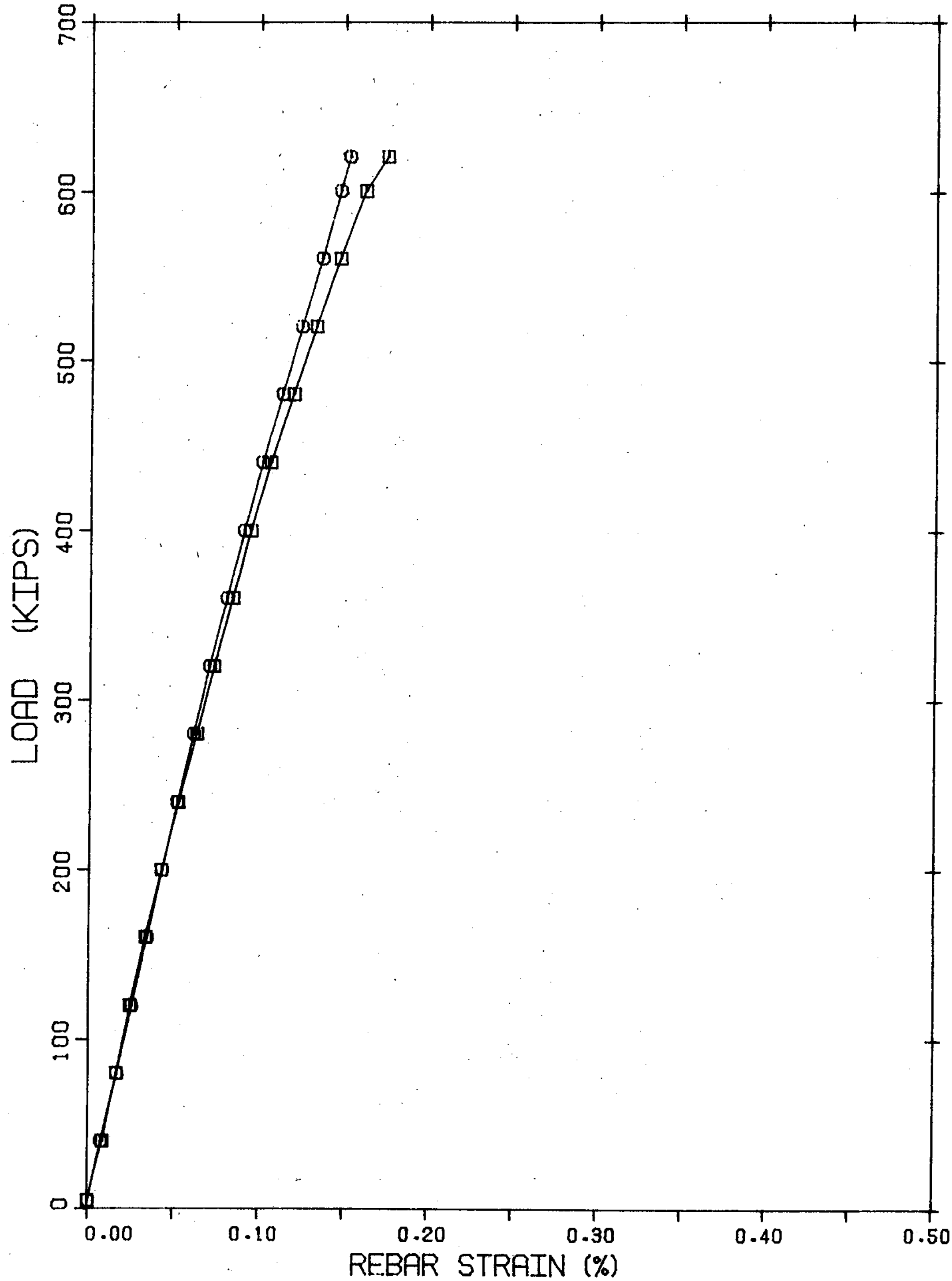

FIGURE B3.21 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B5.2

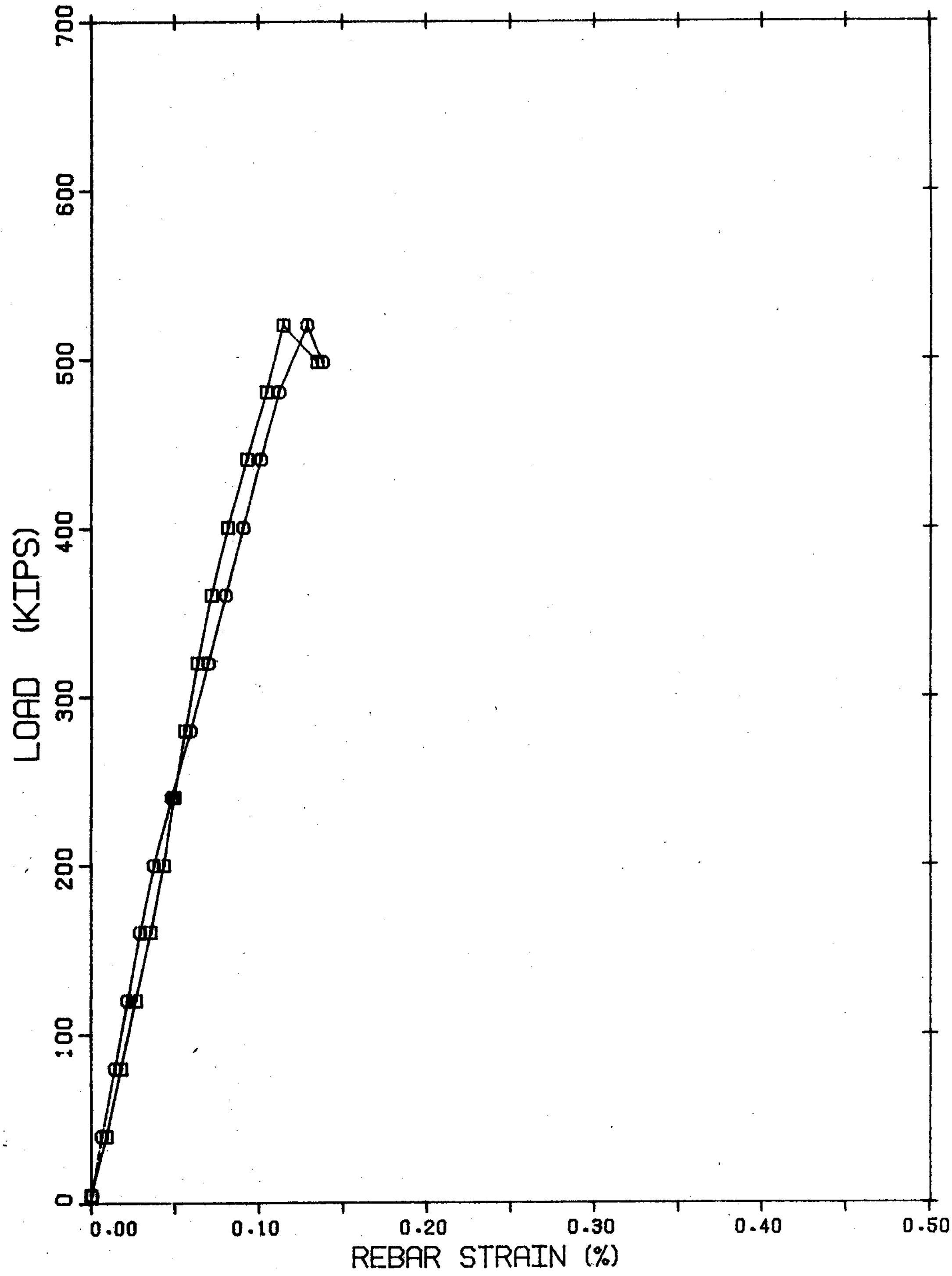

FIGURE B3.22 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B6.1

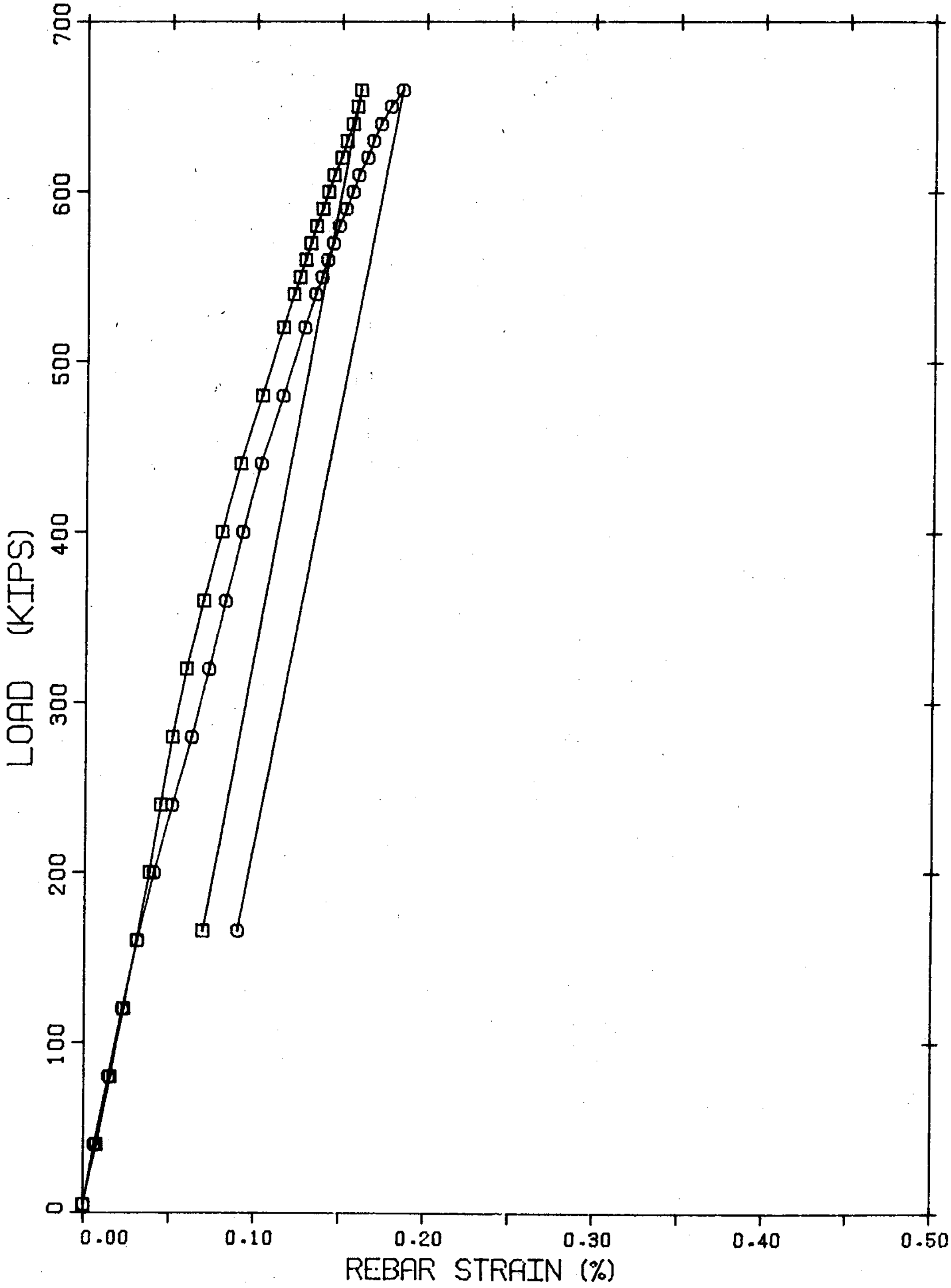

FIGURE B3.23 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B6.2

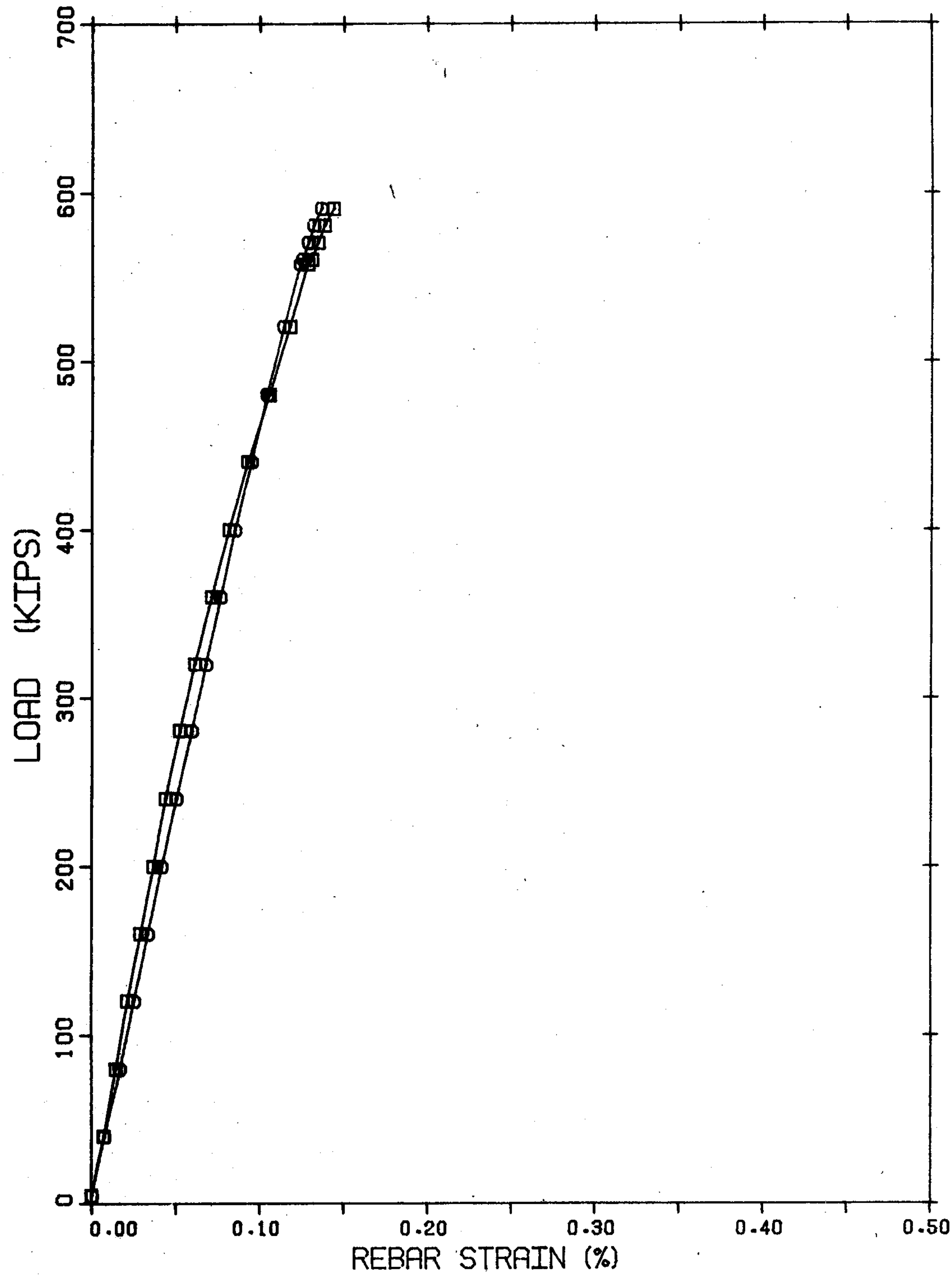

FIGURE B3.24 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B7.1

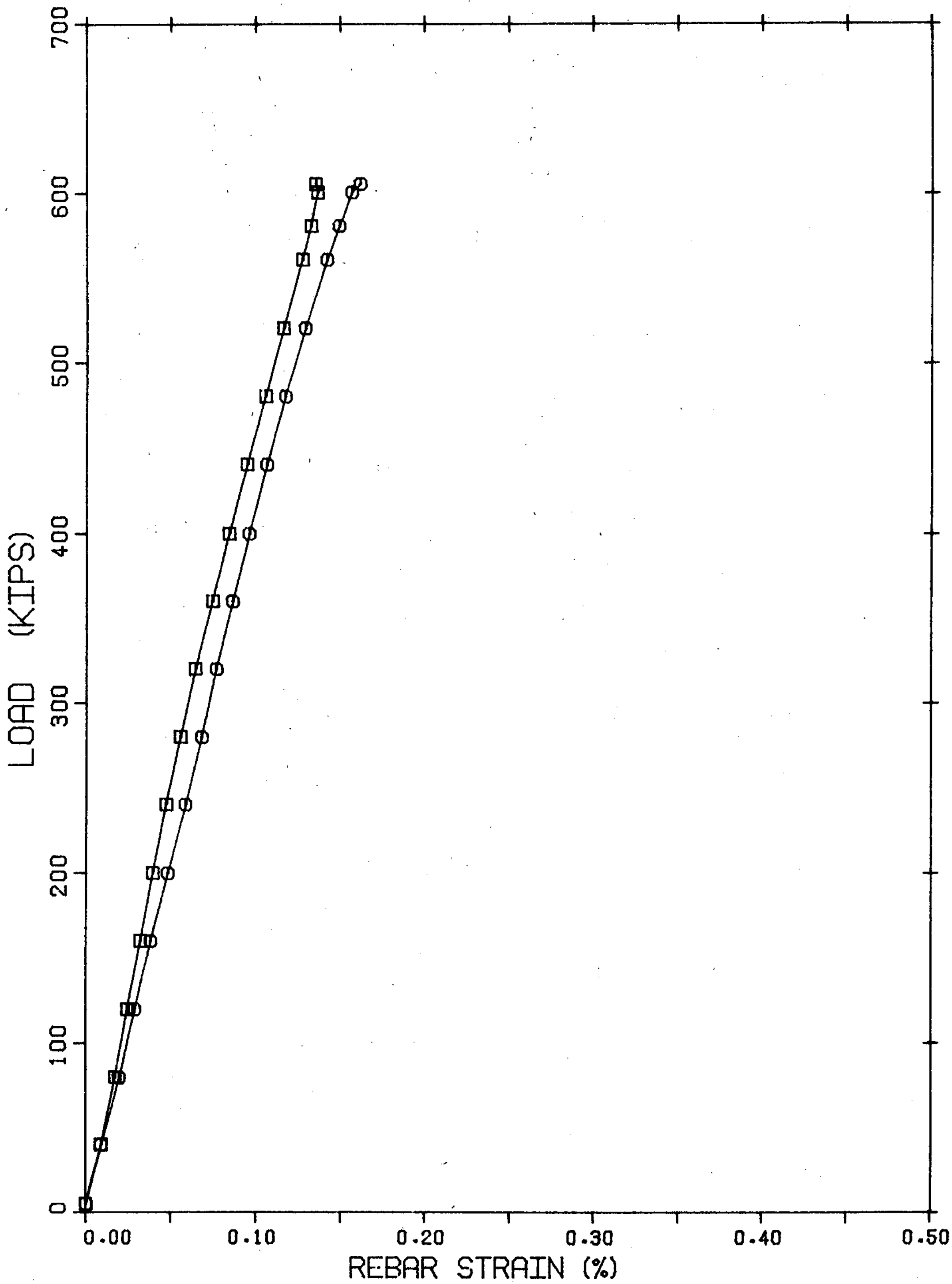

FIGURE B3.25 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN B7.2

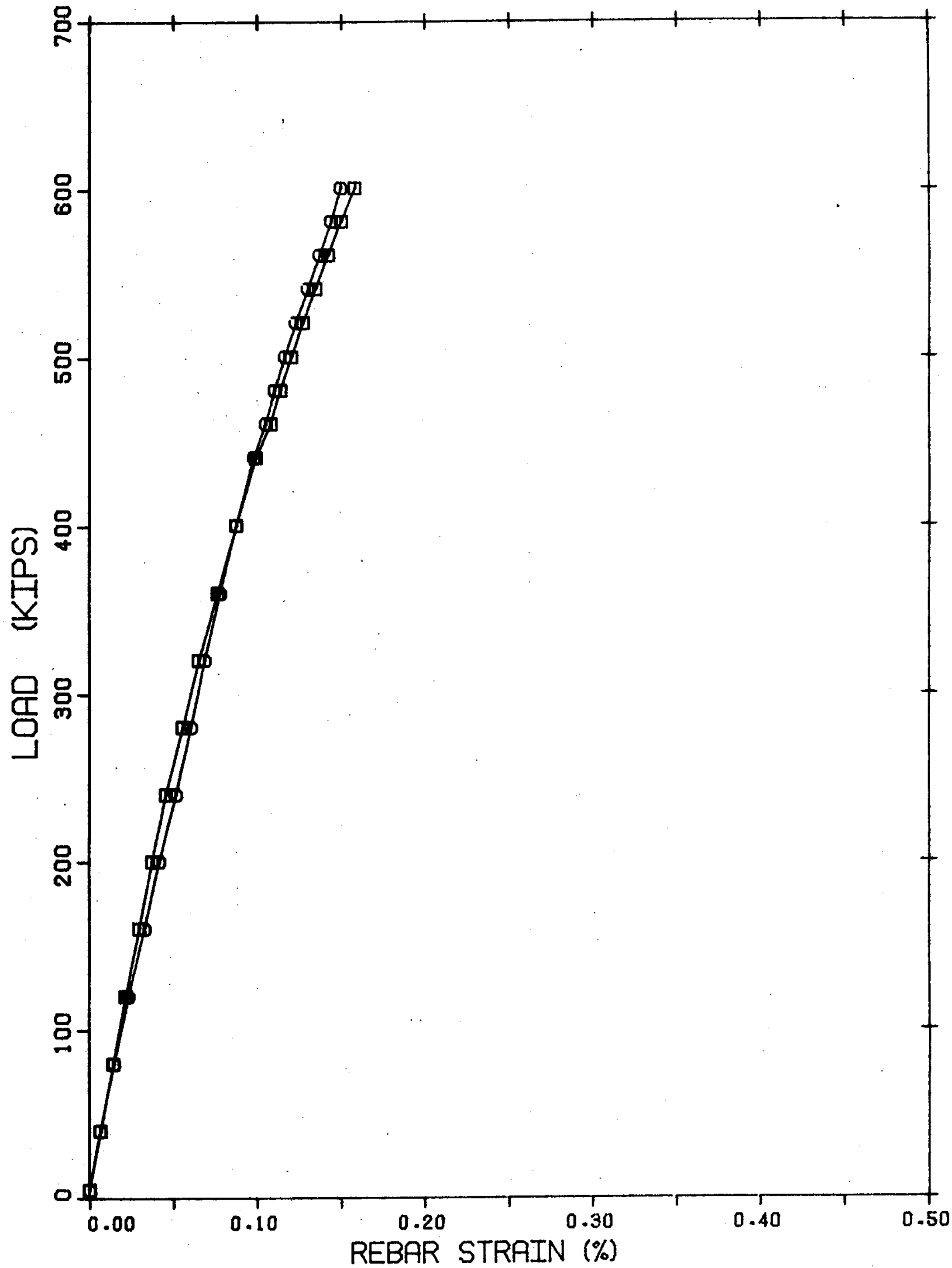

FIGURE B3.26 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN D1.1

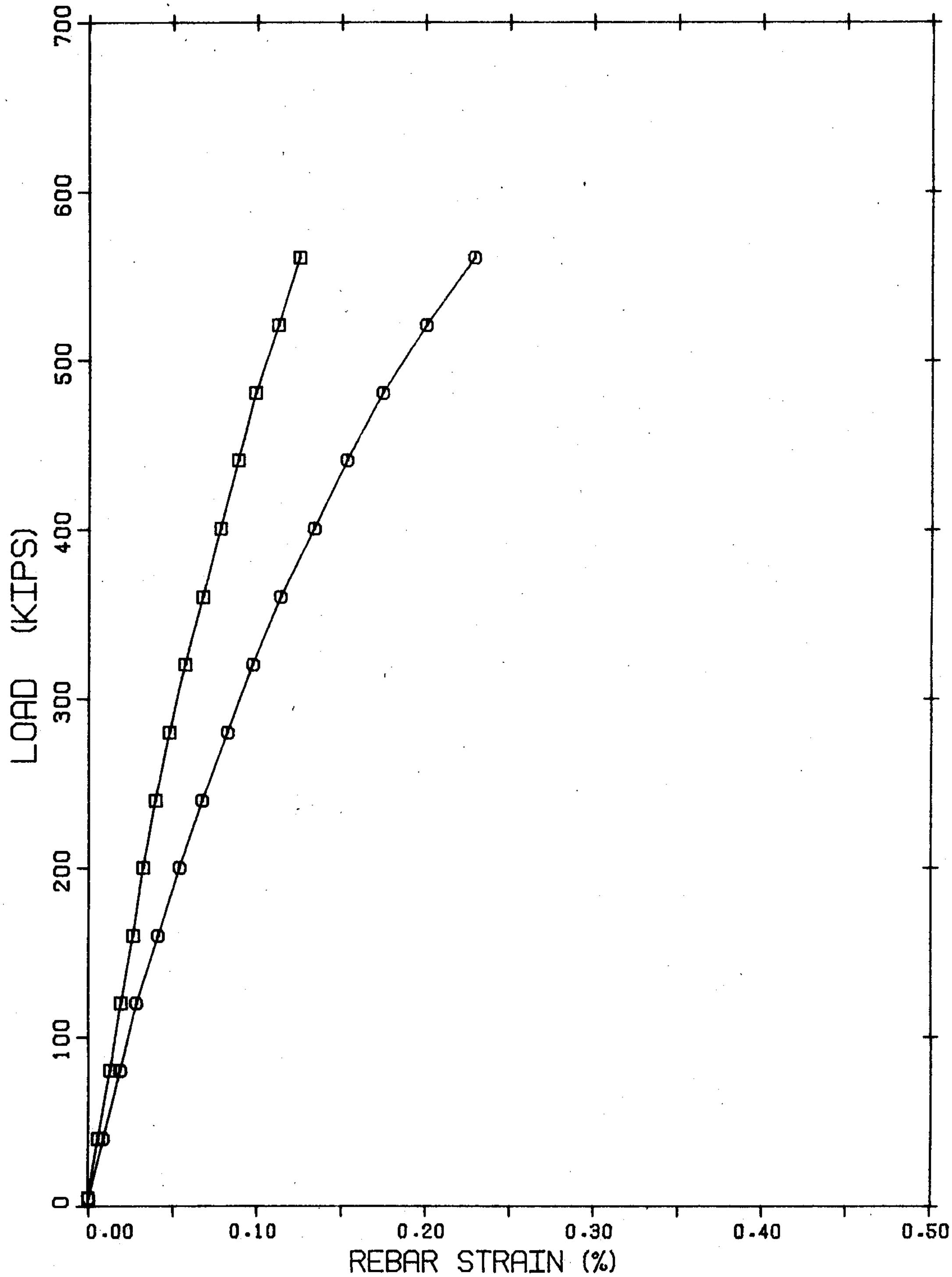

FIGURE B3.27 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN D1.2

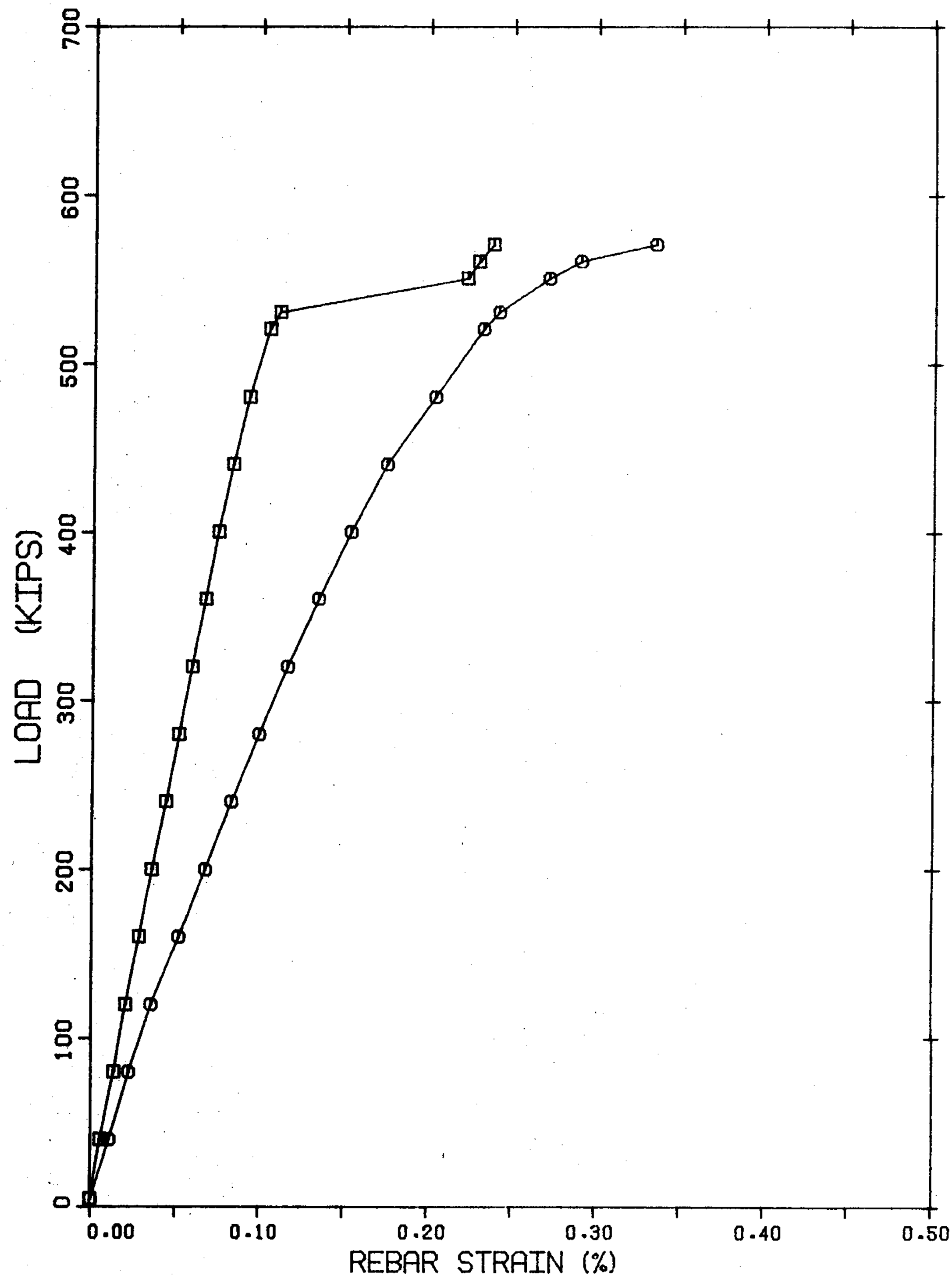

FIGURE B3.28 LOAD VS REINFORCEMENT STRAIN AT MID-HEIGHT OF COLUMN D2.1

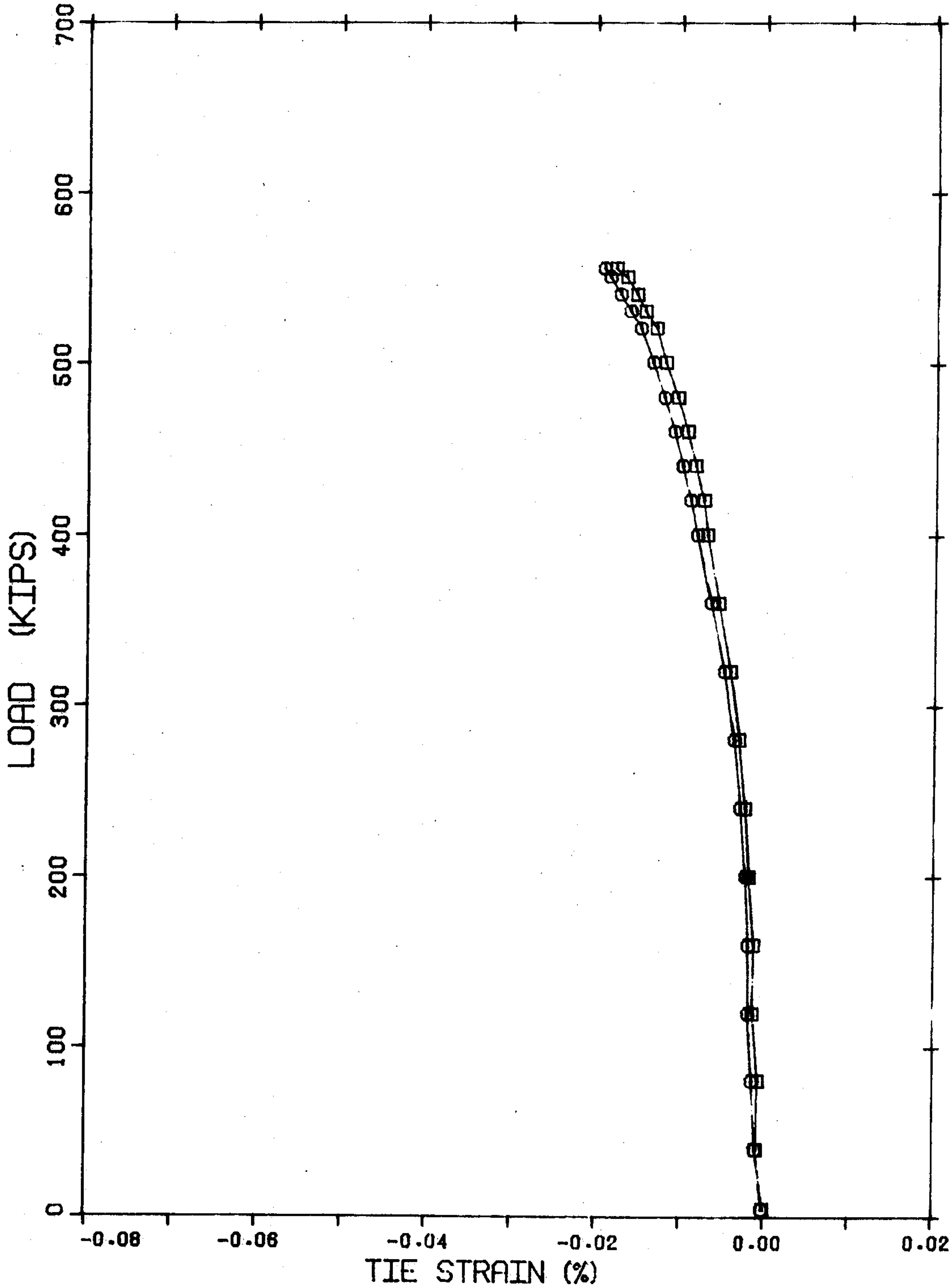

FIGURE B4.1 LOAD VS TIE STRAIN FOR COLUMN A2.1

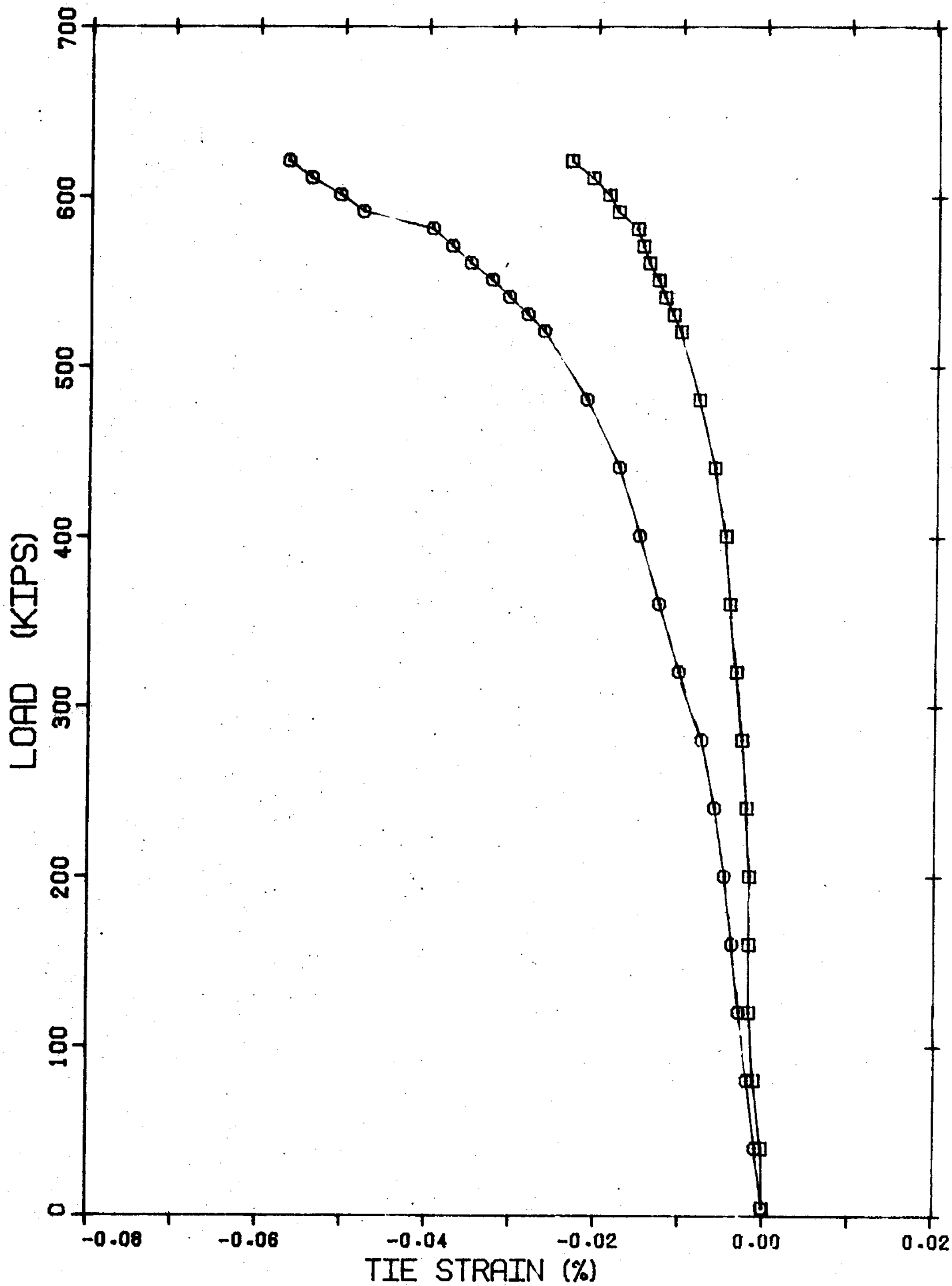

FIGURE B4.2 LOAD VS TIE STRAIN FOR COLUMN A2.2

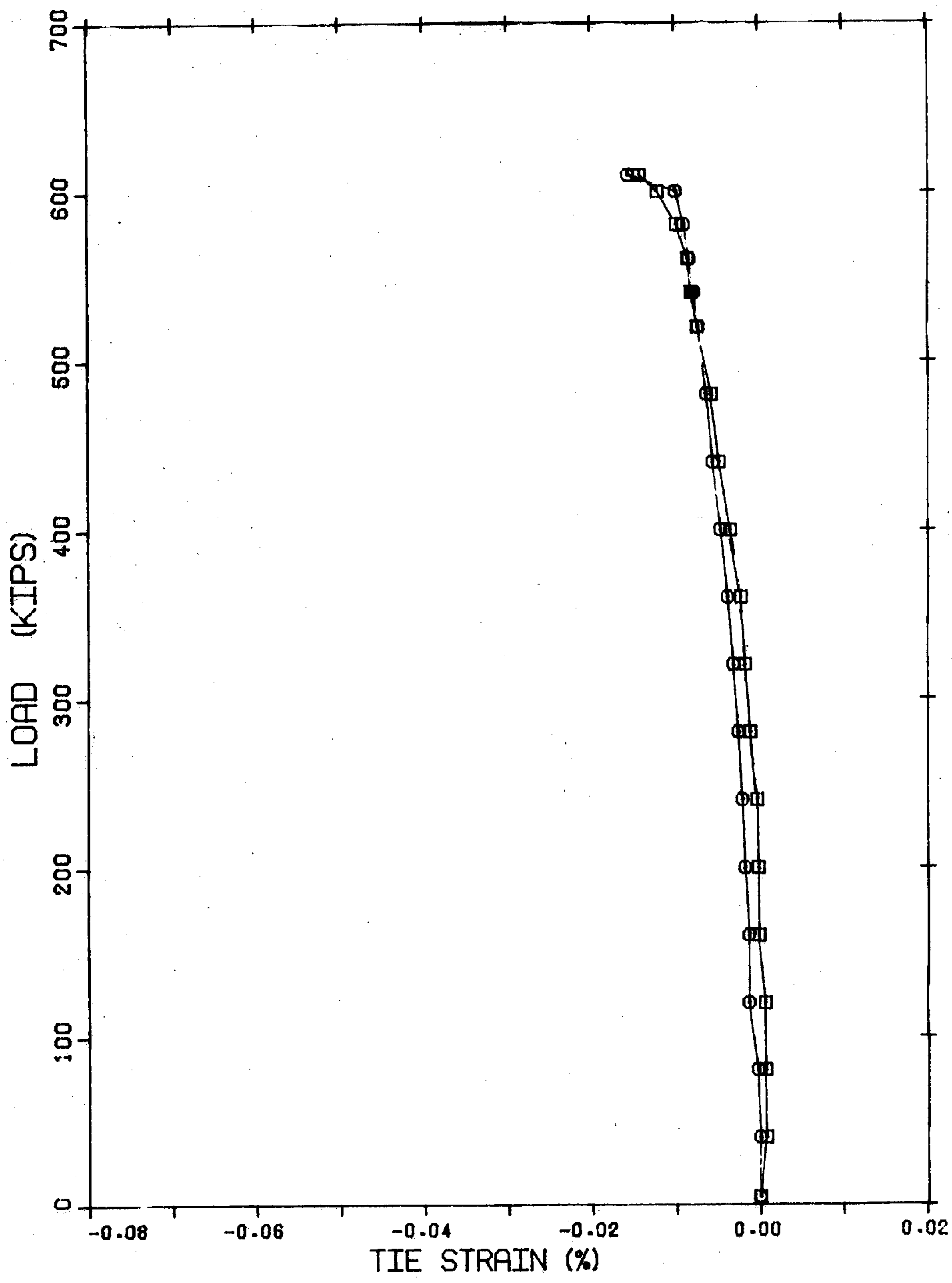

FIGURE B4.3 LOAD VS TIE STRAIN FOR COLUMN A3.1

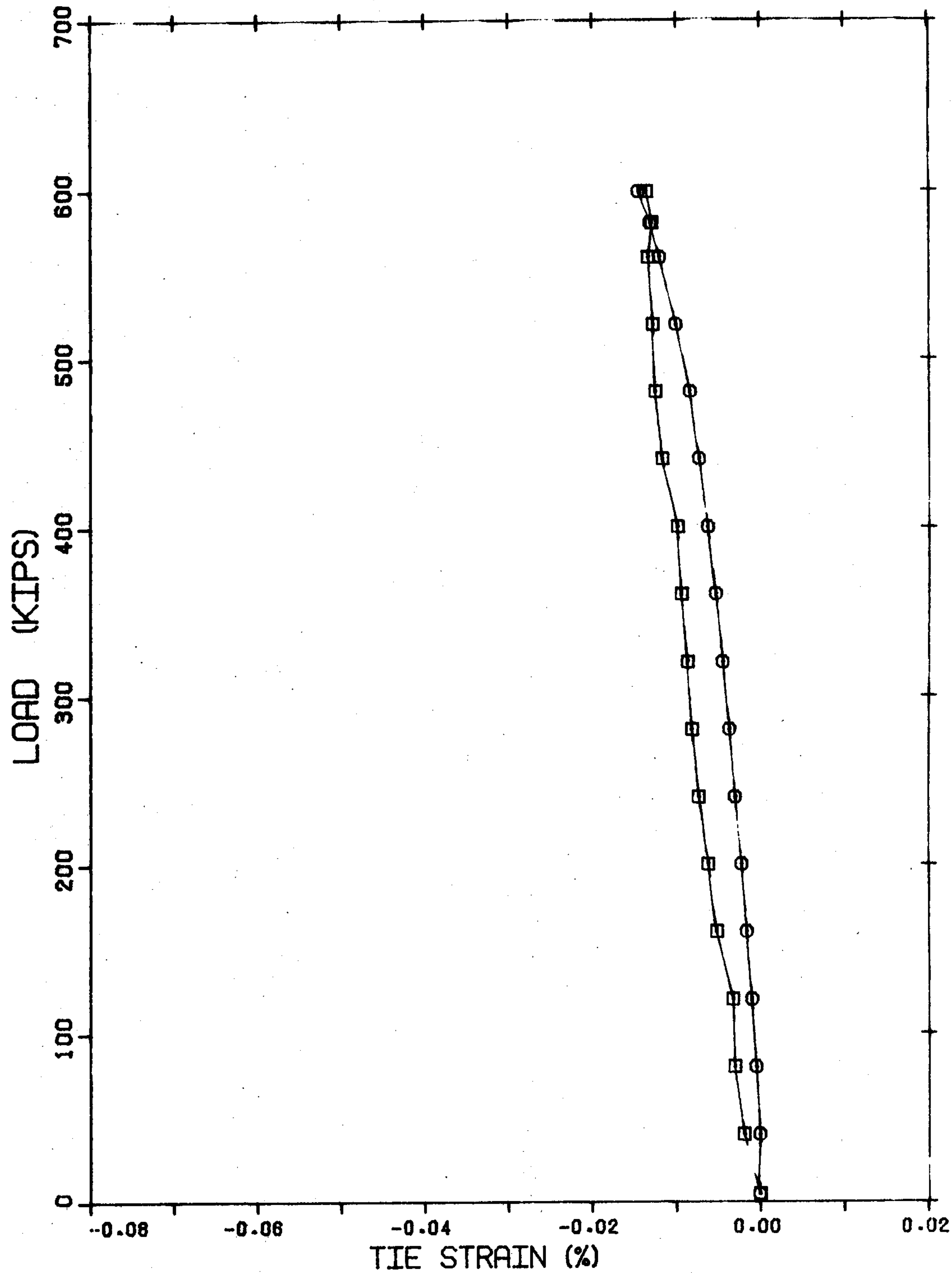

FIGURE B4.4 LOAD VS TIE STRAIN FOR COLUMN A3.2

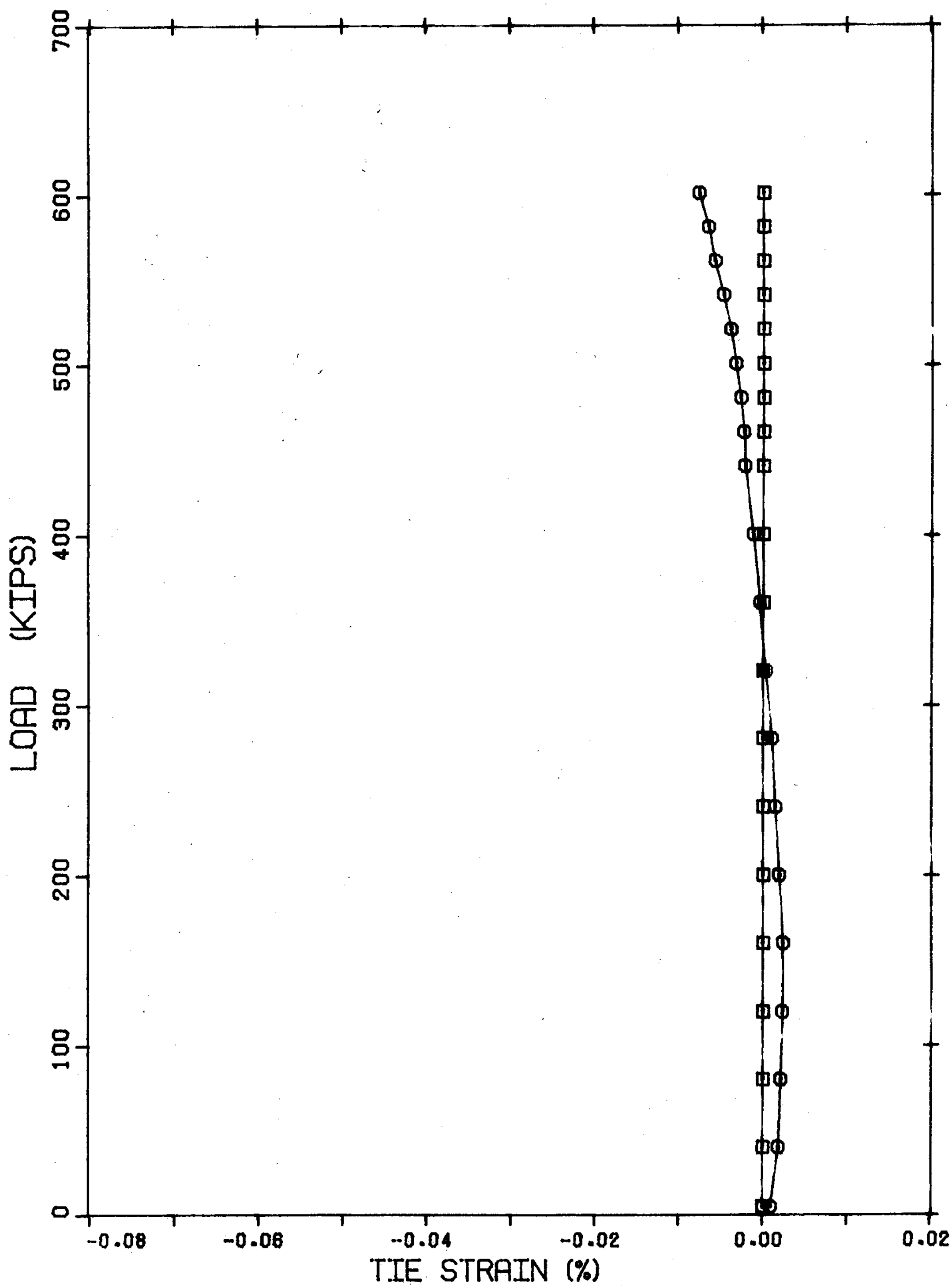

FIGURE B4.5 LOAD VS TIE STRAIN FOR COLUMN D1.1

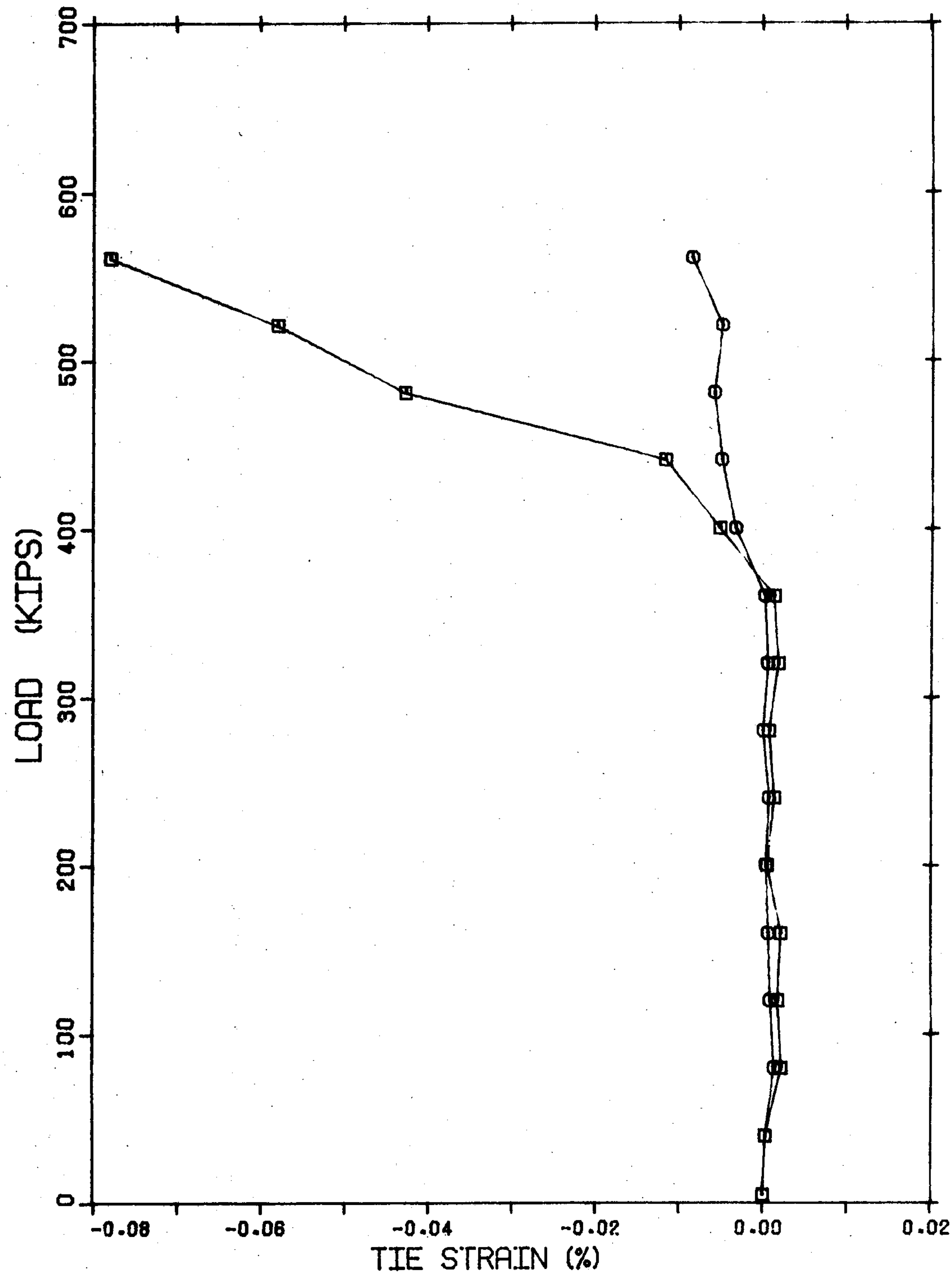

FIGURE B4.6 LOAD VS TIE STRAIN FOR COLUMN D1.2

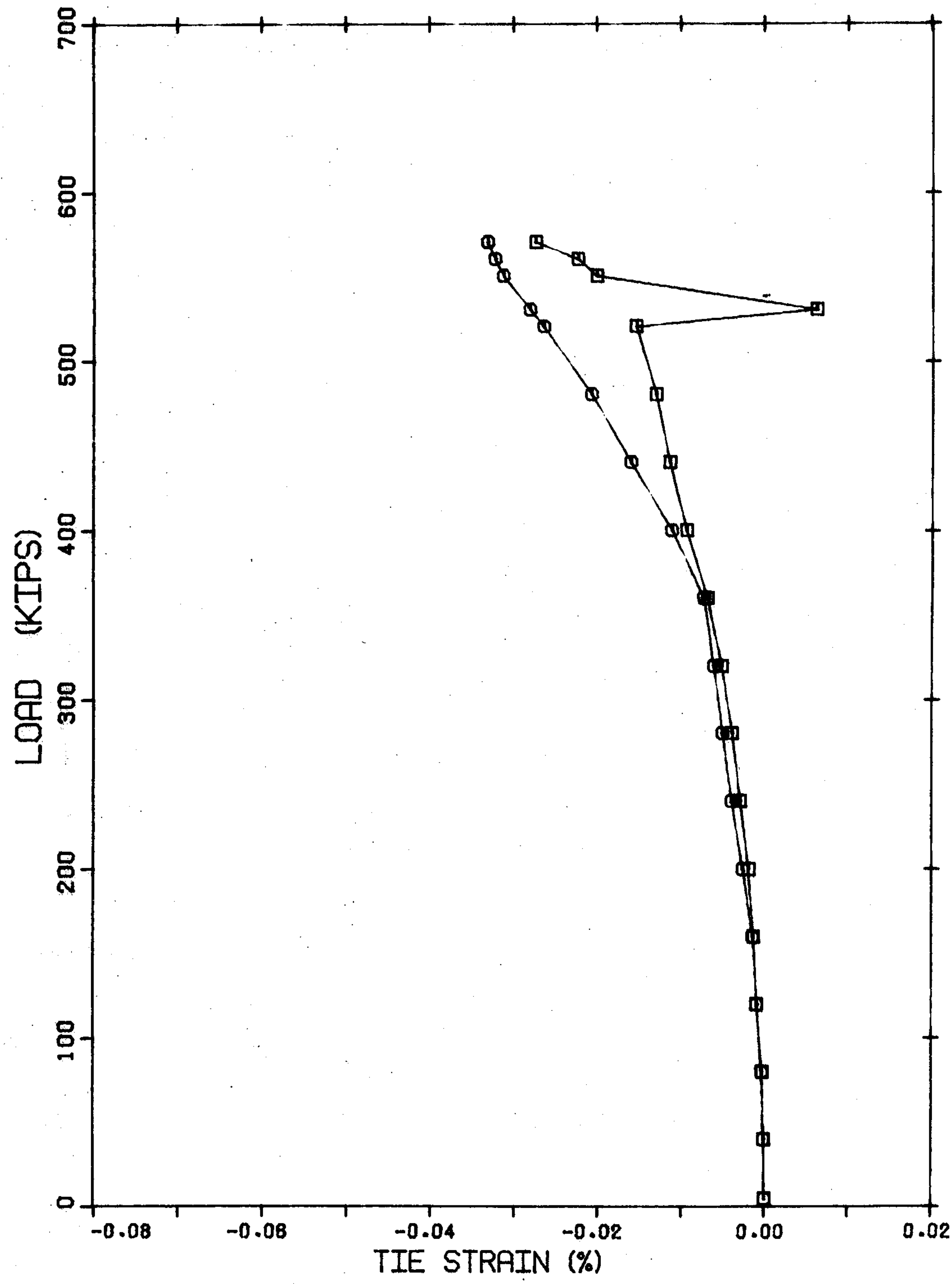

FIGURE B4.7 LOAD VS TIE STRAIN FOR COLUMN D2.1

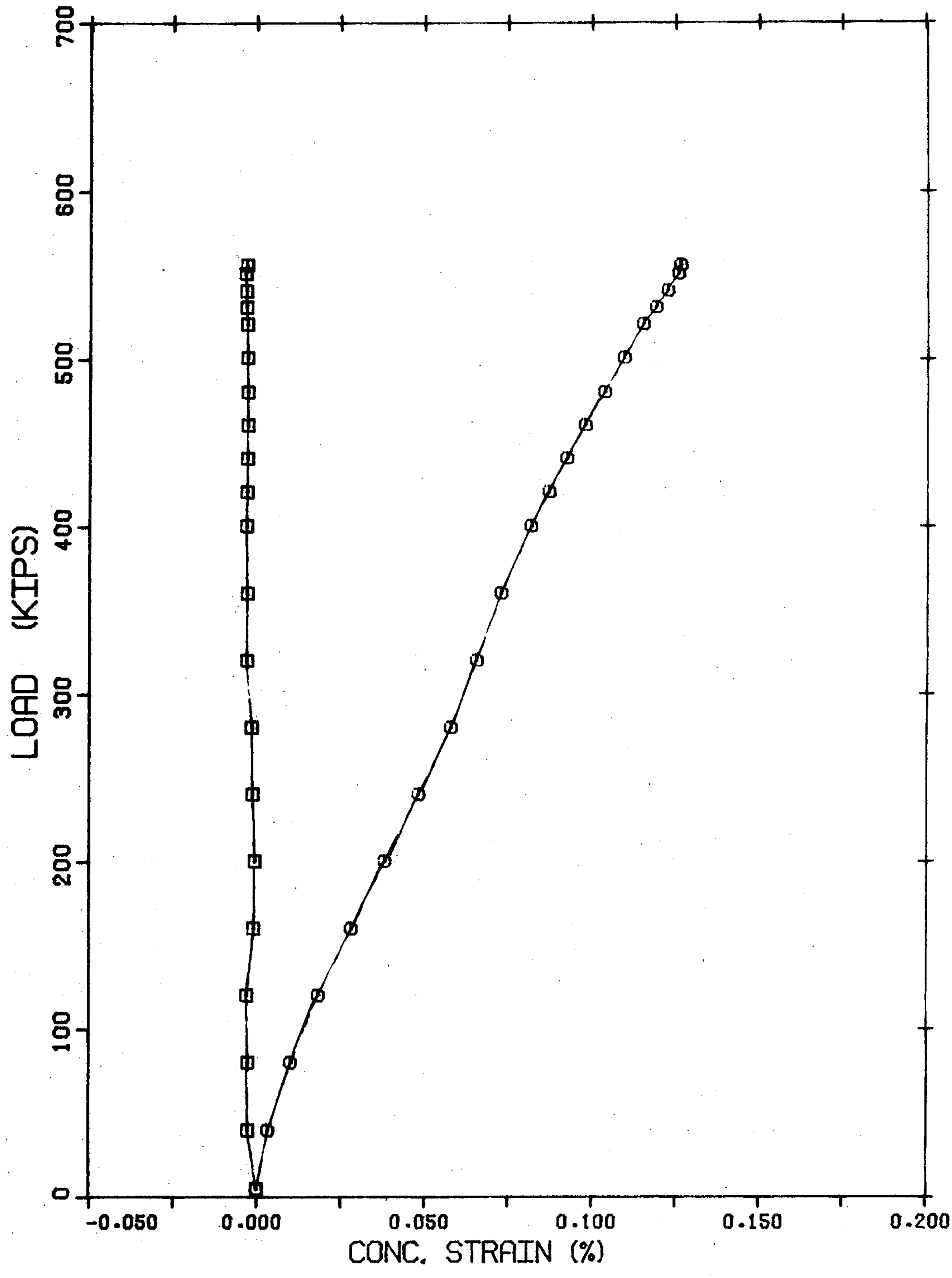

FIGURE B5.1 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN A2.1

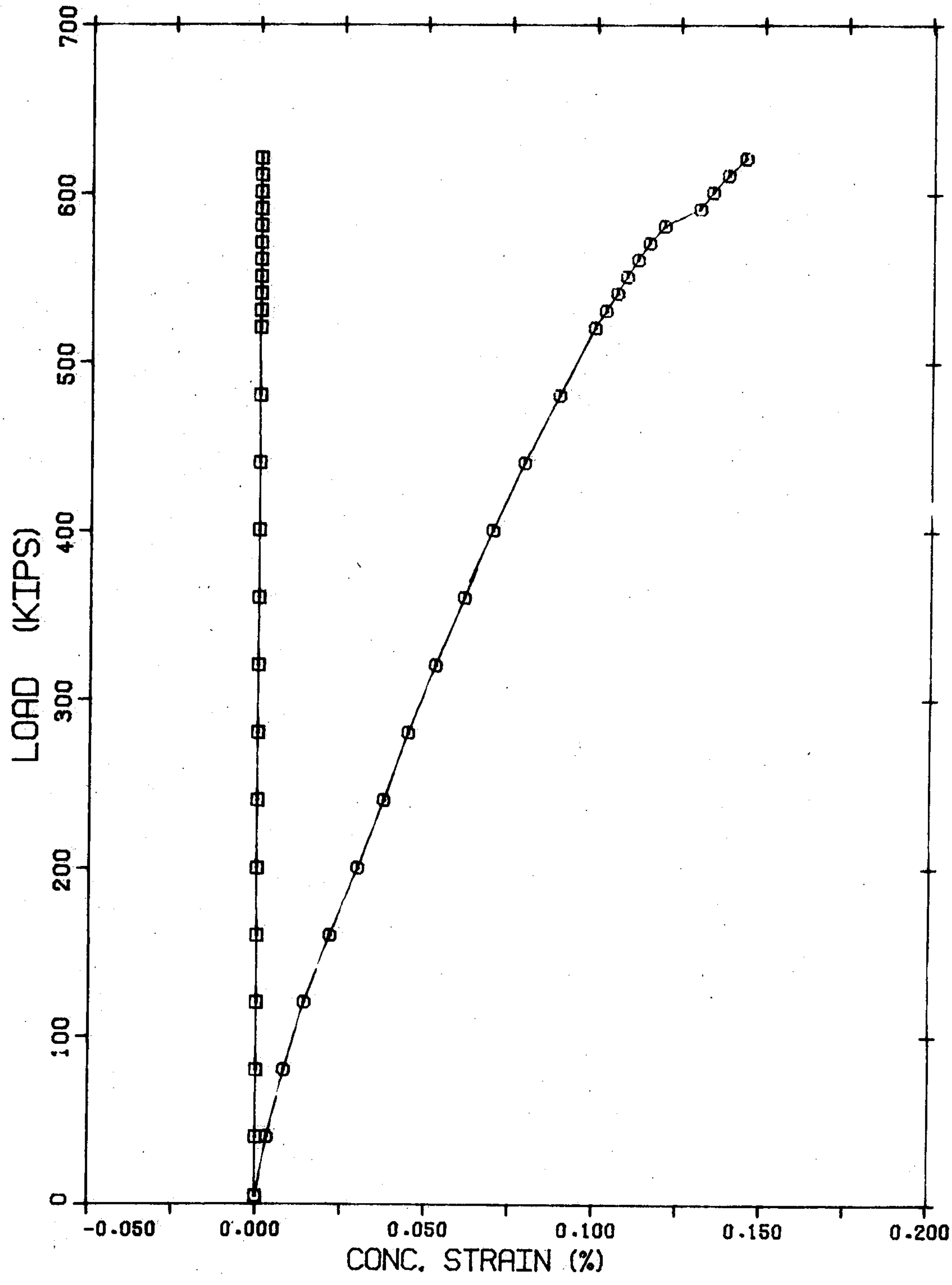

FIGURE B5.2 LOAD VS SURFACE CONCRETE STRAIN

FOR COLUMN A2.2

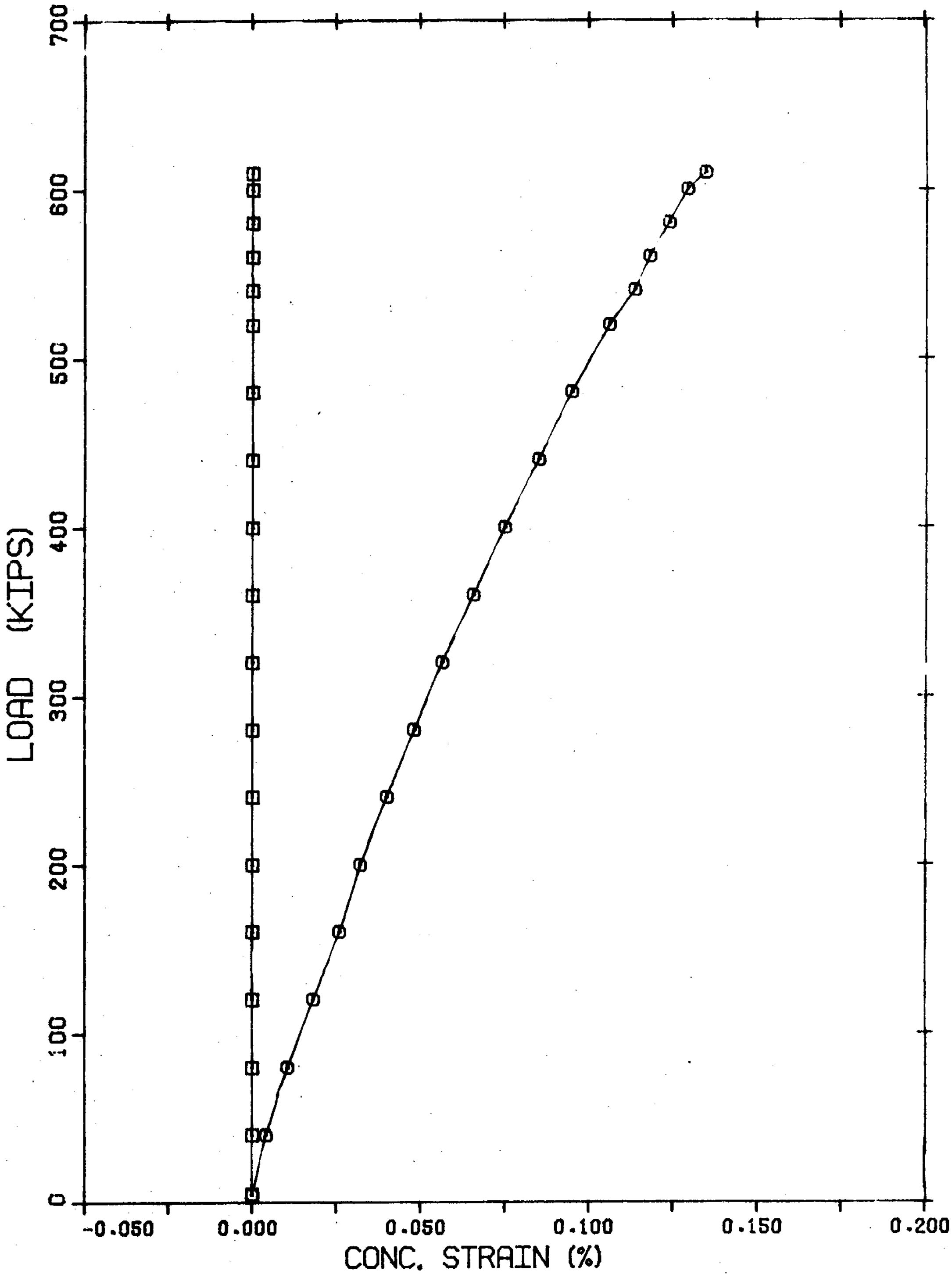

FIGURE B5.3 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN A3.1

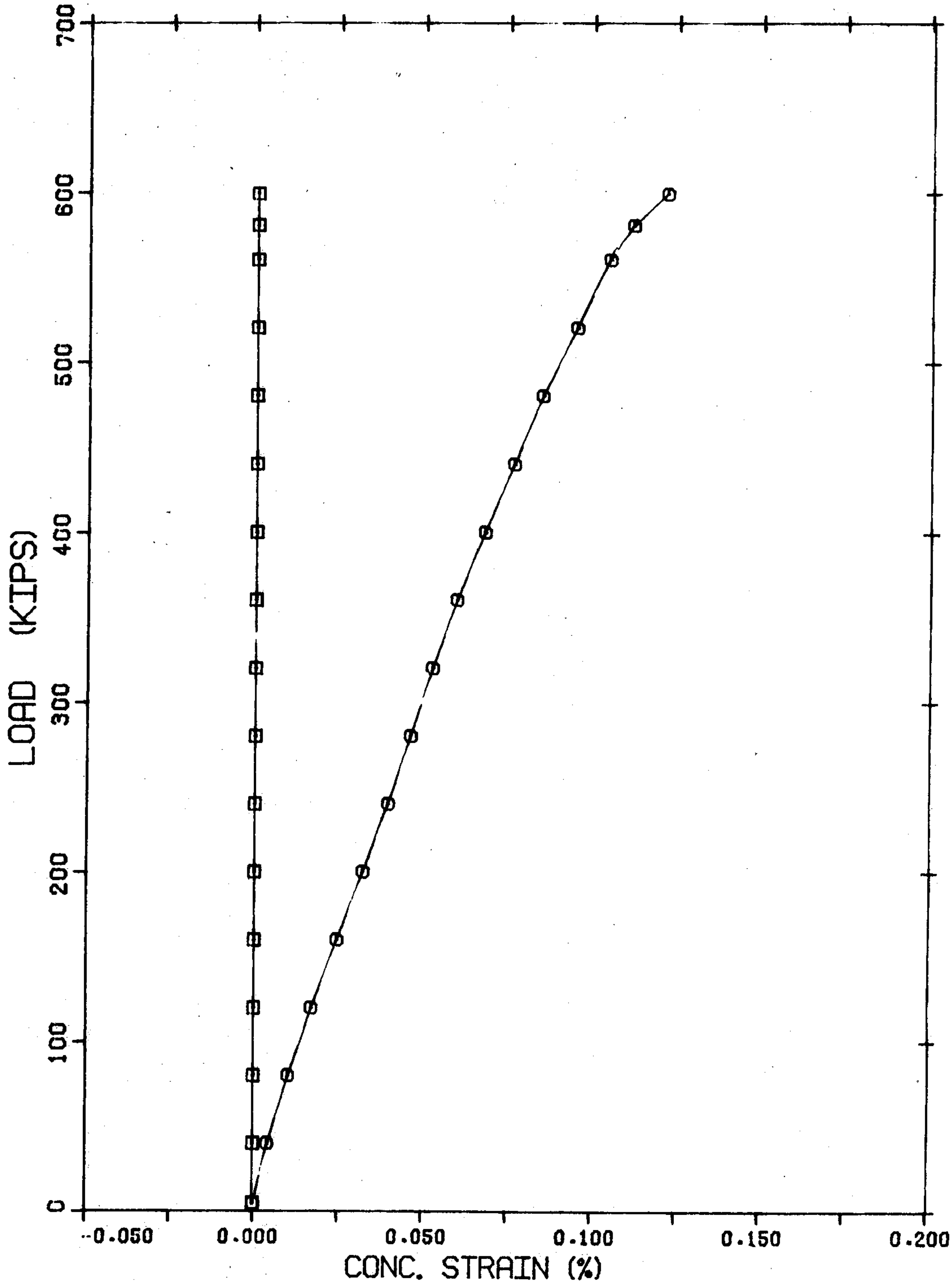

FIGURE B5.4 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN A3.2

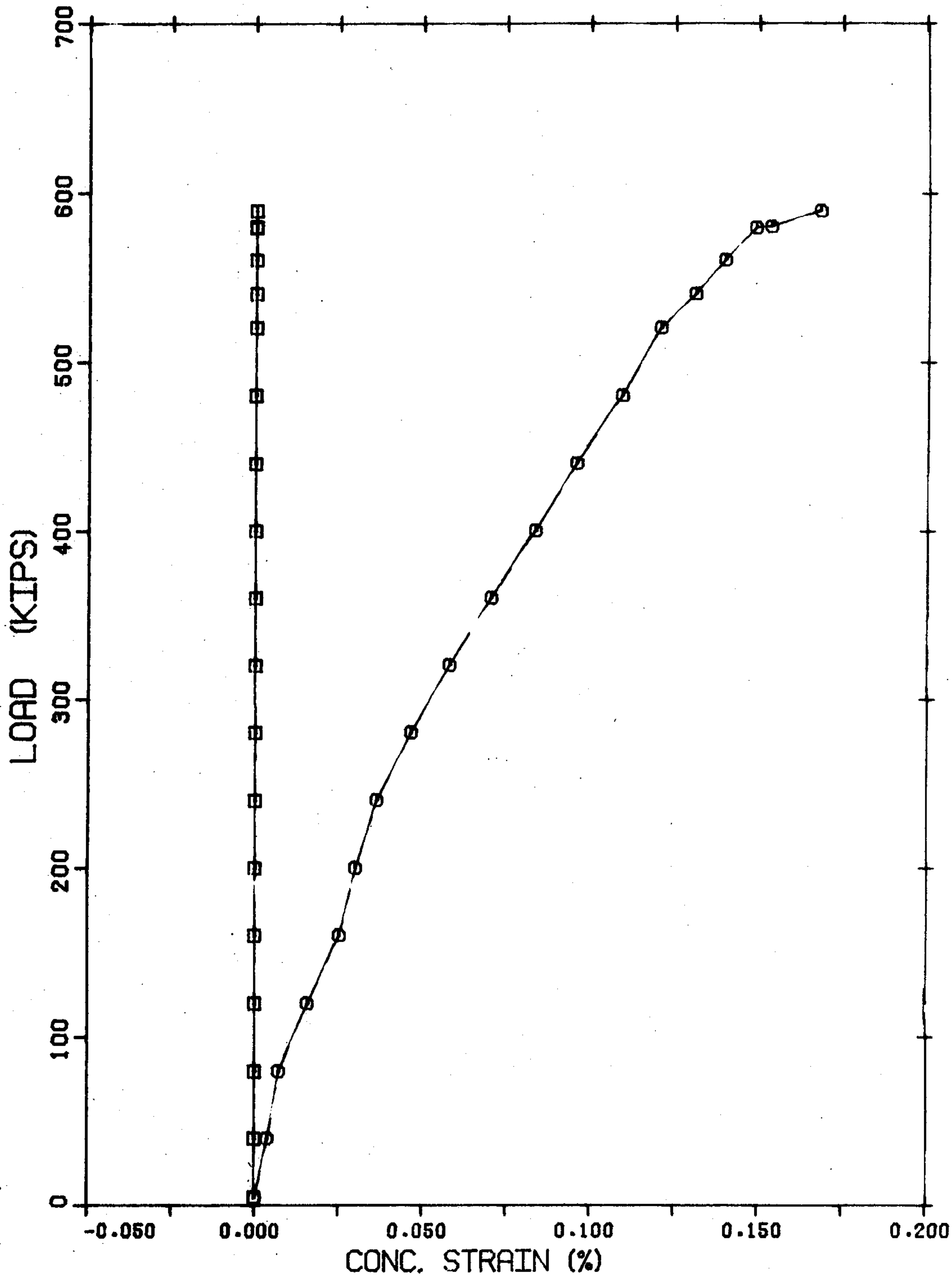

FIGURE B5.5 LOAD VS SURFACE CONCRETE STRAIN

FOR COLUMN A4.1

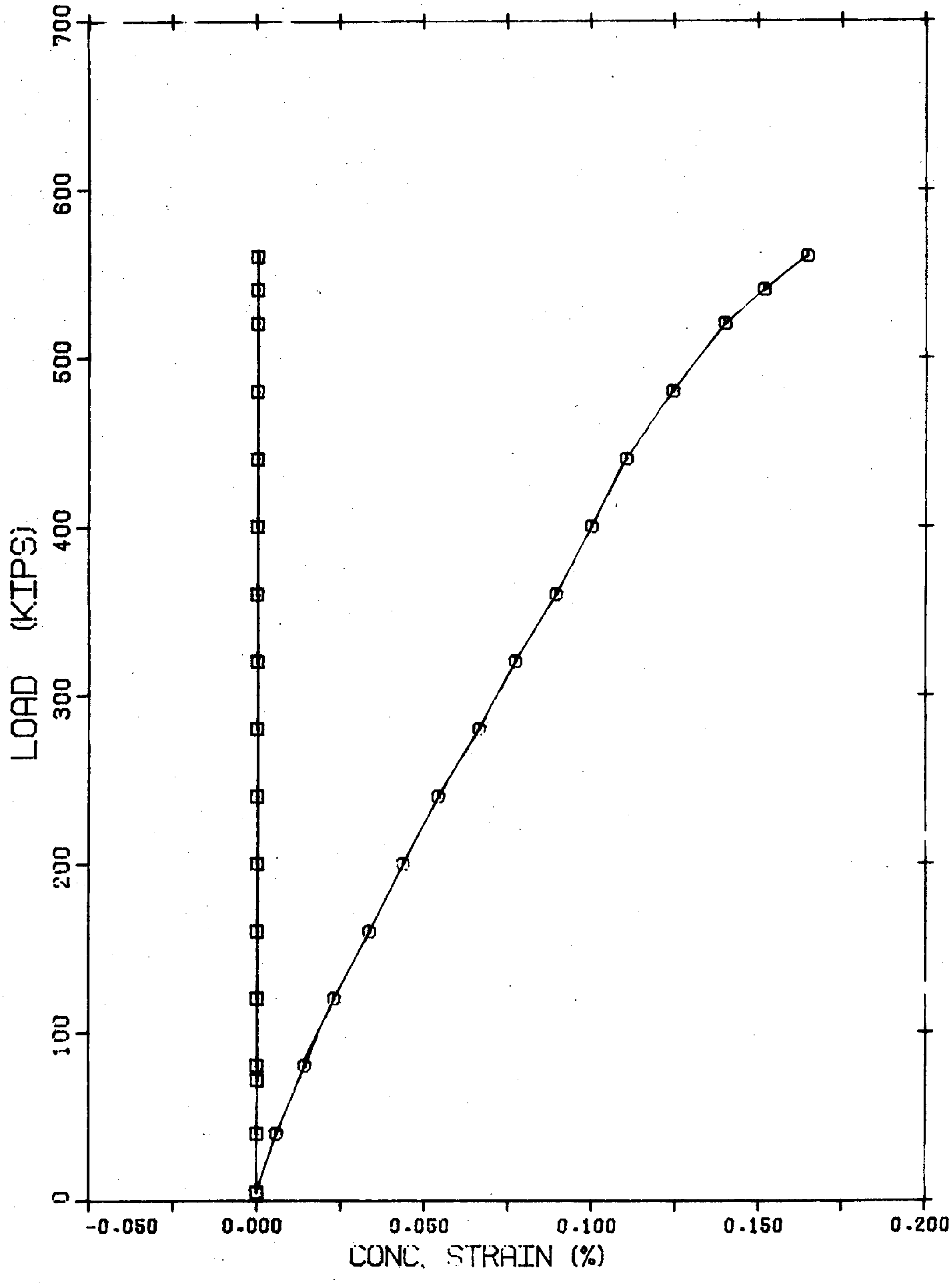

FIGURE B5.6 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN A4.2

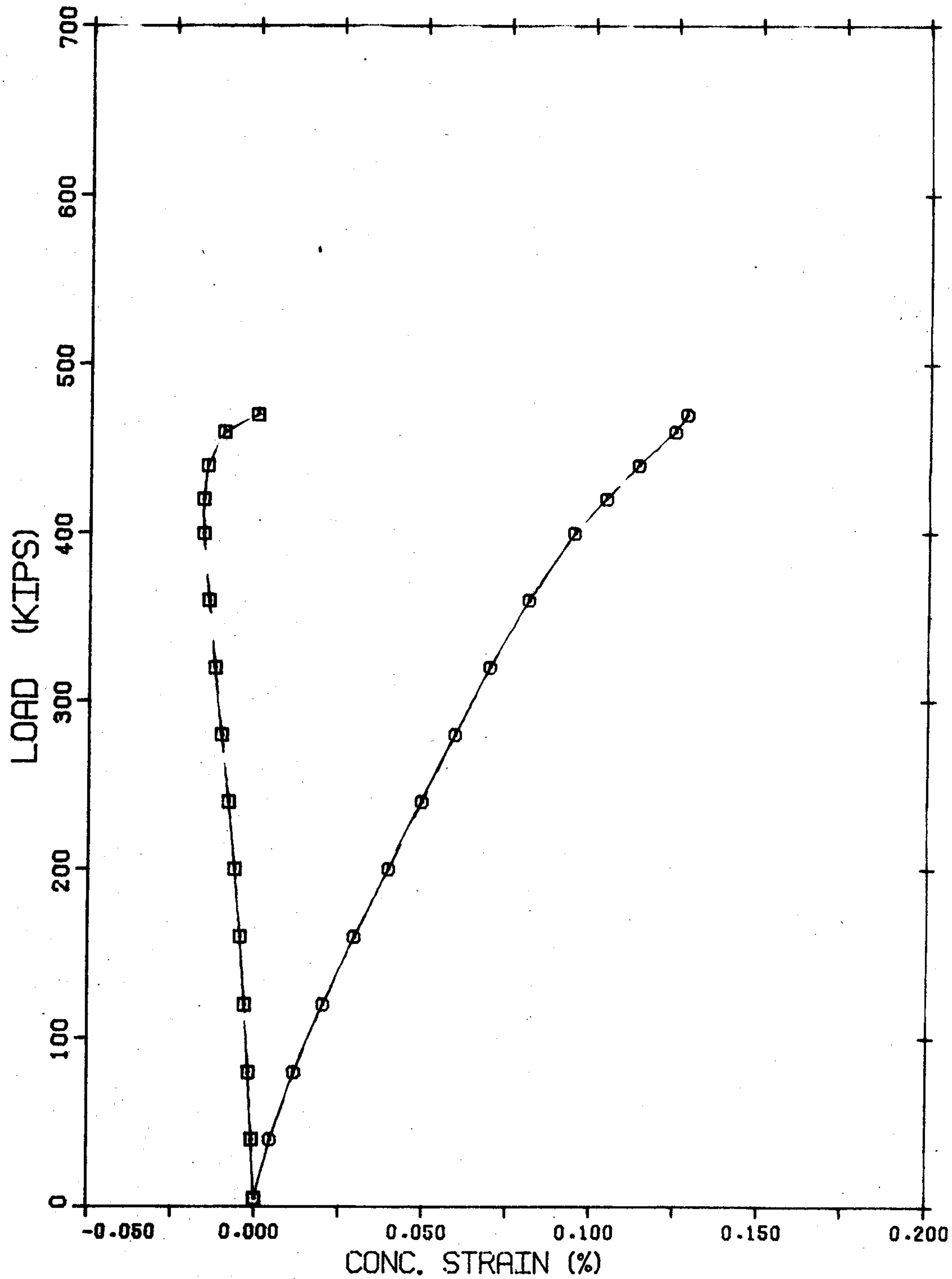

FIGURE B5.7 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B1.1

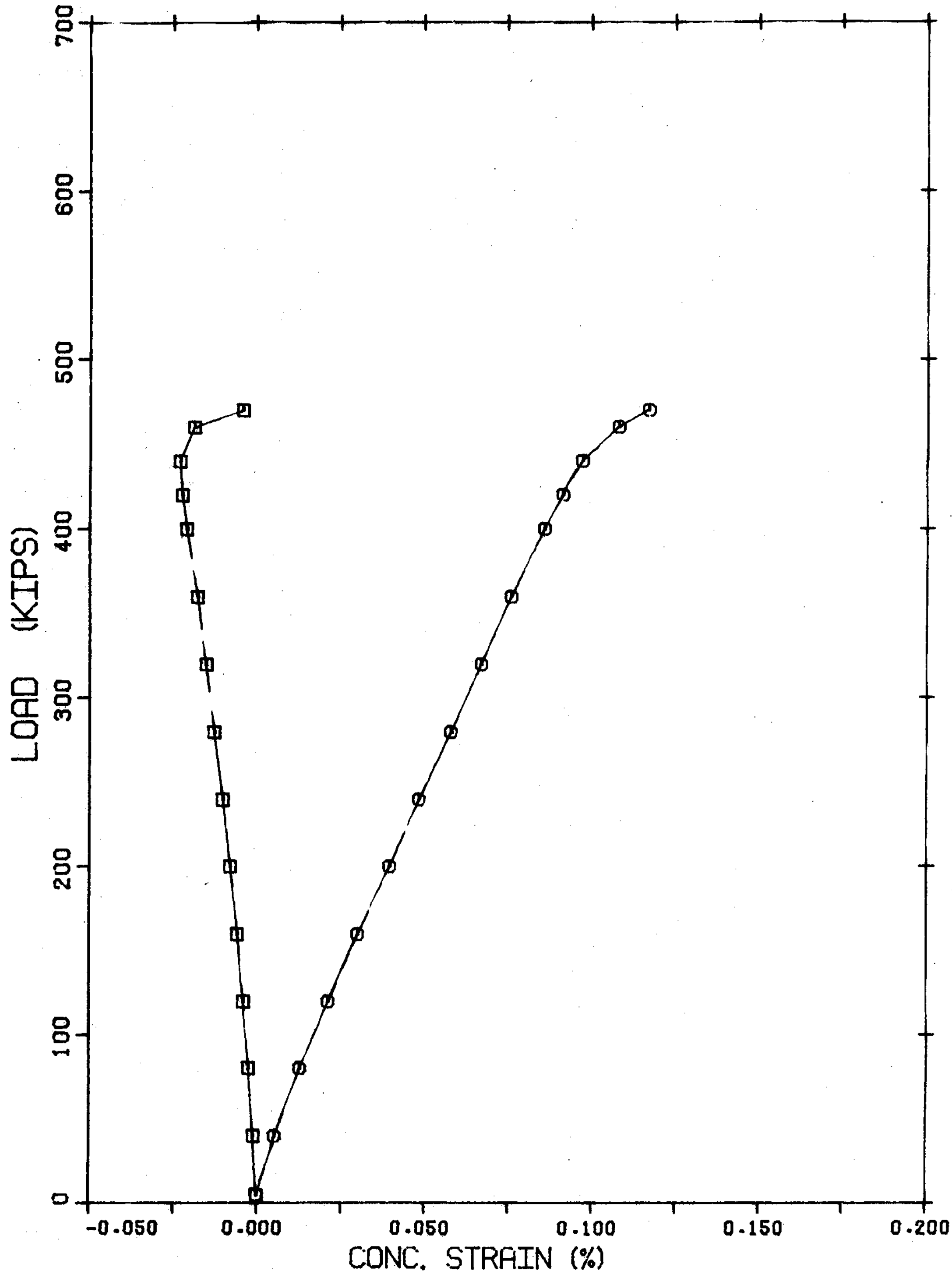

FIGURE B5.8 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B1.2

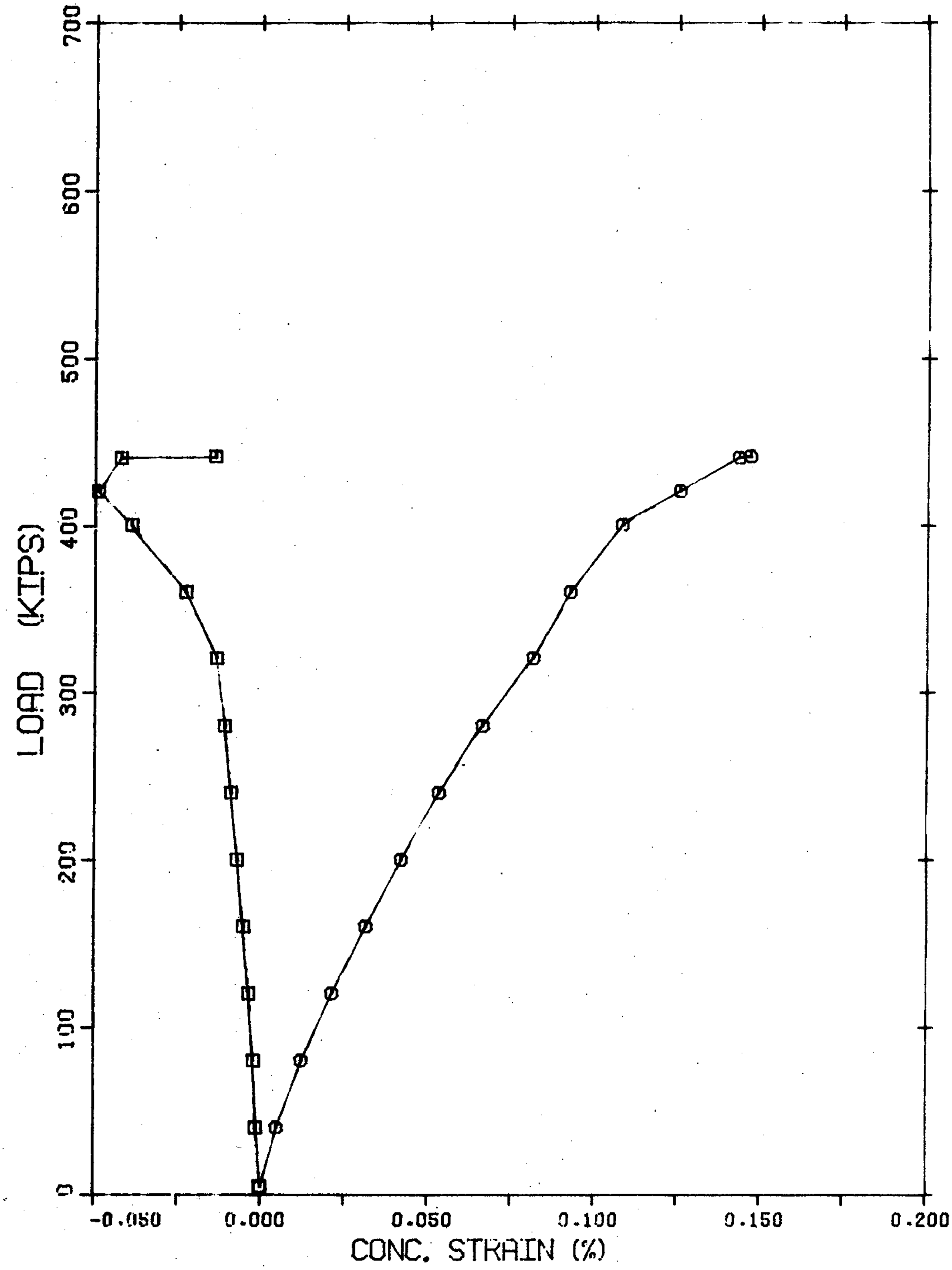

FIGURE B5.9 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN B1.3

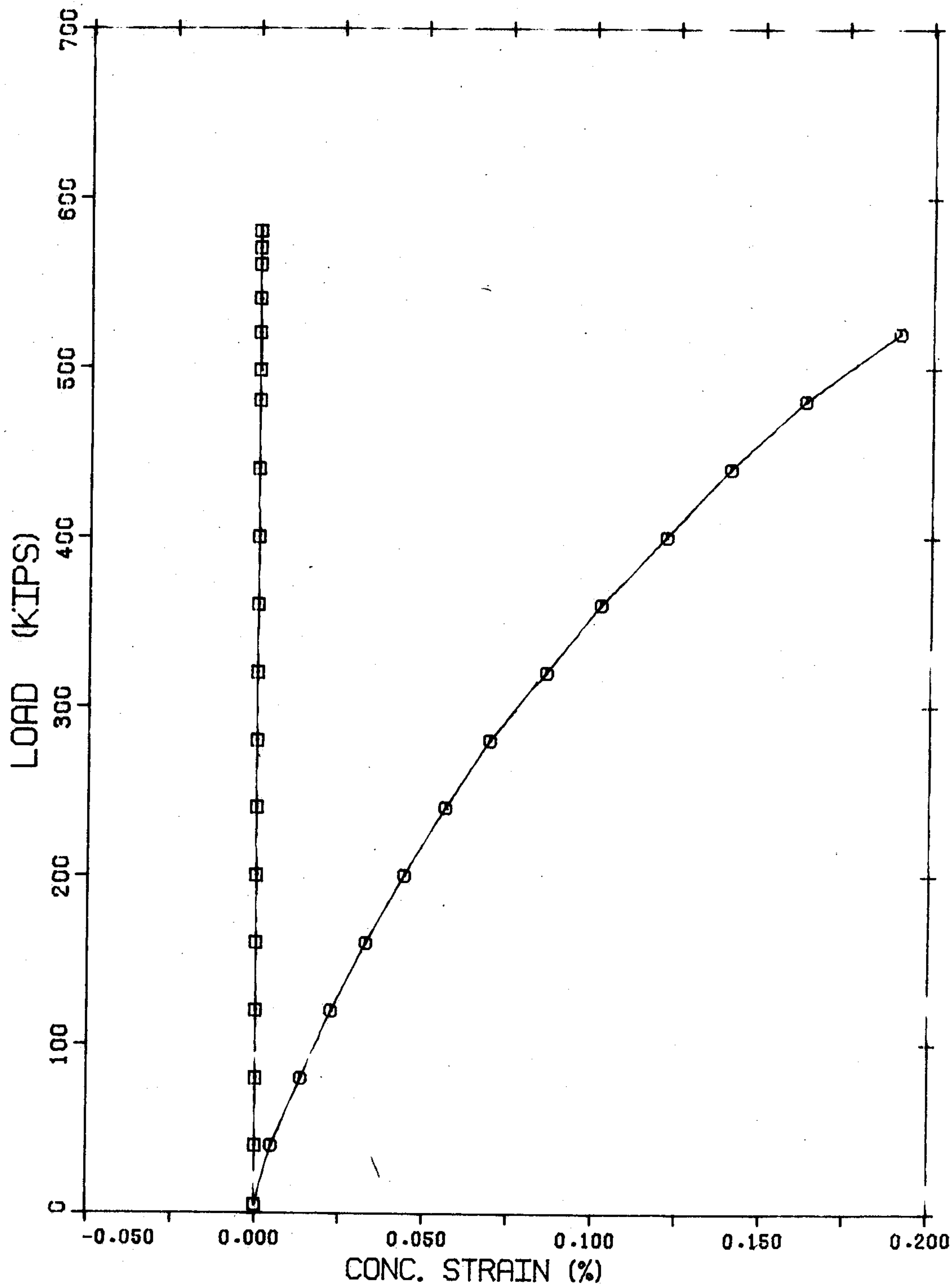

FIGURE B5.10 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B2.4

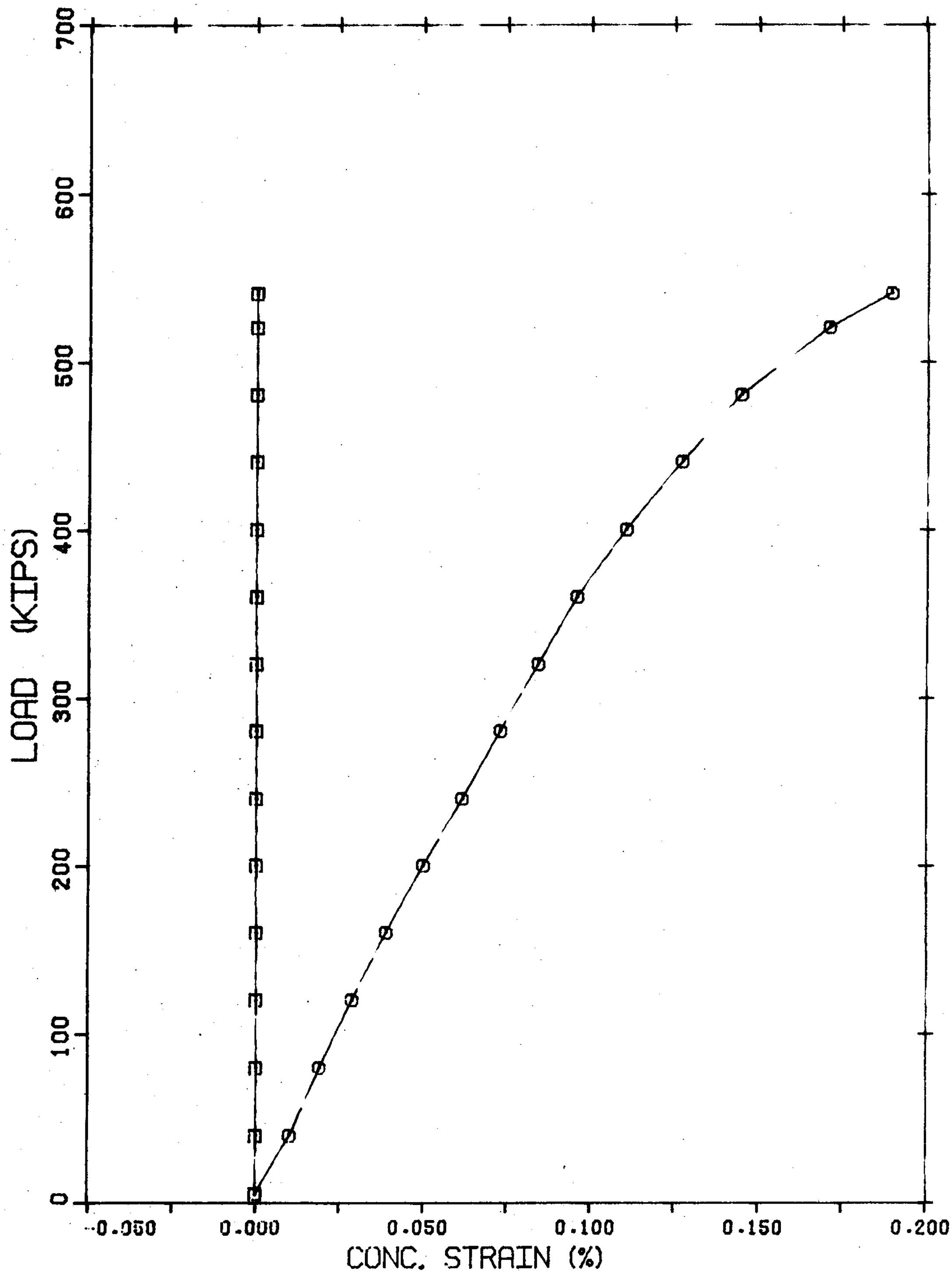

FIGURE B5.11 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN B2.5

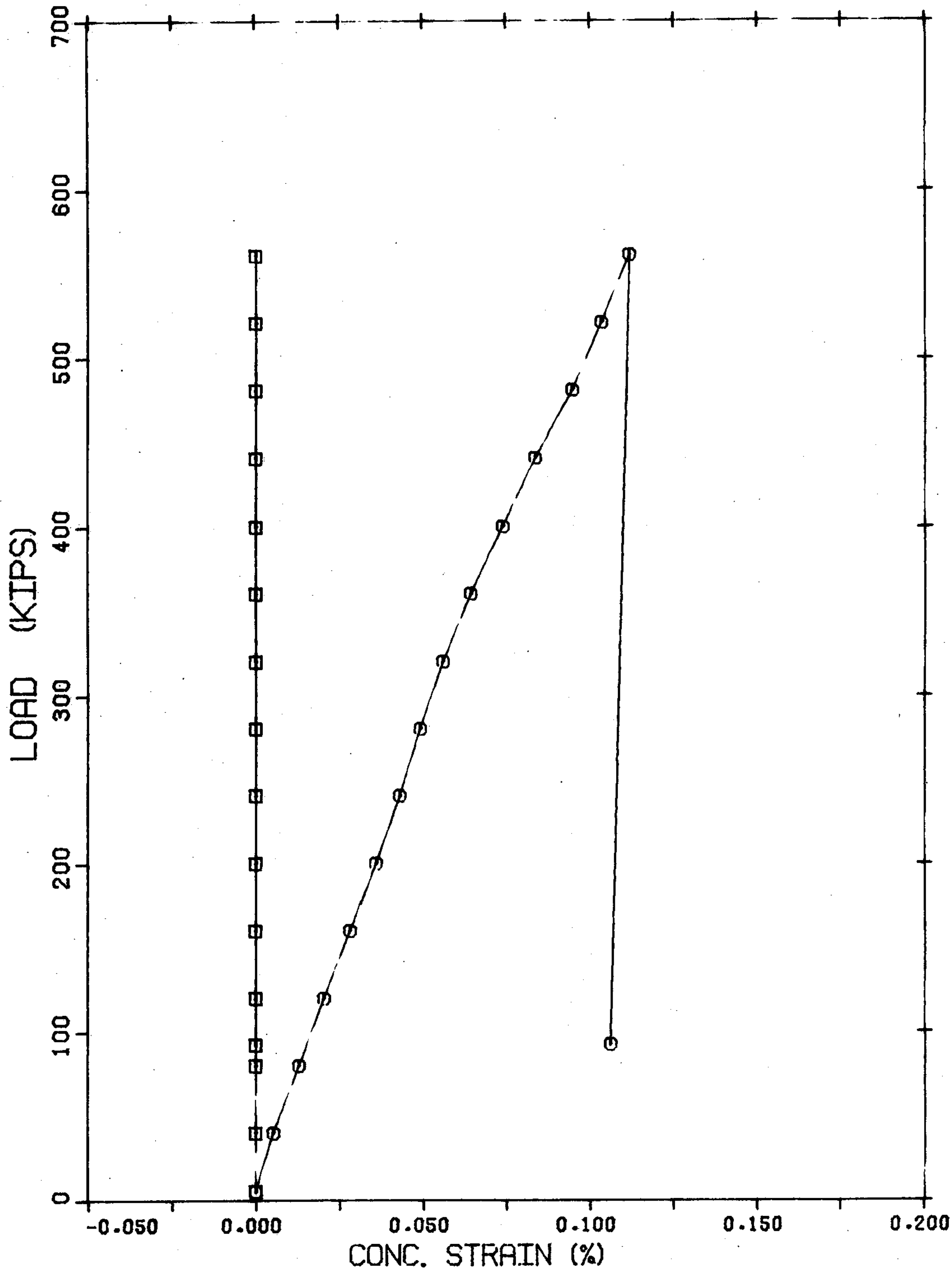

FIGURE B5.12 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B4.1

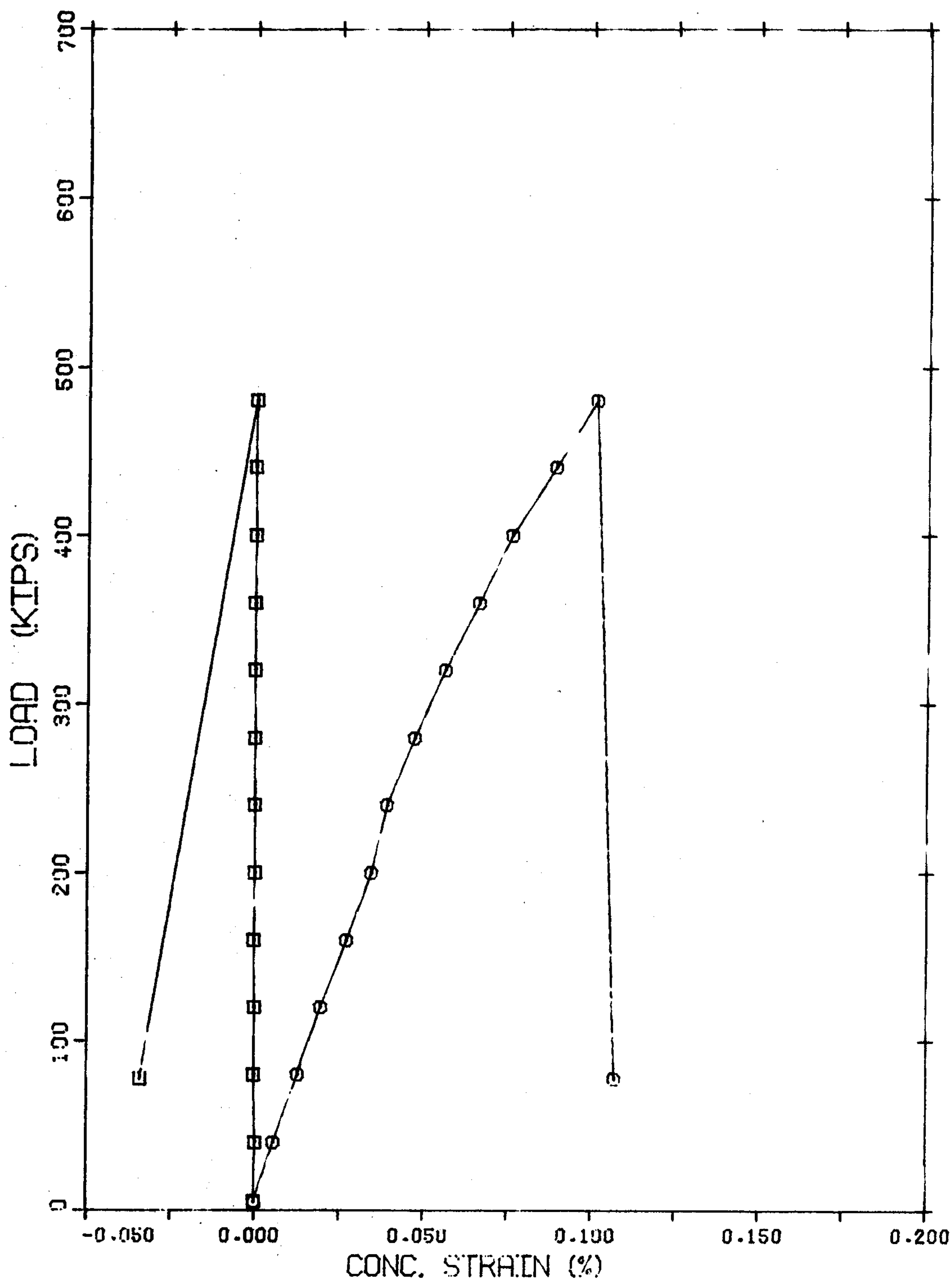

FIGURE B5.13 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B4.2

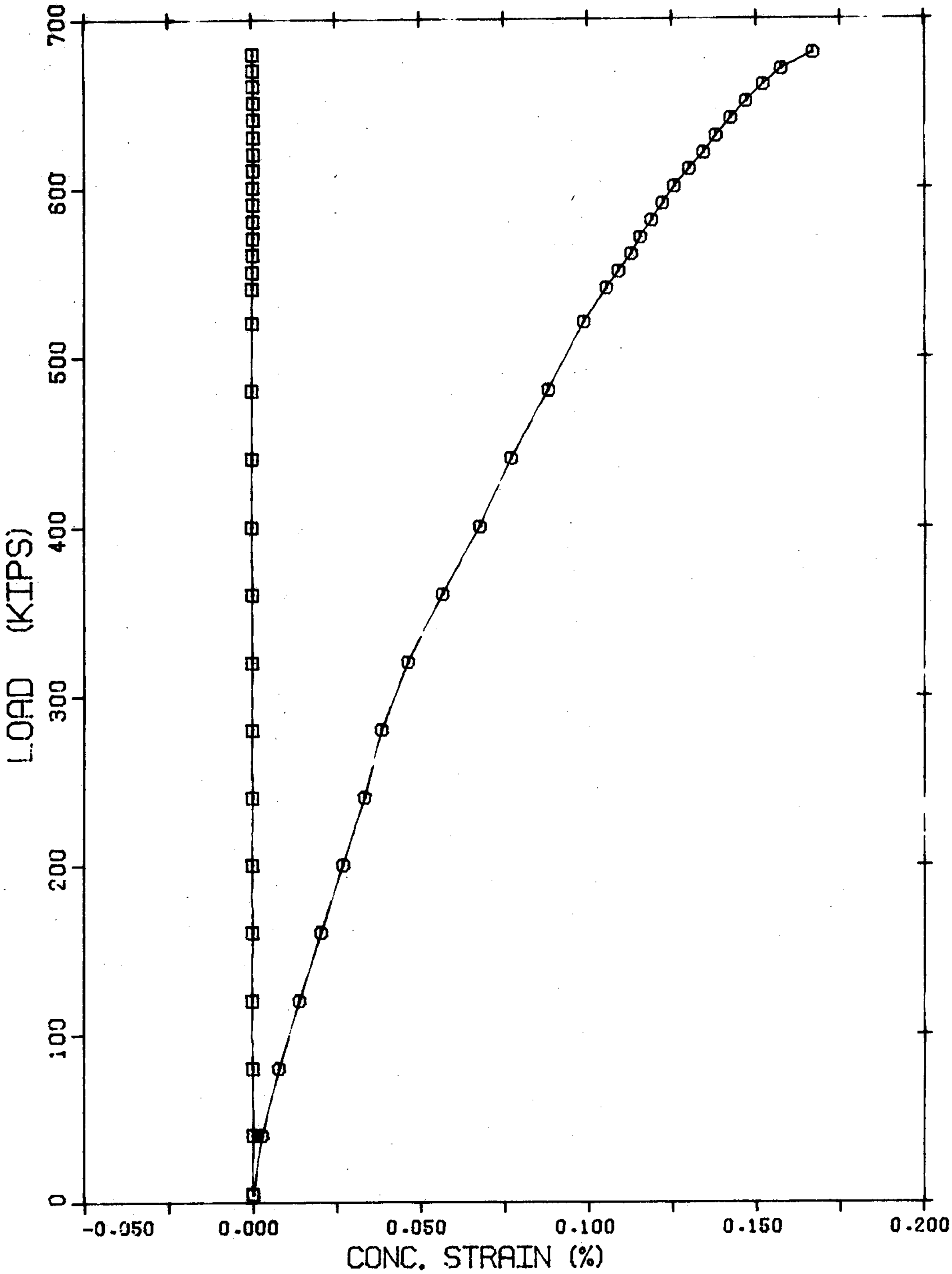

FIGURE B5.14 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B5.1

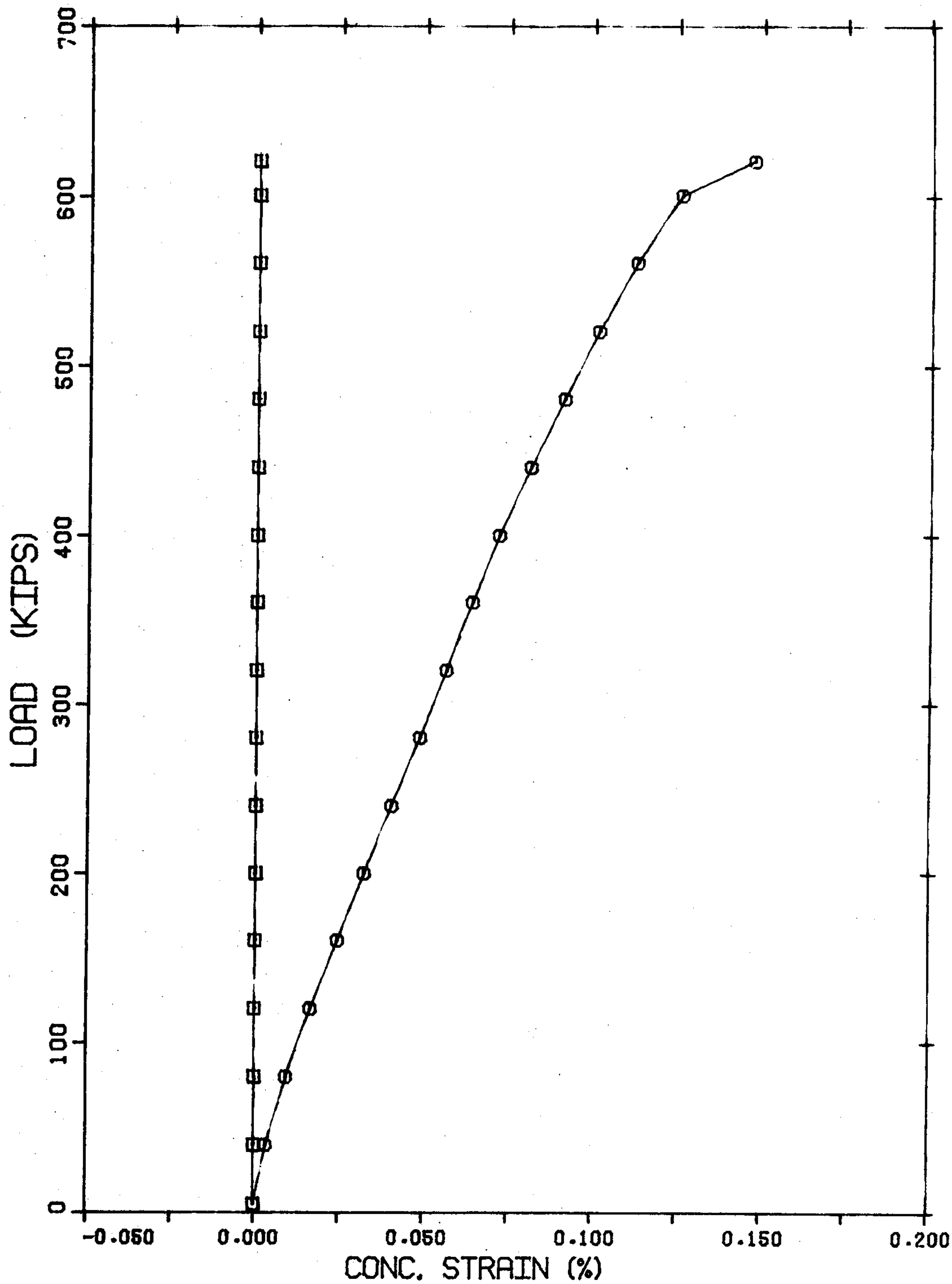

FIGURE B5.15 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B5.2

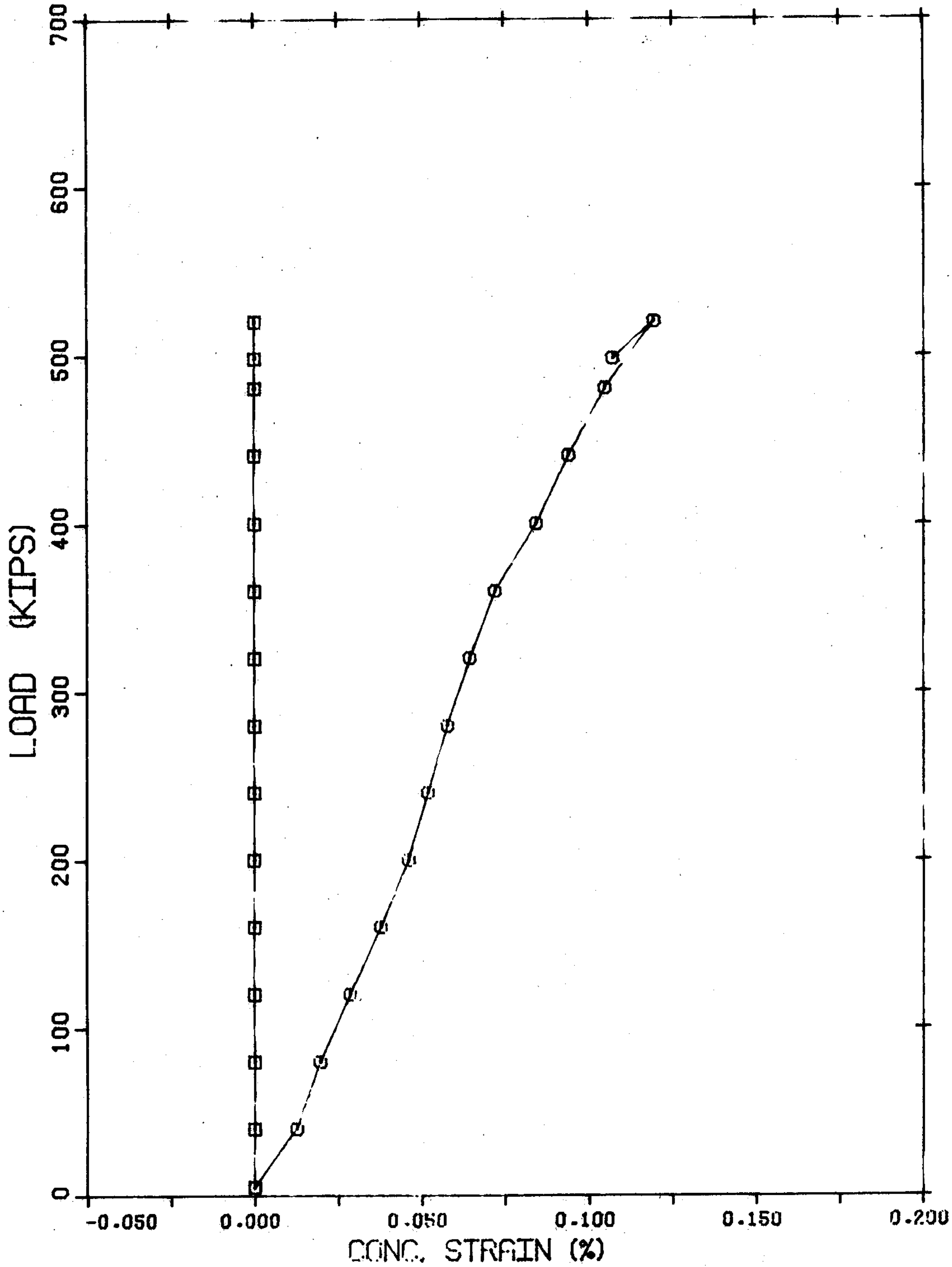

FIGURE B5.16 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN B6.1

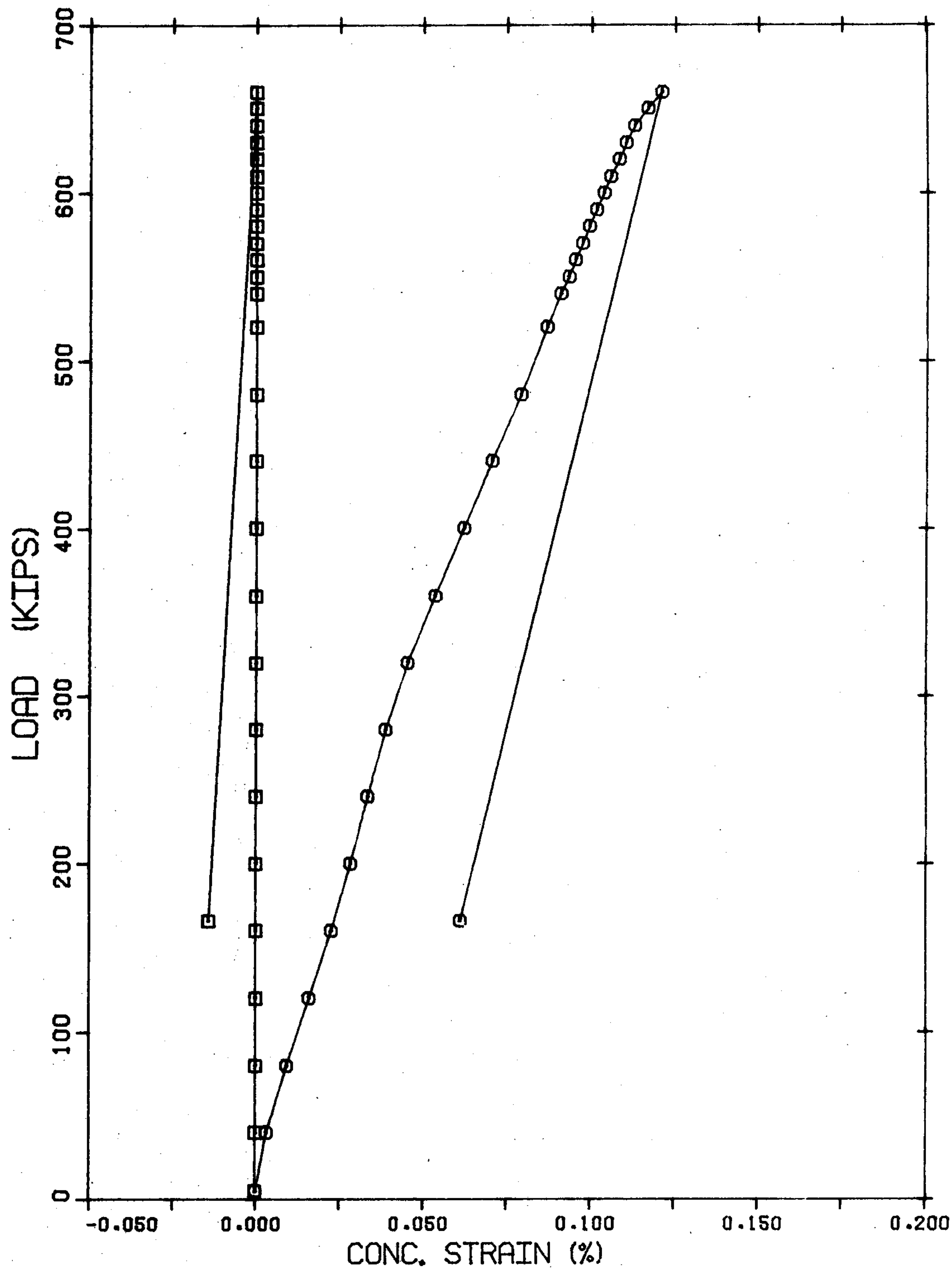

FIGURE B5.17 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B6.2

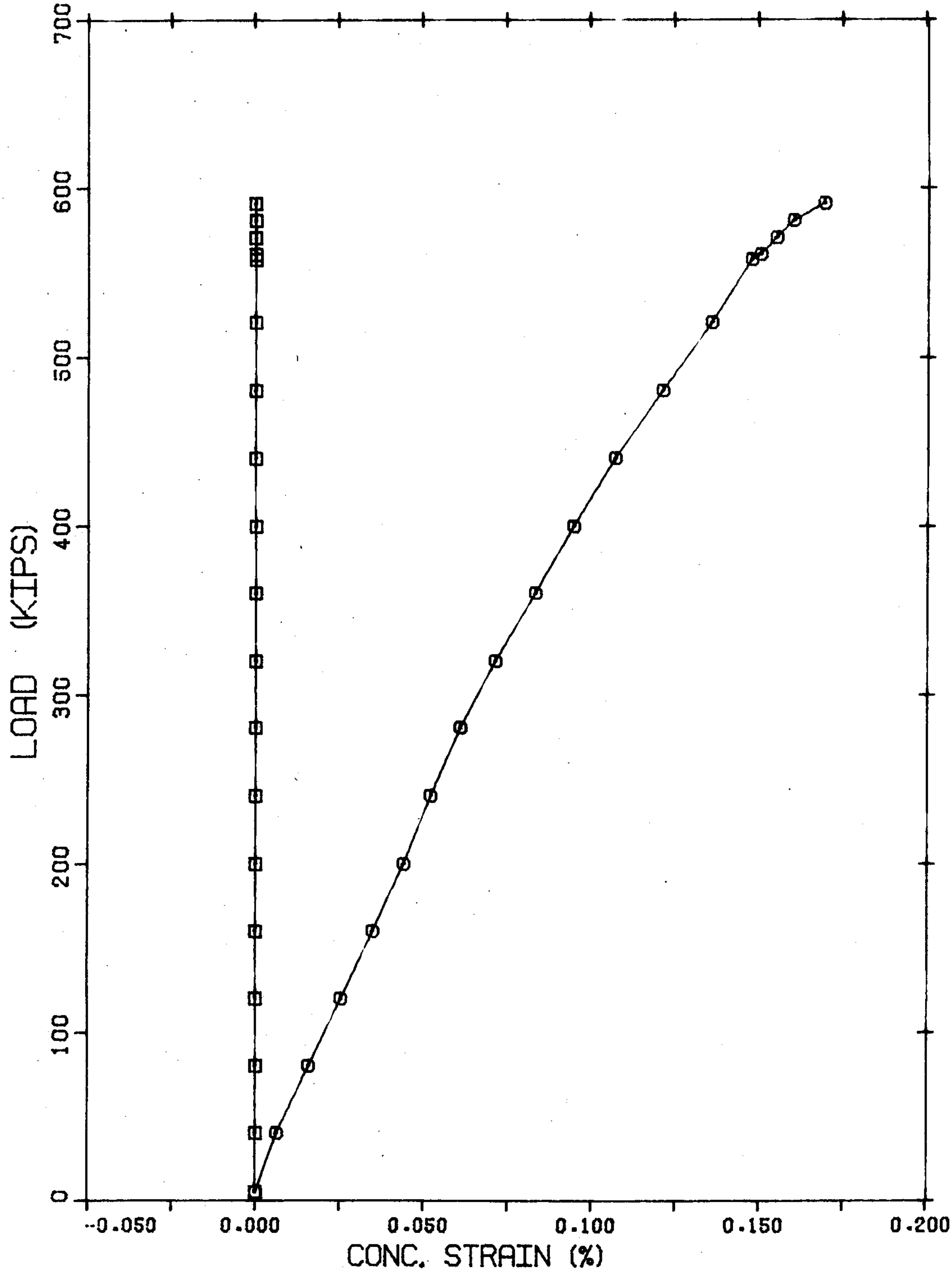

FIGURE B5.18 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN B7.1

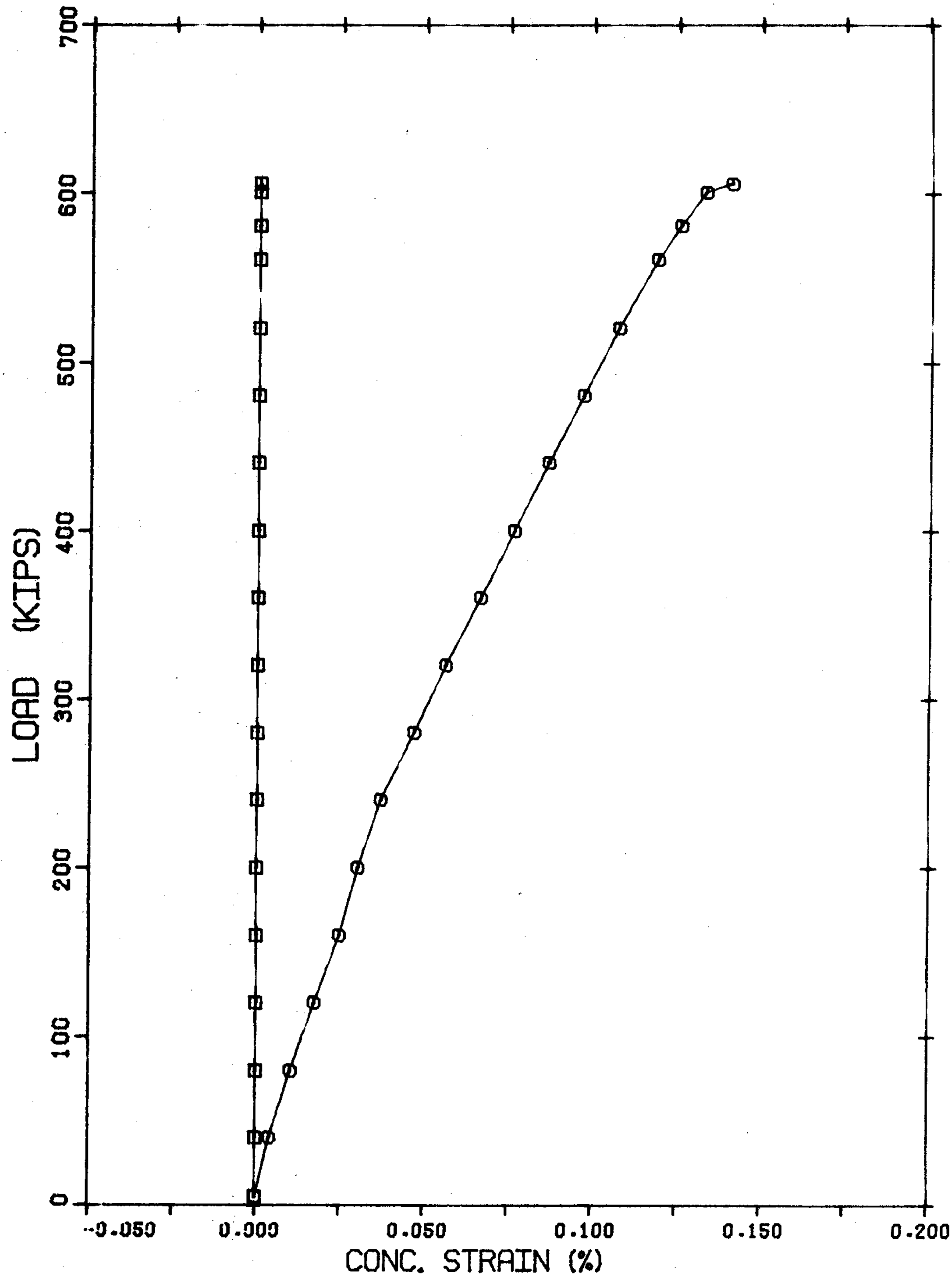

FIGURE B5.19 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN B7.2

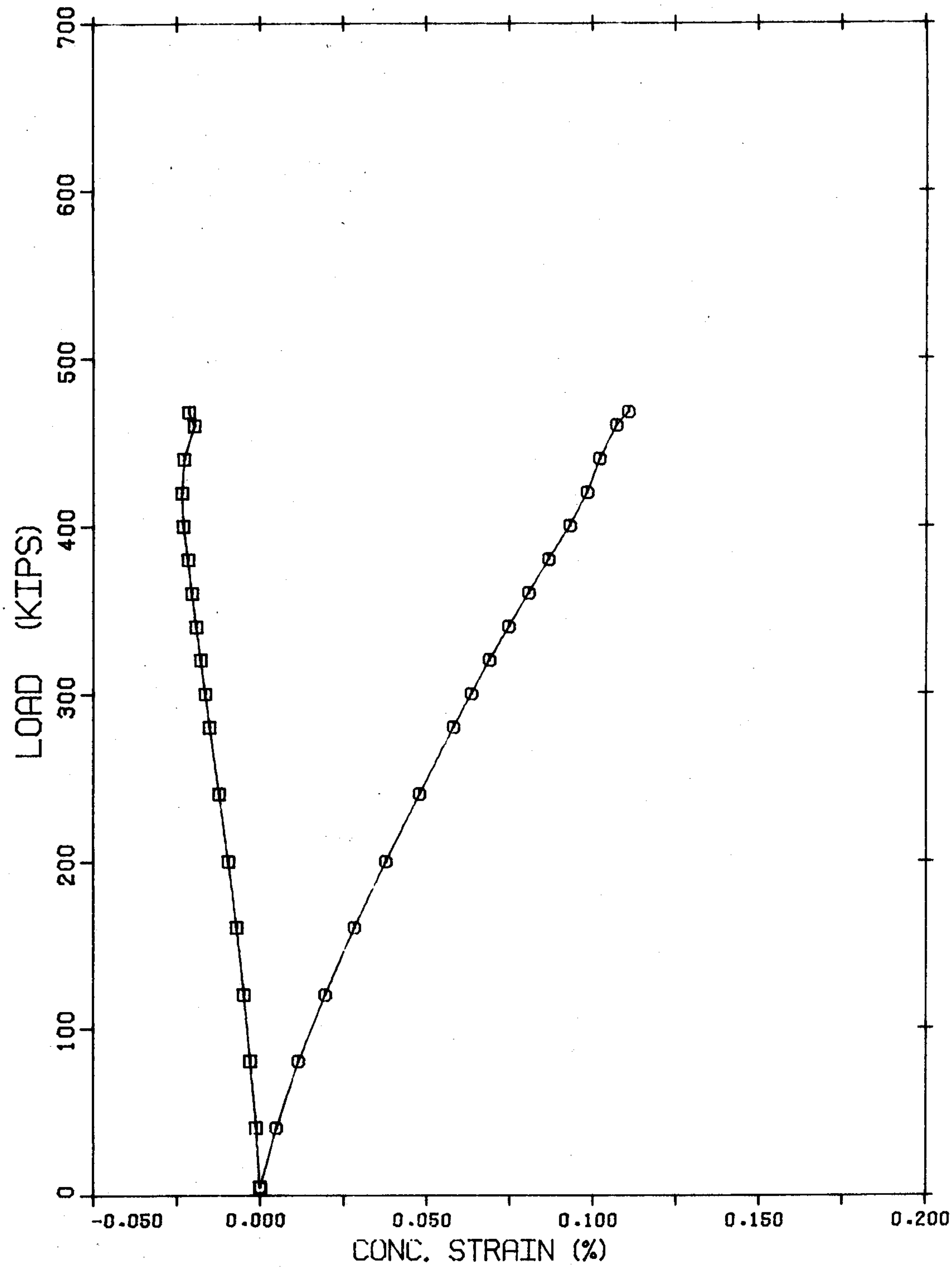

FIGURE B5.20 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN C1.1

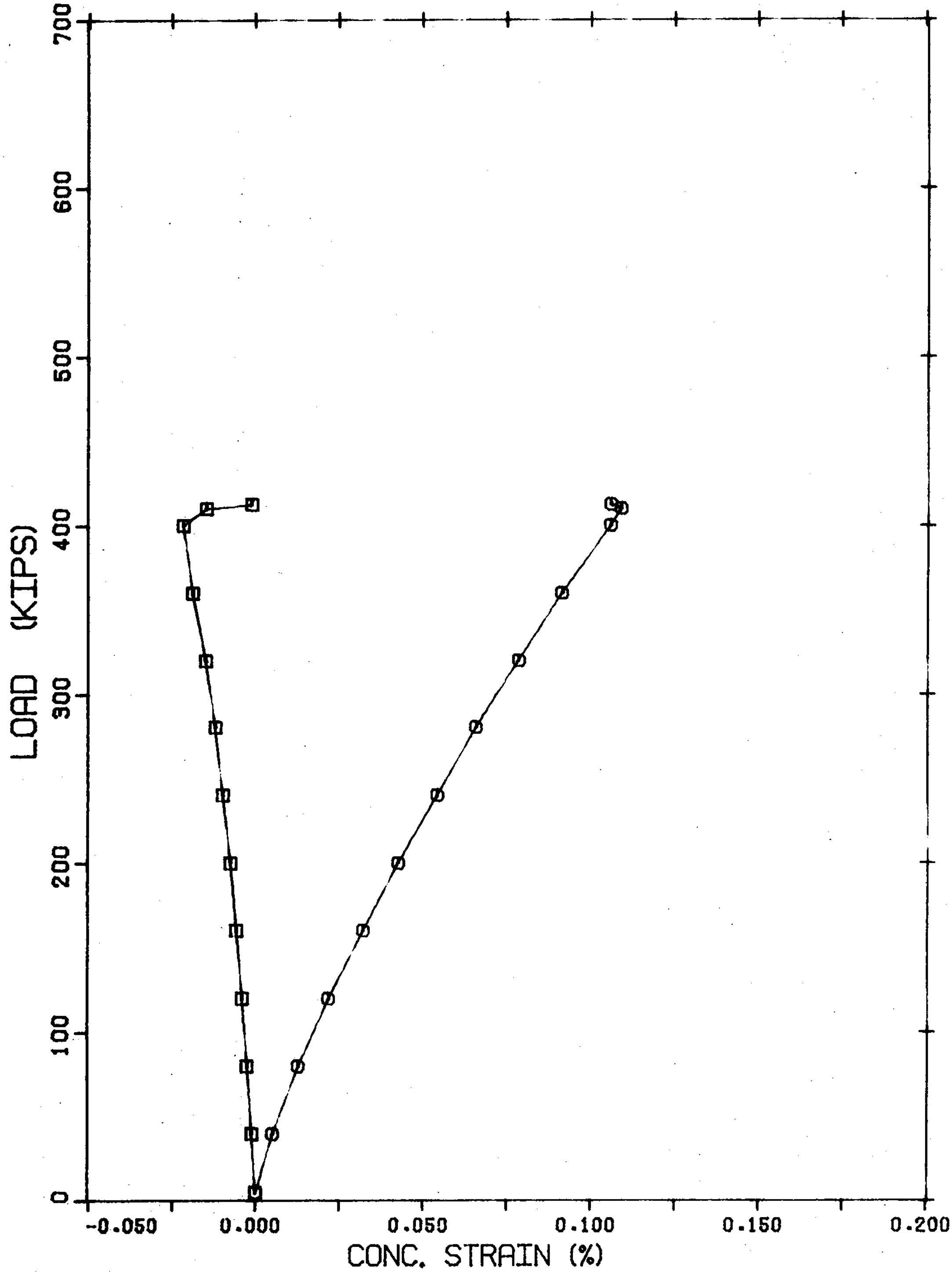

FIGURE B5.21 LOAD VS SURFACE CONCRETE STRAIN

FOR COLUMN C1.2

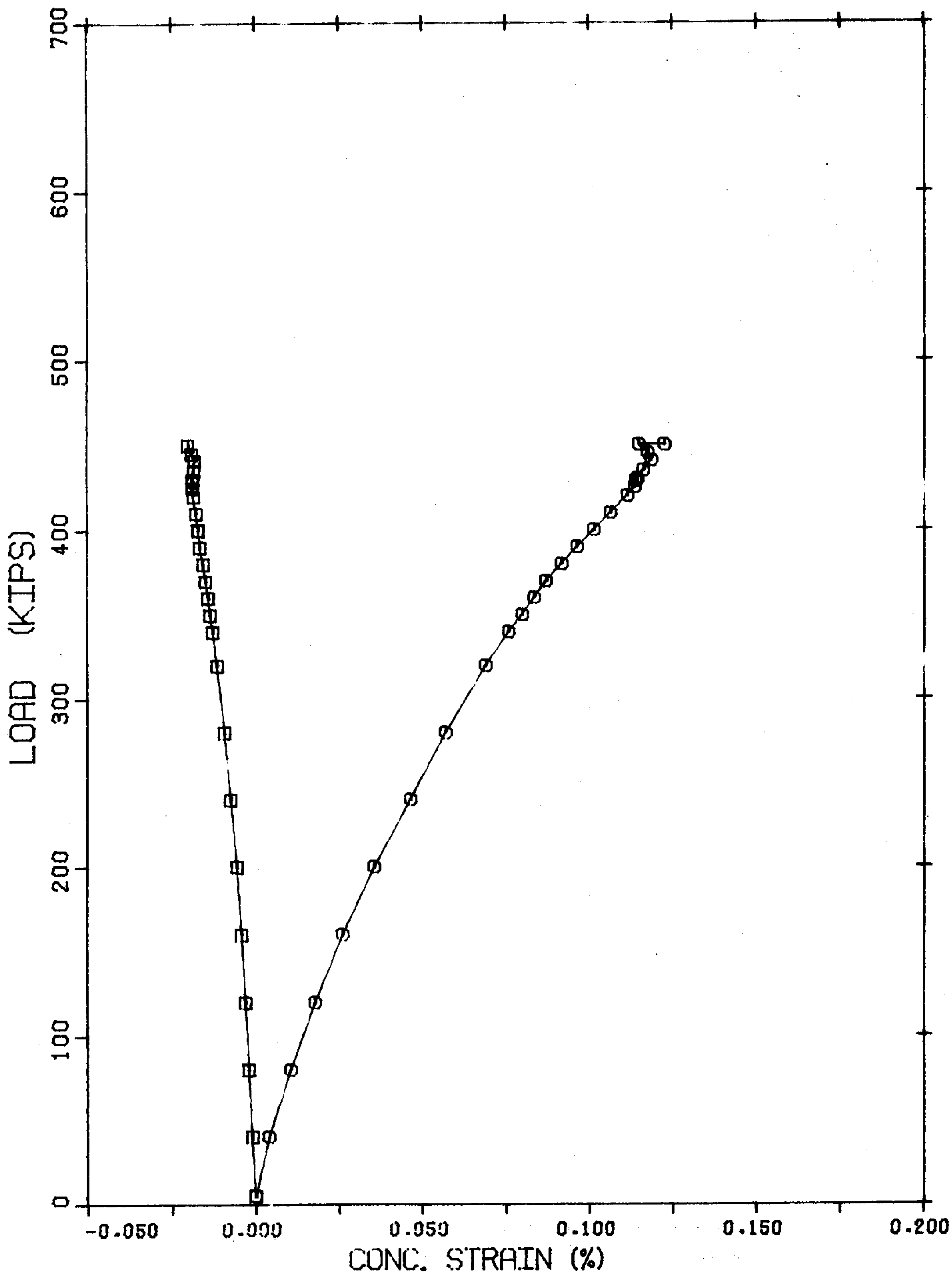

FIGURE B5.22 LOAD VS SURFACE CONCRETE STRAIN

FOR COLUMN C1.3

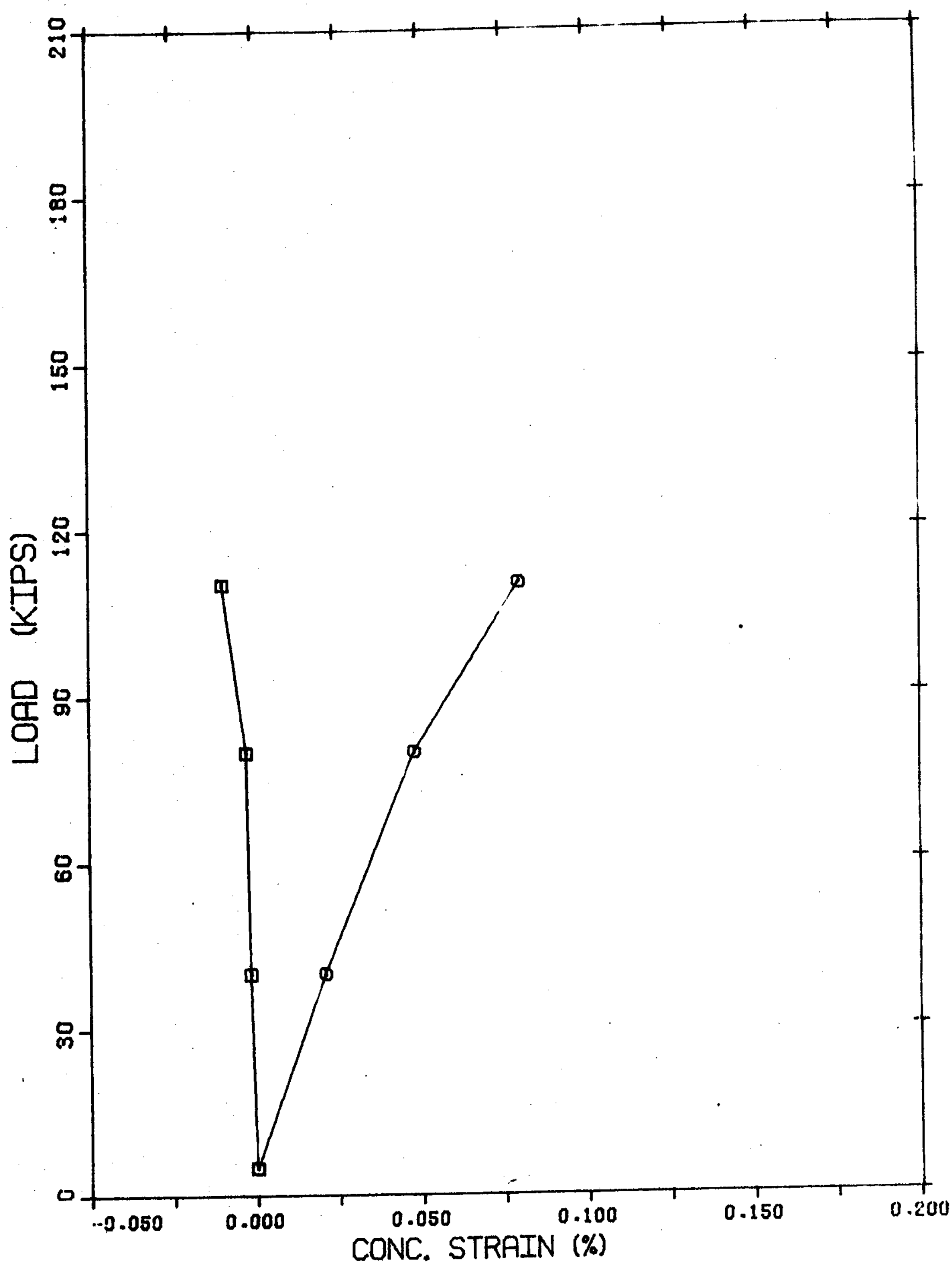

FIGURE B5.23 LOAD VS SURFACE CONCRETE STRAIN
FOR COLUMN C2.1

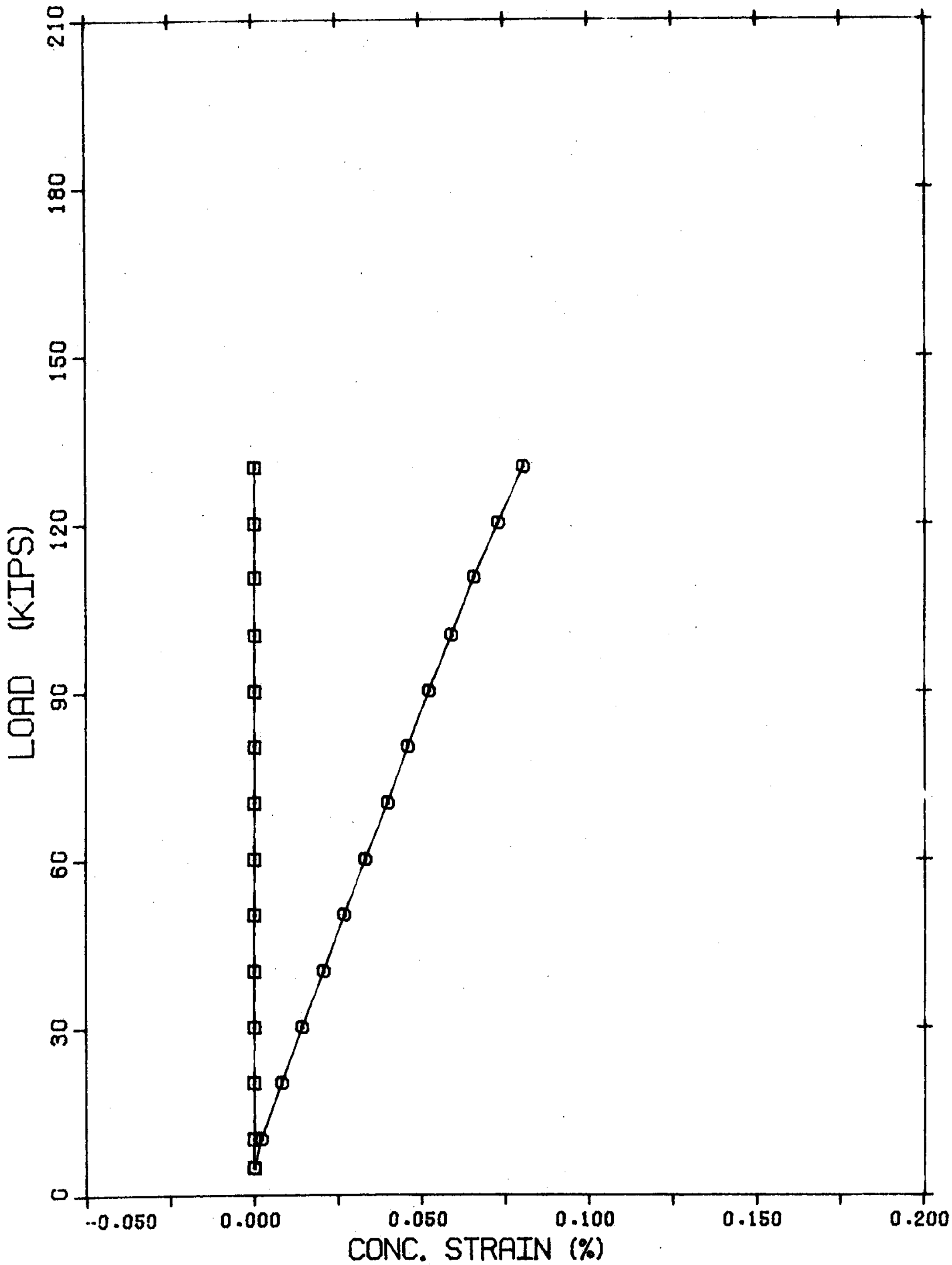

FIGURE B5.24 LOAD VS SURFACE CONCRETE STRAIN

FOR COLUMN C2.2

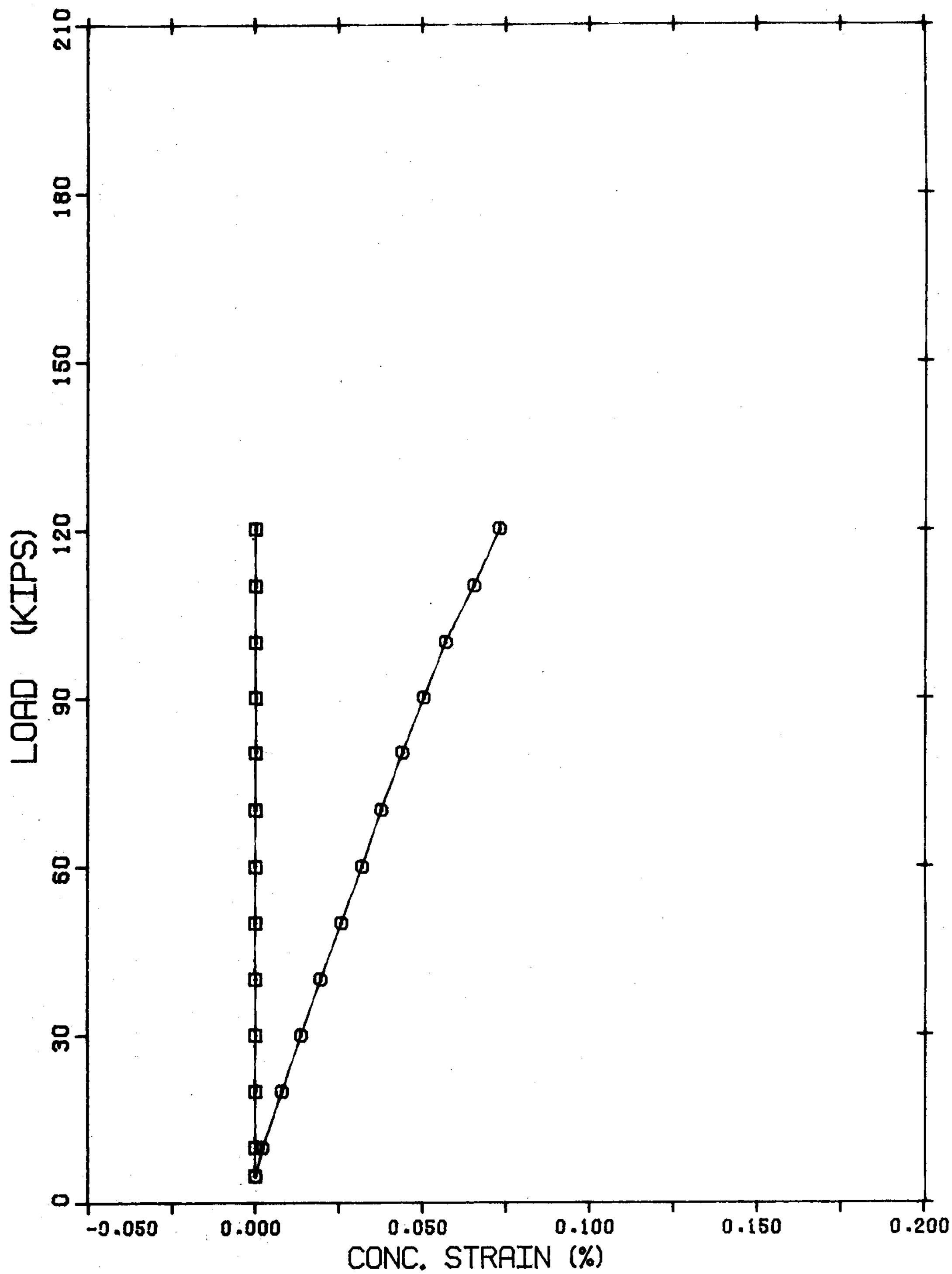

FIGURE B5.25 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN C2.3

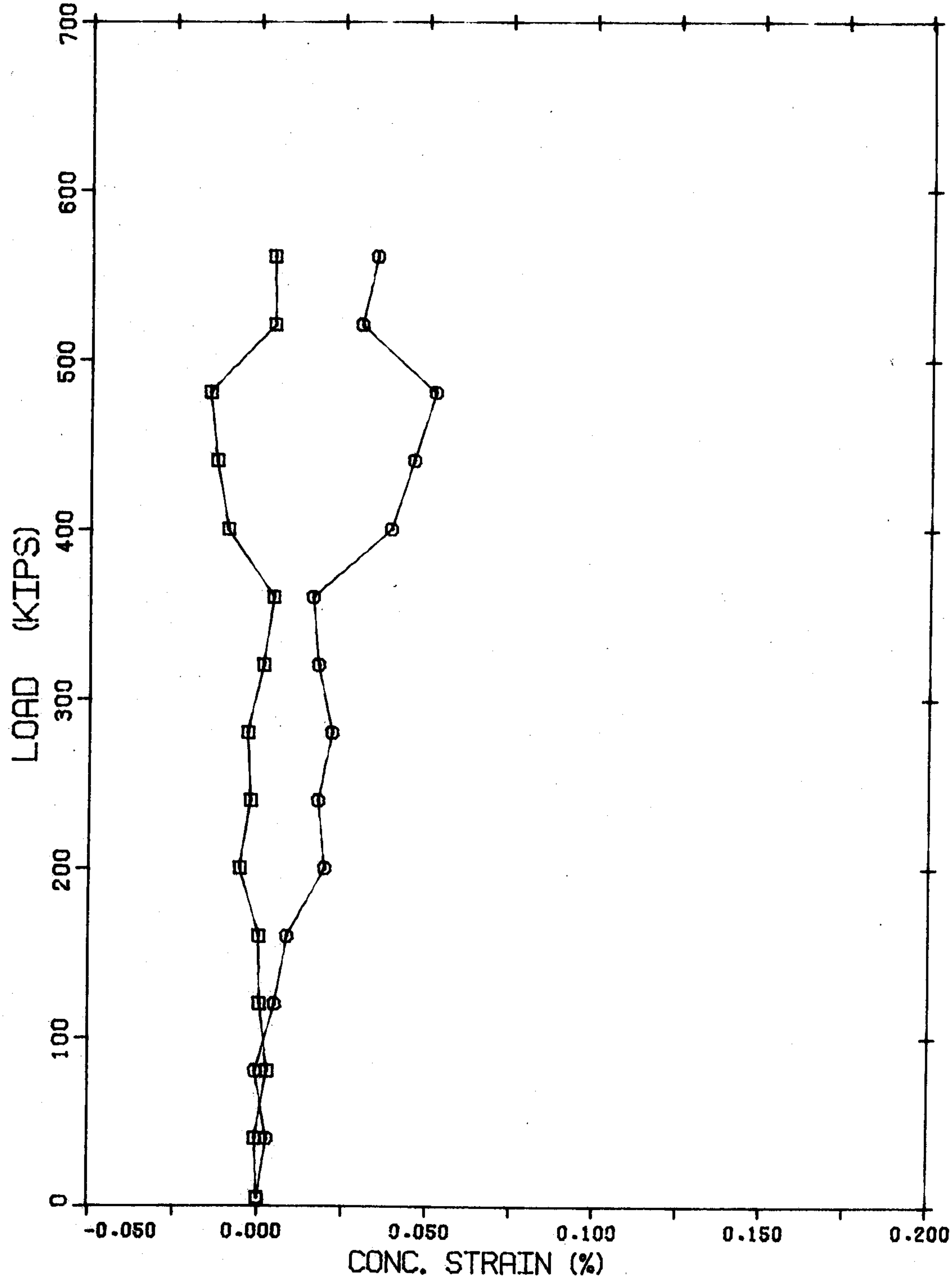

FIGURE B5.26 LOAD VS SURFACE CONCRETE STRAIN FOR COLUMN D1.2

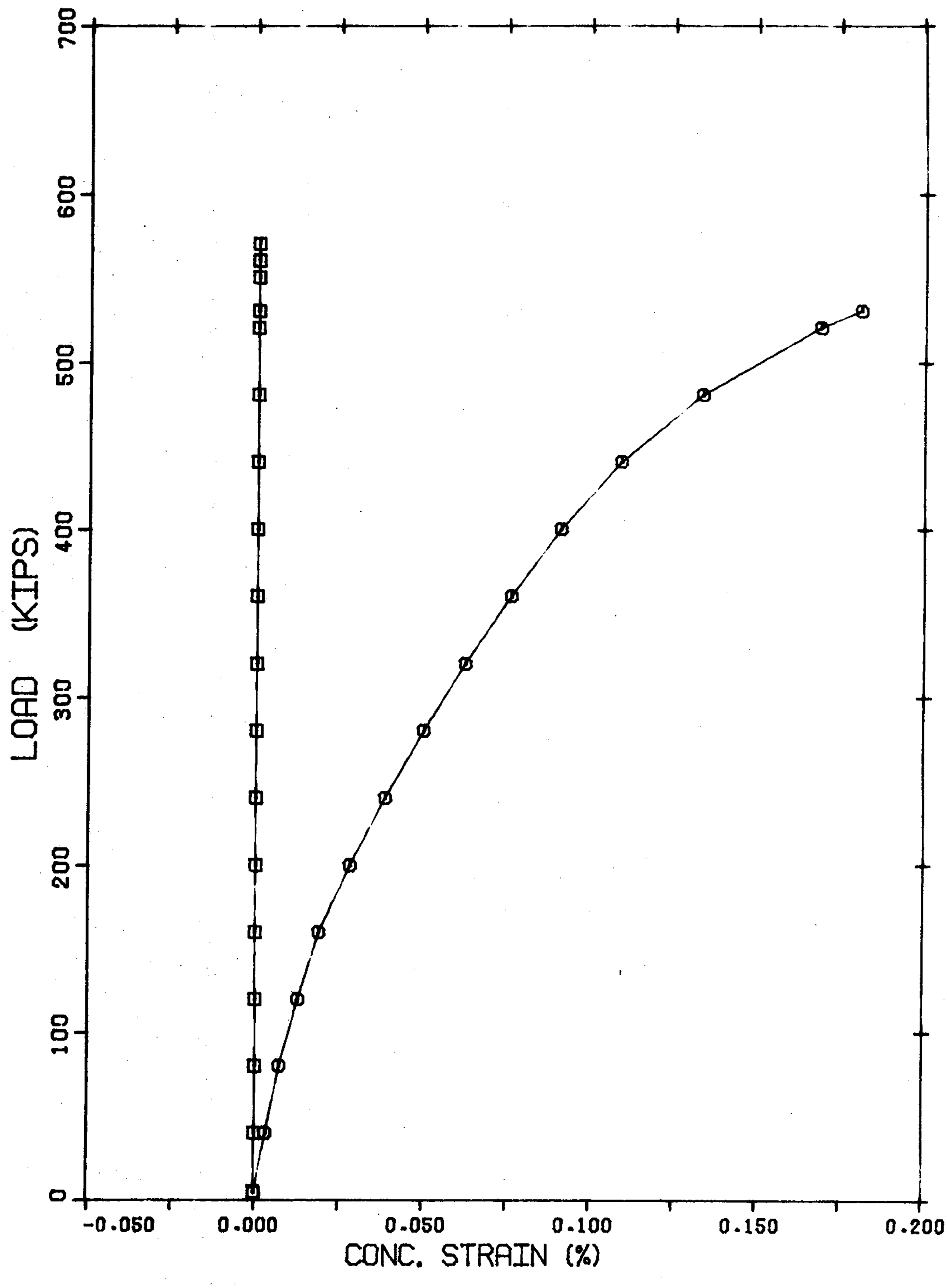

FIGURE B5.27 LOAD VS SURFACE CONCRETE STRAIN

FOR COLUMN D2.1