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FULL SCALE CYCLIC LATERAL LOAD TESTS ON SIX SINGLE PILES IN SAND

by

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This study was performed to determine responses of single piles in sands subjected to cyclic lateral loading and compare the results with predicted responses based on in-situ pressuremeter tests. The pressuremeter method used for predicting pile response to monotonic lateral loading is applicable to piles which will experience little or no degradation in flexural stiffness such as steel piles and prestressed concrete piles loaded less than the prestress.

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PREFACE

This study was performed by the Geotechnical Division, Civil Engineering Department, Texas A&M University, College Station, TX, under contract to the US Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, for the US Army Engineer District, St. Louis. The study was performed under Contract No. DACW39-87-M-0446.

This report was prepared by Mr. Robert L. Little and Dr. Jean-Louis Briaud, Texas A&M University, and reviewed by Mr. G. Britt Mitchell, Chief, Engineering Group, Soil Mechanics Division (SMD), Geotechnical Laboratory (GL). WES. General supervision was provided by Mr. Clifford L. McAnear, Chief, SMD, and Dr. William F. Marcuson III, Chief, GL.

COL Dwayne G. Lee, EN, is Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

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1. INTRODUCTION

1.1 Project Purpose

Six existing piles were readily available for lateral load testing. The purpose of this project was to subject those six piles to cyclic horizontal loads and study the corresponding accumulation of horizontal displacement. These load tests also provided a unique opportunity to study the potential of the pressuremeter for predicting the response of piles in sand subjected to cyclic horizontal loads.

Pressuremeter tests offer an array of advantages over present day methods employed in the design of laterally-loaded piles. The pressuremeter method allows site specific P-y curves developed from point-by-point in-situ measurement to be obtained, rather than curves derived from one or two measured soil parameters. The pressuremeter is a versatile instrument and can be employed in virtually any soil type, including those for which there are no existing recommendations for the derivation of conventional P-y curves. The pressuremeter allows the pile installation method to be modelled directly: pre-bored pressuremeter tests for drilled shafts and driven pressuremeter tests for driven piles. The pressuremeter is also capable of simulating the expected pile loading conditions: sustained pressure increment tests, unload-reload cyclic tests and rapid inflation tests yield site-specific soil responses to creep loading, cyclic loading and dynamic loading respectively.

These advantages over existing methods prompted this project. The chief objective was to incorporate cyclic loading effects into the derivation of P-y curves obtained from pressuremeter tests in order to predict the response of piles in sand subjected to cyclic lateral loading.

1.2 Project Approach

This project was designed to allow for a comparison of measured responses of piles in sand subjected to cyclic lateral loading with predicted responses based on in-situ pressuremeter tests. The project was divided into three phases. In the first phase, a series of pressuremeter (PMT) tests were performed at a site where six individual piles had earlier been installed and load tested vertically. In the second phase, the piles were load tested under cyclic lateral loading and the responses were recorded. In the final phase, predictions of the pile response were prepared based on the PMT tests and the predictions were compared to the measured results.

2. THE SITE AND THE SOIL

2.1 Test Site Location

The pile load test site was located on property under the authority of the Texas State Department of Highways and Public Transportation at the northern end of the Baytown-La Porte tunnel on State Highway 146 near Houston, Texas (Figure 1). The six piles were arranged in a triangular pattern approximately 300 ft south of the tunnel maintenance building near Lagoon Number Three (Figure 2). The piles were originally installed for vertical pile capacity load testing in connection with the construction of a 100 million dollar cable-stayed bridge spanning the Houston ship channel at the same location.

2.2 Soil Conditions and Stratigraphy

A variety of soil tests had been previously performed at the site in conjunction with the vertical load testing of the piles (Briaud Engineers, 1986). The soil was primarily composed of loose to medium dense fine sand in the upper 73 ft underlain by stiff to very stiff clay (Figure 3). A boring log with Standard Penetration Test (SPT) results, grain size analysis curves and cone penetrometer test results are presented in Figures 4 through 6 to complete the documentation of the soil.
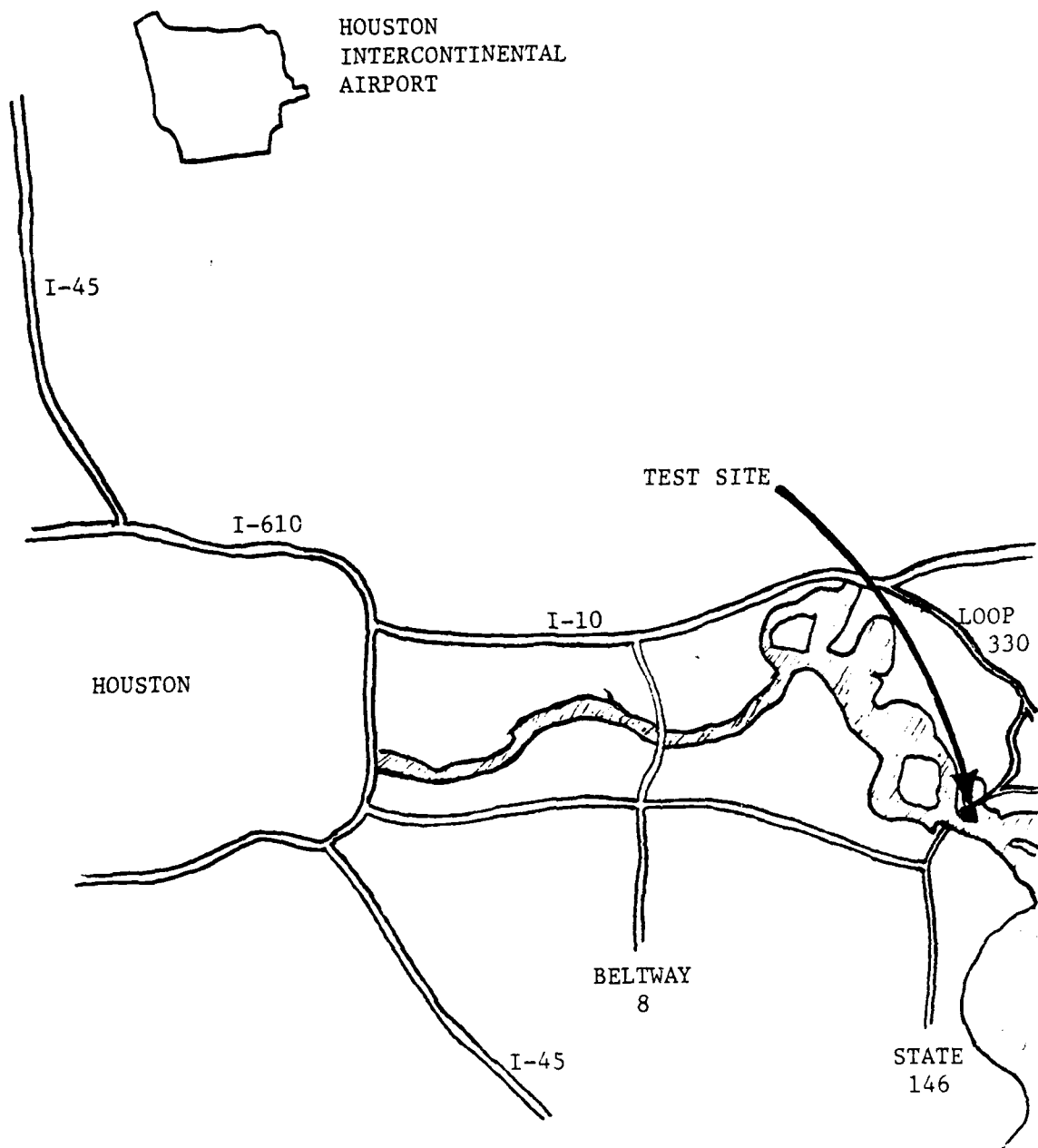
FIGURE 1. Location of Test Site

BAYTOWN
LaPORTE
TUNNEL

0   600'   1200'

SCALE

MAINTENANCE
BUILDING

LAGOON NO. 3

HWY.
146

20' 8"

20' 7"

10' 3"

10' 2"

TEST SITE

LEGEND

☐ 20" SQUARE CONCRETE

○ 24" STEEL PIPE PILE

◉ 36" DRILLED SHAFT

● 42" DRILLED SHAFT
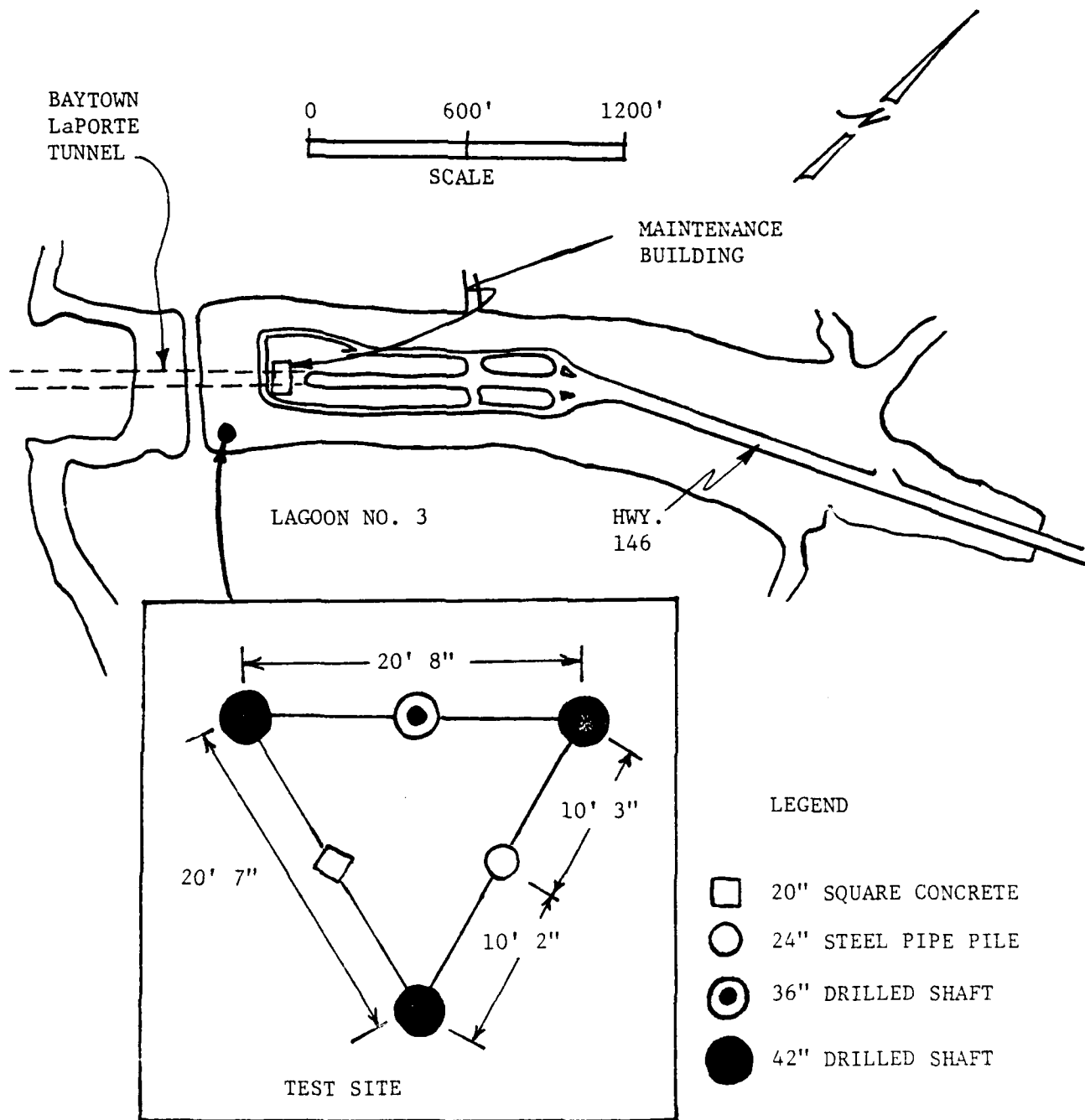
FIGURE 2.   Site Plan and Arrangement of Test Piles

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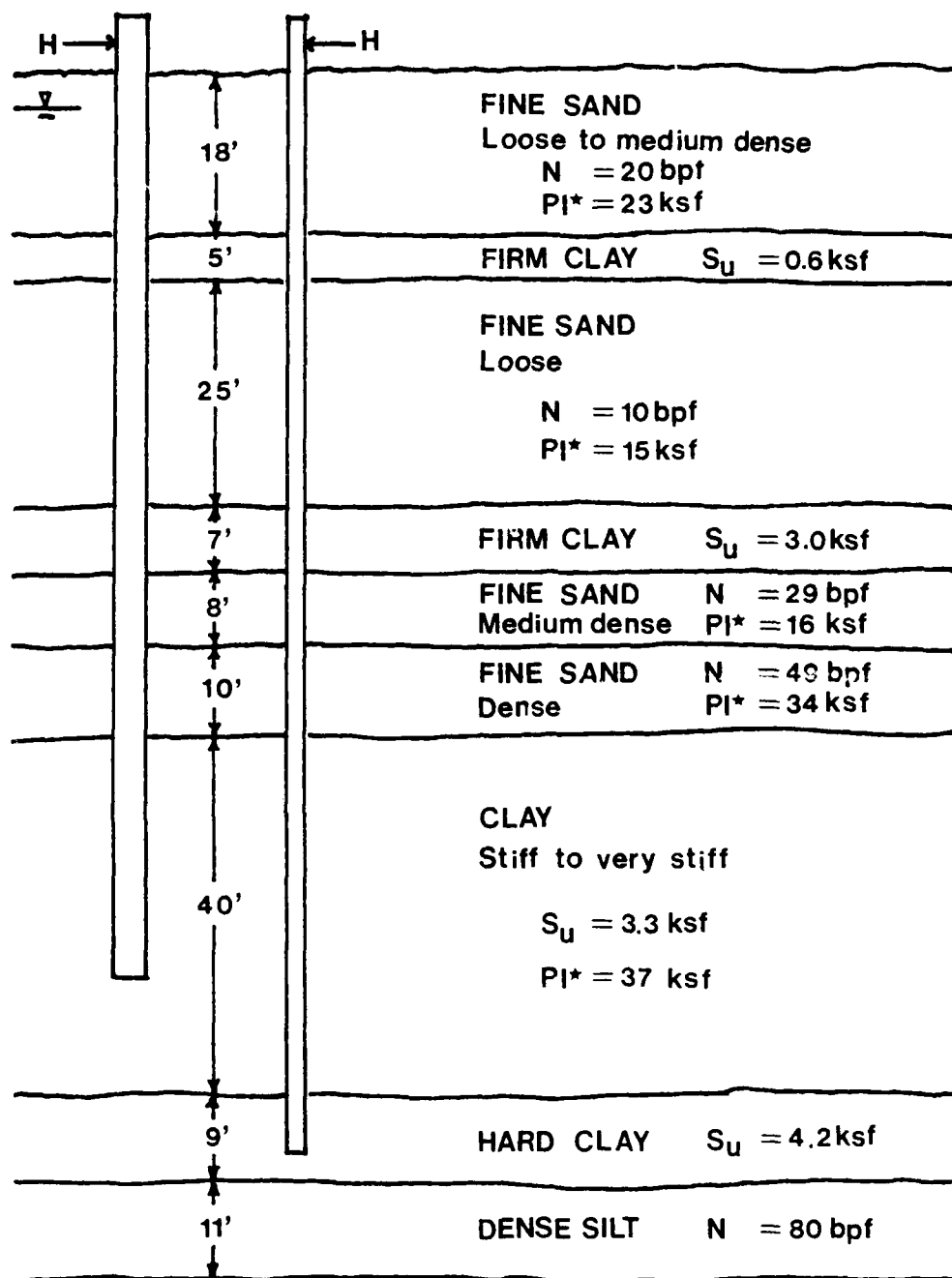
FIGURE 3. Soil Profile at Test Site

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LOG OF BORING NO. 3
NORTH LOAD TEST SITE

DEPTH FT	SYMBOL	SAMPLES	LOCATION Houston Ship Channel Load Test Site SURFACE EL 10.5±	BLOWS PER FT	% PASSING NO 200 SIEVE	UNIT DRY WT LB PER CU FT	WATER CONTENT % Plastic Limit — Natural — Liquid Limit 20 40 60	UNDRAINED SHEAR STRENGTH KIPS PER SQ FT 0.5 1.0 1.5 2.0 2.5 KILOPASCALS 25 50 75 100 125	DEPTH M
			Loose to medium dense light brown fine sand with shell fragments and clay pockets		6				
					5				
10			-silty gray with shell layers below 5'	23	20				
				15	14				
20			Firm olive gray clay and sandy clay with sand pockets			99	←•→ •	■ ▲	
			Loose gray fine sand	15	4				
30				11	7				
				6	10				
			-with clay pockets and seams below 38'	9	16				
40			-with organic matter below 43'	8	52				
50			Firm to very stiff olive gray clay and sandy clay with sand seams and pockets	8			←—•—→		
						114	•	■ ▲— 3.0	
60			Medium dense light gray fine sand with clayey sand layers and clay seams	29	18				
			Dense light brown fine sand	50	6				
70				44	10				
			Stiff reddish brown and light gray clay	13			←•—→	■	
80			-with sand pockets and calcareous nodules 78' to 83'			102	•	▲■	
			-tan and gray 83' to 98'			114	←•→	▲ ■	
90			-very stiff to hard with sand seams and silt partings below 90'			102	•	▲ ■	
						116	←•—→	▲→ 3.7	
100			-reddish brown and gray 98' to 108'			110	•	▲→ ■	

STRENGTH LEGEND
- • Unconfined Compression
- ▲ Unconsolidated-Undrained Triaxial Compression
- ◆ Miniature Vane
- (open symbols indicate remolded tests)
- ➤ Torvane
- ⊠ Hand Penetrometer

FIGURE 4a. Test Site Boring Log, 0' to 100'

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LOG OF BORING NO. 3 (Cont'd)
NORTH LOAD TEST SITE

DEPTH FT	SYMBOL	SAMPLES		BLOWS PER FT	% PASSING NO 200 SIEVE	UNIT DRY WT LB PER CU FT	WATER CONTENT %			UNDRAINED SHEAR STRENGTH KIPS PER SQ FT	DEPTH M
							Plastic Limit	Natural	Liquid Limit	0.5 1.0 1.5 2.0 2.5	
							20	40	60	KILOPASCALS 25 50 75 100 125	
			very stiff to hard reddish brown and gray clay, slickensided			106				4.2	
			-gray and tan below 108'			108				4.7	
110										4.7	
			very stiff to hard light gray silty clay			109				1.6	
120			very stiff red and light gray clay			101					
			very dense light gray silt with clay seams	50/7"	99						
130				50/5"	92						
			Firm to stiff gray clay with silt partings and organic matter	22							
140			-with sand pockets below 143'			83					
150				25							
160											
170											
180											
190											
200											

SAMPLER 1" thin-walled tube
2" split-spoon

DRILLING METHOD Wet Rotary

STRENGTH LEGEND
● Unconfined Compression
▲ Unconsolidated-Undrained Triaxial Compression
◆ Miniature Vane
(open symbols above indicate remolded tests)
→ Torvane
Σ Hand Penetrometer

FIGURE 4b. Test Site Boring Log, 100' to 150'

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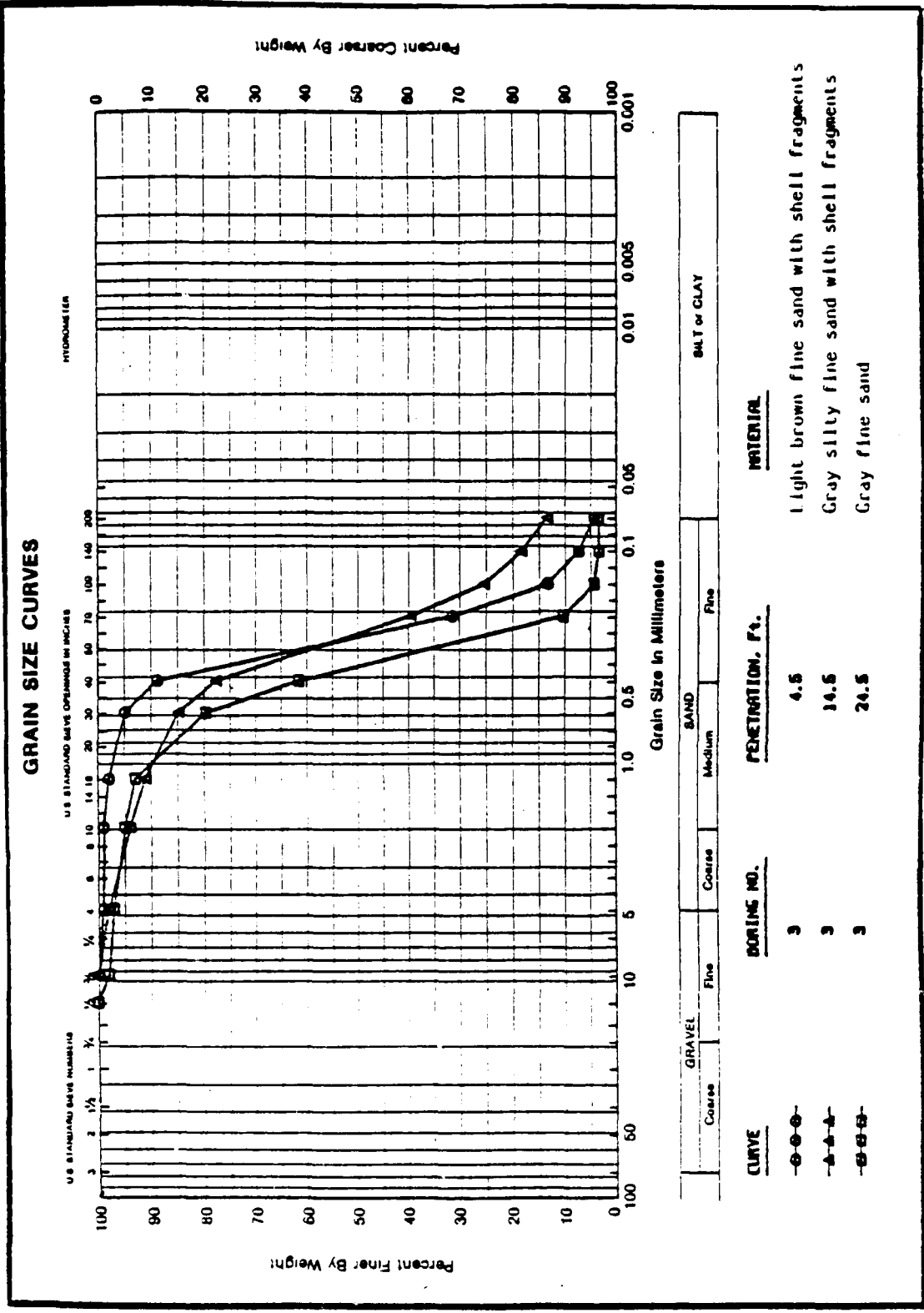
FIGURE 5a. Test Site Grain Size Curves, 0' to 24.5'
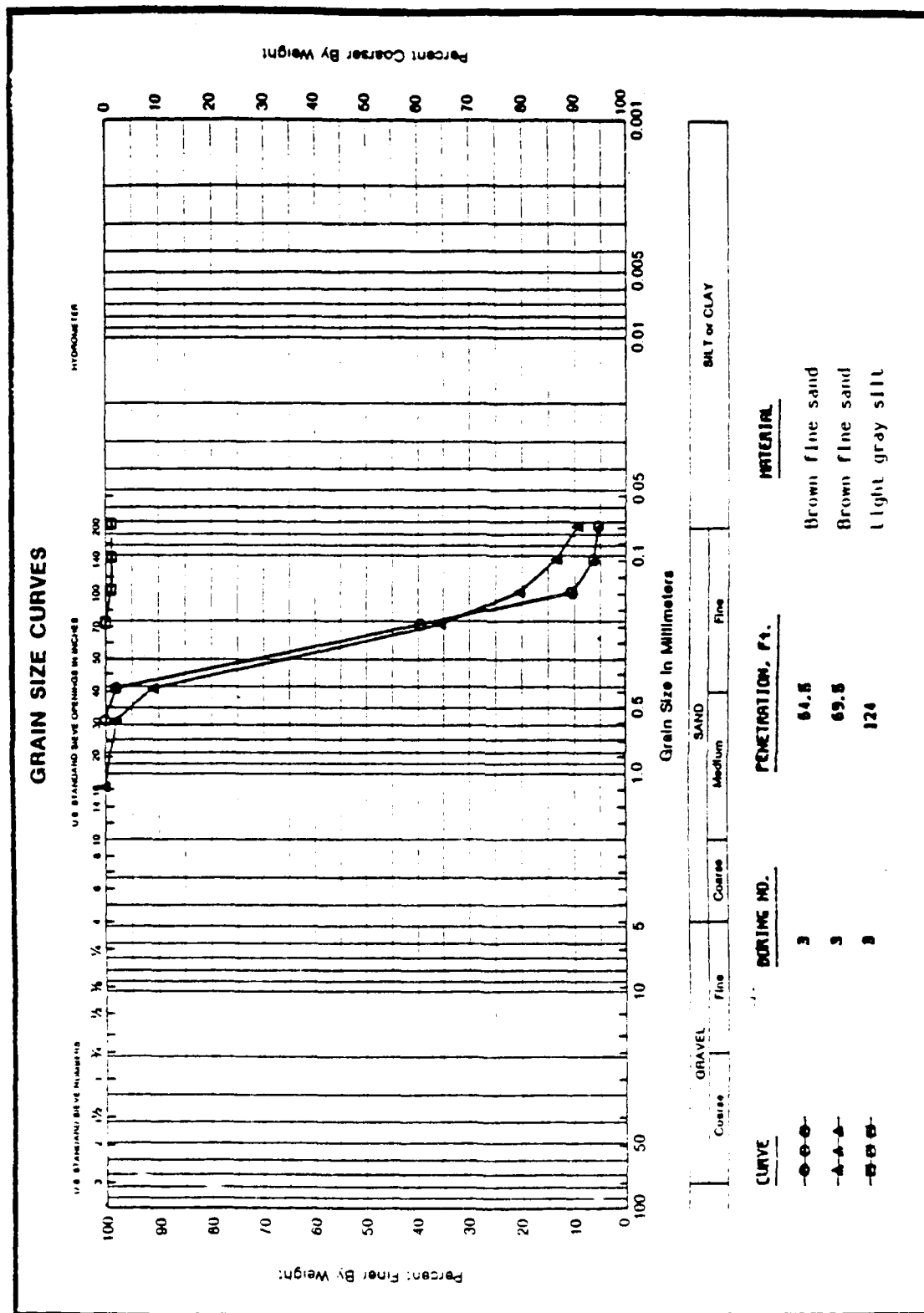
FIGURE 5b. Test Site Grain Size Curves, 64.5' to 124'

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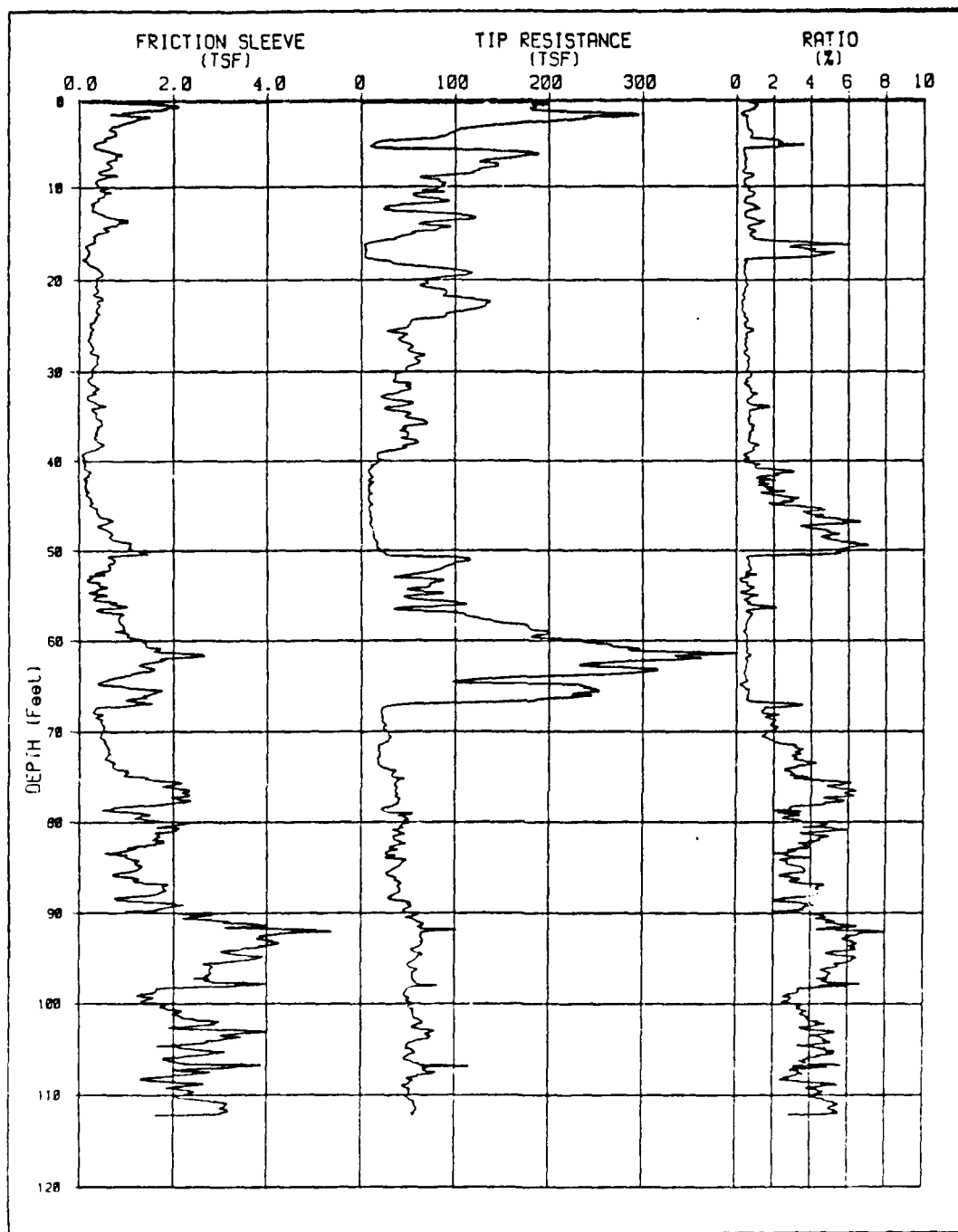
FIGURE 6a. Test Site Cone Penetrometer Data: CPT1
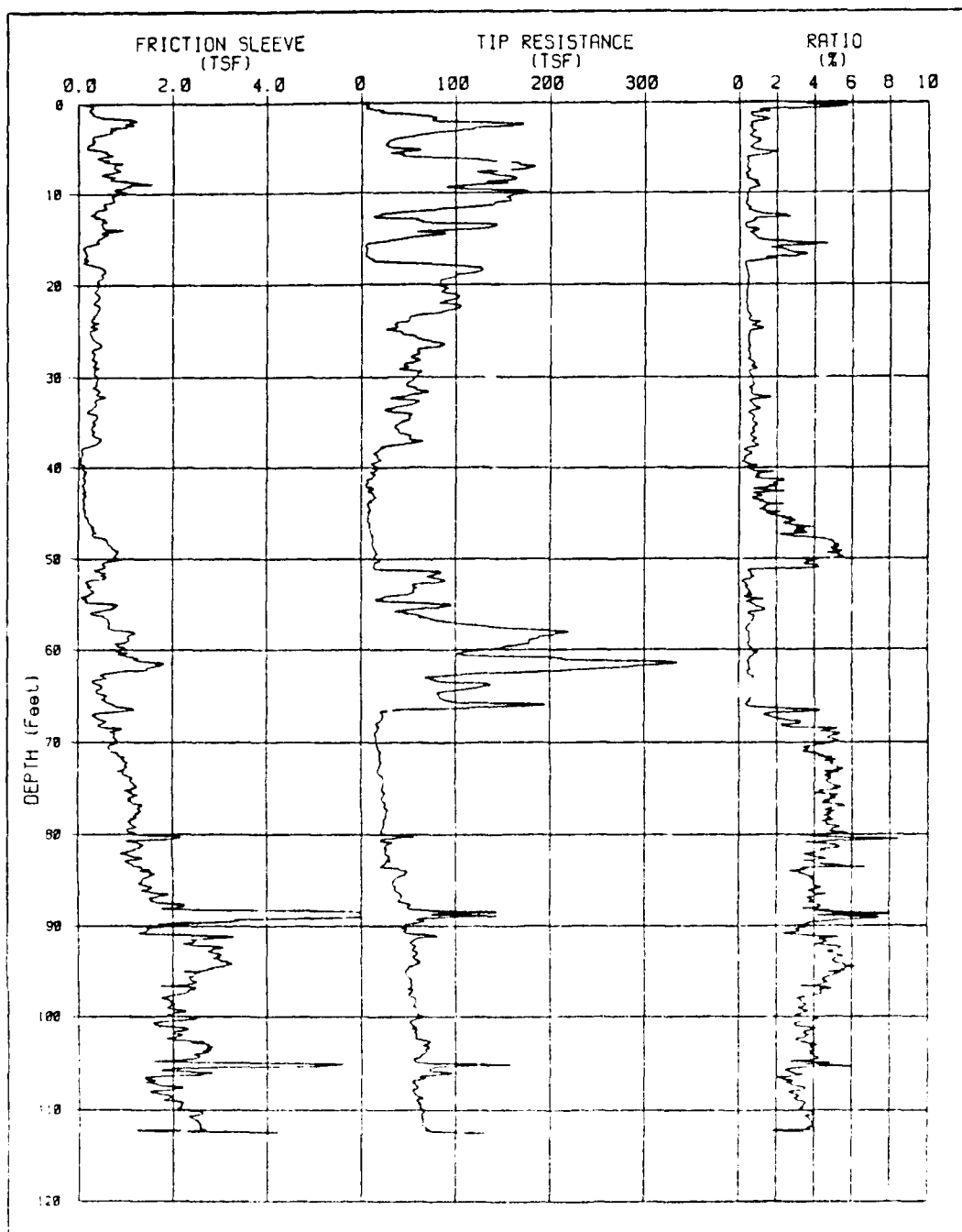
FIGURE 6b. Test Site Cone Penetrometer Data: CPT1

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3. THE PILES

3.1 Layout of the Piles

The six piles at the load test site were arranged in a triangular pattern as shown in Figure 7. This layout was selected at the time the piles were installed to load test vertically the three smaller piles on the legs of the triangle. The three 42-in diameter shafts in the corners were used for vertical reaction.

3.2 Geometry and Properties of the Piles

To accurately predict the response of laterally loaded piles, the geometry and flexural stiffness of the piles must be accurately represented. Correctly selecting the properties of the piles in this study was complicated since the piles had previously been stressed during the vertical load tests. The geometry and properties which were selected for use in the prediction process are presented in Table 1.

TABLE 1. Geometry and Properties of the Test Piles

Pile ID Number	Description	Embedded Length (ft)	Assumed EI ($lb-in^2$)
1	36" Diameter, Reinforced Concrete Drilled Shaft	97	1.70×10^{11}
2	24" O.D., 5/8" Thick Steel Pipe Pile (Open-ended)	20	0.91×10^{11}
3	20" Square Prestressed Concrete Pile	98	0.16×10^{11}
4,5,6	42" Diameter, Reinforced Concrete Drilled Shaft	128	1.80×10^{11}

The 36-in diameter drilled shaft was subjected to axial compression during the vertical load test. The flexural stiffness assumed for the lateral load test predictions was set equal to the stiffness obtained using its cracked moment of inertia. This method assumed that during lateral loading of the pile the portion of the pile cross section in compression transferred stresses to the concrete and to the

EXTENT OF 3-FT EXCAVATION

FIGURE 7. Arrangement and I.D. Numbers for the
          Test Piles
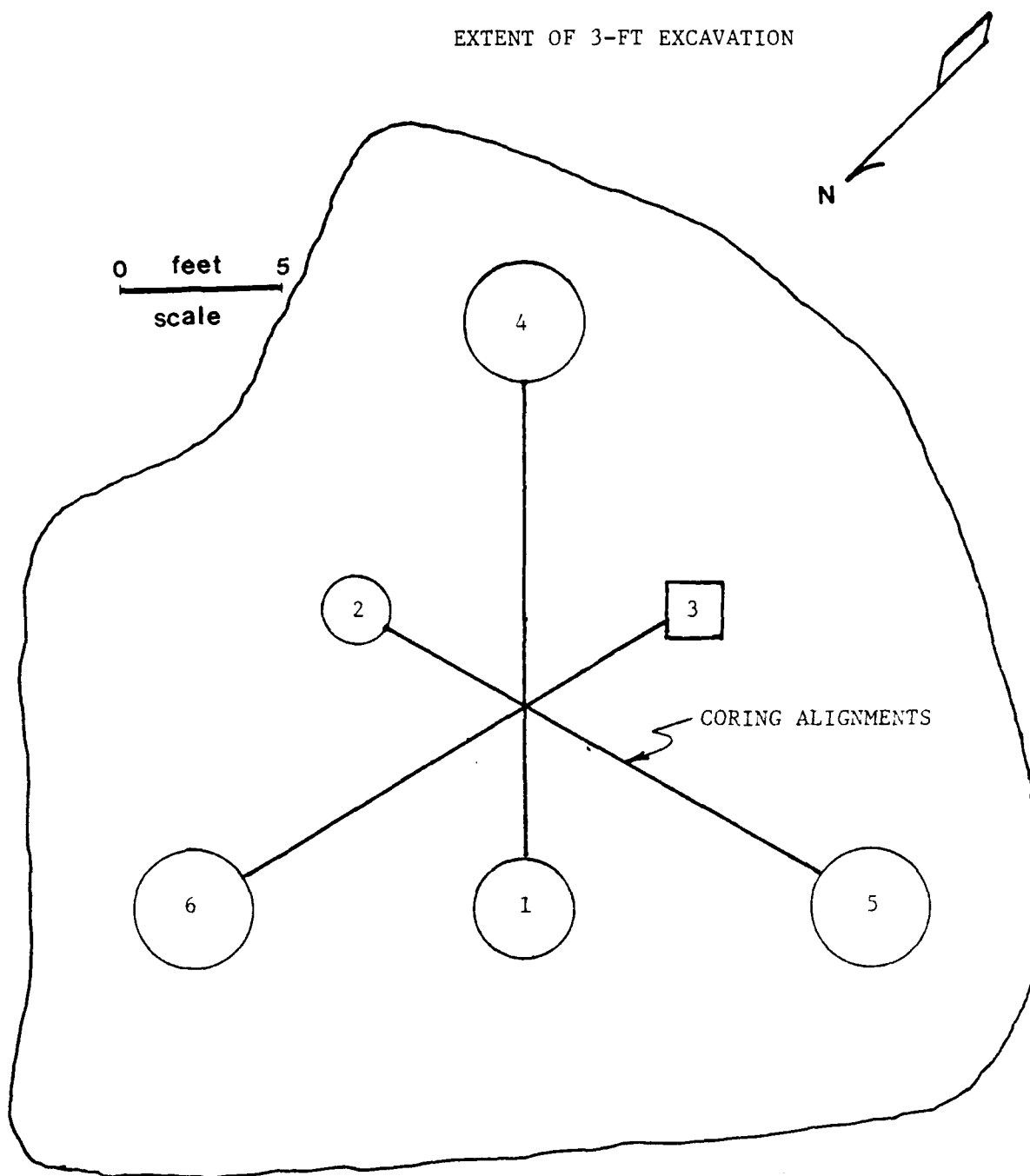
PILE NO.	TYPE
1	36" DIA. R.C. DRILLED SHAFT
2	24" O.D. 5/8" THICK STEEL PIPE PILE
3	20" SQUARE PRESTRESSED CONCRETE PILE
4,5,6	42" DIA. R.C. DRILLED SHAFTS

steel reinforcement. The portion of the cross section in tension, however, was assumed to carry all the stresses in the steel reinforcement alone. The areas of the cross section in compression and tension were assumed to be the compression and tension areas obtained when applying the allowable bending moment to the reinforced concrete section (Wang and Salmon, 1979). For a previously unstressed pile these assumptions may be considered to be conservative.

The 24-in pipe pile was assumed to have an elastic modulus of 29,000 ksi. The moment of inertia selected for the prediction process was based on the pile being completely empty of any soil throughout its length due to the soil plug being drilled out after driving.

For the 20-in square prestressed concrete pile, the stiffness calculation was further complicated by the fact the square cross section was not aligned with the direction of the horizontal load to be applied. The angle between the horizontal load and the sides of the square cross section was 26°. The selected stiffness in Table 1 considered the unusual angle of load application and was based on the cracked moment of inertia as explained for the 36-in drilled shaft. In all inertia computations, the prestressing strands were assumed to carry stresses only in tension, and were not included in the computations for the portion of the cross section in compression.

During the vertical load tests the three 42-in diameter reinforced concrete reaction shafts were subjected to axial tension up to 1000 tons. In the calculations of their flexural stiffness, the elastic modulus and the moment of inertia were substantially reduced from the values that would be assumed for a previously unstressed pile. This was necessary to account for the inevitable tension crack formation that must have occurred during the axial load tests.

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4. THE LATERAL LOAD TESTS

4.1 Site Preparation

The site had been backfilled following completion of the vertical load tests, necessitating excavation before performance of the lateral load tests. The boundaries of the excavation can be seen on the pile layout in Figure 7. The depth of the excavation was approximately 3 ft, allowing sufficient clearance for setting up the loading apparatus and the displacement gages support frame.

4.2 Loading Apparatus and Pile Preparation

The lateral loading of the piles was achieved using the system depicted in Figure 8. Each pile was cored horizontally to allow a length of 1-3/8 in, 150 ksi Dywidag-threadbar to be passed through the pile's central axis. The cored holes through each pile in the corner of the triangular layout were aligned with the cored holes through the pile on the opposite leg of the triangle (Figure 7). A length of threaded bar was passed through the cored hole of a corner pile and a 200-kip load cell was screwed onto the end of the bar near the center of the triangular layout. Another bar, passing through the pile on the opposite leg of the triangle, was screwed onto the other end of the load cell. Steel reaction pads were placed over the threaded bars behind the piles to distribute the lateral load over a wider area and the threaded bar was locked with a nut behind one of the two piles. A 200-kip hollow-core hydraulic jack was locked behind the opposite pile around the threaded bar. As the jack was expanded, the tensile force in the threaded bar pulled the two piles towards each other. The load cell measured the horizontal load applied to each pile.

Dial gages were securely attached to an independently supported displacement measuring frame. Deflections were measured at two points on each pile: one point below the

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200 KIP HYDRAULIC HOLLOW-CORE JACK DISPLACEMENT MEASURING FRAME

42" DIA. R.C. DRILLED SHAFT 9" TRAVEL DIAL GAGE

3" TRAVEL DIAL GAGE TEST PILE

DISPLACEMENT MEASURING FRAME

REACTION PAD 200 KIP LOAD CELL REACTION PAD

DYWIDAG NUT CORED HOLE DYWIDAG-THREADBAR
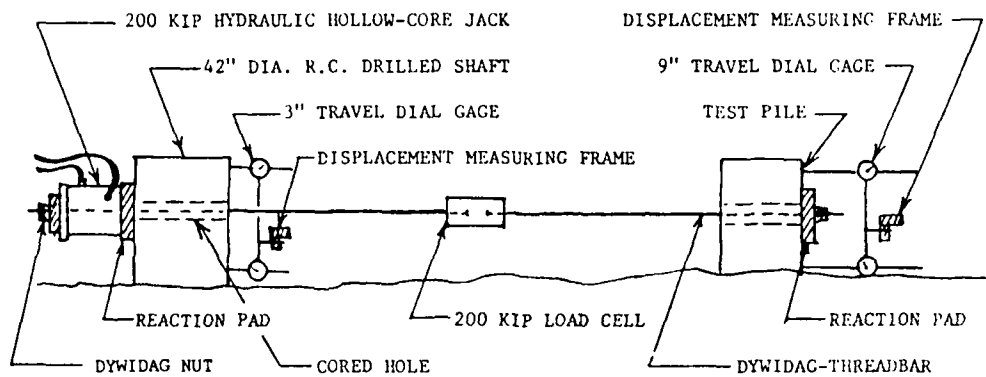
A = Distance from line of loading to top dial gage

B = Distance from line of loading to bottom dial gage

C = Distance from line of loading to ground surface

Pile I.D. No. *	A (in.)	B (in.)	C (in.)
1	24.75	(4.25)**	3.5
2	11.56	4.94	8.4
3	21.06	8.06	10.0
4	10.0	2.0	3.5
5	11.88	4.56	8.4
6	8.5	9.38	10.0

* See Figure 7.

** Above line of loading

FIGURE 8. Horizontal Load Application and
Displacement Measuring System

axis of loading close to the groundline and one above the axis of loading. This allowed the deflection and the slope at the groundline to be obtained. The position of the displacement measuring frame was checked with a transit before and after each load test to guarantee that there was no movement of the frame during testing. The locations of the dial gages and the line of loading are shown on Figure 8.

4.3 General Loading Scheme

The loading scheme for each test followed the same general pattern. Loads were applied in five kip increments. After each increment, displacement readings were taken immediately and at one minute intervals for five minutes as the load was maintained. Two load levels were selected during each test to perform 20 unload-reload cycles. The cycles were performed under load-control conditions. After reaching the first chosen load level, the displacements were recorded during the first five minutes as the load was maintained. The load was then decreased to near zero by completely relaxing the jack. Displacements and load readings were recorded after two minutes and the original cyclic load level was reapplied. A new set of readings were then recorded after an additional two minutes; the cyclic period was thus four minutes. After ten cycles, the bottom, lower load, of each cycle was increased to half of the top cyclic load level. After twenty cummulative cycles, the five-kip, five-minute incremental loading was resumed. When the second chosen cyclic load level was reached, the load was cycled between the chosen load level and one-half of the chosen load level for the first ten cycles and then between the chosen load level and near zero load for the last ten cycles. After completion of the second series of cycles the five-kip, five-minute incremental loading was resumed and continued until the end of the test.

4.4 Results of the Lateral Load Tests

Tabulated results of the lateral load tests are pre-

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sented in Appendix A. Lateral loads versus horizontal de-
flections of the piles are presented graphically in Figures
9 through 20. The displacements are those measured by the
lowest dial gage for each pile, as described in Section
4.2. Two graphs are presented for each pile: one showing
the entire response range during the load test and another
detailing the cyclic response.

4.4.1 Monotonic response envelopes

Curves enveloping the response of the piles to incre-
mental loading intervals are presented as monotonic response
envelopes in Figures 21 and 22. These curves yield a con-
servative estimate of each pile's behavior under strictly
monotonic incremental loading. In reality, the responses
for identical piles not subjected to the two series of
cycles would likely be stiffer. This can be substantiated
by observing the pronounced permanent displacements experi-
enced by each pile during cyclic loading in the case where
the load was decreased to almost zero load (Figures 9
through 20). Furthermore, the concrete piles were subjected
to increased crack propagation during the cyclic series (see
Section 6) effectively reducing their stiffness as the tests
progressed.

The monotonic response envelopes allow comparisons
between the piles to be made. The three 42-in diameter
drilled shafts reponded within a narrow range of values,
showing consistency within the testing method, shaft con-
struction and soil properties. They proved to have the
stiffest response, followed by the 36-in diameter drilled
shaft, the 24-in diameter steel pipe pile and finally the
20-in square prestressed concrete pile.

One of the 42-in diameter drilled shafts failed during
the load tests (Pile No. 6). This premature failure re-
flects the damage incurred by the reaction shafts during the
vertical load tests discussed in Section 3.

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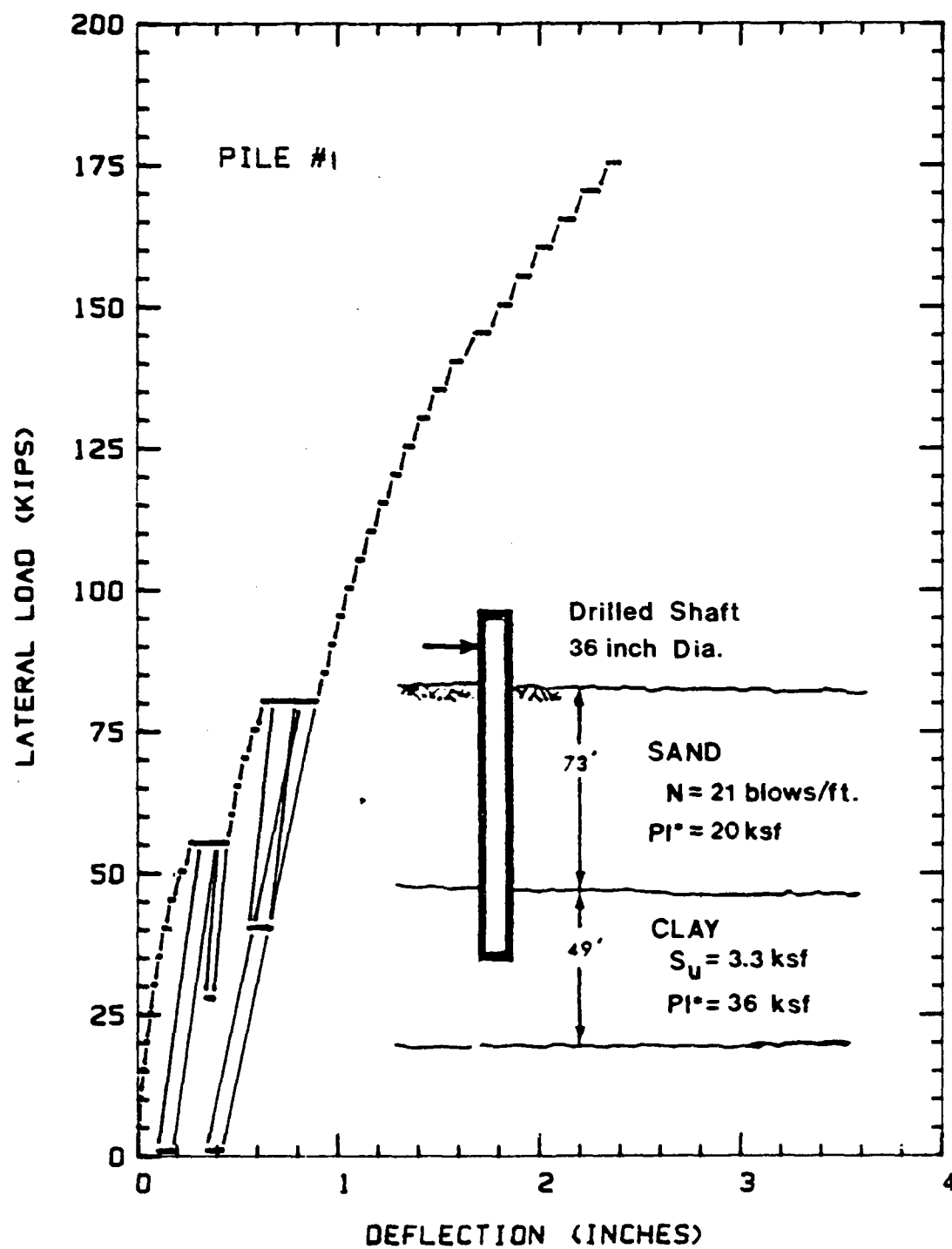
FIGURE 9. Measured Response from Cyclic
Lateral Load Test for Pile No. 1,
0 to 200 Kips Scale.
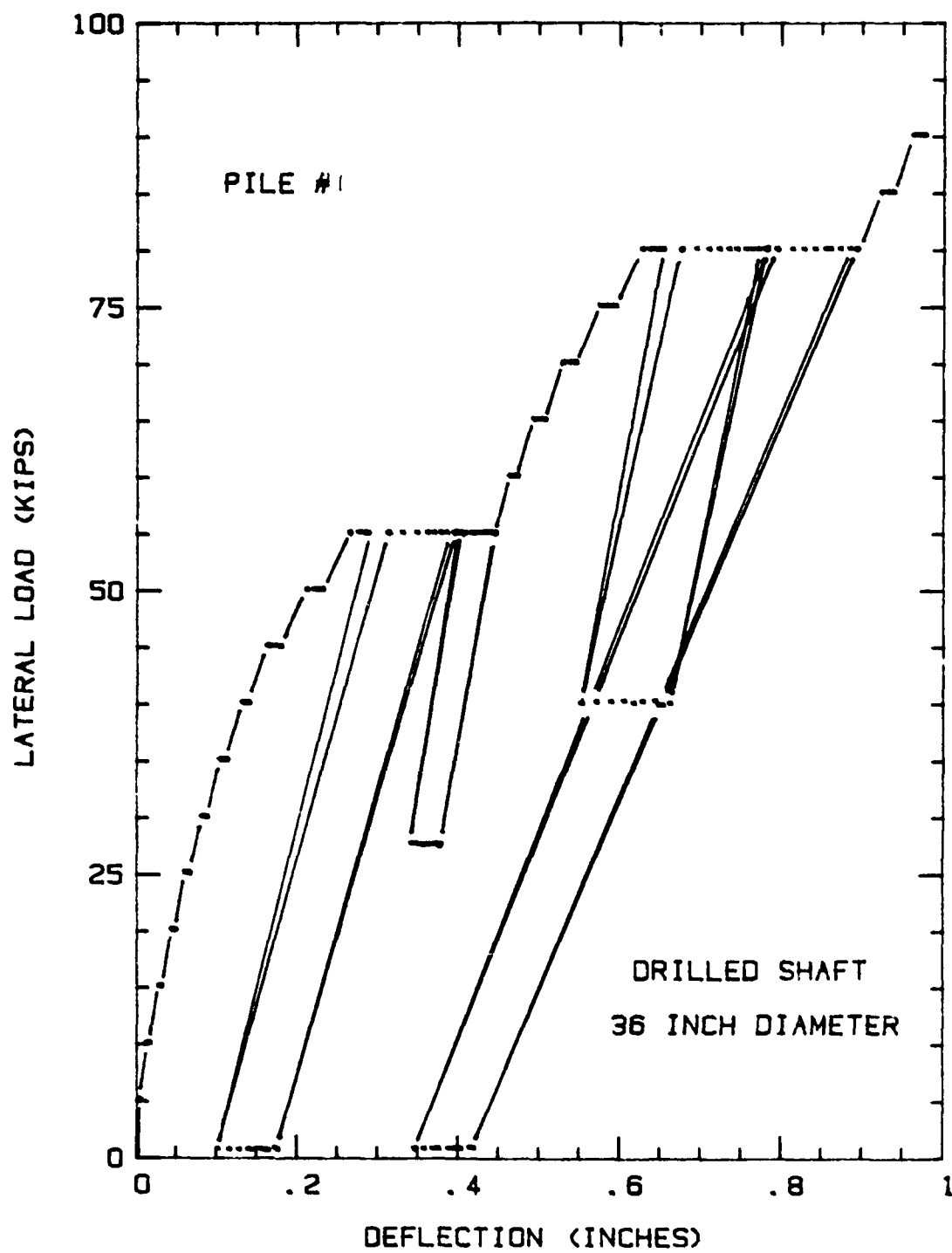
FIGURE 10. Measured Response from Cyclic
Lateral Load Test for Pile No. 1,
Cycling Detail, 0 to 100 Kips Scale.

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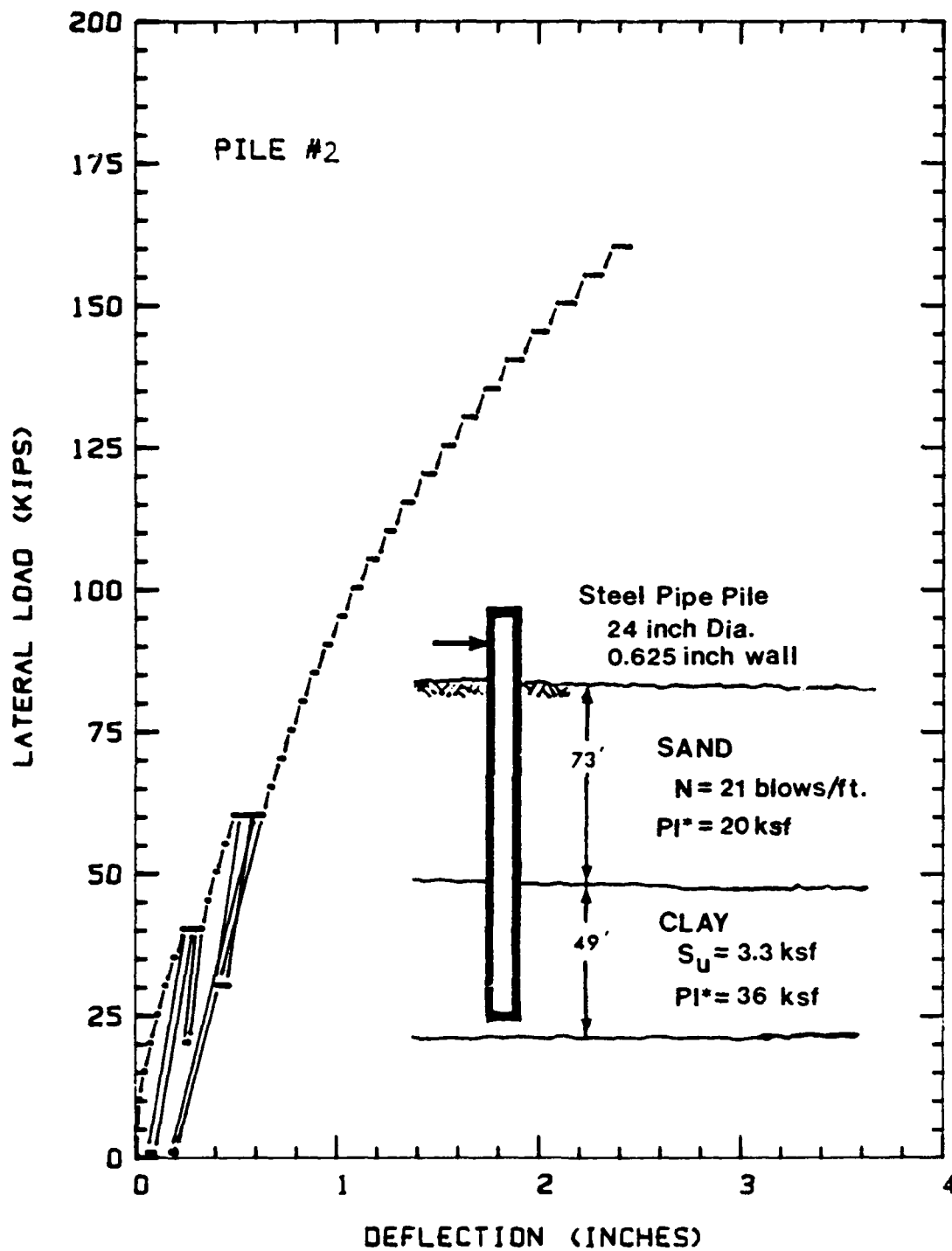
FIGURE 11.   Measured Response from Cyclic Lateral
Load Test for Pile No. 2, 0 to 200
Kips Scale.
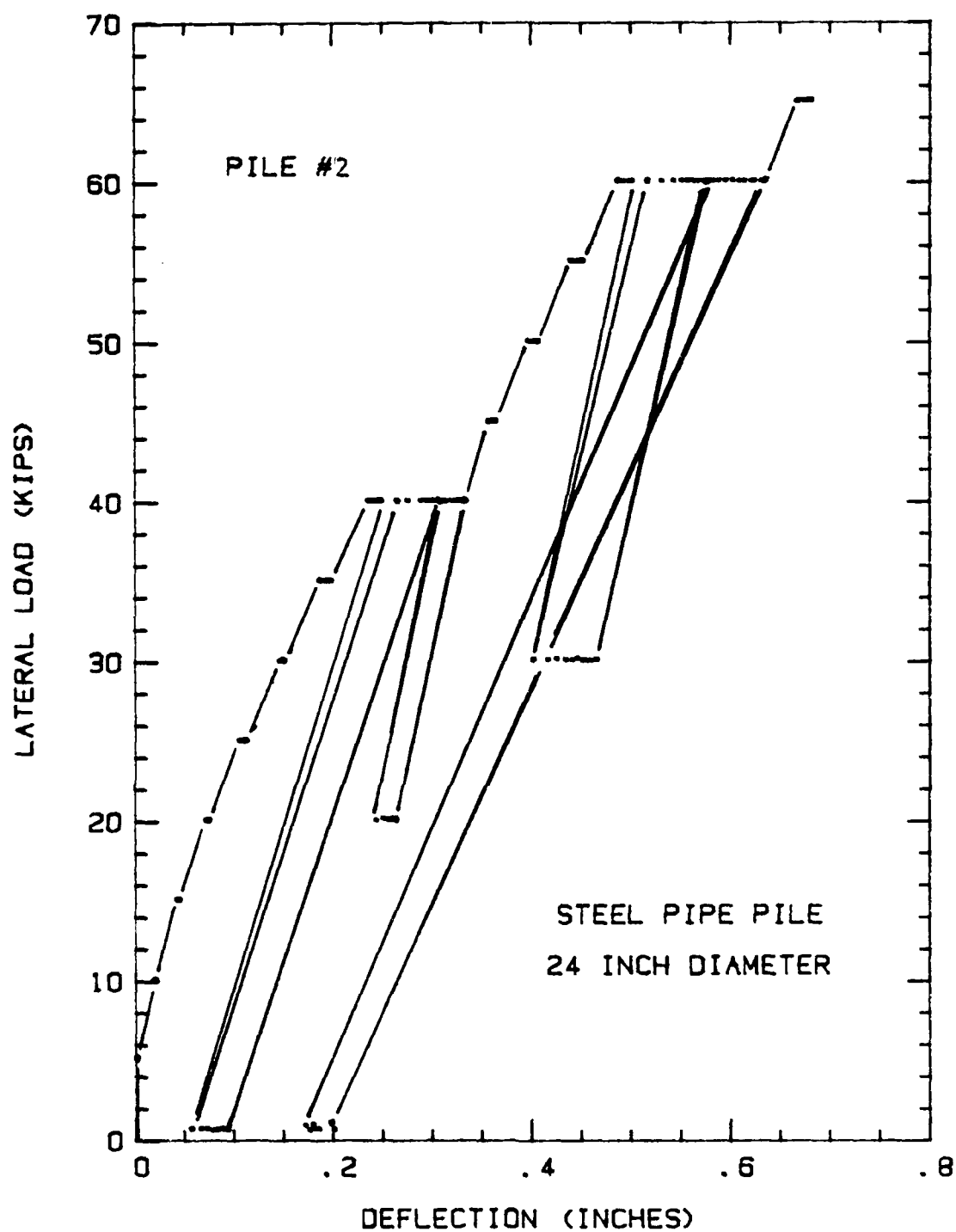
FIGURE 12. Measured Response from Cyclic Lateral
Load Test for Pile No. 2, Cycling
Detail, 0 to 70 Kips Scale.
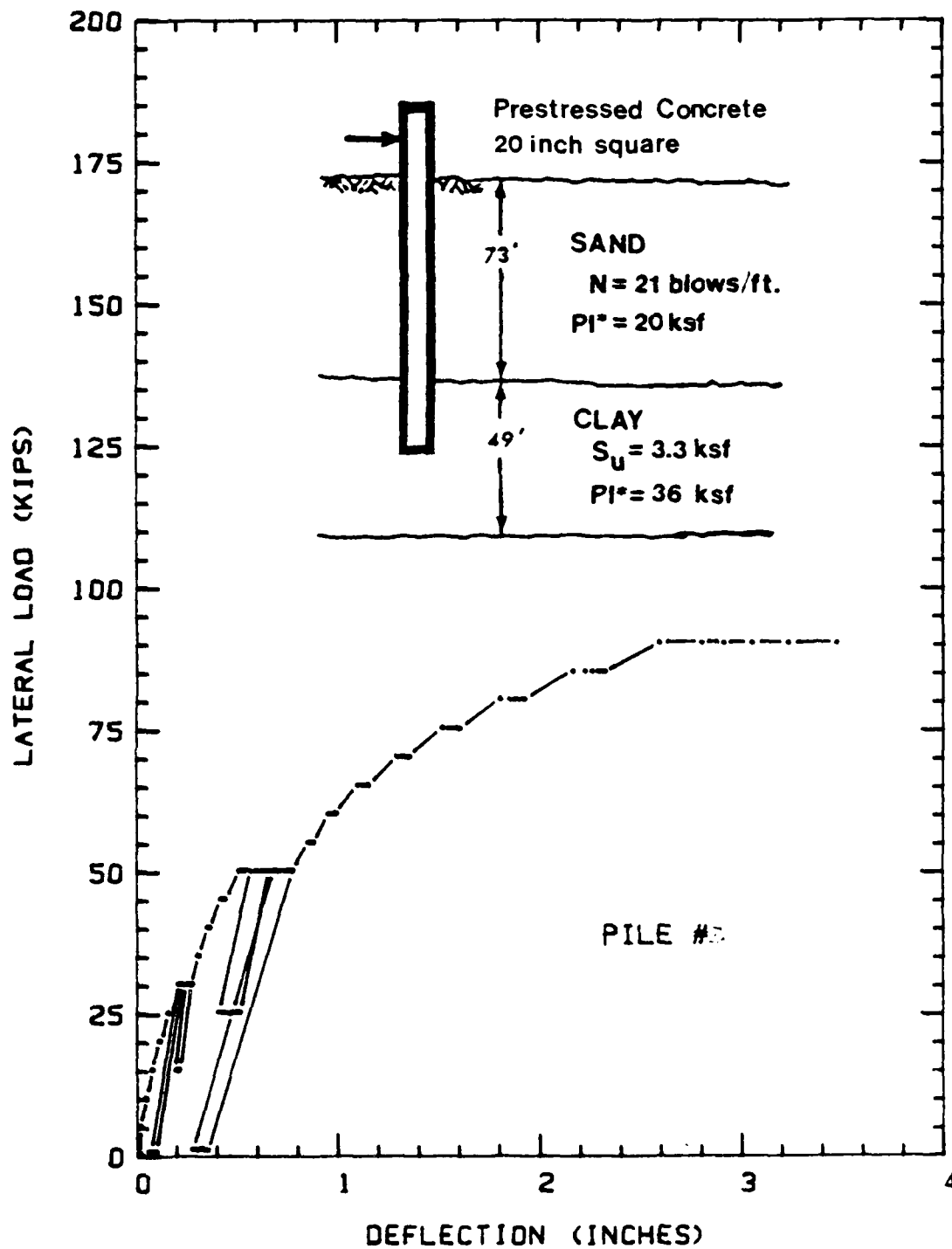
FIGURE 13. Measured Response from Cyclic Lateral Load Test for Pile No. 3, 0 to 200 Kips Scale.
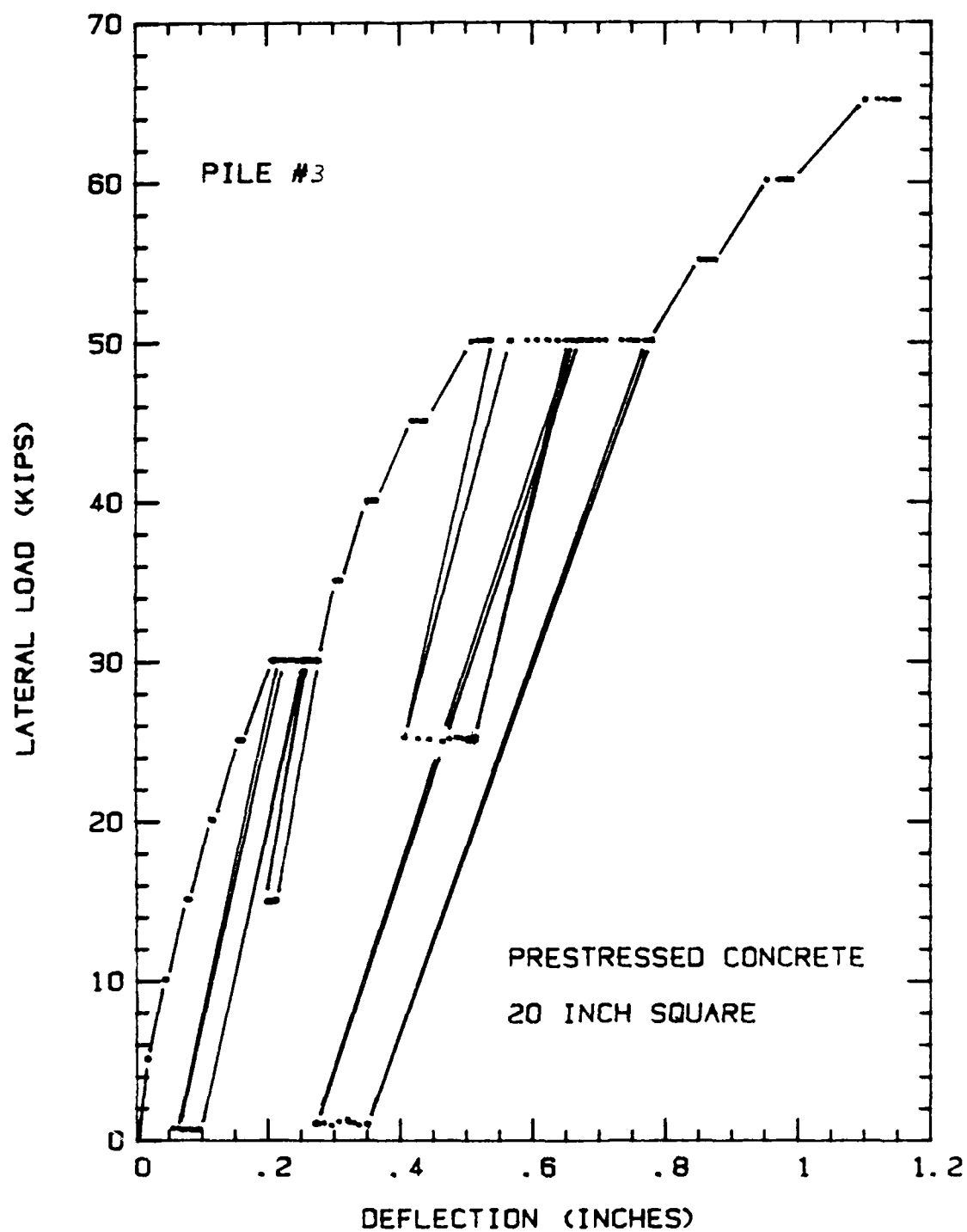
FIGURE 14. Measured Response from Cyclic Lateral Load Test for Pile No. 3, Cycling Detail, 0 to 70 Kips Scale.
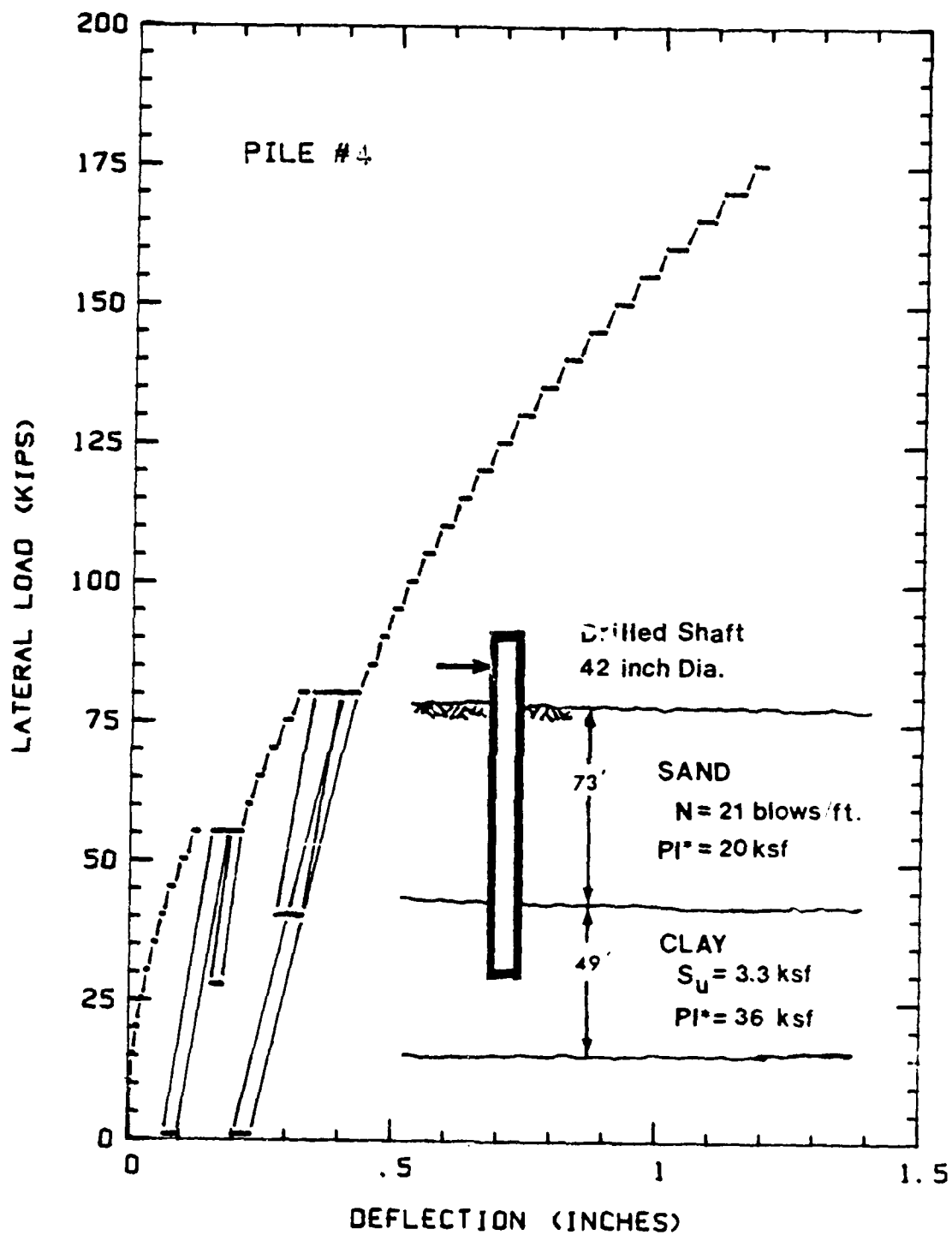
FIGURE 15.   Measured Response from Cyclic Lateral
             Load Test for Pile No. 4, 0 to 200
             Kips Scale.
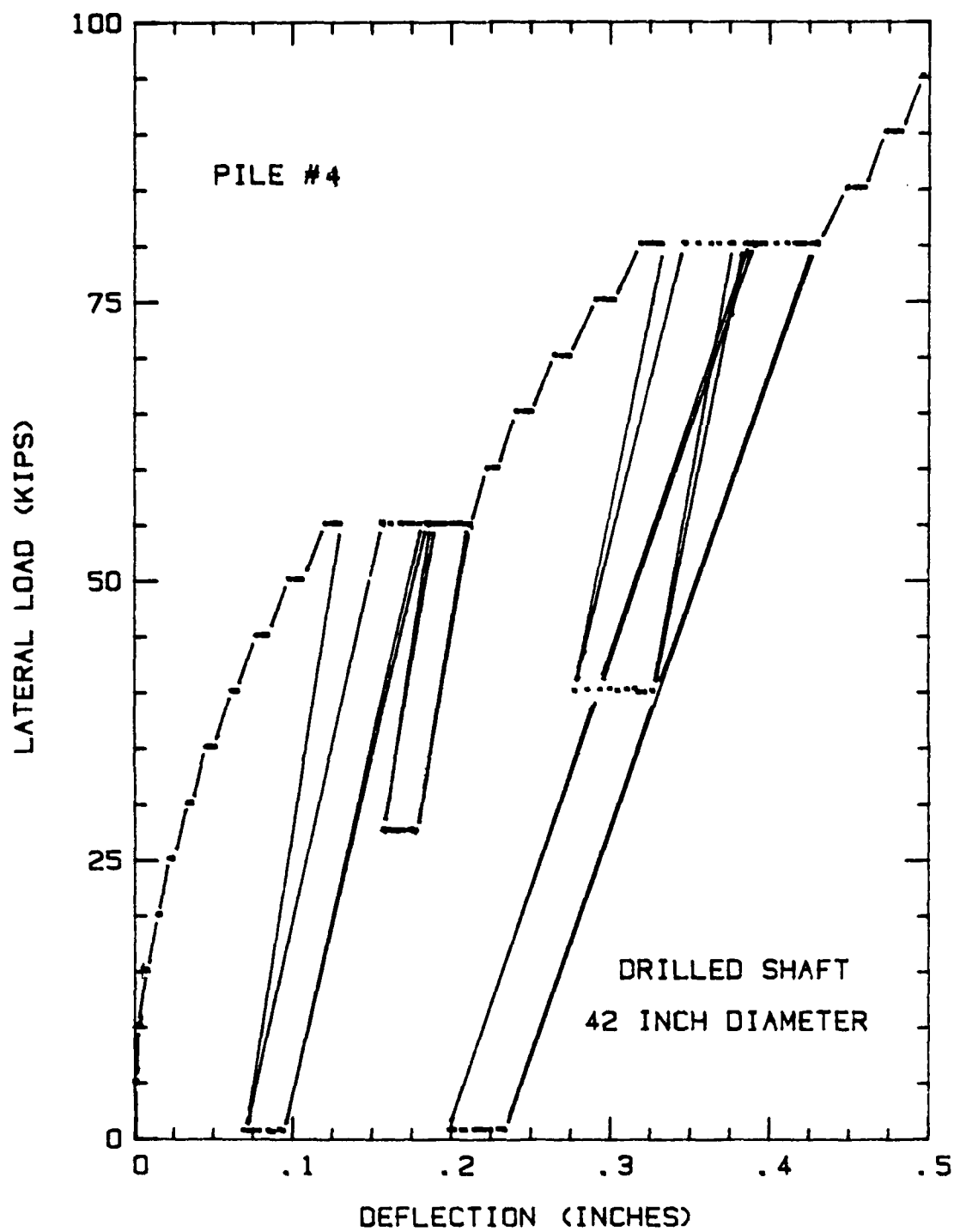
FIGURE 16.   Measured Response from Cyclic Lateral
              Load Test for Pile No. 4, Cycling Detail,
              0 to 100 Kips Scale.
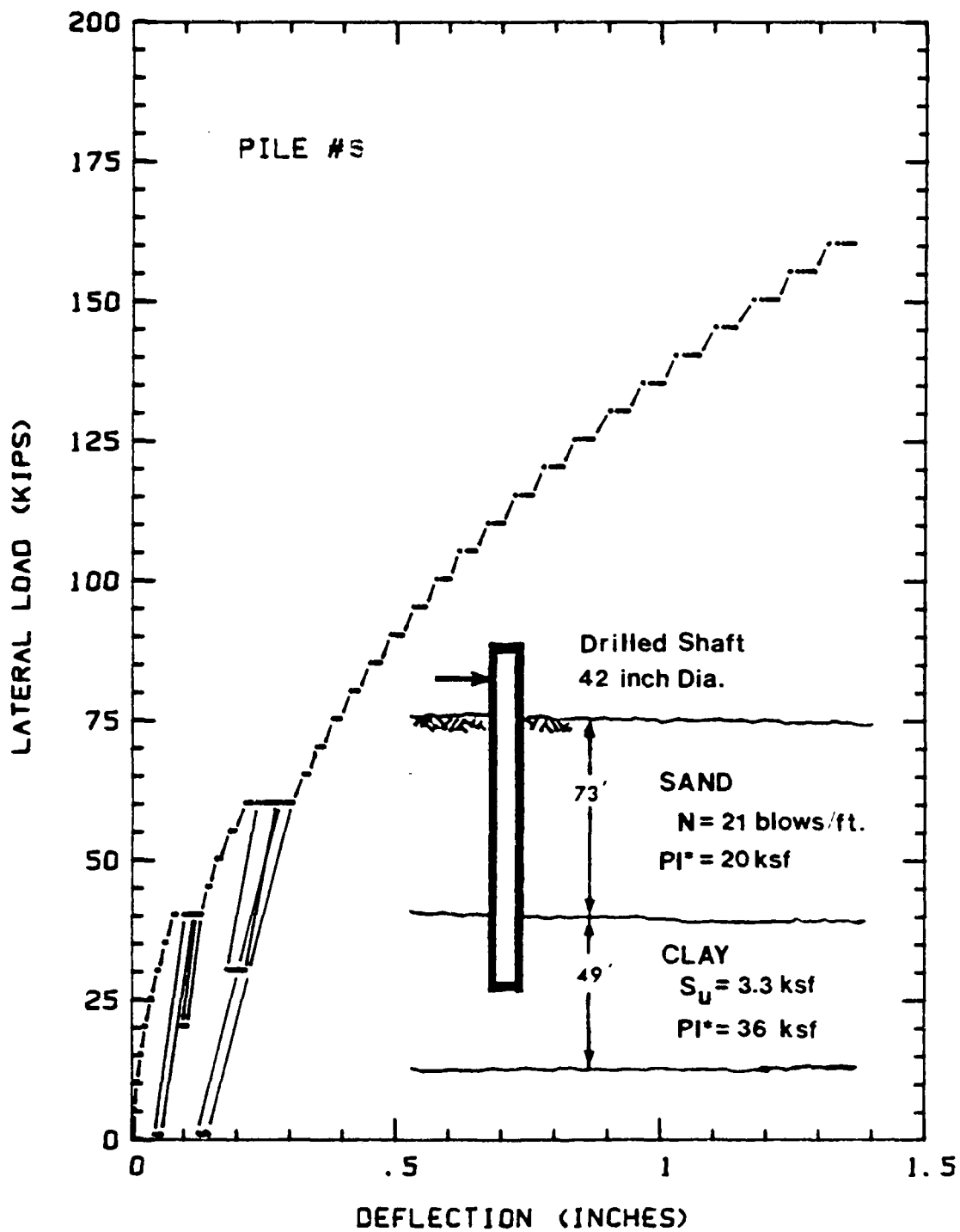
FIGURE 17. Measured Response from Cyclic Lateral
Load Test for Pile No. 5, 0 to 200
Kips Scale.
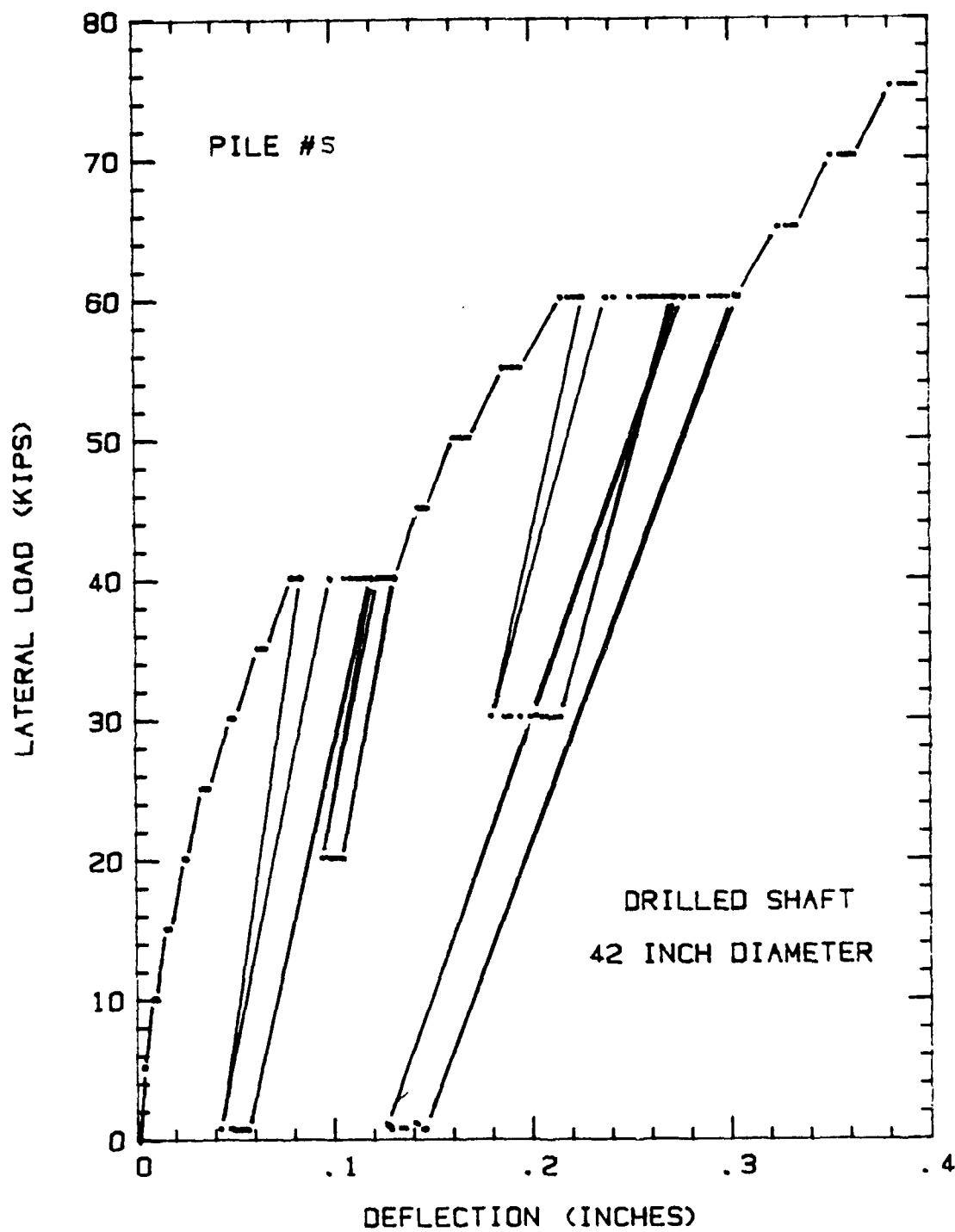
FIGURE 18. Measured Response from Cyclic Lateral
Load Test for Pile No. 5, Cycling Detail,
0 to 80 Kips Scale.

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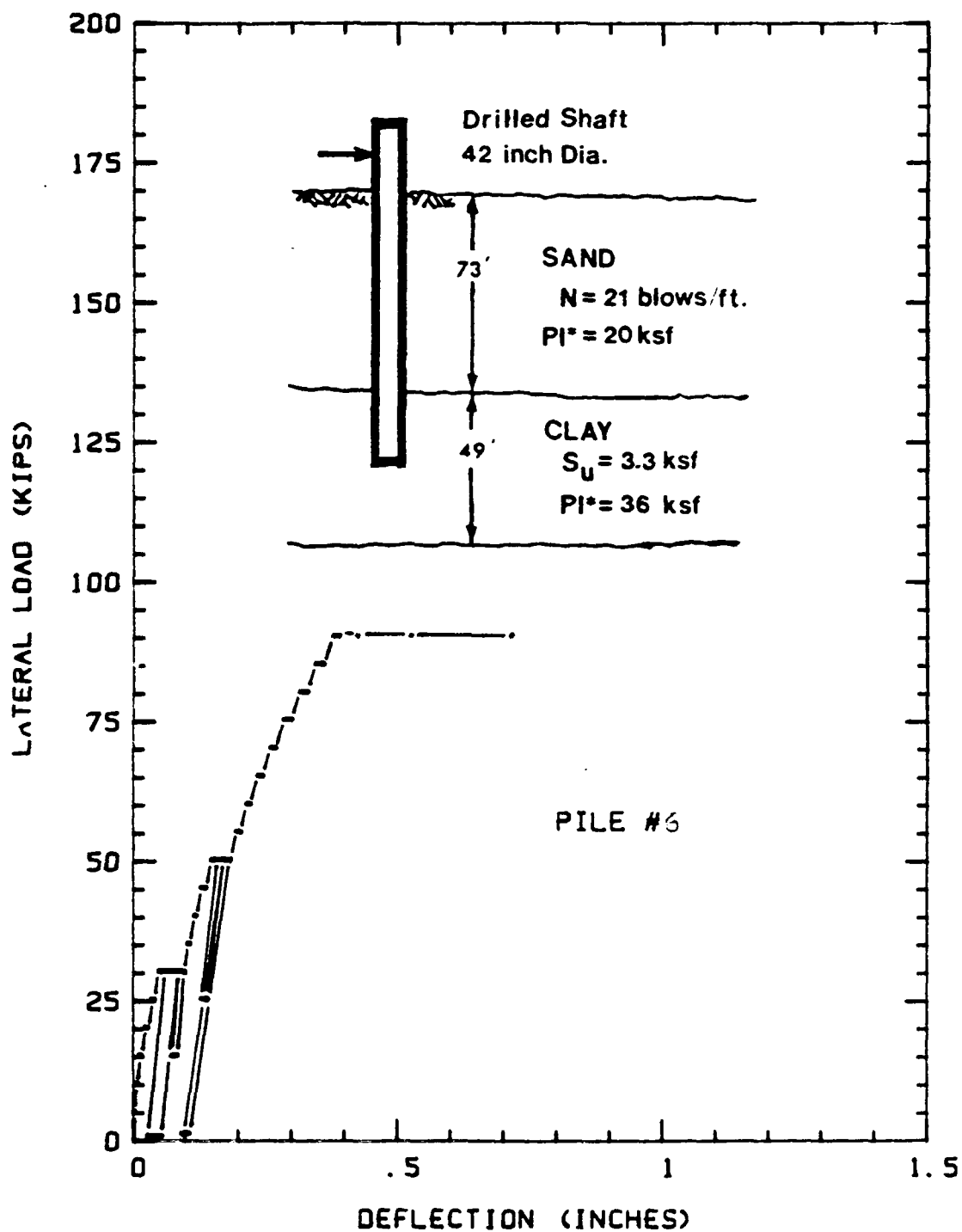
FIGURE 19. Measured Response from Cyclic Lateral Load Test for Pile No. 6, 0 to 200 Kips Scale.

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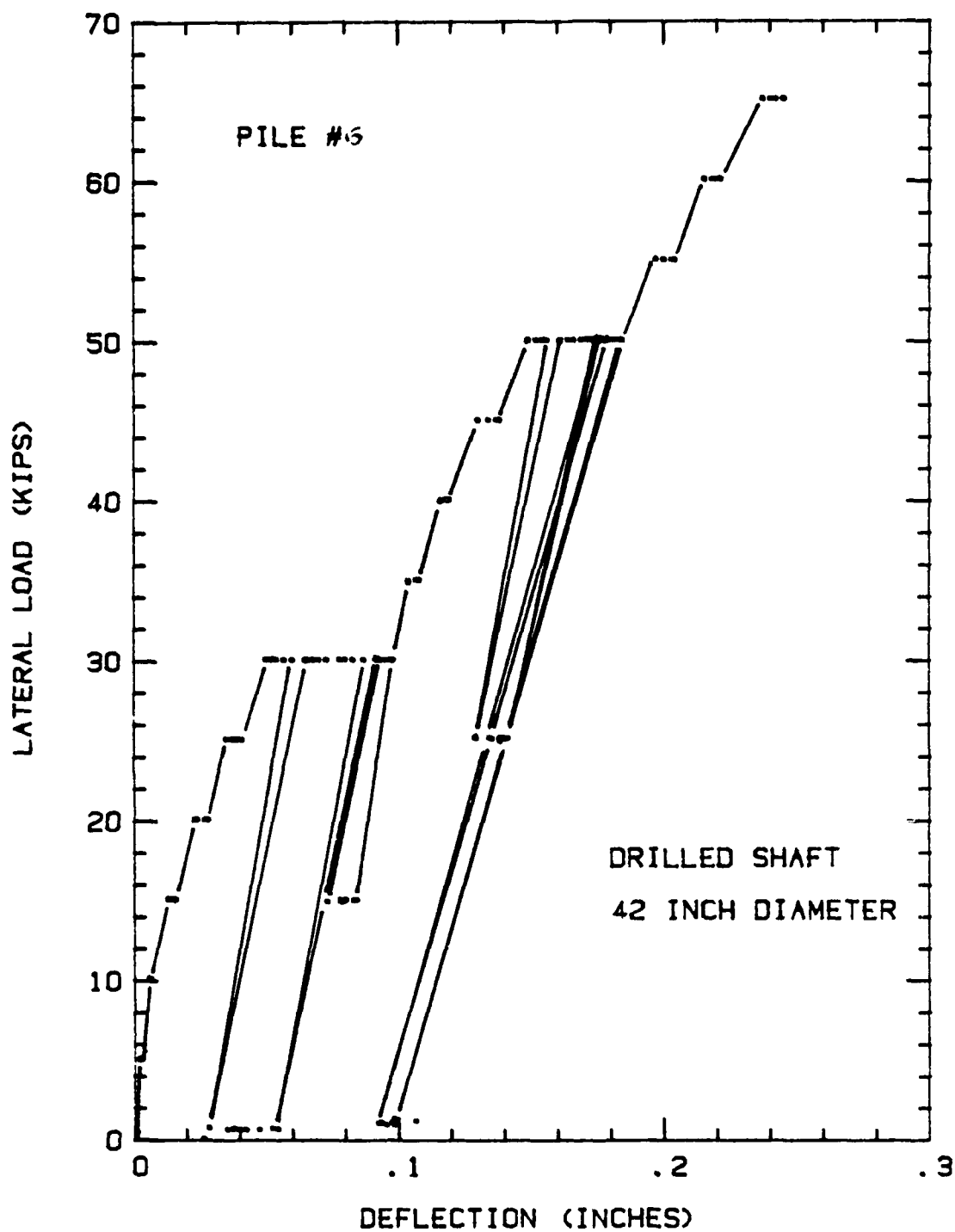
FIGURE 20.   Measured Response from Cyclic Lateral
Load Test for Pile No. 6, Cycling Detail,
0 to 70 Kips Scale.

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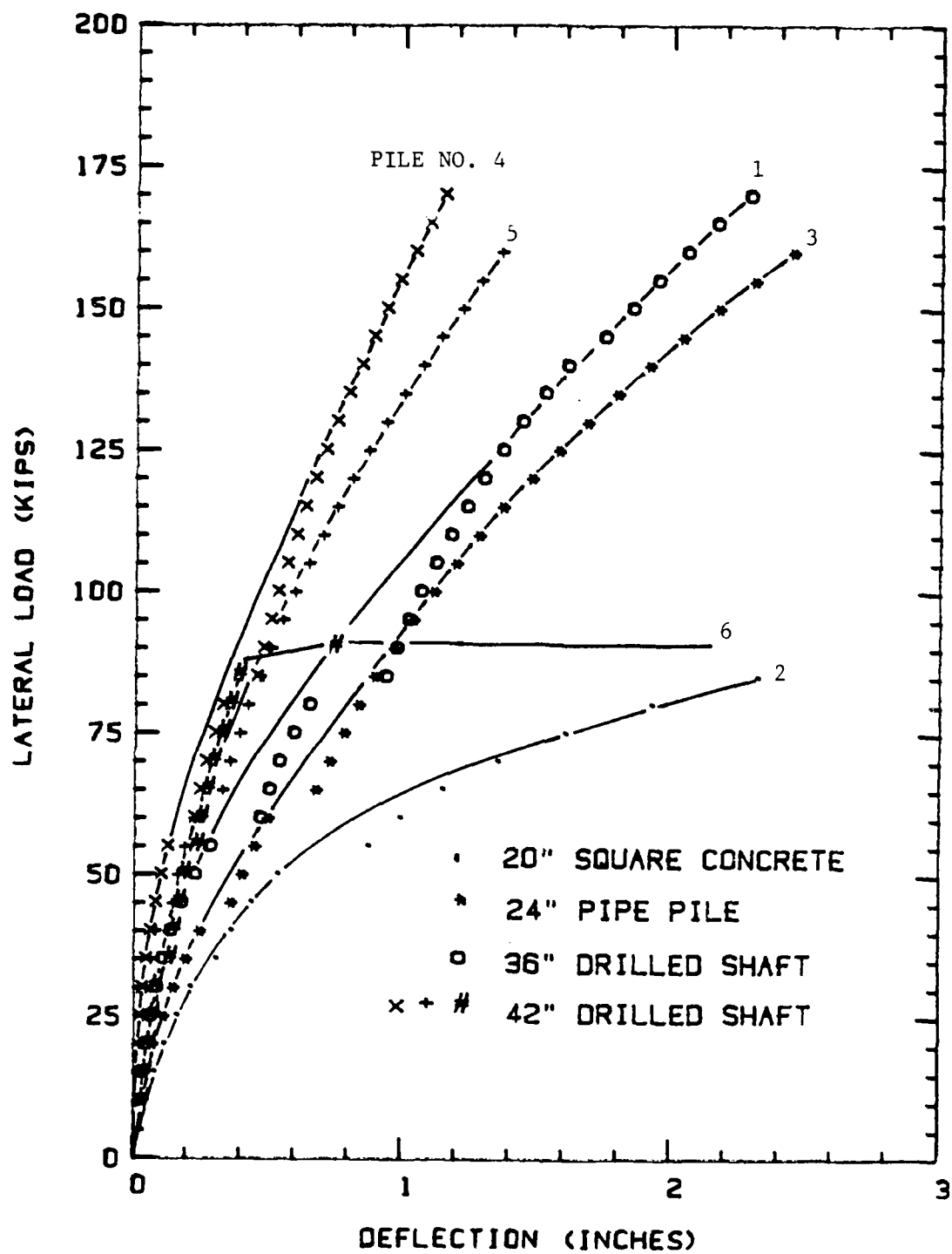
FIGURE 21. Monotonic Response Envelopes Measured
During Pile Load Tests, Full Range Scale.

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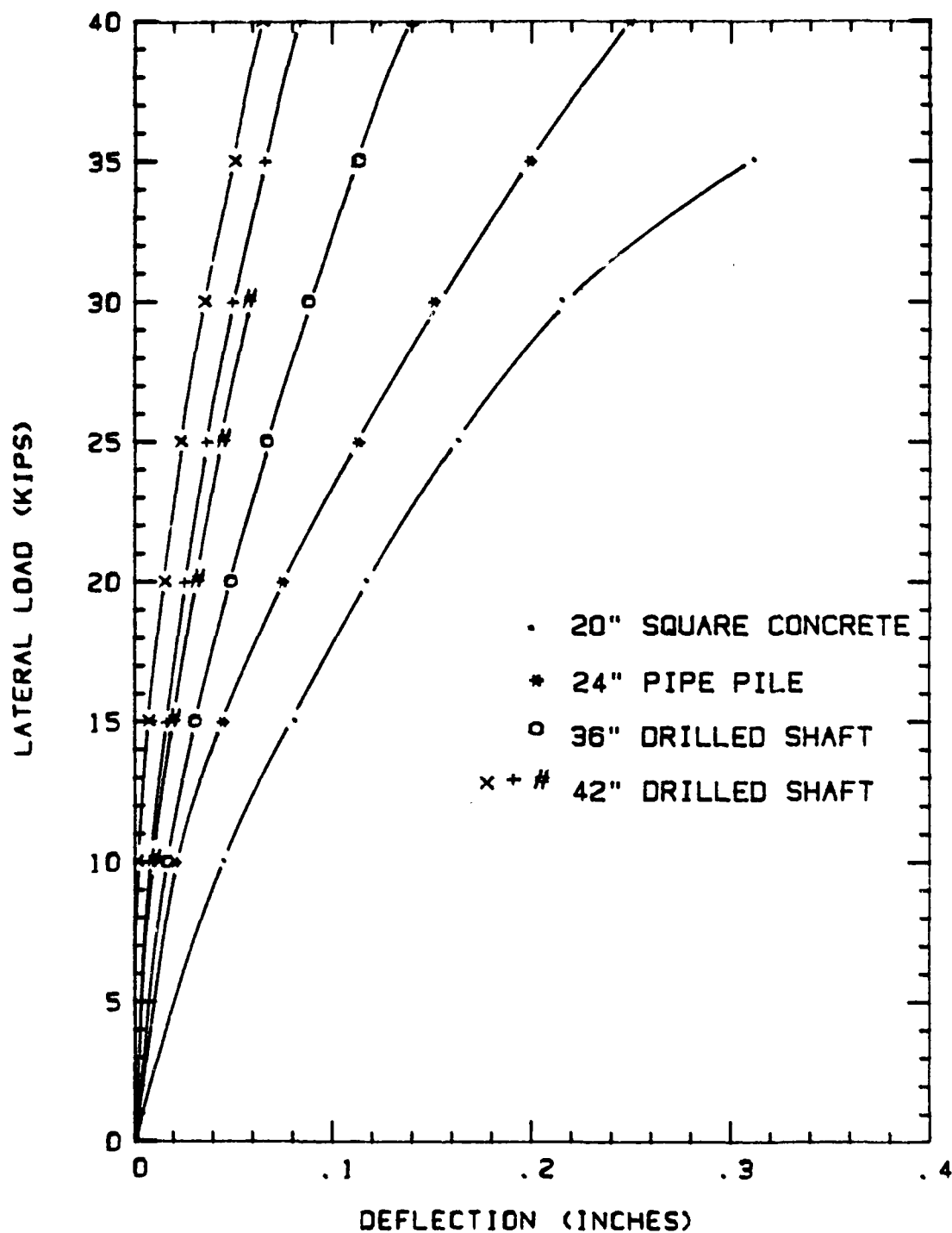
FIGURE 22.   Monotonic Response Envelopes Measured
             During Pile Load Tests, 0 to 40 Kips
             Scale.

4.4.2 Cyclic response and degradation

The cyclic response of the piles is presented in two different ways: 1. as percentage increase in displacement after cyclic loading, and 2. in terms of cyclic degradation of the secant and cyclic shear stiffnesses.

The percentage increase in displacement was measured as shown in Figure 25. Table 2 lists the increase in displacement after 10 and 20 cycles for each pile at each cyclic load level. Several observations concerning the pile's responses may be made from the tabulation.

TABLE 2. Measured Cyclic Percentage Increase in Displacement from the Pile Load Tests

Pile ID No.	Description	First Cycling Level			Second Cycling Level		
		Load (kips)	% Increase in Displacement a 10 cycles	a 20 cycles	Load (kips)	% Increase in Displacement a 10 cycles	a 20 cycles
1	36" Drilled Shaft	55	51.1	66.9	80	17.9	35.4
2	24" Pipe Pile	40	24.9	34.1	60	15.2	25.9
3	20" Square Concrete	30	22.3	27.9	50	26.4	43.1
4	42" Drilled Shaft	55	55.4	72.7	80	17.8	28.3
5	42" Drilled Shaft	40	41.7	56.0	60	19.5	34.1
6	42" Drilled Shaft	30	48.1	79.6	50	11.5	17.9

The four reinforced concrete drilled shafts suffered significantly more loss in pile-soil stiffness during the first cyclic series than the steel pipe or prestressed concrete pile. This greater loss probably reflects the rapid deterioration in the piles' flexural stiffness as the concrete experiences crack propagation, and the more compressible soil left by the drilling process at the concrete/soil interface. At the second, higher, cyclic load level, the relative increase in displacements with increasing number of

35
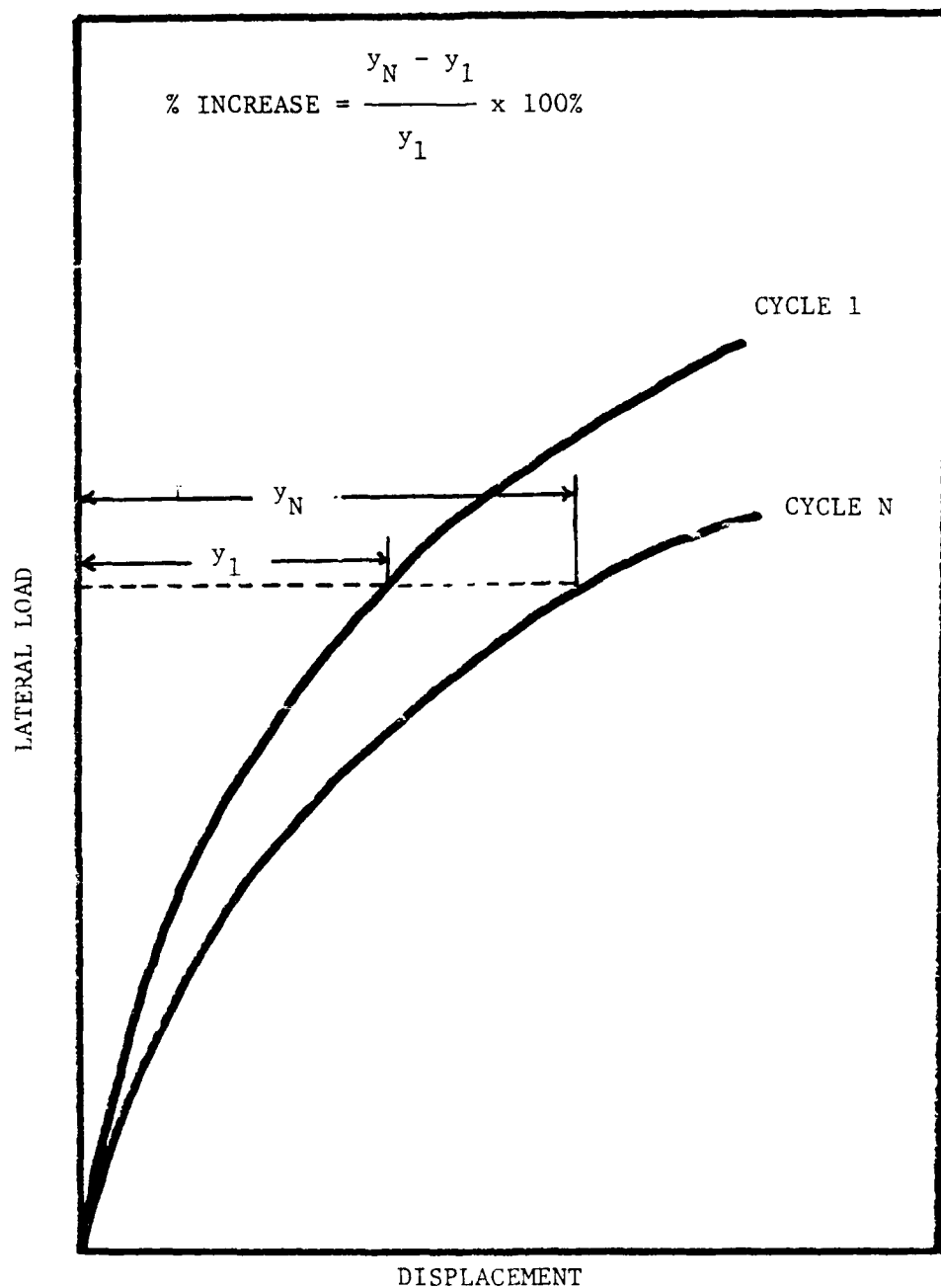
FIGURE 23.   Percentage Increase in Displacement
             Calculation.

36

cycles was much lower. This difference may be explained as follows. During the first series of cycles the piles' flexural stiffness deterioration contributed significantly to the total pile-soil stiffness response. In the second cyclic series, as the piles' flexural stiffness was reaching a limiting value and since the sand had been stiffened by the first series of cycles, the cyclic deterioration of the pile-soil stiffness was much less. As an example, notice that pile 4 was cycled at 55 kips and experienced an increase in displacement of 72.7% after 20 cycles, whereas pile 6 cycled at 50 kips experienced only a 17.9% increase in displacement. The significant difference was that the 55 kips cycling level was the first cyclic series for pile 4 and the 50 kips cycling level was the second cyclic series experienced by pile 6, occurring after a cyclic series at a load level of 30 kips.

The prestressed 20-in square concrete pile and the steel pipe pile did not exhibit the same behavior. Prestressing of the square concrete pile enabled it to resist the effects of crack propagation by keeping a larger portion of the pile cross-section in compression during the lateral loading. As a result, the percentage increase in d splacements of the prestressed concrete pile duplicate more closely the behavior of the steel pipe pile during the first series of cycles. At the second cyclic load level, the prestressed concrete pile showed a significant increase in relative displacement during the last ten cycles. At this load level, the internal bending moment within the pile was probably of sufficient magnitude to put a large portion of the concrete cross-section into tension, causing crack propagation and the associated loss in pile stiffness.

Very little loss in pile stiffness alone was expected in the 24-in diameter steel pipe pile load test, especially at low load levels. It is reasonable to assume then, that

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the majority of the relative increase in displacement due to cyclic loading is the result of the soil stiffness degradation. This assumption seems to be reinforced when comparing the response of the 24-in diameter pipe at the 60-kip load level with the response of the drilled shafts during the second series of cycles. Indeed, all the values are relatively close to one another.

Another method used to evaluate the effect of cyclic loading on the pile-soil stiffness was to evaluate the degradation of the piles secant stiffness $K_{S(N)}$ as described in Figure 24. The cyclic degradation parameter "a" is defined as the negative slope of the best fit line through the points plotted on the graph of the relative secant stiffness $K_{S(N)}/K_{S(1)}$, versus the cycle number, N, on a log-log scale (Figure 24). The "a" values obtained from the pile load tests are presented in Table 3. The actual relative secant stiffness degradation plots are presented in Figures 25 through 30.

TABLE 3. Measured Secant Shear Modulus Degradation Parameters

| Pile ID No. | Description | Top Cyclic Load, H (kips) | "a" Values | | | | |
			Reload a H/2	Reload near 0	Average 1st Level	Average 2nd Level	Overall Average
1	36" Drilled Shaft	55 80	0.064 0.046	0.098 0.136	0.091	0.081	0.086
2	24" Pipe Pile	40 60	0.069 0.038	0.061 0.081	0.065	0.059	0.062
3	20" Square Concrete	30 50	0.031 0.067	0.045 0.109	0.038	0.088	0.063
4	42" Drilled Shaft	55 80	0.095 0.044	0.090 0.091	0.093	0.068	0.080
5	42" Drilled Shaft	40 60	0.066 0.049	0.080 0.096	0.073	0.073	0.073
6	42" Drilled Shaft	30 50	0.082 0.030	0.092 0.068	0.087	0.049	0.068
	Averages		0.057	0.087	0.075	0.070	0.072
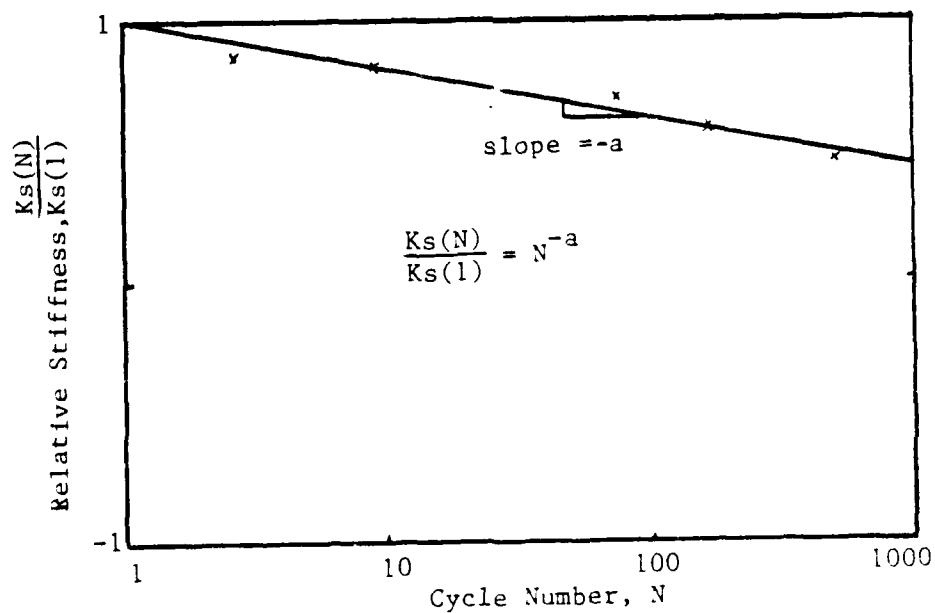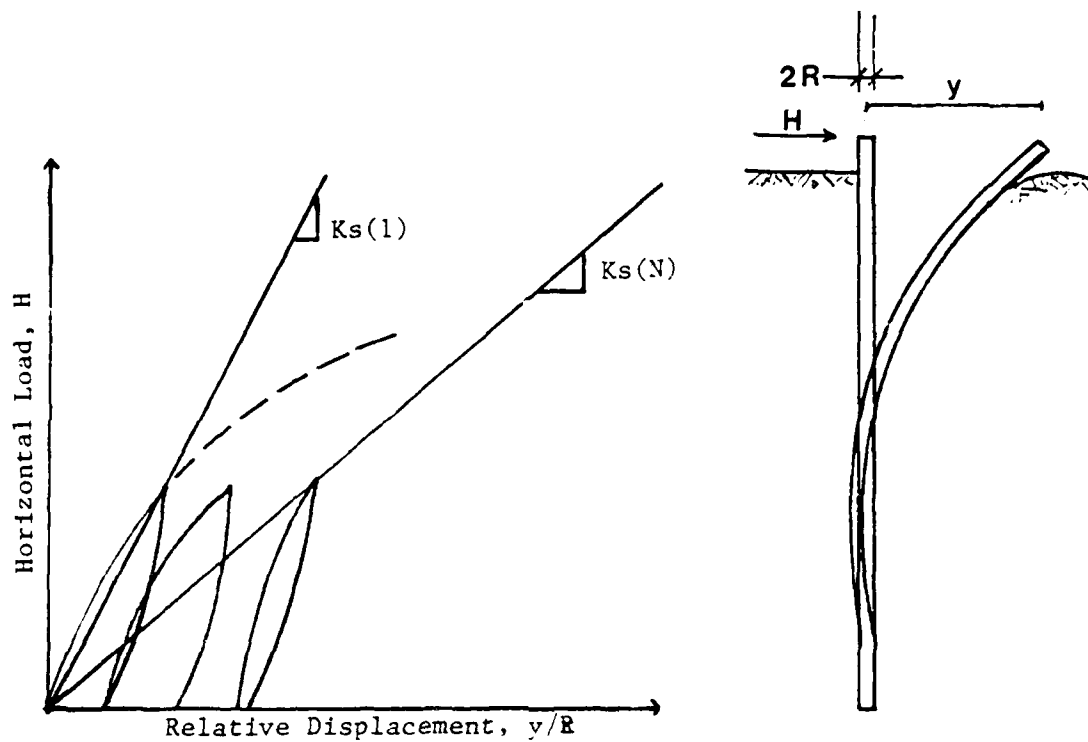
FIGURE 24. Cyclic Parameters Definition (After Makarim and Briaud, 1986).

39
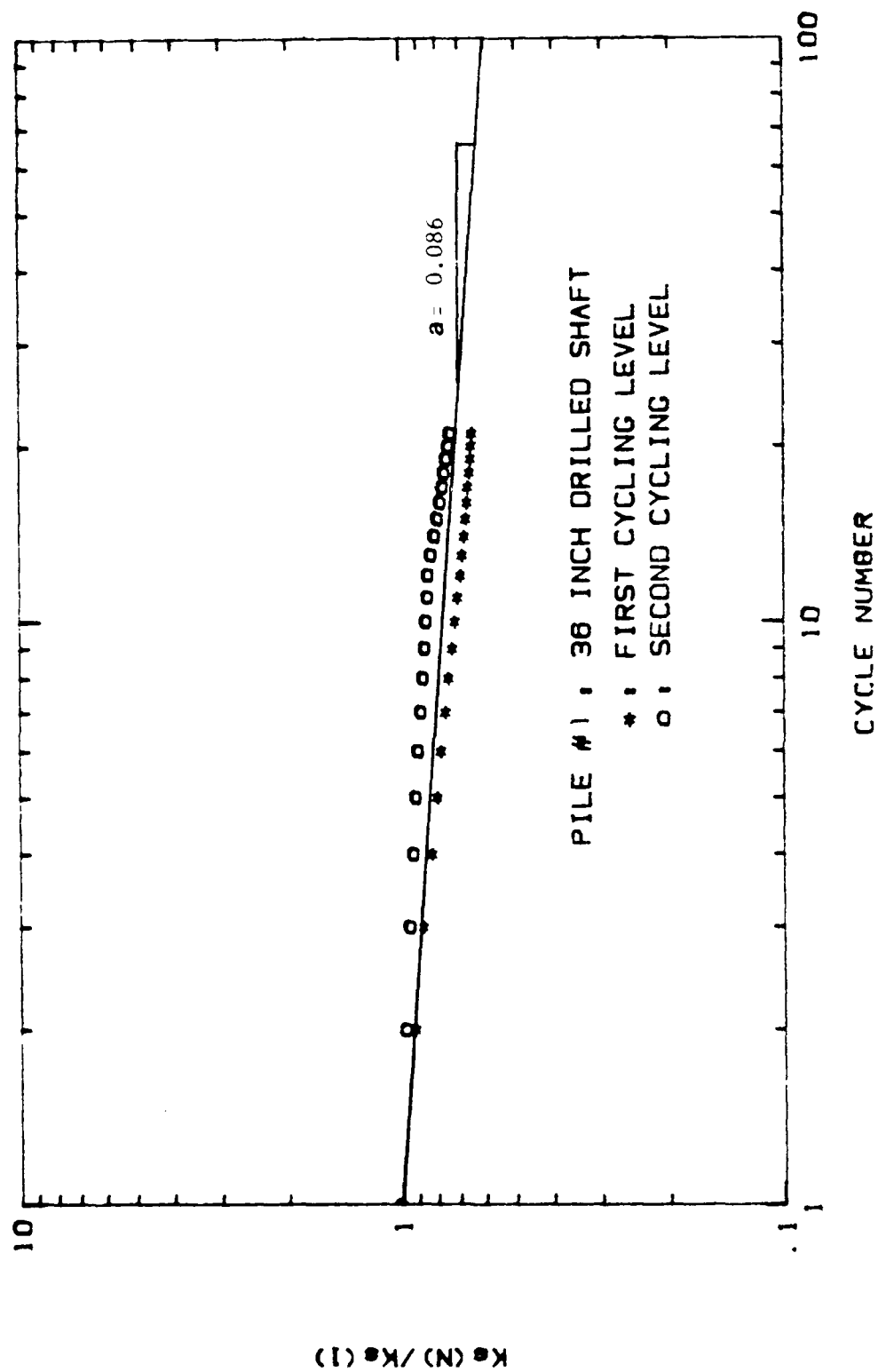
FIGURE 25. Measured Secant Shear Modulus Degradation for Pile No. 1.

40
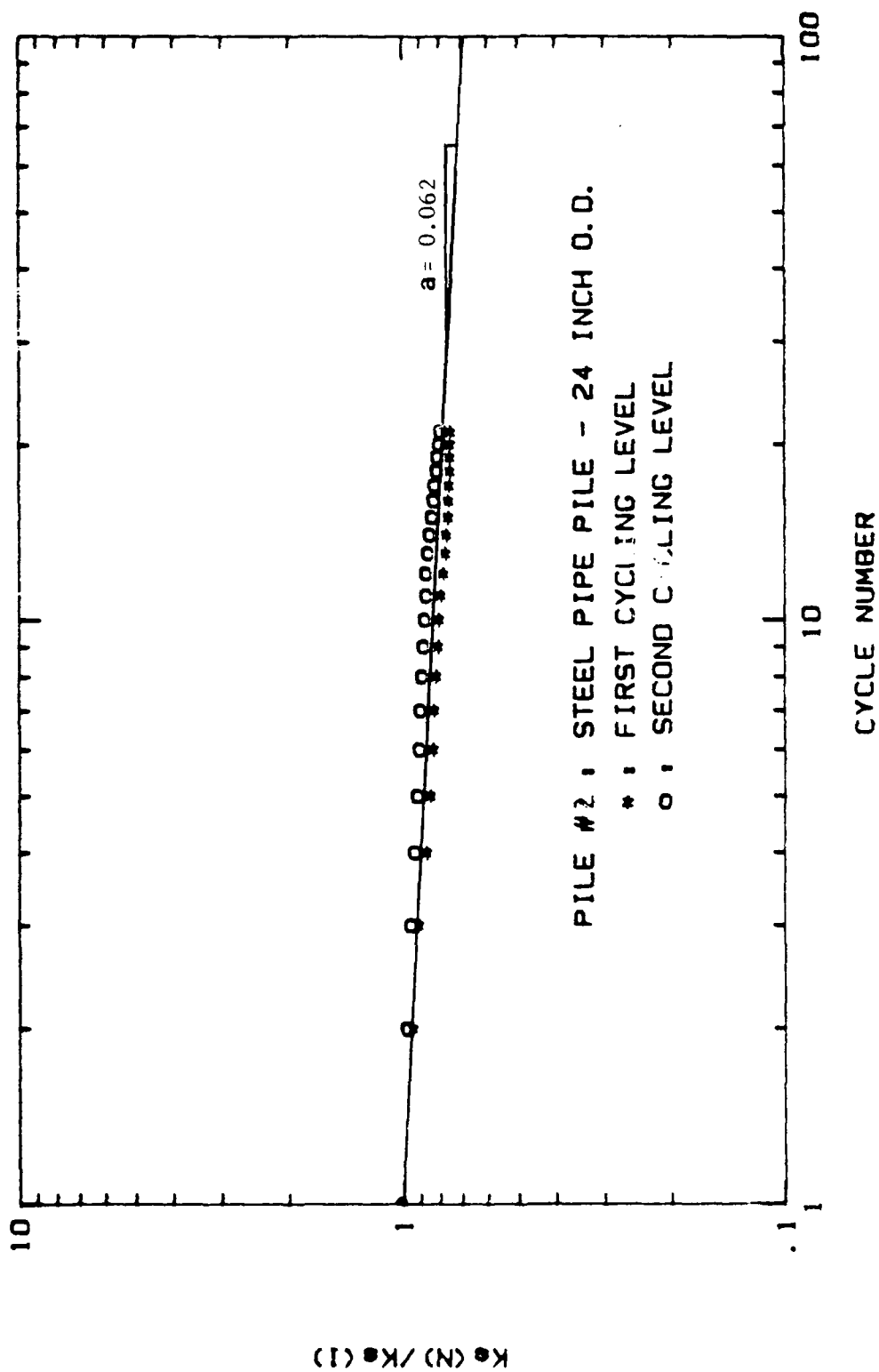
FIGURE 26. Measured Secant Shear Modulus Degradation for Pile No. 2.

41
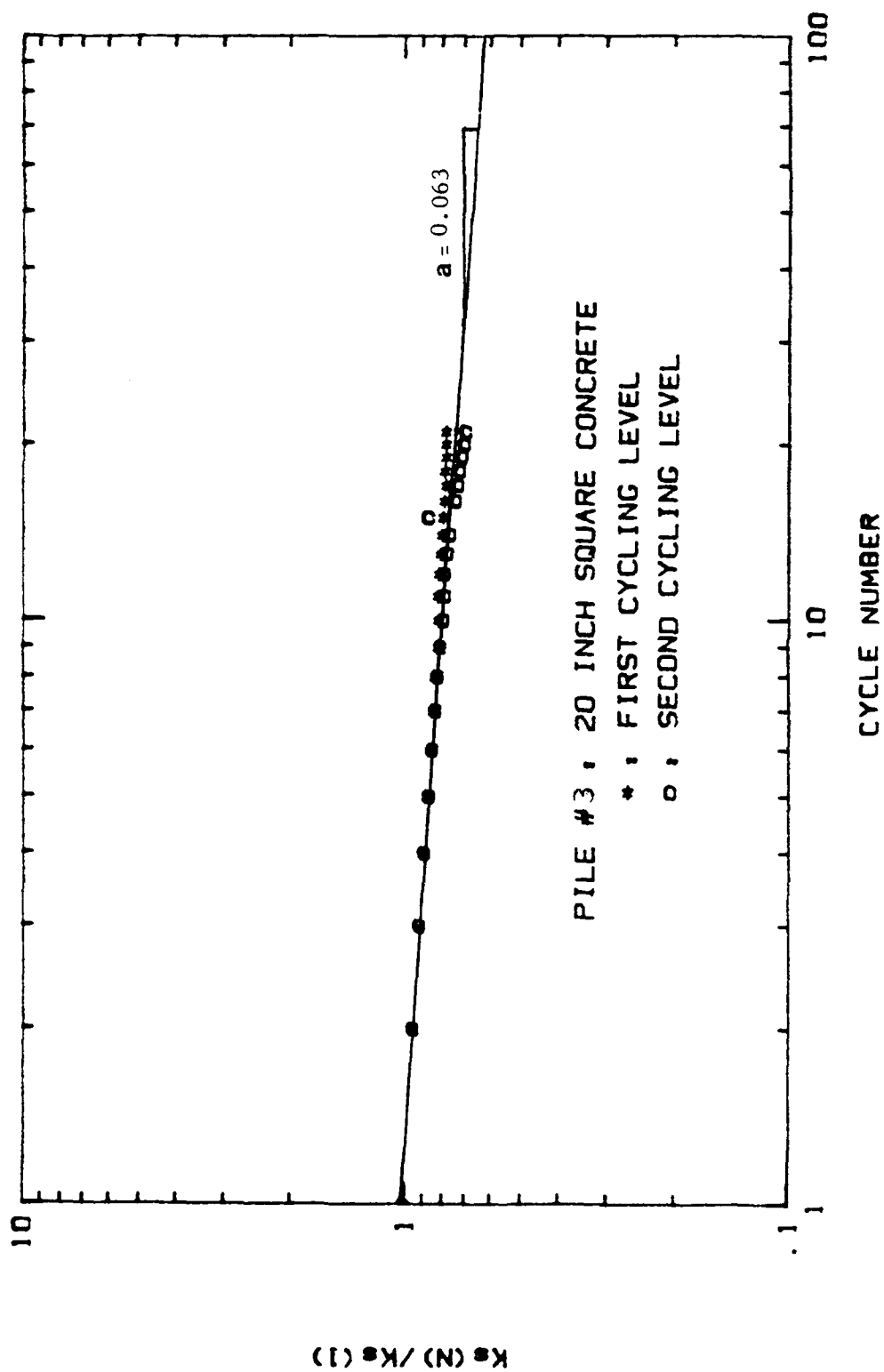
FIGURE 27. Measured Secant Shear Modulus Degradation for Pile No. 3.

42
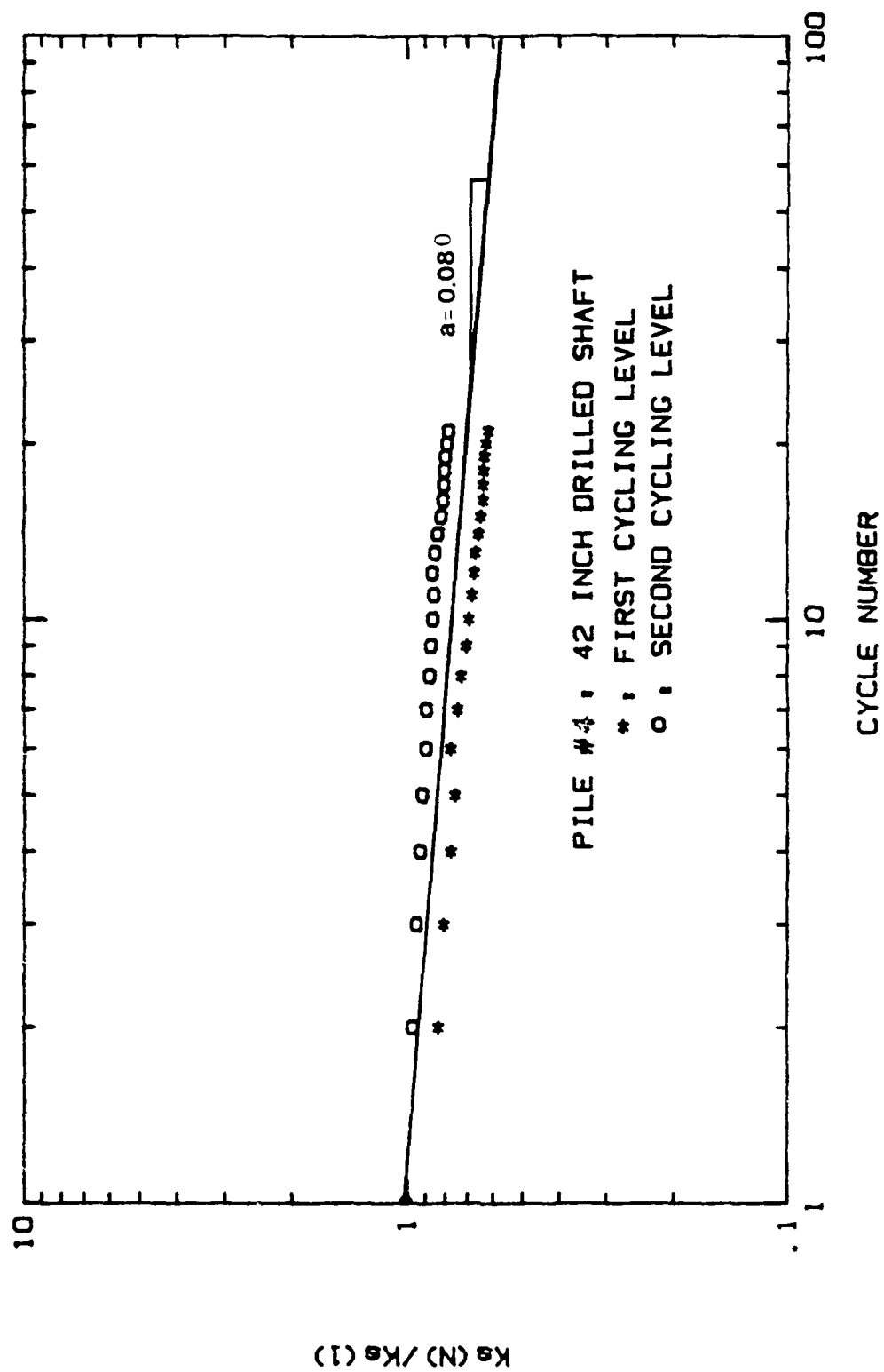
FIGURE 28. Measured Secant Shear Modulus Degradation for Pile No. 4.
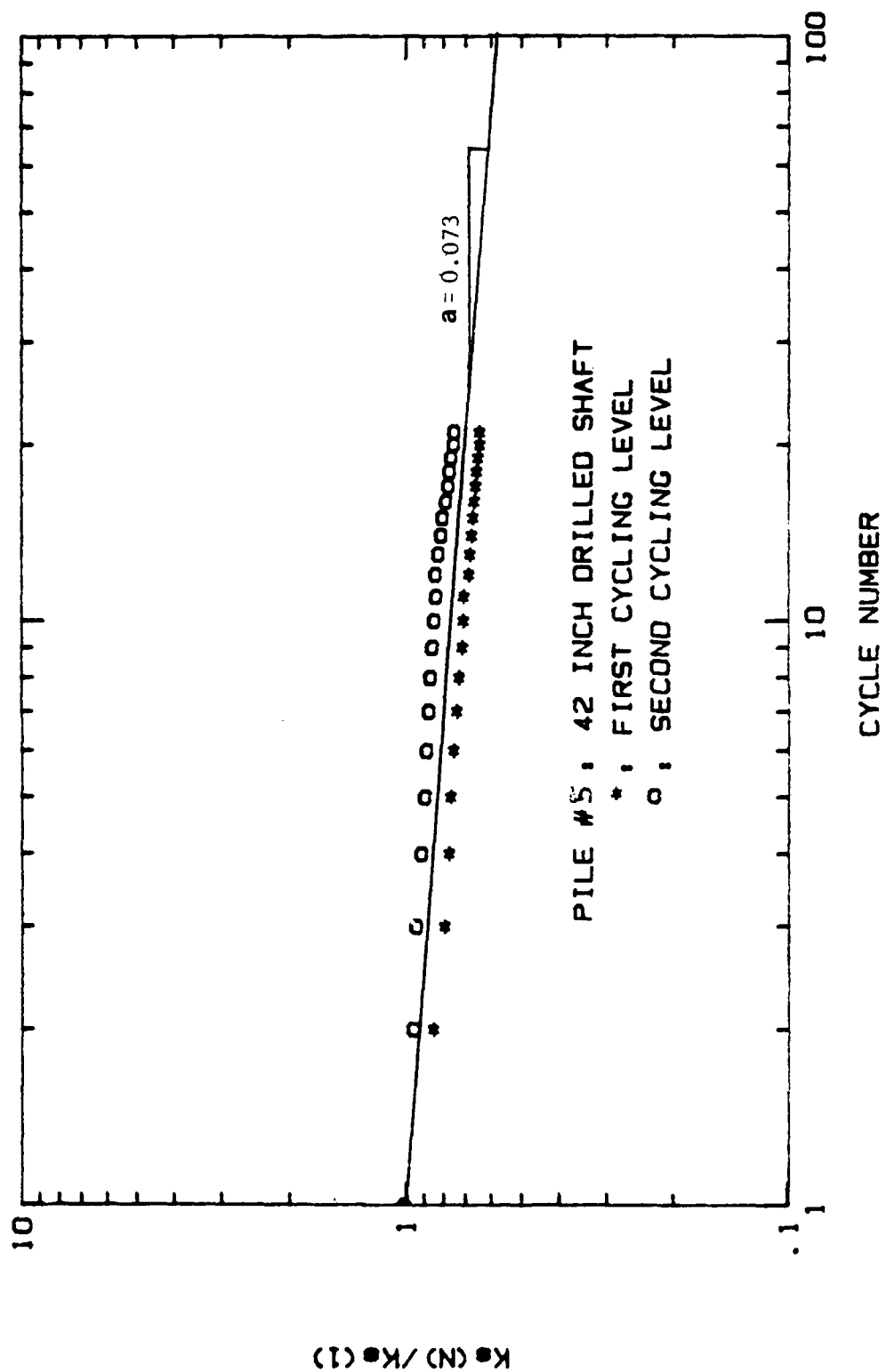
FIGURE 29. Measured Secant Shear Modulus Degradation for Pile No. 5.

44
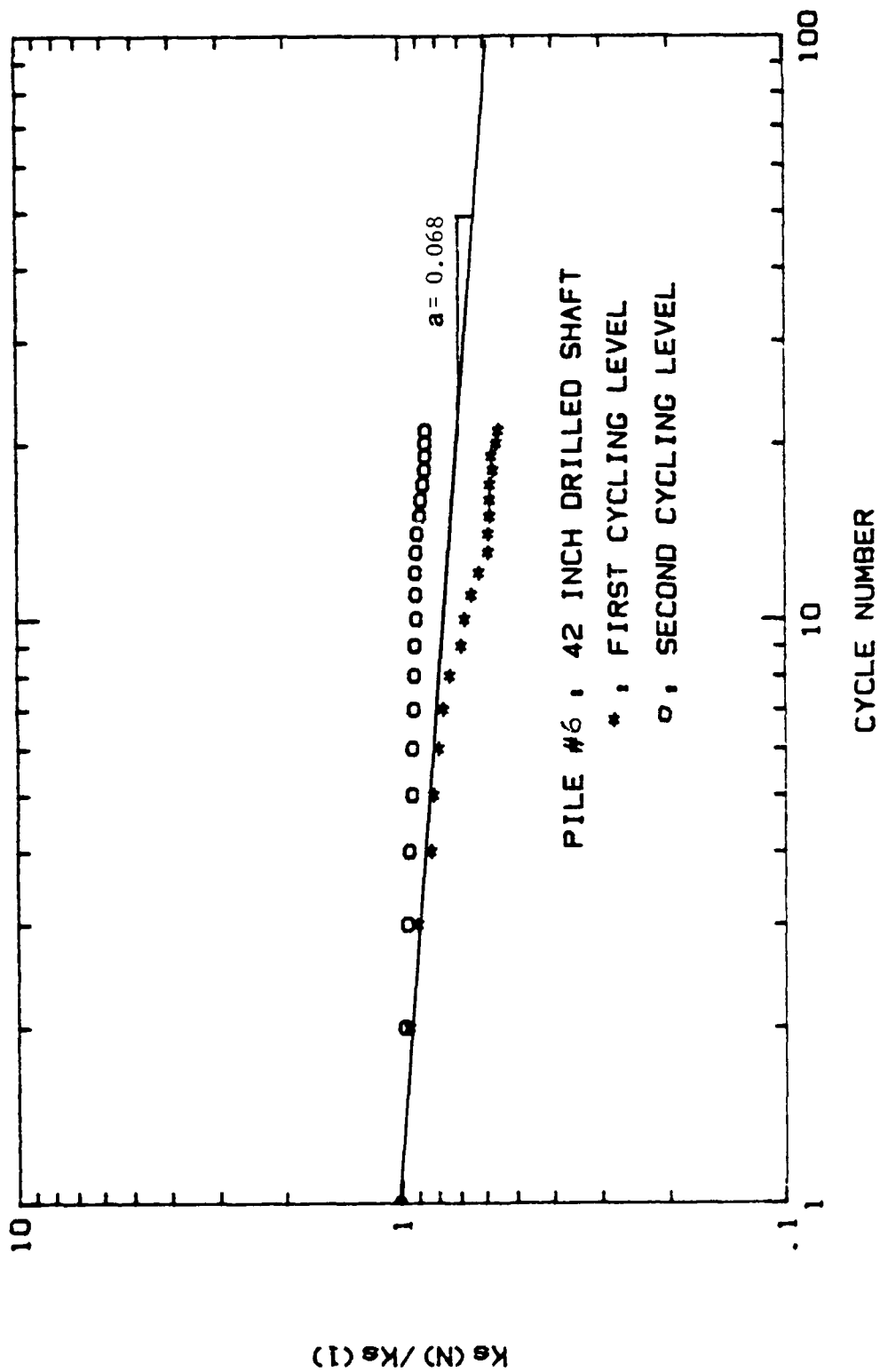
FIGURE 30. Measured Secant Shear Modulus Degradation for Pile No. 6.

45

When comparing the "a" values, it becomes clear that cycling with total unloading causes greater degradation than cycling with only partial unloading (one-half of the top load). This may be the result of greater inward yielding of the soil as the pile is passed through a greater range of displacements.

Also evident from the "a" values is that the first series of cycles were generally more damaging than the following series. This behavior was true for all except the 20-in square prestressed concrete pile. As discussed earlier, a probable reason for the greater degradation in the first cyclic series of the concrete drilled shafts is the decrease in the piles' flexural stiffness. The prestressing in the square concrete pile postponed the crack propagation until the second series of cycles. However, the steel pipe pile also had less degradation during the second series of cycles with "a" values of 0.065 and 0.059 respectively. Assuming that the flexural stiffness of the steel pipe pile itself suffered little or no degradation, it would appear that the soil stiffness also suffers less degradation at the higher load level. The first series of cycles may have caused a slight densification of the soil in front of the pile.

The cyclic stiffness for the pile $K_{C(N)}$, as defined in Figure 31, showed little or no degradation within each portion of the cyclic loading where the difference between the upper and lower loads was constant (Figures 32 through 37). The cyclic stiffnesses were stiffer for cycling where the magnitude of the difference was one-half of the upper load level. This is to be expected from a non-elastic medium.

4.4.3 Creep response

Taking readings every minute for five minutes after reaching a specified load allowed observation of the creep response of the piles. These responses are presented in
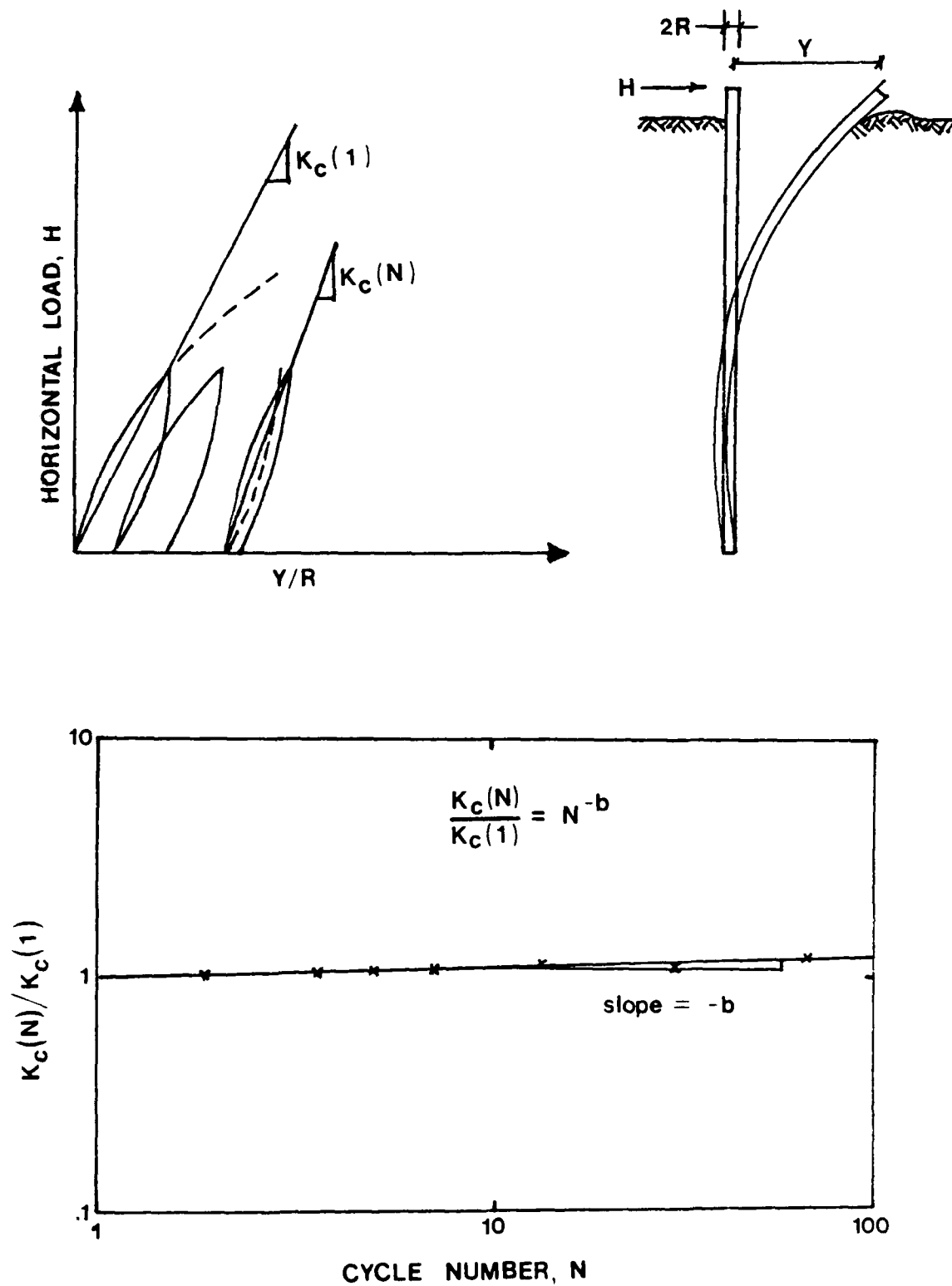
FIGURE 31. Cyclic Shear Modulus Parameters Definition.

47
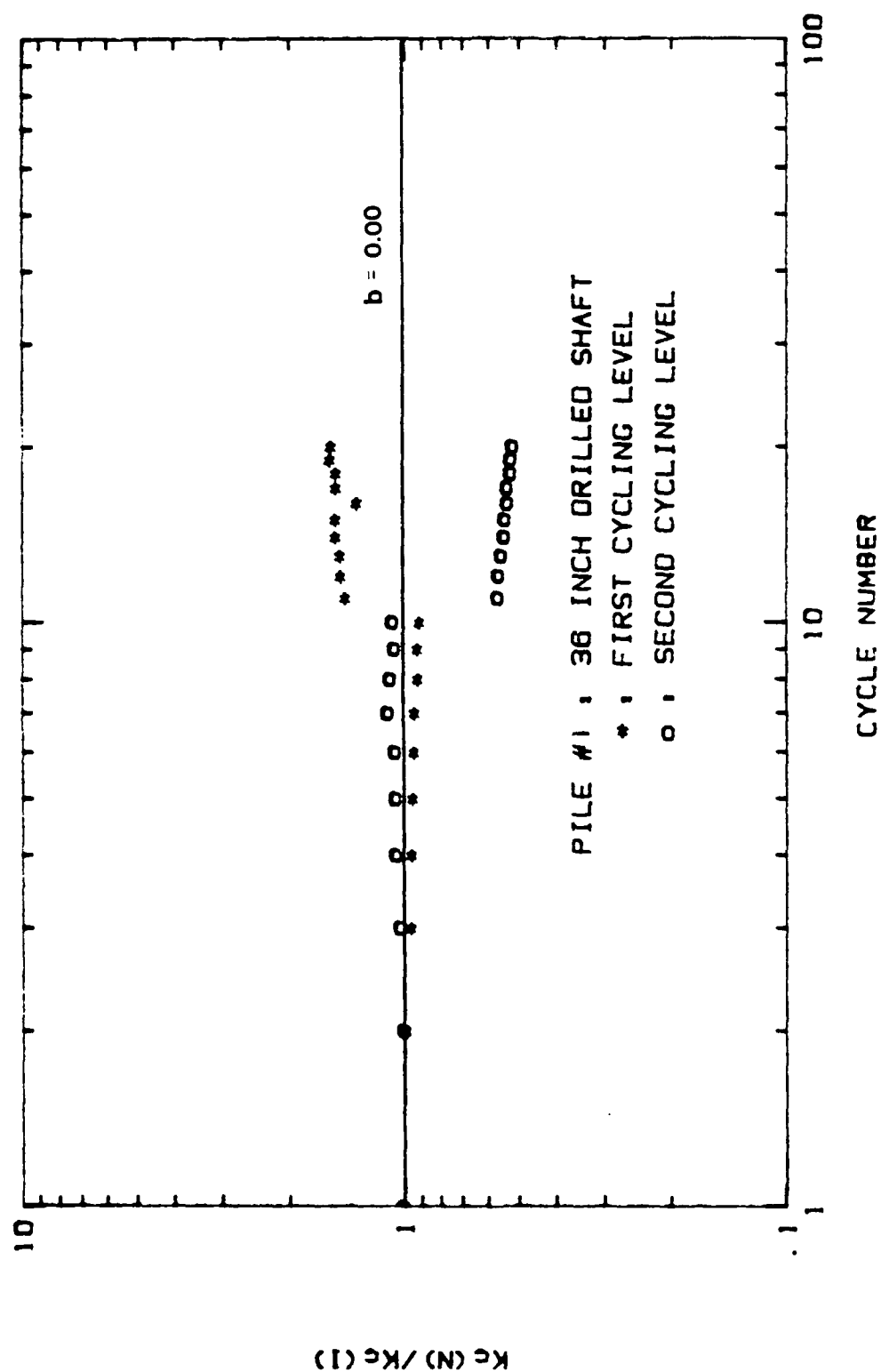
FIGURE 32. Measured Cyclic Shear Modulus Degradation for Pile No. 1.
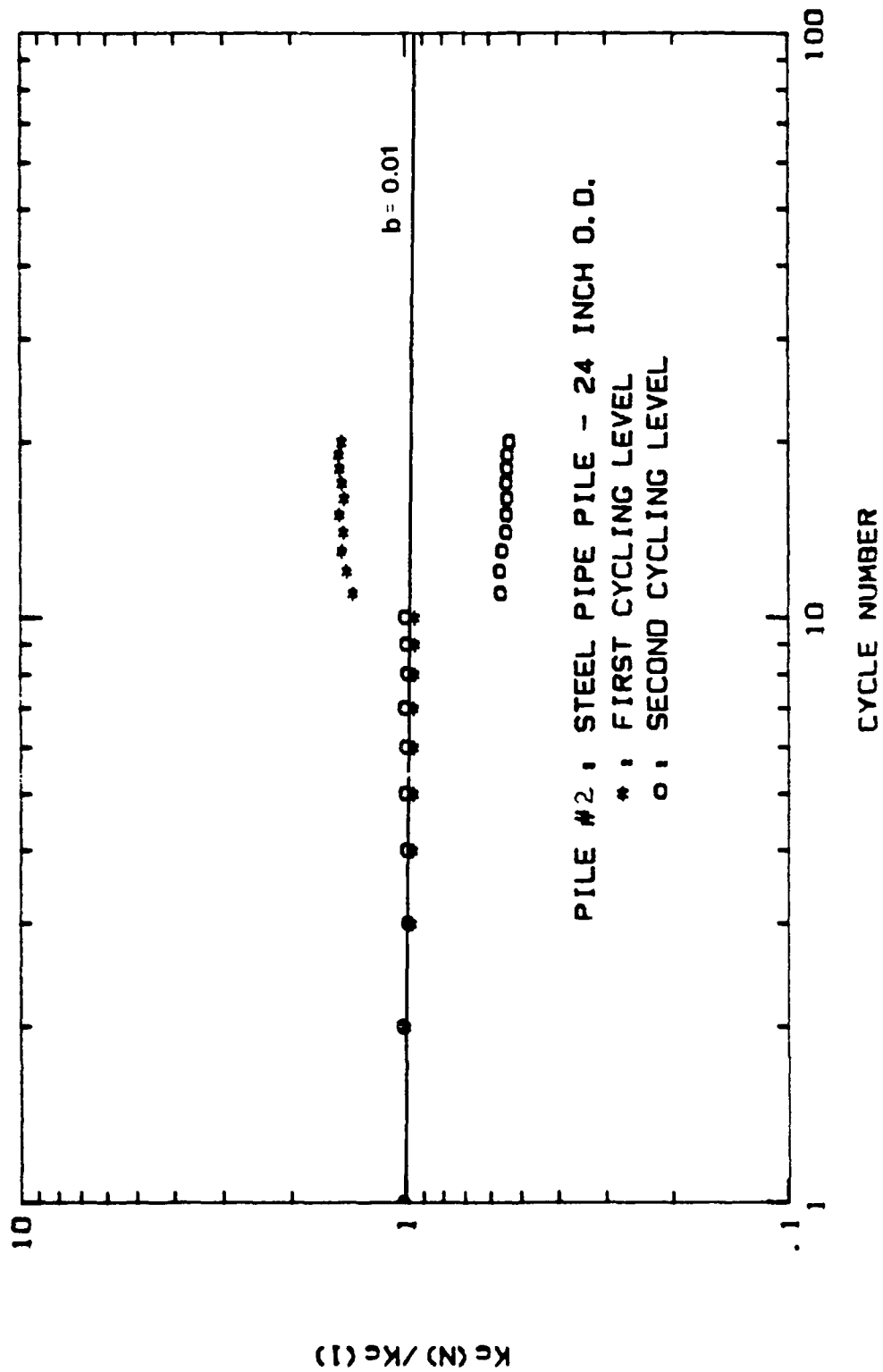
FIGURE 33. Measured Cyclic Shear Modulus Degradation for Pile No. 2.
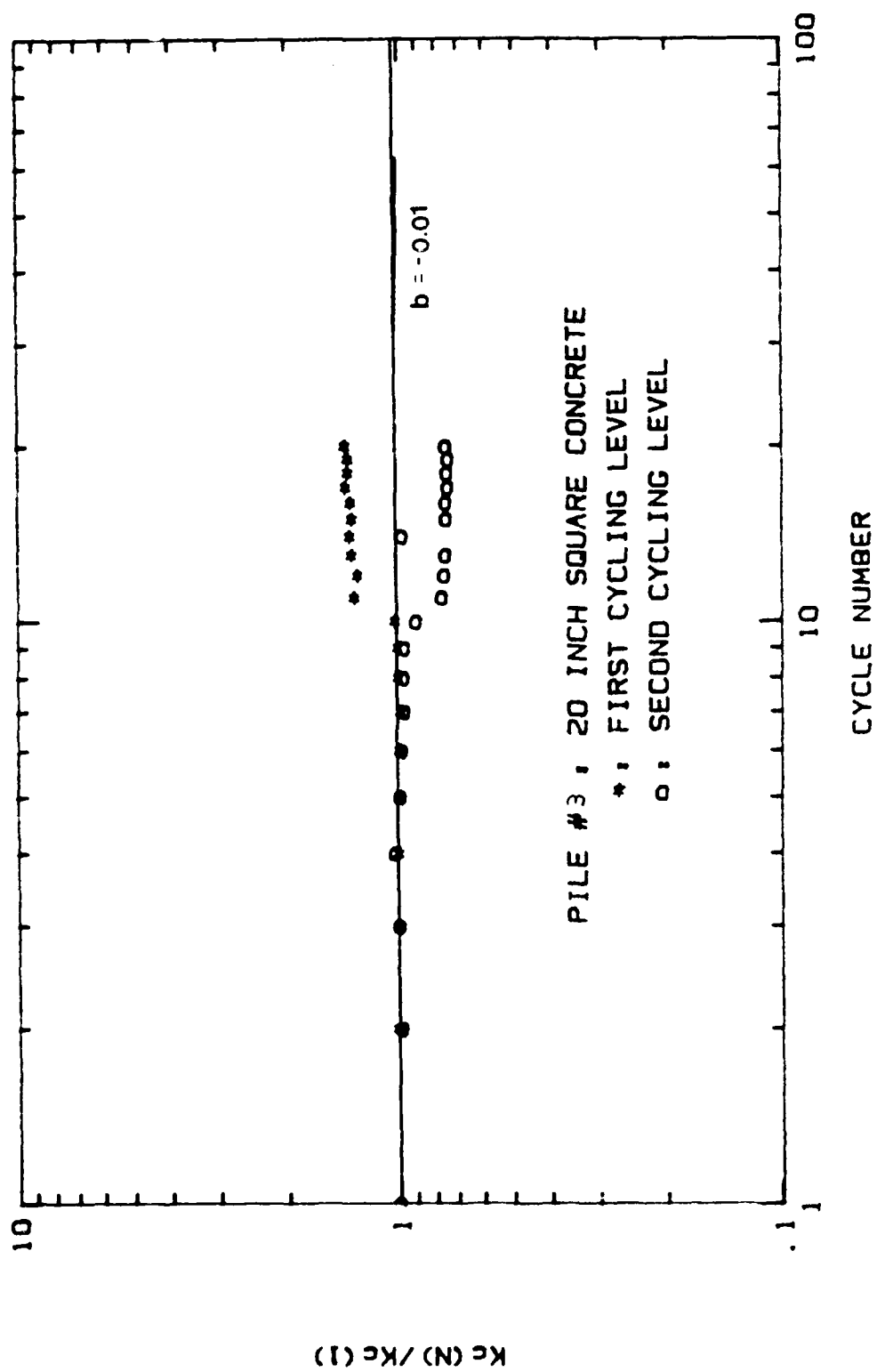
FIGURE 34. Measured Cyclic Shear Modulus Degradation for Pile No. 3.

50
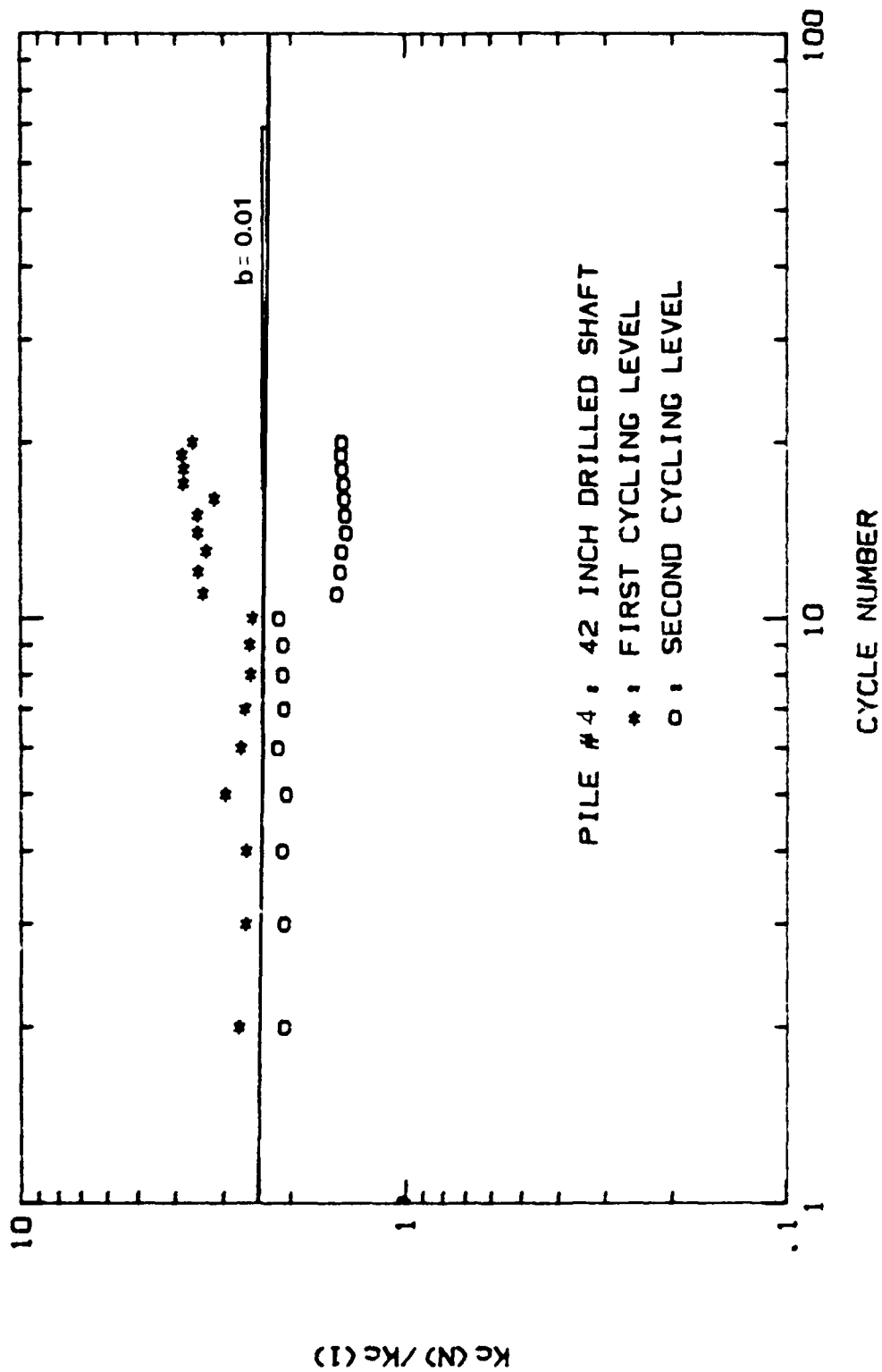
FIGURE 35. Measured Cyclic Shear Modulus Degradation for Pile No. 4.

51
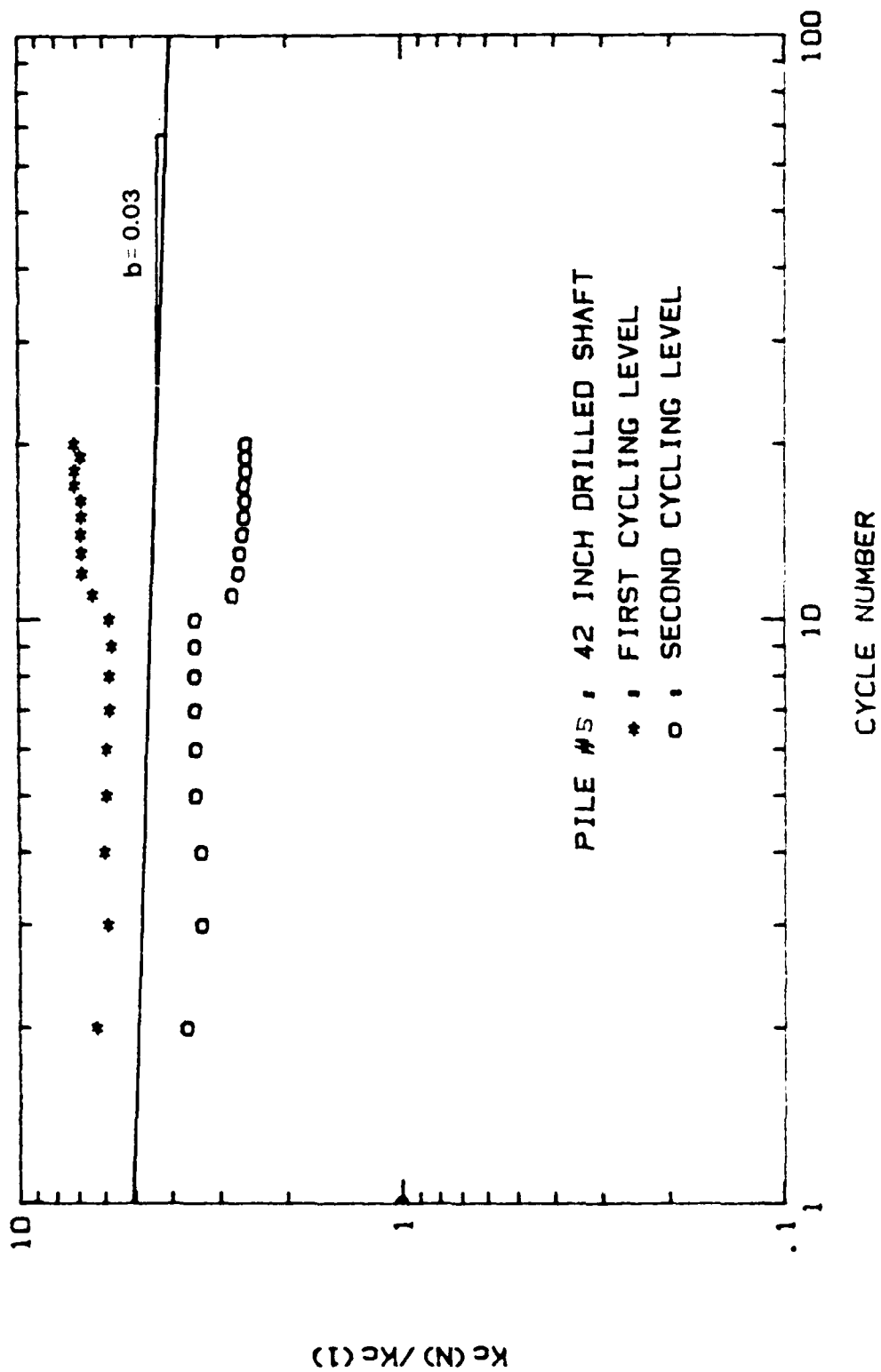
FIGURE 36. Measured Cyclic Shear Modulus Degradation for Pile No. 5.
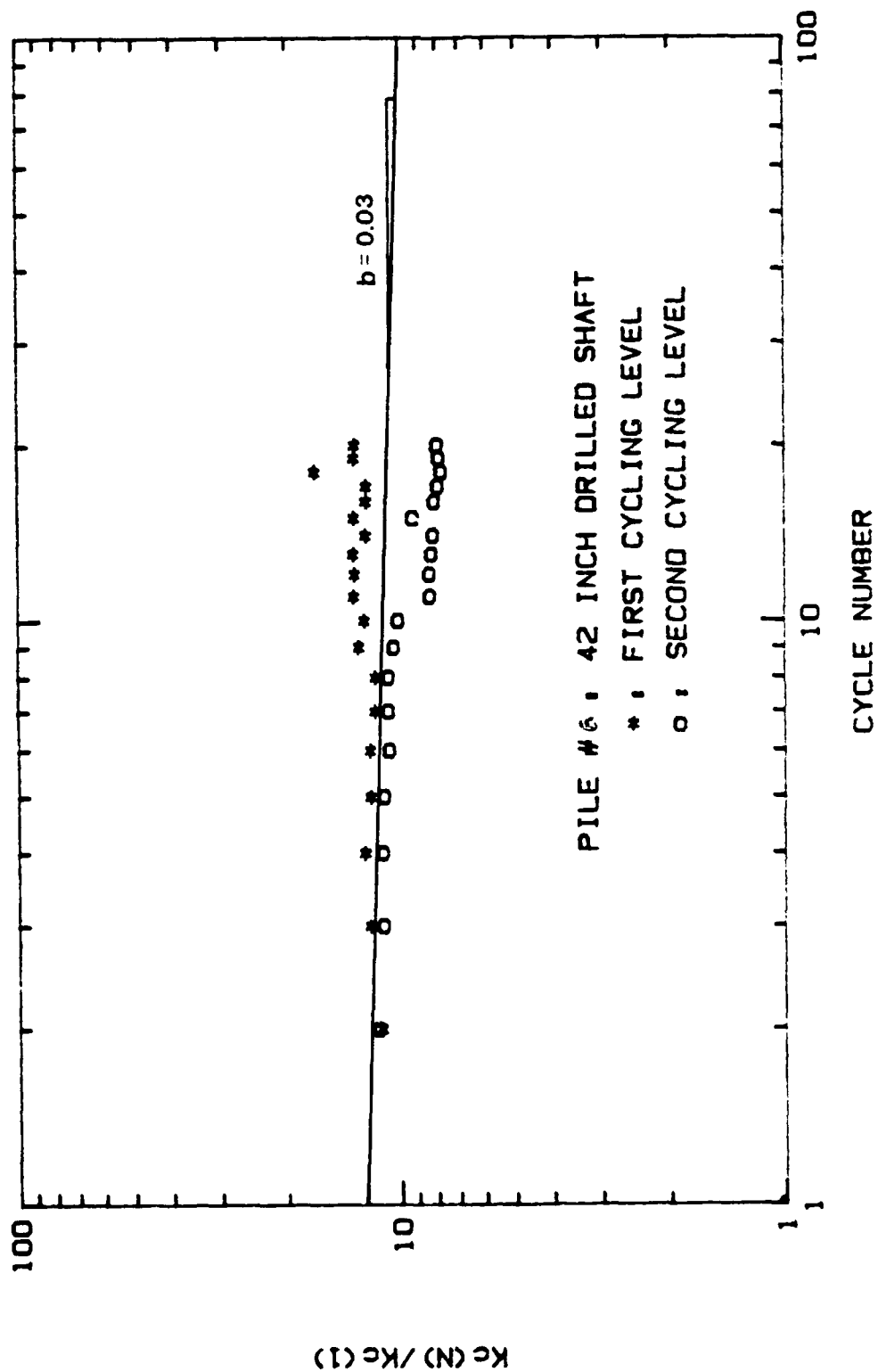
FIGURE 37. Measured Cyclic Shear Modulus Degradation for Pile No. 6.

Figures 38 through 43. These figures show the values S_n/S_o, the displacement at time T_n divided by the displacement at time T_o when the load was initially applied, plotted as a function of the values T_n/T_o on a log-log scale. The slope of each line may then be defined as the creep exponent n:

$$\frac{S_n}{S_o} = \left(\frac{T_n}{T_o}\right)^n \tag{1}$$

Values of n are plotted against the lateral load in Figures 44 through 49. From these figures it can be seen that the creep exponent for two of the 42-in diameter drilled shafts (Piles 4 and 5) dropped from an initially high value to a fairly stable value of about 0.02. The creep exponent for the third 42-in diameter drilled shaft (Pile 6) dropped similarly at first down to the 0.02 level, but then began to climb as the test progressed. The initial high creep may not only be a reflection of initial soil creep, but also the creep associated with crack propagation in the con.rete piles. The stabilization of the n value aroung 0.02 indicates tnat the cracking had stabilized. The upward turn in the n values for pile 6 is indicative of the impending pile failure at 90 kips.

The 36-in diameter drilled shaft behaved similarly to the 42-in diameter drilled shafts, with the values of n dropping initially and stabilizing around 0.015. The steel pipe pile and square prestressed concrete pile had much lower initial n values. This is consistent with the theory that the high initial creep for the drilled shafts relects creep associated with crack propagation in the concrete.

The prestressed concrete pile reached a critical creep load at 90 kips. At this sustained load the increase in deflection began to rise rapidly (Figure 46).
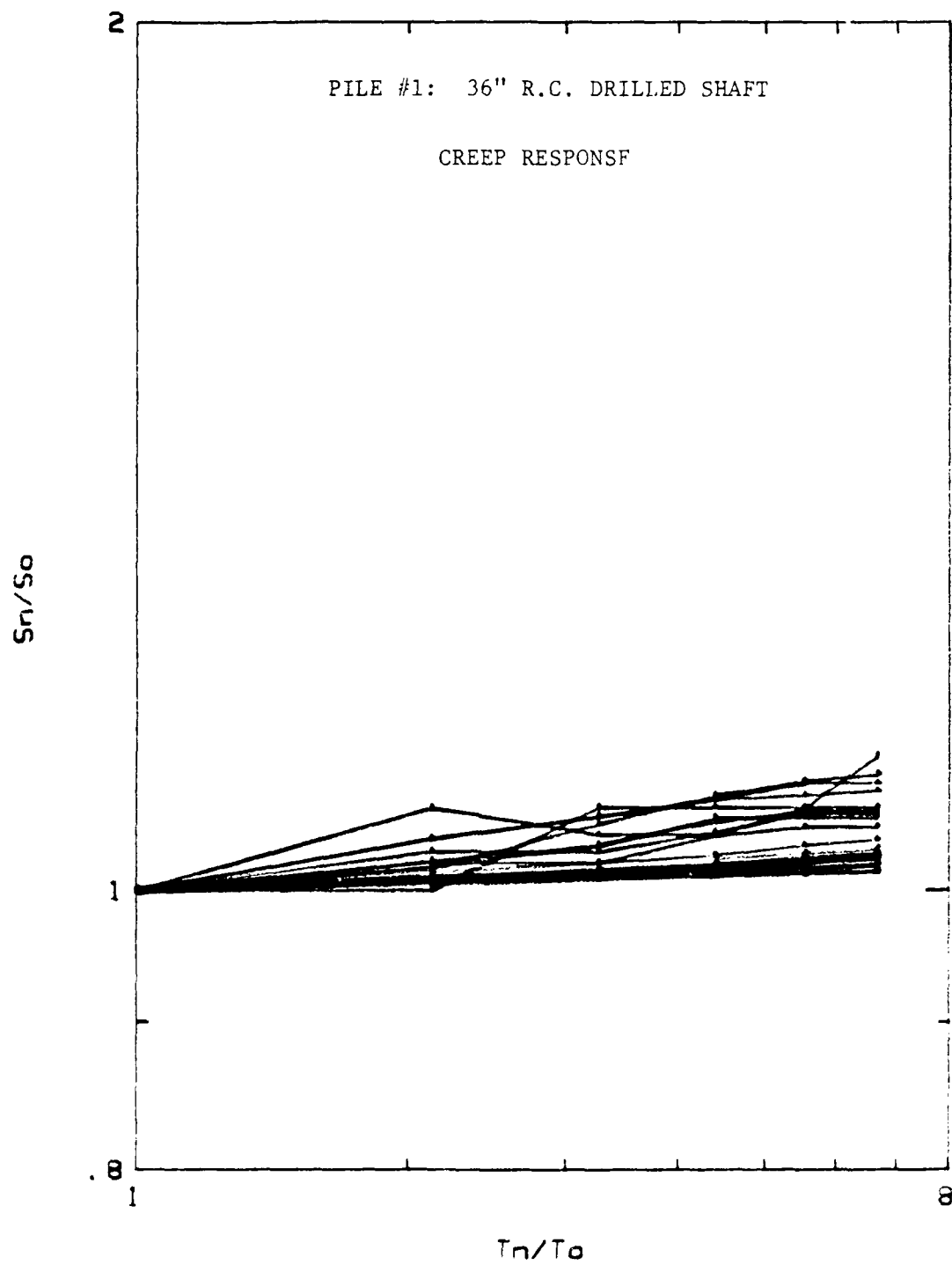
FIGURE 38. Measured Creep Response, Pile No. 1.

55
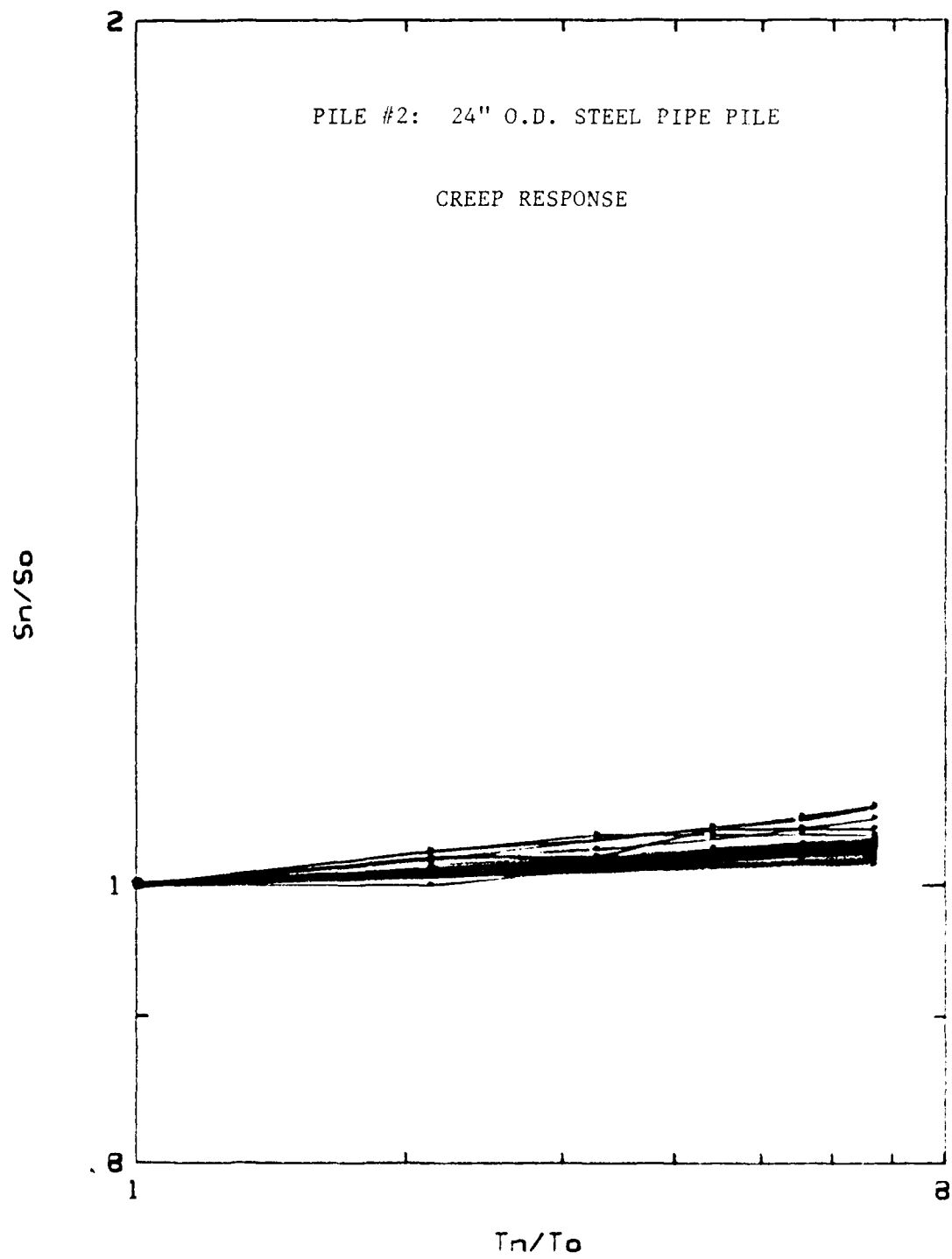
FIGURE 39. Measured Creep Response, Pile No. 2.
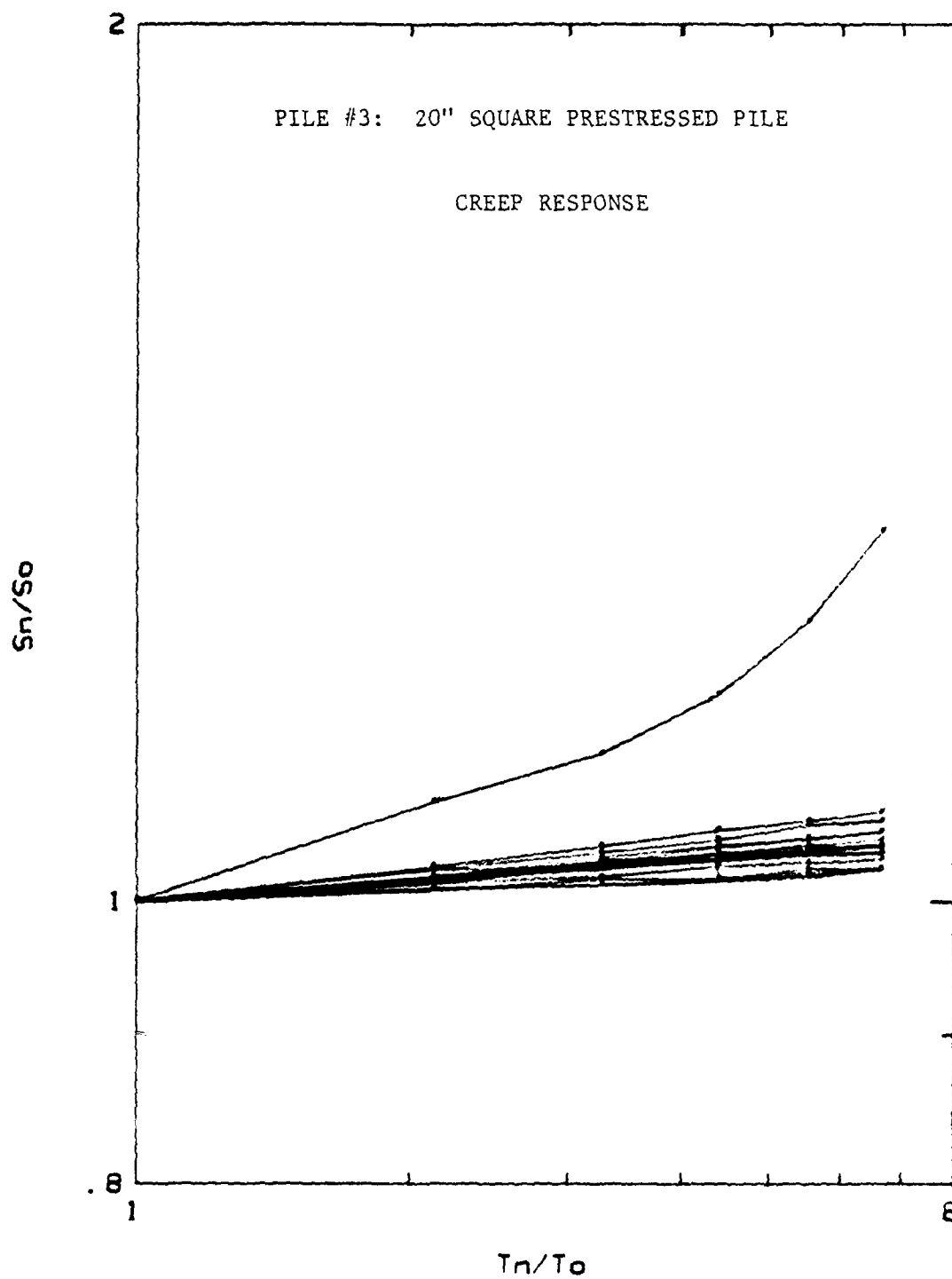
FIGURE 40. Measured Creep Response, Pile No. 3.

57
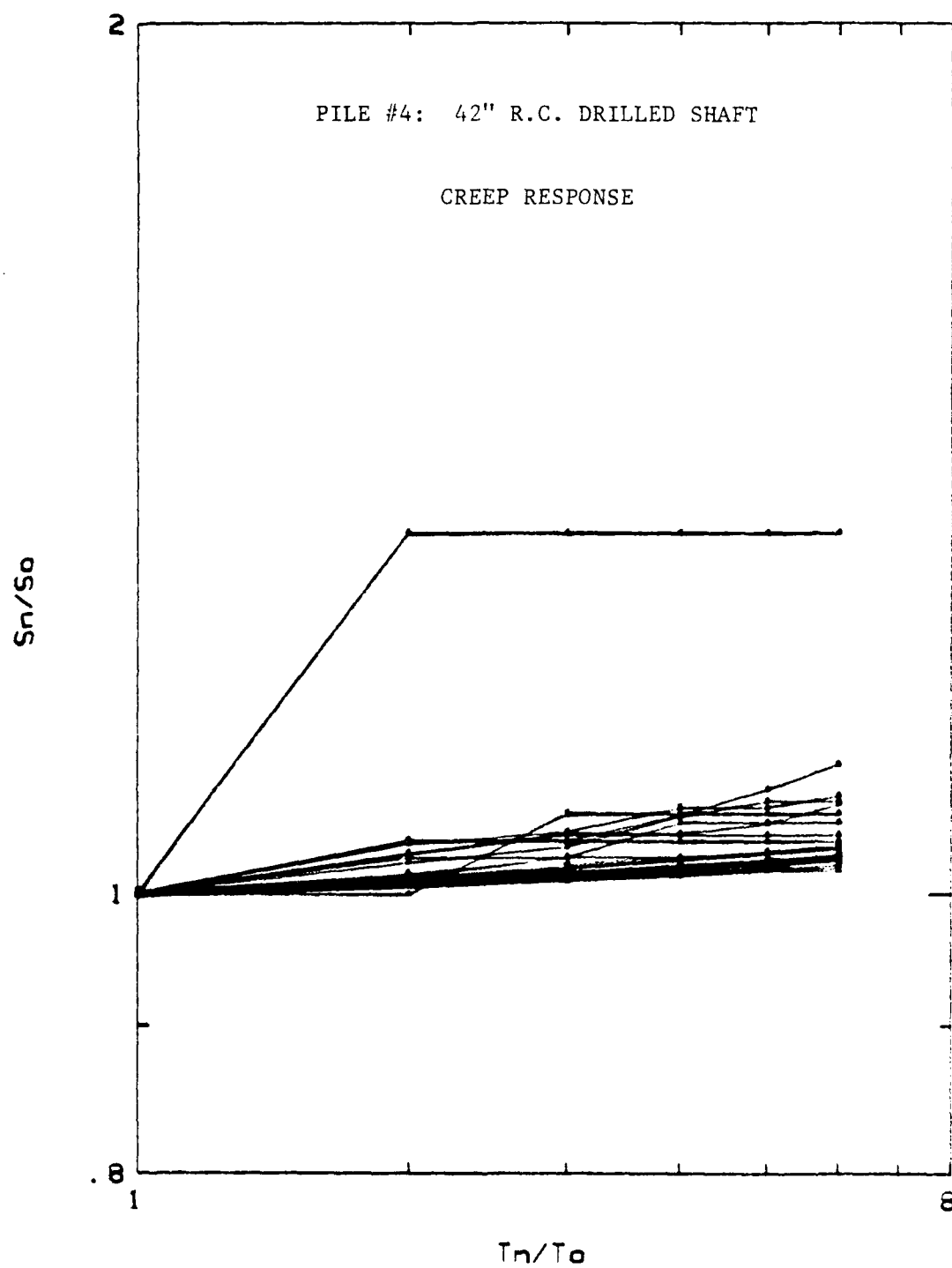
FIGURE 41.   Measured Creep Response, Pile No. 4.
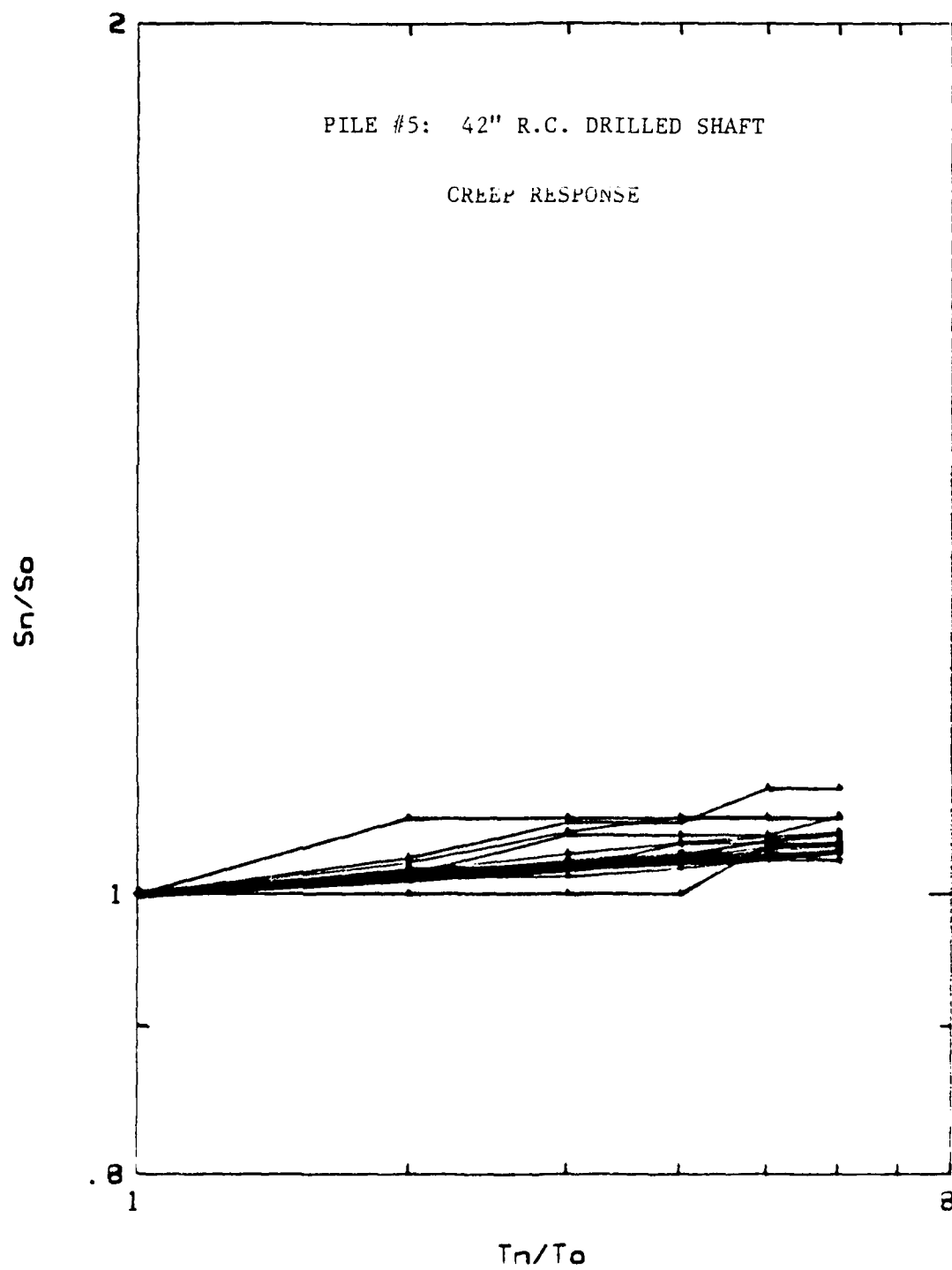
FIGURE 42. Measured Creep Response, Pile No. 5
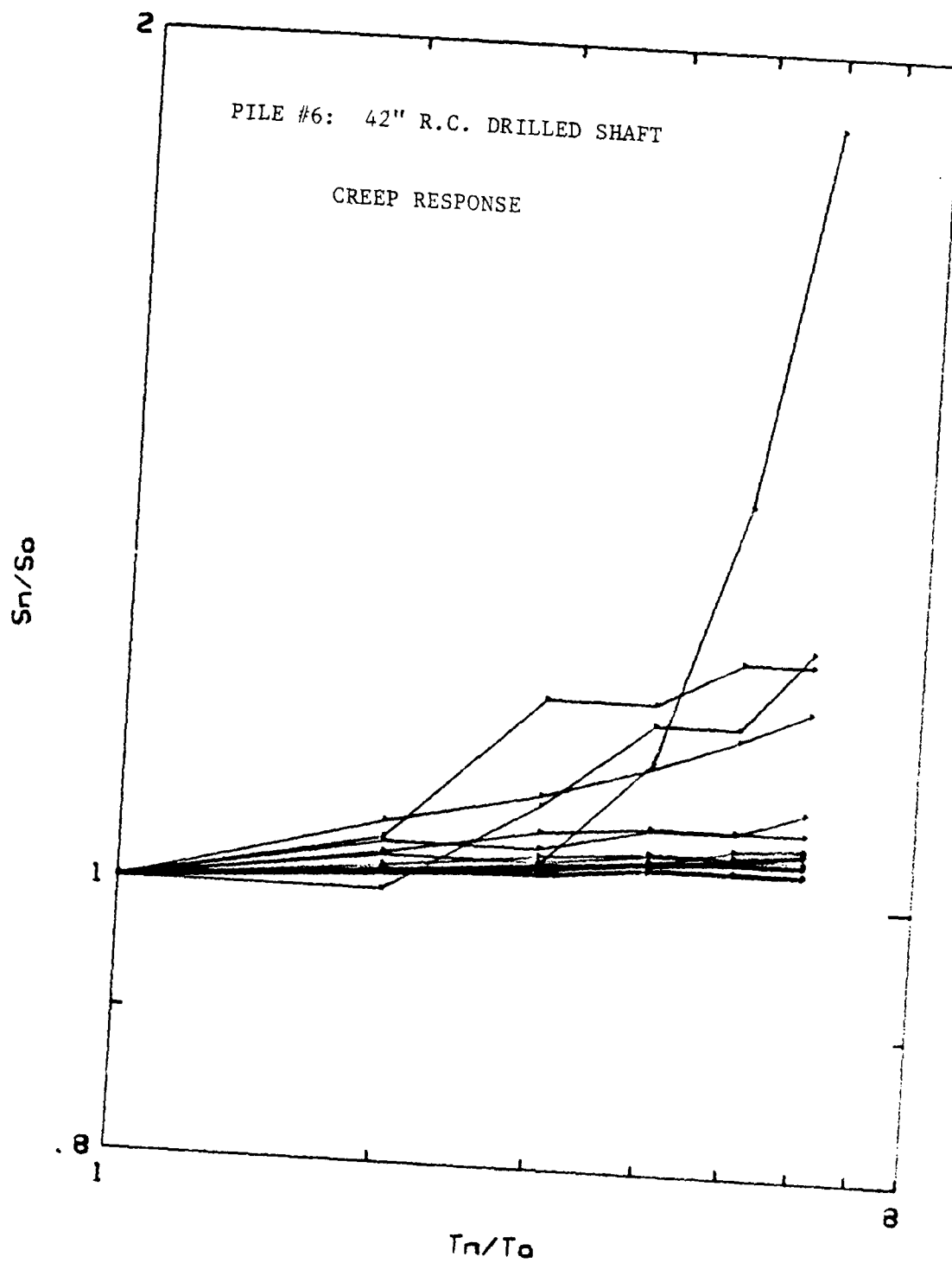
FIGURE 43. Measure Creep Response, Pile No. 6.

60

PILE #1: 36" R.C. DRILLED SHAFT

CREEP EXPONENT RESPONSE
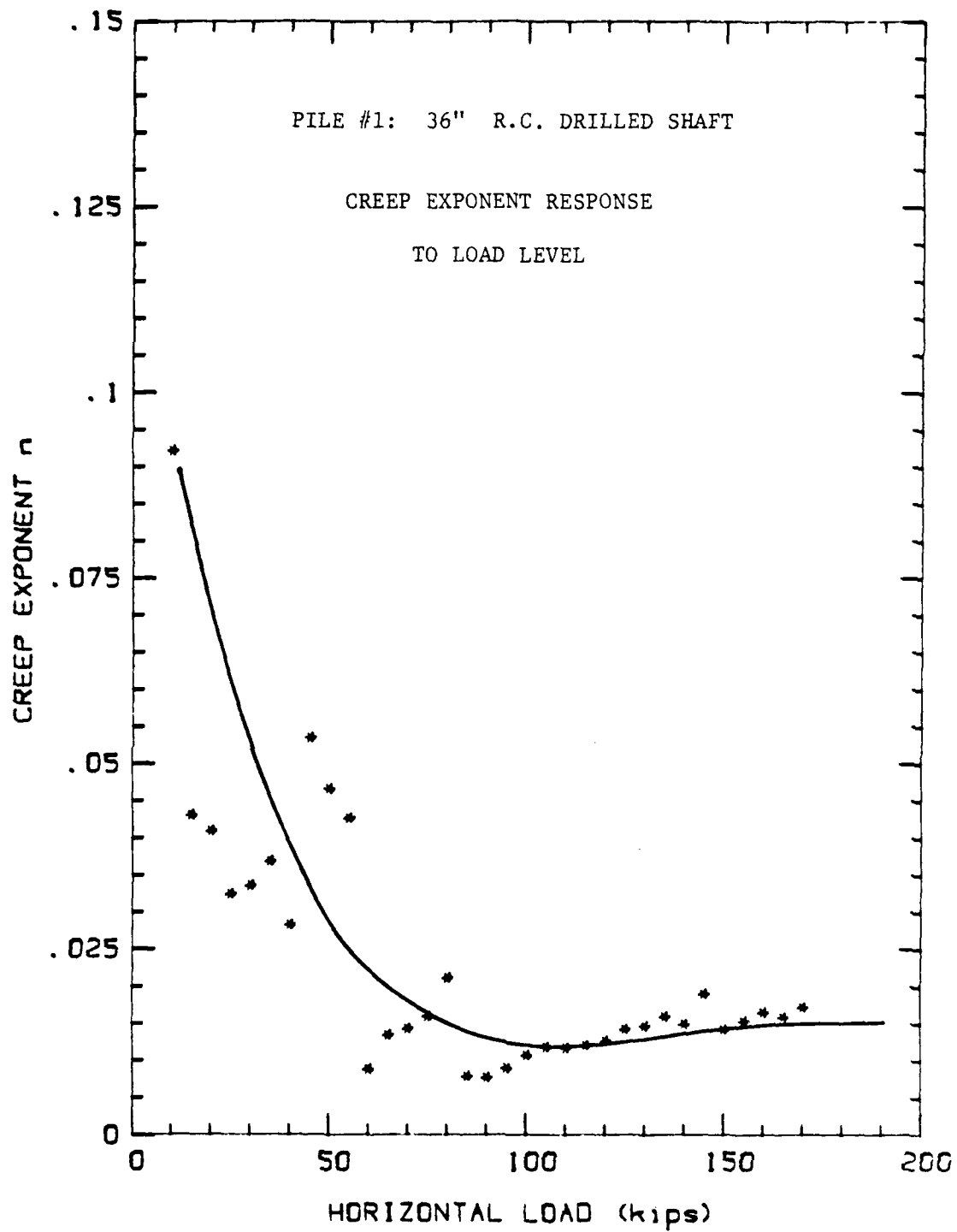
TO LOAD LEVEL

FIGURE 44. Creep Exponent Response to Load
Level, Pile No. 1.

61
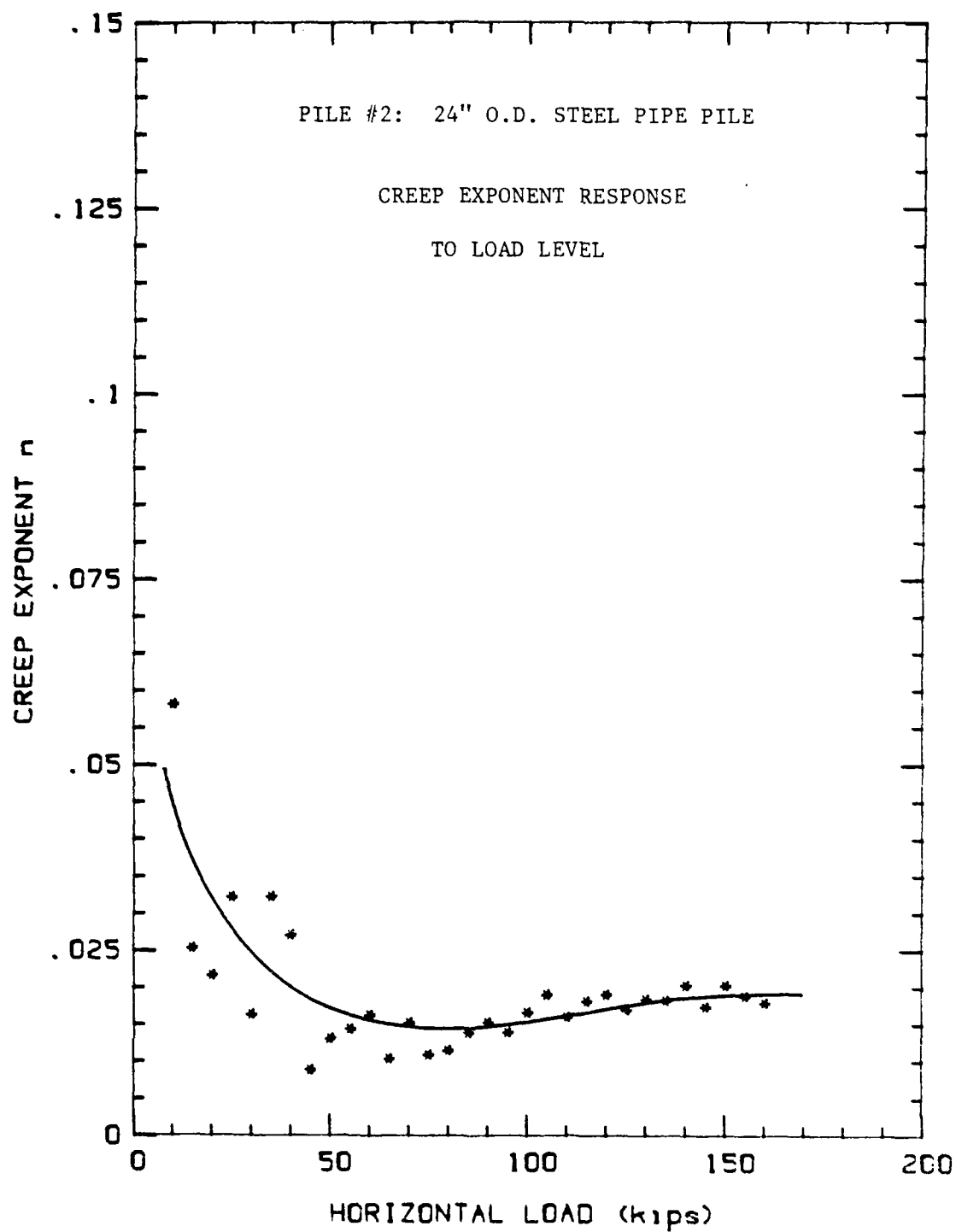
FIGURE 45.   Creep Exponent Response to Load Level,
            Pile No. 2.

62
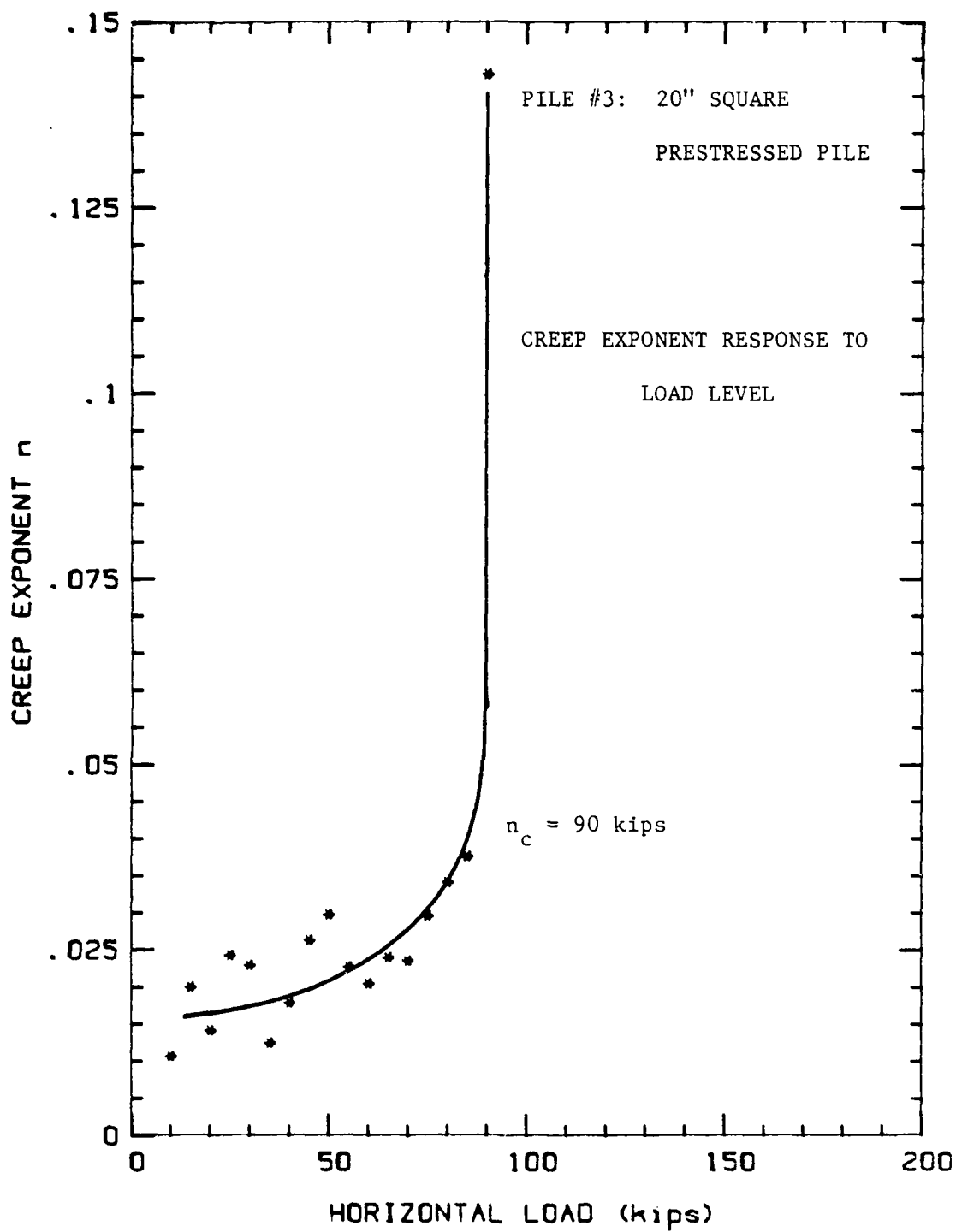
FIGURE 46.   Creep Exponent Response to Load Level,
             Pile No. 3.

63
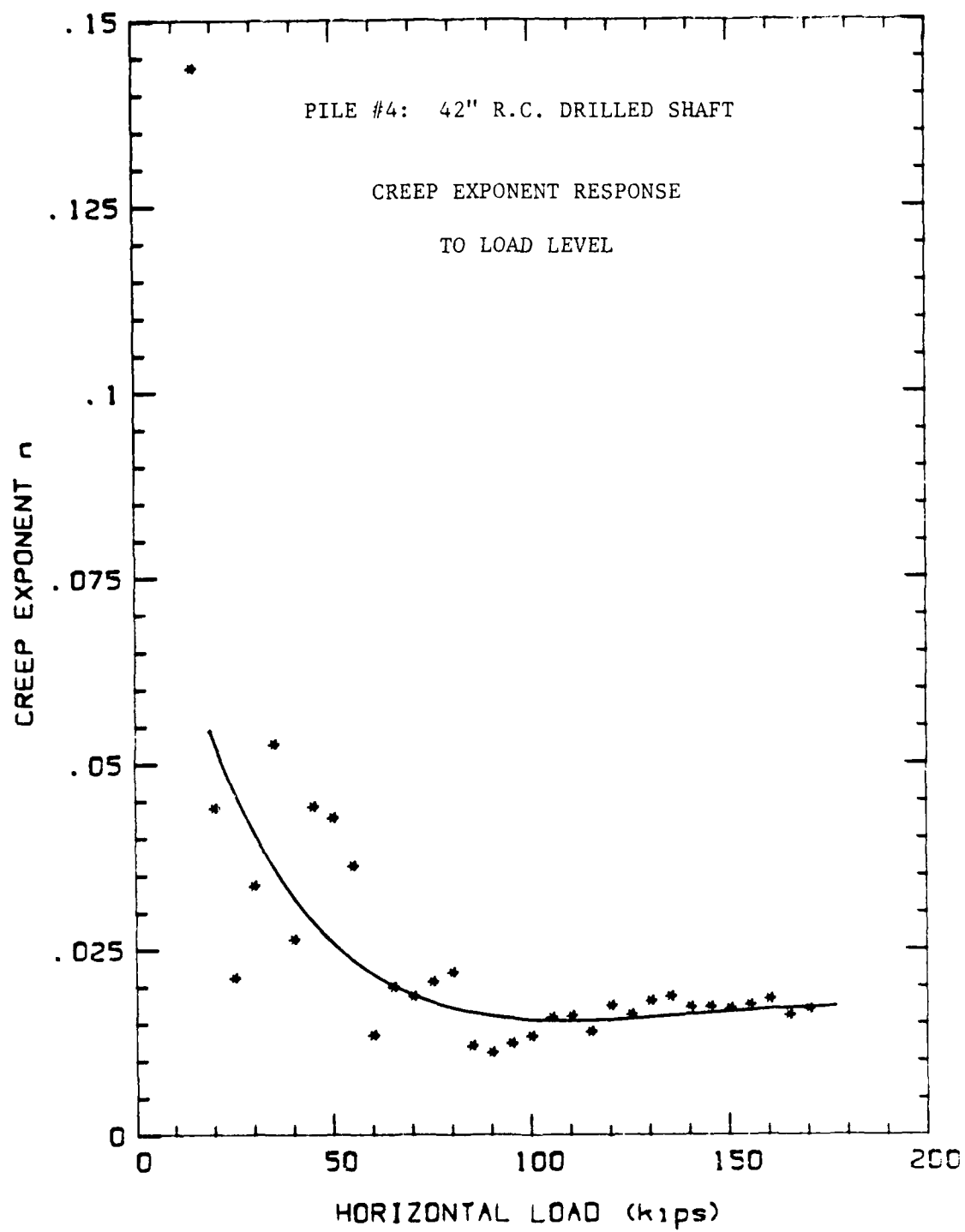
FIGURE 47. Creep Exponent Response to Load Level,
Pile No. 4.

64

PILE #5: 42" R.C. DRILLED SHAFT

CREEP EXPONENT RESPONSE
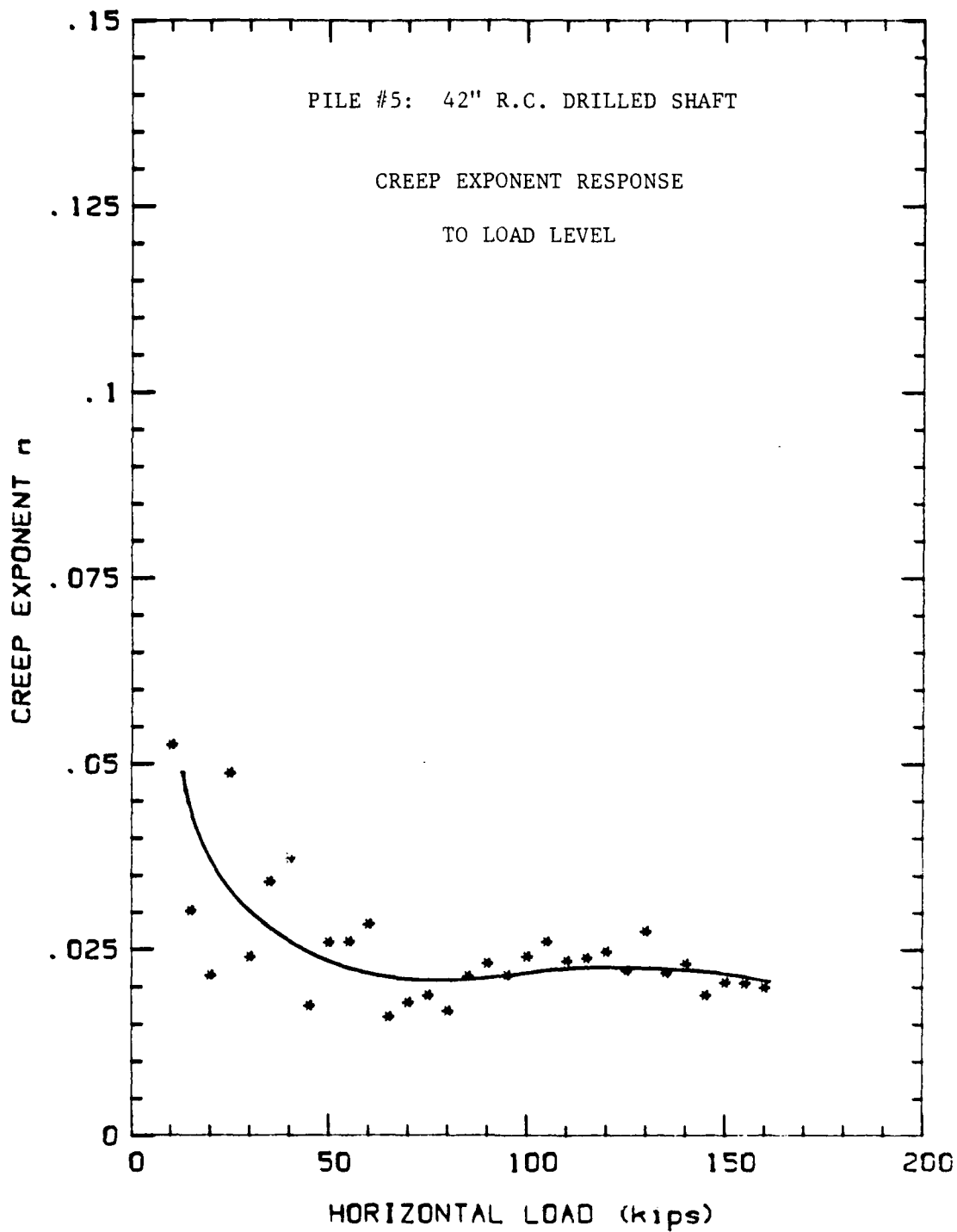
TO LOAD LEVEL

FIGURE 48. Creep Exponent Response to Load Level,
Pile No. 5.
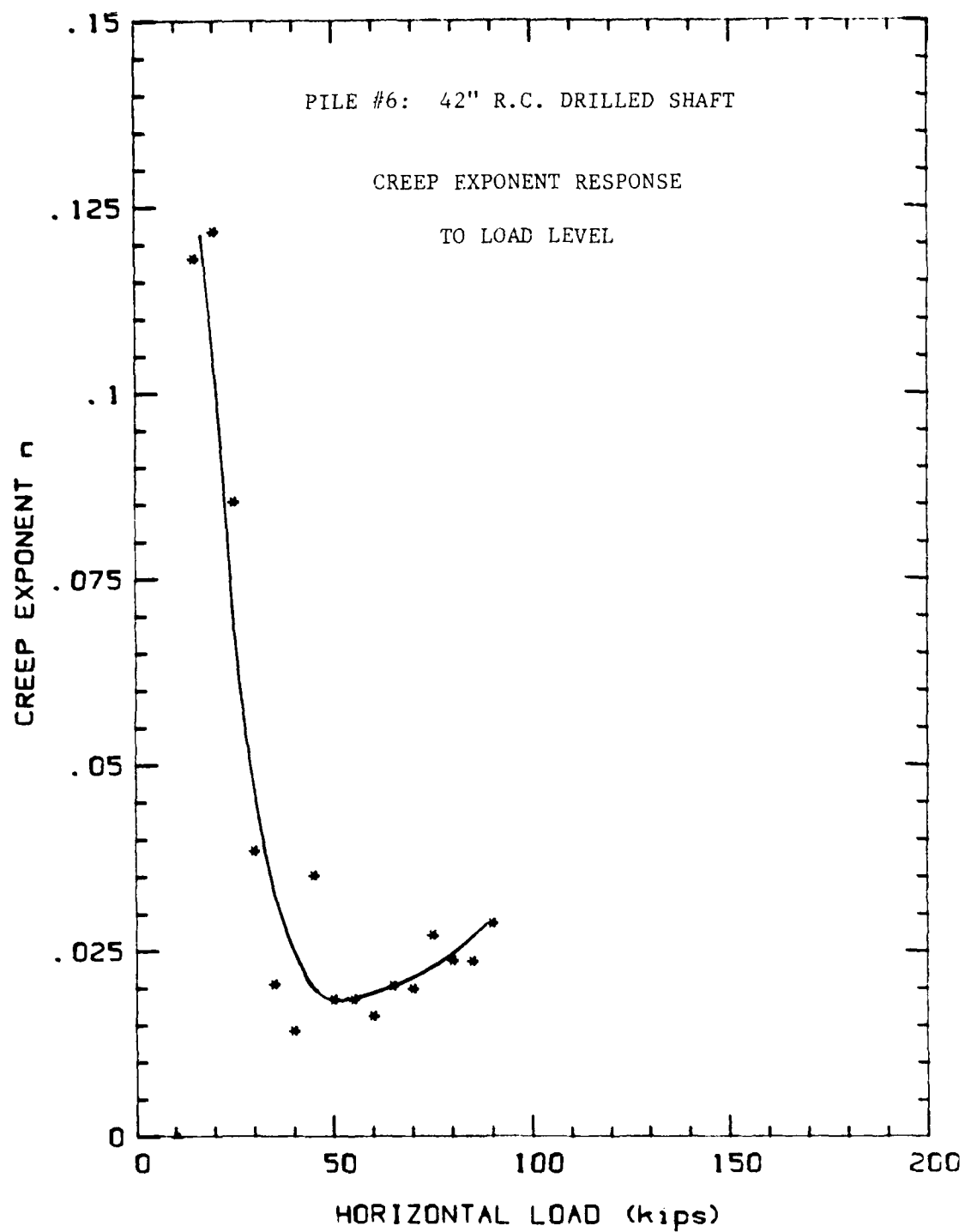
FIGURE 49. Creep Exponent Response to Load Level,
Pile No. 6.

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5. THE PRESSUREMETER TESTS

5.1 PMT Tests at the Site

Two series of pressuremeter tests were conducted prior to the lateral pile load tests and one series of tests afterwards. The first series was conducted in conjunction with the vertical pile load tests and consisted of prebored pressuremeter tests using the TEXAM PMT system (Briaud Engineers, 1986). This test series was performed in June 1986 and included two cone penetrometer test (CPT) soundings, but did not include any cyclic or creep tests. The second series, performed in December 1986, included both prebored TEXAM PMT and driven cone-pressuremeter (CPMT) tests with cyclic and creep tests under pressure-controlled conditions. These tests were concentrated within the upper layers of the soil which have the greatest impact on the response of laterally loaded piles. The third series, performed in January 1987, was also composed of both prebored TEXAM PMT and driven CPMT tests. The tests were conducted after the lateral load testing of the piles to investigate the changes in the soil response following the pile load tests.

The locations of the tests are indicated on the summary of in-situ tests shown in Figure 50. Corrected pressuremeter curves for the cyclic PMT tests used in this report are included in Appendix B.

5.2 PMT Moduli and Net Limit Pressure

The pressuremeter first load moduli, reload moduli and net limit pressure profiles for the site are presented in Figure 51. When compared to the data from the other geotechnical investigators (presented in Section 2) it can be seen that the PMT data confirms the general stratigraphy shown in Figure 3.

5.3 Prebored TEXAM PMT and Driven CPMT Test Results

The PMT tests performed prior to the lateral pile load tests were used to generate the monotonic P-y curves for

Test Hole	PMT Type	Date
B1	TEXAM	12/86
B2	Prebored	1/87
B3	PMT	1/87
D1	Driven	1/87
D2	CPMT	1/87
B4	TEXAM	6/86
CPT1	--	6/86
CPT2		6/86
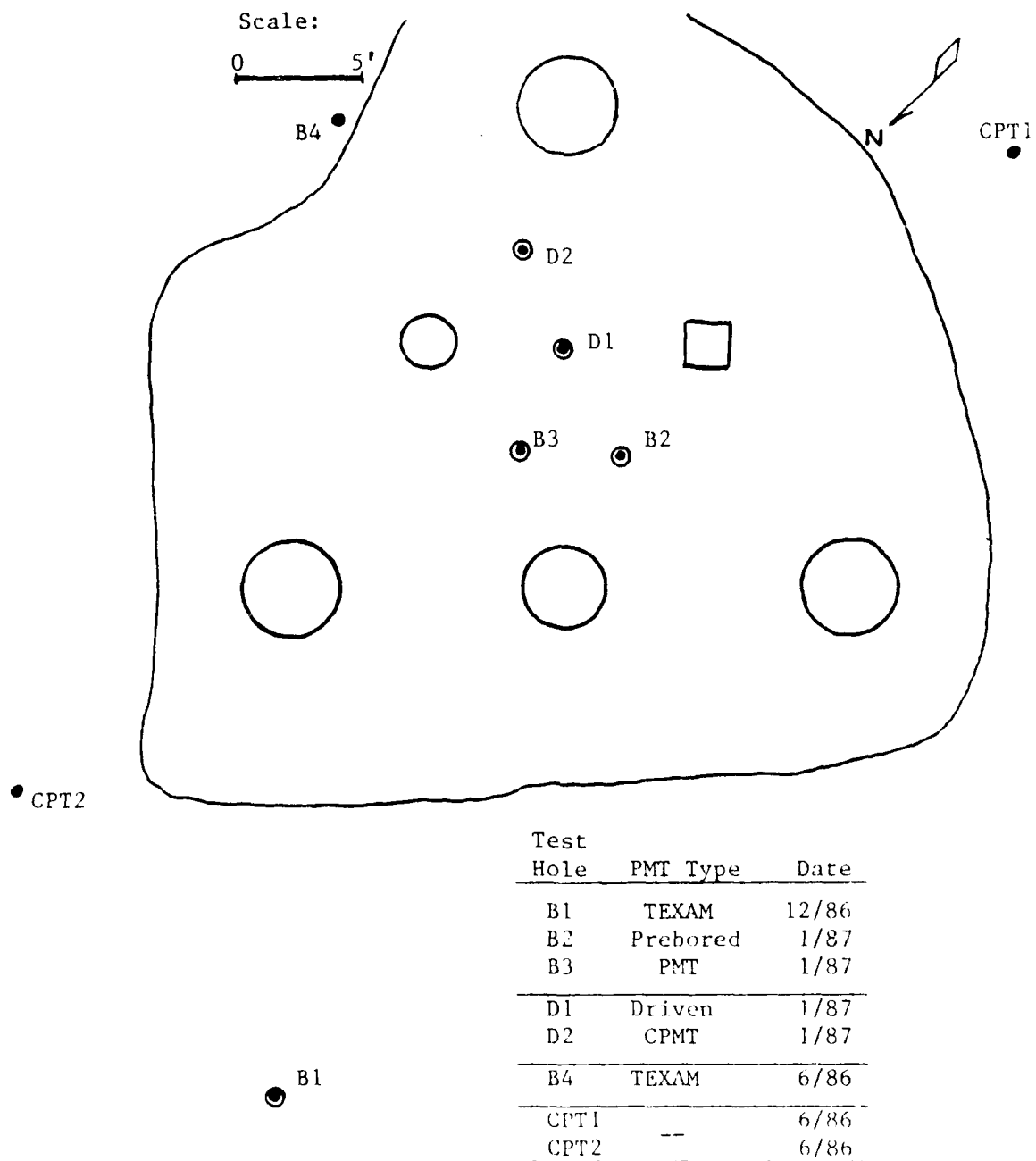
FIGURE 50. Location of In-Situ Tests at Load Test Site
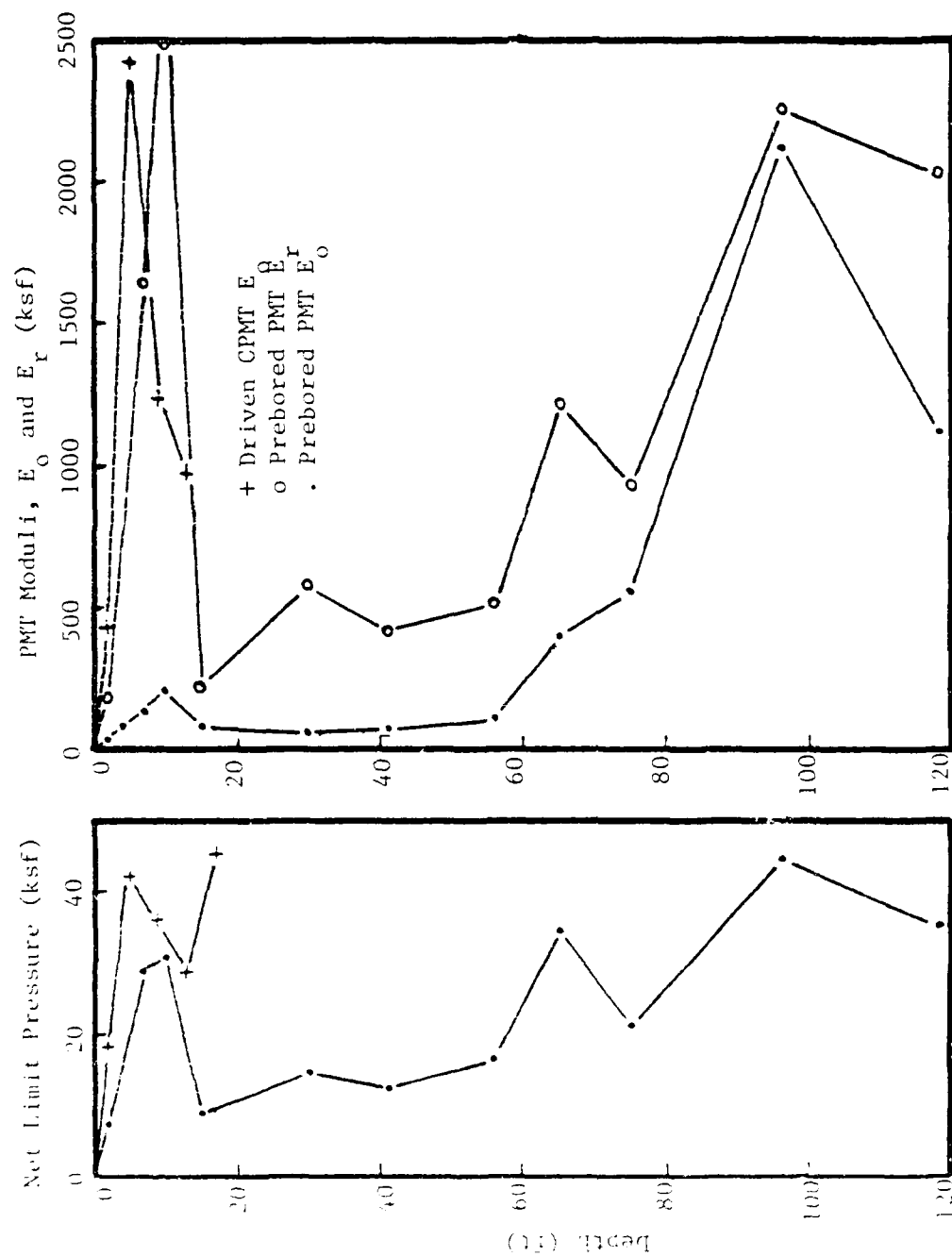
FIGURE 51. Net Limit Pressure, Initial Modulus and Reload Modulus Profiles

each of the pile types and the cyclic degradation and creep response exponents.

5.3.1 PMT generated P-y curves

The procedure for generating P-y curves from pressure-meter data is described in detail by Little and Briaud (1987). Generally, the process uses the analogy between the pressuremeter probe's expansion into the in-situ soil and the horizontal displacement of a laterally loaded pile. It provides a series of curves defining the total resistance to lateral displacement that may be expected during lateral loading of a pile within each layer of the soil stratigraphy. These curves are plots of the total soil resistance per unit length of pile, P, against the lateral pile displacement within each stratum, y.

The P-y curves generated from the pressuremeter tests at the load test site are presented in Figures 52 through 56. The first family of curves (Figure 52) were generated for the three 42-in diameter drilled shafts from the pre-bored TEXAM PMT test results. The P-y curves correspond relatively well to the site stratigraphy as shown in Figure 3. Recalling that the site had been excavated three ft before performance of the lateral load tests, the P-y curves increase in stiffness until a depth of 15 ft. This depth coincides with the first layer of firm clay. The P-y curves in the fine sand layers from 18 to 58 ft are clustered toge-ther. The soil resistance shows a marked increase in the dense sand layer 65 ft below the surface, and drops off in the clay layer below 75 ft.

Two different families of P-y curves were produced for the square prestressed concrete pile (Pile 3). Both sets assumed that the pile was a full displacement driven pile. The first set (Figure 53) was generated by using the driven CPMT test results down to 17 ft and using the prebored TEXAM PMT reload curves below 17 ft (Little and Briaud, 1987).
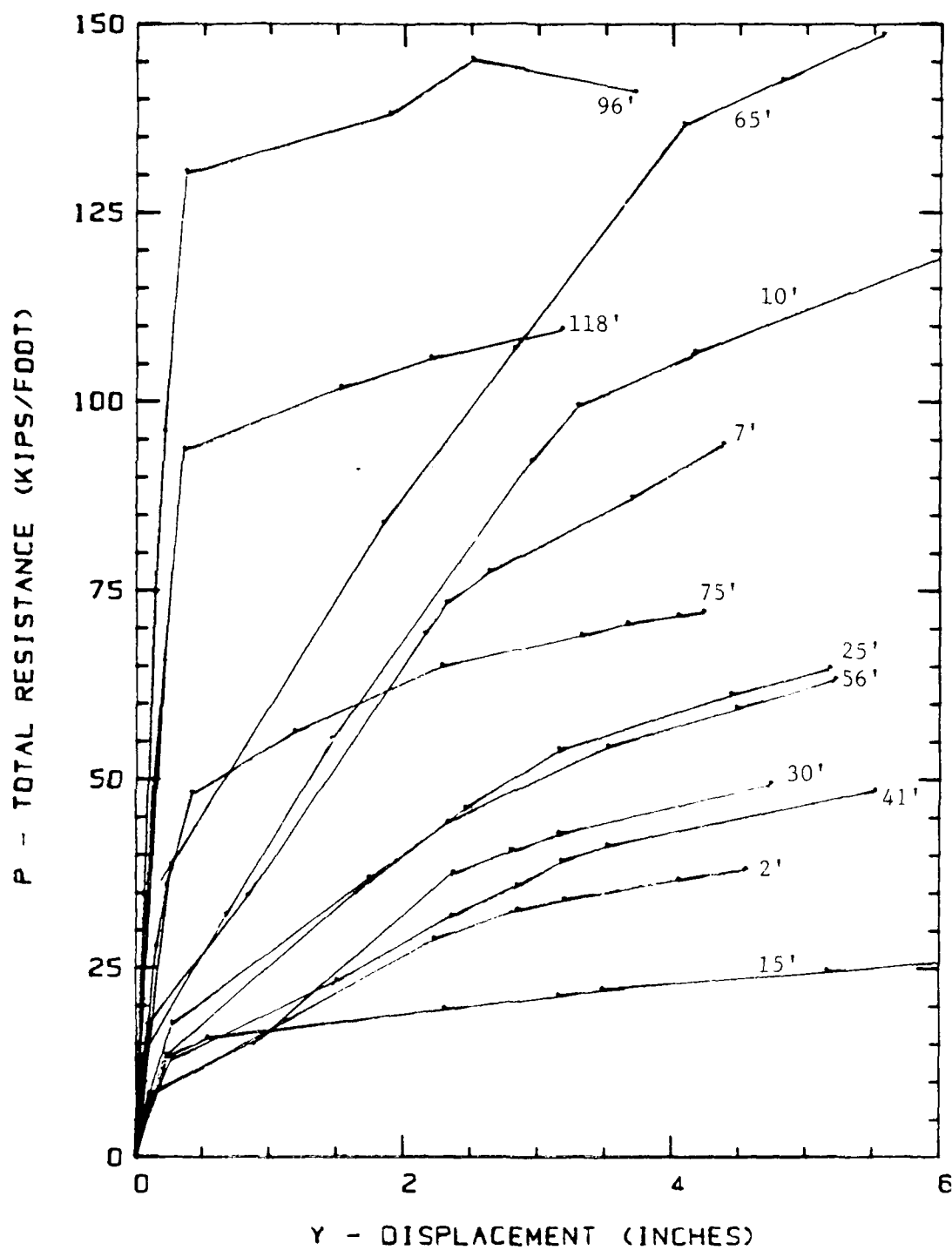
FIGURE 52. Prebored TEXAM PMT Generated P-y Curves for
42" R.C. Drilled Shafts, Pile Nos. 4, 5,
and 6.
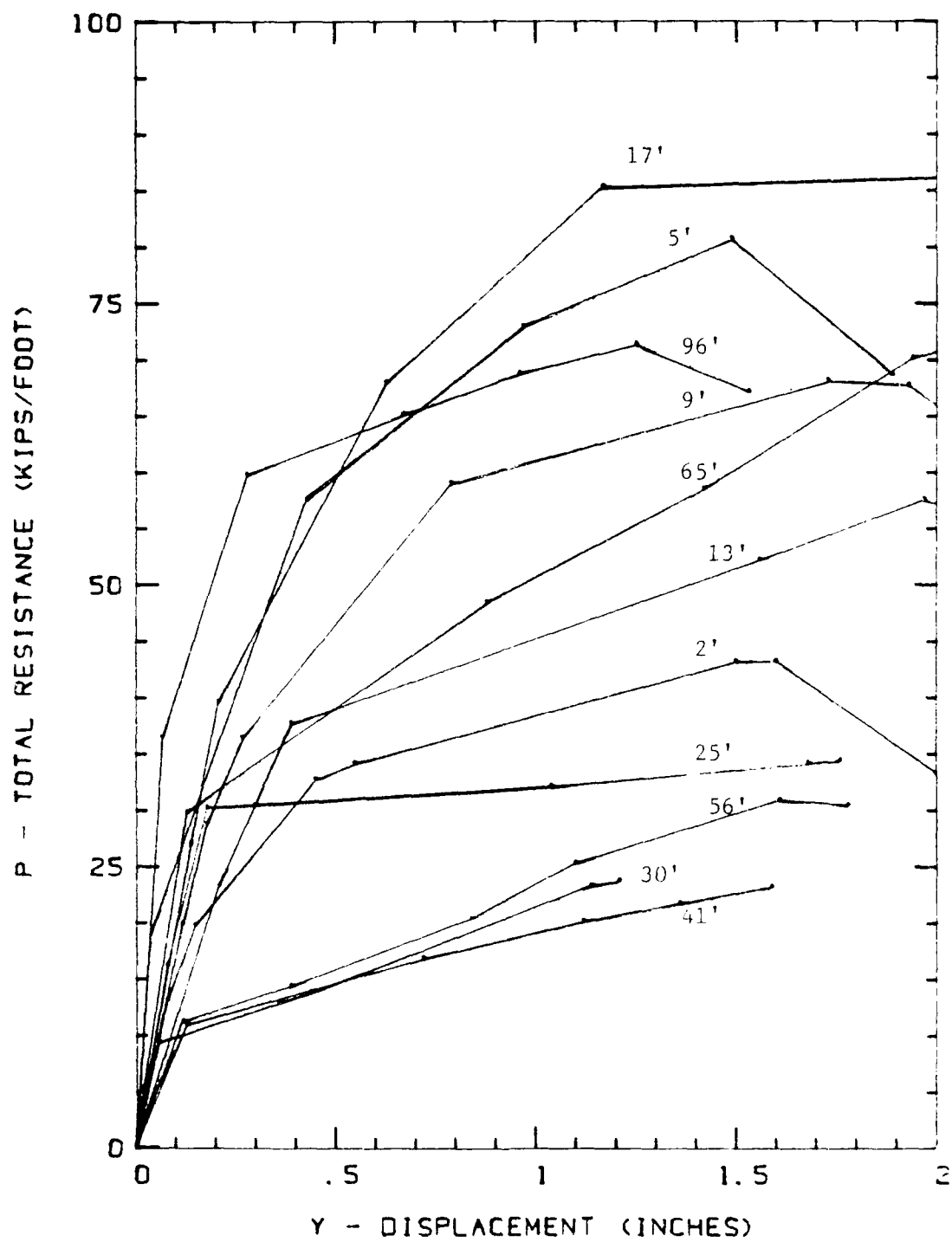
FIGURE 53. Driven CPMT Generated P-y for 20"
Square Concrete, Pile No. 3.
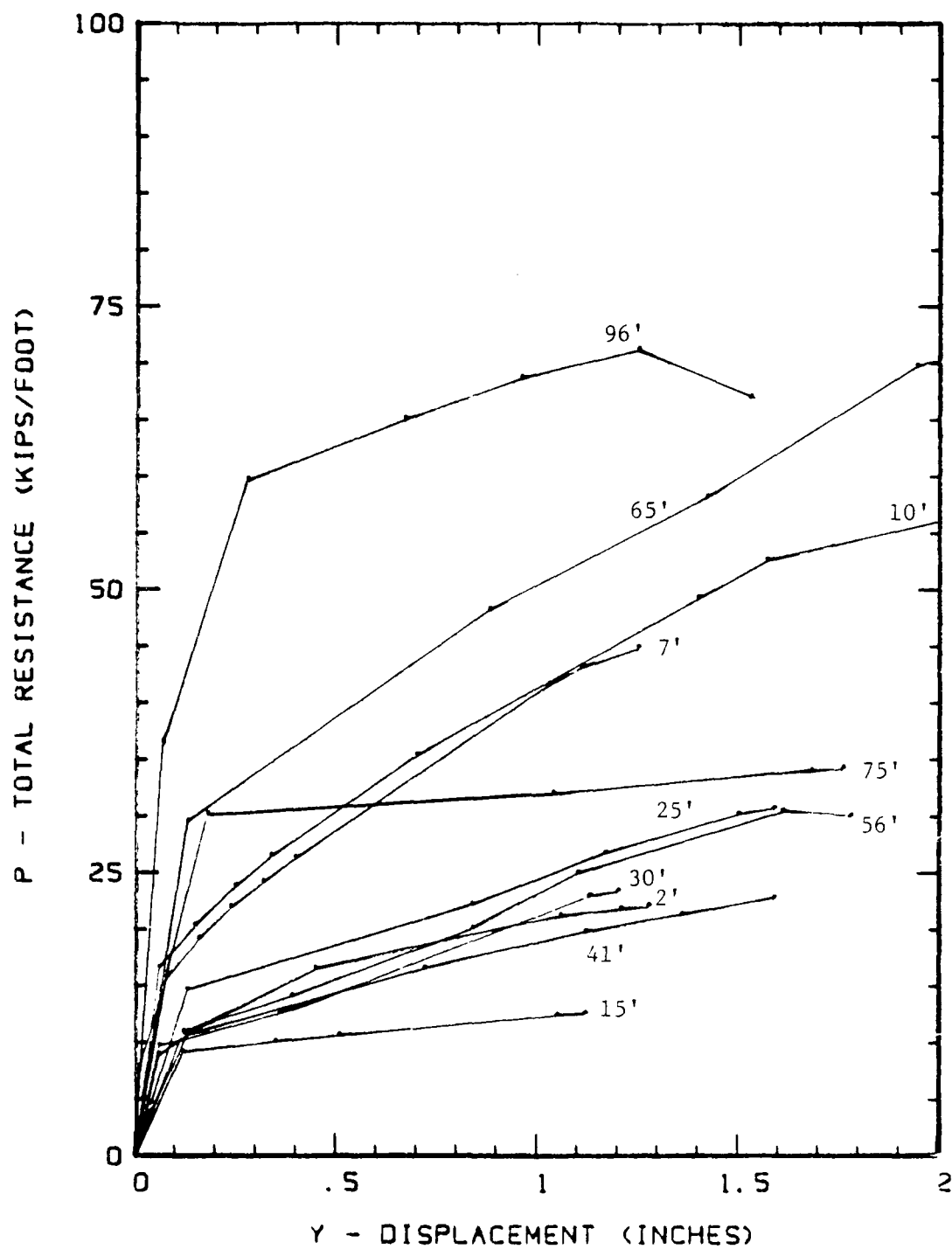
FIGURE 54.   Prebored TEXAM PMT Generated P-y Curves
for 20" Square Concrete, Pile No. 3.
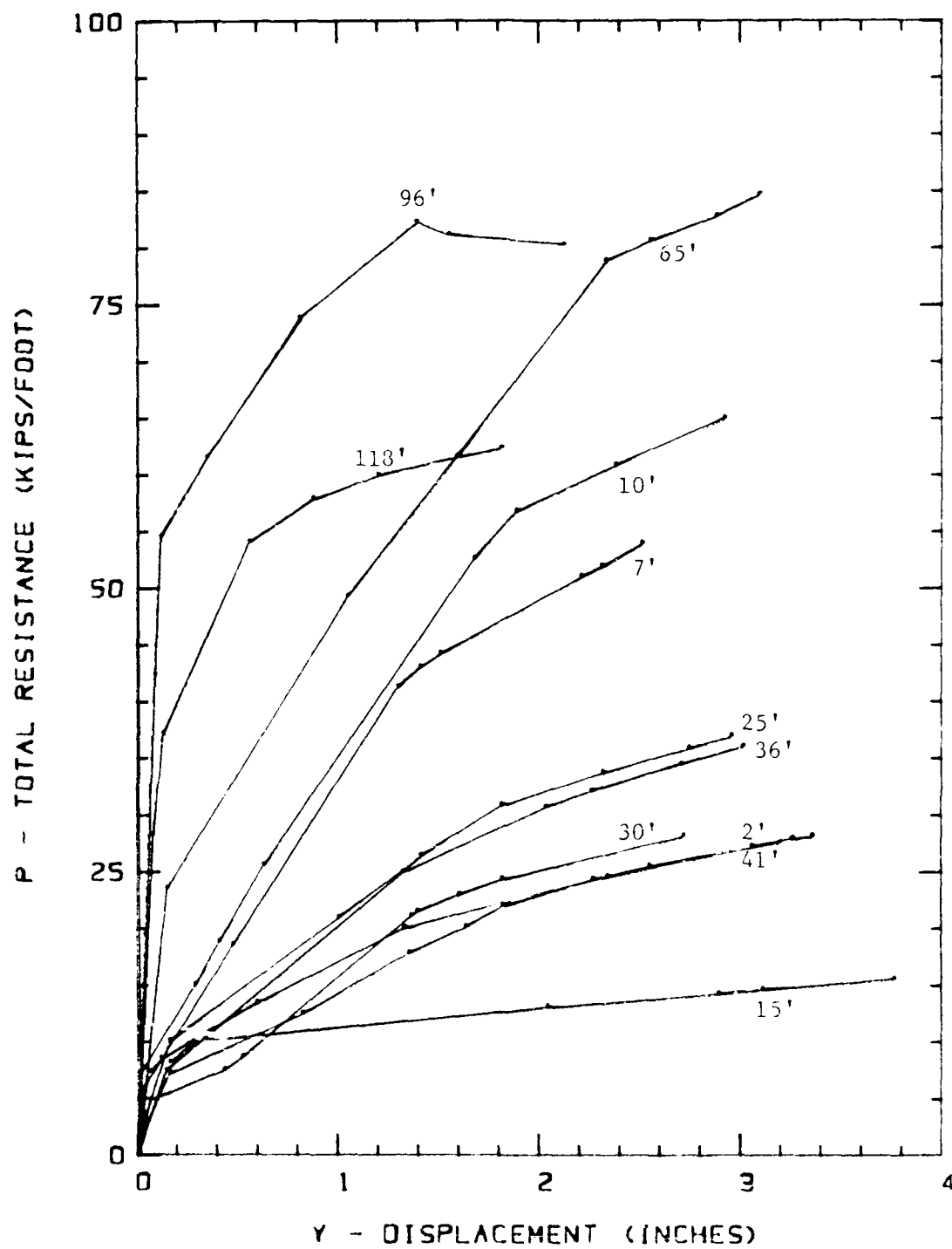
FIGURE 55. Prebored TEXAM PMT Generated P-y Curves for 24" Non-displacement Steel Pipe Pile, Pile No. 2.
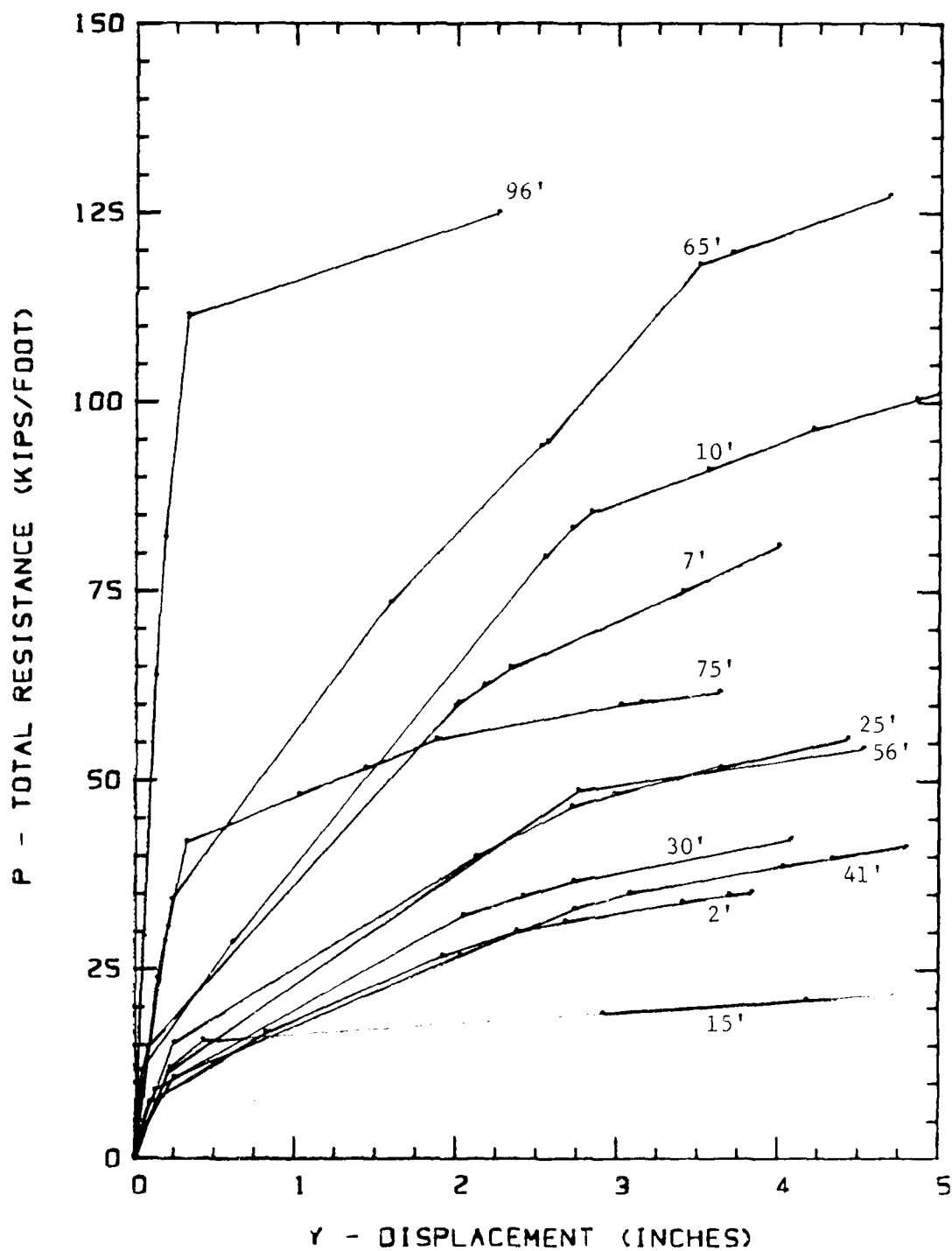
FIGURE 56. Prebored TEXAM PMT Generated P-y Curves for 36" R.... Drilled Shaft, Pile No. 1.

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The second set (Figure 54) was generated using only the prebored TEXAM PMT reload curves. As can be seen in Figures 53 and 54 the driven CPMT tests generally lead to stiffer P-y curves.

The P-y curves for the pipe pile (Figure 55) were generated using the prebored TEXAM PMT test results and assuming that the pile was a non-displacement pile. This assumption is consistent with the fact that the pile did not plug until a significant depth. Therefore, at least in the more important shallow depth region, the pile acted as a non-displacement pile.

The P-y curves for the 36-in diameter drilled shaft (Figure 56) were also generated from the prebored TEXAM PMT test results.

The P-y curves for the 24-in diameter pipe pile and the 36-in diameter drilled shaft prepared using the conventional method (Reese et al., 1974) are shown in Figures 57 and 58. These curves were prepared by McClelland Engineers (1986). Compared to the PMT P-y curves, the conventional P-y curves show a much softer initial response and a lower ultimate soil resistance within the critical upper layers of the soil.

5.3.2 Cyclic degradation parameters

The cyclic degradation parameter, a, for the pressuremeter tests represents the degradation of the secant PMT shear modulus with increasing cycles as defined in Figure 59. The $G_{S(N)}/G_{S(1)}$ versus N curves for each test are presented in Appendix C. A summary of the resulting a values for the secant shear modulus degradation is presented in Table 4.

The cyclic degradation parameters for the driven CPMT and the prebored PMT tests at 2 ft depth are less than the a values of larger depths. A possible explanation for the
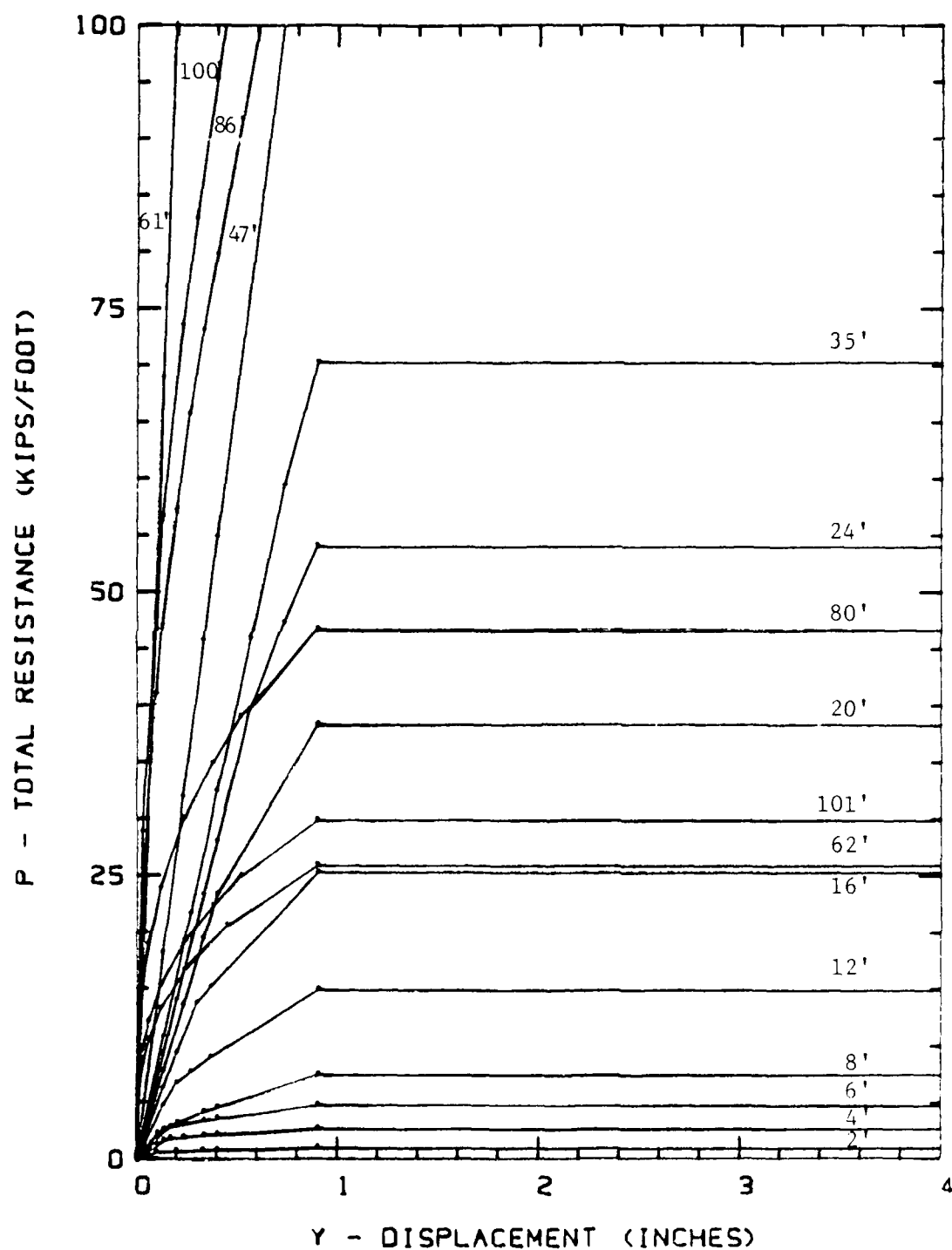
FIGURE 57. Conventionally Prepared P-y Curves for 24"
Non-displacement Pipe Pile (McClelland
Engineers, 1986).
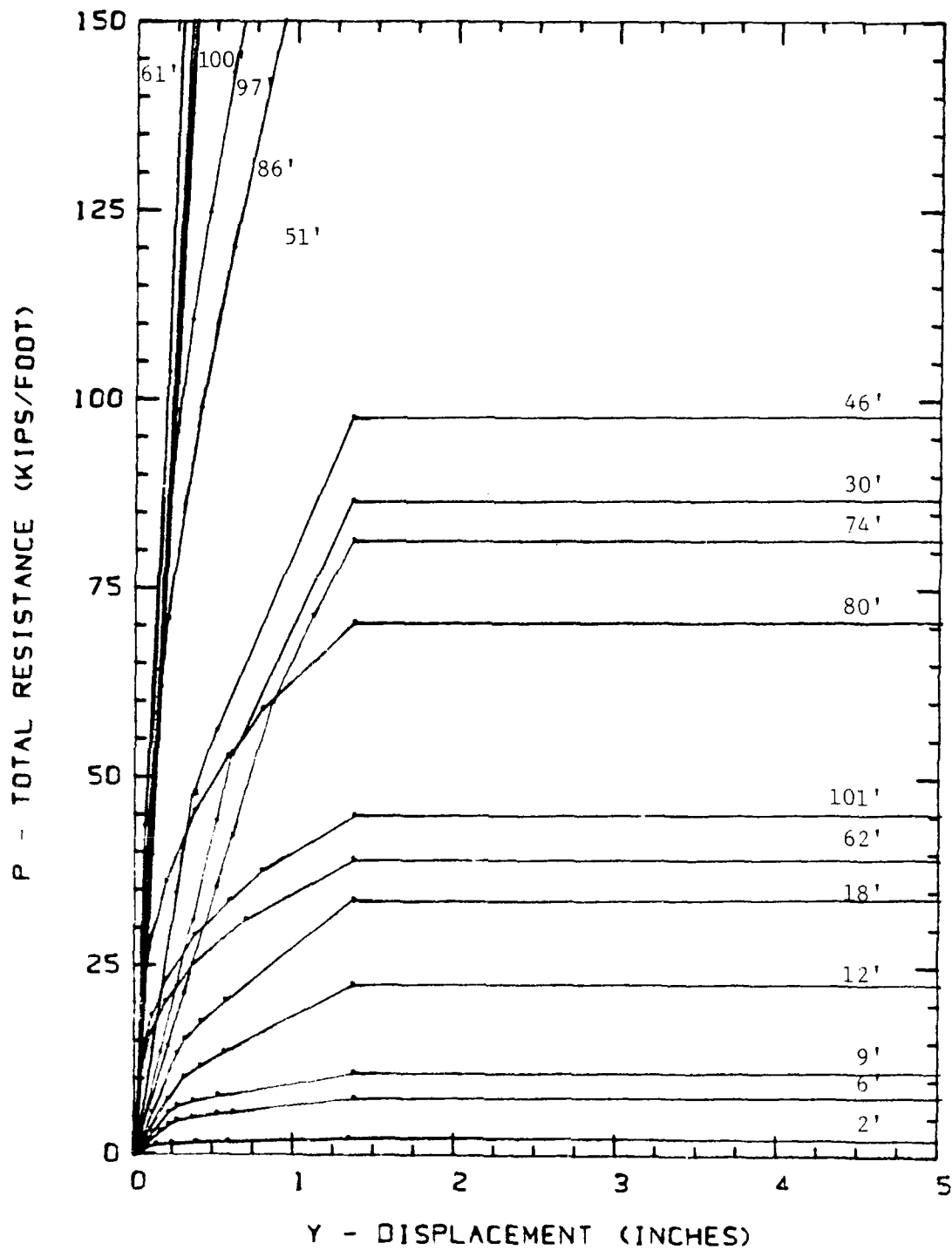
FIGURE 58. Conventionally Prepared P-y Curves for 36"
R.C. Drilled Shaft (McClelland Engineers,
1986).
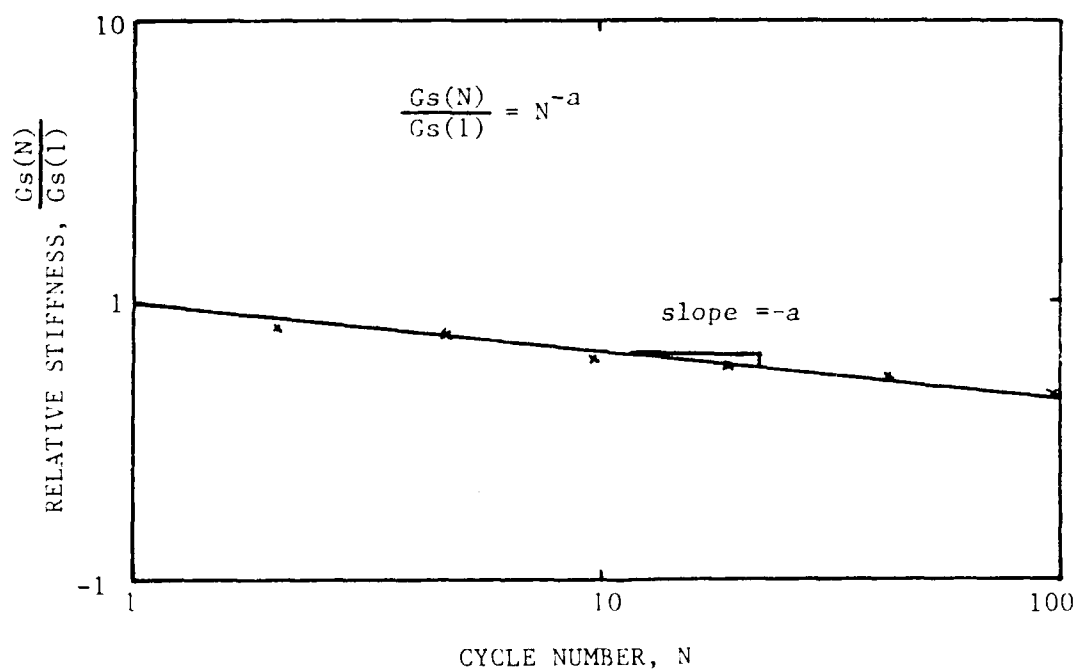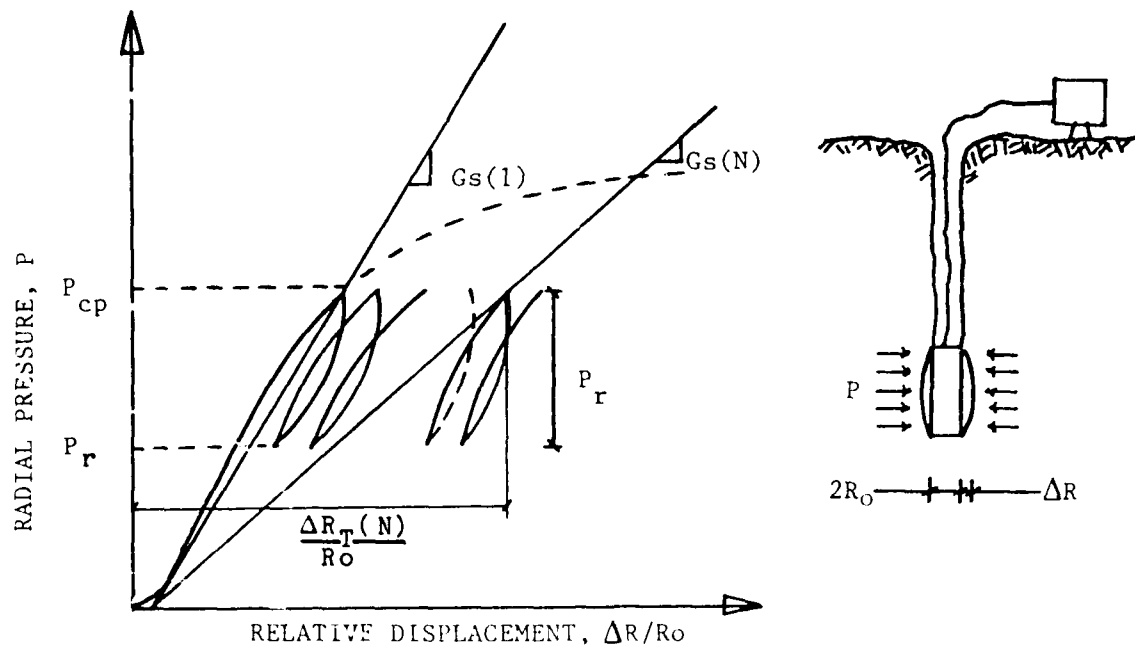
Figure 59. Definition of the Cyclic Degradation
Parameter for the Secant Shear Modulus.

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TABLE 4. Pressuremeter Cyclic Degradation Parameters
for the Secant Shear Moduli

PMT Test No.	PMT Type	Depth (ft)	ΔR/R (%)	a	a Average
B2-2		2	12.8 22.4	0.044 0.038	0.041
B1-7		7	7.9 15.8	0.091 0.062	0.077
B1-10	PREBORED TEXAM	10	5.8 16.0	0.086 0.063	0.075
B1-15		15	17.3	0.064	0.064
B1-30		30	13.3 22.0	0.074 0.056	0.065
Overall Average					0.064
D1-2		2	3.2 7.3	0.054 0.107	0.081
D1-5		5	1.2 5.0	0.115 0.094	0.105
D1-9	DRIVEN CPMT	9	3.2 9.1	0.112 0.108	0.110
D1-13		13	3.3 8.6	0.108 0.109	0.109
D1-17		17	1.8 6.4	0.136 0.096	0.116
Overall Average					0.105
B3-2	PREBORED TEXAM	2	7.6 13.4	0.056 0.030	0.043
D2-2	DRIVEN CPMT	2	3.6 8.5	0.120 0.108	0.114

lower degradation may relate to the degree of saturation of the sand. The water table at the time of testing was located 3.5 ft below the ground surface, indicating that degradation may be greater in saturated sands than in unsaturated sands.

The average a values below the water table were fairly consistent. For prediction purposes an overall average a value was selected for each pile (see Section 6.2).

Another observation on the cyclic degradation entailed the degradation of the cyclic shear modulus as defined in

Figures 60 and 61. The $G_{C(N)}/G_{C(1)}$ versus N curves for the individual PMT tests may be found in Appendix D. The curves for the driven CPMT tests show an apparent degradation of the cyclic shear modulus during the first series of cycles.

5.3.3 Creep response

Near the end of each PMT test the pressure was held constant while recording the increase in volume of the probe. The results are presented graphically in Figure 62 and 63 using the same variables as employed to define creep in the piles (Section 4.4.3). For the prebored TEXAM PMT the creep exponent, n, averaged 0.006. The average n value for the driven CPMT tests was 0.011. Both values fell below the creep exponents found for the load test piles. The difference between the pile creep and the PMT creep exponents may be the result of the creep occuring in the pile material inself (Section 4.4.3).
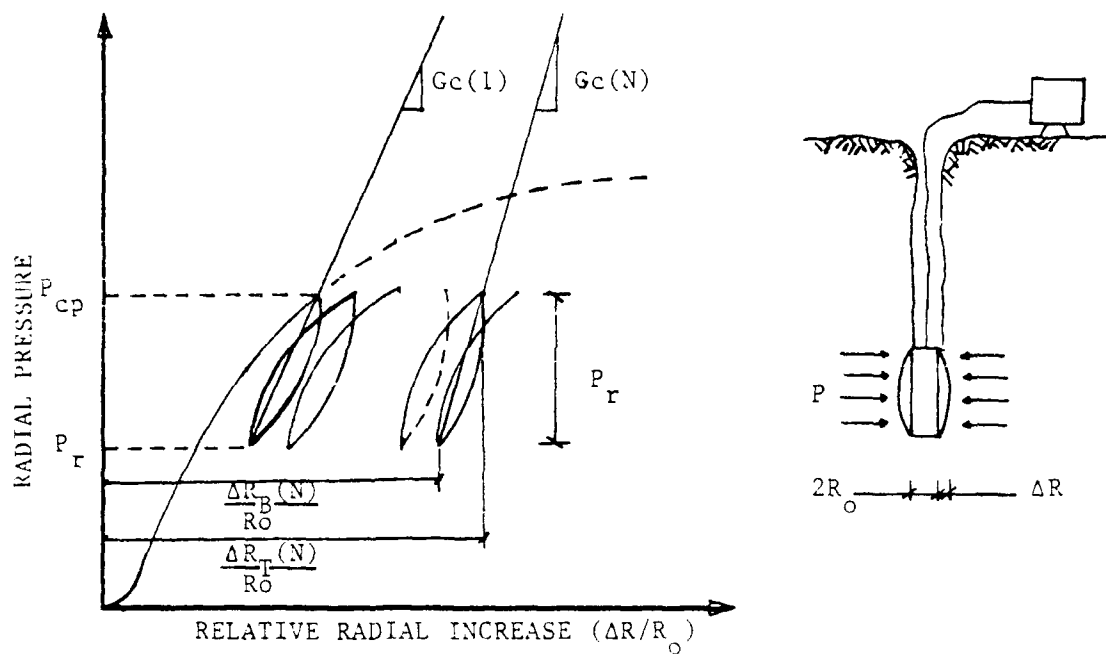
Figure 60. Definition of the Cyclic Shear Modulus.
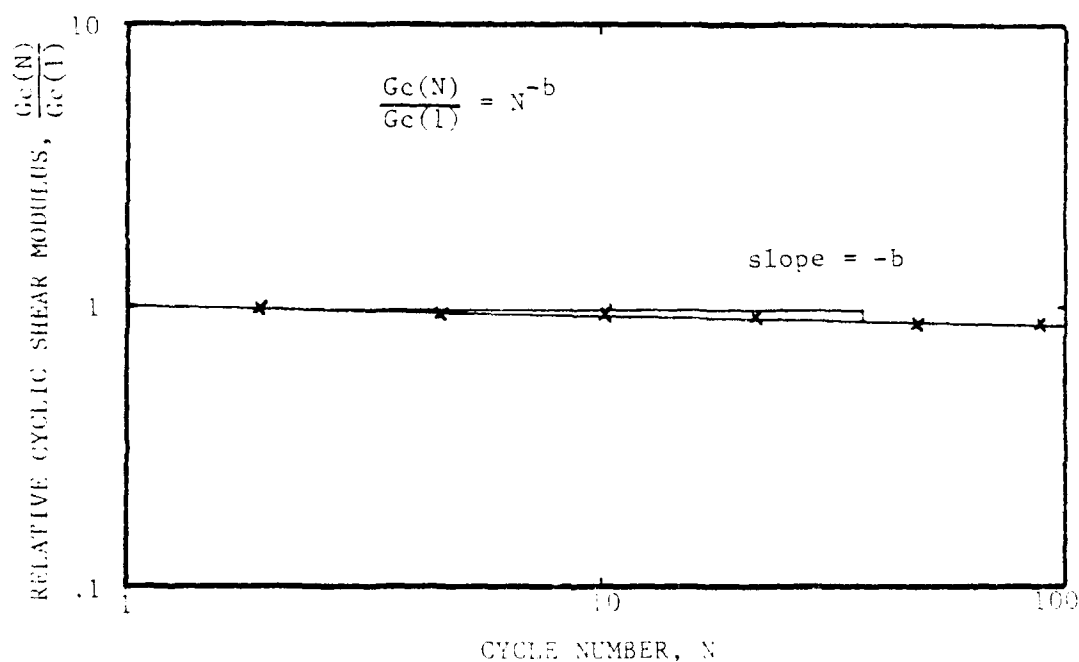
Figure 61. Definition of the Cyclic Degradation Parameter
for the Cyclic Shear Modulus.
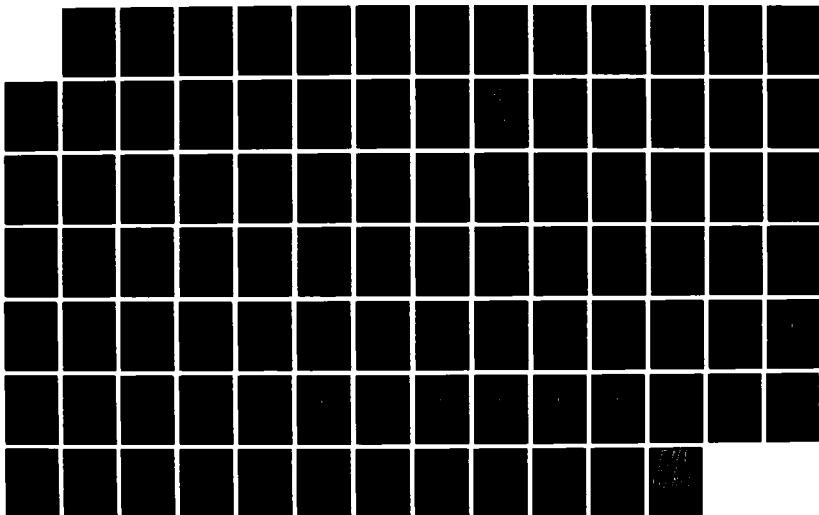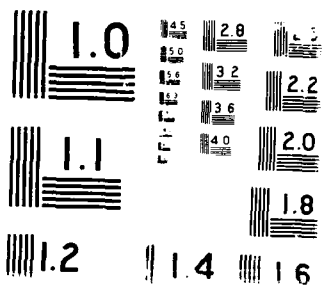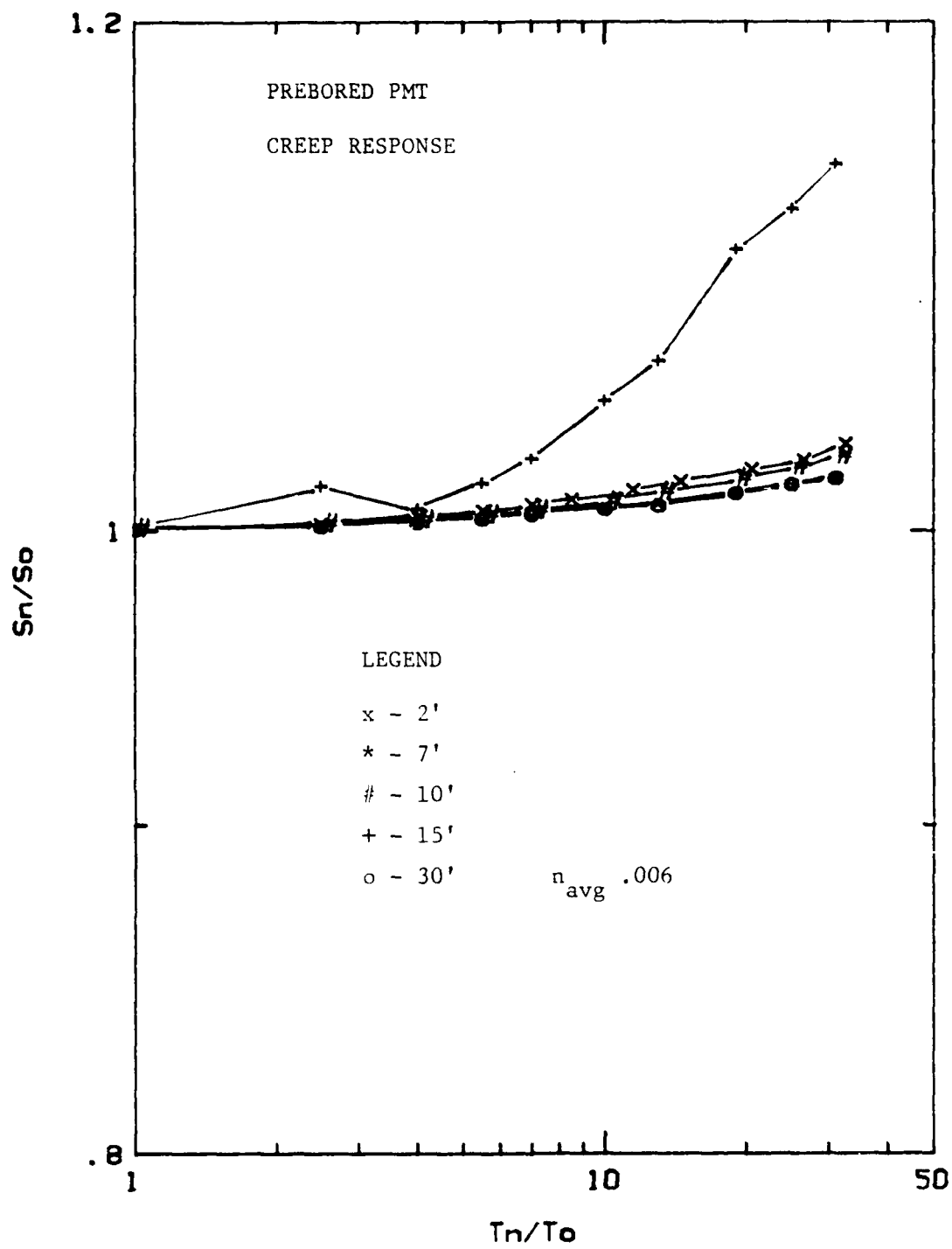
FIGURE 62. Creep Response in the Prebored PMT Tests.

83
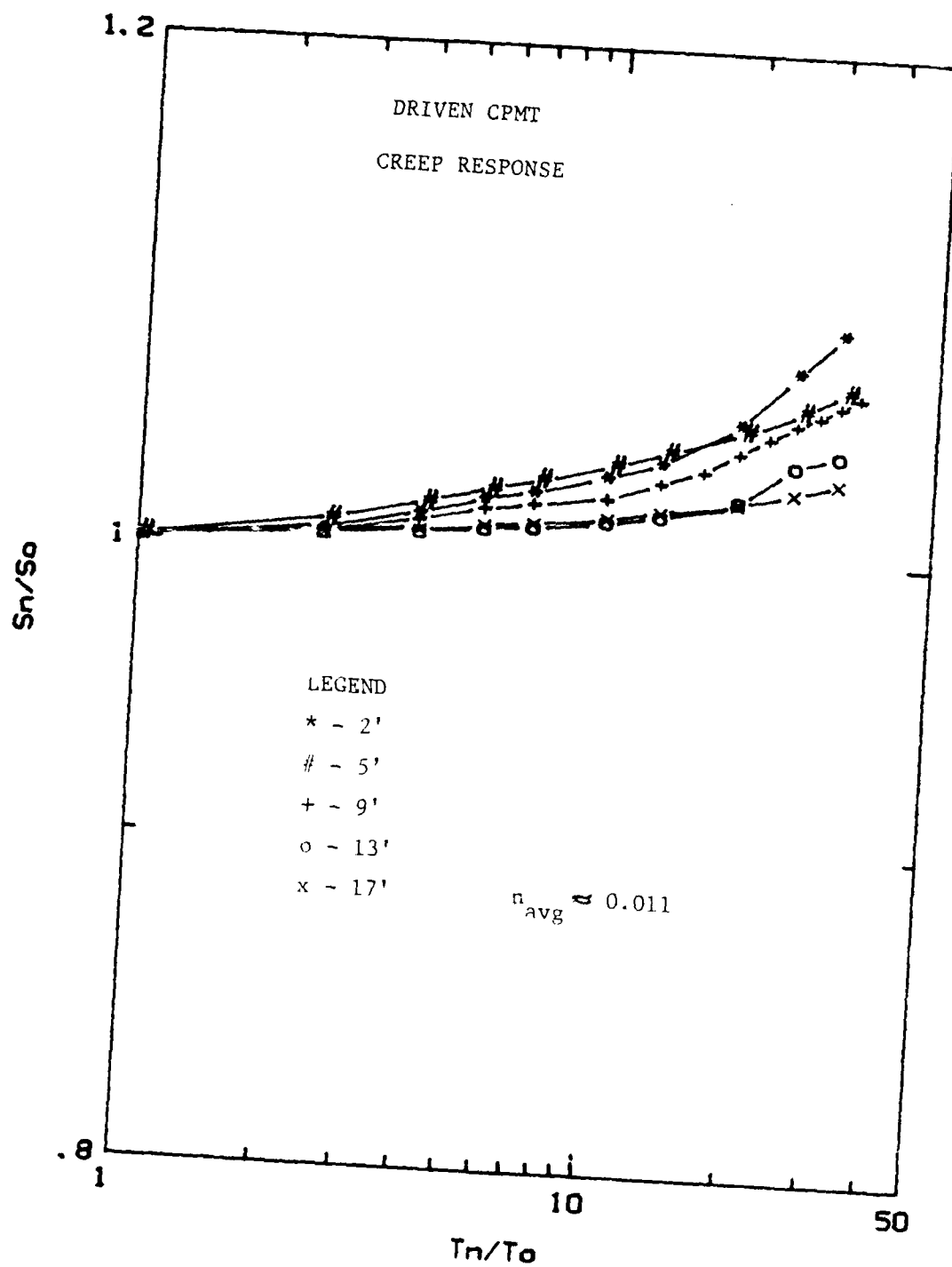
FIGURE 63. Creep Response in the Driven CPMT Tests.

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6. COMPARISON OF PMT AND CONVENTIONAL PREDICTIONS WITH THE MEASURED RESPONSE

The approach employed in this report to predict the monotonic response of the test piles has been presented in detail in an earlier report (Little and Briaud, 1987). For the prediction of the cyclic response, the desired number of cycles is first selected, then each value of y from the monotonic P-y curve is multiplied by N^a to obtain $y(N)$. The deflection $y(N)$ is the deflection after N cycles at the chosen level of soil resistance. The a values were selected as detailed in Section 5.3.2. This process is summarized in Figure 64 and in the following equations:

$$P(N) = P(1) \qquad\qquad (2)$$
$$y(N) = y(1) \times N^a \qquad\qquad (3)$$

where N = cycle number for which the P-y curve is desired,

   P(1) = total soil resistance arrived at in static analysis,

   P(N) = total soil resistance arrived at after N cycles,

   y(1) = the static displacement at P(1),

   y(N) = the displacement at P(N) after N cycles, and

   a = the cyclic degradation parameter otained from the pressuremeter tests.

The cyclic P-y curves were then input as resistances into a beam-column program to obtain the predicted deflections of a pile subjected to a given set of cyclic lateral loads.

6.1 Monotonic Loading Response

The preboring PMT prediction yielded excellent results for the 36-in diameter drilled shaft at loads up to 40 kips (Figure 65). The conventional method predicted a much softer response. At higher loads (Figure 66), after the pile had been subjected to the series of cycles, the PMT predic-

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$$P(N) = P(1)$$

$$y(N) = y(1) \times N^{+a}$$
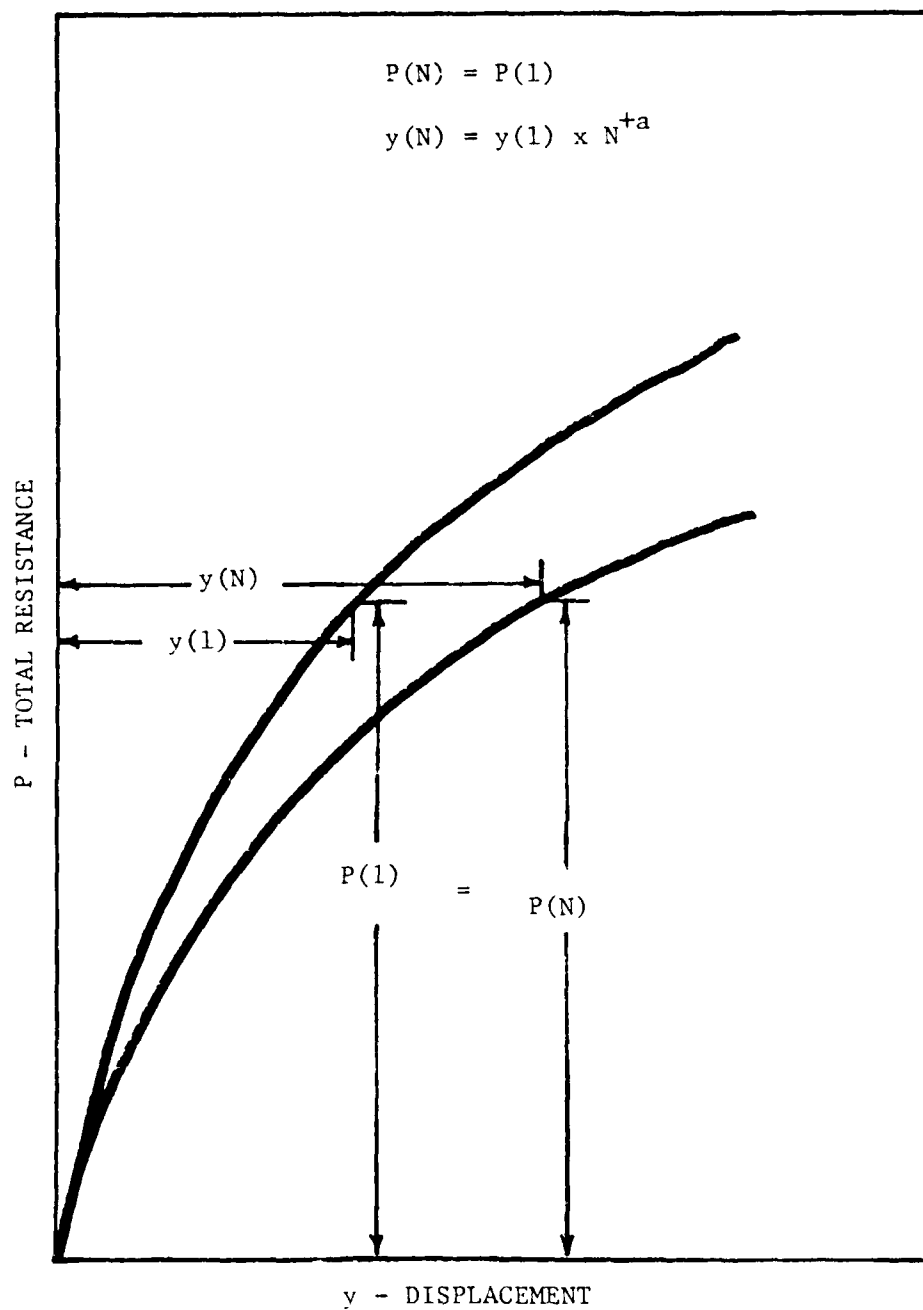
FIGURE 64.   Summary of Method used to Modify a Static
P-y Curve for Cyclic Predictions.

86
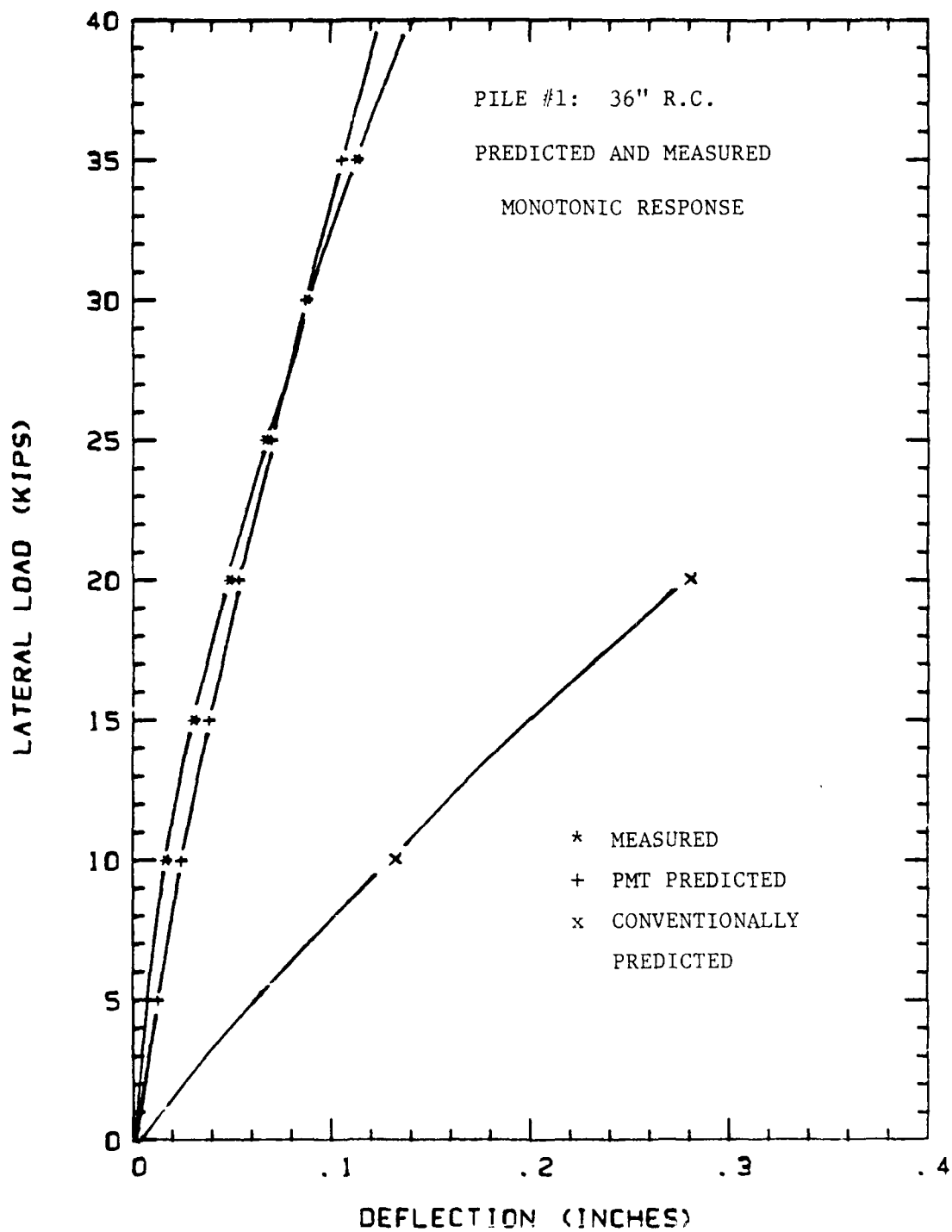
FIGURE 65. Comparison of PMT Predicted, Conventionally Predicted, and Measured Response for Pile No. 1 under Monotonic Loading, 0 to 40 KIP Scale.

87
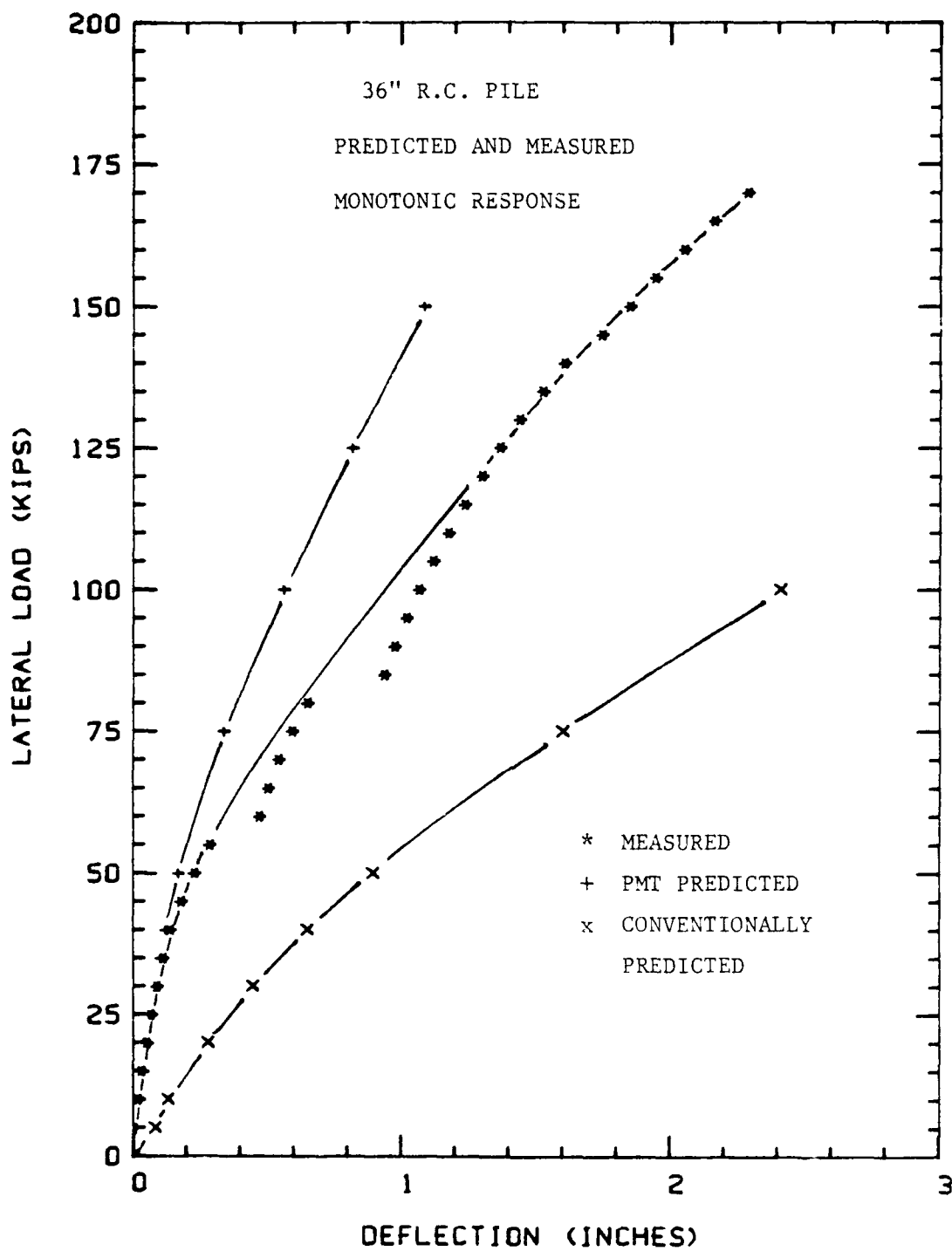
FIGURE 66. Comparison of PMT Predicted, Conventionally Predicted, and Measured Response for Pile No. 1 under Monotonic Loading, 0 to 200 KIP Scale.

tion was stiffer than the measured results. This is probably due to the fact that the cycles induce accentuated curvature in the monotonic envelope and that the flexural stiffness of the pile decreases with increasing load and with increasing number of cycles due to crack propagation. This deterioration was not modeled in the prediction process.

The predicted response for the pipe pile using the preboring PMT and assuming the pile was a non-displacement pile gave excellent results throughout the range of lateral loads applied (Figures 67 and 68). The steel pipe was not subject to the same magnitude of stiffness deterioration as the concrete drilled shafts. At high load levels, after the series of cycles, the PMT method slightly underpredicted the pile displacement. The conventional method, on the other hand, significantly overpredicts the displacements throughout the range of loads applied to the pile.

The square concrete pile was modeled with both the driven CPMT and prebored PMT test results. Both methods produced excellent predictions for loads up to 40 kips (Figure 69). At higher loads the prebored PMT predictions closely followed the measured results until after the second cycling series (Figure 70). It is likely that the deterioration of the pile stiffness (EI-value) associated with cycling was not a factor in the pile-soil response until the effects of the prestressing in the pile were overcome. Therefore, the envelope on measured results up to the second series of cycles probably is an accurate reflection of the soil response alone. The driven CPMT predictions overestimated the pile-soil stiffness response at high loads.

For the three 42-in diameter drilled shafts the pressuremeter method predicted a softer initial response at loads below 30 kips and a stiffer response at load levels over 50 kips (Figures 71 and 72). A partial reason for the predictions of the 42-in diameter drilled shafts not being
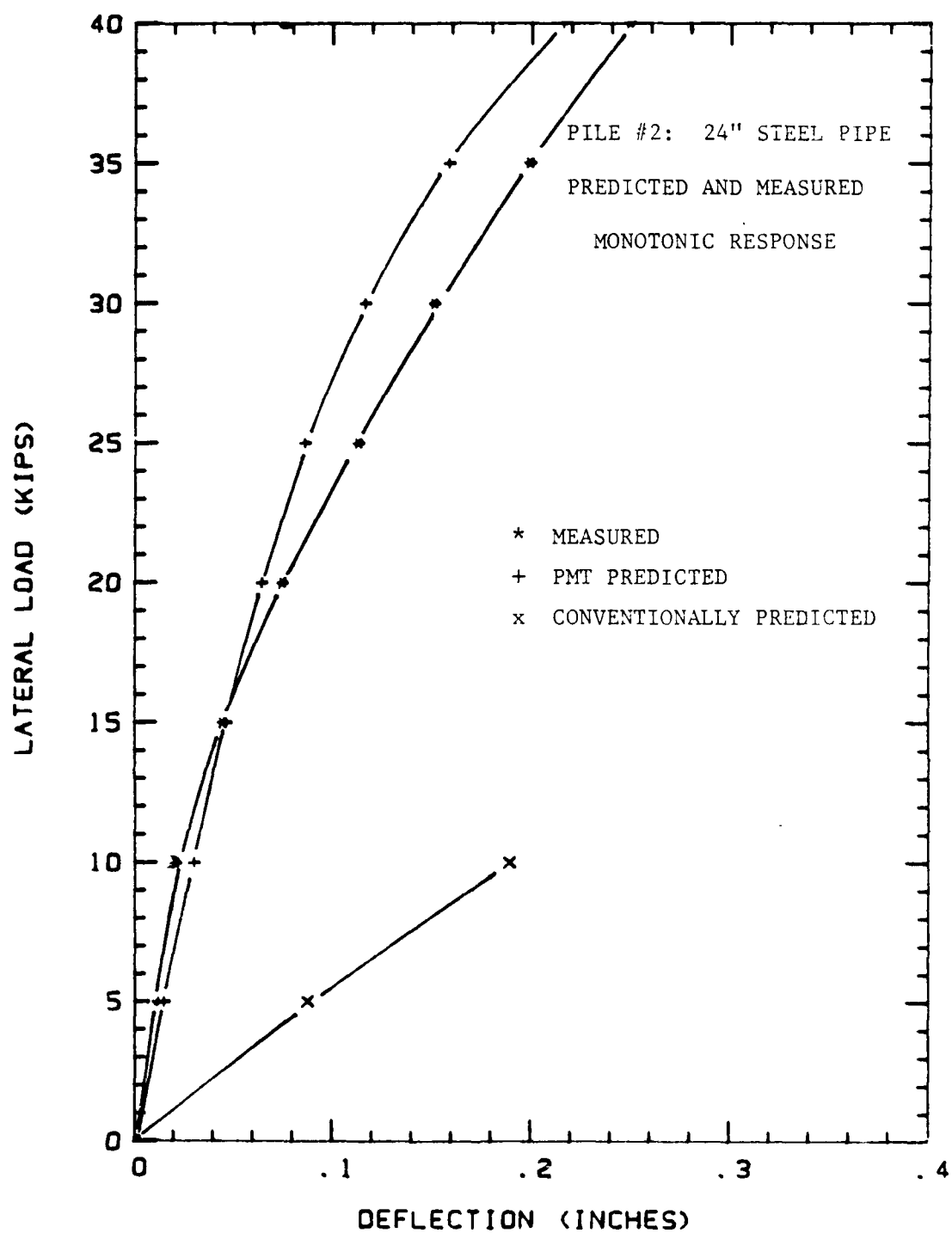
FIGURE 67. Comparison of PMT Predicted, Conventionally
Predicted, and Measured Response for Pile No.
2 under Monotonic Loading, 0 to 40 KIP Scale.

90
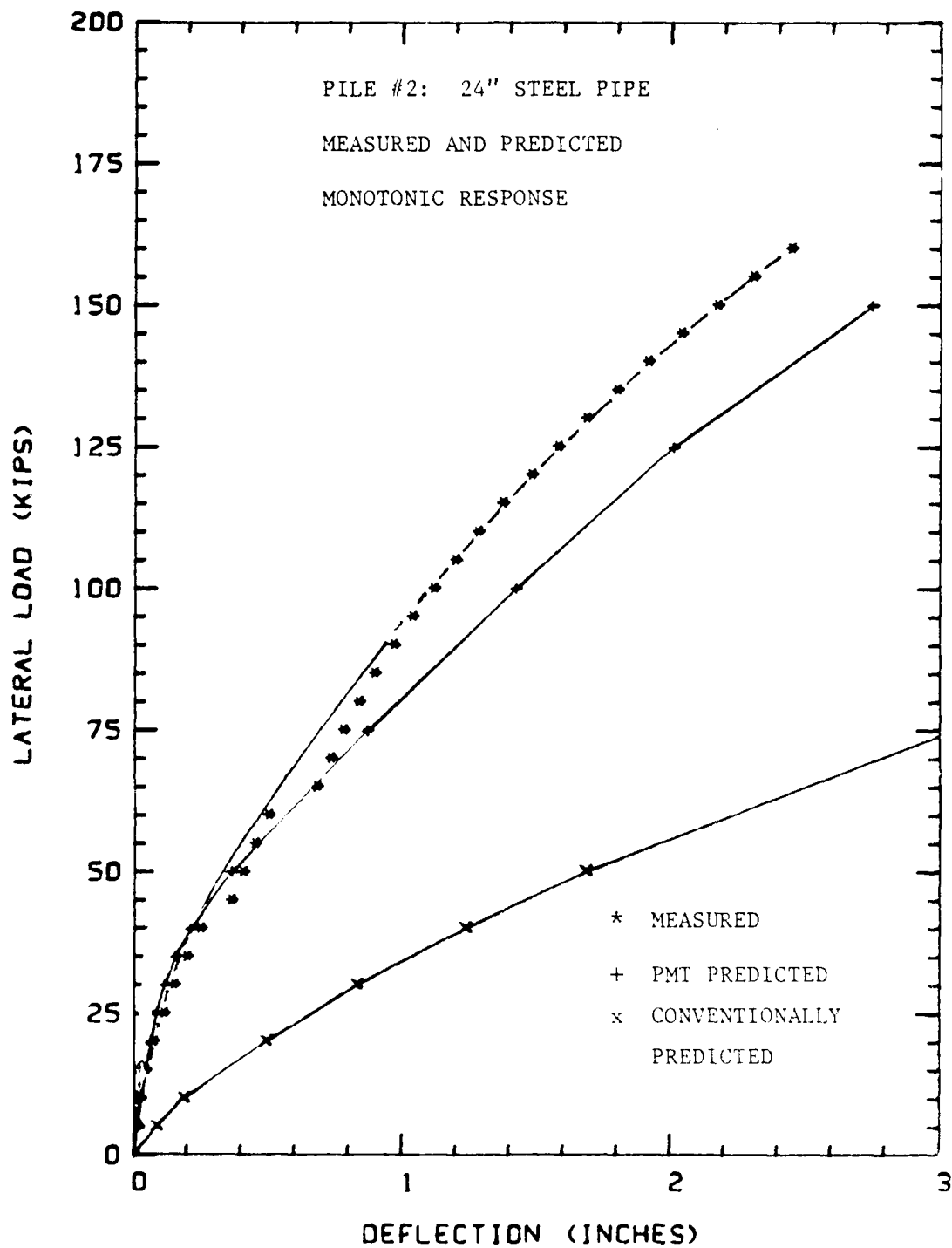
FIGURE 68. Comparison of Predicted, Conventionally Predicted, and Measured Response for Pile No. 2 under Monotonic Loading, 0 to 200 KIP Scale.

91
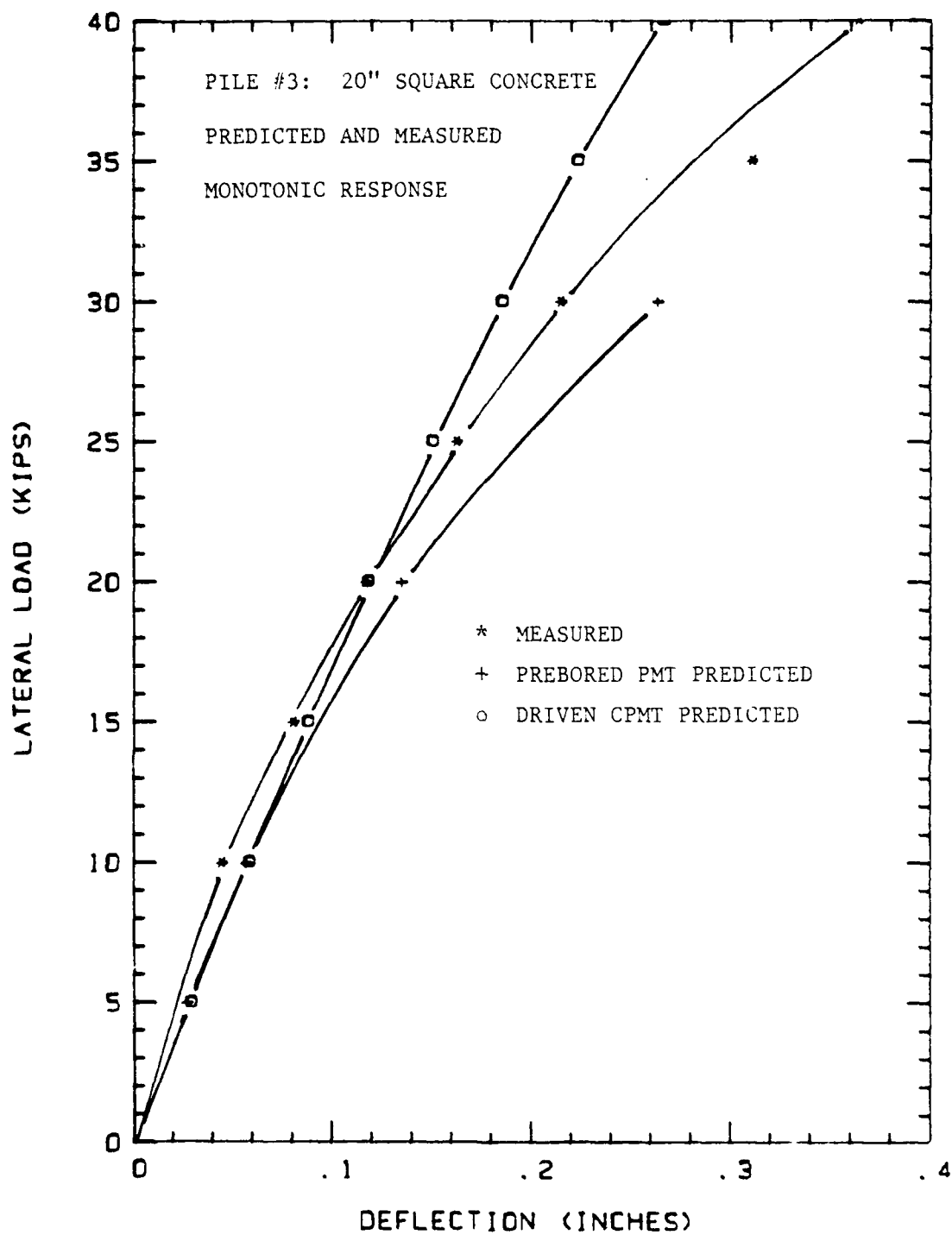
FIGURE 69. Comparison of Measured and PMT Predicted Monotonic Responses for Pile No. 3, 0 to 40 kip Scale.
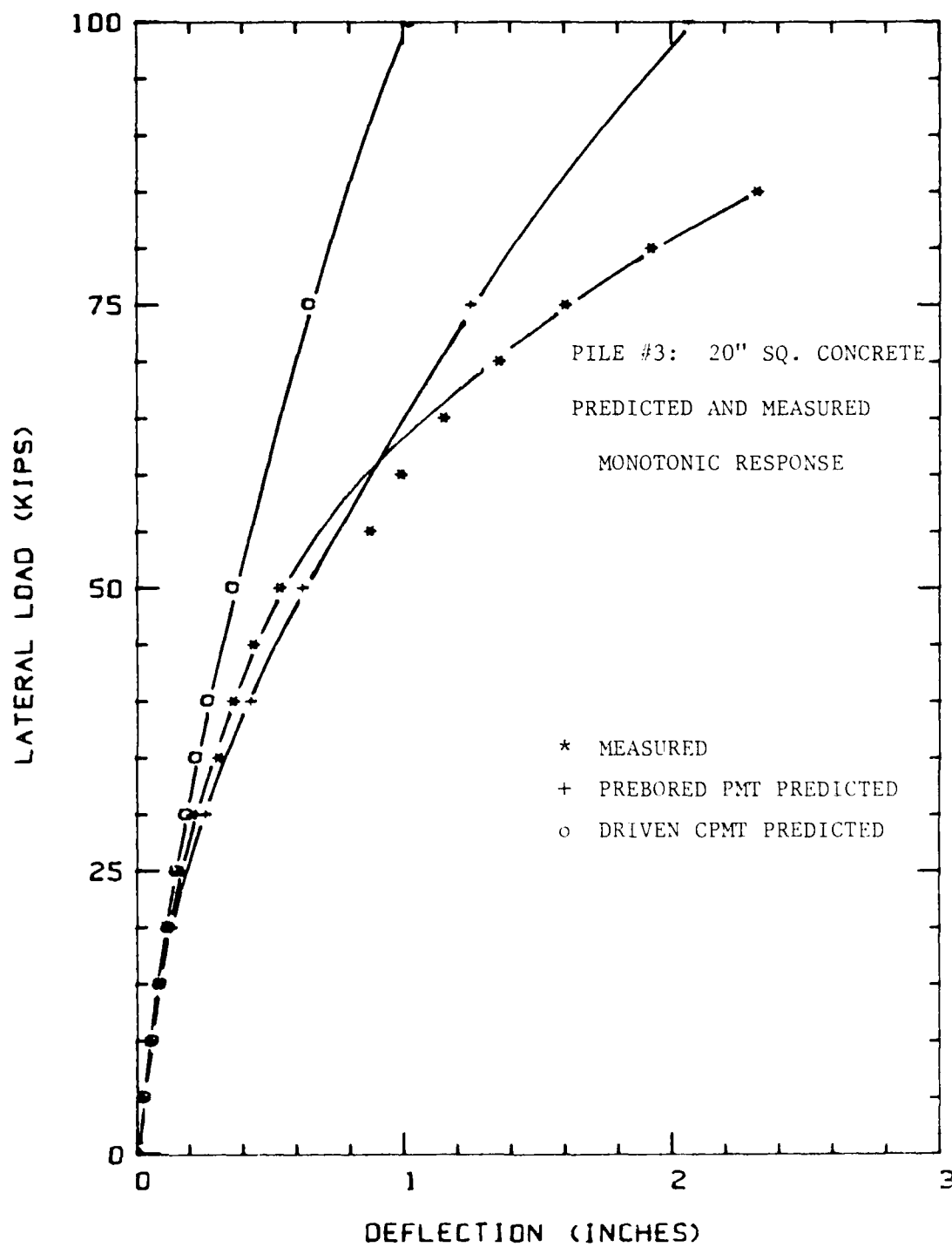
FIGURE 70. Comparison of Measured and PMT Predicted Monotonic Responses for Pile No. 3, 0 to 100 KIP Scale.
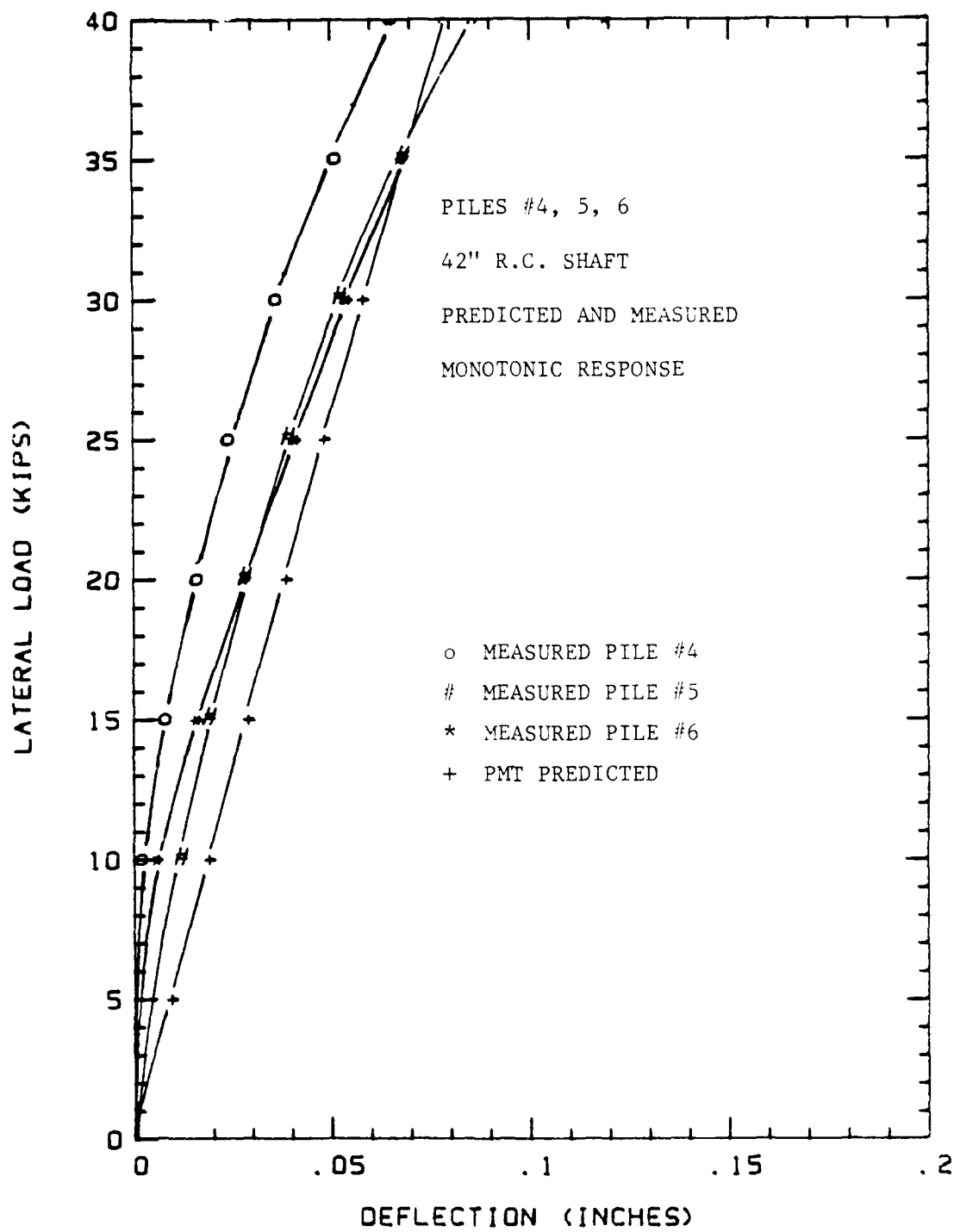
FIGURE 71   Comparison of Measured to PMT Predicted Monotonic
             Response for Pile Nos. 4, 5, and 6, 0 to 40 KIP
             Scale.
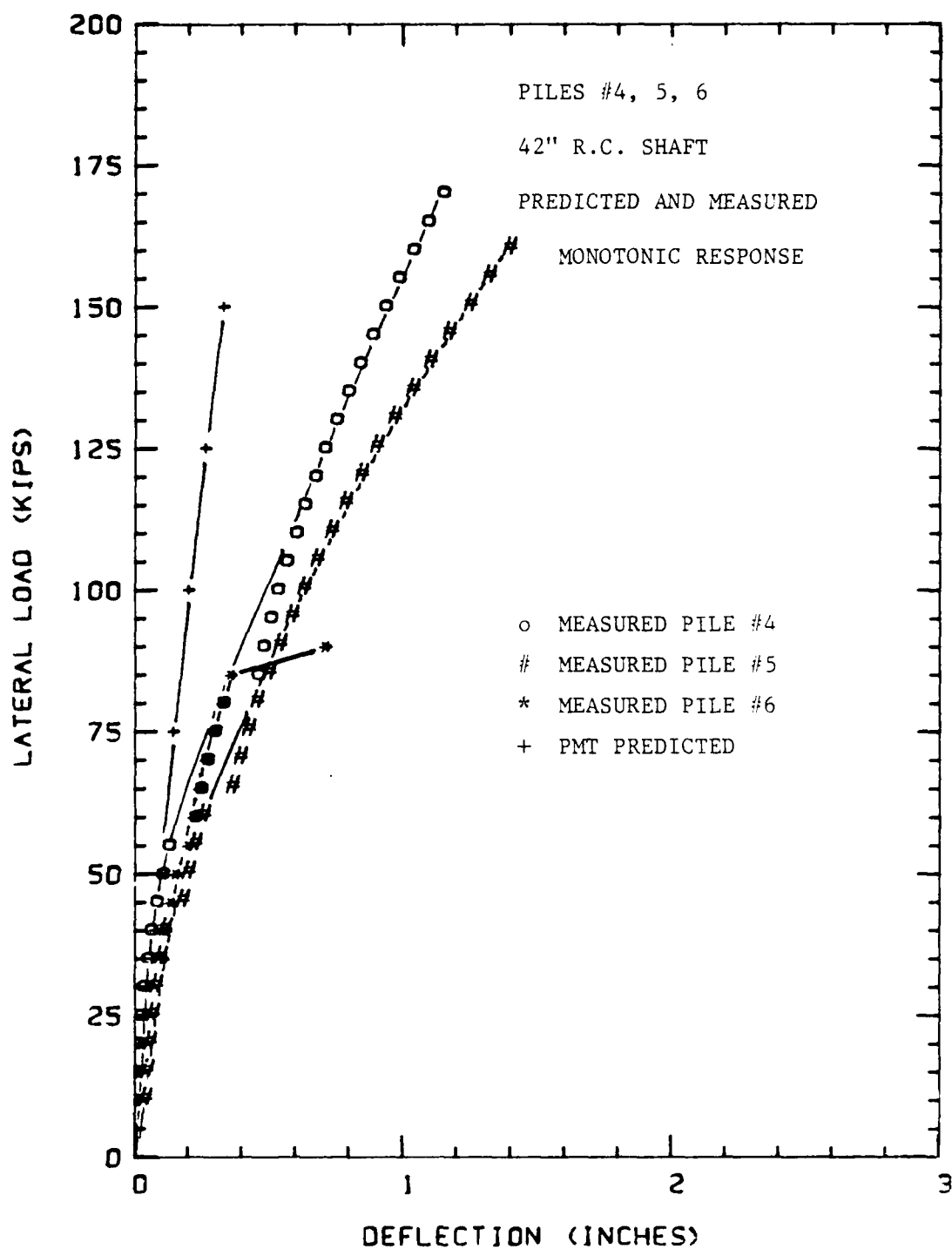
FIGURE 72.   Comparison of Measured to PMT Predicted
             Monotonic Response for Pile Nos. 4, 5, and 6,
             0 to 200 KIP Scale.

as good as the predictions for the other piles is the difficulty encountered in determining the correct pile stiffness to incorporate into the pile-soil stiffness model. As explained in Section 3, these shafts were used as reaction shafts during the previously performed vertical load tests. As such, they were subjected to extreme axial tension stresses. Although in the prediction method a reduced pile stiffness (EI-value) was assumed, it may not have been an accurate model of the actual piles. Judging from the excellent results in predicting the response of the other piles in this study, the less satisfactory results for the 42-in diameter drilled shafts must be due to inaccuracies in modelling the pile itself, and not in modeling the soil response.

6.2 Cyclic Loading Response

The predictions of the cyclic response of the test piles using the results of the cyclic PMT tests are shown in Figures 73 through 77. The predictions are presented as cyclic envelopes. For any given load level the Nth cyclic envelope represents the deflection expected after N cycles at that load level.

The cyclic prediction for the square prestressed concrete pile was obtained from both the preboring and the driven CPMT results. The cyclic predictions for the steel pipe pile and the drilled shafts were obtained from the preboring PMT only.

Table 5 summarizes the cyclic predictions and compares them to the measured responses. In each case, the predicted increase in deflection is less than the measured increase. Four possible reasons for the PMT method underpredicting the cyclic degradation are: (1) the difference in confinement between the pile and the PMT probe, (2) load-control cyclic pile load tests may not cause exclusively load-control cyclic loading of each soil strata, (3) influence of previ-

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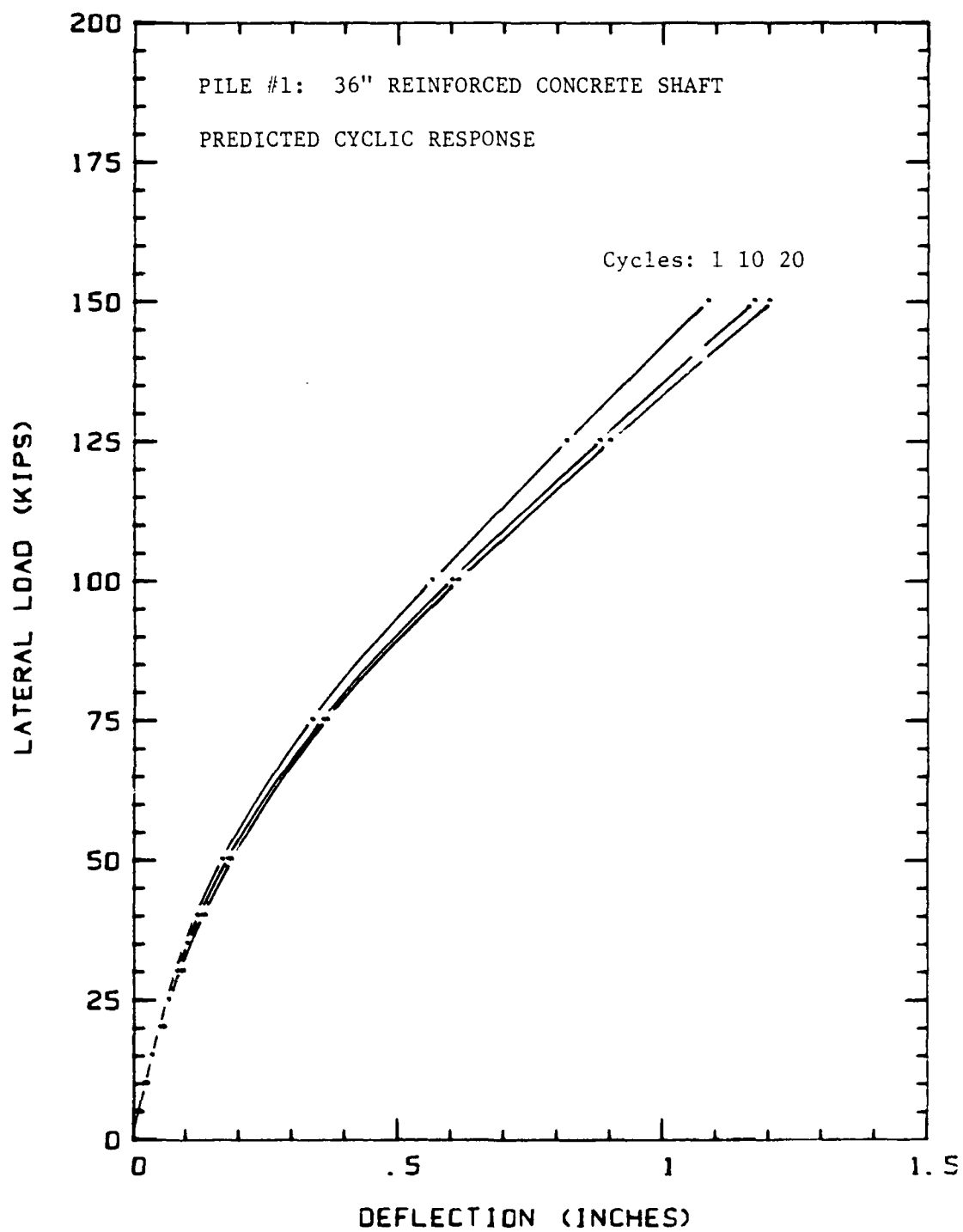
FIGURE 73. Prebored PMT Predicted Cyclic Response, Pile No. 1.

97
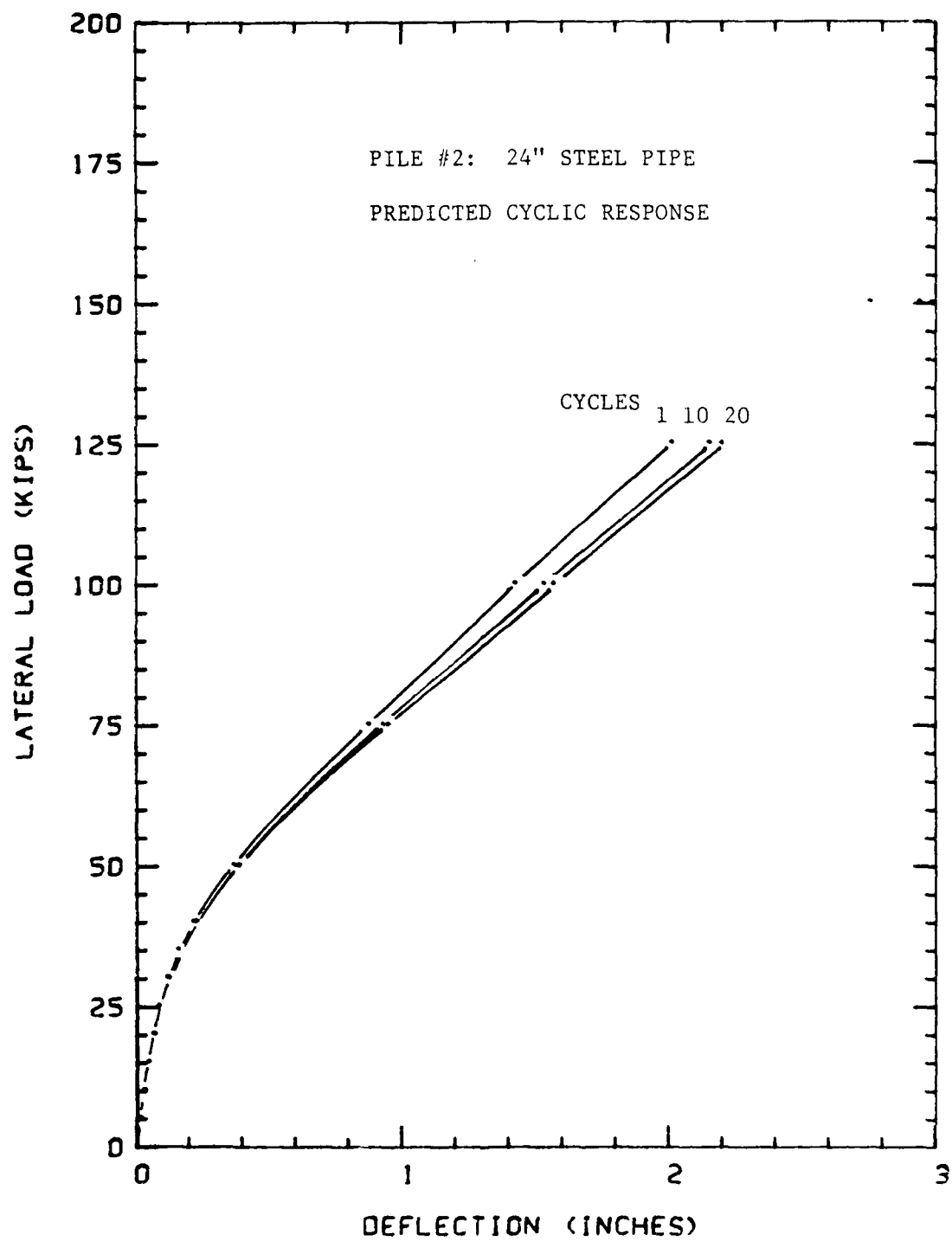
FIGURE 74. PMT Predicted Cyclic Response, Pile 2.
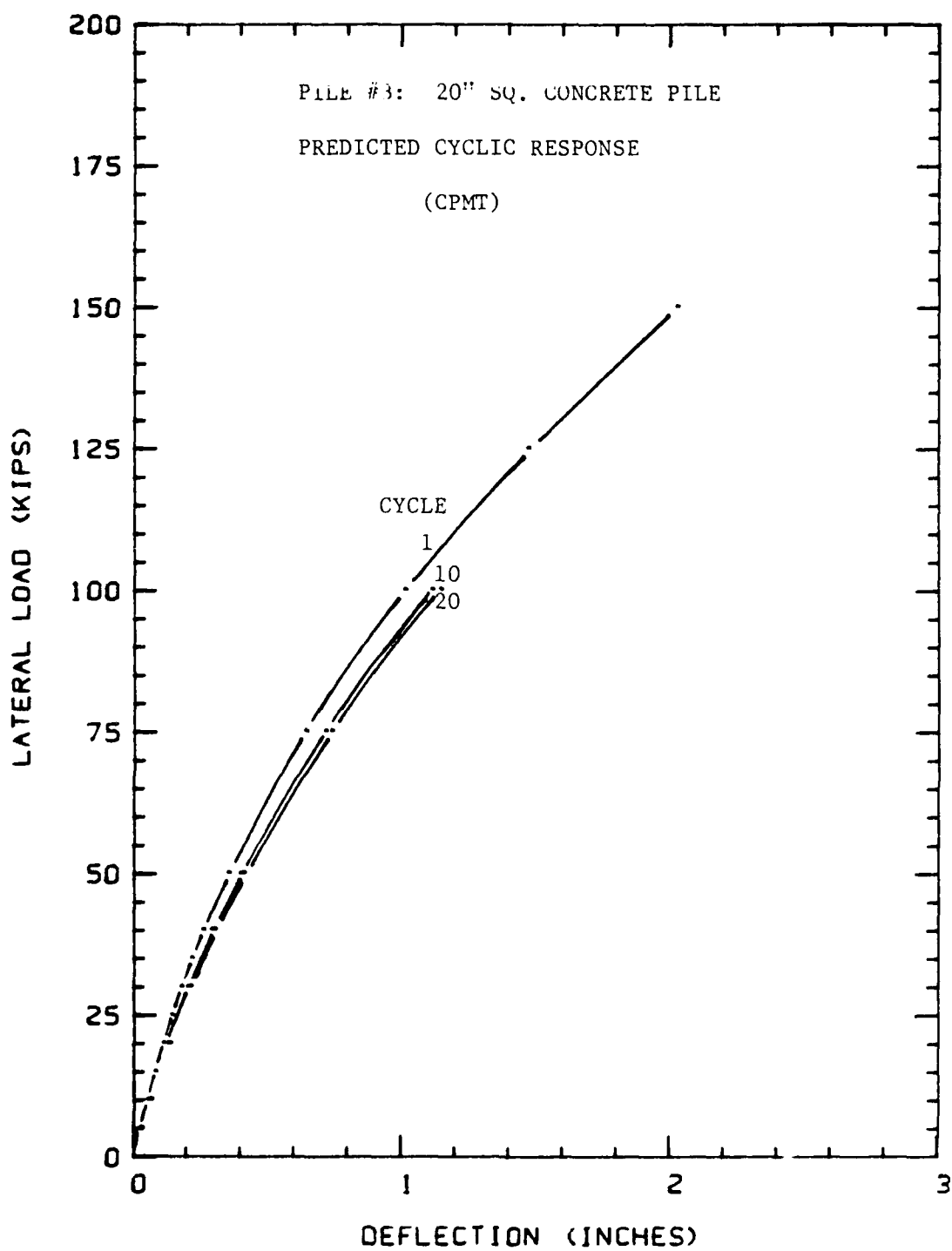
FIGURE 75. Driven CPMT Precicted Cyclic Response, Pile #3.

99
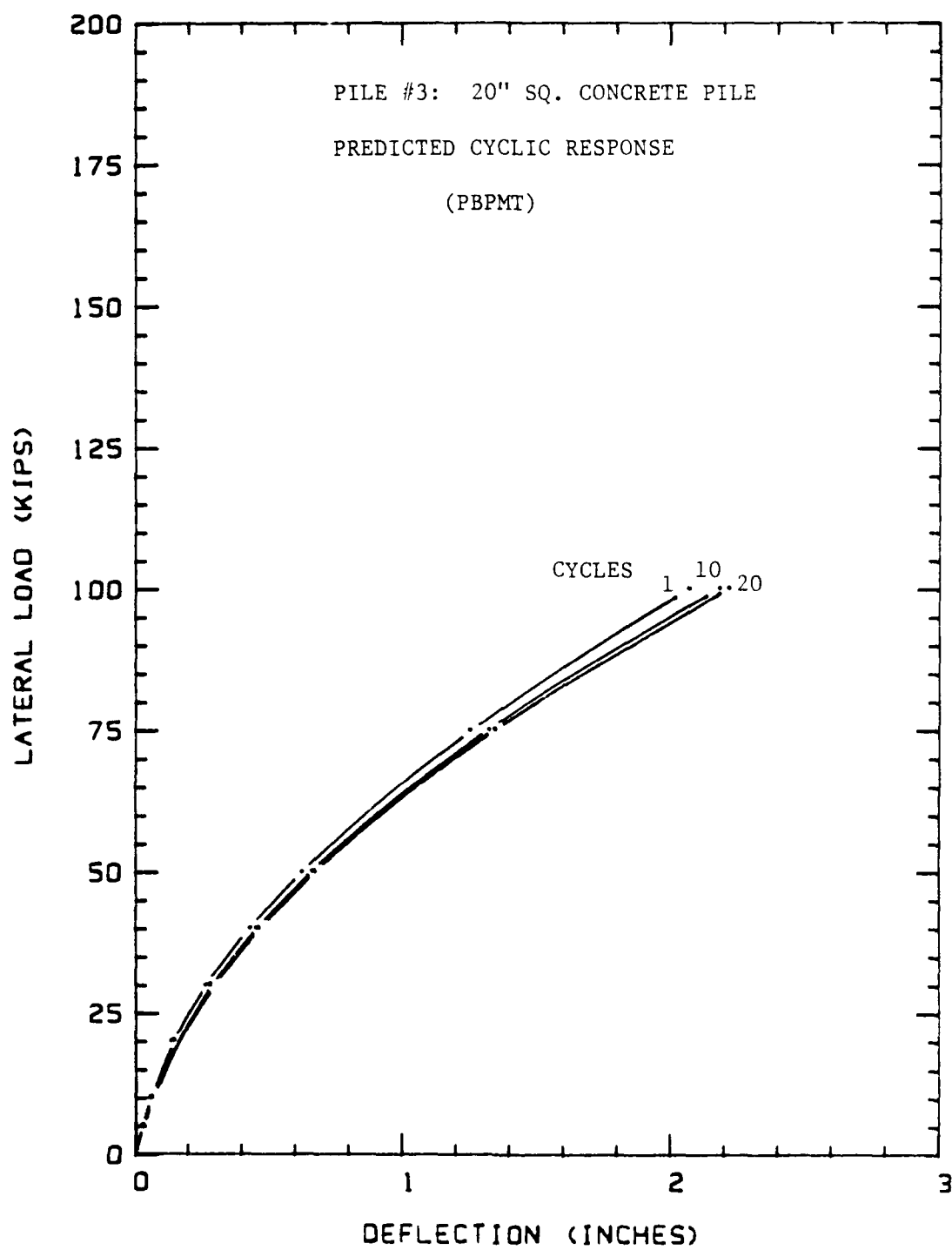
FIGURE 76.   Prebored PMT Predicted Cyclic Response, Pile #3.
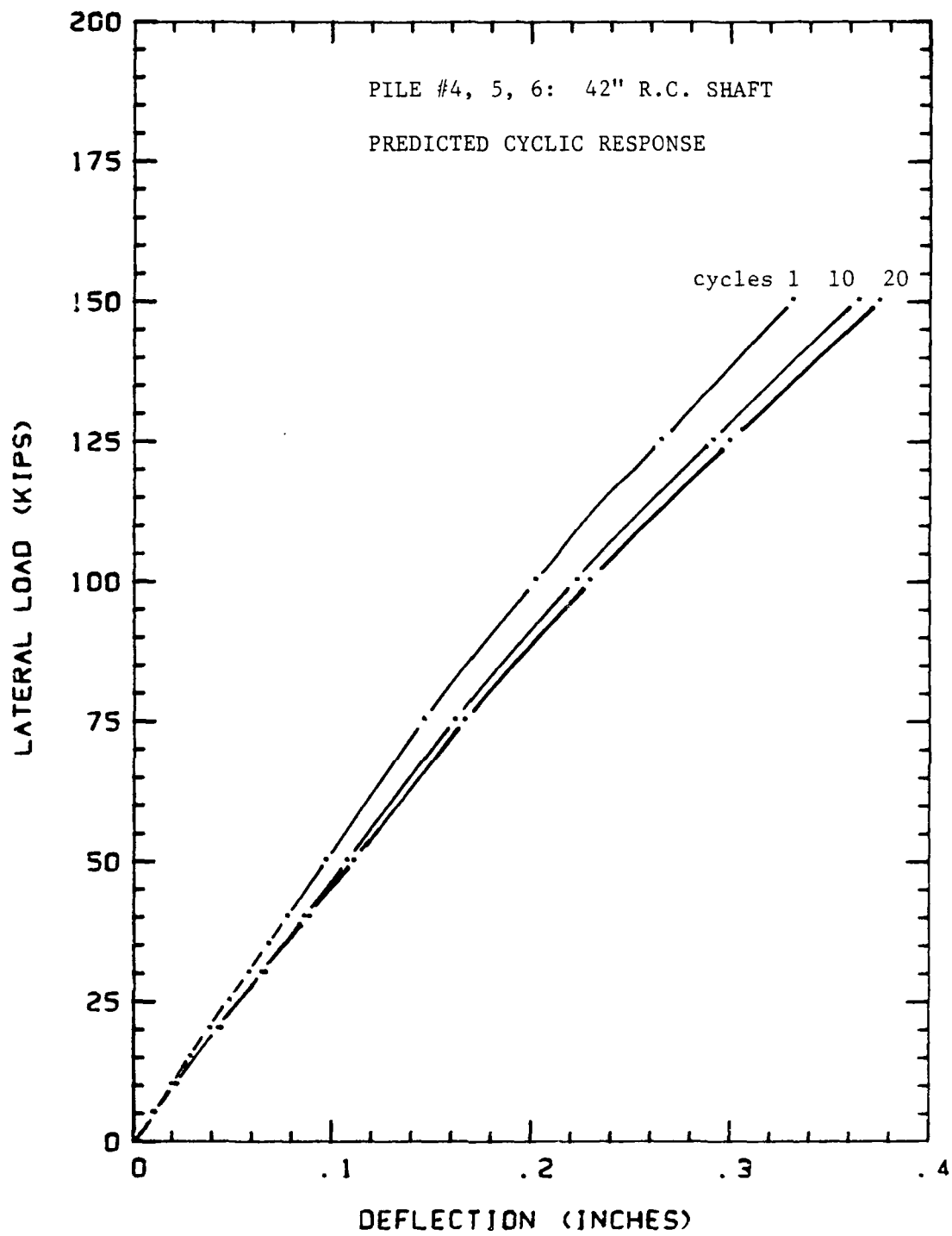
FIGURE 77. Prebored PMT Predicted Cyclic Response, Pile Nos. 4, 5, and 6.

TABLE 5.   Comparison of Percent Increase in Deflection with
           Cycling: Predicted and Measured

Pile ID No.	Pile Description	Cyclic Load Level (kips)	% Increase in Deflection			
			After 10 Cycles		After 20 Cycles	
			Measured	Predicted	Measured	Predicted
1	36" Drilled Shaft	55	51.1	8.5	66.9	11.6
		80	17.9	7.6	35.4	10.3
2	24" Pipe Pile	40	24.9	4.5	34.1	5.8
		60	15.2	4.2	25.9	7.9
3	20" Square Concrete	30	22.3	5.5	27.9	7.2
		50	26.4	6.0	43.1	7.9
4	42" Drilled Shaft	55	55.4	10.8	72.7	14.5
		80	17.8	10.5	28.3	14.1
5	42" Drilled Shaft	40	41.7	11.0	56.0	14.5
		60	19.5	10.6	34.1	14.5
6	42" Drilled Shaft	30	48.1	10.5	79.6	14.5
		50	11.5	11.0	17.9	14.5
	Averages		29.3	9.0	43.5	12.2

ous series of cycles on subsequent series of cycles during a
test and (4) degradation of the pile flexural stiffness
during cyclic loading.

There is a difference in confinement between the pile
and the PMT probe; this is shown in Figure 78. During the
lateral movement of a pile, the soil is able to move towards
the back of the pile where a gap is opening. During a PMT
test, the soil is displaced radially outward. Under mono-
tonic loading, the difference in confinement may not signi-
ficantly affect the pile-pressuremeter probe analogy. Under
repeated cyclic loading, however, the difference in confine-
ment may result in significantly greater degradation in the
soil resistance against the pile than against the pressure-
meter probe.

Another possible explanation for the pressuremeter
predicting less degradation under cyclic loading may arise
from the mode of cycling experienced by the soil during the
cyclic lateral loading of a pile.   In earlier studies
(Makarim and Briaud, 1986; Little and Briaud, 1987) it has

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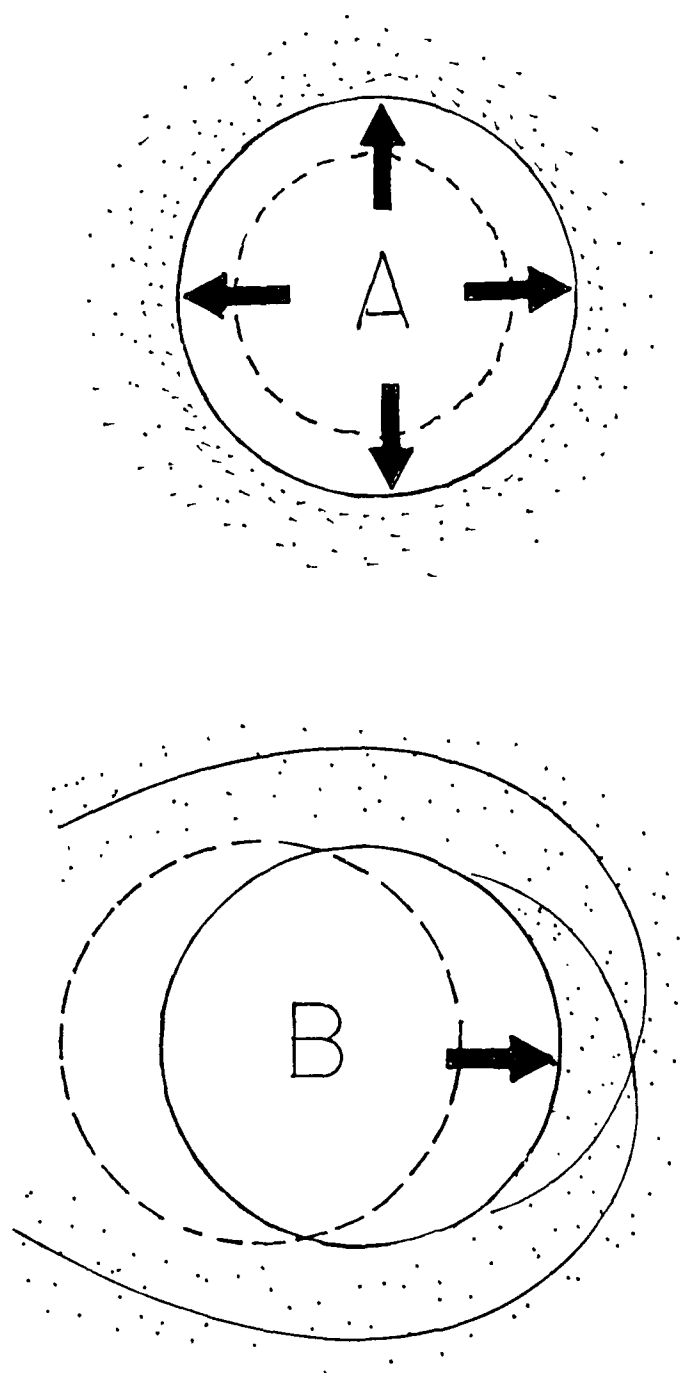
FIGURE 78. Difference in Confinement between the
PMT Probe Expansion (A) and the Lateral
Movement of a Pile (B).

been shown that for pressuremeter tests the soil resistance degrades more rapidly under displacement-controlled conditions than under pressure-controlled conditions. In this study, pressure-controlled cyclic pressuremeter tests were used to predict the soil resistance degradation for the load-controlled cyclic pile load tests. In reality, each soil layer may not be subjected exclusively to load-controlled conditions during a pile load test. The actual loading conditions on the soil may lie somewhere between pressure-control and displacement-control, making the pressure-control-predicted response the most conservative. It is recommended that future pressuremeter predictions be based on a combination of two cycling modes. One possible method may be to perform 10 cycles load-controlled and then 10 cycles displacement-controlled during the pressuremeter tests and calculate the degradation exponent for each mode of cyclic loading.

The influence of the first series of cycles on the soil response during subsequent cycles may also have affected the comparison between predicted and measured results. The first series of cycles in a test is more likely to be influenced by seating problems than subsequent series of cycles.

Particularly damaging to the prediction process employed in this study was the degradation of the pile flexural stiffness associated with cyclic loading. The deterioration of pile stiffness had a profound effect on the concrete piles studied in this report. This can be seen by comparing the measured increase in deflection after 20 cycles on concrete pile 4 at a load of 55 kips with the increase in deflection for concrete pile 5 at a load of 60 kips. The increase is 72.7% for pile 4 and 34.1% for pile 5. The piles were designed and constructed using identical procedures, and since they were installed within the same soil strata it would be logical to assume that the response of

the soil during the series of cycles would be nearly the same. The explanation for the large variation in response between the two identical piles lies in the relative degradation of their flexural stiffnesses.

The primary mechanism for reduction of the pile flexural stiffness was the propagation of cracks through the concrete cross-section during cycling. Increased cracking reduced the strength of the concrete. Crack propagation was most pronounced during the first cycling series for each of the reinforced concrete drilled shafts. After the concrete had suffered significant cracking during the first cycling series, the pile stiffness would tend toward a limit value since the stiffness would primarily be related to the strength of the steel reinforcement. Therefore, the difference mentioned above exists because pile 4 was being cycled for the first time while pile 5 was being cycled for the second time.

An alternative method for using the pressuremter results to predict the pile responses would be to treat the pressuremeter tests as a model pile load test and apply the PMT degradation parameter $\underline{a}$ directly to the monotonic prediction:

$$y(N) = y(1) \times N^a \tag{4}$$
$$H(N) = H(1) \tag{5}$$

where $y(N)$ = pile deflection at the groundline after N cycles at load H,

$\quad y(1)$ = pile deflection at the groundline after monotonic load H,

$\quad\quad a$ = average PMT degradation parameter,

$\quad H(N)$ = horizontal load applied at the top of the Nth cycle,

$\quad H(1)$ = horizontal load applied at the top of the 1st cycle,

$\quad\quad N$ = number of cycles.

The accuracy of this prediction method for the relatively homogeneous strata and piles in this study may be judged by comparing the measured average $\underline{a}$ values with the PMT predicted average of 0.064 (Table 6). This alternative method yielded excellent agreement between predicted average and measured average $\underline{a}$ values for the piles expected to have the least deterioration in pile stiffness, namely the steel pipe pile ($\underline{a}$ = 0.062) and the prestressed concrete pile ($\underline{a}$ = 0.063). The reinforced concrete drilled shafts, on the other hand, showed greater cyclic degradation than predicted ($\underline{a}$ values of 0.086, 0.080, 0.073 and 0.068 for piles 1,4,5 and 6, respectively).

TABLE 6. Comparison of Measured and Predicted Secant Shear Modulus Cyclic Degradation Parameters

Pile No.	Measured $a_{average}$	Predicted $a_{average}$
1	0.086	
2	0.062	
3	0.063	0.064
4	0.080	
5	0.073	
6	0.068	
Overall Average	0.072	0.064

6.3 Comparison of Creep Exponents

In Section 5.3.3 it was shown that the PMT creep exponents were 0.006 for the TEXAM preboring PMT tests and 0.011 for the driven CPMT tests. These exponents were lower than the exponents backcalculated from the pile load tests since these exponents stabilized around 0.015 to 0.02. The discrepancy may be the result of two of the same mechanisms cited for the underprediction of cyclic degradation, namely the difference in confinement between the pile and the pressuremeter probe and the creep of the pile flexural stiffness under a sustained load.

CONCLUSIONS AND RECOMMENDATIONS

The main conclusions to be gathered from this study are the following:

- The four drilled shafts exhibited significantly more cyclic degradation during the first series of cycles than during the second series. This may be due to two things. First, the pile stiffness may degrade due to crack propagation from the cycling. By the second series of cycles, the pile stiffness is mainly obtained from the steel reinforcement which will not exhibit much degradation. This is thought to be the major cause. Second, the sand is densified by the first series of cycling, causing a stiffening of the response in the second series.

- The steel pipe pile showed a somewhat stiffer response during the second cyclic series than in the first. Since the stiffness of the steel should experience little or no degradation at these low load levels, the increase in stiffness is probably due entirely to the stiffening of the soil due to previous cyclic loads.

- The prestressed concrete pile showed more degradation of stiffness during the second cyclic series than in the first. This is thought to be caused by a postponement of cracking of the concrete due to the prestressing of the pile. The bending moment in the pile at the lower cyclic load level was not enough to cause tension cracks in the concrete. However, at the second, higher, cyclic load level, the effect of the prestressing was overcome and tension cracks began to form, thus reducing the pile stiffness.

- The cyclic degradation parameter, "a", values for cycles which unloaded to zero load were 53% higher

than for cycles which unloaded only to one-half the top cyclic load.

- The creep exponent, "n", values for all the piles except the prestressed concrete pile exhibited the same behavior. The n values started between 0.05 and 0.075 then reduced and stabilized between 0.015 and 0.02. For the 42-in diameter drilled shaft which failed, the n values showed an upward turn towards the end of the test indicative of the impending failure. The n values for the prestressed concrete pile began around 0.015 then increased during the test, reaching a critical load at about 90 kips. This may be due to creep in the concrete as the effect of the prestressing is overcome.

- The prediction of the monotonic loading curves by the pressuremeter method (Little and Briaud, 1987) was very good for all piles in the working load range based on the prebored PMT test data. As the loads increased, the measured deflections of the drilled shafts increased much faster than the predictons due to crack propagation in the concrete. The predictions for the steel pipe pile were good throughout the entire loading range. The predictions were too stiff for the prestressed concrete pile at larger loads using both the prebored PMT and the driven CPMT test curves, with the prebored data yielding the best results of the two. From these results it can be concluded that the pressuremeter method predicts well the soil response but does not include any pile stiffness degradation.

- The conventional P-y curves overpredicted the displacement of the piles throughout the entire loading range.

- For all piles the predicted increase in deflection due to cyclic loading was much less than the measured increase. Four possible reasons for this difference are: (1) the difference in confinement between the pile and the pressuremeter, (2) load-control cyclic pile load tests may not cause exclusively load-control cyclic loading of each soil strata, (3) influence of previous series of cycles on subsequent series of cycles during a test and (4) degradation of the pile flexural stiffness during cyclic loading.

- The average cyclic degradation parameter from the pressuremeter tests (a = 0.064) matches very well the average cyclic degradation parameter from the pile load tests which should experience little or no degradation of the pile flexural stiffness, namely the steel pipe pile (a = 0.062) and the prestressed concrete pile (a = 0.063). The reinforced concrete drilled shafts showed much higher cyclic degradation.

- The creep exponents from the PMT tests were 0.006 for the prebored TEXAM tests and 0.011 for the driven CPMT tests. These exponents were lower than the creep exponents backcalculated from the pile load tests which stabilized around 0.015 to 0.02. The difference may be caused by two of th mechanisms cited for the underprediction of cyclic degradation, namely the difference in confinement between the pile and the pressuremeter probe and the creep of the pile flexural stiffness under a sustained load.

The following recommendations are made based on the results of this study:

- The pressuremeter method used in this study for predicting pile response to monotonic lateral loading (Little and Briaud, 1987) is applicable to piles which will experience little or no degradation in

flexural stiffness (such as steel piles, prestressed concrete piles loaded less than the prestress, etc.) due to the applied loading.

- Further study needs to be done in four main areas in order to apply the pressuremeter method to cyclic and creep loading. These areas are (1) the effects of the difference in confinement between the pile and the pressuremeter probe, (2) determining what type of loading each soil strata actually undergoes due to various loading at the top of the pile, (3) the influence of previous series of cycles on subsequent series of cycles and (4) the degradation of the pile flexural stiffness during loading (monotonic, cyclic and creep).

The fourth item relating to degradation of the pile flexural stiffness during loading is felt to be the most critical factor for prediction of the behavior of reinforced concrete drilled shafts.

REFERENCES

BRIAUD ENGINEERS, 1986, "Pressuremeter and Cone Penetrometer Testing for SH 146 Bridge Over the Houston Ship Channel," Report for the Texas State Department of Highways and Public Transportation, College Station, Texas.

LITTLE, R.L. AND BRIAUD, J.-L., 1987, "A Pressuremeter Method for Single Piles Subjected to Cyclic Lateral Loads in Sand," Research Report No. 5357, Civil Engineering Department, Texas A&M University, College Station, Texas.

MAKARIM, C.A. AND BRIAUD, J.-L., 1986, "Pressuremeter Method for Single Piles Subjected to Cyclic Lateral Loads in Overconsolidated Clay," Research Report No. 5112, Civil Engineering Department, Texas A&M University, College Station, Texas.

McCLELLAND ENGINEERS, 1986, Private communication.

REESE, L.C., COX, W.R. AND KOOP, F.D., 1974, "Analysis of Laterally Loaded Piles in Sand," 6th Offshore Technology Conference, Houston, Texas.

WANG, C.K. AND SALMON, C.G., 1979, Reinforced Concrete Design, 3rd edition, Thomas Y. Crowell, Harper and Row Publishers, New York, New York.

APPENDIX A

PILE LOAD TEST DATA

TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: 20 inch SQUARE CONCRETE   Date: 1-6-87

Reaction Pile: 42 inch DRILLED SHAFT

Displacement gage locations:
#1: 3/8" TO RIGHT OF LOAD AXIS & 21 1/16 ABOVE (ON BACK SIDE)
#2: 1/4" TO RIGHT OF LOAD AXIS & 8 3/4" BELOW (OF PILE)
#3: 1 3/4" TO RIGHT OF LOAD AXIS & 8 1/2" ABOVE (ON INSIDE)
#4: 2 1/8" TO RIGHT OF LOAD AXIS & 9 1/8" BELOW (OF PILE)

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
0	4?0	560	7.800	6.428	0.143	0.026
30s	5600	800	7.794	6.411	0.148	0.028
1:30	4950	800	7.794	6.411	0.148	0.028
2:30	5050	800	7.794	6.410	0.148	0.029
3:30	5000	800	7.794	6.410	0.148	0.029
4:30	5000	800	7.794	6.410	0.148	0.029
5:30	10000	1080	7.754	6.384	0.154	0.032
6:30	10000	1080	7.756	6.363	0.154	0.032
7:30	10000	1080	7.755	6.383	0.154	0.032
8:30	"	"	7.755	6.383	0.154	0.032
9:30	"	"	7.754	6.383	0.154	0.032
10:30	"	"	7.754	6.381	0.154	0.032
11:30	15050	1380	7.707	6.350	0.162	0.039
12:30	15080	"	7.704	6.348	0.162	0.039
13:50	"	"	7.703	6.348	0.163	0.040
14:30	"	"	7.703	6.347	0.164	0.041
15:30	"	"	7.703	6.347	0.164	0.041
16:30	"	"	7.702	6.347	0.165	0.042
17:30	20550	1500	7.648	6.314	0.173	0.049
18:30	"	"	7.643	6.213	0.174	0.050
19:30	"	"	7.641	6.212	0.176	0.053
20:30	"	"	7.640	6.212	0.176	0.053
21:30	"	"	7.639	6.211	0.177	0.054
22:30	"	"	7.639	6.211	0.177	0.054
23:30	25000	1700	7.581	6.272	0.186	0.061
24:30	"	"	7.576	6.269	0.187	0.062
25:30	"	"	7.571	6.267	0.186	0.064
26:30	"	"	7.572	6.266	0.187	0.065
27:30	"	"	7.571	6.265	0.189	0.066
28:30	"	"	7.569	6.265	0.189	0.067
29:30	30000	1920	7.512	6.222	0.199	0.076
30:30	"	"	7.506	6.219	0.200	0.078
31:30	"	"	7.503	6.216	0.201	0.078
32:30	"	"	7.500	6.215	0.202	0.078
33:30	"	"	7.498	6.214	0.202	0.079
34:30	"	"	7.496	6.213	0.202	0.080

13:23

Left margin notes:
At 1:20 Jack for reseal of ... and check the 6 minutes over ~20 s

load starts going / immediately after this ... and takes ~20 s to go to next load.

hairline crack gap behind

Square pile twist slightly 1/8" hor.

TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Date: Jan 6, 87.

Time mm:sec.	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
36:30	650	600	7.725	6.373	0.166	0.054
38:30	30000	1900	7.482	6.202	0.205	0.083
40:30	650	600	7.720	6.364	0.168	0.054
42:30	30000	1900	7.471	6.194	0.208	0.086
44:30	550	600	7.708	6.360	0.173	0.061
46:30	30000	1920	7.464	6.187	0.212	0.091
48:30	660	600	7.706	6.354	0.175	0.063
50:30	30000	1920	7.460	6.183	0.214	0.092
52:30	660	600	7.699	6.348	0.177	0.064
54:30	30000	1920	7.455	6.175	0.216	0.094
56:30	550	600	7.694	6.345	0.178	0.066
58:30	30000	1920	7.456	6.273	0.218	0.096
60:30	550	600	7.690	6.343	0.180	0.068
62:30	30000	1900	7.450	6.170	0.220	0.097
64:30	550	600	7.687	6.336	0.183	0.073
66:30	30000	1920	7.455	6.165	0.224	0.104
68:30	600	600	7.696	6.335	0.186	0.078
70:30	30050	1920	7.453	6.165	0.226	0.106
72:30	600	600	7.616	6.333	0.186	0.080
74:30	30000	1920	7.450	6.166	0.217	0.109
76:30	14850	1200	7.545	6.231	0.210	0.099
78:30	30000	1920	7.447	6.164	0.230	0.112
80:30	14950	1150	7.538	6.229	0.212	0.102
82:30	30000	1900	7.444	6.161	0.231	0.118
84:30	14850	1100	7.533	6.225	0.212	0.104
86:30	30000	1920	7.441	6.159	0.231	0.118
88:30	14950	1150	7.536	6.221	0.213	0.104
90:30	30000	1900	7.440	6.157	0.232	0.119
92:30	14900	1120	7.533	6.221	0.214	0.105
94:30	30000	1900	7.437	6.155	0.231	0.119
96:30	15000	1120	7.528	6.217	0.214	0.104
98:30	30000	1900	7.436	6.152	0.232	0.119
100:30	15000	1150	7.528	6.217	0.215	0.106
102:30	30000	1900	7.435	6.154	0.234	0.121
104:30	14950	1150	7.527	6.217	0.216	0.109
106:30	30000	1900	7.434	6.153	0.234	0.120
108:30	14950	1150	7.527	6.217	0.216	0.109
110:30	30000	1900	7.433	6.153	0.234	0.123
112:30	15000	1150	7.525	6.216	0.218	0.110
114:30	30000	1900	7.433	6.153	0.232	0.124
115:30	35000	2120	7.389	6.125	0.245	0.130
116:30	"	"	7.385	6.122	0.247	0.133

Margin notes:
hoist
2nd jack
Soil crack in back of reaction shaft
15:02
bottom 19
top 20
bottom 20
top 21

Continuation of: Date:

Time (min:sec)	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (lower)(in)	Reaction Pile Disp. #3 (in)	#4 (upper)(in)
117:30	35000	2120	7.382	6.121	0.248	0.133
118:30	"	"	7.380	6.120	0.248	0.134
119:30	"	2150	7.379	6.119	0.248	0.134
120:30	"	"	7.377	6.117	0.249	0.134
121:30	40,000	2500	7.317	6.076	0.260	0.142
122:30	"	"	7.308	6.071	0.262	0.144
123:30	"	"	7.304	6.069	0.263	0.144
124:30	"	2480	7.301	6.066	0.264	0.145
125:30	"	"	7.299	6.065	0.264	0.145
126:30	"	"	7.299	6.064	0.264	0.145
127:30	45000	2700	7.233	6.010	0.278	0.156
128:30	"	2680	7.221	6.003	0.281	0.160
129:30	"	"	7.213	5.997	0.281	0.163
130:30	"	"	7.208	5.994	0.282	0.164
131:30	"	"	7.205	5.991	0.282	0.164
132:30	"	"	7.202	5.989	0.283	0.164
133:30	50000	2850	7.111	5.919	0.298	0.175
134:30	"	"	7.099	5.909	0.302	0.178
135:30	"	"	7.087	5.901	0.304	0.180
136:30	"	2840	7.081	5.896	0.305	0.181
137:30	"	"	7.075	5.893	0.306	0.181
138:30	"	"	7.071	5.890	0.307	0.182
140:30	25150	1580	7.285	6.022	0.266	0.155
142:30	50000	2820	7.046	5.860	0.309	0.187
144:30	25100	1580	7.266	6.001	0.268	0.160
146:30	50000	2800	7.019	5.836	0.311	0.190
148:30	25050	1560	7.246	5.984	0.270	0.161
150:30	50000	2800	6.989	5.821	0.312	0.192
152:30	24900	1580	7.222	5.964	0.273	0.164
154:30	50000	2800	6.964	5.804	0.314	0.195
156:30	25100	1580	7.212	5.954	0.272	0.164
158:30	50000	2800	6.952	5.790	0.315	0.195
160:30	25150	1580	7.199	5.943	0.273	0.165
162:30	50000	2800	6.935	5.778	0.316	0.197
164:30	25100	1600	7.184	5.936	0.273	0.166
166:30	50000	2800	6.918	5.768	0.316	0.198
168:30	25050	1580	7.173	5.925	0.274	0.166
170:30	50000	2800	6.904	5.757	0.317	0.198
172:30	25100	1580	7.159	5.916	0.275	0.167
174:30	50000	2800	6.893	5.748	0.318	0.200
176:30	25050	1600	7.173	5.921	0.272	0.165
178:30	50000	2800	6.880	5.742	0.316	0.199

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TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Date:

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
180:30	950	580	7.505	6.157	0.217	0.118
182:30	50000	2800	6.891	5.740	0.317	0.199
184:30	950	580	7.500	6.153	0.217	0.119
186:30	50000	2800	6.877	5.729	0.320	0.200
188:30	1000	580	7.494	6.145	0.217	0.119
190:30	50000	2800	6.858	5.718	0.322	0.201
192:30	850	580	7.490	6.133	0.218	0.121
194:30	50000	2800	6.842	5.805	0.325	0.204
196:30	1100	580	7.472	6.122	0.220	0.132
198:30	50000	2800	6.827	5.694	0.326	0.205
2n:30	1250	600	7.459	6.109	0.221	0.124
202:30	50000	2800	6.816	5.683	0.328	0.207
204:30	1050	600	7.455	6.107	0.220	0.124
206:30	50000	2800	6.805	5.675	0.328	0.209
208:30	1000	600	7.447	6.100	0.220	0.123
210:30	50000	2800	6.797	5.670	0.327	0.210
212:30	850	600	7.439	6.092	0.221	0.124
214:30	50000	2800	6.782	5.658	0.331	0.210
216:30	900	600	7.424	6.079	0.222	0.125
218:30	50000	2800	6.770	5.650	0.332	0.210
219:30	55000	3040	6.680	5.575	0.350	0.223
220:30	"	"	6.667	5.567	0.352	0.226
221:30	- No Rdg -	---	---	---	---	---
222:30	55000	3050	6.654	5.560	0.355	0.229
223:30	"	3050	6.650	5.556	0.355	0.230
224:30	"	"	6.646	5.553	0.356	0.230
225:30	60000	3240	6.535	5.475	0.372	0.241
226:30	"	3250	6.515	5.457	0.376	0.244
227:30	"	"	6.504	5.449	0.378	0.245
228:30	"	"	6.497	5.444	0.380	0.247
229:30	"	"	6.492	5.440	0.381	0.247
230:30	"	3240	6.485	5.437	0.381	0.247
231:30	65000	3490	6.343	5.327	0.399	0.263
232:30	"	"	6.319	5.308	0.404	0.266
233:30	"	"	6.304	5.297	0.406	0.268
234:30	"	"	6.290	5.286	0.408	0.271
235:30	"	"	6.285	5.281	0.409	0.271
236:30	"	"	6.277	5.277	0.410	0.271
237:30	70000	3680	6.690	5.120	0.432	0.286
238:30	"	"	6.660	5.109	0.436	0.290
239:30	"	3660	6.637	5.096	0.440	0.292
240:30	"	"	6.627	5.086	0.442	0.294

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TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Date:

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (6.42 at) (in)	Reaction Pile Disp. #3 (in)	#4 (in)
241:30	70,000	3660	6.013	5.077	0.443	0.295
242:30	"	"	6.003	5.071	0.445	0.295
243:30	75000	3840	5.772	4.911	0.466	0.311
244:30	"	3820	5.727	4.883	0.473	0.315
245:30	"	"	5.705	4.863	0.476	0.318
246:30	"	"	5.687	4.845	0.479	0.320
247:30	"	"	5.672	4.834	0.480	0.324
248:30	"	"	5.658	4.825	0.481	0.325
249:30	80000	4080	5.406	4.622	0.502	0.341
250:30	"	4100	5.331	4.576	0.511	0.346
251:30	"	"	5.302	4.551	0.515	0.349
252:30	"	"	5.276	4.532	0.517	0.351
253:30	"	"	5.242	4.510	0.520	0.353
254:30	"	"	5.232	4.503	0.522	0.355
255:30	85000	4380	4.891	4.266	0.545	0.371
256:30	"	4400	4.803	4.206	0.553	0.376
257:30	"	"	4.757	4.170	0.558	0.380
258:30	"	"	4.721	4.141	0.561	0.382
259:30	"	"	4.696	4.124	0.564	0.384
260:30	"	"	4.669	4.107	0.566	0.386
261:30	90000	4600	4.195	3.840	0.591	0.405
262:30	"	4650	4.015	3.630	0.598	0.411
263:30	"	4680	3.850	3.510	0.603	0.415
264:30	"	4720	3.655	3.380	0.633 x	0.450 x
265:30	"	"	3.418	3.200	0.753	0.550
266:30	"	"	3.040	2.960	>1.000	0.740 9
—	00000	Reading at "No Load" condition after test			—	

major cracks sq. pile (1/4 in wide, 1 ft long)

TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _24" DIA. OPEN ENDED PIPE PILE_   Date: _1-7-87_

Reaction Pile: _42" DIA DRILLED SHAFT_

Displacement gage locations:
#1: _11 9/16" ABOVE LOAD AXIS, 1/4" TO THE RIGHT_ (AS FACING OUTSIDE OF PILE)
#2: _4 15/16" BELOW LOAD AXIS, 3/8" TO THE RIGHT_ (OF PILE)
#3: _11 7/8" ABOVE LOAD AXIS, 7/16" TO THE LEFT_ (AS FACING INSIDE OF PILE)
#4: _4 9/16" BELOW LOAD AXIS, 7/16" TO THE LEFT_ (OF PILE)

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
—	00000	— "NO LOAD" READING BEFORE TEST SET UP				
0	0	500	+5.548	+6.521	-0.008	+0.036
30 sec	5150	800	5.529	6.520	-0.002	0.040
1:30	5000	800	5.528	6.520	-0.002	0.040
2:30	"	"	5.528	6.510	-0.002	0.040
3:30	"	"	5.528	6.520	-0.002	0.040
4:30	"	780	5.528	6.520	-0.002	0.040
5:30	10000	1000	5.501	6.502	0.006	0.045
6:30	"	"	5.500	6.502	0.006	0.046
7:30	"	"	5.500	6.501	0.006	0.046
8:30	"	"	5.499	6.501	0.006	0.046
9:30	"	"	5.499	6.500	0.006	0.046
10:30	"	"	5.499	6.500	0.007	0.046
11:30	15000	1320	5.468	6.478	0.015	0.052
12:30	"	"	5.466	6.477	0.015	0.053
13:30	"	"	5.465	6.477	0.016	0.053
14:30	"	"	5.465	6.476	0.016	0.053
15:30	"	"	5.464	6.476	0.016	0.053
16:30	"	"	5.464	6.476	0.016	0.053
17:30	20000	1420	5.429	6.449	0.026	0.061
18:30	"	"	5.427	6.447	0.027	0.061
19:30	"	"	5.426	6.446	0.027	0.061
20:30	"	"	5.425	6.446	0.027	0.061
21:30	"	"	5.424	6.446	0.027	0.062
22:30	"	"	5.424	6.446	0.028	0.062
23:30	25000	1680	5.384	6.415	0.037	0.070
24:30	"	"	5.380	6.412	0.040	0.071
25:30	"	"	5.378	6.411	0.041	0.072
26:30	"	"	5.377	6.410	0.041	0.072
27:30	"	"	5.376	6.409	0.042	0.073
28:30	"	"	5.375	6.408	0.042	0.073
29:30	30000	1900	5.331	6.374	0.055	0.084
30:30	"	"	5.327	6.374	0.057	0.085
31:30	"	"	5.324	6.372	0.058	0.085
32:30	"	"	5.323	6.371	0.058	0.085
33:30	"	1910	5.322	6.370	0.058	0.085

11:50

TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Pipe pile   Date: 1-7-87

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (=521) (in)	Reaction Pile Disp. #3 (in)	#4 (=0.036) (in)
34:30	30,000	1920	5.321	6.370	0.059	2.086
35:30	35000	2200	5.274	6.334	0.073	0.098
36:30	"	"	5.270	6.330	0.075	0.099
37:30	"	"	5.267	6.327	0.076	0.101
38:30	"	"	5.265	6.325	0.077	0.101
39:30	"	"	5.264	6.324	0.078	0.101
40:30	"	"	5.262	6.322	0.078	0.102
41:30	40000	2420	5.215	6.285	0.093	0.115
42:30	"	"	5.209	6.280	0.096	0.117
43:30	"	"	5.205	6.278	0.097	0.119
44:30	"	"	5.203	6.276	0.098	0.120
45:30	"	"	5.201	6.274	0.099	0.120
46:30	"	"	5.198	6.272	0.100	0.120
48:30	650	520	5.466	6.464	0.017	0.078
50:30	40000	2420	5.180	6.256	0.100	0.135
52:30	650	520	5.455	6.455	0.020	0.084
54:30	40000	2420	5.165	6.245	0.103	0.142
56:30	650	520	5.448	6.449	0.022	0.083
58:30	40000	2420	5.153	6.232	0.106	0.145
60:30	550	520	5.444	6.444	0.025	0.085
62:30	40000	2420	5.146	6.226	0.109	0.146
64:30	550	520	5.440	6.440	0.026	0.086
66:30	40000	2410	5.140	6.221	0.111	0.148
68:30	600	520	5.437	6.438	0.028	0.088
70:30	40000	2420	5.136	6.219	0.113	0.152
72:30	650	540	5.433	6.434	0.030	0.089
74:30	40000	2420	5.130	6.215	0.115	0.152
76:30	650	520	5.430	6.432	0.031	0.091
78:30	40000	2420	5.127	6.211	0.117	0.154
80:30	600	520	5.428	6.431	0.032	0.091
82:30	40000	2420	5.123	6.210	0.119	0.155
84:30	800	540	5.425	6.428	0.033	0.092
86:30	40000	2450	5.132	6.207	0.119	0.155
88:30	20000	1350	5.229	6.278	0.082	0.130
90:30	40000	2480	5.113	6.201	0.123	0.59
92:30	30000	1340	5.221	6.271	0.085	0.133
94:30	40000	2480	5.110	6.197	0.124	0.160
96:30	20050	1350	5.217	6.267	0.086	0.134
98:30	40000	2450	5.107	6.195	0.126	0.161
100:30	20000	1350	5.214	6.265	0.088	0.135
102:30	40000	2450	5.103	6.192	0.127	0.162
104:30	20050	350	5.211	6.262	0.089	0.136

Margin notes:
- 1/8" gap behind pipe pile
- bottom of 1 (at 48:30)
- Cycle numbers in left margin: 2 (52:30), 3 (56:30), 4 (60:30), 5 (64:30), 6 (68:30), 7 (72:30), 8 (76:30), 9 (80:30), 10 (84:30), 11 (88:30), 12 (92:30), 13 (96:30), 14 (100:30), 5 (104:30)

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TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Pipe Pile   Date: 1-7-87

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in) (5.568)	#2 (in) (6.521)	Reaction Pile Disp. #3 (in)	#4 (in) (0.0xx)
106:30	40000	2460	5.102	6.191	0.128	0.163
108:30	20050	1350	5.210	6.262	0.090	0.137
110:30	40000	2450	5.100	6.189	0.129	0.164
112:30	20050	1340	5.207	6.260	0.092	0.139
114:30	40000	2450	5.098	6.188	0.131	0.165
116:30	20050	1350	5.206	6.259	0.093	0.140
118:30	40000	2450	5.097	6.188	0.132	0.166
120:30	20000	1340	5.205	6.258	0.094	0.140
122:30	40000	2450	5.096	6.187	0.132	0.167
124:30	20000	1320	5.205	6.259	0.094	0.141
126:30	40000	2460	5.095	6.187	0.133	0.167
127:30	45000	2660	5.063	6.163	0.147	0.179
128:30	"	2650	5.060	6.160	0.149	0.181
129:30	"	2640	5.058	6.159	0.150	0.181
130:30	"	2650	5.057	6.158	0.151	0.182
131:30	"	2650	5.056	6.157	0.152	0.183
132:30	"	"	5.055	6.157	0.153	0.184
133:30	50000	2820	5.023	6.124	0.169	0.197
134:30	"	2840	5.017	6.120	0.174	0.200
135:30	"	2820	5.014	6.117	0.176	0.201
136:30	"	"	5.013	6.116	0.177	0.202
137:30	"	"	5.011	6.115	0.178	0.204
138:30	"	"	5.010	6.114	0.179	0.205
139:30	55000	3020	4.968	6.081	0.199	0.222
140:30	"	3000	4.962	6.077	0.204	0.225
141:30	"	"	4.959	6.074	0.206	0.227
142:30	"	"	4.956	6.072	0.208	0.228
143:30	"	"	4.954	6.070	0.210	0.230
144:30	"	"	4.953	6.069	0.211	0.231
145:30	60000	3360	4.908	6.034	0.235	0.251
146:30	"	"	4.900	6.030	0.240	0.255
147:30	"	"	4.897	6.025	0.243	0.258
148:30	"	"	4.893	6.022	0.245	0.260
149:30	"	"	4.890	6.020	0.247	0.261
150:30	"	"	4.858	6.020	0.249	0.262
152:30	30100	1700	5.023	6.120	0.174	0.215
154:30	60000	3320	4.870	6.004	0.255	0.274
156:30	30050	1700	5.022	6.106	0.182	0.222
158:30	60000	3360	4.860	5.990	0.250	0.278
160:30	30100	1700	5.018	6.097	0.184	0.225
162:30	60000	3360	4.846	5.978	0.267	0.286
164:30	30050	1700	5.006	6.089	0.186	0.230

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TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Date: 1-7-87

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
166:30	60000	3360	4.836	5.970	0.273	0.291
168:30	30050	1700	4.997	6.081	0.195	0.235
170:30	60000	3380	4.829	5.964	0.276	0.294
171:30	30150	1750	4.991	6.076	0.198	0.238
174:30	60000	3380	4.822	5.959	0.280	0.297
176:30	30000	1700	4.986	6.071	0.202	0.241
178:30	60000	3380	4.816	5.954	0.284	0.300
180:30	30000	1700	4.980	6.067	0.205	0.244
182:30	60000	3400	4.809	5.948	0.287	0.303
184:30	30000	1700	4.975	6.062	0.208	0.247
186:30	60000	3400	4.804	5.944	0.290	0.306
188:30	30050	1700	4.969	6.057	0.211	0.250
190:30	60000	3400	4.799	5.940	0.292	0.309
192:30	850	540	5.340	6.350	0.108	0.162
194:30	60000	3360	4.804	5.939	0.293	0.308
196:30	950	540	5.337	6.342	0.107	0.162
198:30	60000	3380	4.797	5.933	0.297	0.312
200:30	700	540	5.338	6.347	0.107	0.164
202:30	60000	3380	4.787	5.926	0.302	0.317
204:30	550	540	5.340	6.346	0.108	0.164
206:30	60000	3360	4.779	5.919	0.307	0.320
208:30	650	540	5.335	6.340	0.111	0.169
210:30	60000	3350	4.771	5.913	0.312	0.326
212:30	650	540	5.331	6.336	0.115	0.171
214:30	60000	3340	4.764	5.907	0.316	0.329
216:30	1050	560	5.315	6.324	0.121	0.176
218:30	60000	3340	4.757	5.900	0.319	0.332
220:30	950	540	5.316	6.324	0.122	0.177
222:30	60000	3340	4.750	5.897	0.324	0.335
224:30	600	540	5.311	6.321	0.126	0.180
226:30	60000	3360	4.745	5.890	0.328	0.339
228:30	600	550	5.313	6.321	0.126	0.181
230:30	60000	3350	4.741	5.886	0.329	0.340
231:30	65000	3520	4.676	5.853	0.356	0.361
232:30	"	"	4.690	5.848	0.353	0.365
233:30	"	"	4.686	5.845	0.362	0.368
234:30	"	"	4.684	5.842	0.364	0.369
235:30	"	"	4.682	5.841	0.365	0.370
236:30	"	"	4.681	5.840	0.366	0.370
237:30	70000	3750	4.615	5.810	0.387	0.387
238:30	"	"	4.630	5.797	0.392	0.391
239:30	"		4.627	5.794	0.395	0.394

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PILE LOAD TEST DATA

Continuation of: Date: 1-7-87

Time mm:Sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
240:30	70,000	3720	4.635	5.792	0.397	0.396
241:30	"	3700	4.632	5.790	0.398	0.397
242:30	"	"	4.631	5.788	0.400	0.399
243:30	75,000	3920	4.591	5.756	0.422	0.417
244:30	"	"	4.583	5.751	0.428	0.422
245:30	"	"	4.578	5.747	0.431	0.425
246:30	"	"	4.575	5.744	0.434	0.427
247:30	"	"	4.573	5.742	0.436	0.429
248:30	"	"	4.570	5.740	0.438	0.430
249:30	80,000	4200	4.529	5.703	0.463	0.450
250:30	"	"	4.518	5.695	0.471	0.457
251:30	"	4220	4.514	5.692	0.474	0.460
252:30	"	"	4.510	5.689	0.477	0.462
253:30	"	4200	4.507	5.687	0.479	0.464
254:30	"	"	4.505	5.684	0.481	0.461
255:30	85,000	4450	4.460	5.648	0.507	0.487
256:30	"	4440	4.450	5.640	0.515	0.494
257:30	"	"	4.443	5.634	0.519	0.497
258:30	"	"	4.439	5.630	0.523	0.500
259:30	"	"	4.435	5.627	0.525	0.503
260:30	"	"	4.432	5.625	0.527	0.505
261:30	90,000	4690	4.390	5.583	0.554	0.526
262:30	"	"	4.378	5.571	0.563	0.534
263:30	"	"	4.372	5.566	0.568	0.538
264:30	"	"	4.366	5.561	0.572	0.541
265:30	"	"	4.362	5.558	0.575	0.545
266:30	"	"	4.359	5.555	0.578	0.547
267:30	95,000	4800	4.312	5.512	0.605	0.569
268:30	"	"	4.300	5.505	0.613	0.577
269:30	"	"	4.292	5.497	0.619	0.581
270:30	"	"	4.286	5.493	0.623	0.585
271:30	"	"	4.281	5.489	0.627	0.588
272:30	"	"	4.277	5.485	0.630	0.590
273:30	100,000	5000	4.234	5.440	0.656	0.611
274:30	"	"	4.219	5.427	0.668	0.620
275:30	"	"	4.209	5.420	0.674	0.626
276:30	"	5040	4.202	5.414	0.679	0.630
277:30	"	"	4.196	5.409	0.683	0.634
278:30	"	5060	4.192	5.405	0.686	0.636
279:30	105,000	5320	4.151	5.368	0.709	0.655
280:30	"	5340	4.138	5.350	0.724	0.660
281:30	"	5350	4.117	5.340	0.722	0.675

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TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Pipe Pile   Date: 1-7-87

Time min:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
282:30	105000	5350	4.110	5.334	0.738	0.680
283:30	"	"	4.104	5.330	0.741	0.682
284:30	"	"	4.099	5.325	0.745	0.685
285:30	110000	5500	4.044	5.278	0.775	0.709
286:30	"	"	4.024	5.264	0.788	0.720
287:30	"	"	4.014	5.255	0.795	0.727
288:30	"	5600	4.005	5.248	0.802	0.731
289:30	"	"	4.000	5.244	0.805	0.735
290:30	"	"	3.995	5.240	0.809	0.738
291:30	115000	5800	3.943	5.196	0.835	0.759
292:30	"	"	3.923	5.181	0.851	0.771
293:30	"	"	3.910	5.170	0.860	0.779
294:30	"	5780	3.900	5.161	0.864	0.783
295:30	"	"	3.892	5.155	0.870	0.787
296:30	"	"	3.886	5.150	0.875	0.791
297:30	120000	6000	3.841	5.095	0.903	0.813
298:30	"	"	3.816	5.076	0.919	0.827
299:30	"	6050	3.803	5.065	0.928	0.835
300:30	"	6080	3.793	5.056	0.933	0.840
301:30	"	"	3.783	5.048	0.939	0.844
302:30	"	6100	3.775	5.042	0.945	0.849
303:30	125000	6300	3.722	4.995	0.972	0.872
304:30	"	"	3.707	4.980	0.985	0.881
305:30	"	"	3.693	4.968	0.994	0.889
306:30	"	"	3.683	4.960	1.001	0.895
307:30	"	"	3.672	4.951	1.007	0.901
308:30	"	"	3.665	4.945	1.012	0.906
309:30	130000	6500	3.606	4.896	1.040	0.938
310:30	"	"	3.583	4.875	1.057	0.951
311:30	"	"	3.569	4.862	1.064	0.958
312:30	"	"	3.558	4.853	1.071	0.964
313:30	"	"	3.548	4.844	1.078	0.969
314:30	"	"	3.540	4.838	1.083	0.972
315:30	135000	6680	3.479	4.786	1.111	1.000
316:30	"	6700	3.457	4.768	1.131	1.014
317:30	"	"	3.447	4.756	1.140	1.022
318:30	"	"	3.428	4.742	1.149	1.029
319:30	"	"	3.416	4.733	1.155	1.034
320:30	"	6720	3.408	4.725	1.162	1.039
321:30	140000	7000	3.354	4.680	1.186	1.062
322:30	"	7050	3.325	4.656	1.207	1.074
323:30	"	"	3.309	4.644	1.217	1.088

Concrete shaft

pipe pile 93 15/16

7:13

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TUNNEL SITE
PILE LOAD TEST DATA

Continuation of: Date:

Pile Gage VF-1-N
6 Ft
behind pile pile

0.5 → in
after unload pile pile was permanently bent

Time hm:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	Test Pile Disp. #2 (in) -6.521"	Reaction Pile Disp. #3 (in)	Reaction Pile Disp. #4 (in)
324:30	140000	7020	3.299	4.624	1.226	1.095
325:30	"	7040	3.288	4.616	1.233	1.101
326:30	"	"	3.280	4.609	1.239	1.106
327:30	145000	7250	3.217	4.550	1.273	1.137
328:30	"	7240	3.191	4.530	1.286	1.152
329:30	"	7220	3.174	4.514	1.297	1.160
330:30	"	"	3.160	4.501	1.304	1.168
331:30	"	"	3.150	4.493	1.310	1.172
332:30	"	"	3.140	4.484	1.311	1.174
333:30	150000	7400	3.079	4.430	1.345	1.209
334:30	"	"	3.046	4.405	1.366	1.224
335:30	"	7420	3.027	4.388	1.377	1.233
336:30	"	"	3.015	4.368	1.387	1.240
337:30	"	7440	3.004	4.357	1.395	1.248
338:30	"	7460	2.993	4.349	1.402	1.254
339:30	155000	7800	2.934	4.297	1.432	1.277
340:30	"	"	2.903	4.270	1.448	1.292
341:30	"	"	2.883	4.252	1.463	1.302
342:30	"	"	2.866	4.237	1.473	1.311
343:30	"	"	2.853	4.226	1.480	1.317
344:30	"	"	2.841	4.216	1.488	1.324
345:30	160000	8020	2.770	4.155	1.520	1.350
346:30	"	8020	2.741	4.131	1.540	1.366
347:30	"	8020	2.717	4.110	1.550	1.378
348:30	"	"	2.700	4.095	1.564	1.385
349:30	"	"	2.687	4.084	1.573	1.392
350:30	"	8000	2.676	4.074	1.581	1.396
3.1	155000	7800	2.654	4.051	1.597	1.411

17:52

TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _36" Drilled Shaft / loadline = 3.5' above ground_ Date: _1-8-87_

Reaction Pile: _4'∅ Drilled Shaft / load line = 3.5' above ground line_

Displacement gage locations:
#1: _24" 3/4 above load line / 2" right of vert_
#2: _4" 1/4 above load line / vert on load line_
#3: _10" above load line / 1" right of vert_
#4: _2" below load line / 1" right of vert_

Time mm:Sec	Load Cell Reading (lbs.)	Pressure in Jack	Test Pile Disp. #1	Test Pile Disp. #2	Reaction Pile Disp. #3	Reaction Pile Disp. #4
0	− 1500	540	1.621	0.868	0.681	0.105
:30 Sec	5000	800	1.632	0.870	0.685	0.105
1:30	"	"	1.629	0.871	0.685	0.105
2:30	"	"	1.630	0.871	0.685	0.105
3:30	"	"	1.630	0.871	0.685	0.105
4:30	"	"	1.630	0.871	0.685	0.105
5:30	10000	1020	1.643	0.882	0.692	0.106
6:30	"	1000	1.644	0.883	0.692	0.107
7:30	"	"	1.645	0.883	0.693	0.107
8:30	"	"	1.645	0.884	0.693	0.107
9:30	"	"	1.646	0.884	0.693	0.107
10:30	"	"	1.646	0.885	0.693	0.107
11:30	15000	1240	1.661	0.897	0.701	0.111
12:30	"	1220	1.662	0.897	0.702	0.113
13:30	"	"	1.664	0.899	0.702	0.113
14:30	"	1240	1.665	0.899	0.702	0.113
15:30	"	"	1.665	0.899	0.703	0.113
16:30	"	"	1.665	0.899	0.703	0.113
17:30	20,000	1500	1.682	0.912	0.712	0.120
18:30	"	"	1.684	0.915	0.712	0.120
19:30	"	"	1.685	0.914	0.713	0.121
20:30	"	"	1.685	0.914	0.713	0.121
21:30	"	"	1.686	0.915	0.713	0.121
22:30	"	"	1.687	0.917	0.713	0.121
23:30	25000	1720	1.705	0.931	0.723	0.128
24:30	"	1740	1.707	0.933	0.724	0.129
25:30	"	"	1.708	0.933	0.724	0.129
26:30	"	1720	1.709	0.934	0.725	0.129
27:30	"	1740	1.709	0.935	0.725	0.129
28:30	"	1740	1.711	0.935	0.725	0.129
29:30	30,000	1940	1.730	0.951	0.736	0.139
30:30	"	"	1.732	0.953	0.736	0.140
31:30	"	1960	1.733	0.954	0.736	0.140
32:30	"	1950	1.735	0.956	0.739	0.141
33:30	"	"	1.734	0.956	0.739	0.141
34:30	"	"	1.735	0.956	0.739	0.141

10:02

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TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _Drilled Shaft_   Date: _1-6-67_

Reaction Pile: _____

Displacement gage locations:
    #1: _____
    #2: _____
    #3: _____
    #4: _____

Time	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	Test Pile Disp. #2 (in) -0.868	Reaction Pile Disp. #3 (in)	Reaction Pile Disp. #4 (in) (0.105)
35:30	35000	2150	1.757	0.974	0.752	0.151
36:30	"	"	1.761	0.976	0.754	0.153
37:30	"	"	1.763	0.978	0.755	0.153
38:30	"	"	1.764	0.980	0.755	0.154
39:30	"	2140	1.766	0.981	0.756	0.155
40:30	"	250	1.767	0.981	0.757	0.156
41:30	40000	2360	1.792	1.001	0.770	0.167
42:30	"	"	1.796	1.004	0.773	0.169
43:30	"	"	1.798	1.004	0.774	0.170
44:30	"	"	1.800	1.007	0.775	0.170
45:30	"	"	1.801	1.008	0.775	0.170
46:30	"	"	1.801	1.008	0.776	0.170
47:30	"	"	1.802	1.009	0.776	0.171
48:30	45000	2560	1.829	1.032	0.791	0.183
49:30	"	"	1.833	1.035	0.793	0.185
50:30	"	"	1.842	1.041	0.795	0.186
51:30	"	"	1.847	1.045	0.796	0.188
52:30	"	"	1.849	1.047	0.797	0.189
53:30	"	"	1.851	1.048	0.798	0.189
54:30	50000	2780	1.888	1.080	0.814	0.203
55:30	"	"	1.901	1.089	0.819	0.207
56:30	"	"	1.907	1.093	0.820	0.208
57:30	"	"	1.910	1.096	0.822	0.210
58:30	"	"	1.914	1.099	0.823	0.210
59:30	"	"	1.915	1.099	0.824	0.211
60:30	55000	2900	1.956	1.134	0.842	0.226
61:30	"	"	1.972	1.145	0.848	0.230
62:30	"	"	1.980	1.150	0.851	0.232
63:30	"	"	1.982	1.154	0.851	0.232
64:30	"	"	1.985	1.155	0.852	0.233
65:30	"	"	1.986	1.156	0.853	0.235
67:30	650	580	1.751	0.963	0.738	0.173
69:30	55000	3080	2.006	1.182	0.864	0.262
71:30	650	580	1.772	0.983	0.742	0.176
73:30	650					
75:30	5500	3080	2.036	1.199	0.864	0.267

bottom

less of power head and for L arm.

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TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _____   Date: _____

Reaction Pile: _____

Displacement gage locations:
#1: _____
#2: _____
#3: _____
#4: _____

Time mm:sec	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in) (0.66r)	Reaction Pile Disp. #3 (in)	#4 (in) (0.105)
77.30	600	580	1.787	0.995	0.747	0.179
79.30	55000	3080	2.059	1.217	0.677	0.274
81.30	600	600	1.799	1.007	0.75?	0.153
93.30	55000	3080	2.074	1.230	0.881	0.278
85.30	550	600	1.809	1.014	0.801?	0.189
87.30	55000	3080	2.089	1.238	0.883	0.273
89.30	650	600	1.818	1.022	0.756	0.169
91.30	55000	3080	2.099	1.247	0.886	0.281
93.30	550	580	1.826	1.027	0.759	0.190
95.30	55000	3080	2.103	1.254	0.889	0.285
97.30	550	580	1.831	1.031	0.761	0.192
99.30	55000	3080	2.119	1.263	0.894	0.290
101.50	750	600	1.840	1.040	0.765	0.194
103.30	55000	3050	2.124	1.270	0.893	0.293
105.30	660	600	1.842	1.043	0.767	0.198
107.30	55000	3050	2.135	1.276	0.899	0.297
109.30	27650	1700	2.050	1.209	0.846	0.262
111.30	55000	3050	2.143	1.284	0.903	0.299
113.30	27600	1680	2.058	1.215	0.850	0.265
115.30	55000	3050	2.149	1.288	0.906	0.301
117.30	27550	1700	2.064	1.221	0.853	0.267
119.30	55000	3050	2.156	1.294	0.909	0.305
121.30	27550	1680	2.071	1.226	0.857	0.271
123.30	55000	3060	2.160	1.297	0.911	0.307
125.30	27550	1700	2.075	1.230	0.860	0.274
127.30	55000	3060	2.163	1.301	0.914	0.210
129.30	27500	1700	2.065	1.222	0.856	0.270
131.30	55000	3050	2.168	1.303	0.915	0.310
133.30	27600	1700	2.083	1.236	0.863	0.278
135.30	55000	3050	2.173	1.307	0.917	0.311
137.30	27650	1700	2.086	1.239	0.865	0.279
139.30	55000	3050	2.176	1.310	0.918	0.312
141.30	27400	1700	2.090	1.243	0.867	0.281
143.30	55000	3050	2.178	1.312	0.920	0.314
145.30	27600	1700	2.092	1.245	0.666	0.282
147.30	55000	3050	2.182	1.314	0.922	0.317

gage sticking figure this number

TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _____   Date: _____

Reaction Pile: _____

Displacement gage locations:
#1: _____
#2: _____
#3: _____
#4: _____

Time	Load Cell Reading (lbs.)	Pressure in Jack	Test Pile Disp.		Reaction Pile Disp.	
			#1 (in)	#2 (in) -0.868	#3 (in)	#4 (in) -0.105
148:30	60,000	3280	2.203	1.333	0.936	0.329
149:30	60,000	3280	2.207	1.336	0.939	0.331
150:30	"	"	2.209	1.338	0.940	0.332
151:30	"	"	2.211	1.339	0.941	0.332
152:30	"	"	2.212	1.340	0.942	0.333
153:30	"	"	2.214	1.341	0.942	0.334
154:30	65,000	3480	2.239	1.362	0.957	0.346
155:30	"	"	2.245	1.366	0.962	0.350
156:30	"	"	2.249	1.370	0.963	0.352
157:30	"	"	2.251	1.371	0.965	0.353
158:30	"	"	2.253	1.373	0.966	0.354
159:30	"	"	2.254	1.375	0.967	0.355
160:30	70,000	3680	2.285	1.398	0.984	0.370
161:30	"	"	2.295	1.404	0.988	0.374
162:30	"	"	2.299	1.406	0.992	0.378
163:30	"	"	2.300	1.409	0.993	0.378
164:30	"	"	2.303	1.411	0.994	0.378
165:30	"	"	2.305	1.413	0.995	0.379
166:30	75,000	3900	2.340	1.444	1.015	0.396
167:30	"	"	2.349	1.450	1.019	0.400
168:30	"	"	2.353	1.454	1.022	0.402
169:30	"	"	2.356	1.456	1.024	0.404
170:30	"	"	2.361	1.460	1.026	0.406
171:30	"	"	2.365	1.462	1.028	0.407
172:30	80,000	4120	2.404	1.495	1.048	0.424
173:30	"	4150	2.418	1.504	1.054	0.426
174:30	"	4160	2.425	1.509	1.057	0.431
175:30	"	"	2.431	1.513	1.060	0.433
176:30	"	"	2.435	1.518	1.062	0.435
177:30	"	"	2.438	1.521	1.063	0.437
178:30	40,000	2180	2.315	1.420	0.982	0.382
180:30	80,000	4180	2.457	1.544	1.077	0.452
182:30	40,000	2200	2.334	1.438	0.990	0.390
184:30	80,000	4200	2.486	1.562	1.086	0.460
186:30	40,000	2200	2.357	1.456	1.000	0.398
188:30	80,000	4180	2.505	1.577	1.094	0.468

Notes: 1 (178:30), 2 (182:30), 3 (186:30)

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TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _____   Date: _____

Reaction Pile: _____

Displacement gage locations:
   #1:_____
   #2:_____
   #3:_____
   #4:_____

	Time (min:sec)	Load Cell Reading (lbs.)	Pressure in Jack (psi)	Test Pile Disp. #1 (in)	#2 (in)	Reaction Pile Disp. #3 (in)	#4 (in)
4	190:30	40200	2180	2.374	1.472	1.008	0.404
	192:30	80000	4200	2.522	1.590	1.101	0.473
5	194:30	40000	2200	2.390	1.484	1.014	0.409
	196:30	80000	4180	2.536	1.602	1.108	0.480
6	198:30	40150	2150	2.401	1.495	1.020	0.415
	200:30	80000	4180	2.548	1.612	1.113	0.482
7	202:30	40200	2160	2.415	1.511	1.026	0.420
	204:30	80000	4160	2.559	1.623	1.119	0.490
8	206:30	39850	2120	2.421	1.515	1.028	0.422
	208:30	80000	4180	2.568	1.630	1.123	0.492
9	210:30	39850	2120	2.430	1.520	1.033	0.426
	212:30	80000	4180	2.576	1.638	1.127	0.496
10	214:30	40000	2150	2.442	1.529	1.038	0.431
	216:30	80000	4180	2.585	1.645	1.131	0.499
11	218:30	700	500	2.050	1.213	0.880	0.303
	220:30	80000	4160	2.590	1.649	1.129	0.495
12	222:30	700	500	2.055	1.225	0.882	0.306
	224:30	80000	4180	2.605	1.662	1.133	0.502
13	226:30	700	500	2.066	1.236	0.888	0.311
	228:30	80000	4160	2.627	1.681	1.141	0.509
14	230:30	700	500	2.073	1.243	0.891	0.313
	232:30	80000	4160	2.644	1.695	1.147	0.516
15	234:30	750	500	2.088	1.255	0.897	0.319
	236:30	80000	4160	2.663	1.709	1.154	0.521
16	238:30	750	500	2.094	1.261	0.902	0.322
	240:30	80000	4160	2.675	1.722	1.158	0.523
17	242:30	700	500	2.102	1.268	0.904	0.325
	244:30	80000	4160	2.685	1.728	1.159	0.525
18	246:30	700	500	2.103	1.270	0.907	0.329
	248:30	80000	4160	2.700	1.740	1.165	0.527
19	250:30	800	500	2.117	1.282	0.912	0.334
	252:30	80000	4160	2.704	1.750	1.169	0.531
20	254:30	750	500	2.122	1.286	0.911	0.337
	256:30	80000	4140	2.712	1.761	1.172	0.535
	257:30	85000	4370	2.751	1.792	1.194	0.554
	258:30	"	"	2.758	1.798	1.198	0.558

TUNNEL SITE
PILE LOAD TEST DATA

Test Pile: _____   Date: _____

Reaction Pile: _____

Displacement gage locations:
#1: _____
#2: _____
#3: _____
#4: _____

Time	Load Cell Reading (lbs.)	Pressure in Jack	Test Pile Disp. #1	#2	Reaction Pile Disp. #3	#4
259:30	85000	4320	2.763	1.801	1.201	0.560
260:30	"	"	2.766	1.803	1.203	0.561
261:30	"	"	2.768	1.805	1.204	0.563
262:30	"	"	2.769	1.806	1.206	0.564
263:30	90000	4560	2.800	1.831	1.223	0.578
264:30	"	4540	2.806	1.835	1.226	0.581
265:30	"	4560	2.810	1.839	1.229	0.583
266:30	"	"	2.813	1.842	1.232	0.585
267:30	"	"	2.816	1.843	1.233	0.587
268:30	"	4580	2.817	1.845	1.234	0.587
269:30	95000	4800	2.849	1.870	1.252	0.601
270:30	"	"	2.857	1.877	1.257	0.607
271:30	"	"	2.861	1.880	1.259	0.608
272:30	"	"	2.865	1.883	1.262	0.610
273:30	"	"	2.868	1.885	1.263	0.611
274:30	"	"	2.870	1.888	1.265	0.613
275:30	100000	5020	2.904	1.913	1.283	0.628
276:30	"	5000	2.914	1.922	1.289	0.633
277:30	"	"	2.919	1.926	1.292	0.636
278:30	"	5020	2.923	1.930	1.295	0.638
279:30	"	5040	2.927	1.932	1.297	0.639
280:30	"	"	2.930	1.935	1.298	0.641
281:30	105000	5240	2.963	1.964	1.316	0.657
282:30	"	5250	2.976	1.973	1.324	0.662
283:30	"	"	2.981	1.977	1.326	0.665
284:30	"	"	2.981	1.987	1.330	0.668
285:30	"	5260	2.991	1.985	1.332	0.671
286:30	"	5240	2.995	1.988	1.335	0.673
287:30	110000	5450	3.033	2.018	1.355	0.690
288:30	"	"	3.045	2.027	1.361	0.694
289:30	"	"	3.055	2.034	1.366	0.698
290:30	"	"	3.059	2.037	1.368	0.700
291:30	"	"	3.063	2.041	1.372	0.704
292:30	"	"	3.067	2.044	1.375	0.708
293:30	115000	5620	3.107	2.077	1.395	0.725
294:30	"	"	3.121	2.087	1.402	0.730

131

Time mm:sec	Load lb	Press Tank psi	Gage 1 (in)	Gage 2 (in)	Gage 3 (in)	Gage 4 (in)	Gap Penetr
295:30	115 000	5620	3.129	2.094	1.406	0.733	45 5/8"
296:30	"	"	3.136	2.098	1.410	0.737	
297:30	"	"	3.141	2.102	1.412	0.738	
298:30	"	5800	3.145	2.105	1.416	0.741	
299:30	120000	5820	3.183	2.136	1.435	0.758	
300:30	"	"	3.199	2.147	1.443	0.765	
301:30	"	5850	3.208	2.155	1.448	0.768	
302:30	"	5860	3.214	2.160	1.452	0.771	
303:30	"	5880	3.221	2.166	1.456	0.776	
304:30	"	"	3.226	2.169	1.459	0.780	
305:30	125000	6100	3.263	2.200	1.480	0.796	
306:30	"	"	3.281	2.212	1.488	0.802	
307:30	"	6120	3.294	2.222	1.493	0.807	
308:30	"	6200	3.301	2.228	1.498	0.811	
309:30	"	"	3.307	2.232	1.502	0.814	
310:30	"	"	3.312	2.236	1.504	0.816	
311:30	130000	6420	3.355	2.270	1.527	0.834	
312:30	"	"	3.373	2.283	1.535	0.842	
313:30	"	"	3.384	2.292	1.541	0.648	
314:30	"	6440	3.393	2.300	1.546	0.852	
315:30	"	"	3.401	2.305	1.550	0.855	
316:30	"	"	3.406	2.309	1.553	0.858	
317:30	135000	6660	3.438	2.348	1.576	0.877	
318:30	"	6680	3.454	2.360	1.584	0.883	
319:30	"	"	3.467	2.370	1.591	0.891	
320:30	"	"	3.477	2.378	1.596	0.896	
321:30	"	6660	3.483	2.387	1.600	0.900	
322:30	"	6650	3.489	2.394	1.604	0.902	
323:30	140000	6850	3.534	2.430	1.625	0.920	

Time mm.sec	Load lbs	Press. Jack psi	Gage 1 in	Gage 2 in	Gage 3 in	Gage 4 in
324:30	"	6850	3.555	2.445	1.636	0.930
325:30	"	6840	3.566	2.454	1.643	0.935
326:30	"	6820	3.576	2.462	1.648	0.939
327:30	"	6820	3.580	2.470	1.652	0.943
328:30	"	"	3.587	2.475	1.656	0.946
329:30	145000	7080	3.694	2.554	1.676	0.965
330:30	"	7100	3.718	2.571	1.690	0.976
331:30	"	"	3.732	2.582	1.697	0.981
332:30	"	"	3.740	2.600	1.702	0.986
333:30	"	"	3.747	2.608	1.708	0.990
334:30	"	"	3.754	2.614	1.711	0.992
335:30	150000	7340	3.793	2.668	1.735	1.013
336:30	"	7360	3.815	2.684	1.746	1.023
337:30	"	7380	3.832	2.694	1.754	1.029
338:30	"	7480	3.840	2.704	1.761	1.035
339:30	"	"	3.850	2.710	1.765	1.038
340:30	"	"	3.859	2.718	1.769	1.041
341:30	155000	7620	3.905	2.758	1.790	1.060
342:30	"	"	3.924	2.775	1.801	1.069
343:30	"	"	3.943	2.792	1.809	1.076
344:30	"	"	3.950	2.799	1.815	1.082
345:30	"	"	3.962	2.806	1.821	1.087
346:30	"	"	3.971	2.814	1.826	1.090
347:30	160000	7800	4.025	2.858	1.851	1.111
348:30	"	"	4.047	2.875	1.862	1.120
349:30	"	"	4.067	2.890	1.871	1.126
350:30	"	"	4.075	2.904	1.880	1.13
351:30	"	"	4.088	2.913	1.886	1.140
352:30	"	"	4.097	2.920	1.890	1.144

133

Time mm:Sec	Load (lb)	Press. Jack (psi)	Gage 1 (in)	Gage 2 (in)	Gage 3 (in)	Gage 4 (in)
353:30	165000	8050	4.151	2.967	1.915	1.168
354:30	"	8060	4.179	2.988	1.928	1.176
355:30	"	8100	4.200	3.003	1.937	1.184
356:30	"	8100	4.214	3.013	1.943	1.189
357:30	"	"	4.225	3.023	1.950	1.195
358:30	"	8120	4.236	3.032	1.955	1.199
359:30	170000	8350	4.288	3.082	1.979	1.220
360:30	"	8360	4.320	3.105	1.993	1.231
361:30	"	8380	4.338	3.119	2.001	1.236
362:30	"	8400	4.355	3.133	2.008	1.244
363:30	"	8400	4.373	3.147	2.015	1.250
364:30	"	"	4.384	3.156	2.021	1.255
365:30	175000	8600	4.443	3.205	2.043	1.275
366:30	"	"	4.482	3.234	2.057	1.286
367:30	"	"	4.516	3.258	2.068	1.295
16:19	74200	4800	3.854	2.876	1.725	1.047

Note: Load Cell and Jack broke!
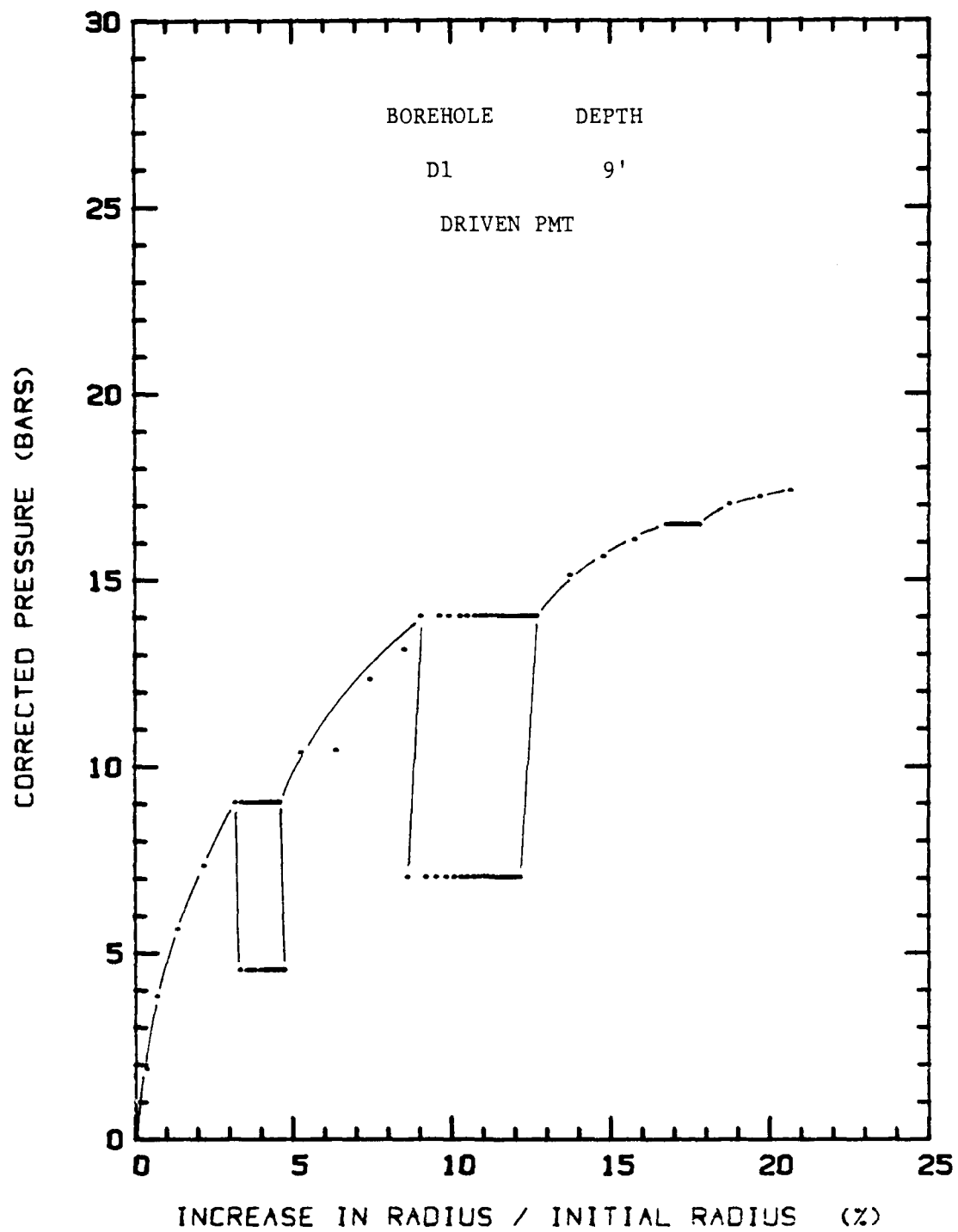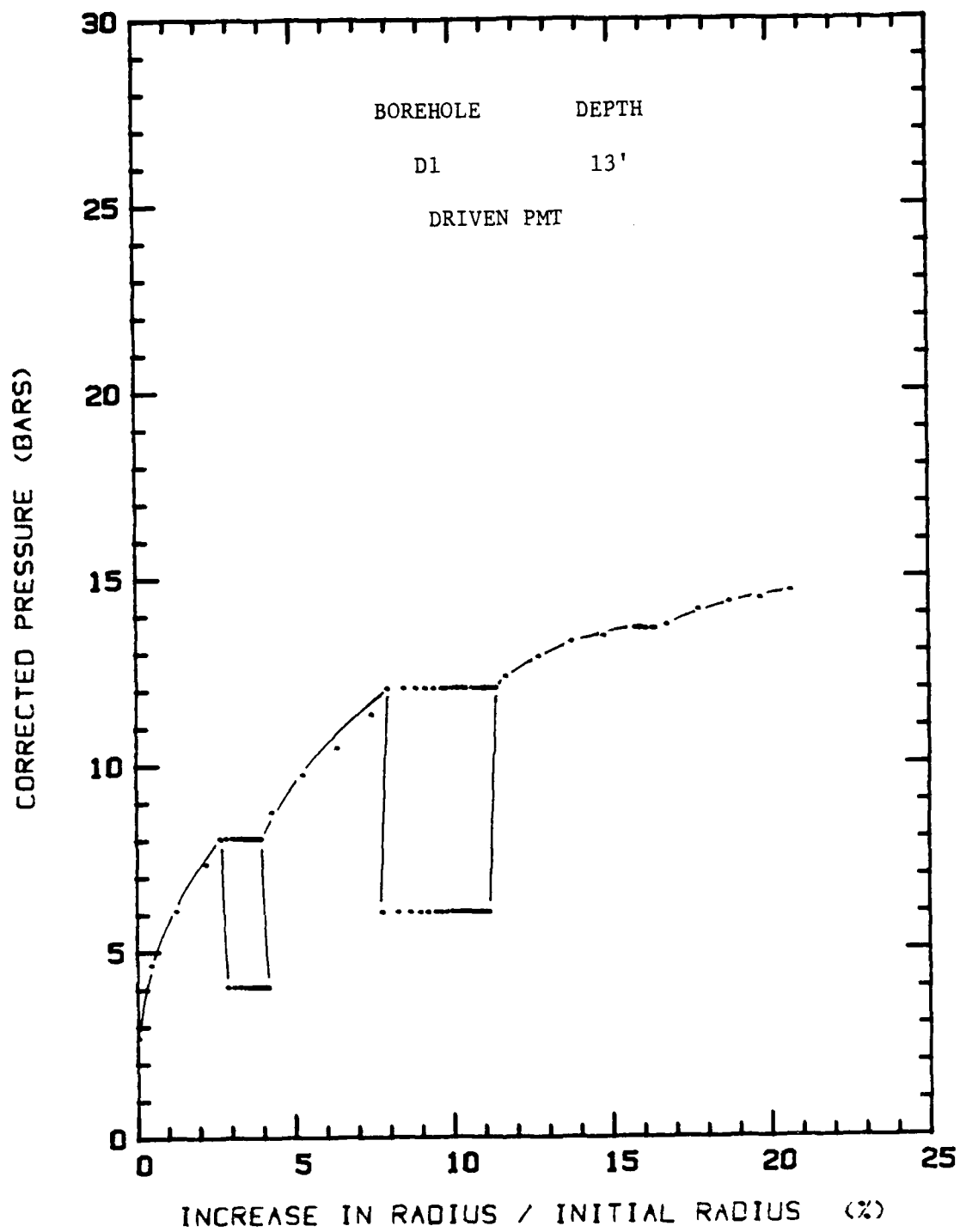
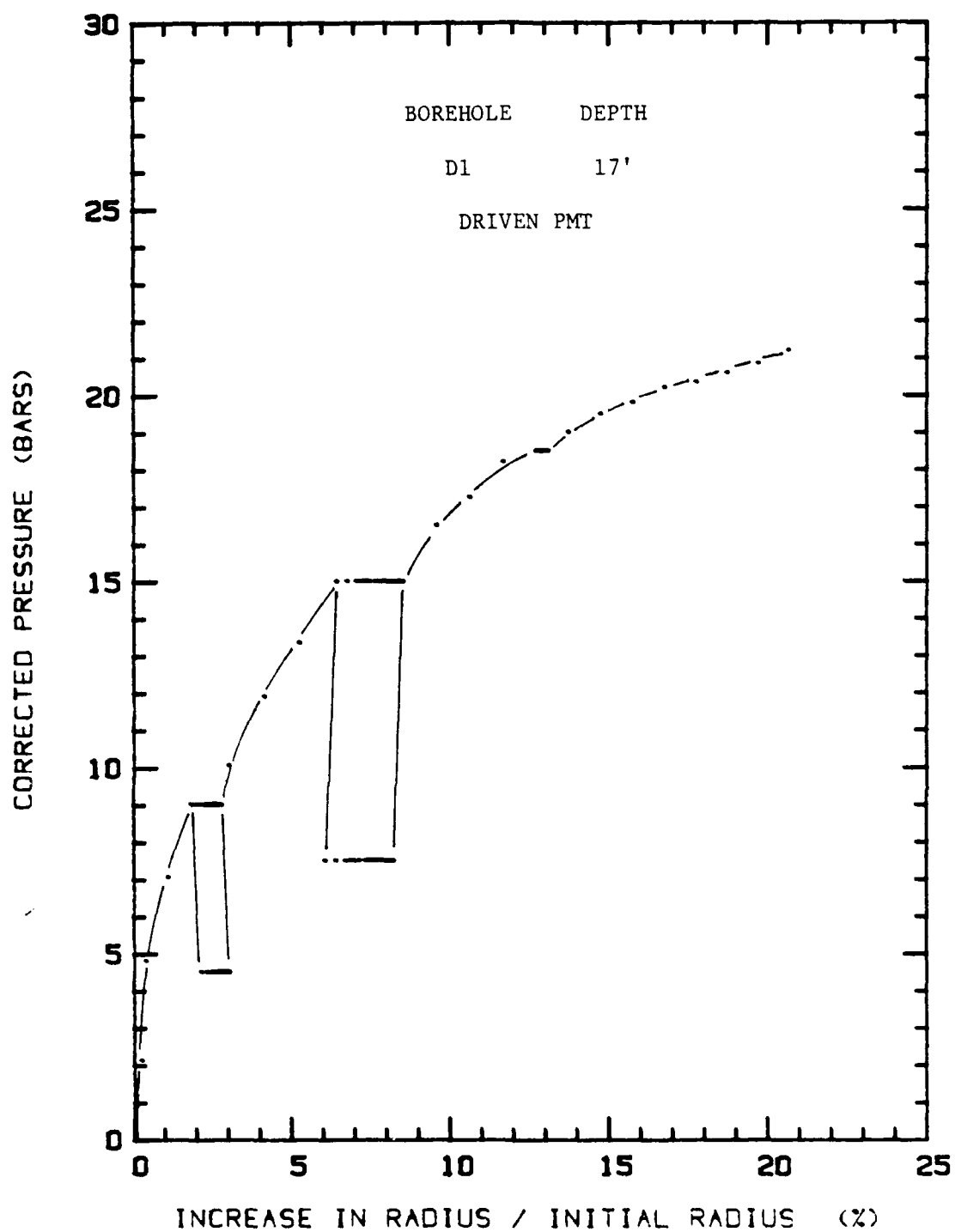Gap in back of drilled shaft (36") = 1.25" wide
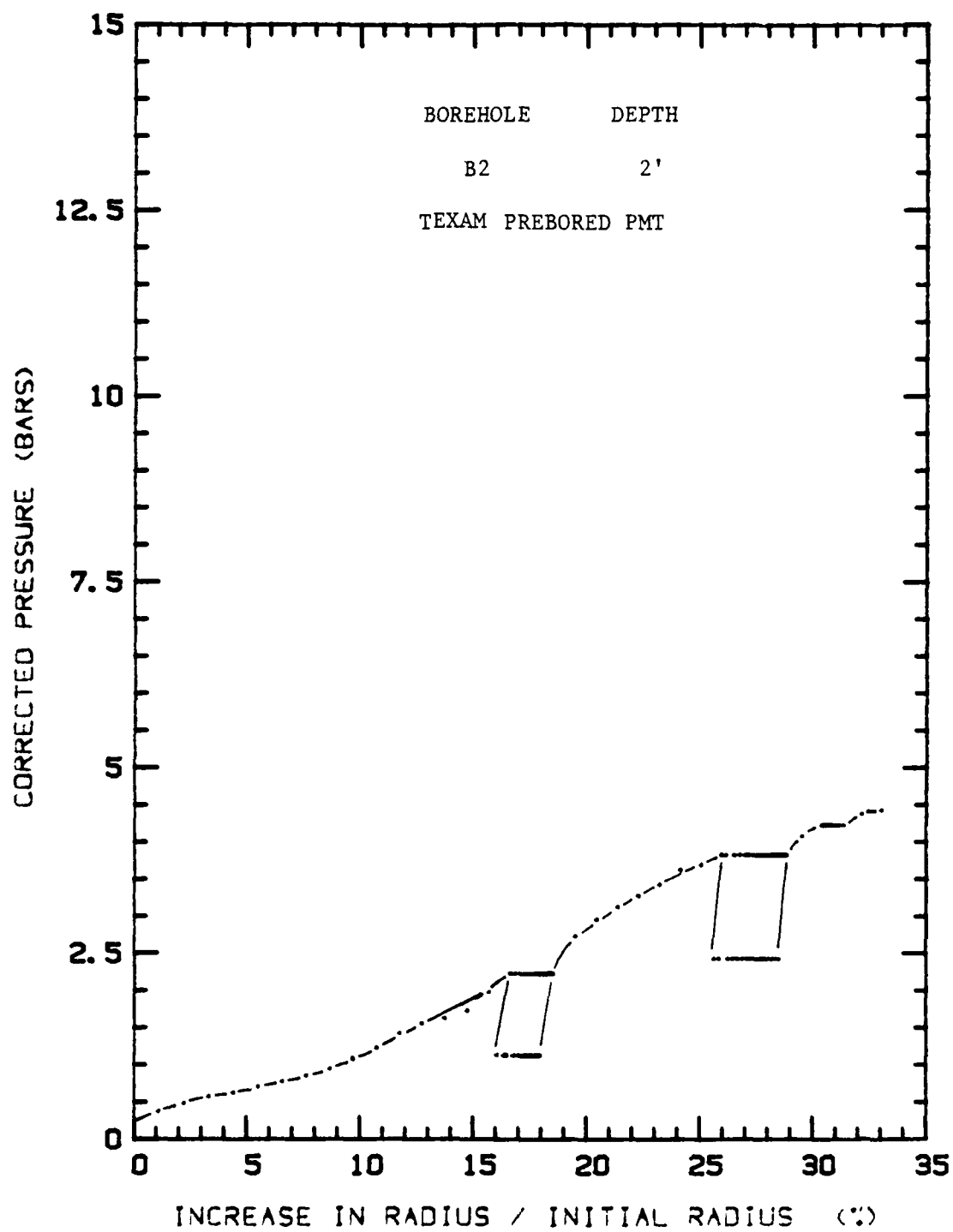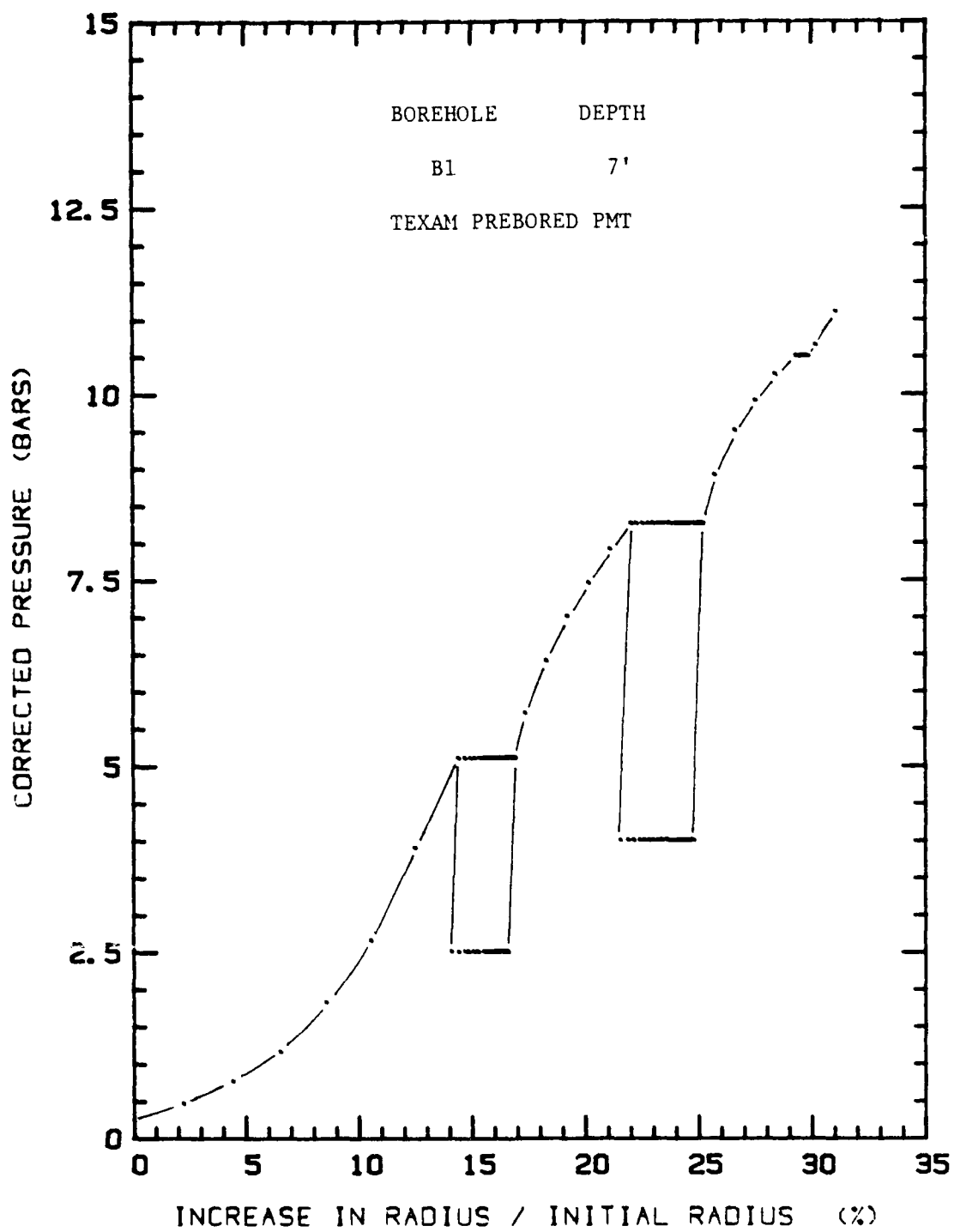
= 45.5/8" deep

134

APPENDIX B

CORRECTED PMT CURVES

135

BOREHOLE   DEPTH

D1   5'

DRIVEN PMT

BOREHOLE   DEPTH

B2   2'

TEXAM PREBORED PMT

CORRECTED PRESSURE (BARS)

INCREASE IN RADIUS / INITIAL RADIUS (%)

BOREHOLE   DEPTH

B1   7'

TEXAM PREBORED PMT

INCREASE IN RADIUS / INITIAL RADIUS   (%)

142

BOREHOLE   DEPTH

B1   10'

TEXAM PREBORED PMT

143

BOREHOLE   DEPTH

B1   15'

TEXAM PREBORED PMT

144

BOREHOLE   DEPTH

B1   30'

TEXAM PREBORED PMT

145
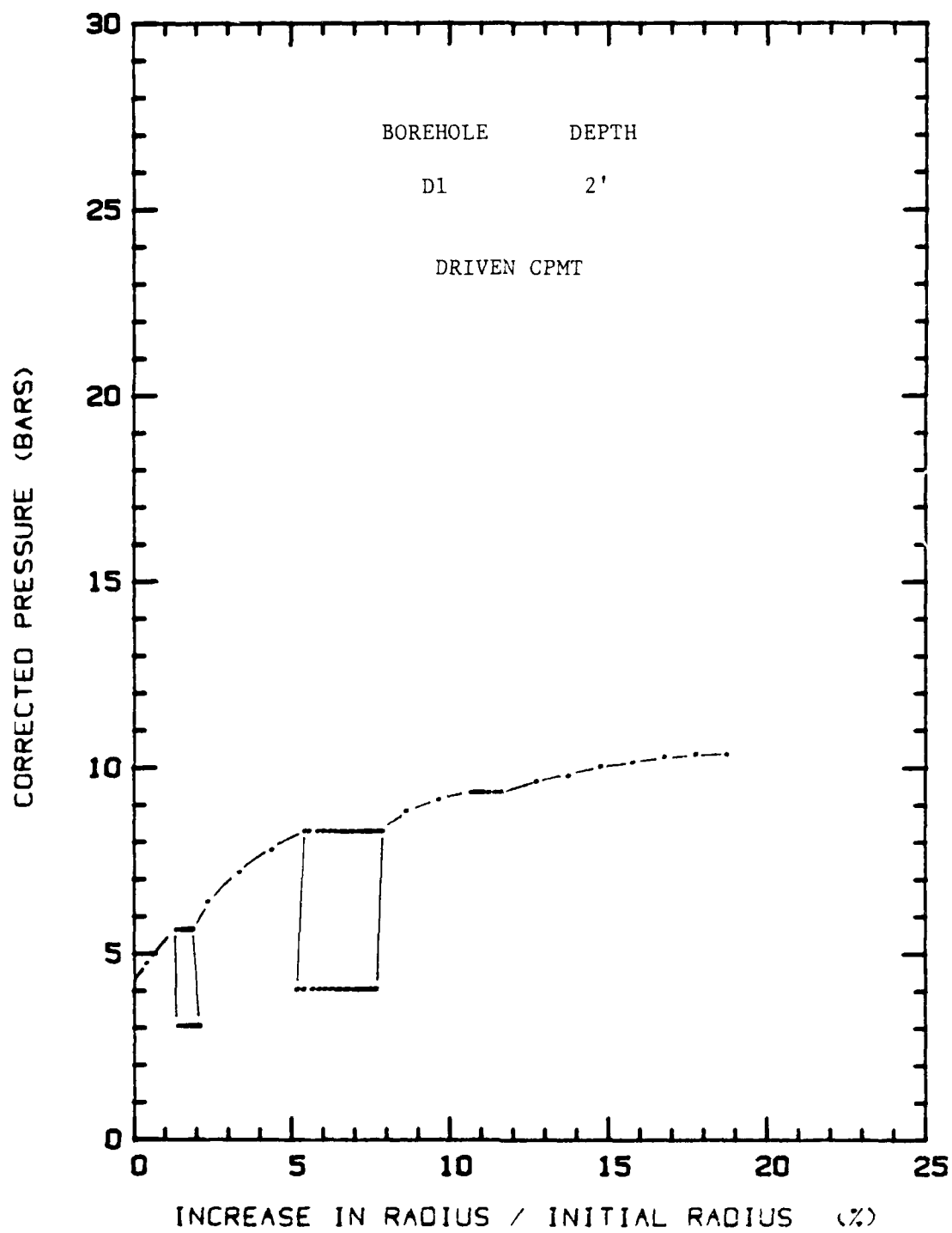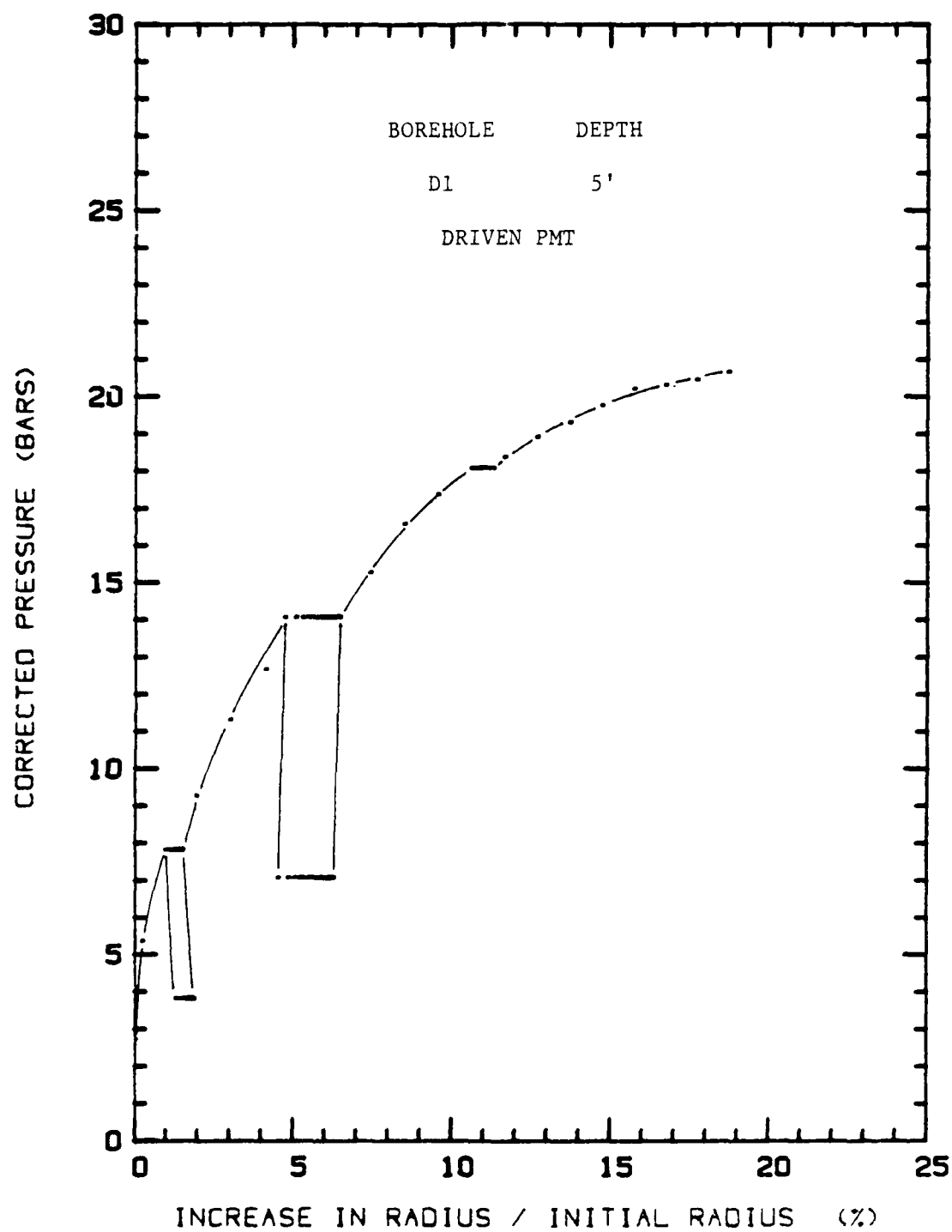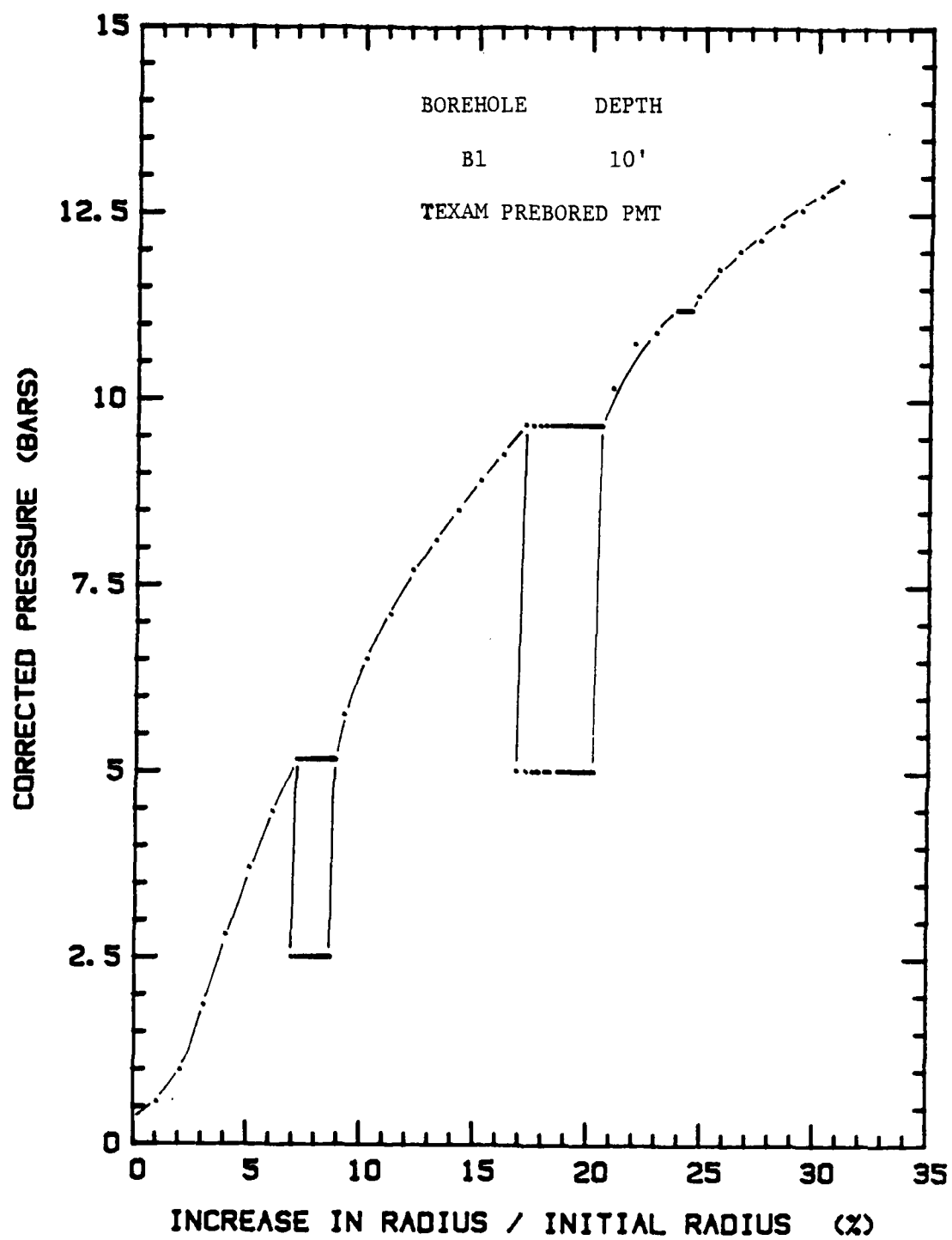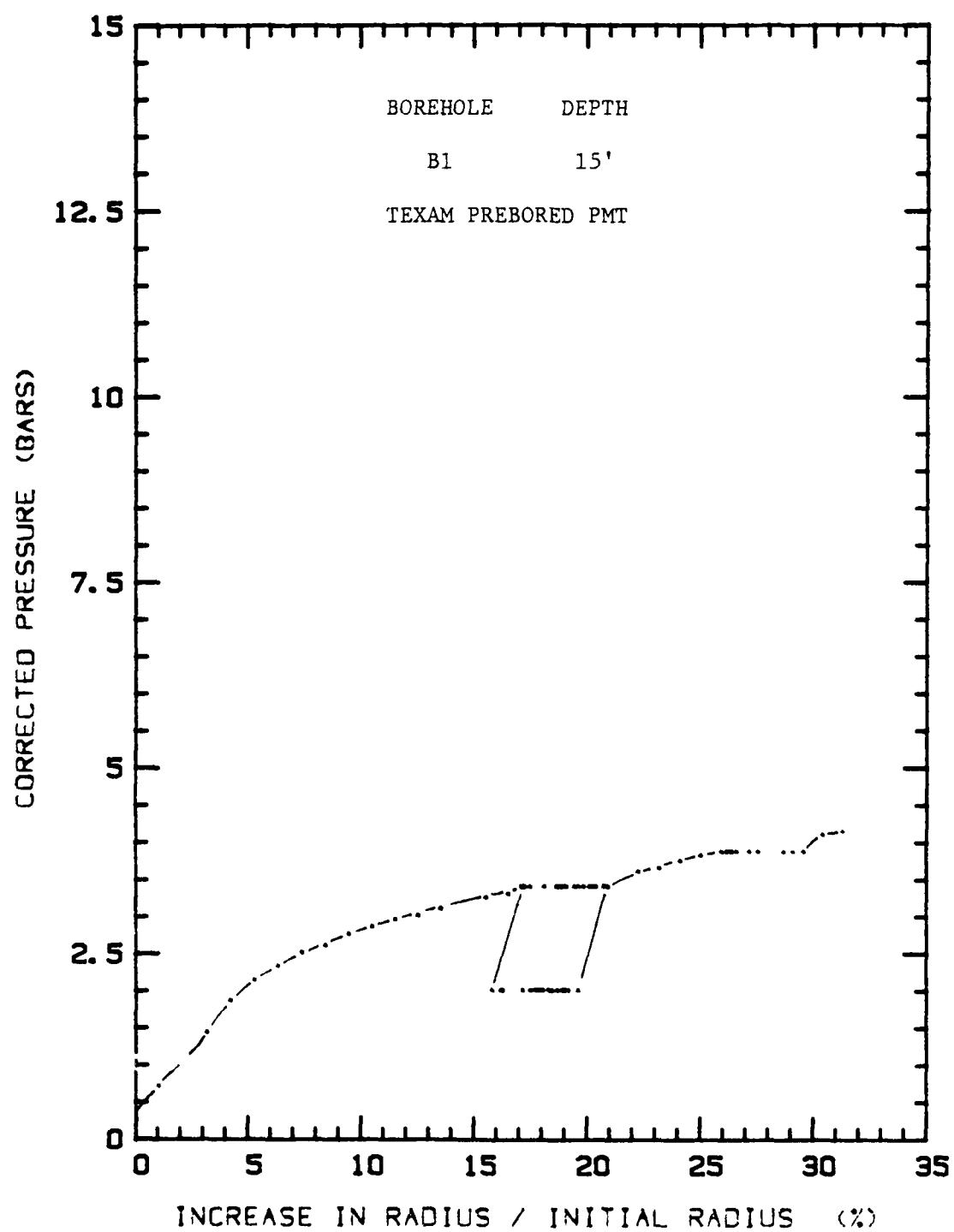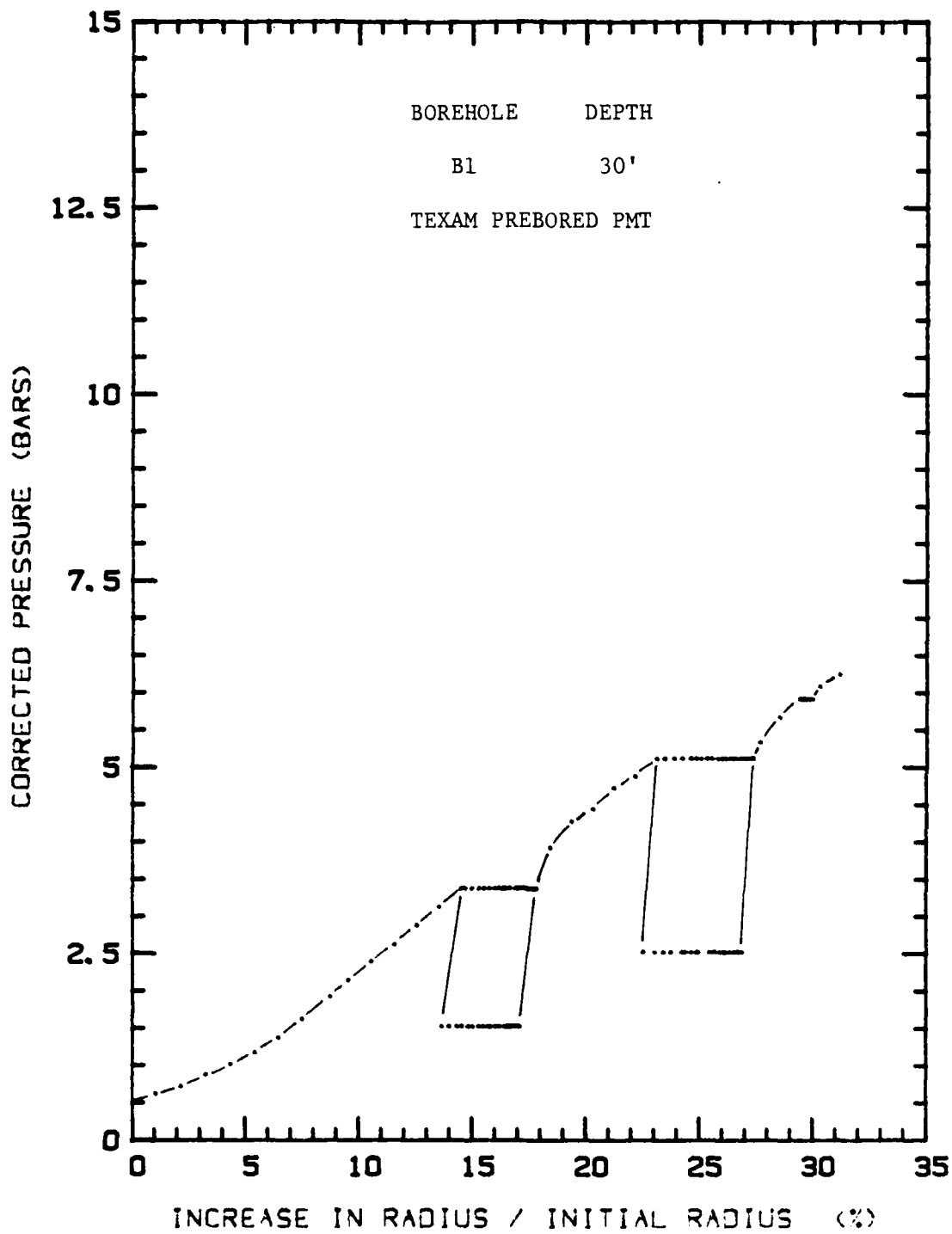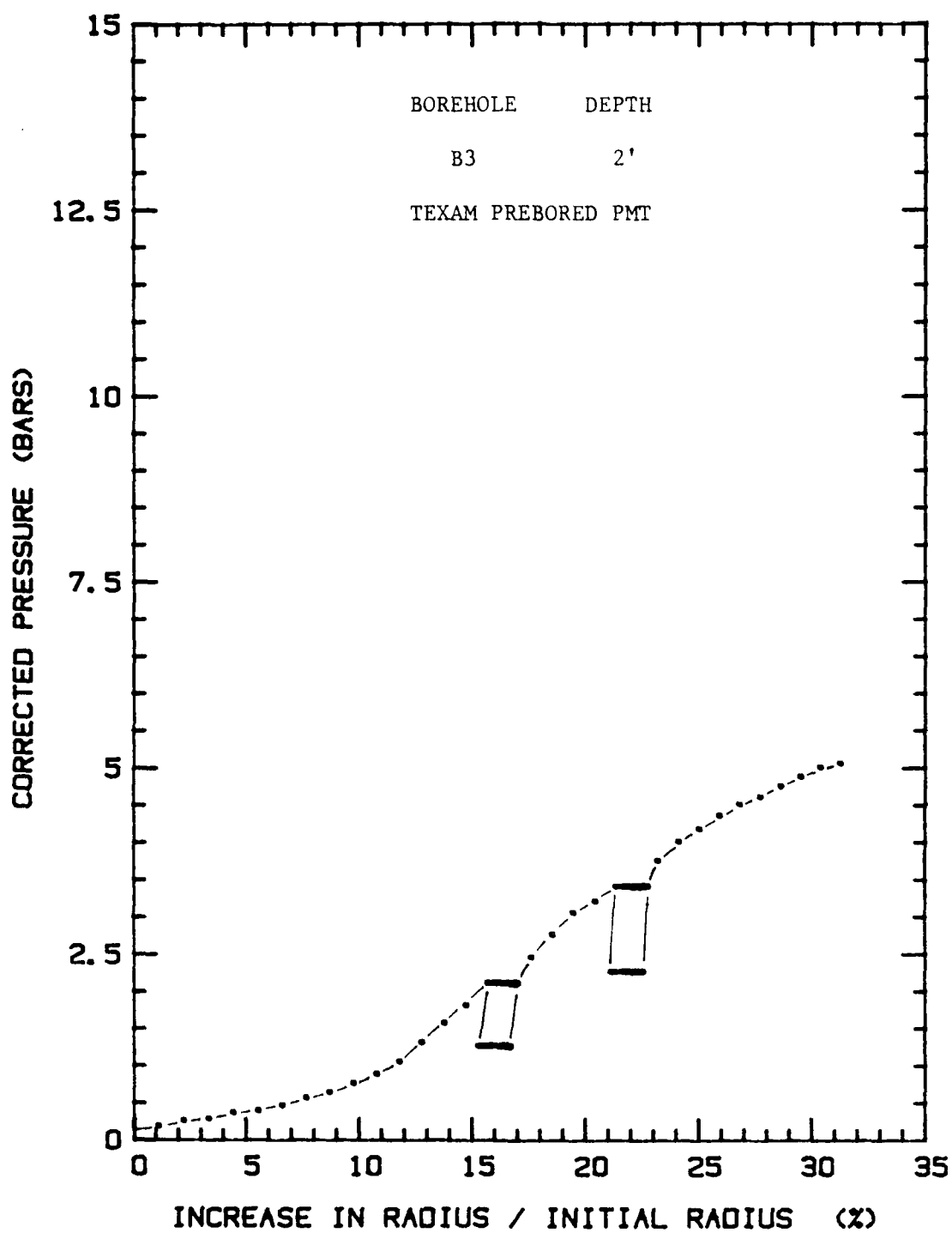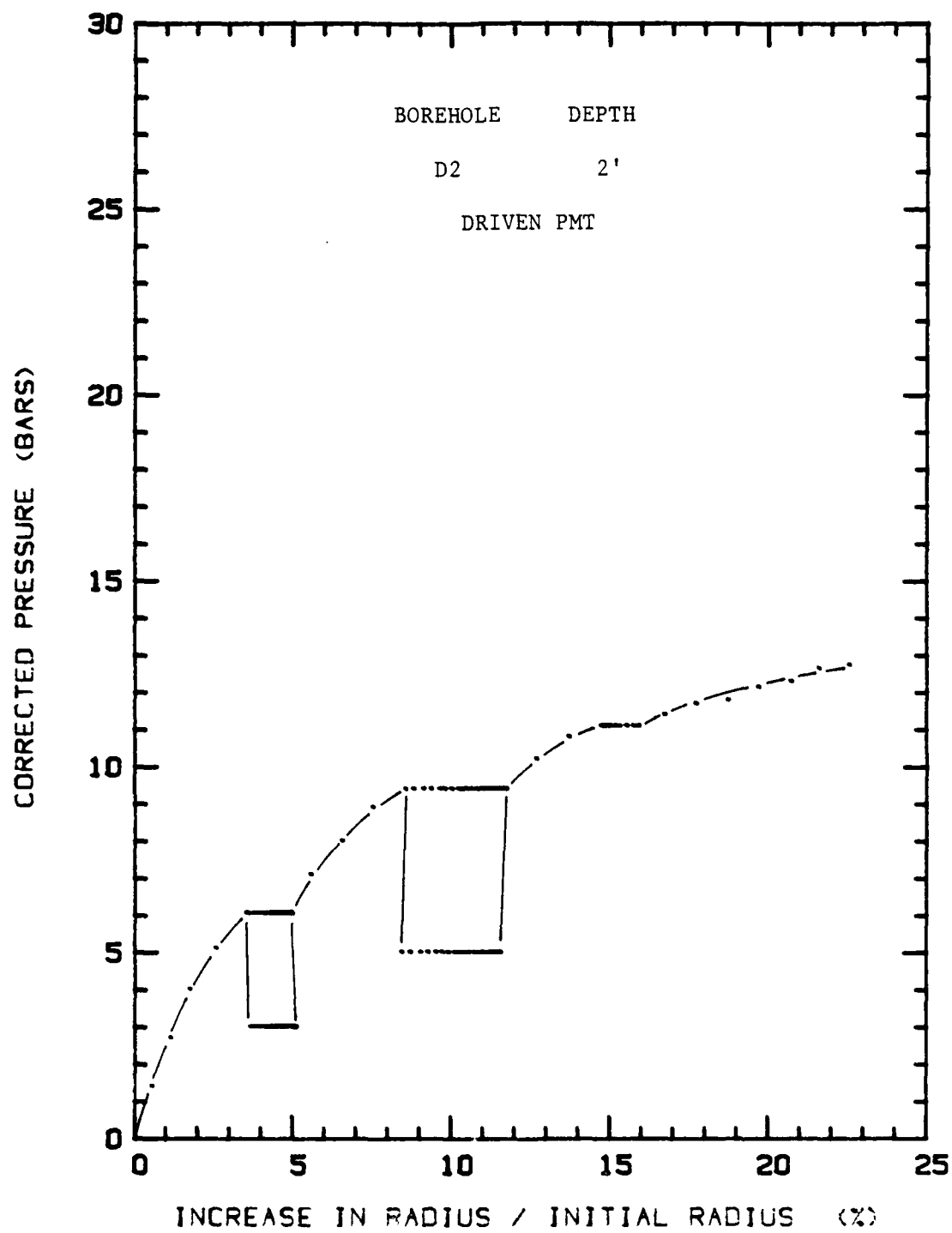
APPENDIX C

CYCLIC DEGRADATION OF THE

PMT SECANT SHEAR MODULUS

149

Borehole D1
Depth 2 ft

* load level 1
o load level 2

CYCLE NUMBER (N)

Cs(N)/Cs(1)

150

Borehole D1
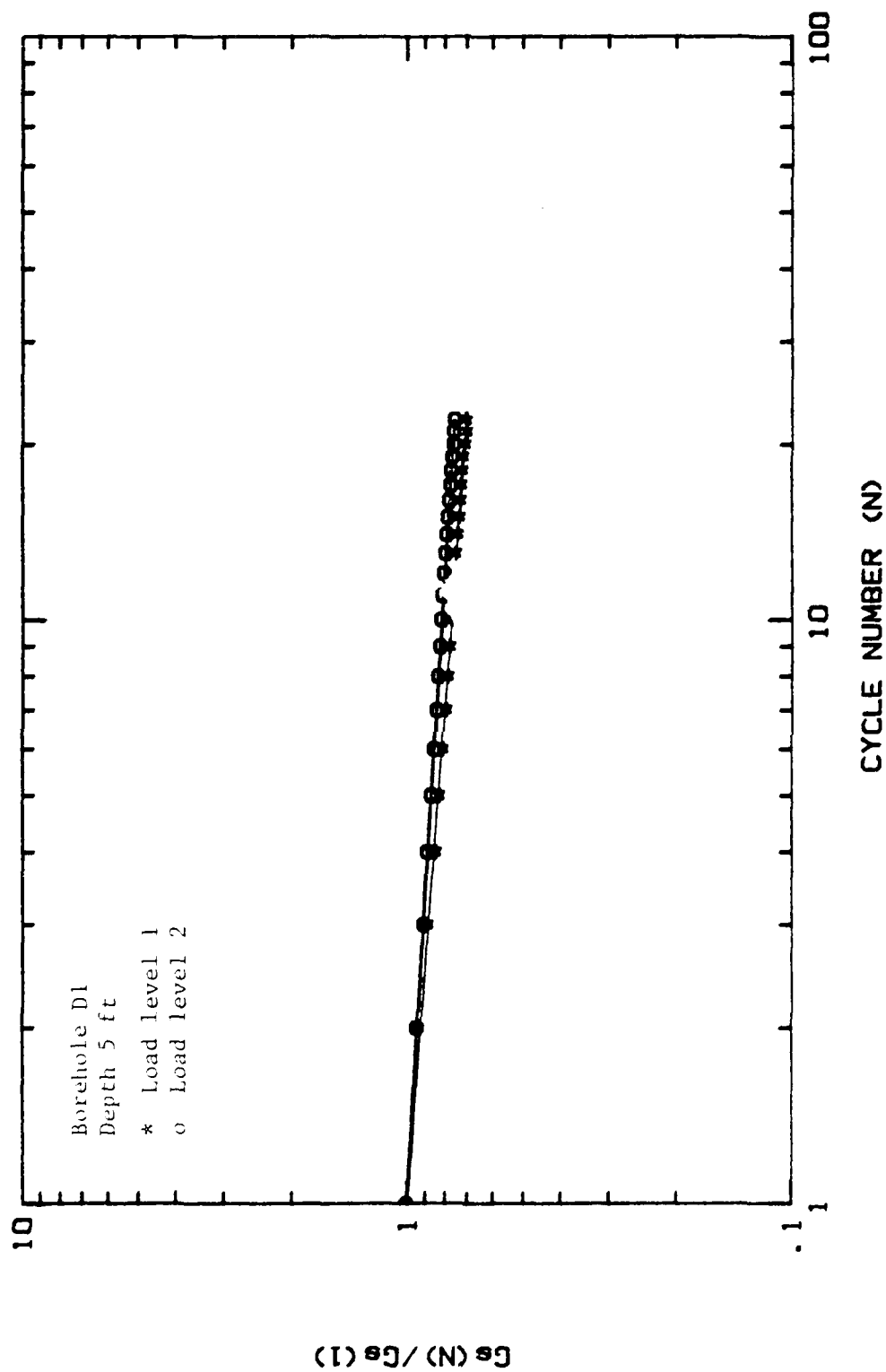Depth 5 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gs(N)/Gs(1)

151

Borehole D1
Depth 9 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gs (N) / Gs (1)

152

Borehole D1
Depth 13 ft
* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gs (N)/Gs (1)

Borehole D1
Depth 17 ft

* Load level 1
o Load level 2

Gs (N) / Gs (1)

CYCLE NUMBER (N)

154

Borehole B1
Depth 7 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gs (N)/Gs (1)

155

Borehole Bl
Depth 10 ft

* Load level 1
o Load level 2

Gs (N)/Gs (1)

CYCLE NUMBER (N)

156

Borehole B1
Depth 15 ft
* Load level 1

Gs (N)/Gs (1)

CYCLE NUMBER (N)

157

Borehole B1
Depth 30 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gs(N)/Gs(1)

158

Borehole B2
Depth 2 ft
* Load level 1
o Load level 2

Gs (N) / Gs (I)

CYCLE NUMBER (N)

159

Borehole B3
Depth 2 ft
* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gs(N)/Gs(1)

160

Borehole D2
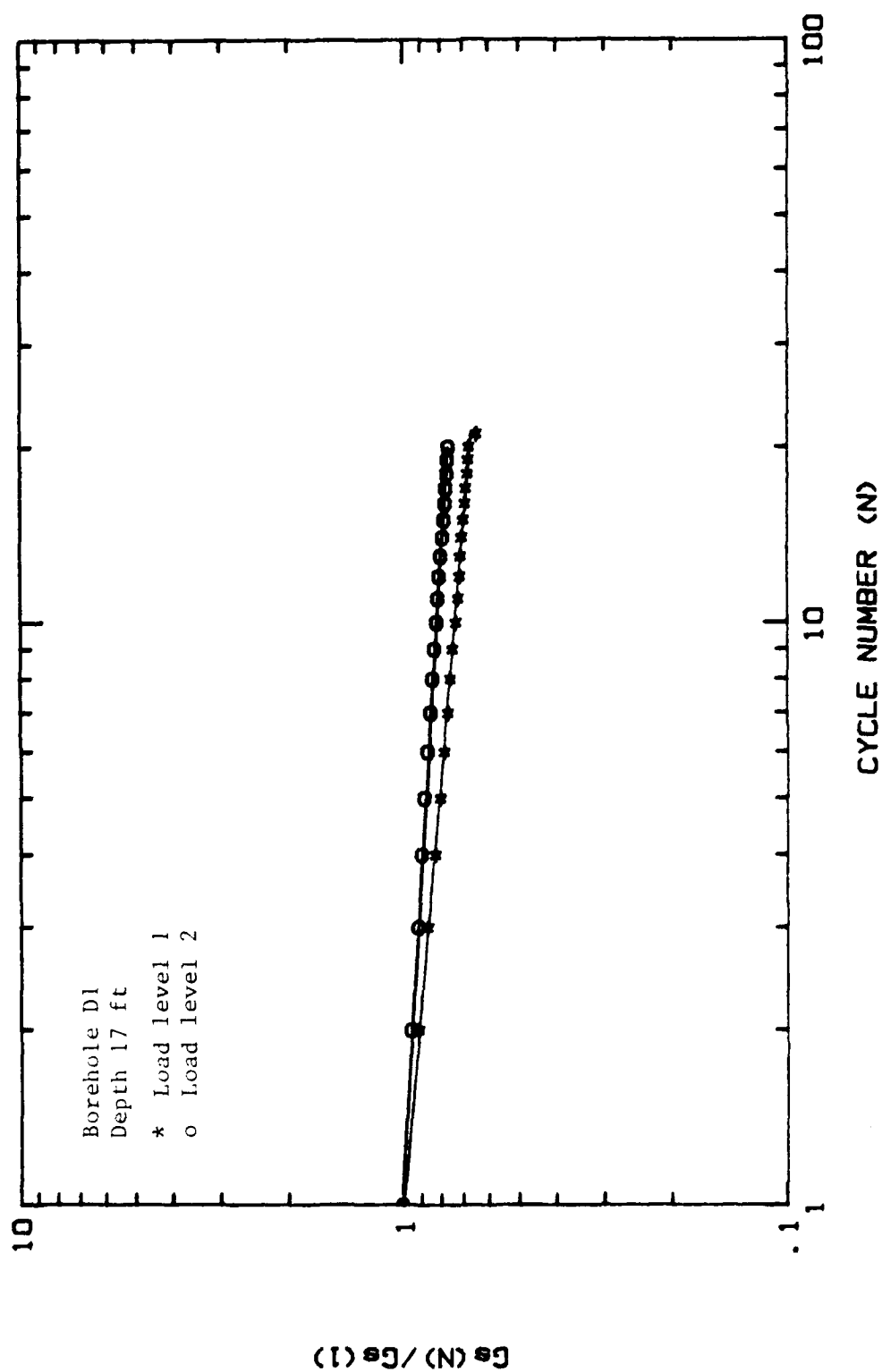Depth 2 ft

* Load level 1
o Load level 2

Gs (N) / Gs (1)

CYCLE NUMBER (N)

APPENDIX D

CYCLIC DEGRADATION OF THE

PMT CYCLIC SHEAR MODULUS

Borehole D1
Depth 2 ft
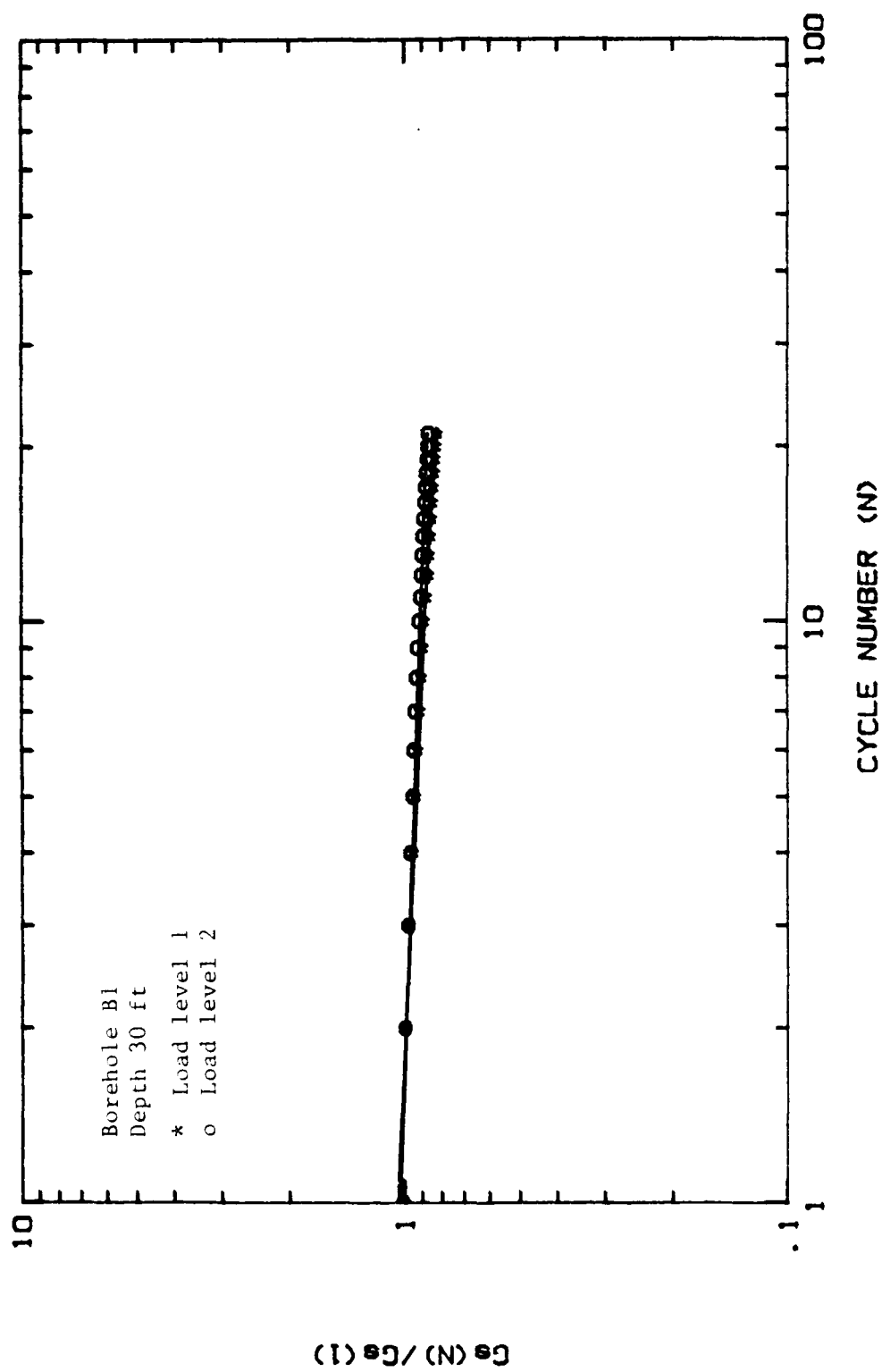
* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

164

Borehole D1
Depth 5 ft
* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

Borehole D1
Depth 9 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc (N) /Gc (1)

Borehole D1
Depth 13 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

167

Borehole D1
Depth 17 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc (N)/Gc (1)

Borehole B2
Depth 2 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

Borehole B1
Depth 7 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

170

Borehole Bl
Depth 10 ft
* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc (N)/Gc (1)

171

172

Borehole B1
Depth 30 ft
* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

173

Borehole B3
Depth 2 ft

* Load level 1
o Load level 2

Gc (N)/ Gc (1)

CYCLE NUMBER (N)

174

Borehole D2
Depth 2 ft

* Load level 1
o Load level 2

CYCLE NUMBER (N)

Gc(N)/Gc(1)

175