1973

Analysis of reinforced concrete pile caps

Abayomi O. Oyeledun
Lehigh University

ANALYSIS OF REINFORCED CONCRETE PILE CAPS

by

ABAYOMI O. OYELEDUN

A Thesis

Presented to the Graduate Committee

of Lehigh University

in Candidacy for the Degree of

Master of Science

in

Civil Engineering

June 1973

This thesis is accepted and approved in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

May 1, 1973

Roger G. Slutter

Roger G. Slutter
Professor in Charge

David A. VanHorn

David A. VanHorn
Chairman, Department of
Civil Engineering

ACKNOWLEDGMENT

This thesis represents the results of full scale tests of reinforced concrete pile caps conducted at the Fritz Engineering Laboratory of Lehigh University, Bethlehem, Pennsylvania. Dr. D. A. VanHorn is the Chairman of the Department of Civil Engineering and Dr. L. S. Beedle is the Director of the Laboratory. This investigation was sponsored by the American Iron and Steel Institute.

The author wishes to thank Dr. Roger G. Slutter, his thesis director, for his interest, advice, and guidance throughout the investigation and writing of this thesis.

Special thanks are due Mrs. Dorothy F. Fielding who typed the manuscript, the personnel of the Operations Division, the Machine Shop and the Graphic Office of the Laboratory.

TABLE OF CONTENTS

		Page
	ABSTRACT	1
1.	INTRODUCTION	3
2.	TEST SPECIMENS AND TESTING PROCEDURE	5
	2.1 Instrumentation	6
	2.2 Testing Procedure	6
3.	TEST RESULTS	7
4.	THEORETICAL ANALYSIS	10
5.	ANALYSIS OF TEST RESULTS AND CONCLUSIONS	14
	TABLES	19
	FIGURES	21
	REFERENCES	39
	APPENDIX A - PILE CAP ELEVATION	40
	APPENDIX B - SECTION PROPERTIES - TEST 1	42
	TEST 2	43
	APPENDIX C - COMPARISON OF ULTIMATE SHEAR LOAD BY ACI BUILDING CODE FORMULA AND TEST LOAD RESULT	44
	VITA	49

LIST OF TABLES

Table		Page
1	CONCRETE COMPRESSION TEST DATA	19
2	COUPON TESTS OF PILE MATERIAL	20

LIST OF FIGURES

Figure		Page
1	PILE CAP 1 - SOUTH ELEVATION	21
2	PLAN VIEW OF PILE CAP 1	22
3	PILE CAP 2 - SOUTH ELEVATION	23
4	PLAN VIEW OF PILE CAP 2	24
5	SETUP FOR TESTING PILE CAP	25
6a	SOUTH FACE OF PILE CAP 1 - CRACK PATTERNS	26
6b	SOUTH FACE OF PILE CAP 2 - CRACK PATTERNS	26
7a	WEST END OF PILE CAP 1 - CRACK PATTERNS	27
7b	WEST END OF PILE CAP 2 - CRACK PATTERNS	27
8	STRESS IN REINFORCING STEEL - CAP 1	28
9	PENETRATION OF PILING INTO CAP 1	29
10	DISTRIBUTION OF LOADS IN PILES - TEST 1	30
11	LOAD DEFLECTION CURVE - CAP 2 (LONGITUDINAL DEFLECTION)	31
12	LOAD DEFLECTION CURVE - CAP 2 (TRANSVERSE DEFLECTION)	32
13	STRESS IN REINFORCING STEEL - CAP 2	33
14	BUCKLING IN PILE C-2	34
15	PENETRATION OF PILING INTO CAP 2	35
16	DISTRIBUTION OF LOADS IN PILES - TEST 2	36
17	PILES AFTER TESTING - TEST 2	37
18	CONCRETE AT EXTERIOR CORNER AFTER REMOVAL OF PILE - TEST 2	37
19	CONCRETE AT CENTER AFTER REMOVAL OF PILES - TEST 2	38

ABSTRACT

This report describes full scale tests of reinforced concrete pile caps. The testing program consisted of two pile caps with six steel H-piles embedded six inches into each cap. The cap was concentrically loaded in both tests by applying a column load on a wide-flange column seated on a steel base plate.

The purpose of the investigation was to study

a. the mode of failure at ultimate load,

b. bond anchorage of high strength reinforcing bars,

c. the effect of pile cap plates on the end condition of embedded piles, and

d. load distribution among the piles at ultimate load.

The pile cap design depends to a great extent on the arrangement of piles in the group. The caps tested were designed for a specified working load and loading applicable to the steel H-piles independent of whether the piles were used as end-bearing piles or as friction piles.

Diagonal cracks were developed at high loads and a sketch of the crack patterns revealed a typical arch pattern. The caps failed in shear and the ACI Code equations for computing shear stresses in deep

beams gave good correlation with test load values. It was determined that web reinforcement was not necessary for reinforced concrete pile caps. In one test the yield load of the piles was reached at the ultimate load.

1. INTRODUCTION

Column loads are normally transferred to a group of piles by a pile cap. This is a structural element which closely resembles a spread footing and which in fact is sometimes termed a pile footing. The cap is usually a reinforced concrete member, the size of which depends on the number and spacings of the piles in a group. The design must also provide for the specified edge thickness and clearances for the reinforcing bars. The design is based on the assumption that the cap would derive no support whatever from contact with the soil on which it was formed.

The American Concrete Institute Building Code(1) includes provisions for the design of footings with regards to:

1. Loads and reactions
2. Bending moment
3. Shear and development of reinforcement
4. Minimum edge thickness

These provisions may be found in Chapter 15 of the Code and in other related chapters as well.

The two pile caps tested in this investigation were designed according to the provisions of the American Concrete Institute Standard Building Code Requirements for Reinforced Concrete (ACI 318-63)(2). The design was based on Working Stress Design for concrete pile caps without web reinforcement. The depth to the centroid of the reinforcing steel was computed to satisfy flexure and shear stress requirements. Computations for reinforcing steel are based on using bars having deformations conforming to American Standard of Testing Materials Requirements (ASTM A615). Appendix A shows typical elevation of a pile cap. The section properties of the two pile caps tested are given in Appendix B.

Computations and design tables could be found in the United States Steel publication on pile cap design for H-pile foundations[3].

Six steel H-piles were embedded six inches into each cap. The cap was concentrically loaded in both tests by applying a column load on a wide-flange column seated on a steel base plate.

The pile cap was treated as a deep beam member in this report and it had been found that design based on shear load at ultimate is more appropriate than earlier designs which had been based on the maximum bending moment at the critical section. The critical section was taken midway between the column face and the edge of the base plate.

2. TEST SPECIMENS AND TESTING PROCEDURE

Each pile cap was designed for a different working load. Figures 1 through 4 show the dimensions and reinforcing steel for the caps tested. The concrete was moist cured for two weeks and air cured for two weeks prior to testing. The average concrete strength from compression tests of cylinders is shown in Table 1.

The 10HP42 pile sections were embedded six inches into the pile cap. Table 2 shows the results of coupon tests of pile sections used in the second test. The piles were spaced thirty-six inches center to center. Pile spacing requirements are covered in most building codes. Spacing in general should be based on practical considerations. If the piles are spaced too closely together, they may be both inefficient and uneconomical. Piles spaced too far apart will increase the cost of the footings without commensurate benefit. Piles bearing on rock are normally spaced center to center, a minimum distance of 1.75 times the diagonal of the pile section but not less than twenty-four inches. Piles driven into other material are usually spaced center to center, a minimum distance of 1.75 times the diagonal but not less than thirty inches. In certain cases it may be advisable to increase this spacing to secure development of somewhat greater loads per individual pile.

The main reinforcing steel was placed nine inches from the bottom. The reinforcing steel had a specified minimum yield strength of 60 ksi. No end hooks were used since the deformed bars used were able to develop adequate bond with the concrete.

2.1 Instrumentation

The piles were seated on homosote pads and bearing plates so as to provide a more uniform distribution of loading throughout the test. Dial gages were used to measure settlement of the homosote pads, deflections of the pile cap, penetrations of the piles and overall movement of the test specimen. Mechanical strain gage points were located on the concrete surface at the level of the main reinforcing steel. Electrical resistance strain gages were placed on the reinforcing steel to provide an indication of stresses in the steel. Electrical strain gages were also used on the pile sections. All the concrete surfaces were white washed so that the crack patterns could be observed.

2.2 Testing Procedure

The column load was applied at the geometric center of the pile cap by a concentrically loaded wide-flange column on a steel base plate. The specimens were tested in a 5,000,000 lb. Universal Testing Machine. Figure 5 shows a typical test setup. The procedure during testing was to apply an increment of load, read all instrumentation, examine the specimen for cracking of the concrete and inspect for yield lines on the exposed surface of the pile sections.

3. TEST RESULTS

The pile cap in Test 1[4] was designed for a 900 kip column load. This is a design load of 150 kip load per pile.

The pile cap was initially very stiff and at the working load of 900 kips, the centerline deflection was only 0.018 inches. A record of the crack patterns was made. There were no visible cracks up to a load of 450 kips and at the working load of 900 kips, the centerline crack had penetrated to about 15 inches from the top of the cap. Horizontal cracking at the level of reinforcing steel began at a load of 1050 kips. This, however, propagated to nearly the full length of the reinforcing steel before failure of the cap. The crack patterns on both the south face and the west end of the cap are sketched in Figs. 6a and 7a.

The main reinforcing steel stress increased slowly until a large amount of cracking had developed at an applied load of 1050 kips. Some of the reinforcing had yielded at ultimate load but the stress in most of the reinforcing bars remained below yield. The reinforcing steel stress versus load is plotted in Fig. 8.

The pile sections without the steel cap plates had penetrated twice as much as the pile sections with the cap plates. Pile penetration measurements are shown in Fig. 9. The load on individual piles throughout the test versus applied load could be obtained from the strain gages mounted on the pile sections. The load distribution in the piles was relatively uniform up to a load of 1800 kips. Load distribution in the piles is shown in Fig. 10.

The pile cap in Test 2[5] was designed for a 1500 kip column load. This is a design load of 250 kip load per pile.

The deflection data for this test are summarized in two sets of curves. The curves in Fig. 11 give the average deflection at the center of the pile cap relative to the deflection measured at the four corners of the cap. At the working load of 1500 kips the deflection at the center exceeded that of the corners by 0.01 inch. The transverse deflection of the pile cap is shown in Fig. 12. These curves were obtained by plotting the difference between the average of the deflection measured at the center of the end faces and the average deflection measured at the corners.

Cracks due to flexure were first observed at a load of 900 kips. At the working load level of 1500 kips the principal flexural crack had penetrated to the middle height of the pile cap. Cracks, however, were not visible on the ends of the pile cap until a load of 2100 kips was reached. At 2500 kips the principal cracks on the ends had extended to middle height of the cap. The crack patterns on the south face and the west end are sketched in Figs. 6b and 7b.

A curve of reinforcing steel stress versus applied load is shown in Fig. 13. The stress indicated in this curve is the stress in the #4 bars which were located six inches above the main steel.

The stress level remained below the level of the yield stress in the main steel throughout the test.

Pile penetration into the pile cap was highest for the piles on the east end (Piles C1 and C2). Figure 14 shows Pile C2 which had buckled noticeably at the end of the test. The center piles exhibited the next largest penetration and the piles on the west end penetrated the least amount. At the working load of 1500 kips the average amount of penetration was approximately 0.09 inch. The curve of pile penetration versus the total applied load is shown in Fig. 15.

The load distribution among the piles in the group is shown in Fig. 16. The load at which a pile would yield based on a yield stress of 36 ksi is shown. The yielding of all six piles at the conclusion of the test was evident from flaking of the white wash. Figure 17 shows the condition of the six piles after the test. The concrete section at the end of the piles were exposed after the test. Figures 18 and 19 show crushing of the concrete at an exterior corner and center of the pile cap.

4. THEORETICAL ANALYSIS

Because of the complex nature of the stress distribution in beams failing in shear, investigators have had to resort to empirical methods to interpret test results in order to develop expressions for shear strength(6). As a result there are numerous formulas for estimating the strength of various shapes under various loading conditions.

The elastic behavior of deep beams have been shown to be different from that of more common flexural members. This difference in behavior is mainly attributed to the significant effects of vertical normal stresses and shear deformations.

In the ACI Code(1) the following equation for computing nominal shear stress for deep beams is given in Section 11.9

$$V_c = (3.5 - 2.5 \frac{M_u}{V_u d}) \times (1.9 \sqrt{f_c'} + 2500 \rho_w \frac{V_u d}{M_u}) \qquad (11\text{-}22)$$

where

$\rho_w = A_s/b_w d$

A_s = area of non-prestressed tension reinforcement

b_w = web width, or diameter of circular section

d = distance from extreme compression fiber to centroid of tension reinforcement

f_c' = compressive strength of concrete

M_u = applied design load moment at critical section

V_u = total applied design shear force at critical section

This equation expressed the nominal shear stress carried by the concrete as a function of concrete strength, the cross-sectional dimensions and the shear-to-moment ratio at the critical section along the span.

The first term of Eq. (11-22) is provided to account for the fact that it is assumed that diagonal cracking occurs at the same nominal stress as for ordinary beams, but that the shear stress carried by the concrete will be greater than the shear stress causing diagonal cracking. The first term is hence set not to exceed 2.5 which is a conservatively established limit for the ratio by which the stress is increased.

The shear stresses in excess of the shear stress causing diagonal cracking have always been thought of as causing cracks of unsightly width unless shear reinforcement is provided in the beam. However experiments have shown that there was only a slight increase in strength depending on the mode of failure when web reinforcement was included in beams[7]. It had been concluded that the presence of web reinforcement did not appear to influence the capacity beyond cracking for beams failing in shear.

The provisions of Section 11.10 of the ACI Code[1] had often been used in designing the pile cap. However the reinforced concrete pile cap can justifiably be treated as a deep beam member and be designed in accordance with the provisions of Section 11.9 of the ACI Code[1]. The justifications are as follows:

a. The member is deep with span to depth ratio of 1.20 and 0.97 for the two tests.

b. The crack pattern is that of a deep beam member with cracks in the critical region being nearly vertical rather than the 45° cracks.

c. The critical section defined in Section 11.10.2 of the ACI Code[1] is not critical in the tests conducted because of arch action.

The shear stress carried by the concrete, V_c, was computed using Equation (11-22) from the ACI Code. This was used in the shear computation in Appendix C and was found to give good correlations at ultimate load with test results.

Since proper embedment length of reinforcement would ensure the development of required tension in the reinforced concrete members, the reinforcing bars were checked for proper embedment. The ACI Code[1] equations given in Section 12.5 were used to determine the proper embedment length for the reinforcing bars.

The basic development length ℓ_d of deformed bars in tension for #11 or smaller bars is given by:

$$\ell_d = 0.04\ A_b\ f_y/\sqrt{f_c'}$$

but not less than 0.0004 $d_b\ f_y$ where

A_b = area of individual bars

d_b = nominal diameter of bars

f_c' = compressive strength of concrete

f_y = yield strength of non-prestressed reinforcement

The computations for the embedment length of reinforcing bars are shown in Appendix C. The embedment length of reinforcing bars used in the two pile caps were found to be adequate.

5. ANALYSIS OF TEST RESULTS AND CONCLUSIONS

The crack patterns of the pile caps tested are shown sketched in Figs. 6a and b and 7a and b. The inclined cracks are most important since they have the greatest influence on the behavior of deep beams. These originate near the support and propagate upward and toward mid-span of the cap. The formation of these cracks eliminates the inclined principal tensile stress necessary for beam action. This leads to a redistribution of the internal stresses and results in the formation of a tied arch. The reinforcement thus acts as the tension tie and the part of the concrete outside the inclined cracks acts as the arch rib in compression. The typical arch appearance is depicted in the sketch of the crack patterns on the face of the pile caps.

The load deflection curve plot shows linear behavior to approximately the working load level. It then becomes non-linear and exhibits a ductile behavior. The analytical determination of the pile cap deflections would require an extensive analysis because of the cracking and behavior as a deep beam. Simple beam deflection theory is probably sufficient for design to approximate the magnitude of deflection. Deflections of the pile cap tend to be small compared to deformation of the piles.

In the second test, the 10HP42 pile sections did not behave adequately as columns at ultimate load. The appearance of yield lines on the exposed surface of the piles was recorded at a load lower than the ultimate load. The static yield stress of the piling in Test 2 was 35.6 ksi (Table 2). The static yield may be as much as 15 percent lower than the yield reported in a mill report[8]. Also the effect of residual stress on the behavior of a column section is pronounced[9]. This may cause the apparent yield behavior of a column to be considerably lower than indicated by tensile coupon yield stress values. Yield lines were visible on the piles at a steel stress of approximately 30.2 ksi. The yielding of the piles produced excessive lateral pressure on the concrete which surrounds the ends of the pile. This produced additional stress resulting in failure of the pile cap as a whole.

Cap plates on the end of piles in the concrete pile cap have been used to reduce penetration of the steel section into the concrete. These plates may be made from channels, punched plates or tee sections. Reinforcing bars through burned holes in the piling have also been used for this purpose. A series of compression tests conducted by the State of Ohio, Department of Highways[10] provided evidence indicating that the strength of the connection between properly sized and reinforced concrete caps and steel H-piles was adequate without cap plates. Steel H-piles without cap plates embedded only six inches into the concrete still proved as effective as those with cap plates. This result was confirmed in Test 1 in which the two piles had cap plates while the other four were without cap plates. Although the piles without cap plates penetrated more than the two with cap plates, the strength of the pile cap was not affected by this. In Test 2 no cap plates were used.

The penetration of the center two piles in Test 1 exceeded greatly the penetration of the end piles. This was primarily due to cracking of the cap. It was this observation which led to the conclusion that the center two piles would support the same load as the end piles but no more. This assumption was used in calculating the ultimate strength of the pile caps.

The special provisions included in the ACI Code[1] for deep beams show good correlation between calculated loads and test loads at ultimate load. The equation for nominal shear stress carried by the concrete given in Section 11.9 of the ACI Code was used in the calculations in Appendix C. The ratio of test load to calculated load at ultimate $P_{test}/P_{calulated}$ was found to be 1.17 for Test 1 and 1.00 for Test 2. Yielding of the piles caused this load ratio to be lower for Test 2.

The ACI Code[1] equation in Section 12.5 was used to compute the proper embedment length required to provide adequate tension in the reinforced concrete caps (Appendix C). The embedment length of reinforcing bars was found to be sufficient for the two pile caps tested.

The ACI Code equation for required strength, Equation 9-1 in Code[1], was used to determine the range of working loads for the two pile caps if dead load to live load ratio ranges from 1.0 to 2.0. The working load varies from 1097 kips to 1134 kips for the pile cap in Test 1 and from 1658 kips to 1713 kips for the pile cap in Test 2. There were no noticeable excessive cracking of the concrete at this working load range in either of the pile caps. The deflection behavior is still within the elastic region in each case.

The following conclusions can be drawn from the results of this investigation:

1. The reinforced concrete pile cap should be treated as a deep beam member provided the dimensions meet requirements of Section 11.9 of the ACI Code(1).
2. The depth to the centroid of the reinforcing steel can be computed to satisfy the requirements for shear stress using the special provisions for deep beams in the ACI Code(1).
3. Anchorage of reinforcing steel will not be required as long as the proper embedment length of reinforcing bars is used according to the provisions of Section 12.5 of the ACI Code(1).
4. There is no need to use web reinforcement in design of pile caps as long as it can be treated as a deep beam.
5. Care should be taken in choosing adequate steel H-pile sections.

6. Steel plates are not required on top of the embeded ends of steel H-piles in the concrete cap.

TABLE 1 CONCRETE COMPRESSION TEST DATA

	Specimen Number	Age (days)	Load (lbs.)	Compression Stress (psi)
Test 1				
	1	28	117,730	4160
	2	↓	117,730	4160
	3	↓	116,600	4120
	4	↓	117,730	4160
	5	↓	116,600	4120
	6	↓	118,860	4200
		Average		4150
Test 2				
	1	28	124,500	4400
	2	↓	119,500	4220
	3	↓	126,500	4470
	4	↓	131,500	4650
	5	↓	124,500	4400
		Average		4430

TABLE 1a SPLITTING CYLINDER TESTS

	Specimen Number	Age (days)	Load (lbs.)	f'_{sp} (psi)
Test 2				
	1	28	45,250	400
	2	↓	52,000	460
	3	↓	52,500	464
	4	↓	43,500	385
		Average		427

TABLE 2 COUPON TESTS OF PILE MATERIAL

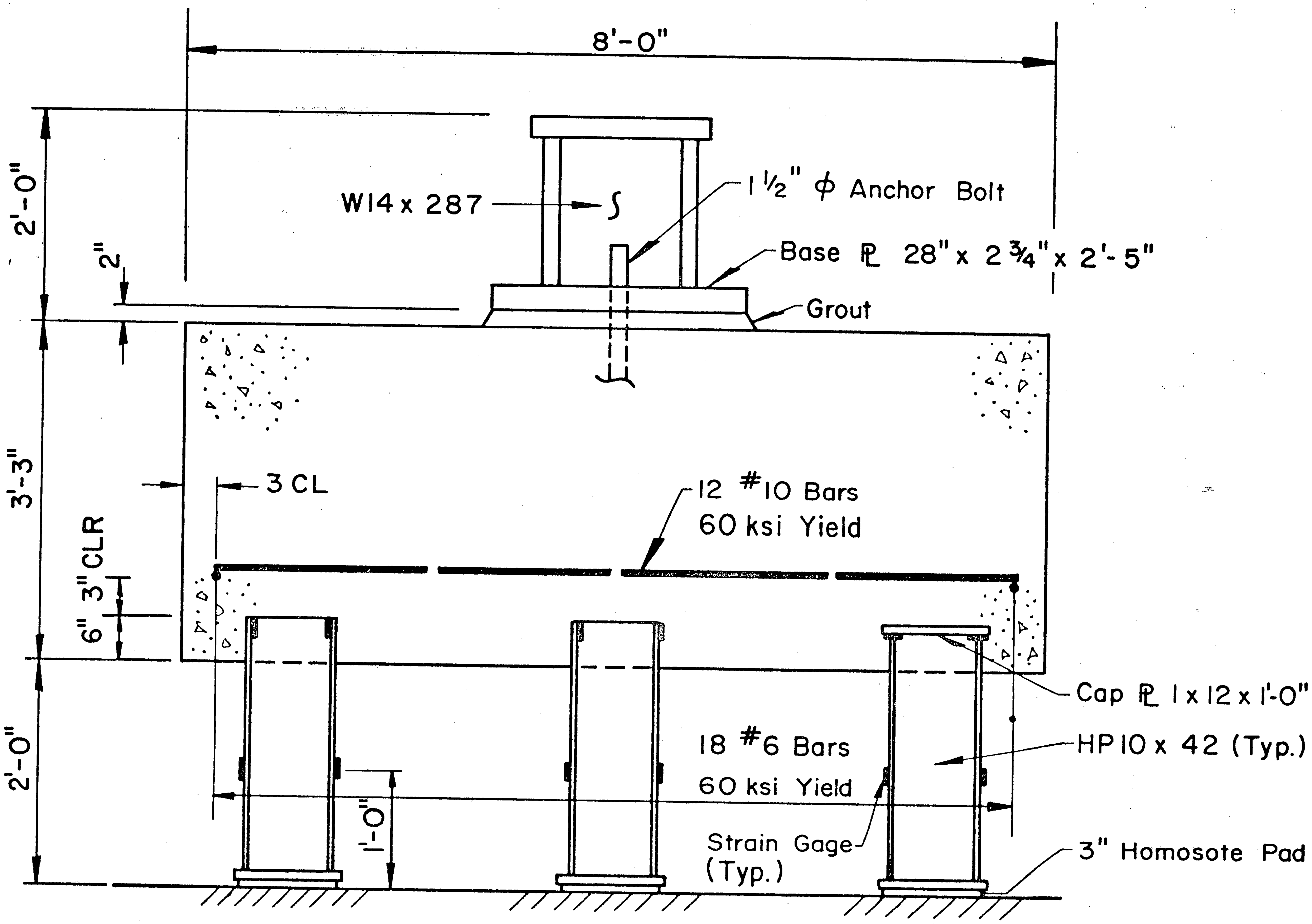
Test 2 - Flange Specimens

(a) Tension Test

	(1)	(2)
Upper Yield Stress (ksi)	40.6	40.2
Dynamic Yield Stress (ksi)	39.7	39.3
Static Yield Stress (ksi)	35.6	36.8
Tensile Strength (ksi)	63.2	63.4
Elongation in 4 inches (%)	40.0	33.0
Reduction in Area (%)	50.7	55.3

(b) Compression Test

Yield Point Stress (ksi)	44.8
Static Yield Stress (ksi)	40.9
Modulus of Elasticity (ksi)	30,400



PILE CAP SOUTH ELEVATION

Fig. 1 PILE CAP 1 - SOUTH ELEVATION

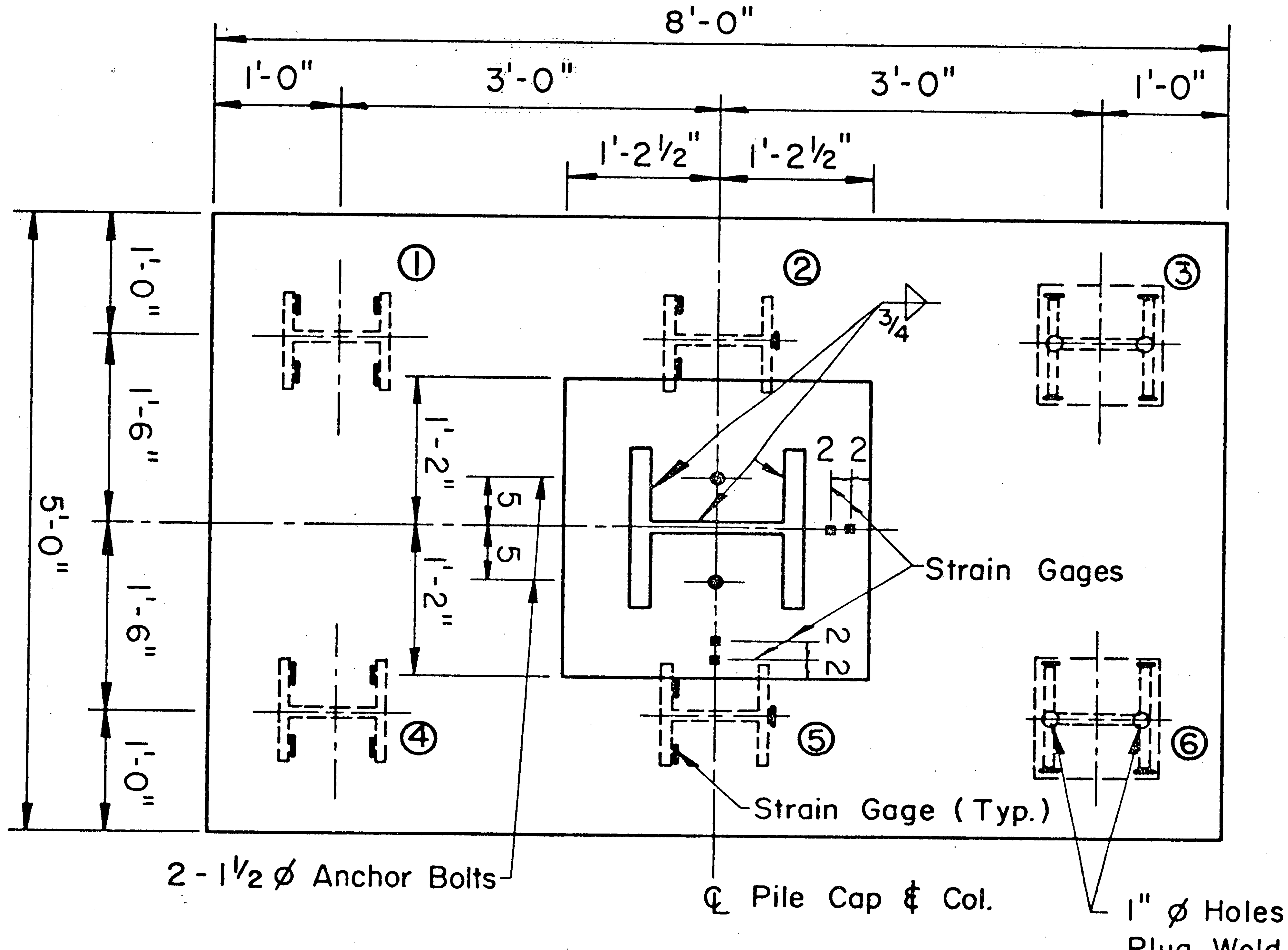


Fig. 2 PLAN VIEW OF PILE CAP 1

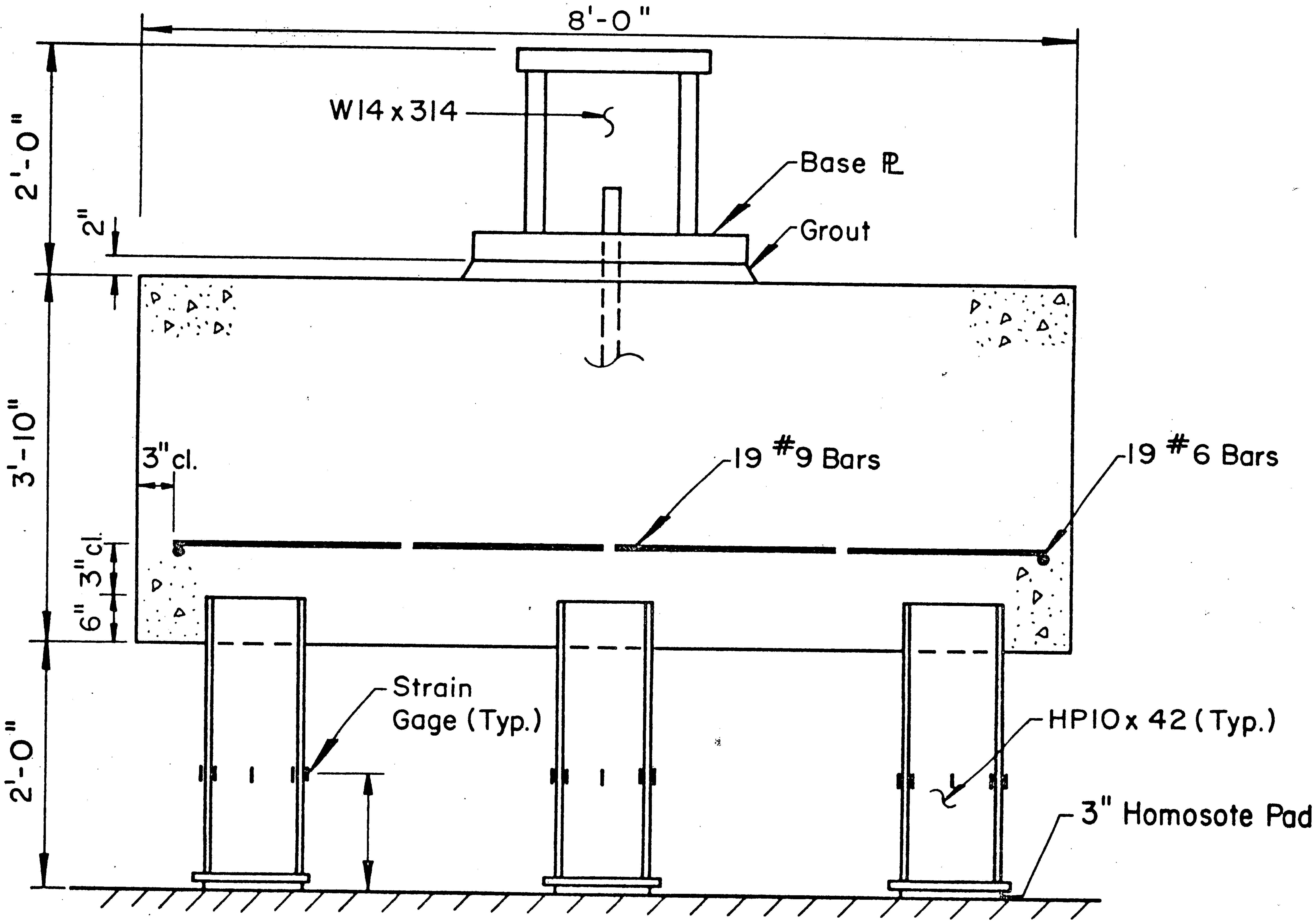


Fig. 3 PILE CAP 2 - SOUTH ELEVATION

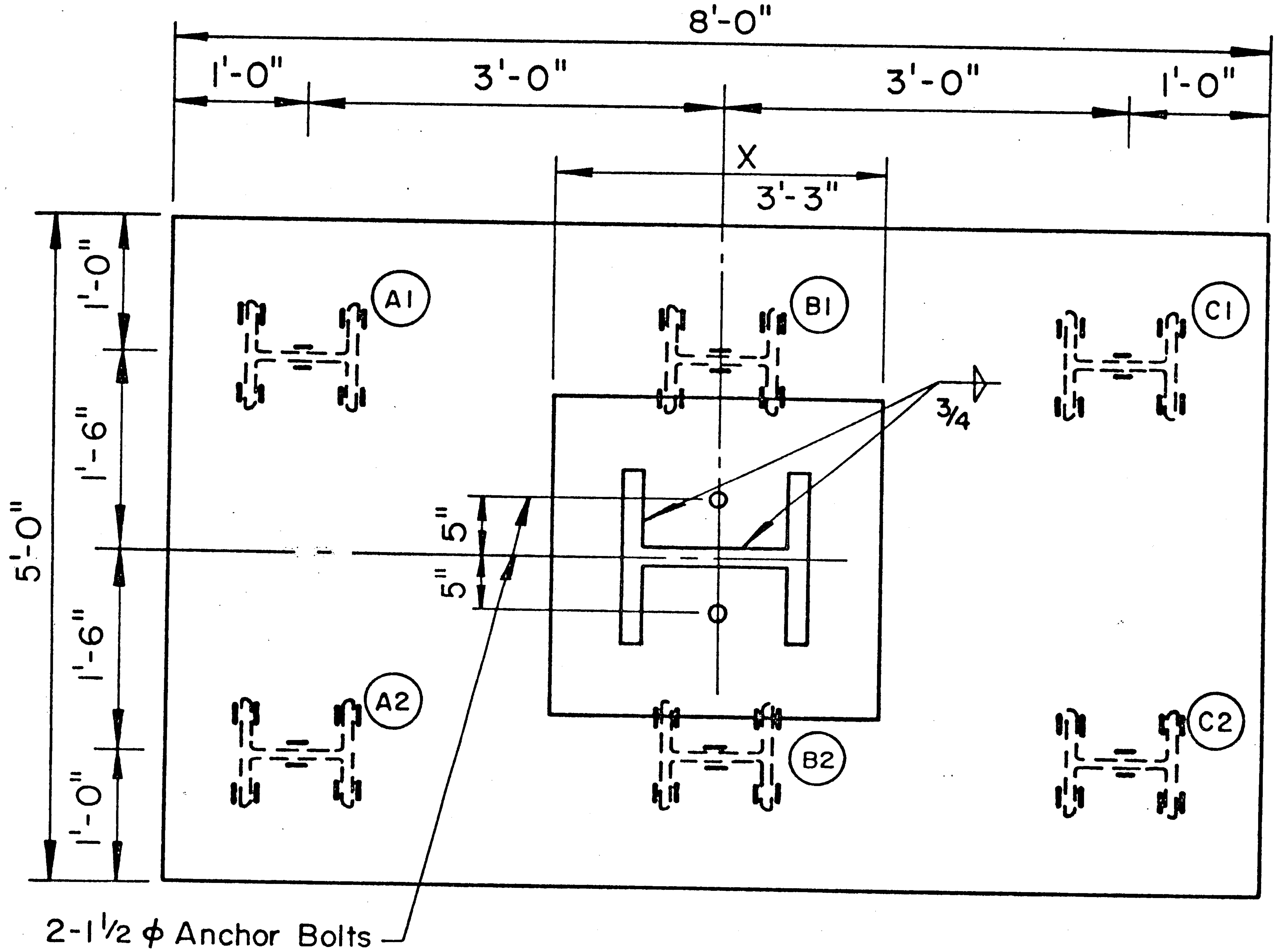


Fig. 4 PLAN VIEW OF PILE CAP 2

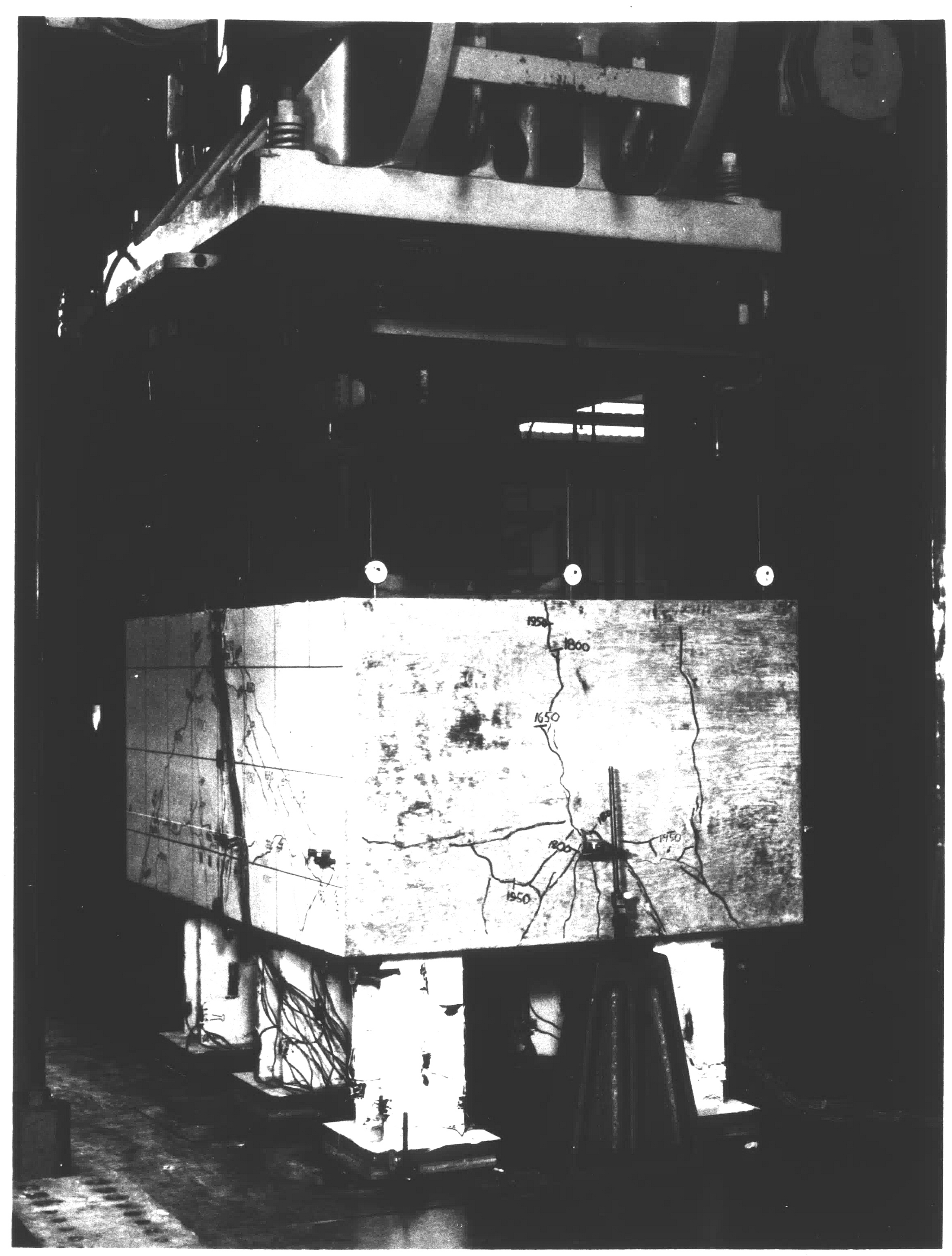


Fig. 5 SETUP FOR TESTING PILE CAP

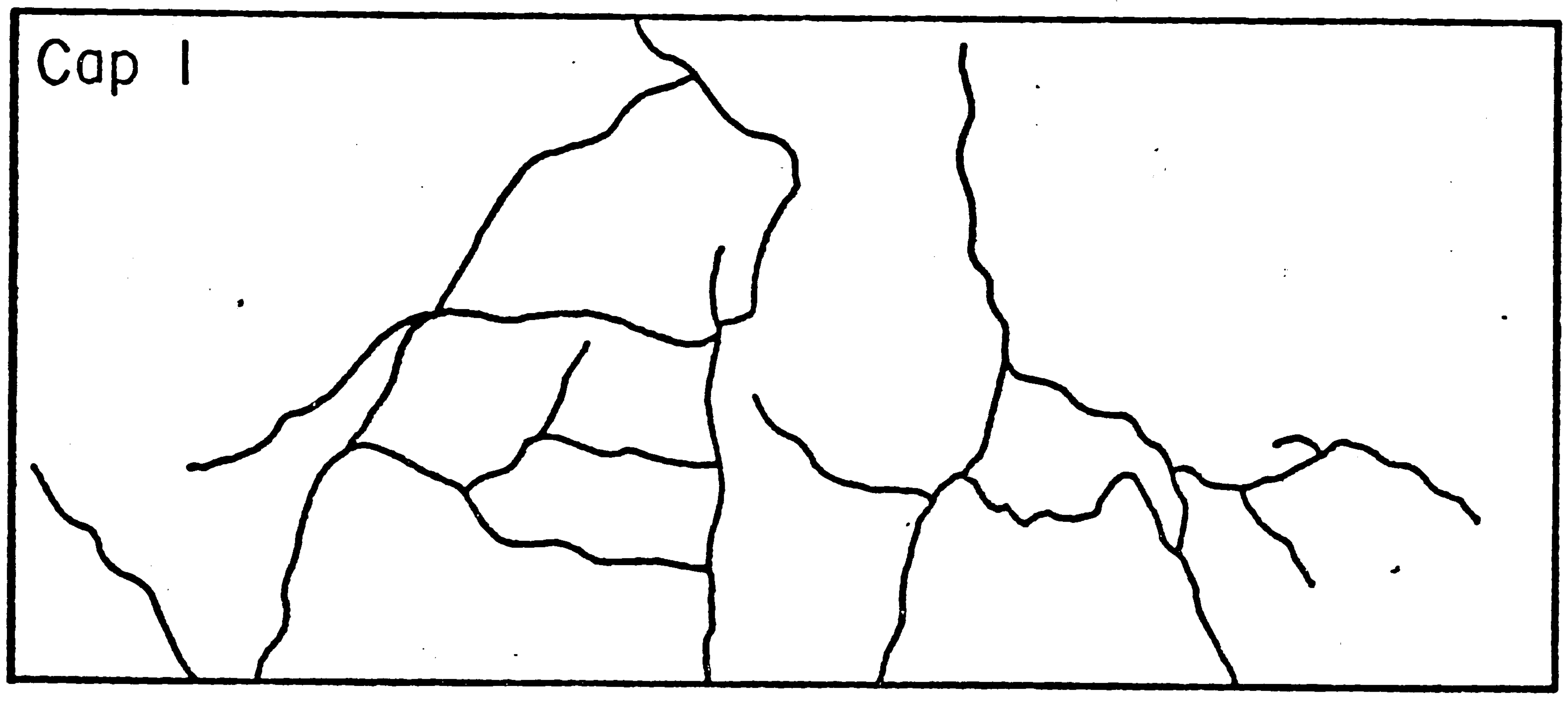


Fig. 6a SOUTH FACE OF PILE CAP 1 - CRACK PATTERNS

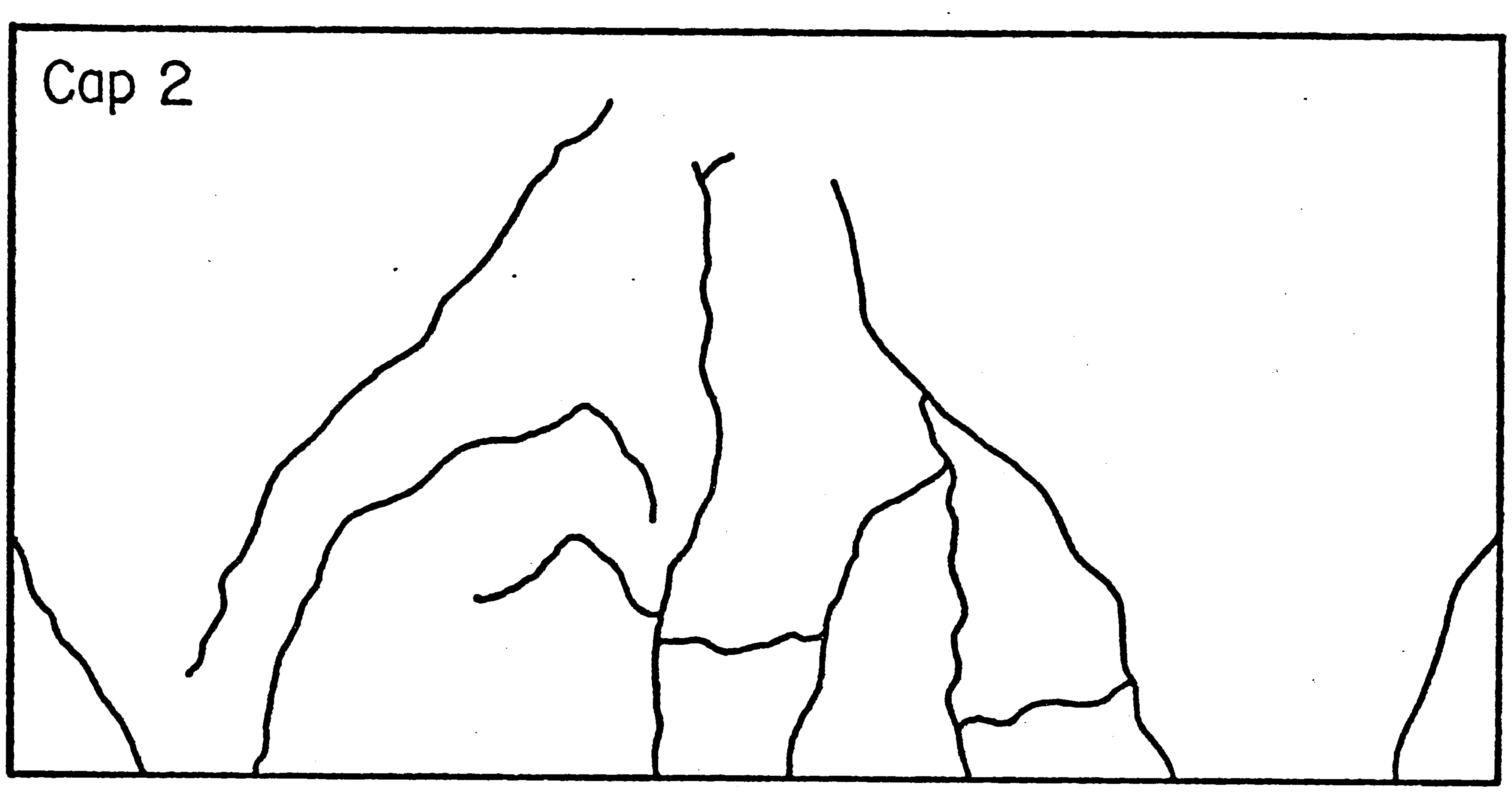


Fig. 6b SOUTH FACE OF PILE CAP 2 - CRACK PATTERNS

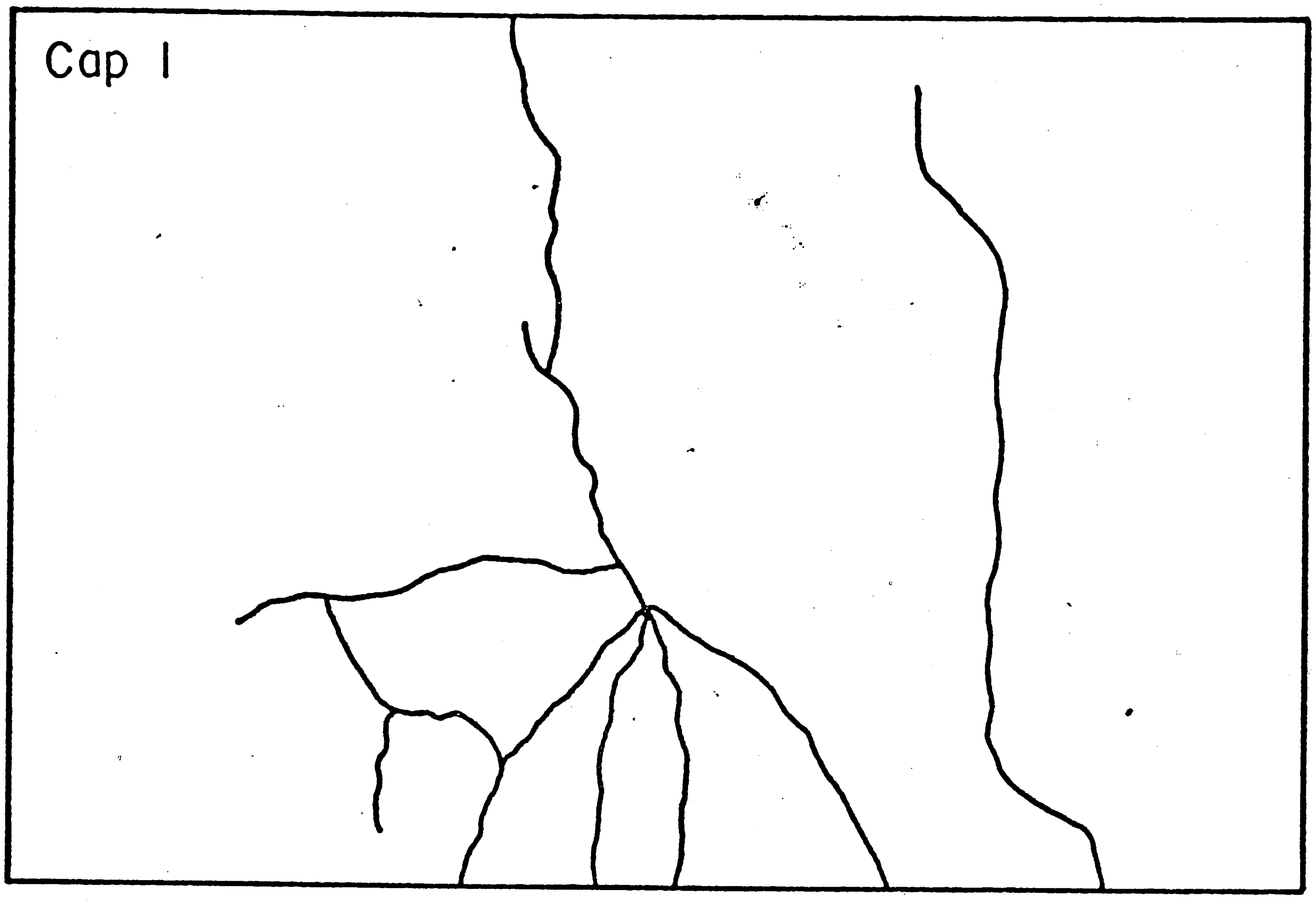


Fig. 7a WEST END OF PILE CAP 1 - CRACK PATTERNS

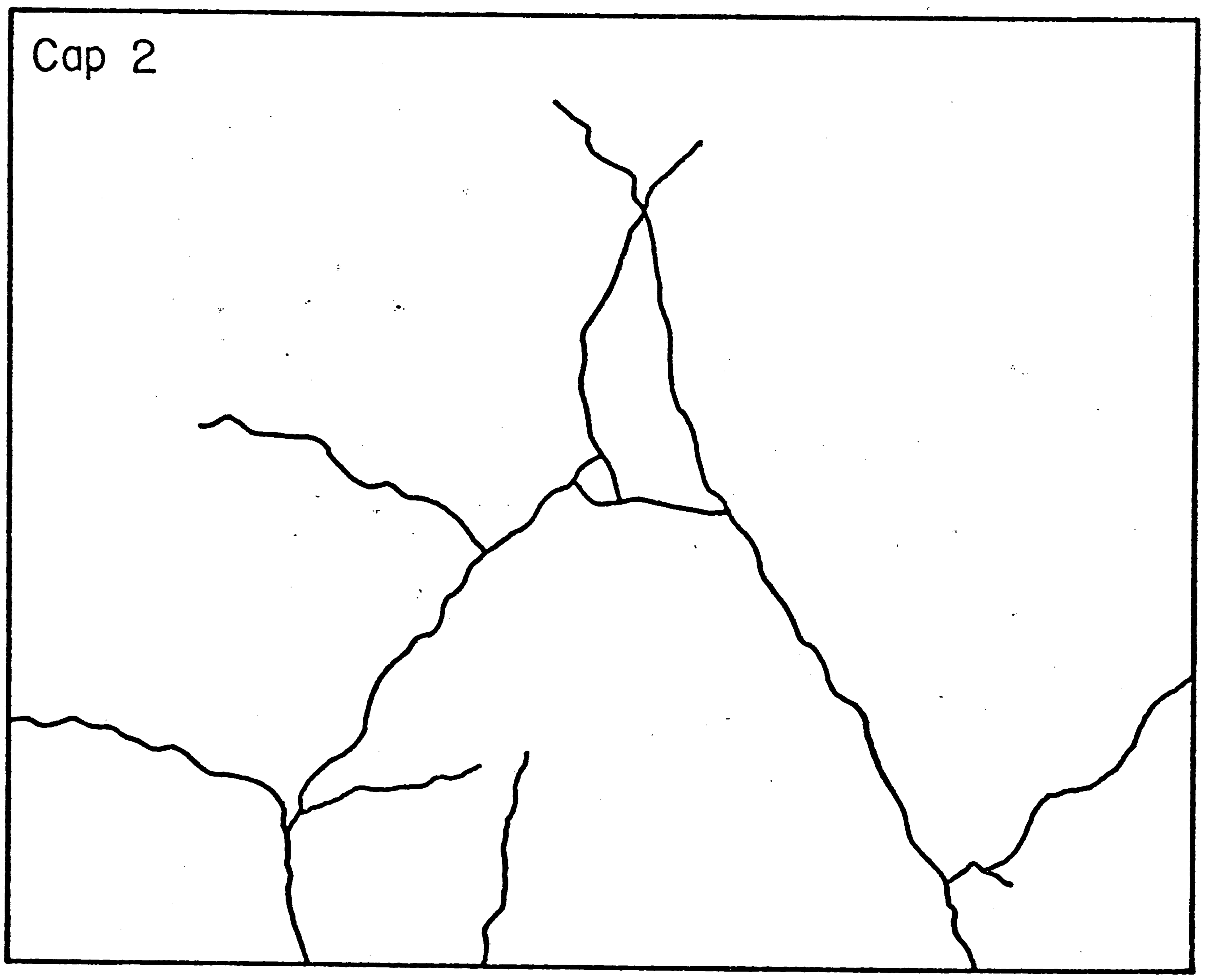


Fig. 7b WEST END OF PILE CAP 2 - CRACK PATTERNS

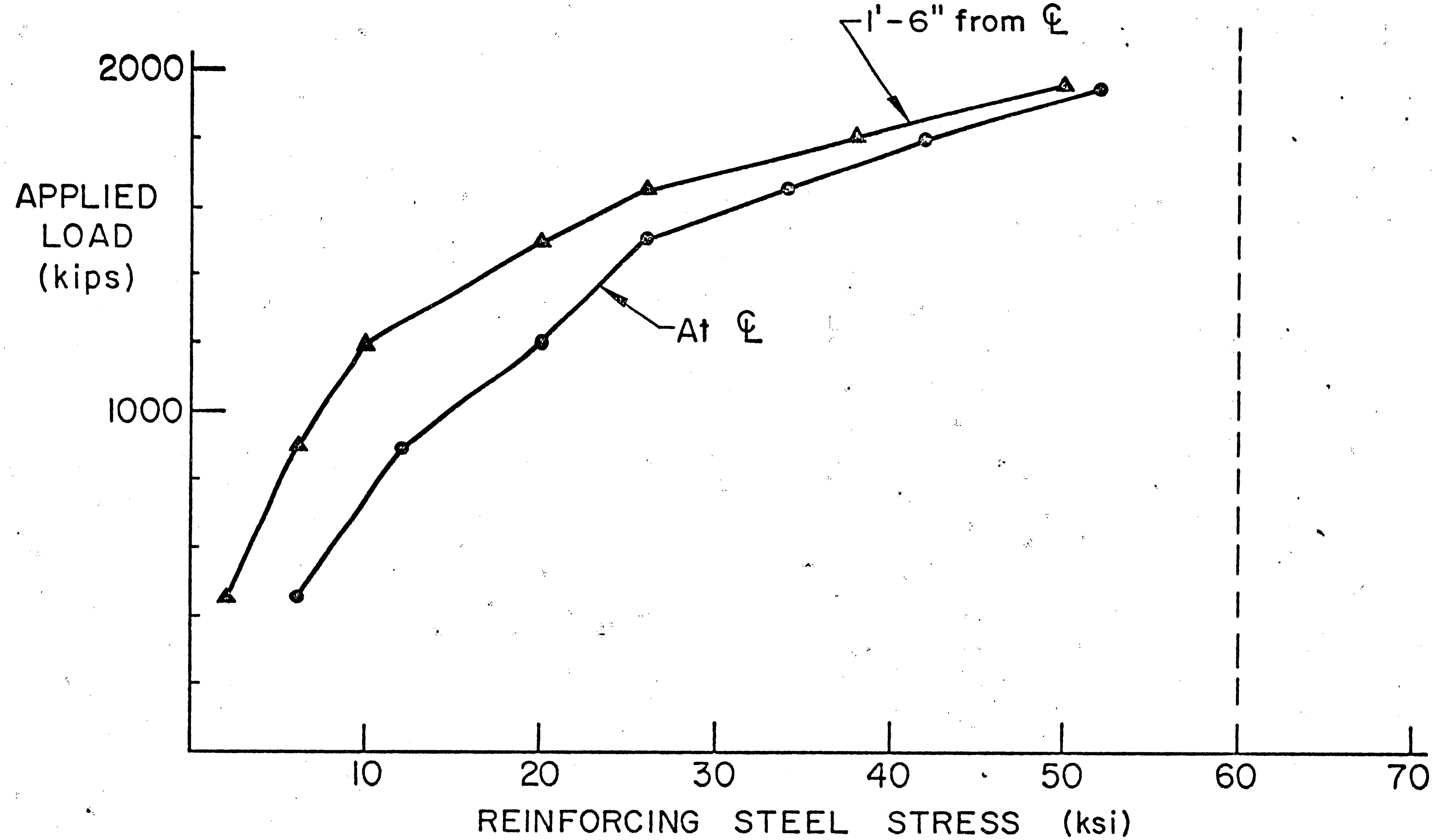


Fig. 8 STRESS IN REINFORCING STEEL - CAP 1

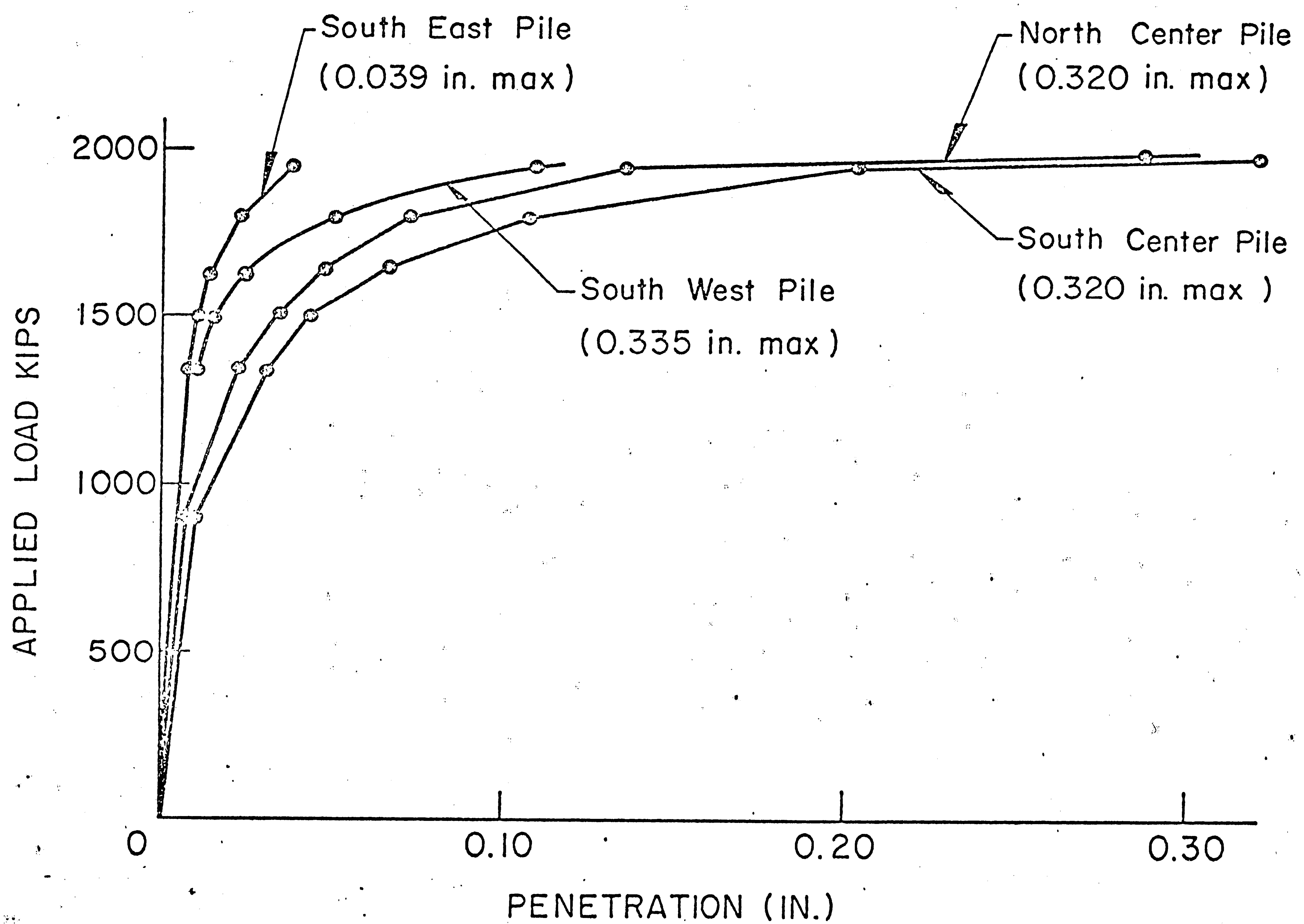


Fig. 9 PENETRATION OF PILING INTO CAP 1

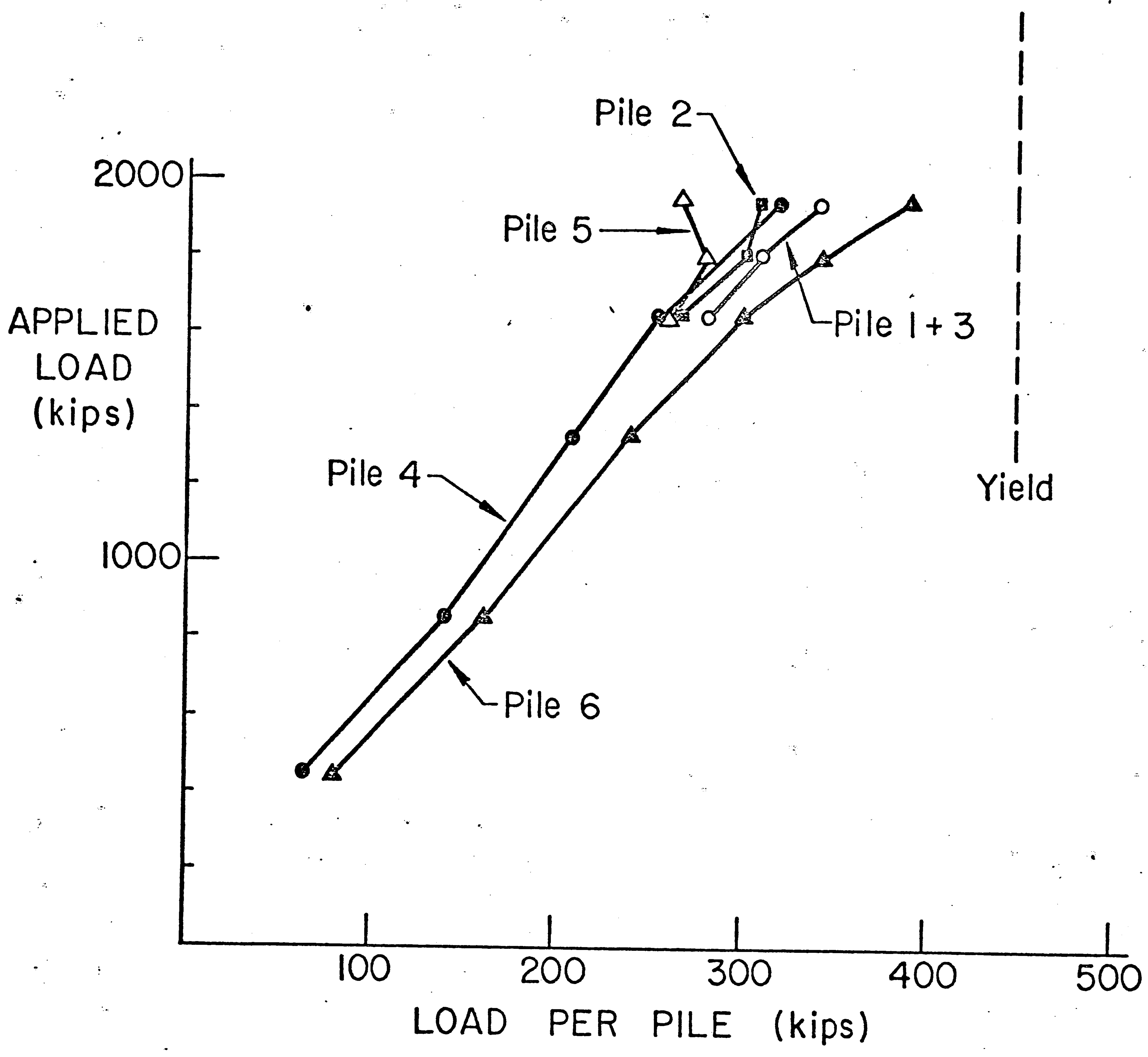


Fig. 10 DISTRIBUTION OF LOADS IN PILES - TEST 1

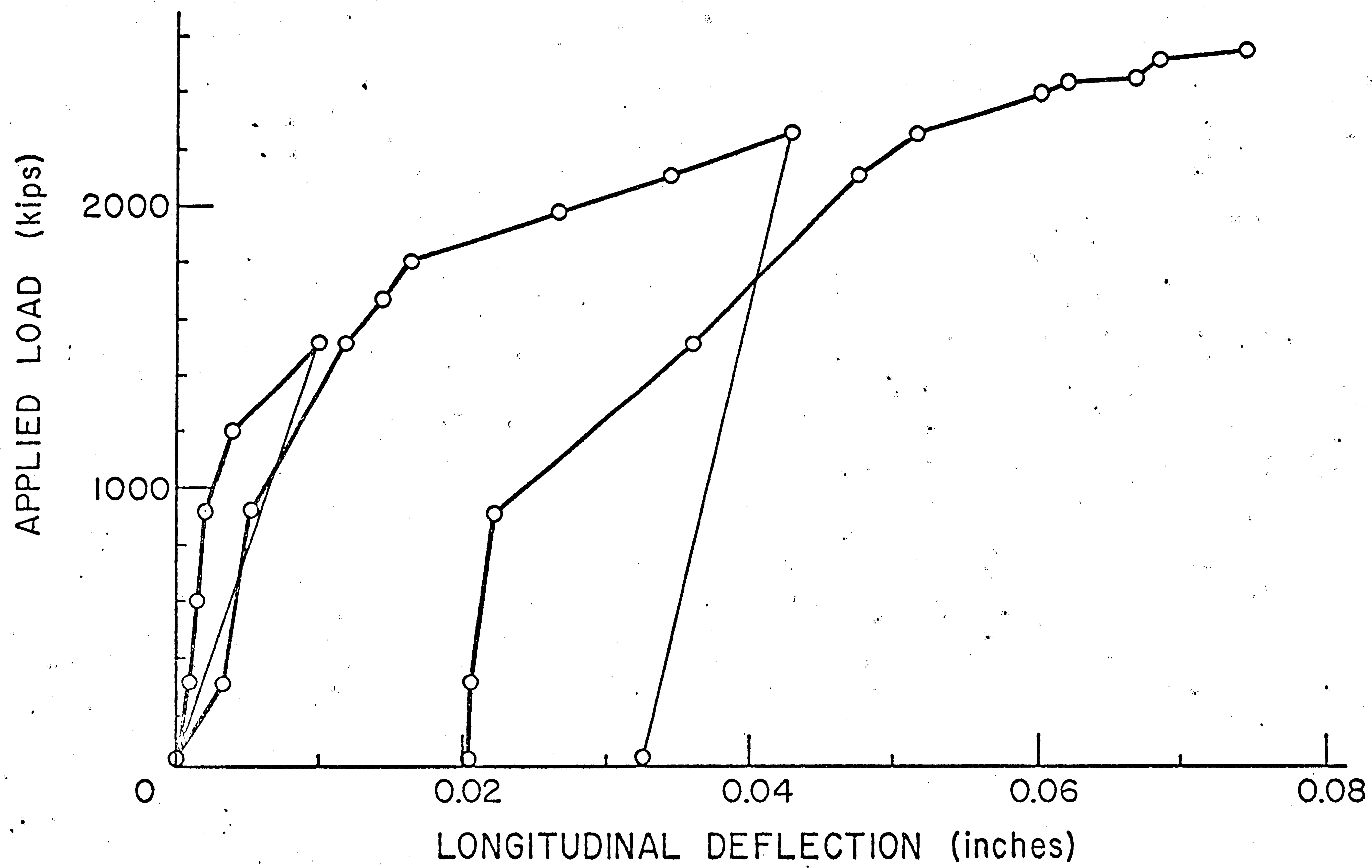


Fig. 11 LOAD DEFLECTION CURVE - CAP 2 (LONGITUDINAL DEFLECTION)

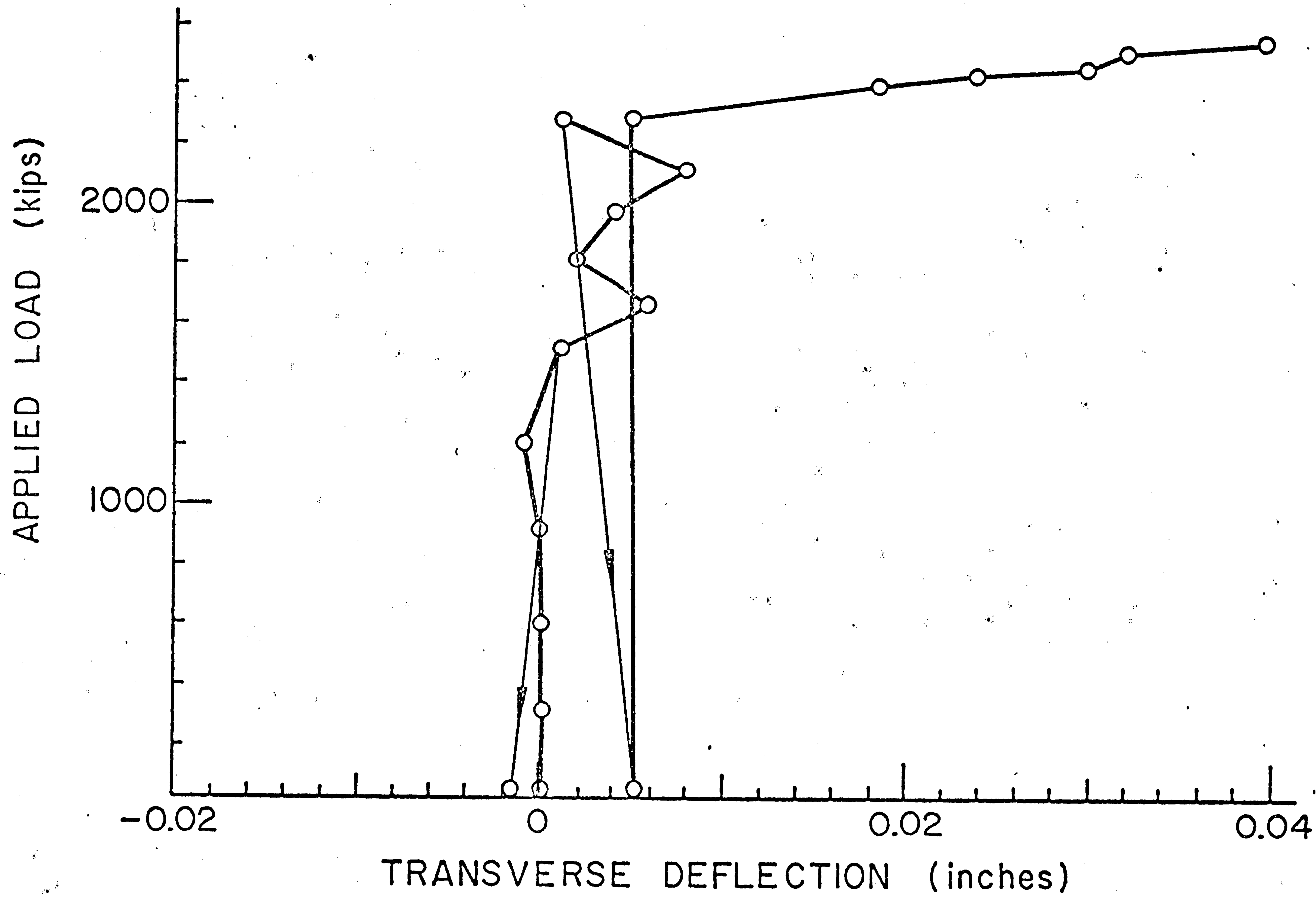


Fig. 12 LOAD DEFLECTION CURVE - CAP 2 (TRANSVERSE DEFLECTION)

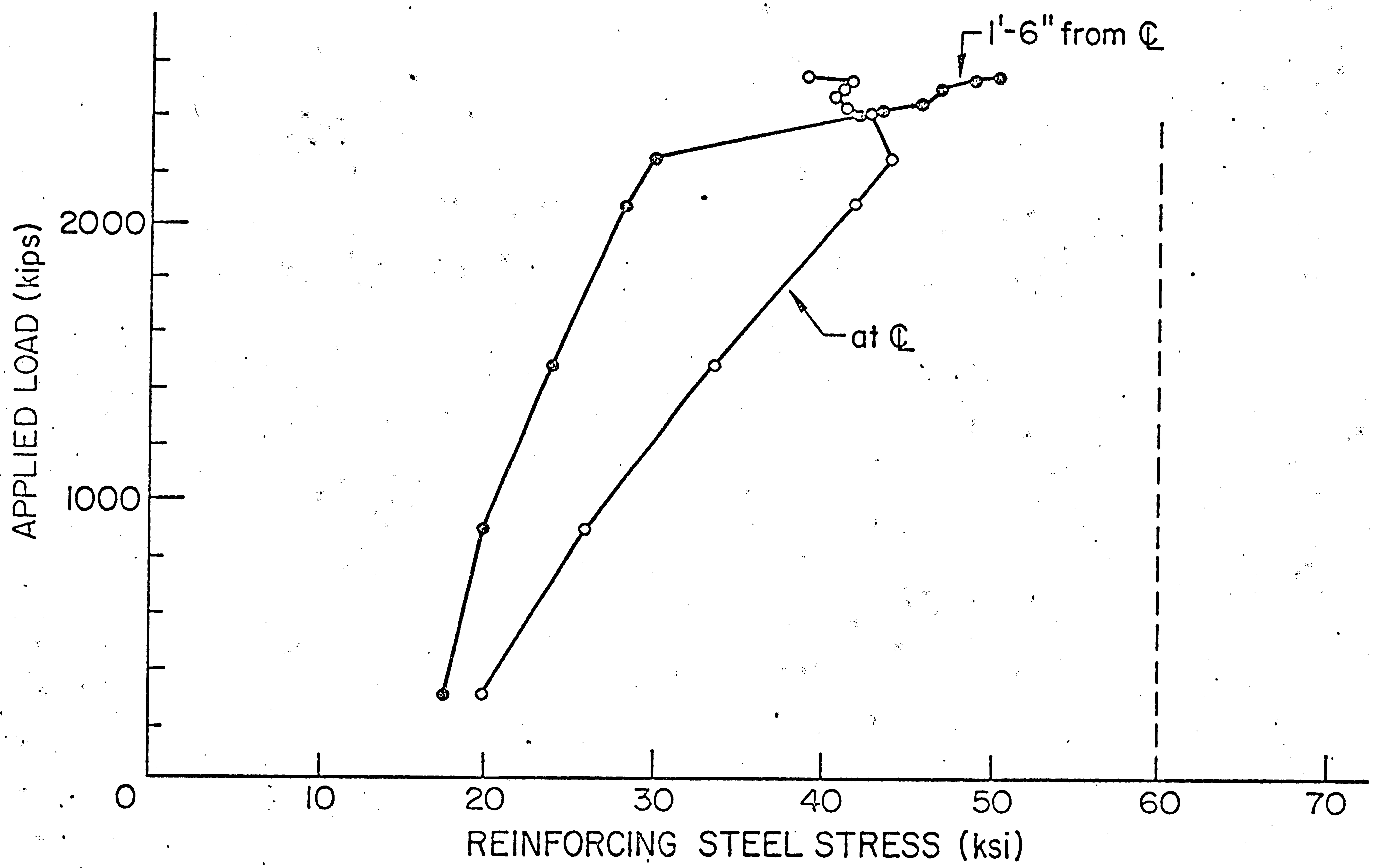
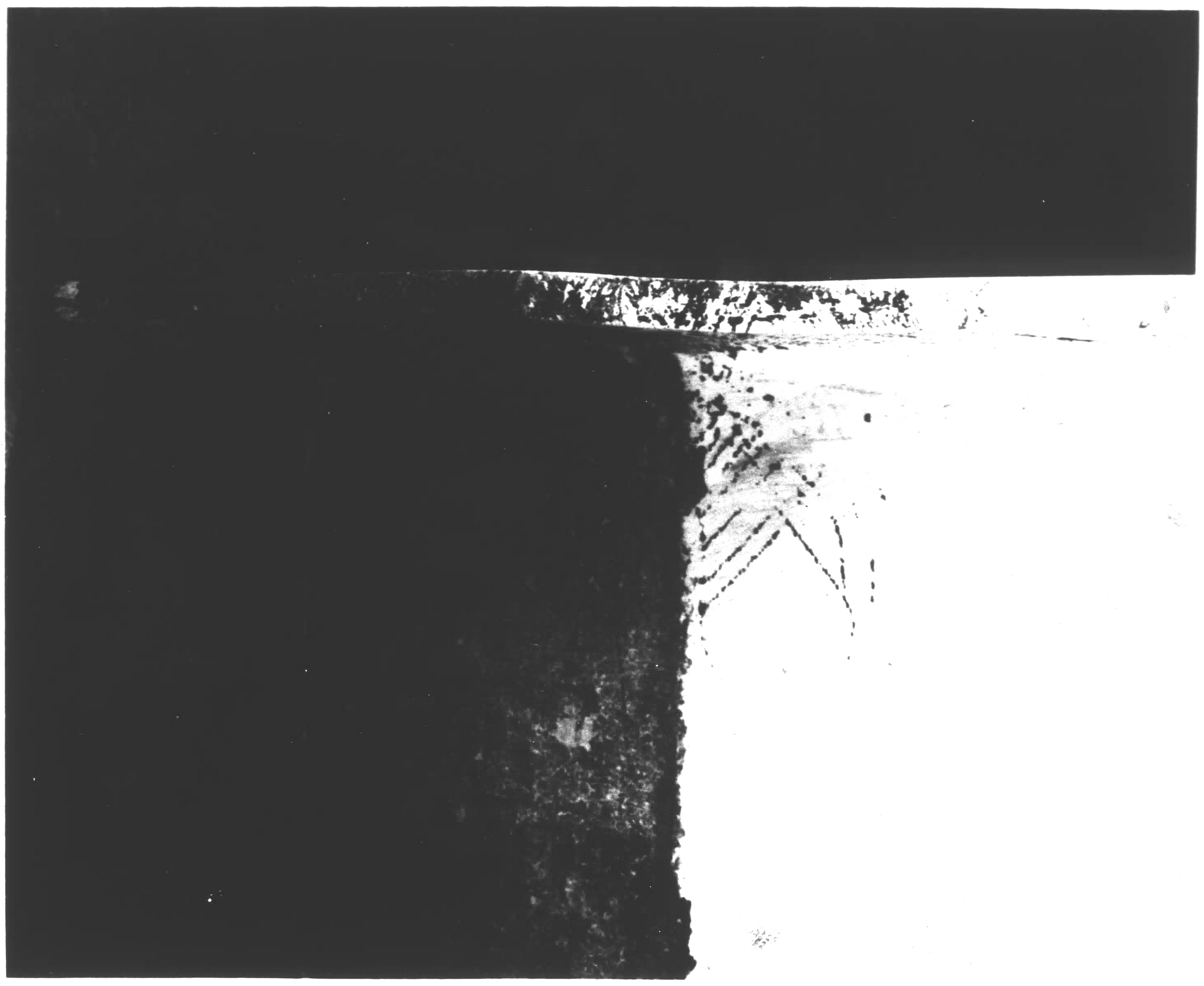
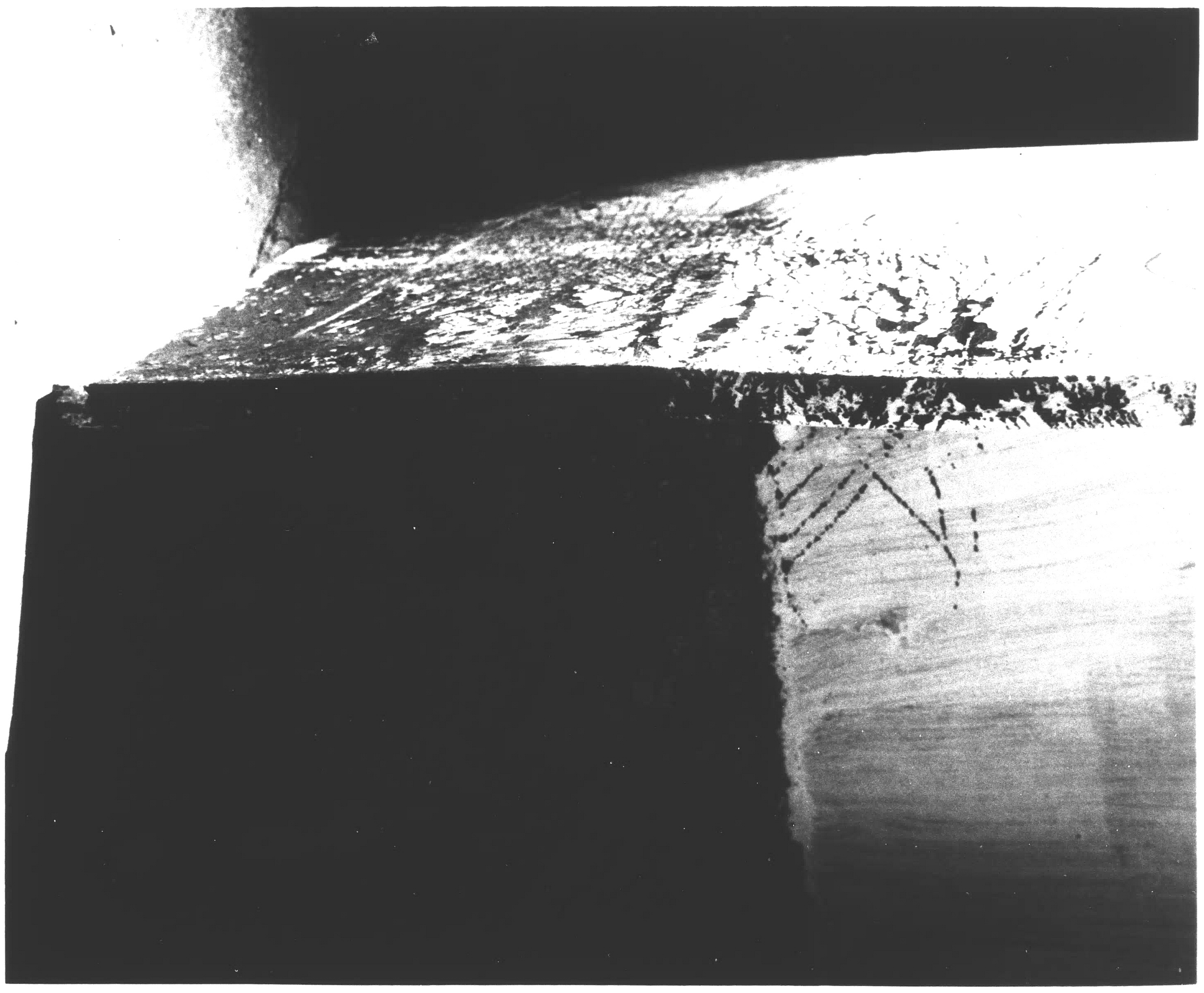


Fig. 13 STRESS IN REINFORCING STEEL - CAP 2



(a)



(b)

Fig. 14 BUCKLING IN PILE C-2

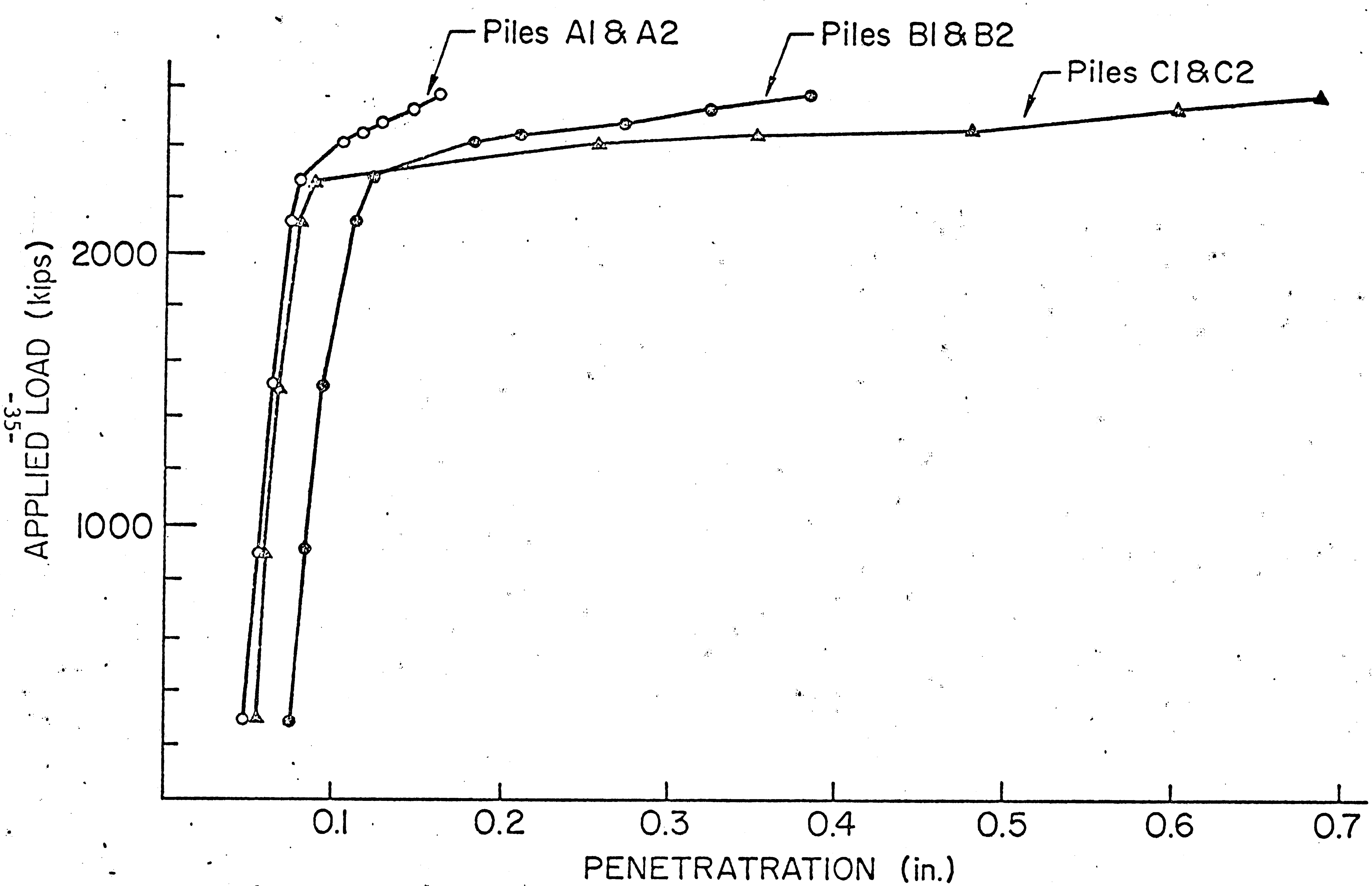


Fig. 15 PENETRATION OF PILING INTO CAP 2

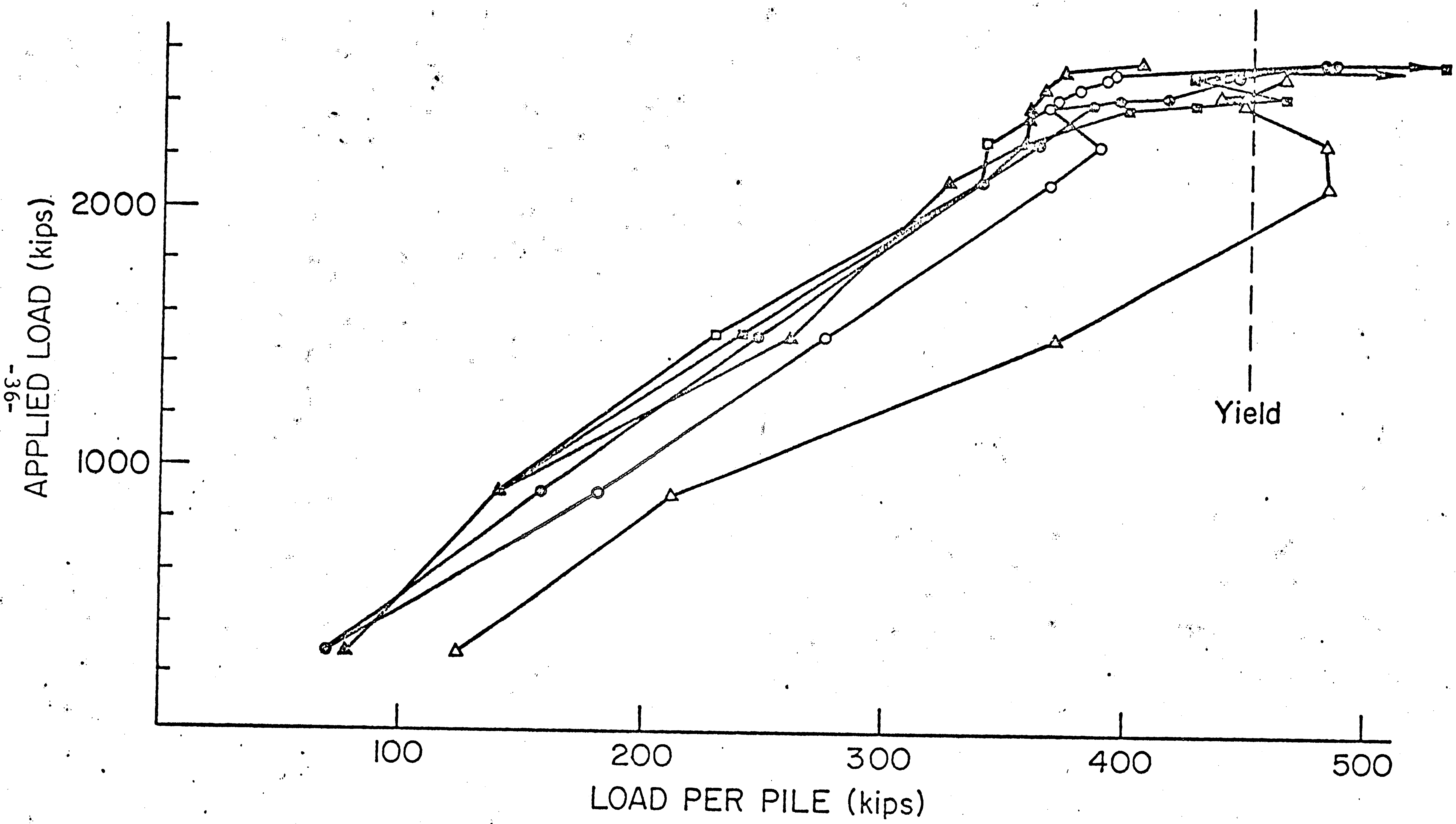


Fig. 16 DISTRIBUTION OF LOADS IN PILES - TEST 2

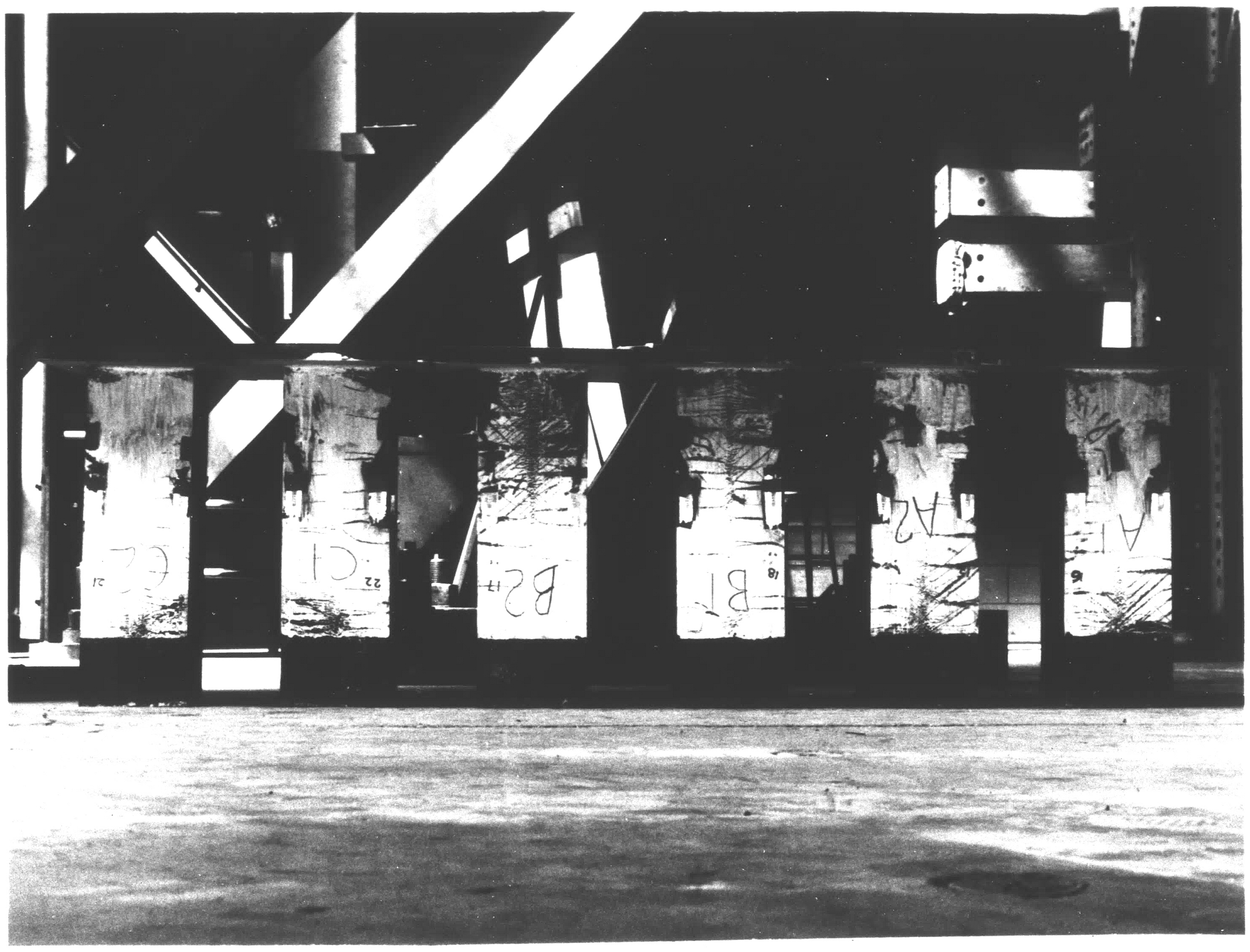


Fig. 17 PILES AFTER TESTING - TEST 2

Fig. 18 CONCRETE AT EXTERIOR CORNER AFTER REMOVAL OF PILE - TEST 2

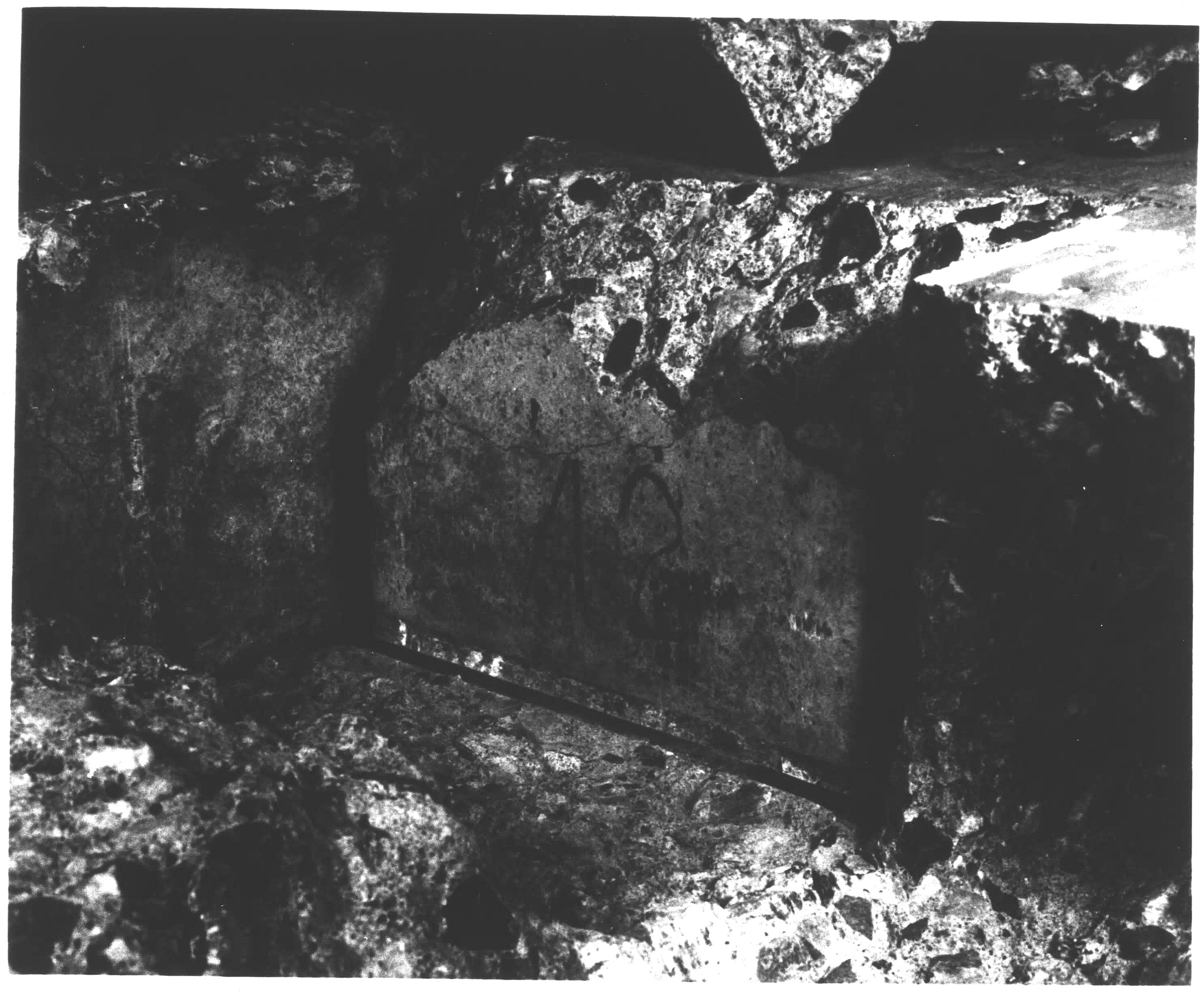


Fig. 19 CONCRETE AT CENTER AFTER REMOVAL OF PILE - TEST 2

REFERENCES

1. American Concrete Institute
   BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE,
   ACI Standard, Detroit, February 1971

2. American Concrete Institute
   BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE,
   ACI Standard, Detroit, June 1963

3. United States Steel Corporation
   PILE CAP DESIGN FOR H-PILE FOUNDATIONS,
   Aduss 91081-40, Pittsburgh, July 1968

4. Seek, Walter G.
   PILE CAP TEST #1,
   Fritz Engineering Laboratory Report No. 200.71.476.1
   Lehigh University, June 1971

5. Oyeledun, Abayomi O. and Slutter, Roger G.
   PILE CAP TEST #2,
   Fritz Engineering Laboratory Report No. 200.72.476.1,
   Lehigh University, June 1972

6. Anstin, W. J., Egger, W., Untraner, R. E. and Winemiller, J. R.
   AN INVESTIGATION OF THE BEHAVIOR OF DEEP BEAM MEMBERS OF
   REINFORCED CONCRETE AND STEEL,
   Structural Research Series #187, Civil Engineering Department,
   University of Illinois, Urbana

7. dePaiva, H. A. R. and Siess, C. P.
   STRENGTH AND BEHAVIOR OF DEEP BEAMS IN SHEAR,
   Proceedings, ASCE, Vol. 91, ST5, Part 1, October 1965

8. Marshmann, John C.
   THE INFLUENCE OF PLASTIC STRAIN RATE ON THE YIELD STRENGTH
   OF MILD STEEL,
   Unpublished Master's Thesis, Lehigh University, June 1956

9. Huber, A. W. and Beedle, L. S.
   RESIDUAL STRESS AND THE COMPRESSIVE STRENGTH OF STEEL,
   Welding Journal, Vol. 33, December 1954

10. State of Ohio, Department of Highways
    INVESTIGATION OF THE STRENGTH OF THE CONNECTION BETWEEN A
    CONCRETE CAP AND THE EMBEDDED END OF A STEEL H-PILE,
    Research Report #1, December 1947

APPENDIX A

PILE CAP ELEVATION

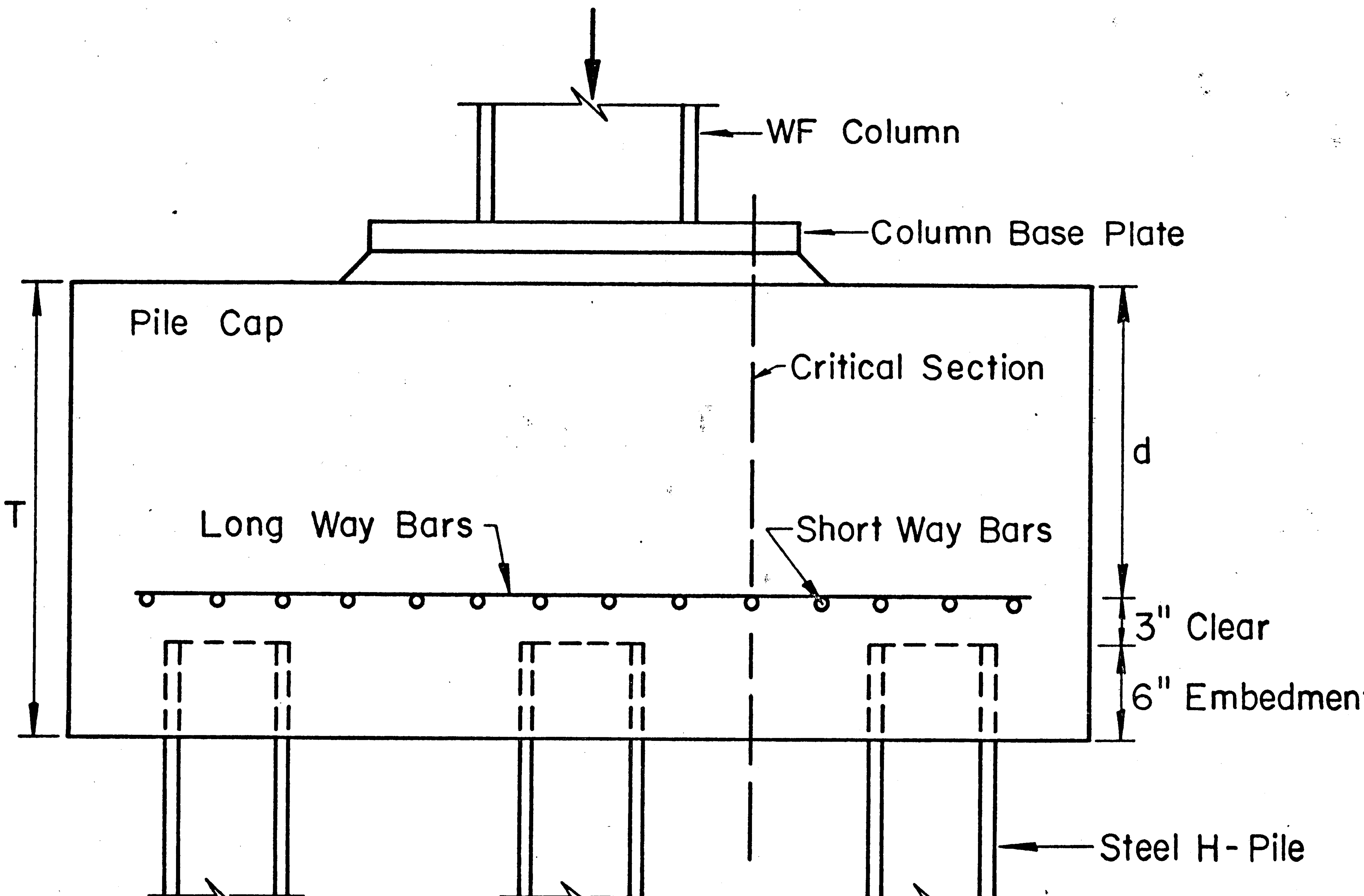


Fig. A1 Pile Cap Detail Showing Design Parameters and Critical Section

APPENDIX A

PILE CAP ELEVATION

Reinforcing Steel

"Long Way" parameters apply to bars parallel to long axis of pile cap.

"Short Way" parameters apply to bars parallel to short axis of pile cap.

Critical Section

Located midway between the column face and the edge of the base plate.

APPENDIX B

SECTION PROPERTIES - TEST 1

Pile Cap Plan Dimensions:

length (L) = 8'-0"

width (b) = 5'-0"

thickness (T) = 3'-0"

Base Plate Dimensions:

long way = 29"

short way = 28"

thickness = 2-3/4"

Reinforcing Steel:

long way; 12 #10 bars A_b = 15.24 sq.-in.

short way; 18 #6 bars A_b = 7.92 sq.-in.

Loading Column:

W14x287

b_f = 16.13

Steel H-piles:

Area of pile = 12.4 sq.-in.

Number of piles = 6

APPENDIX B

SECTION PROPERTIES - TEST 2

Pile Cap Dimensions:

length (L)	=	8'-0"
width (b)	=	5'-0"
thickness (T)	=	3'-10"

Base Plate Dimensions:

long way	=	39"
short way	=	35"
thickness	=	4-1/8"

Reinforcing Steel:

long way;	19 #9 bars	A_b = 19.00 sq.-in.
short way;	19 #6 bars	A_b = 8.36 sq.-in.

Loading Column:

W14x314

$b_f = 16.24$

Steel H-piles:

Area of pile = 12.4 sq.-in.

Number of piles = 6

APPENDIX C

COMPARISON OF ULTIMATE SHEAR LOAD BY ACI BUILDING CODE[1]

FORMULA AND TEST LOAD RESULT

Section 11.9 in the Code is titled "Special Provision for Deep Beams".

The concrete caps tested were considered as deep beam members. The nominal shear stress is given by the following equation:

$$V_c = (3.5 - 2.5 \frac{M_u}{V_u d}) \times (1.9 \sqrt{f_c'} + 2500 \rho_w \frac{V_u d}{M_u}) \qquad (11\text{-}22)$$

The following restrictions are imposed:

(i) $(3.5 - 2.5 \frac{M_u}{V_u d}) < 2.5$

(ii) $V_c < 6 \sqrt{f_c'}$

Test 1

Column Load = 900 kips

Load/pile = 900/6 = 150 kips

V = (150)(2) = 300 kips

X = 0.50a = 18 in.

M = (300)(18) = 5400 kip-in.

d = 30 in.

f_c' = 4150 psi

Test 1 (continued)

$$\rho_w = A_s/b_w\, d = \frac{(12)(1.27)}{(60)(30)} = 0.0085$$

Test 2

Column Load = 1500 kips

Load/pile = 250 kips

$V = (250)(2) = 500$ kips

$X = 0.50a = 18$ in.

$M = (500)(18) = 9000$ kip-in.

$d = 37$ in.

$f_c' = 4430$ psi

$$\rho_w = A_s/b_w\, d = \frac{(19)(1.0)}{(60)(37)} = 0.0086$$

COMPARISON OF SHEAR LOAD

Test 1 - Critical Section for Shear from ACI Code Section 11.9.3

$$V_c = (3.5 - 2.5\frac{M}{Vd}) \times (1.9\sqrt{f_c'} + 2500\,\rho_w \frac{Vd}{M})$$

$$= (3.5 - (2.5)(0.600)) \times (122 + (2500)(0.0085)(1.67))$$

$$= 315 \text{ psi} < 6\sqrt{f_c'} \quad \text{O.K}$$

$$V_u = V_c\, b_w d$$

$$= (.315)(60)(30)$$

$$= 567 \text{ kips}$$

$$P_{cal} = (567)(3) = 1701 \text{ kips}$$

cf Test Load 1986 kips

$$\frac{P_{test}}{P_{cal}} = 1.17$$

<u>Test 2</u> - Critical Section for Shear from ACI Code Section 11.9.3

$$V_c = (3.5 - 2.5\frac{M}{Vd}) \times (1.9\sqrt{f_c'} + 2500\,\rho_w \frac{Vd}{M})$$

$$= (3.5 - (2.5)(0.487)) \times (126 + (2500)(0.0086)(2.053))$$

$$= 387 \text{ psi} < 6\sqrt{f_c'} \quad 0 \cdot K$$

$$V_u = V_c\, b_w d$$

$$= (.387)(60)(37)$$

$$= 859 \text{ kips}$$

$$P_{cal} = (859)(3) = 2577 \text{ kips}$$

cf Test Load 2570 kips

$$\frac{P_{test}}{P_{cal}} = 1.00$$

APPENDIX C

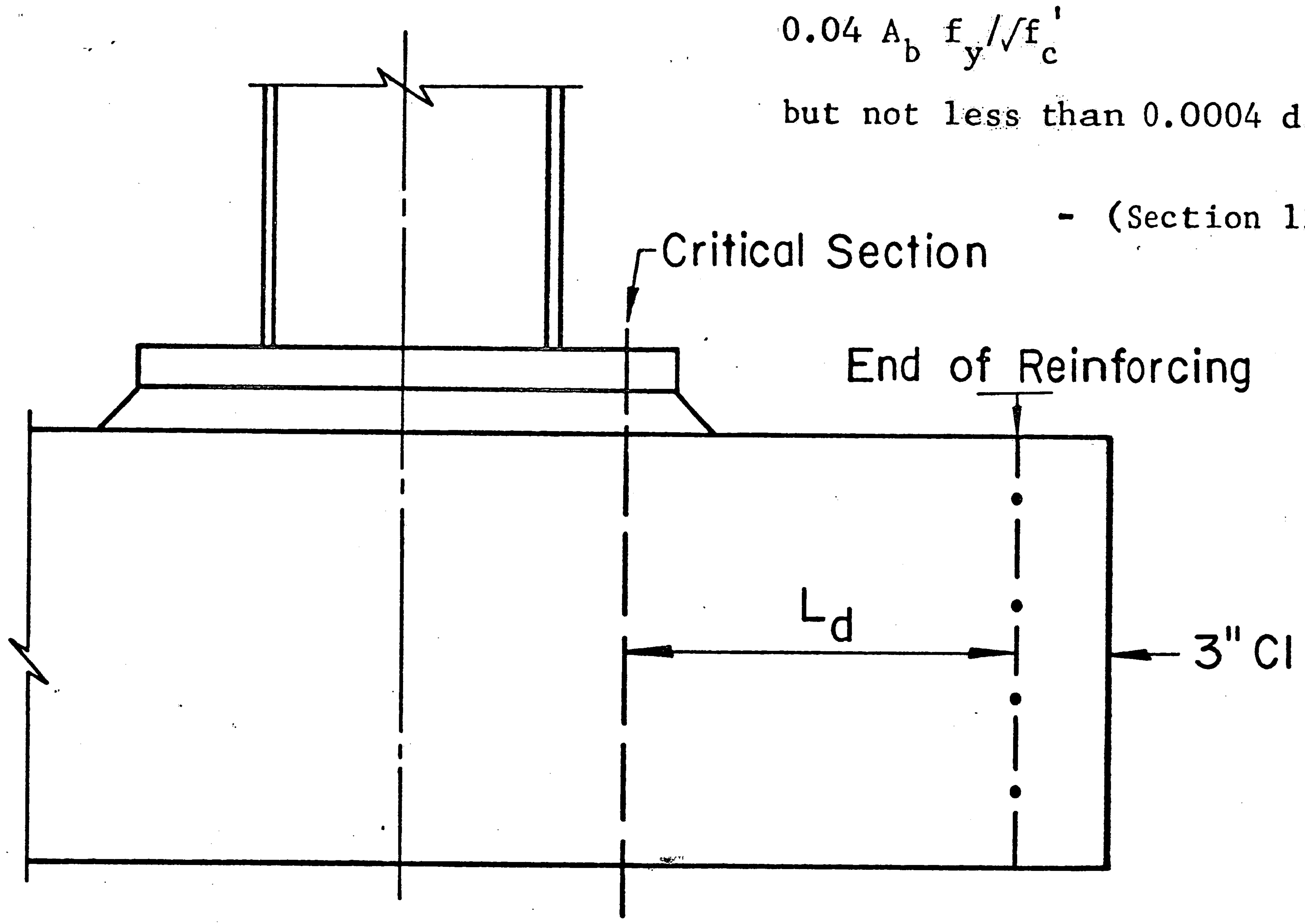
DEVELOPMENT LENGTH OF DEFORMED BARS IN TENSION

The Code specifies that the development length ℓ_d in inches for #11 or smaller bars shall be:

$$0.04\ A_b\ f_y/\sqrt{f_c'}$$

but not less than $0.0004\ d_b\ f_y$

\- (Section 12.5)



Test 1 - Long Way Reinforcement

12 #10 bars

$A_b = 1.27 \text{ in.}^2 \qquad d_b = 1.27 \text{ in.}$

$f_y = 60{,}000$ psi

$f_c' = 4150$ psi

$$\ell = 0.04\, A_b\, f_y / \sqrt{f_c'}$$

$$= (0.04)(1.27)(60{,}000) / \sqrt{4150}$$

$$= 47.31 \text{ in.}$$

$$\ell > 0.0004\, d_b\, f_y$$

$$> (0.0004)(1.27)(60{,}000)$$

$$> 30.48 \text{ in.}$$

However, $\ell_d = 33.71 \text{ in.} > 30.48'' \quad \therefore 0 \cdot K$

Test 2 - Long Way Reinforcement

19 #9 bars

$A_b = 1.00 \text{ in.}^2 \qquad d_b = 1.128 \text{ in.}$

$f_y = 60{,}000$ psi

$f_c' = 4430$ psi

$$\ell = 0.04\, A_b\, f_y / \sqrt{f_c'}$$

$$= (0.04)(1.00)(60{,}000) / \sqrt{4430}$$

$$= 36.06 \text{ in.}$$

$$\ell > 0.0004\, d_b\, f_y$$

$$> (0.0004)(1.128)(60{,}000)$$

$$> 27.07 \text{ in.}$$

However, $\ell_d = 31.19 \text{ in.} \quad > 27.07 \text{ in.} \quad \therefore 0 \cdot K$

VITA

The author received a Bachelor of Science degree in Civil Engineering from Lafayette College, Easton, Pennsylvania in 1971. He joined the staff of Fritz Engineering Laboratory in September 1971 and will be receiving the degree of Master of Science in Civil Engineering in May 1973.