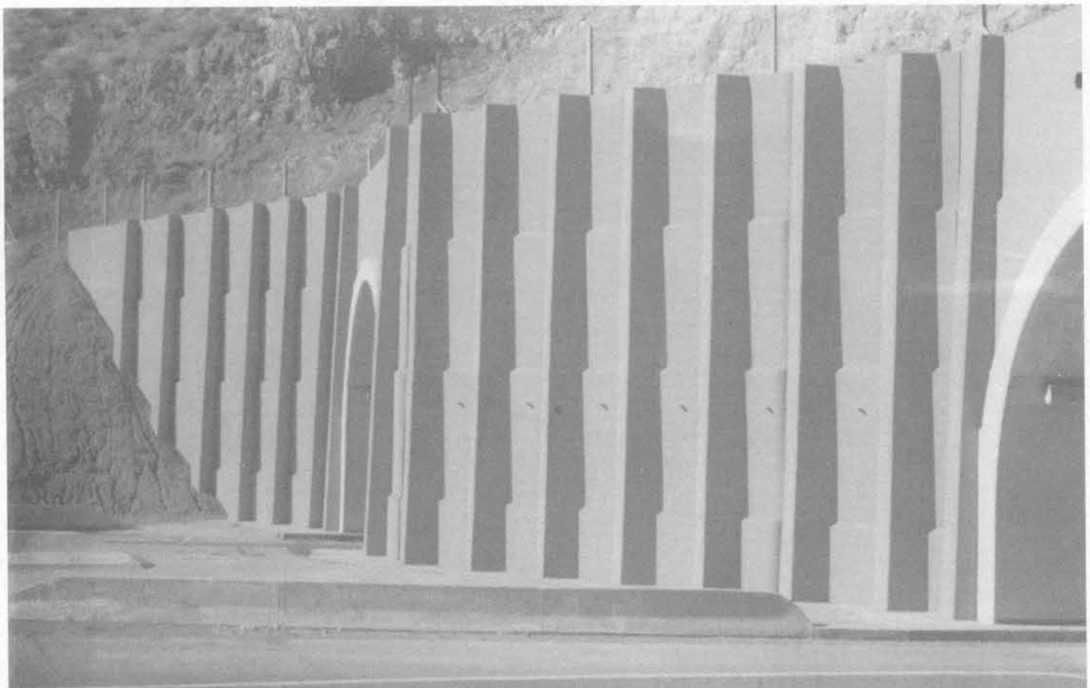
U.S. Department of Transportation
Federal Highway Administration

Demonstration Projects Program

DP-68
PERMANENT GROUND ANCHORS

Volume 2
Field Demonstration Project Summaries



Demonstration Projects Division
Office of Highway Operations

FHWA-DP-90-068-003
April 1990

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-DP-90-068-003		

4. Title and Subtitle	5. Report Date
DP-68 - Permanent Ground Anchors Volume 2, Field Demonstration Project Summaries	April 1990
	6. Performing Organization Code HHO-42
7. Author(s) Richard S. Cheney	8. Performing Organization Report No.
9. Performing Organization Name and Address HHO-30/HHO-40 Federal Highway Administration 400 Seventh Street, SW. Washington, D.C. 20590	10. Work Unit No. (TRAIS)
	11. Contract or Grant No.
12. Sponsoring Agency Name and Address HHO-40 Federal Highway Administration 400 Seventh Street, SW. Washington, D.C. 20590	13. Type of Report and Period Covered Final Report
	14. Sponsoring Agency Code

15. Supplementary Notes

FHWA: Theodore Ferragut

16. Abstract

Ground anchors, often called tiebacks, are structural elements which receive their support in soil or rock and act to retain earth masses and/or applied structural loads. The Federal Highway Administration recognized that the use of permanent ground anchors in highway cut sections could affect substantial benefits in both economy and safety. The specific purpose of the permanent ground anchor demonstration project was to introduce the concept of permanent ground anchor use into American construction practice. A manual numbered FHWA-DP-68-1R and titled "Permanent Ground Anchors" was prepared and several thousand copies distributed to highway engineers. Permanent ground anchors on several projects were monitored to validate the concepts addressed in FHWA-DP-68-1R. This report summarizes the results of both the field monitoring and the history of the permanent ground anchor demonstration project.

17. Key Words	18. Distribution Statement
Permanent ground anchors, tiebacks, anchors, anchor specifications, anchor test projects, anchor acceptance criteria, anchor test procedures	No restriction - NTIS

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

TABLE OF CONTENTS

REPORT NO. 1

REPORT OF FIELD PERFORMANCE
PERMANENT TIEBACK WALL

DEMONSTRATION SITE
I-75 - ATLANTA, GEORGIA

GEORGIA DEPARTMENT OF TRANSPORTATION
by
LAW/GEOCONSULT INTERNATIONAL

PROJECT PEI-75-2(41)
MAY 1985

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES CONTAINED
IN APPENDIX A

TABLES

FIGURES

1. INTRODUCTION

The Georgia Department of Transportation (Georgia DOT) is presently conducting an extensive refurbishing program on the interstate highways that pass through Atlanta's urban environment. An important element of this program is the implementation of many new innovations in design and construction in cooperation with the Federal Highway Administration (FHWA) to develop a modern and economical interstate system. One such innovation involved the design and construction of a permanent tieback retaining wall using ground anchors in lieu of a conventional design using a cantilevered wall in front of a temporary excavation support system.

The permanent tieback wall design and construction project presented in this report was funded through the FHWA's Demonstration Projects Program to "promote and accelerate the adoption of new research results and innovative planning, engineering, and construction practices." The overall objective of this demonstration project was to instrument and monitor the long-term performance of the wall in order to evaluate the design and construction procedures for future Georgia DOT and FHWA-funded highway construction projects. This report describes the instrumentation plan used to monitor the wall and the ground anchors, the testing procedure, the test results of data gathered during the first eight months of monitoring, and presents conclusions and recommendations. It is the intent of the Georgia DOT and the FHWA to monitor the wall's performance over a three year period.

To accomplish the project objective, a comprehensive work plan was developed to verify the wall design assumptions and construction methods and to collect and interpret long-term performance data. The project scope included the following tasks:

- Task A. Measure deflection of the wall face to an accuracy of 0.1 inches.
- Task B. Measure deflection of the wall and ground with slope indicators in the unbonded length of the tieback and beyond the bonded length.
- Task C. Measure load variations in the tieback tendons within the unbonded and bonded lengths at specified stations along the wall.
- Task D. Measure other variables which are deemed necessary for the evaluation of the wall's performance.

This demonstration project is one of many that the FHWA has funded as part of its efforts to transfer technology into useful processes, products and programs. This report presents results of an innovative permanent tieback retaining wall used for the first time by the Georgia DOT. The dissemination of the results will assist other State highway agencies to determine the applicability of permanent tieback walls to their highway construction needs.

2. PROJECT DESCRIPTION

This demonstration project is included as part of Georgia DOT's Project P.E. I-75-02 (41) in Fulton County. This construction project involved the widening of I-75/85 in Atlanta near Fifth Street and in close proximity to the Penthouse Motel and the Coca Cola Bottling Company. The widening required the construction of a retaining wall as shown in Figure 1. The permanent tieback retaining wall section is approximately 500 feet long and its' height ranges from seven to 30 feet. This site was selected to demonstrate the permanent tieback design because of the right-of-way limitations and potential effects of construction on the adjacent structures which limited the type of retaining wall that could be built at this location.

<u>Design Method</u>

The original design, provided by Georgia DOT and the FHWA, included both a cast-in-place concrete cantilever wall and a permanent anchor wall as bid alternates. The general contractor, S. J. Groves, chose the permanent anchor solution and submitted a revised wall designed by Nicholson Construction Company and their subcontractor, Chastain and Tindel, Inc. This revised permanent anchor wall design was reviewed and accepted by Georgia DOT and the FHWA. The design loading on the wall included active earth pressure, and impact and surcharge loads, which were included due to the close proximity of traffic on Williams Street. These loads were to be resisted by the pile embedment (passive earth pressure) and the tieback loads. Active earth pressure loading was assumed to be a uniform distribution acting on the soldier piles and wood lagging above excavation levels. This distribution was assumed to be 65 percent of the computed active case hydrostatic load at the excavation level, as described by Terzaghi and Peck (1967). Hydrostatic earth pressure distributions were assumed to act on the soldier piles below excavation levels. The active component of the hydrostatic load was assumed to act on a 2.5 foot wide pile (effective width of concrete filled drilled hole). Passive earth pressure resistance was assumed in front of the piles below excavation levels. In addition to the 2.5 foot wide pile, as allowed by the specifications, an effective pile width factor of 3 was used. This factor allows the distribution of passive earth pressure to act on an effective pile width (7.5 feet) to account for a soil arching effect in front of the piles. While no direct factor of safety was applied to the passive earth pressure coefficient, a safety margin was included in the design by assuming only one-half of the actual pile embedment in the calculations. No water pressure loading was assumed to act on the retaining wall since groundwater levels were deeper than excavation levels. Also, drainage fabric was to be installed behind the concrete facing to carry away any groundwater that might collect behind the wall.

Tieback loads were calculated using a structural computer program and were checked by hand calculations. Both methods consisted of summing forces and moments and checking for equilibrium. One level of tiebacks was included in the lower portion of the wall, while the remaining portion of the wall included two levels. Design tieback loads ranged from 57 kips (in the lowest portion of the wall) to 138 kips. In cases where the design load in a tieback was not achieved during field proof testing, the requirement for an additional tieback was also calculated by summing moments and forces.

The soldier piles were designed as simply supported steel beams where two or more tiebacks were installed on the soldier pile. Both final configuration and critical construction sequence loadings were examined. The latter calculations were made for intermediate excavation levels to check for maximum moments in the piles occurring during construction rather than at the deepest excavation levels.

Construction Method

The method of construction used in this tieback wall involved several components and sequences. The wall consisted of soldier piles, timber lagging, ground anchors and concrete face panels. First, 30 inch diameter holes were augered from the ground surface to design depths ranging from seven to nine feet below the proposed bottom of the wall. Soldier piles were inserted into the holes. The piles were located on eight or nine foot centers approximately 15 inches behind the proposed wall face.

The structural members used for the soldier piles were double beams (ranging from W12 x 26 to W18 x 46 sections) joined with steel plates. Concrete (3000 psi strength) was placed around the piles from the bottom of the hole to the proposed ground line. Lean concrete was placed above the proposed ground line. As earth in front of the piles was excavated, the lagging was installed between the piles bearing against the steel beam flanges. At designed locations and angles, approximately 6 inch diameter holes were drilled between the two joined beams through the lean concrete and into the soil for placement of the ground anchors. Casing was placed as the hole was advanced to allow insertion of the anchor. The ground anchors were stranded tendons approximately 50 feet long placed at angles from 20 to 30 degrees from the horizontal. Each tendon consisted of from two to five strands with each strand made up of seven wires. The majority of tendons in the higher portions of the wall had either four or five strands. The five strand tendon had an approximate diameter of

0.6 inches and total cross-sectional area of 1.075 square inches. In the unbonded portion of the tendon, a grease filled plastic sheath covered each strand. A corrugated plastic sheath was fitted around the entire tendon in both the unbonded and bonded zone. Grout was injected through the casing as the casing was withdrawn from the hole. The grout eventually filled boththe annular space between the tendon and the corrugated tube, and the corrugated tube and the ground. Grout pressures of approximately 70 pounds per square inch were utilized. Grout pressures were measured with gages located at both the drill head and grout pump to ensure that the pressure was maintained.

After curing of the grout, the anchors were load-tested and post tensioned to a predetermined load. The load test procedure is described in detail in Section 4. In some cases, an anchor would not hold the predetermined design load. In these cases, the anchor was assigned a "safe" load based on the load test results. The designer would then assess whether this reduced load capacity was sufficient. If not, an additional anchor was installed. Additional anchors were required at the northern instrumentation section (Piles 65, 66, and 67).

The process of lagging installation and anchor installation was repeated until the excavation in front of the wall was complete. The final sequence was to install strips of drainage fabric along the lagging and cast concrete face panels over the piles, lagging and tieback retaining system. The concrete face panels were attached to the soldier piles by a series of studs welded to the soldier piles and embedded in the cast-in-place concrete.

Subsurface Conditions

Subsurface conditions at the site were determined by borings performed by the Georgia DOT. The over-burdened soils were described as "medium dense to dense yellowish-brown and pinkish micaceous sandy silts." The micaceous nature of the site soils was of interest because of its potential effect on anchor capacity and particularly long-term creep. The residual soils in the Piedmont geologic province are typically micaceous due to their derivation from parent geneses and sects. Mica content has been estimated (Sowers, 1963) in the Southern Piedmont area to typically range from 5 to 25 percent. Quantification of soil mica content is not commonly performed; therefore, it was not included by Georgia DOT as part of their investigation.

Standard Penetration Test resistances ranged from 11 blows per foot to 60 blows with no penetration. These values generally increased with depth. However, harder and softer layers were frequently encountered in interbedded fashion. Groundwater was encountered approximately 40 feet below the ground surface. The depth to hard rock varies from 45 to 65 feet.

During construction, thin black, iron-manganese coated zones were observed. These zones are common in the Southern Piedmont and are locally termed "slickensides," but are actually unrelated to faulting (St. John, et al, 1969).

3. INSTRUMENTATION PLAN

Background

The instrumentation of the permanent tieback wall consists of several monitoring devices. The instrumentation plan was recommended by Mr. John Dunncliff and is appended to this report. Table I briefly presents all monitoring devices, including type and source of instruments and their primary purpose within the program. A discussion of the instrument plan is contained in the following paragraphs.

A significant change resulted early in the instrumentation program when stranded tendons were selected over the initially planned bar anchors. Strain gages were initially planned to measure strain at designated locations within the bonded zone and thus allow calculation of load transfer. However, it was not deemed feasible to mount strain gages to the tendon strands. Thus, wire telltales, with a custom-fitted fixation to the strand, were used to measure deformation within the bonded zone.

Description

Two monitoring stations spaced approximately 135 feet apart along the wall were selected to monitor the wall performance in higher portions of the wall (Figures 2 and 3). At each monitoring station, two production anchors (upper and lower) on a single pile line were designated for "primary" instrumentation. The production anchor on the piles on both sides of the "primary" anchor was designated for "secondary" instrumentation. The instrumentation for two secondary anchors was destroyed during construction (discussed in Section 7). Thus, a total of four primary and six secondary anchors were instrumented.

Instrumentation for the primary instrumented anchors included: a permanent load cell; short and long rod telltales; and five wire telltales. This arrangement is shown in Figure 4 . The five wire telltales were fixed approximately 7.5 feet apart from the top to the bottom of the bonded zone (Instrument Positions 1 through 5). The short rod telltale was fixed at the top of the bond zone (Instrument Position 1). The long rod telltale was fixed at the bottom of the bond zone (Instrument Position 5). Rod and wire telltales were attached to the same tendon strand. A basic assumption in application of the wire and rod telltales was that all strands within the anchors behaved similarly and that instrumentation of one strand would predict the behavior of the entire anchor.

The secondary instrumented anchors had only one rod telltale fixed to one of the five strands at Position 1. The load cell mounted at the anchor head during load testing was not left in place permanently.

In addition to the anchor instrumentation, each monitoring station included three inclinometers. One inclinometer was placed in a pipe which was welded to the soldier pile for the primary anchors (Piles 51 and 68). The other two inclinometers were located approximately 10 and 50 feet behind the wall.

Extensive horizontal and vertical survey data all along the permanent tieback wall was provided by the Georgia DOT. Survey points were established on the wall top and face. Ground survey points were also established at several distances (10 and 30 feet) behind the wall. Points 50 feet behind the wall were included for a limited portion of the wall. Movement of the inclinometer tops was also measured. Horizontal survey was performed by electronic distance measurement from points across the existing interstate. Vertical survey was performed by routine level measurements.

Each instrument was given a code for identification. This code is presented below.

W - XY - Z

where <u>W</u> = <u>Instrument Type</u>

RT = Rod telltale

WT = Wire telltale

LC = Load cell

SI = Slope inclinometer

<u>X</u> = <u>Soldier Pile Number</u>

See Figures 2 and 3

<u>Y</u> = <u>Anchor Locations</u>

U = Upper Row or L = Lower Row

<u>Z</u> = <u>Instrument Position Designation</u>

See Figure 4

4. LOAD TESTING PROCEDURE

All project anchors were subjected to a load test during installation. The load tests were either a "proof test" or a "performance test". All anchors were to be loaded to 150 percent of their design load. Incremental cycling of load was part of the performance load testing. Anchors were loaded with a center-pull hydraulic jack with an electric pump. Displacement of the loading head was measured with a dial gage attached to a tripod set on the ground.

The acceptance criteria for the anchors was based on the movement of the anchor head (measured by the dial gage) during load testing and creep movement at the maximum test load. An anchor was acceptable if:

A. The total elastic movement obtained exceeded 80 percent of the theoretical elastic elongation of the free length and was less than the theoretical elastic elongation of the free length plus 50 percent of the bond length.

B. The creep movement did not exceed 0.080 inches during a single log-time cycle (five to 50 minutes for performance testing, and one-half to five minutes for proof testing) regardless of tendon length and load.

Anchors which met the above acceptance criteria were unloaded and locked-off at 115 percent of design load. The additional 15 percent of load was included to account for load losses due to lock-off (wedge setting) and potential long-term creep losses. Anchors which did not meet the above criteria were redesigned. This typically involved assuming a new design load equal to 67 percent of the maximum safe load determined during the load test. The anchor was unloaded and locked-off at 115 percent of the "new" design load.

Both anchors which met the original criteria and redesigned anchors were actually unloaded to below the design load and then reloaded to the lock-off load. This is because the jack pressure (which determines load) is considered to be more accurate in loading than in unloading. Thus, by loading up to the lock-off load, a more accurate reading is obtained.

Several differences in the load test procedures were utilized for the instrumented anchors (primary and secondary). A 150-ton and a 300-ton capacity hydraulic jack was utilized for loading the secondary anchors and primary anchors, respectively. The primary anchors required a larger jack because the center hole in the 150-ton jack was not large enough to accommodate the instrumentation and the anchor. In addition to the jack load reading, a load cell was used to monitor the anchor loads. For this reason, it was not necessary to unload and reload the anchors to obtain an accurate lock-off load. A different loading sequence occurred at four anchor locations. At anchor locations 52-upper, 67-upper and 67-lower the anchors were unloaded to the seating load (alignment load) and then loaded back to the lock-off load. At anchor location 66-lower, the load was cycled (loaded and unloaded) at approximately 50 percent of design load and cycled again at 150 percent of design load. Some details of the load testing at each of the instrumented anchors are shown on Figures 6 through 15 (Load Displacement Curve - Load Test).

5. TEST RESULTS

Test results presented and discussed within the following section are divided into those obtained during load testing of anchors and those obtained during the long-term (approximately eight months) monitoring program.

<u>Load Test: Load-Displacement</u>

Loads applied during load testing of both primary and secondary anchors were measured with both the jack pressure gage and load cell. However, the load data presented within this section were measured with the load cell because it is generally considered to be more accurate. Displacements of the anchor head were measured with a dial gage which read displacement independent of anchor movement. Plots of load versus head displacement are shown in Figures 6 through 15. The acceptance criteria (A) discussed in the Section 4 are also shown on these figures. The free and bond lengths used for determination of the acceptance criteria are shown on Figures 6 through 15. Some results of the load tests have been included in Table IV, Summary of Anchor Performance.

Displacement of the long and short rod telltales and wire telltales #1 and #5 for the primary anchors are also shown on Figures 6 through 9. The long rod telltale and wire telltale #5 measure displacement of the pulling head with respect to the anchor bottom. As may be seen only minor differences exist between the dial gage and long rod telltale readings for primary Anchors 51-upper, 51-lower, and 66-lower. This indicates that the bottoms of these anchors showed no or very little movement during the load test. Anchor 66-upper (Figure 8) indicates a difference of approximately 0.1 inches between the dial gage and the long rod or wire #5 telltale. Thus, a displacement of this amount of the anchor bottom may have resulted during load testing.

The modulus of elasticity of the unbonded length can be determined during the load test for the primary and secondary anchors by comparison of load versus elongation of the unbonded length. This elongation is measured with the short rod telltale, or wire #1, both of which are anchored at the interface between the unbonded and bonded zones. Modulus values determined in this manner are presented in Table III. The reported manufacturer's modulus was 28.5×10^6 pounds per square inch. As may be seen in that table, the calculated modulus values during loading are considerably less than the manufacturer's modulus. A potential explanation for this difference in slippage of the rod and wire telltale anchors relative to the tendon, causing the telltales to overestimate elongation. The slippage could be caused by the tendon "breaking out" of the grout with the telltale anchors restrained by the grout. The possibility of slippage cannot be discounted, despite a soft material which was placed immediately in front of the telltale anchors (around the tubes containing the rods or wires) designed to alleviate this problem. As discussed later, an evaluation of load transfer data tends to support the hypothesis of slippage of these telltale anchors.

As discussed previously, five and 50 minute creep tests were performed at the maximum test load for the secondary and primary anchors, respectively. The results of these tests are presented in Table II.

We have also utilized the creep test data to make a preliminary assessment of creep potential. For this assessment, we have used a procedure presented in the report "Permanent Ground Anchors: Soletanche Design Criteria," dated September 1982 (Report No. FHWA/RD-81/150). This procedure involves the performance of several "design load tests" prior to the installation of production anchors. In these tests, the anchor load is held for one hour at different increasing load levels. These "design load tests" were not performed as part of either the design or construction of the permanent tieback wall. However, we have utilized the results from 50 minute creep tests performed both for the primary instrumentation anchors and as part of performance testing for selected production anchors. This data is plotted (Figure 16) in a manner similar to that suggested in the Soletanche Report. The data appears to generally suggest that the "critical creep tension" was not reached during these tests.

<u>Load Test: Displacement and Load Distribution in Bonded Zone</u>

Displacement along the bonded anchor zones of the four primary anchors was measured using a series of five wire telltales. The spacing between the wire telltales, as shown in Figure 4, was approximately 7.5 feet. Displacement at the top and bottom of the bonded zone was also measured with the short and long rod telltales which were attached at the same place on the tendon strand as the wire telltales #1 and #5, respectively. A comparison of the short rod versus wire #1, and the long rod versus wire #5, is shown in Figures 17 and 18 (Appendix A) for the four primary anchors. Because the rod and wire telltale are fixed at the same location on the tendon strand, they should measure approximately equal deflection (that is, approximately zero relative deflection on Figures 17 and 18). Relatively good agreement (less than 0.05 inch relative deflection) was obtained for Anchors 51-upper, 66-upper, and 51-lower (wire #1 versus short rod, only). Lesser agreement was obtained for Anchor 86-lower (0.05 to 0.10 inch relative deflection) and Anchor 51-lower, wire #5 versus long rod (0.10 to 0.15 inch relative deflection). For both Anchors 51-lower and 66-lower we have also shown relative deflection of the long rod versus dial gage, which should be approximately equal if no deflection occurs at the bottom of the bonded zone. Good agreement between the dial gage and long rod was obtained for Anchor 51-lower, thus indicating that wire #5 overestimated deflection. In Anchor 88-lower, the dial gage was greater than the long rod which could indicate either displacement at the bottom of the bonded zone or relative instrument accuracy. Wire #5 deflection exceeded dial gage deflection, which probably indicates that wire #5 overestimated deflection.

The displacement measured by each wire telltale with respect to wire telltale #1 (relative deformation) during load testing is shown on Figures 19 through 22. The difference between wire telltale curves at a given load represents the relative deformation between points. Example: the difference between curves 2-1 and 3-1 represents the relative displacement between points 2 and 3). Positive relative deformations between any two points (2-1, 3-1, 3-2, etc.) represents elongation of that segment of the tendon. The displacement of the long rod telltale with respect to the short rod telltale is also shown on these figures.

The relative deformations within the bonded zone must meet two criteria which are based on two closely related assumptions. These criteria and their related assumptions are:

A. Relative deformation (elongation) should increase with increased distance between points (the 3-1 difference should be greater than the 2-1 difference, etc.) assuming that the tendon is in tension. The wire 5-1 difference or the long rod-short rod difference (whichever is more correct) should form the upper bound of the plots shown on Figures 19 through 22.

B. The elongation of adjacent equal length segments must decrease moving down the tendon. That is, the wire 2-1 difference must be greater than the wire 3-2 difference (3-2 = 3-1 minus 2-1), etc. This assumes that tension in the tendon decreases with increased distance from the top of the bond zone.

The relative deformations presented in Figures 19 through 22 may be examined utilizing these two criteria. Anchor 51-lower, with the displacements measured by wire #3, #4, and #5 all exceeding the long rod displacement, will be excluded from this consideration. Examination based on data for the remaining three anchors (Figures 19, 21, and 22) yields the following general points:

A. The wire 2-1 difference is less than the wire 3-1 difference (Criterion I) in all three cases (including Anchor 66-upper for which the wire 2-1 difference was negative). However, the wire 2-1 difference is less than the wire 3-2 difference (3-2 = 3-1 minus 2-1) in all three cases, which is contrary to Criterion II.

B In two of three anchors (88-upper and 66-lower) the wire 3-1 difference is less than the wire 4-1 difference (Criterion I). In both cases, the wire 4-3 difference is less than the wire 4-3 difference (Criterion II).

C. The wire 4-1 difference is less than the wire 5-1 difference in Anchor 51-upper (and also less than the wire 3-1 difference) and close to the wire 5-1 difference in Anchors 66-upper and 66-lower. In general, deformations in this portion of the bonded zone appear to be in the range of the data accuracy. Thus, it appears difficult to resolve deformations in this portion of the bonded zone.

In summary, the wire 2-1 difference appears too small in all cases, while the validity of the remaining data appears mixed. A potential reason for this apparent underestimation of the 2-1 relative deformation (elongation) is slippage of the wire #1 (and short rod) telltale anchorage. As discussed previously, this slippage could cause an overestimation of elongation of the unbonded length and, thus, an underestimation of elongation of the bonded zone (particularly the upper 7.5 feet of the bonded zone, when wire #1 is compared to wire #2). Slippage of the telltale anchors, caused by the grout restraining free movement of the telltale anchor along with the tendon, appears more likely at Point #1 than at the other telltale anchorage points. This is because higher tensions and displacements occur in this portion of the tendon than in deeper portions of the bonded zone. The higher tensions and displacements increase the possibility that the bond between the tendon and the grout could be broken and thus that relative displacement between the two could occur. In addition, the grout within the unbonded zone (free length) is not stressed by the tendon and thus acts to restrain the outward movement of grout in the upper portion of the bonded zone.

The amount of possible slippage of the wire #1 and short rod telltale anchors can be estimated by making the assumption that telltale anchor slippage is the only cause of difference between the apparently underestimated modulus values determined during the load test and the manufacturer's modulus. Thus, the difference between the elongation compatible with the manufacturer's modulus and the elongation measured by wire #1 (or the short rod) could approximately equal the amount of slippage. The amount of estimated slippage increases with increasing load and is approximately 0.3 inches at maximum loads. If the wire #1 and short rod readings are corrected for the estimated slippage, all the wire differences (2-1, 3-1, 4-1, 5-1, and the long rod-short rod) plotted on Figures 19 through 22 would be moved to the right with respect to the ordinate.

The purpose of deflection measurements within the bonded zone was to provide an indirect calculation of load transfer. The question of slippage of the wire #1 and short rod telltale anchor significantly effects the results of this calculation. This effect will be further discussed below. First, however, we will discuss the manner in which load transfer is estimated from relative deflections within the bonded zone.

Elongation of a segment of the tendon is a function of strains along that segment length. The elongation equals the integration of strain versus length (the area under the strain versus length plot). Thus, elongation is directly related to the average strain within that segment. Stress (and load) are directly related to strain at any point along the tendon (Hooke's Law). Thus, the shape of load versus length plot must be directly related to the shape of the strain versus length plot, and average load directly related to average strain. Further, the average load within a segment, as a measure of the area under the load distribution curve, is directly related to the elongation of the segment, and can be calculated from that elongation. The simplifying assumption that strains and loads change linearly within a segment allows positioning the calculated average load at the mid-point of the segment. The change in load from one point in the tendon to another point is the load transfer between those points.

As discussed previously the question of slippage of the wire #1 and short rod telltale anchors significant affects our calculation of load transfer. Thus, the two possible outcomes of the question lead to two separate lines of reasoning, which are developed separately below:

A. <u>Assumption of No Telltale Anchor Slippage</u>: If we assume that slippage of the wire #1 and short rod telltale anchors has not occurred, then the wire 2-1 differences become too small compared to the wire 3-2 differences (Criterion II). Thus, both these wire differences appear in question and it appears impossible to utilize them to calculate average loads and load transfer in the upper 15 feet of the bonded zone. However, if slippage has not occurred, then a good estimate of overall bond zone elongation appears possible utilizing the wire 5-1 or long rod-short rod difference. This data is supported by the previously discussed agreement between the short rod versus wire #1, and the long rod versus wire #5. A significant result of the previous discussion on calculation of load transfer is that elongation of the entire bonded zone (or any segment of the bonded zone) is directly proportional to the area under the load distribution curve. Thus, although it is not possible to calculate average loads within segments of the bonded zone, it may be possible to derive general trends in the load distribution. Anchors 51-upper and 66-lower both showed approximate 0.6 inch bonded zone elongation at 150 kips above the seating load. Thus, Part A of Figure 23 shows three hypothetical load distribution diagrams with elongation of the entire bonded zone equal to 0.6 inches. Examination of these diagrams shows that significant load transfer must occur within the first two segments of the bonded zone to result in 0.6 inches of elongation. Lesser load transfer rates in this portion of the bonded zone would result in increased area under the curve and thus increased bonded zone elongation.

B. <u>Assumption of Telltale Anchor Slippage</u>: If we assume that slippage of the wire #1 and short rod telltales has occurred, then the wire difference curves on Figures 19 through 22 are shifted to the right, as discussed previously. As stated, we may calculate the amount of slippage based on the manufacturer's modulus and use it to correct the wire #1 data. This increases the wire 2-1 difference, but does not affect the relative differences between the other curves (the wire 3-2, 4-3, and 5-4 differences). The overall elongation of the bond zone (the wire 5-1 or long rod-short rod difference) is, of course, also increased. The maximum slippage, as previously discussed, is estimated to be approximately 0.3 inches. Adding this amount to the 0.6 inches discussed previously for Anchors 51-upper and 66-lower, yields a total bond zone elongation of approximately 0.9 inches. Part B of Figure 23 shows hypothetical load distribution diagrams with elongation of the entire bonded zone equal to 0.9 inches. In addition, we may use the adjusted wire 2-1 difference and the other wire differences to calculate average segmental loads and thus derive a load distribution plot which is based on the slippage assumption. This has been performed for Anchors 66-upper and 66-lower. After adjustment of the data from all four anchors for assumed slippage, we judge the data from these two anchors to be more reasonable than the other two primary anchors. Load distribution plots for these two anchors are shown on Figure 24. Load distribution for selected loading increments are shown. At each load increment we have plotted calculated average load within bonded zone segments. We have also shown a smoothed load distribution curve which meets the criteria, stated previously, that area under the curve must be directly related to elongation of the entire bonded zone. These curves intersect the ordinate of the plots at the load cell reading, which is assumed to equal load at the top of the bonded zone. The calculated average segmental loads and the smoothed load distribution curve are both based on deflection data which begins at the seating load. Thus, these loads must be added to assumed load distribution at the seating load. This assumed distribution is shown on the plots.

In summary, our estimate of load distribution within the bonded zone is significantly affected by the question of slippage of the wire #1 and short rod telltale anchors. The assumption of no slippage results in our inability to utilize the wire differences to estimate loads, and results in relatively high load transfers within the upper portions of the bonded zone (based on total elongation of the bonded zone). The assumption of anchor slippage is based on the low modulus values calculated for the unbonded zone during the load test. Application of the estimated slippage makes the wire differences appear more reasonable and allows calculation of average segmental loads. Load transfer based on the anchor slippage assumption appear more reasonable than that based on the no slippage assumption. Thus, there appears to exist several points supporting the slippage assumption. However, we do not feel that it is possible to decide conclusively between these two assumptions and associated results. Future projects might provide insight into this question.

Load Test: Comparison between Load Cell and Hydraulic Jack

Figures 25 through 30 show jack load reading divided by load cell reading versus load cell reading during load testing for primary and secondary anchors. The load cell reading was chosen for the abscissa because it is assumed to be more accurate that the jack load reading. As may be seen the jack load reading in almost all cases is greater than or equal to the load cell during loading and less than the load cell during unloading. The load as measured by the jack has an apparent accuracy of + 5 to 10 percent. This agrees with most published data which generally cite off-center loading and end effects as probable cause of inaccuracy.

Long term: Load at Anchor Head

Long-term variation in load at the anchor head was measured on the primary anchors with load cells and short rod telltales (telltale load cells), and on the secondary anchors with only short rod telltales.

The functioning of the short rod telltale as a load indicator is as follows. (A sketch and description are also contained in Figure 3 of the appended instrumentation plan. The short rod telltale measures displacements between the top of the bonded zone and the anchor head. Thus, the short rod telltale measures changes in length of the unbonded anchor strand. These changes in length can be converted into load changes (via Hooke's Law) within the unbonded zone. It is assumed that no load transfer occurs within this unbonded zone. Thus, these load changes are also indicative of load changes at the anchor head. Because the wire telltale #1 also measures changes in length of the unbonded zone, it can be used as a telltale load cell in a similar manner. The unbonded length in the final lock-off position (the length utilized in the telltale load cell calculations) is less than the unbonded length during the load test. This is because a portion of tendon was cut off during the lock-off. This reduction in length was typically 4 to 5 feet.

Utilization of Hooke's Law, as discussed above, requires assumption of a modulus of elasticity (E). As discussed previously, modulus values of the unbonded length determined during load testing were significantly less than the reported manufacturer's modulus and may have been caused by slippage of wire #1 and short rod telltale anchors. For this reason, it was decided to use the manufacturer's modulus.

Long-term load variation at the anchor head is shown for the primary anchors on Figures 31 and 32. Load change measured by the load cell and calculated by both the rod and wire telltale load cells are shown. As may be seen, there is good agreement in three of four primary anchors between the loads changes measured with the load cell and those calculated from the rod telltale. This agreement tends to support the decision to use the manufacturer's modulus in the telltale load cell calculations. The difference between the load indicators in Anchor 51-lower cannot be explained. Based on the comparisons, it appears that the short rod telltale is a relatively reliable indicator of long-term load change. Figures 33 through 35 show load variations in the six secondary anchors as measured with the rod telltale load cell. Measurement of long-term load change for both primary and secondary anchors was begun after setting wedges to lock-off the anchor into final configuration.

Approximately eight months of load variation data has been collected for all ten instrumented anchors. Seven of the anchors have shown approximately 5 kips or less variation from their lock-off condition (Anchors 50-upper, 51-upper, 51-lower, 52-upper per, 65-upper, 65-lower, and 66-upper). Of the remaining three anchors, two have shown increase in load of approximately 10 kips (Anchors 66-lower and 67-lower) while Anchor 67-upper has shown a decrease in load of approximately 15 kips. The load changes after eight months have been added to the seating loads, and are shown in Table IV. We have also attempted to estimate the long-term load loss (creep) in anchors. These results are also shown in Table IV. These estimated creep values should be considered preliminary and should be reevaluated when additional long-term data is available.

Long-term: Displacements

Three inclinometers (SI) are located behind each instrumented station (Piles 50-52 and 65-67). The Number 1 inclinometers were installed in a steel pipe welded to Piles 51 and 66. The Number 2 and 3 inclinometers are located in the soil approximately 10 and 50 feet behind the retaining wall, respectively. The horizontal inclinometer deflections observed at different stages of construction are shown in Figures 36 and 37.

The inclinometer casings were extended approximately 10 feet below the bottoms of the soldier piles as shown in Figures 36 and 37. As may be seen by examination of the inclinometer profiles on those figures, fixity was not exactly achieved at the base of the casings. The amount of base deviation was small and the effect on the overall readings appears negligible. This is generally confirmed by horizontal survey of the inclinometer casing tops. Comparisons of survey data versus inclinometer top data are shown in Figures 41 and 42. The procedure of "check sums" was used to determine inclinometer accuracy. This confirmed that the inclinometers were providing reliable data.

The maximum horizontal retaining wall movement measured by the inclinometers was about 3/4 inches toward the excavation. The magnitude of the horizontal movements 10 feet behind the wall (Number 2 inclinometers) are similar to that of the retaining wall; however, the vertical distribution of the movement is somewhat different. Somewhat greater deflection occurred in Inclinometer SI-66-2 than in Inclinometer SI-66-1. The deflection

Downward deflections of the surveyed pile monitor points indicated a downward maximum deflection of approximately 0.08 feet (1.0 inch). As may be seen in Figure 44 these deflections were erratic with respect to wall height. The pile settlement was probably caused by the downward component of tieback or earth pressure loads. Ground survey points 10 feet and 30 feet behind the wall showed a maximum settlement of 0.07 feet (0.8 inch) and 0.02 feet (0.2 inch), respectively. As may be seen in Figure 44 a good correlation apparently exists between wall height and surveyed ground settlements. Surveyed vertical ground deflections averaged approximately 50 percent of horizontal deflections.

Maximum horizontal wall movements measured by the inclinometers and by survey were approximately 0.8 inches and 1.2 inches, respectively. These correspond to approximately 0.2 percent and 0.3 percent of wall height, respectively. Goldberg, Jaworski and Gordon (1976) reported a range of horizontal movements of 0.1 percent and 0.8 percent of wall height for tieback walls in sand and gravel. Experience with standard cantilever retaining walls in the Atlanta area indicates that development of the active earth pressure case requires a horizontal wall movement of approximately 0.3 to 0.6 percent of wall height. In our opinion, movements of the permanent tieback wall are in the range of those generally associated with the active earth pressure condition for soils in the Atlanta area.

One purpose of the survey data was to determine whether or not the instrumented sections were typical of the rest of the wall. Examination of Figure 43 and 44 appears to indicate that they were typical of the higher portion of the wall.

Long-term: Displacements and Load Distribution in Bonded Zone

The wire #5 and long rod telltales measure relative deformations between the anchor head and the bottom of the bonded zone (i.e. the entire anchor). Examination of the long term data indicates elongations of the anchor ranging from 0.05 to 0.20 inches. This is compared to outward movements of the wall and anchor head, of approximately 3/4 inch. Thus, as the wall deformed outward, the entire anchor (including the bonded zone) also moved outward approximately 1/2 to 3/4 inch. Inclinometer SI-66-3 was located near the bottom of the anchors (see Figure 37) and measured approximately 0.4 inches of deflection. Thus, movement of the bottom of the anchors may be related to overall movement of the soil mass rather than displacement of the bonded zone in relation to the surrounding soil.

Overall elongation of the entire anchor, as measured by the wire #5 and long rod telltale, consists of unbonded length deformation (indicative of load in the anchor) and bonded length deformation (indicative of load transfer within the bonded zone). Figures 45 and 46 show long-term relative displacement within the bonded zone in a manner similar to that presented previously in Figures 19 through 22. As before, the long rod-short rod difference and the wire 5-1 difference are indicative of elongation (for positive differences) of the entire bonded zone. Good agreement was obtained between the long rod-short rod difference and the wire 5-1 difference for three of the four primary anchors (the exception being Anchor 51-upper). For the three anchors for which agreement was obtained, the elongation of the entire bonded zone was approximately 0.1 inch in each case. As before, elongation of the bonded zone is directly related to area under the load distribution curve. Thus, the additional long-term elongation indicates relatively less load transfer near the top of the bond zone and relatively increased load transfer deeper within the bond zone.

6. DISCUSSION OF WALL PERFORMANCE

The object of the discussion within this report section is to bring together the instrumentation results and use them to evaluate the wall design.

Maximum outward movements of the wall have been approximately 3/4 to 1 1/4 inches which corresponds to 0.2 to 0.3 percent of wall height, respectively. This range of movement appears to generally conform with that anticipated for the active soil pressure condition, for which the wall was designed. Soil movement has apparently occurred some distance behind the wall, creating an outward movement of the bonded zone of the anchors.

In the long term (eight months), seven of the instrumented anchors have maintained their lock-off loads, two have increased somewhat (approximately 10 kips), and one has decreased somewhat (approximately 15 kips). Table IV shows the anchor load after eight months with respect to design loads. In that the anchor loads have not significantly increased above the design loads appears to indicate that the design loading conditions and resulting design tieback loads were not significantly underestimated. Had this been the case, the loading in the anchors would have increased to withstand these higher loads.

This discussion on long-term tieback load with respect to design loading assumptions is generally true for the three anchors which experienced somewhat greater load variation. For the two anchors which experienced an increase in load (Anchors 66-lower and 67 lower), it is possible that the design somewhat underestimated the actual loading. However, it is also possible that the design assumptions overestimated resistance of the pile embedment in this area, thus requiring the lower anchors to carry additional load. For the anchor which experienced a decrease in load (Anchor 67-upper) two possibilities exist. One is that the anchor bonded zone displaced an additional amount toward the wall in addition to the general outward movement in the wall and anchor. A second is that the pile moved inward due either to stressing of the intermediate anchor, or possibly a redistribution of load toward the bottom of the wall (as reflected in increased load in Anchor 67-lower). However, the second possibility is not supported by either Anchor 65-upper or 66-upper both of which had an intermediate anchor below them. In addition, Anchor 66-lower experienced a load increase similar to that of Anchor 67-lower without a decrease in load in Anchor 66-lower.

An intermediate level of tiebacks was required at the northern instrumentation section (piles 65, 66, and 67) because of failure of the upper level of anchors to achieved proposed design loads. The design loads for these intermediate anchors were calculated in the same manner as the original design loads, including the actual load for the upper level of anchors. Thus, while long-term load change data is not available for the intermediate anchors, it appears that the conclusions within this section on wall performance and the relation to wall design are also applicable to this case.

7. INSTRUMENTATION RECOMMENDATIONS

The instrumentation on this project has generally provided the intended information. While some problems have surfaced, these are to be expected in an instrumentation program for which there was little precedent. The successful implementation of the program required intensive coordination between ourselves, the instrumentation consultant, the tieback contractor, the instrument suppliers, and the overseeing agencies. The customized design of the instrumentation necessitated many iterations between these parties. The need for this coordination cannot be overemphasized. The ability to select and procure the proper instrumentation on a "preferred sole source" basis was critical to the successes of the program. If similar instrumentation is planned on future projects we recommend that the work be performed under a professional services contract rather than be subjected to bid as part of the construction contract.

beyond the retaining wall and therefore induced a moment at the spherical bearing location. Methods to stabilize the assembly during testing included very high alignment loads, supporting the jack with an overhead crane, and bolting the assembly to the soldier pile with long threaded rods and nuts. A convenient method of assembly was never developed. Future considerations to handle this problem could be: omission of the spherical bearings; or a "chair" similar to the one used for the jack which could be modified to support the load cell also. Bracing from the soldier pile and/or the ground below could be used for additional support.

Connection of the wire leads to the load cell was inconvenient. On future projects each of the load cells should have a permanently installed lead wire with environmental connectors.

<u>Rod Telltales</u>: Installation of the rod telltales was generally straight forward. For the short rod telltale on two of the intended secondary anchors it was impossible to insert the rod through the tube to the telltale anchor. The rod for one telltale could be inserted only a short distance, presumably because grout had entered the tube either at the telltale anchor connection, at a break in the tube, or at the top cap. In remaining installations this top cap was glued in place and cut off after grouting. The rod for the second unsuccessful telltale could be inserted almost to the telltale anchor. It was concluded that the tube had be come disconnected from the telltale anchor at their threaded connection. This connection could be improved in future installations by providing a longer threaded section.

Twisting of the tendon and attached rod and wire telltales was recognized as a potential problem prior to initiation of the instrumentation program. This twist is believed to be a major cause of the problems when interpreting aspects of the wire and rod telltale data. After attachment of the telltales and placement of the tendon in the anchor hole, there was no way to maintain alignment of the instruments during grouting. As the grout was pumped, the casing was twisted and pulled. Difficulty in inserting the rod into the rod telltale sleeve is believed to be evidence of significant twist. However the actual degree of rotation experienced by the telltales and tendon is unknown. We believe there is no way to alleviate this problem in future installations involving stranded tendons.

As discussed previously slippage may have occurred at the connection between the short rod telltale and the tendon. This possibility was recognized during the design phase, and substantial precautions were taken, including machining the base of the anchor to match the tendon spiral, gluing the two surfaces together, double-banding around the anchor and tendon, using a high tension banding system, and placing a soft material in front of the telltale anchors. Improvements should be sought in future installations, but space limitations may prevent a solution to the problem. As a minimum, the telltale anchors could be notched, where the steel bands hold it to the strand, to improve mechanical interaction between them. Three bands, instead of two, would make the shear strength of the telltale anchor-strand connection tighter. Additional soft material placed in front of the anchor could help alleviate the forces causing slippage. However, slippage may be impossible to prevent, particularly in the upper portion of the bonded zone where high loads in the tendon create the greatest possibility for differential movement between the tendon and grout.

The telltale load cell (short rod telltale) has, in three of four cases, compared well with data from the load cell in providing long-term load change data. Thus, the telltale load cell appears to be a relatively reliable indicator of long term load change. In the one case where disparity exists it is not possible to determine which is correct, although it would be generally assumed that the load cell is more accurate. Where space within the anchor allows, two telltale load cells attached to different strands could provide redundancy in future installations.

<u>Wire Telltales</u>: The wire telltales were used to measure deformation and thus estimate load transfer within the bond zone. The wire telltales gave more problems than the other instrumentation both in terms of installation, reading, and performance. In comparisons between rod and wire telltale data, good agreement was achieved in some cases while in other cases the wire telltales appears to have overestimated deflection. Several of the other wire telltales appear to have yielded unreasonable information.

Small collars attached to the top of the wire telltales allowed the indicator to grip the telltale. The system was identical to a proven multi-point wire extensometer system which has performed well on other projects. However, when the range of the indicator was exceeded, it was necessary to move these collars. Because the wires were relatively stiff, they crimped easily during placement of the collars and made this operation difficult to perform properly. It is possible that some of the collars may have slipped during the load test and long term readings. In future installations, an improved method of collar attachment should be devised. Alternately, by use of spacers or shims in conjunction with the indicator, it might be possible to eliminate the need for collar relocation.

The Peter Smith Mark I indicator used to read the wire telltales performed poorly. An increased reading range would help alleviate the problem discussed above. The indicator also required that the tensioning load be transmitted through the micrometer used to measure deflection. Thus, the tensioning load was significantly limited by the strength of the micrometer threads. An instrument with a larger range of tensioning loads would provide greater reading accuracy, by allowing a check on the functioning of the wires to be performed by a comparison of readings made at low and high tensions. This multi-tension check was part of the original instrumentation proposal. The inability to perform these multi-tension checks reduced the reliability of the wire telltale data on this project. Such checks should be included as part of future wire telltale instrumentation.

As discussed previously, slippage of the wire and rod telltale anchors may have also contributed to instrument error, and the measures recommended earlier for rod telltale anchors should also be adopted for wire telltale anchors. Where space within the anchor allows, we recommend additional telltales close to the top of the bonded zone. These would provide additional reference points should the question arise of slippage of the first telltale in the bonded zone. In addition, higher tensile loads exist in this zone than deeper in the anchor. This creates greater tendon elongations which are easier to detect amidst the normal data variation.

An assumption common to both the wire and rod telltales is that deformations measured in one strand are representative of the entire tendon. General long term agreement in the telltale load cell (short rod telltale) with respect to the load cell tend to support the assumption. Instrumentation of duplicate strands, which would provide insight into this assumption, is typically limited because of space limitation.

The success of the use of wire telltales on a stranded tendon to estimate load transfer within the bonded zone on this project is in question because of possible anchor slippage. In addition, certain inaccuracies exist in the remainder of the data. The problem of tendon twist during the pulling of casing may be the cause of these inaccuracies. In that case, the problem of twist may be impossible to overcome. However, the implementation of the improvements suggested within this section should improve the data quality and could yield more successful results.

Inclinometers: The inclinometers appear to have provided consistent and reliable information. Although examination of the data indicates that base fixity was not quite obtained, the deviations were small and do not appear to have caused significant error in deflection measurements.

Survey: Survey appears to have provided the reliable data anticipated from this proven instrumentation technique. The survey points were installed, read and reported by the Georgia DOT crews. Thus, credit for success should belong to them. As should be expected, many points on the top of the wall were destroyed before significant data could be obtained.

Long-Term Readings: We recommend continued long-term readings of load cells, short and long rod telltales, inclinometers, and survey points. Long-term reading of wire telltales #1 and #5 should also be considered to provide redundancy for the rod telltales. If this is performed, readings for wire telltales #2, #3, and #4 could also be continued and might provide information concerning long-term redistribution of load transfer within the bonded zone.

8. CONCLUSIONS

The following are our primary conclusions concerning the permanent tieback retaining wall, Georgia DOT Project P.E. 1-75-02 (41).

Instrumentation Results

Conclusions which may be drawn from the test data are summarized in the following points:

1. During the load test, the bottoms of the primary instrumented anchors showed little or no movement. Estimation of modulus values for the anchor free length may have been affected by slippage of the short rod and wire #1 telltale anchors.

2. The question of telltale anchor slippage during the load test also affects the estimation of load transfer within the bonded zone. If slippage did not occur, relatively high load transfer appears to have occurred in the upper portion of the bonded zone. If slippage did occur, estimated load transfer appears more reasonable.

3. Comparison of jack pressure gage loads with load cell results indicates that jack gage overestimates load during loading and underestimates load during unloading. Load as measured by the jack has an apparent accuracy of + 5 to 10 percent.

4. Significant variation in load at the anchor head has not occurred for seven of the ten instrumented anchors. Two of the other anchors have gained load (approximately 10 kips) while one has lost load (approximately 15 kips).

5. Most of the measured wall deflection occurred within one to two months after excavation. Long-term measurements indicate that deflection has stabilized. The magnitude of both horizontal and vertical deflections appear to be related to wall height, and are well within the range of past experience.

6. The anchors' bonded zones were displaced a distance of approximately 1/2 to 3/4 inch during outward deflection of the wall and anchor head. This appears to be the result of overall soil deformation. Some load redistribution within the bonded zone occurred during this displacement.

Wall Performance

Based on the results of the load testing and long-term load and displacement performance monitoring to date, it appears that the permanent tieback wall is performing satisfactorily. The assumptions and procedures used to design the wall and the construction methods used to accomplish the design also appear satisfactory. Thus, it can be stated that the permanent tieback retaining wall system is a suitable design and construction alternative for future highway construction in similar soils in the State of Georgia and in the southeastern United States.

It is recognized that one of the major concerns of owner-agencies is the anchor permanency against long-term corrosion and/or creep; or: "What is the design life of the anchor wall?" This concern can be addressed by relating the past 25-40 years of experience in the use of permanent tieback walls to the design, installation and testing procedures used in this project.

The design life of anchor permanency can be ensured through establishing the technical feasibility for a permanent tieback wall for the specific site, an evaluation of the risk by the owners based on the feasibility study, the pre-qualification of designers and contractors, the selection of tieback type including state-of-the-art corrosion protection, and the establishment of load testing acceptance criteria and long-term monitoring.

To estimate the design life of this particular permanent anchor wall, one must evaluate the short-term load holding capacity of the anchors, the long-term load holding behavior which is eight months to date, and consider the design and construction procedures. The test results presented in this report indicate that the wall is behaving as designed. The design and construction was carried out by qualified professionals, and the corrosion protection system selected and installed is assumed to be providing the long-term corrosion performance for which it was designed. Therefore, the life expectancy of this permanent tieback wall can be estimated to be comparable to a reinforced concrete structure designed and constructed with similar professional care.

Safe Design Loads

Ultimate capacity of the instrumented anchors is shown in Table IV. Ultimate loads obtained on this project typically ranged from four to five kips/foot of bond length. Anchor loads for the subject wall were reduced from these ultimate loads by use of a safety factor. The data for the instrumented anchors and overall wall system indicate that under similar subsurface and construction conditions, preliminary safe design loads on other projects could be derived from these ultimate loads and a similar safety factor. Of course, actual loads must be confirmed with load testing.

TABLE I

SUMMARY OF INSTRUMENTATION

INSTRUMENT TYPE	MANUFACTURER	PARAMETER MEASURED	EQUIPMENT (MAJOR PARTS)
Inclinometer	Slope Indicator, Co.	Horizontal deformations of wall face and retained soil	. Inclinometer system with Digitilt sensor, LCD indicator, 2.75" o.d. plastic casing installed using grout backfill.
Load Cell	Brewer Engineering Laboratories, Inc.	Load in anchor	. 225-kip capacity with BLH 120 Indicator Box, and digital readout, bearing washers and spherical bearing plates. Gage and calibrated pull bar to re-establish zero reading at any time.
Wire Telltales	Geokon, Inc.	Load distribution in anchor bond zone	. s/s wire type 16 G.A. . 1/4" oil-filled nylon tube around each wire . 5/8" grooved s/s anchor attached to the strand using clamps and epoxy . 3/4" pvc pipe on the axis of the entire anchor with slots for grout entry and nylon tube entry. . Peter Smith Mark I extensometer readout. . Readings made using a chair attached to the anchor head.

APPENDIX A

TABLE I (con't.)

SUMMARY OF INSTRUMENTATION

INSTRUMENT TYPE	MANUFACTURER	PARAMETER MEASURED	EQUIPMENT (MAJOR PARTS)
Rod Telltales	Geokon, Inc.	Load change in anchor Provides data redundancy in bond zone	. Flush-coupled 1/4" s/s rod within an oil-filled pvc pipe. . Anchors same as wire telltale . 1-7/8" s/s reference plate required on the anchor head. . s/s chair for in-situ calibration . micrometer for readout
Survey	Provided by Georgia DOT	Horizontal and vertical deformation of wall face and top of retained soil	. Electronic distance meter (EDM) for horizonzontal deflections . conventional leveling techniques.

TABLE II

SUMMARY OF ACCEPTANCE CRITERIA
CREEP TESTING
INSTRUMENTED ANCHORS

ANCHOR	Creep/Cycle (inches)	LENGTH OF TEST (minutes)
51 upper	0.003	50
51 lower	0.020	50
66 upper	0.001	5
66 lower	0.002	50
67 upper	0.008	5
67 lower	0.017	10
65 upper	0.029	5
65 lower	0.009	10
52 upper	0.013	5
50 upper	0.008	5

The creep per log cycle computed by:

$$\text{creep/cycle} = \frac{d_2 - d_1}{\log T_2/T_1}$$

d - deflection
T - time

TABLE III

APPARENT MODULUS OF UNBONDED TENDON LENGTH

DETERMINED DURING LOAD TESTS

	Short Rod Telltale Data		Wire Telltale #1 Data	
	E Loading (psi)	E Unloading (psi)	E Loading (psi)	E Unloading (psi)
PRIMARY ANCHORS				
51-lower	21.4x10^{6}	30.7x10^{6}	21.9x10^{6}	29.2x10^{6}
51-upper	22.4x10^{6}	26.3x10^{6}	22.4x10^{6}	29.5x10^{6}
66-lower	22.7x10^{6}	28.9x10^{6}	26.3x10^{6}	27.3x10^{6}
66-upper	19.3x10^{6}	30.6x10^{6}	20.0x10^{6}	29.8x10^{6}
SECONDARY ANCHORS				
67-lower	23.9x10^{6}	25.7x10^{6}	-	-
67-upper	24.0x10^{6}	23.9x10^{6}	-	-
52-upper	24.2x10^{6}	25.2x10^{6}	-	-
50-upper	19.3x10^{6}	36.2x10^{6}	-	-
65-lower	22.4x10^{6}	32.8x10^{6}	-	-
65-upper	26.2x10^{6}	30.4910^{6}	-	-

TABLE IV

SUMMARY OF INSTRUMENTED ANCHOR PERFORMANCE

ANCHOR	BOND LENGTH (FEET)	ULTIMATE LOAD LOAD (KIPS)	ULTIMATE LOAD KIPS PER FOOT	DESIGN LOAD (KIPS)	LOAD AFTER SETTING WEDGES (KIPS)	LOAD AFTER EIGHT MONTHS (KIPS)	CREEP KIPS PER LOG CYCLE
51-U	32	189.9*	5.9	132.6**	128.4	132.9	††
51-L	35	188.0	5.4	125.3	126.0	126.0	3.9
66-U	32	131.9*	4.1	91.8**	94.1	92.3	††
66-L	35	155.0	4.4	104.1	99.9	108.9	0.5
50-U	33	190.5	5.8	129.7	147.6†	145.6	1.9
52-U	33	201.7	6.1	138.8	154.6†	152.4	3.4
65-U	32	114.2*	3.6	78.4**	83.0	79.3	††
65-L	35	147.2	4.2	98.0	94.7	94.1	0
67-U	32	130.2*	4.1	91.1**	94.2	79.2	0.7
67-L	35	158.7	4.5	105.8	106.1	117.3	†††

NOTES:

Grout pressure of approximately 70 psi used on all anchors

* Approached acceptance criteria before reaching predetermined ultimate load; other anchors achieved predetermined load without approaching criteria.

** New design load based on results of load test.

† Lock-off Load (load after setting wedges not determined)

†† Load variation judged too erratic for accurate creep determination.

††† Anchor gained load.

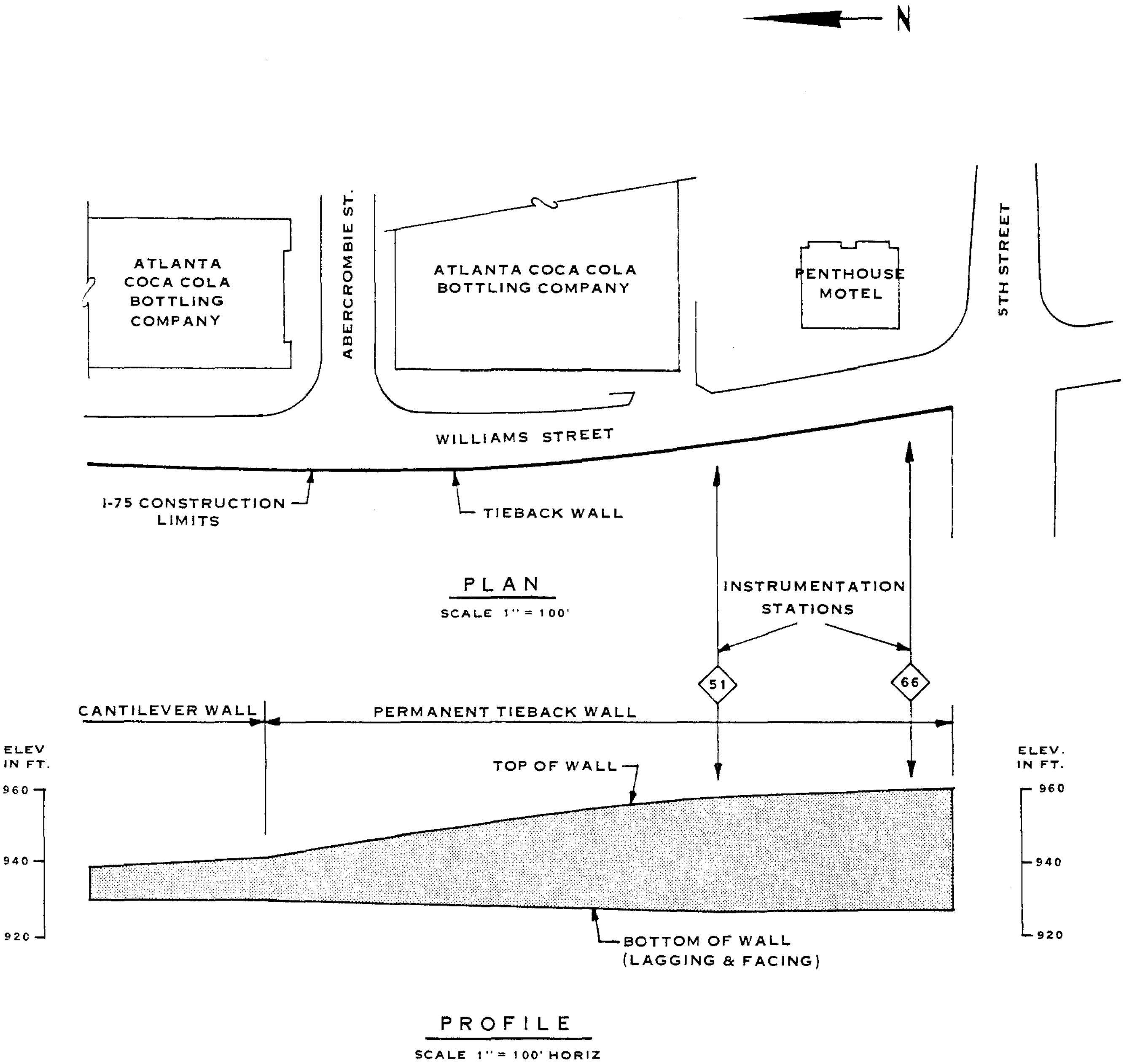


FIGURE 1 - PLAN AND PROFILE OF PERMANENT TIEBACK WALL

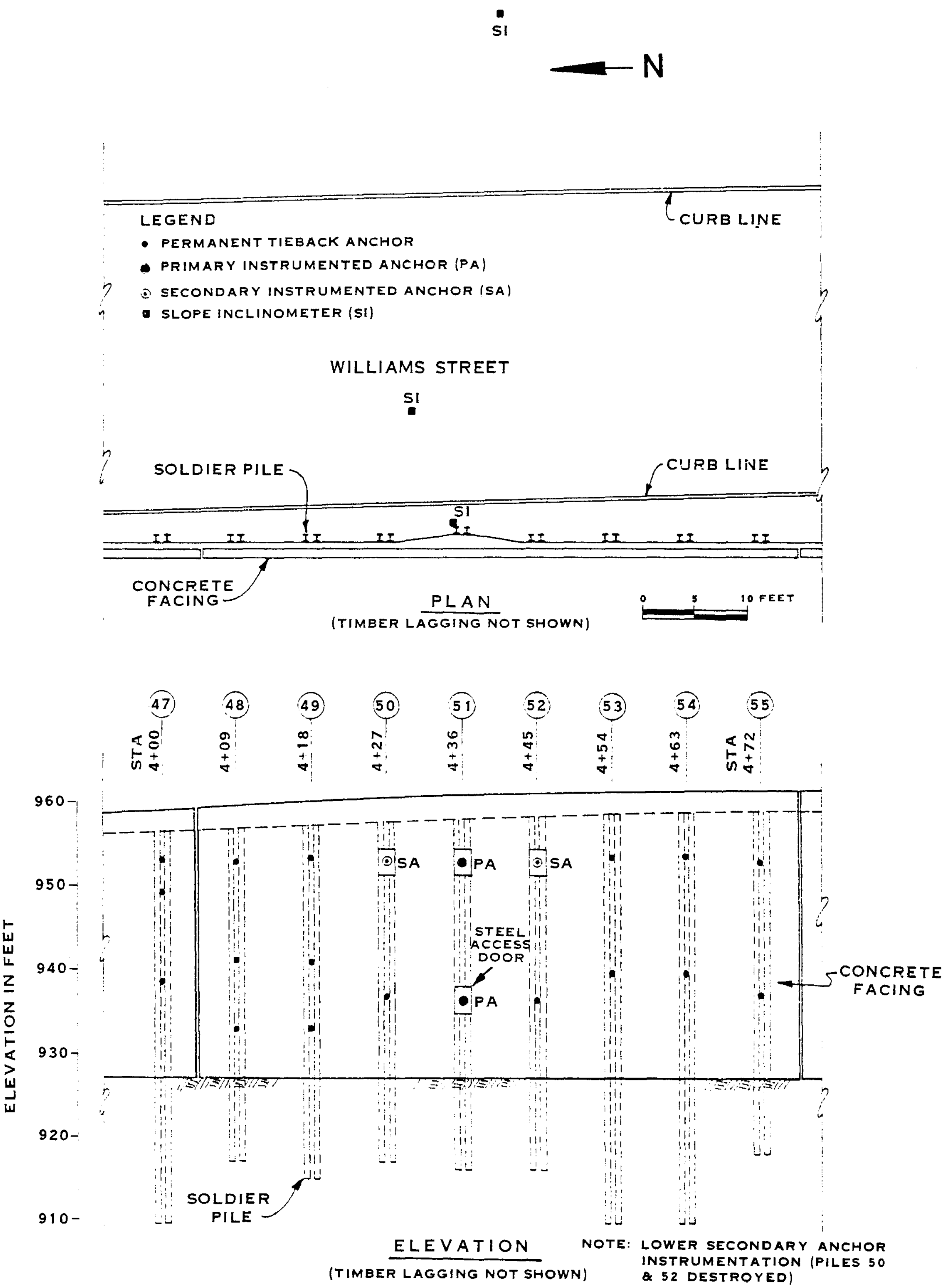


FIGURE 2 - PERMANENT TIEBACK WALL INSTRUMENTATION

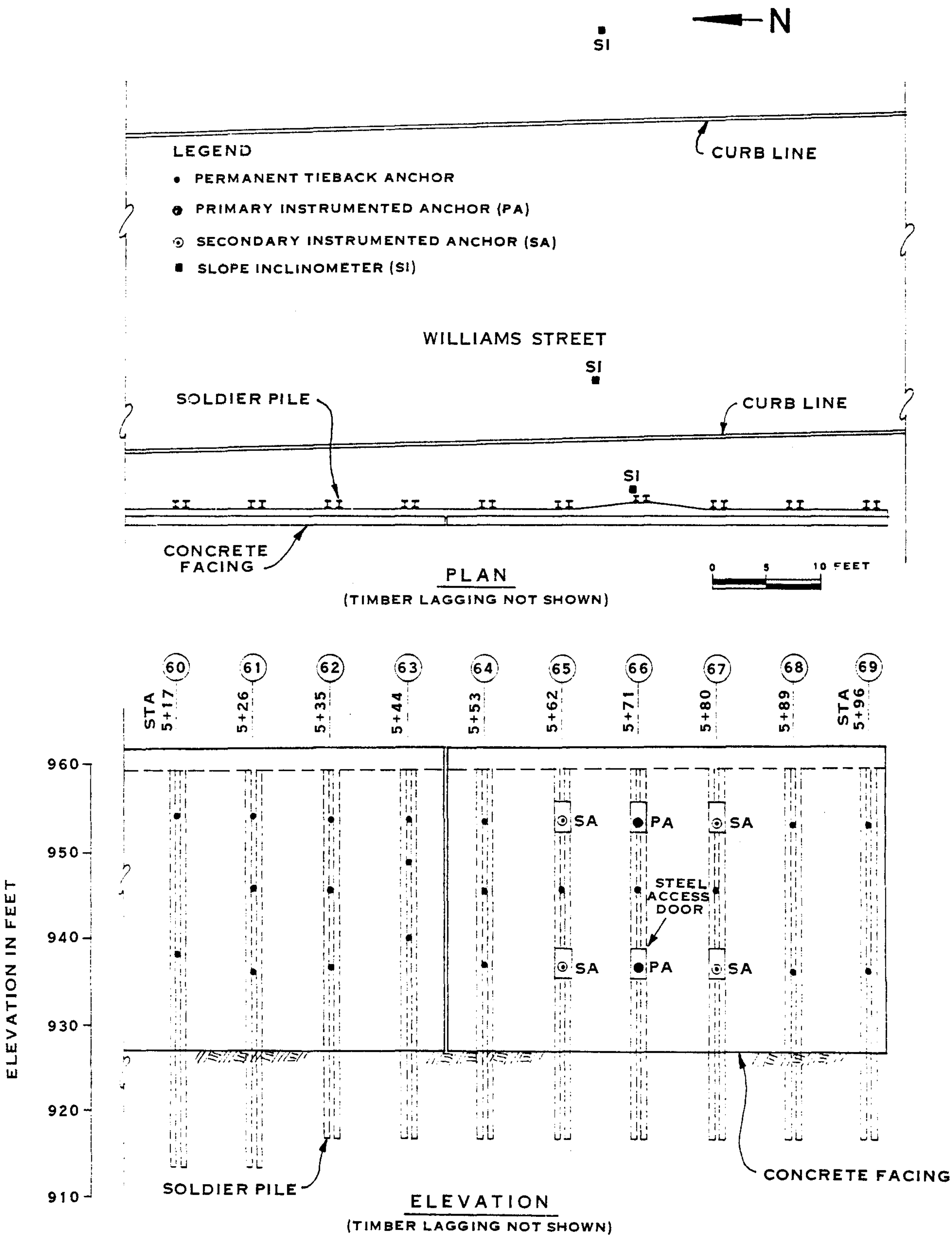


FIGURE 3 - PERMANENT TIEBACK WALL INSTRUMENTATION

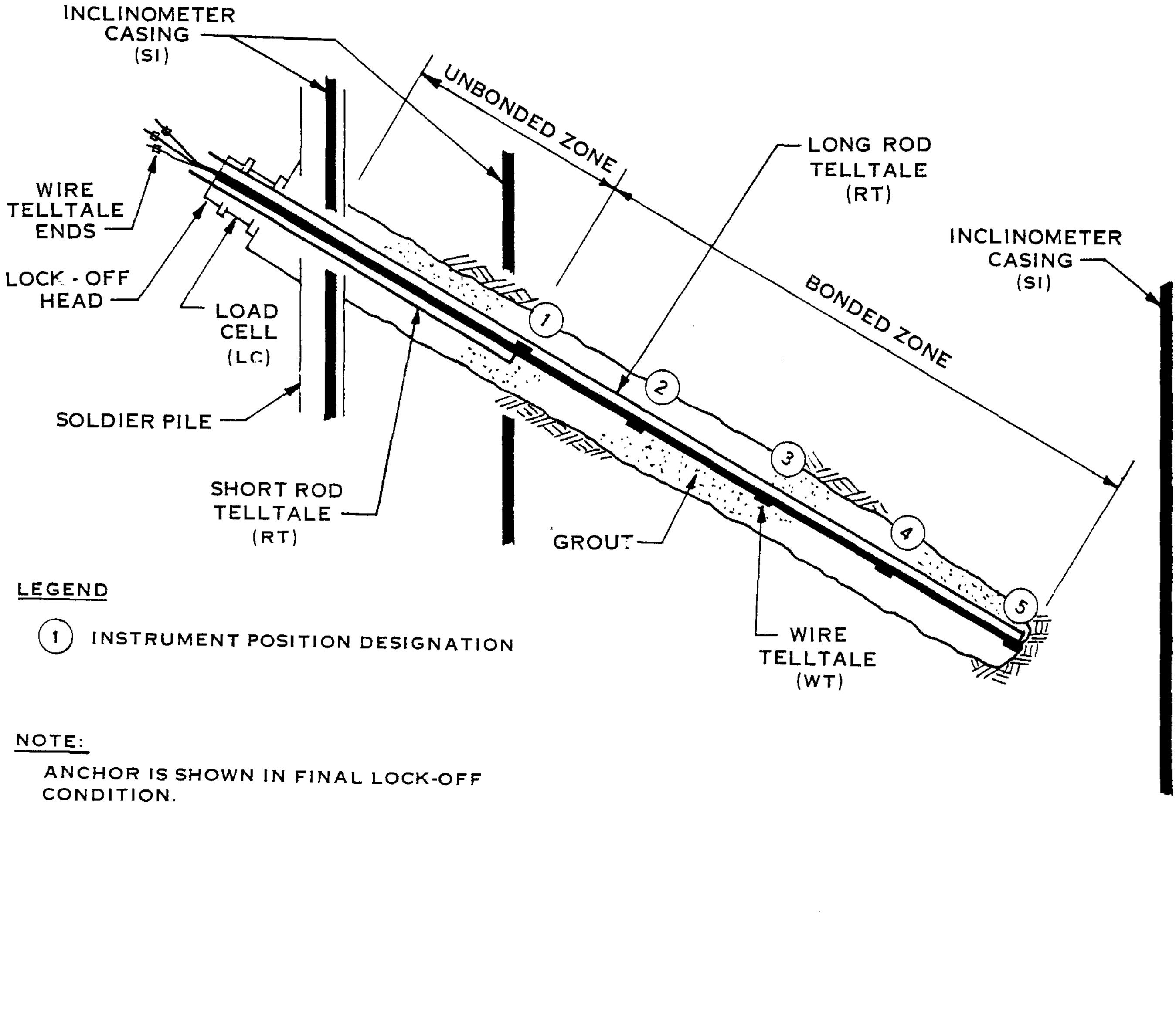


FIGURE 4 - SCHEMATIC OF PRIMARY INSTRUMENTED TIEBACK ANCHOR

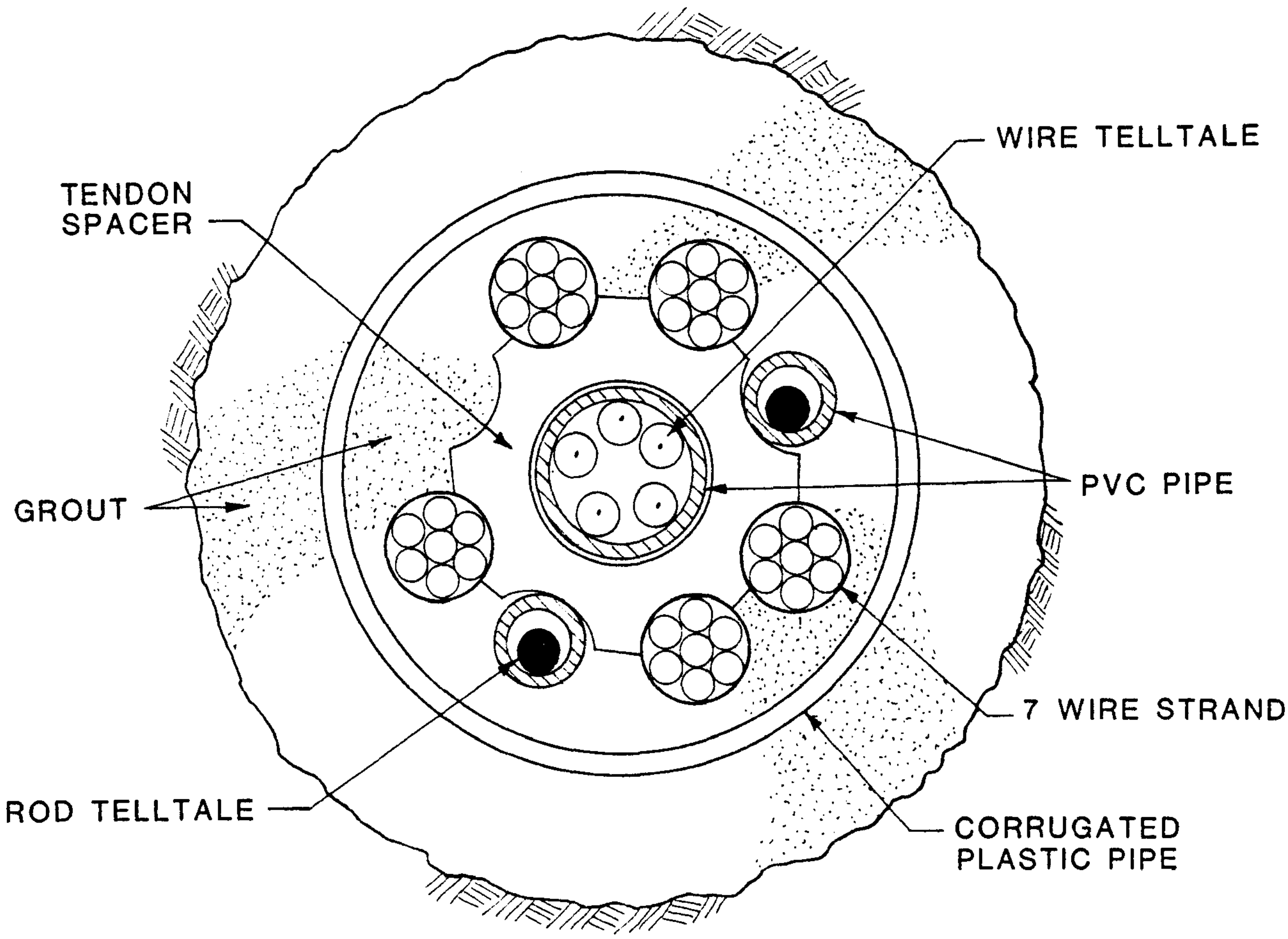


FIGURE 5 - CROSS SECTION OF INSTRUMENTED TIEBACK ANCHOR

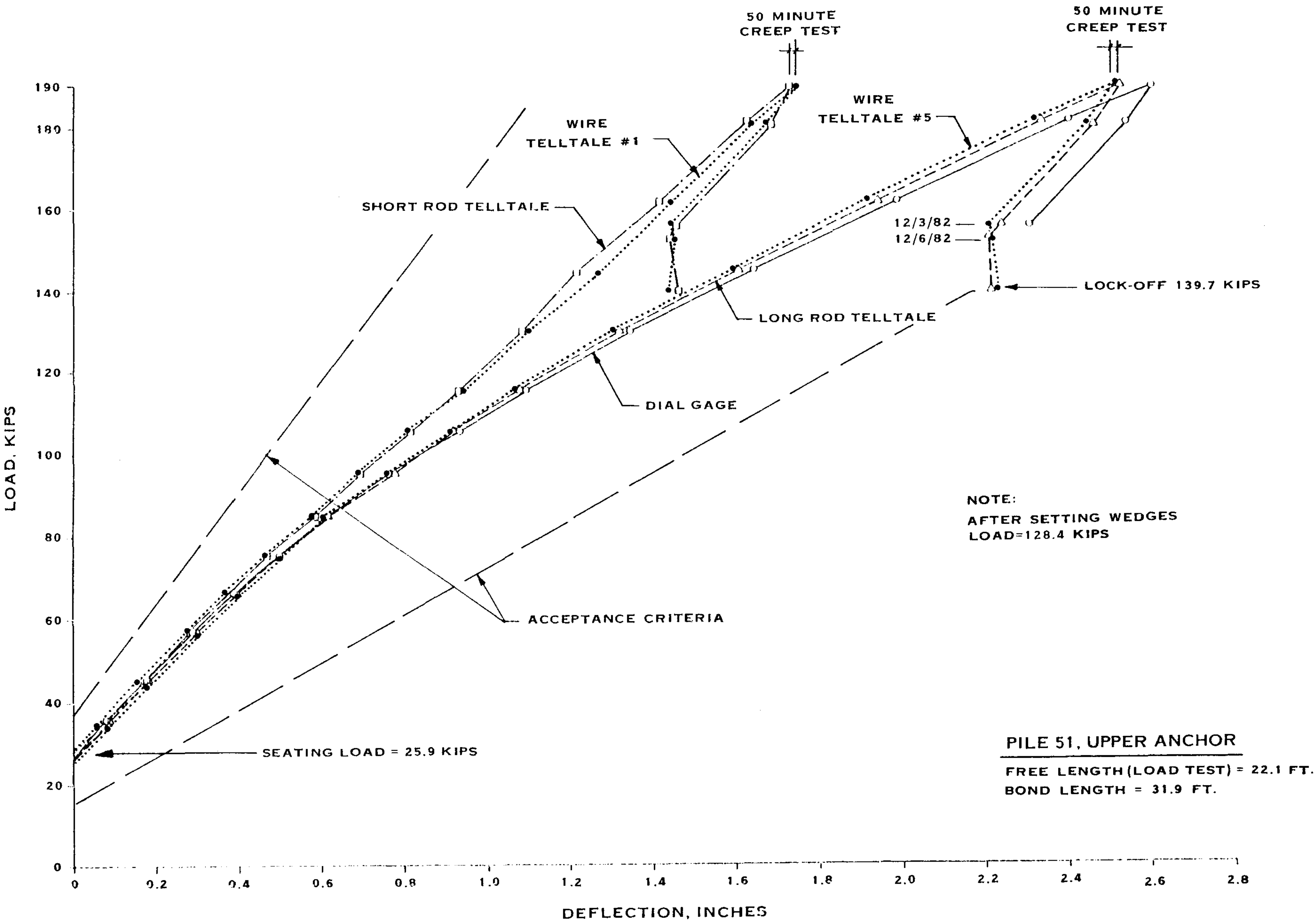


FIGURE 6 - LOAD DISPLACEMENT CURVE - LOAD TEST

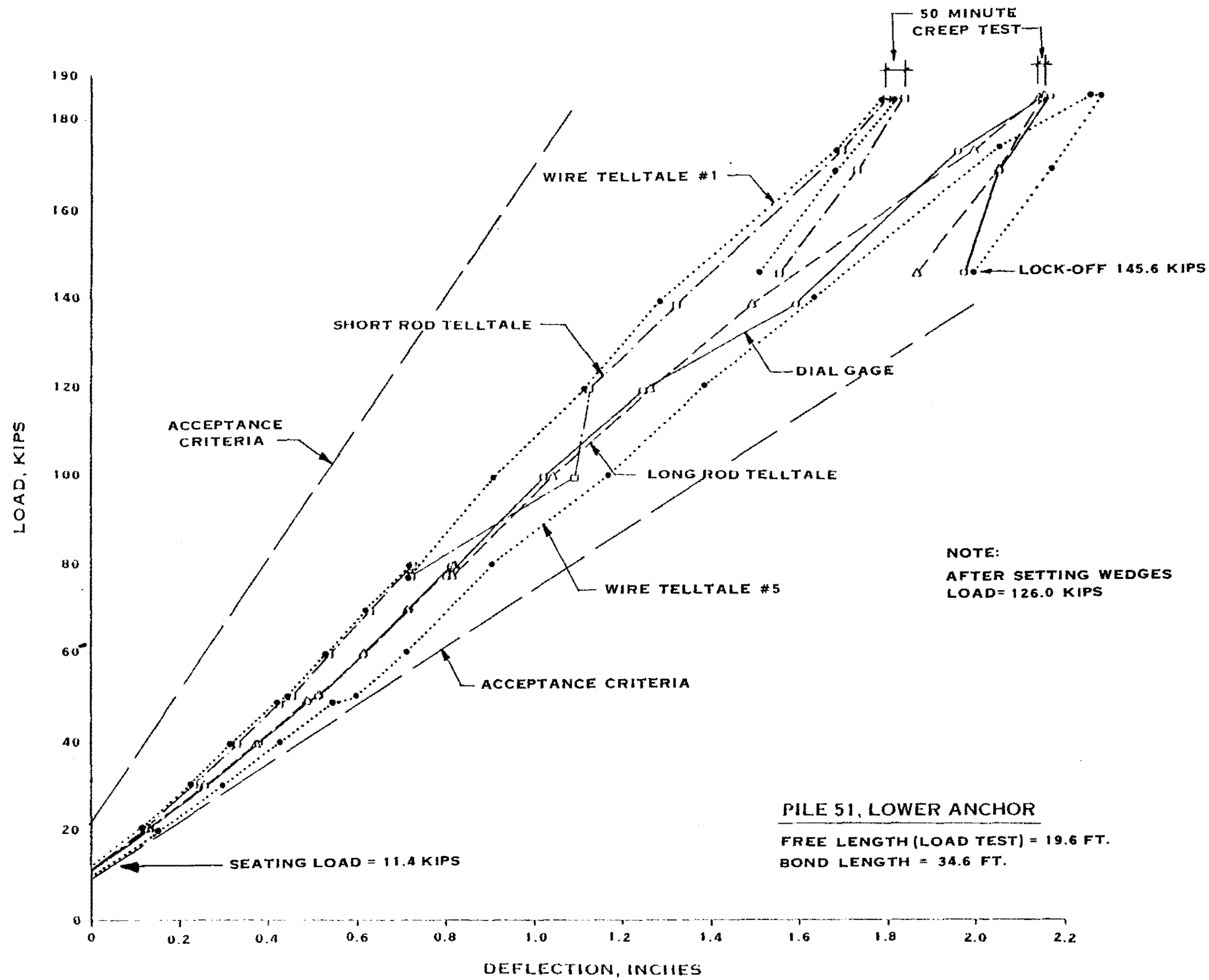


FIGURE 7 - LOAD DISPLACEMENT CURVE - LOAD TEST

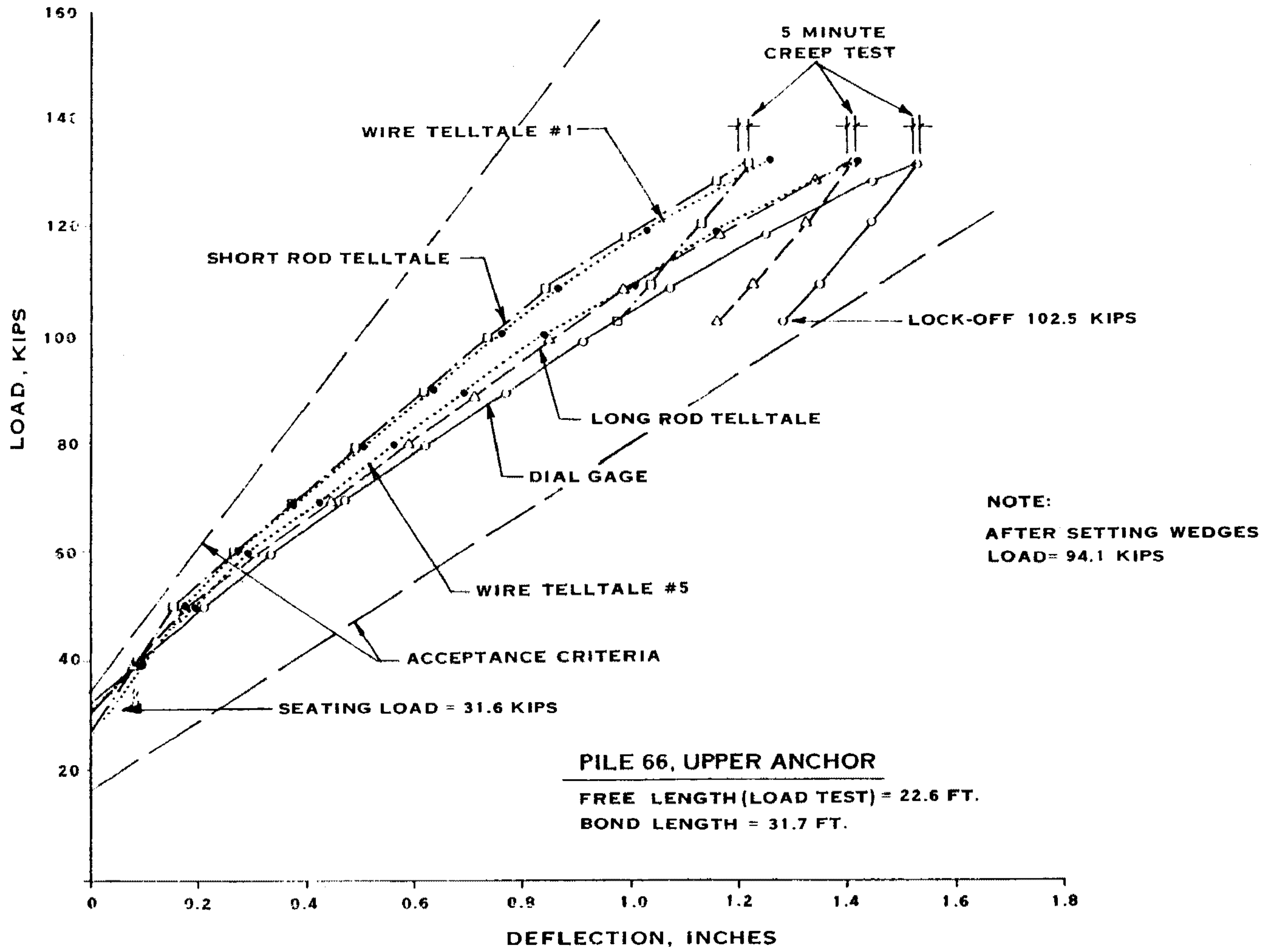


FIGURE 8 - LOAD DISPLACEMENT CURVE - LOAD TEST

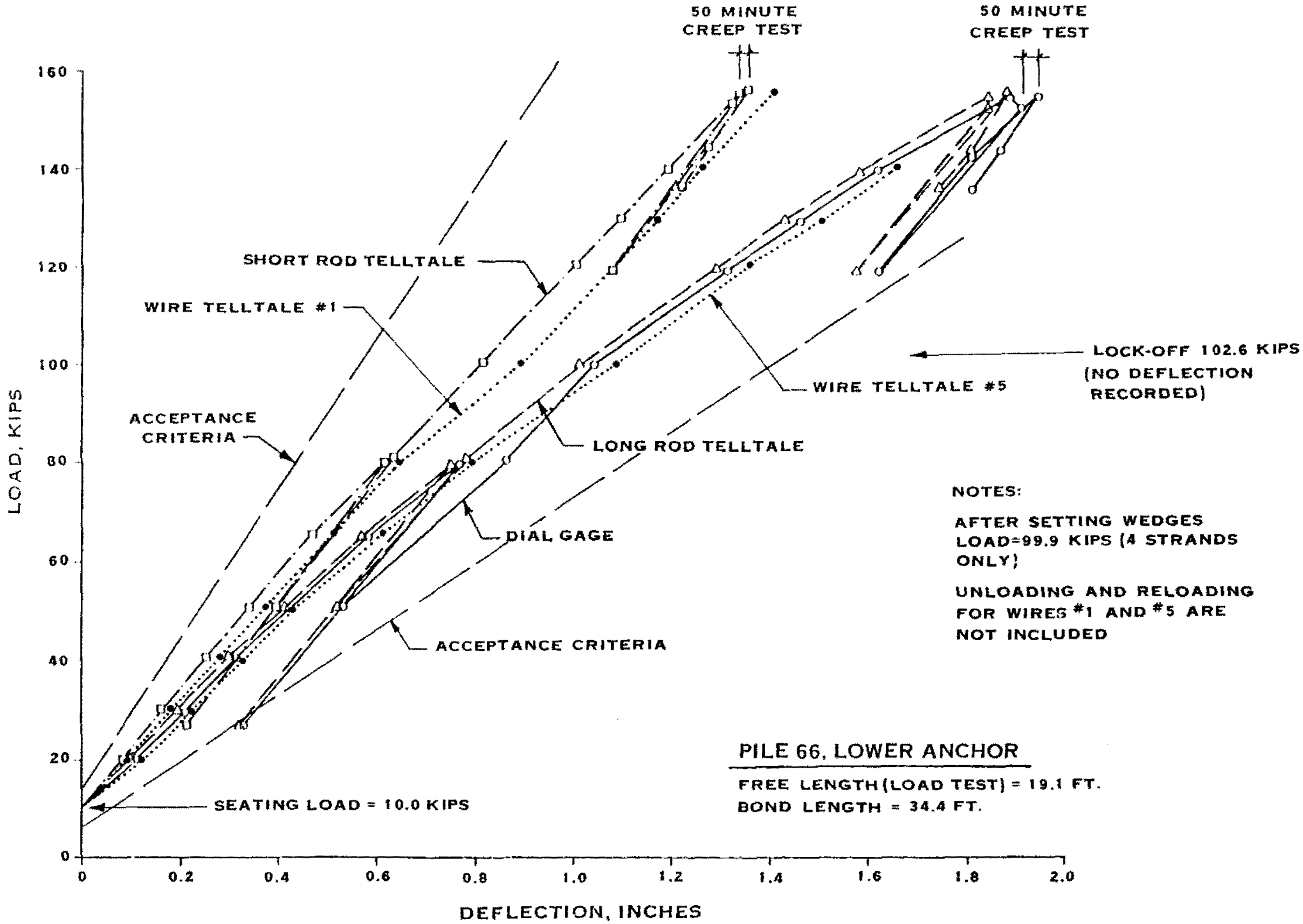


FIGURE 9 - LOAD DISPLACEMENT CURVE - LOAD TEST

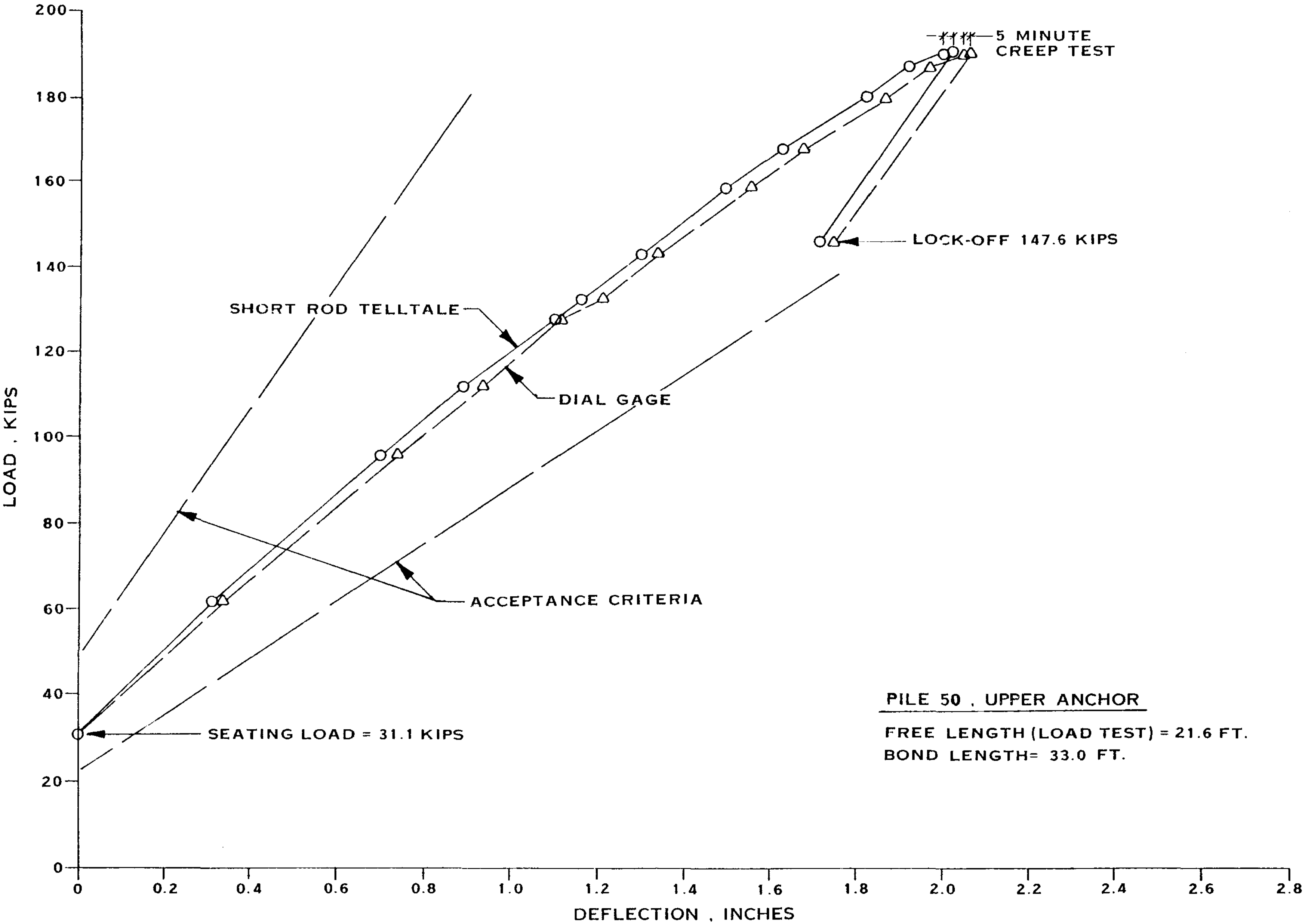


FIGURE 10.-LOAD DISPLACEMENT CURVE - LOAD TEST

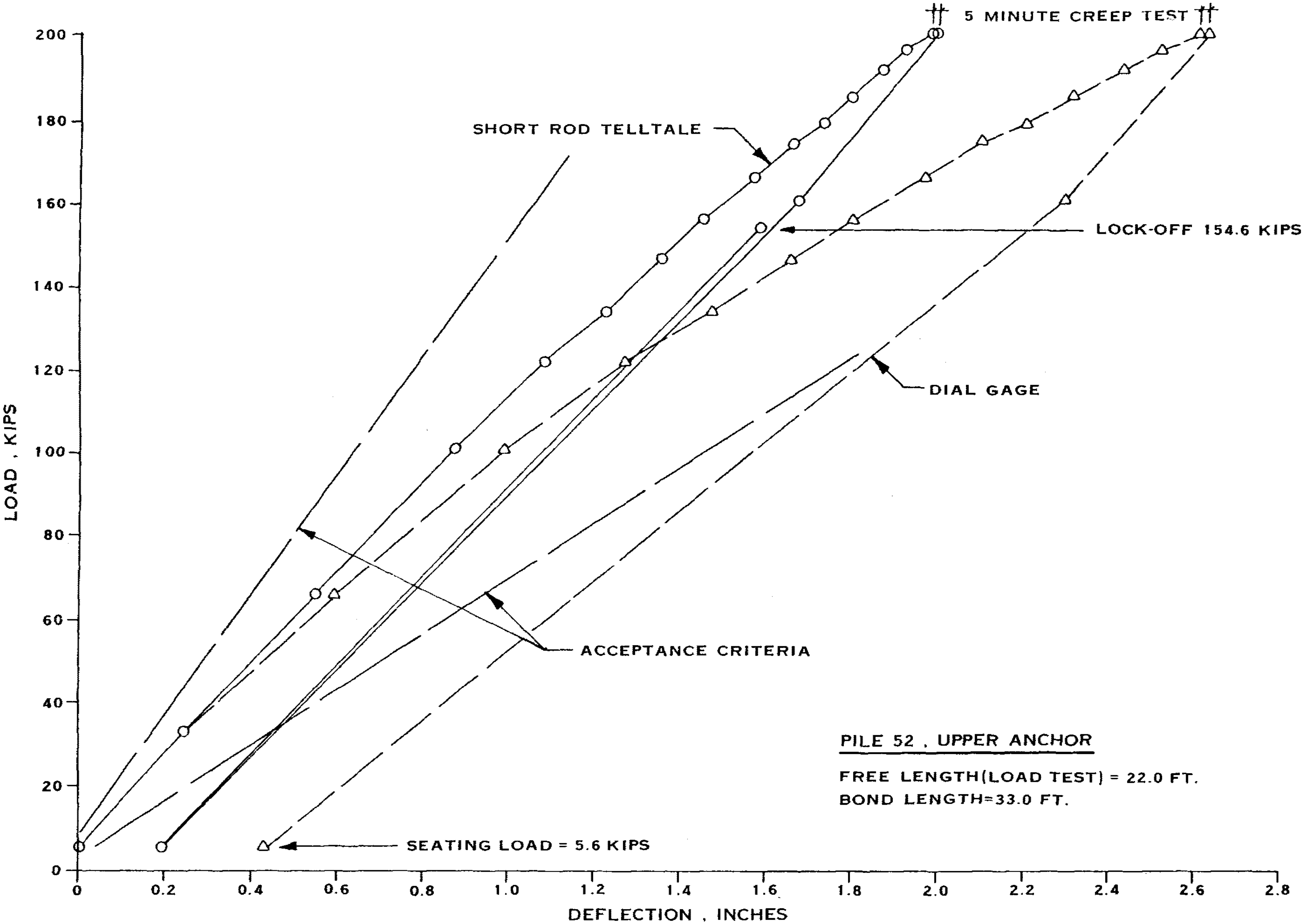


FIGURE 11-LOAD DISPLACEMENT CURVE - LOAD TEST

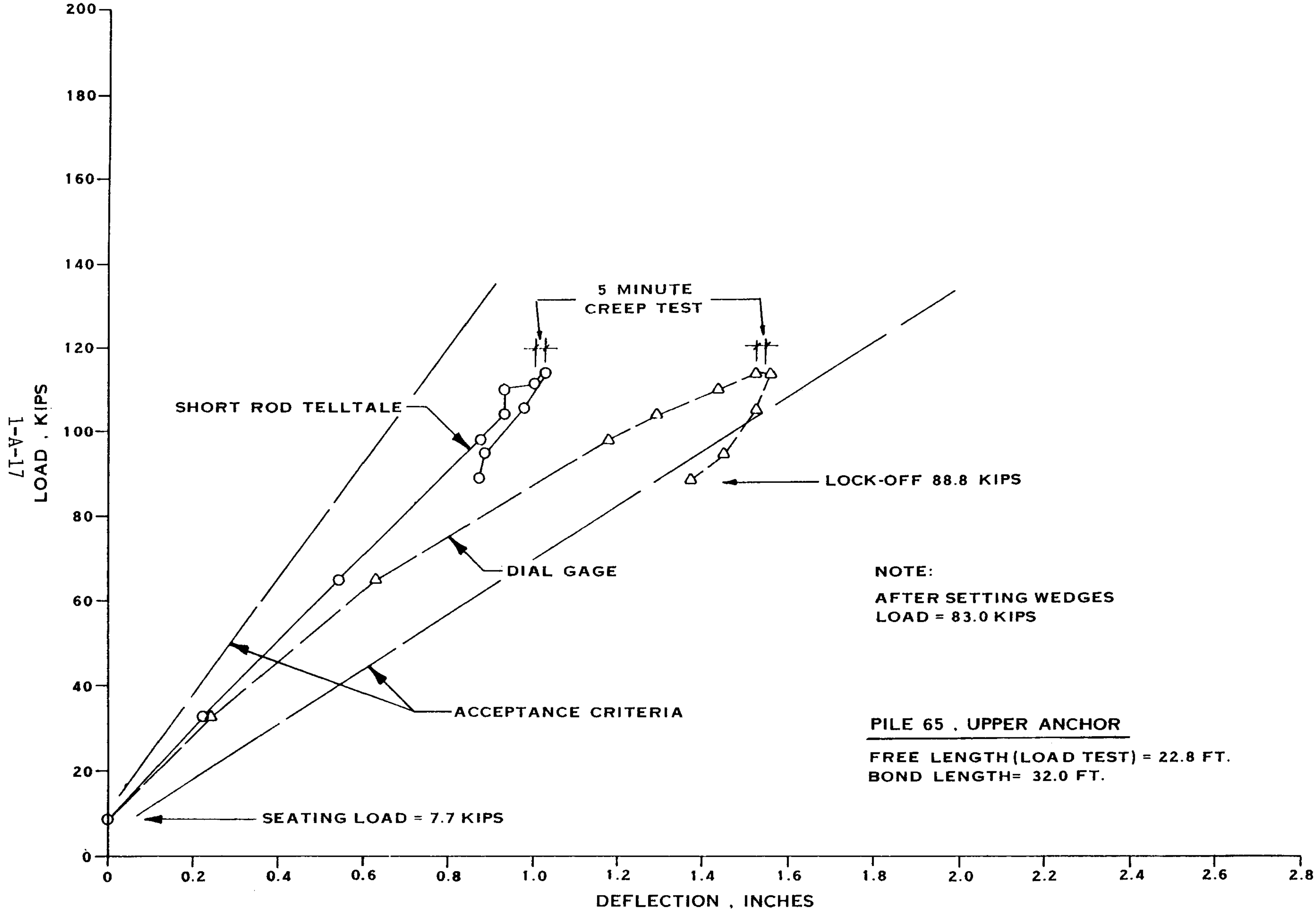


FIGURE 12-LOAD DISPLACEMENT CURVE - LOAD TEST

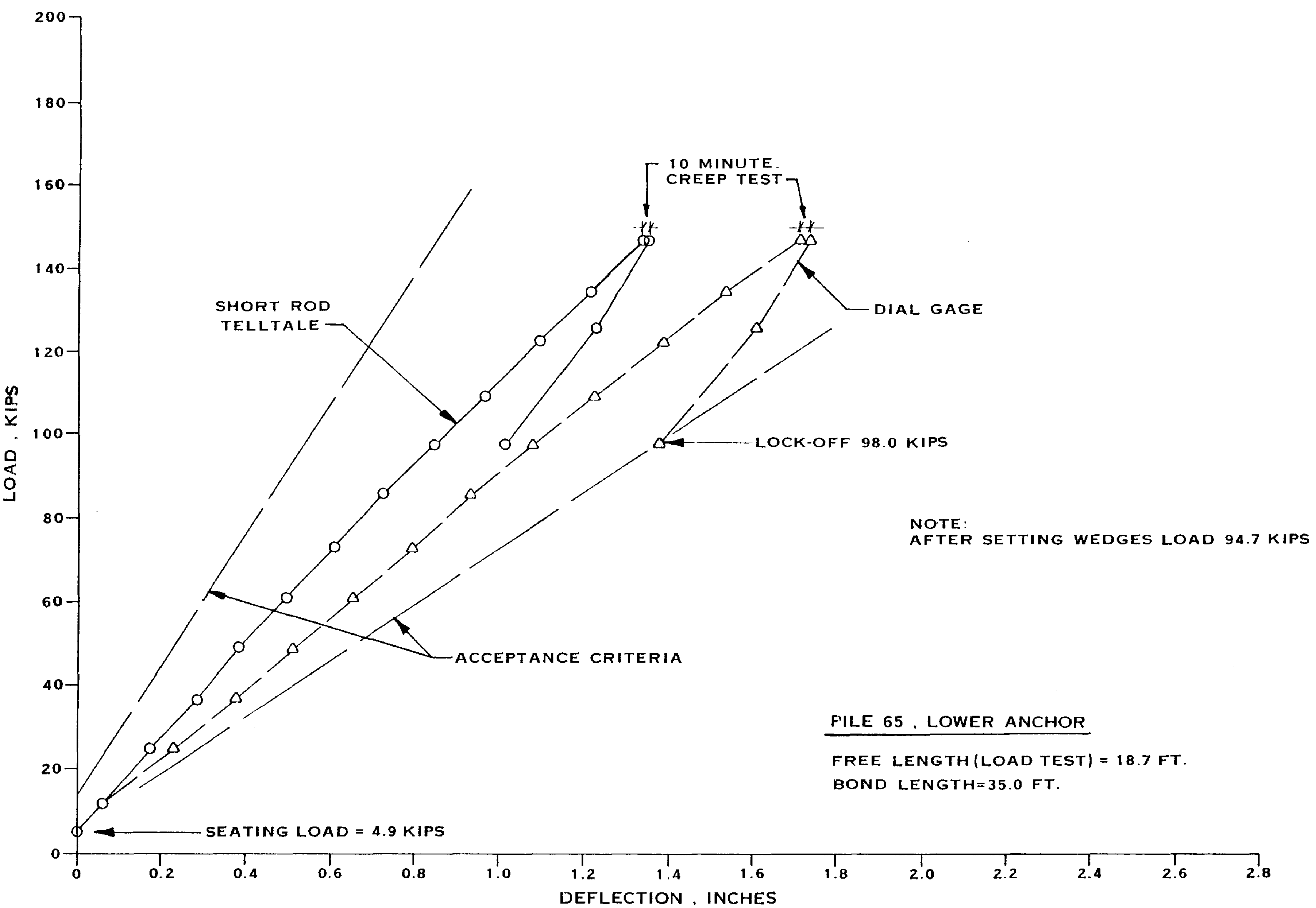
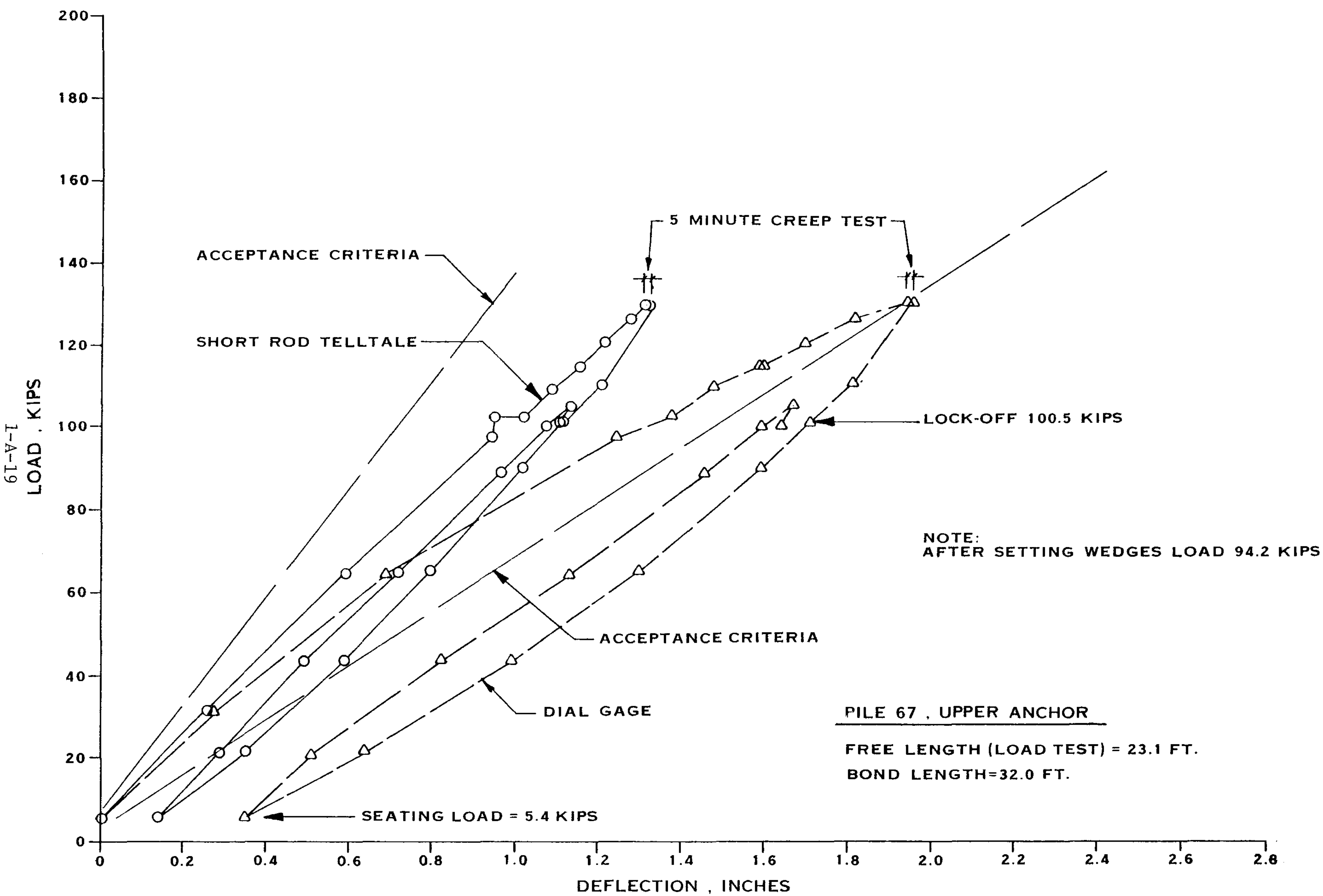


FIGURE 13 - LOAD DISPLACEMENT CURVE - LOAD TEST



1-A-19

FIGURE 14 - LOAD DISPLACEMENT CURVE - LOAD TEST

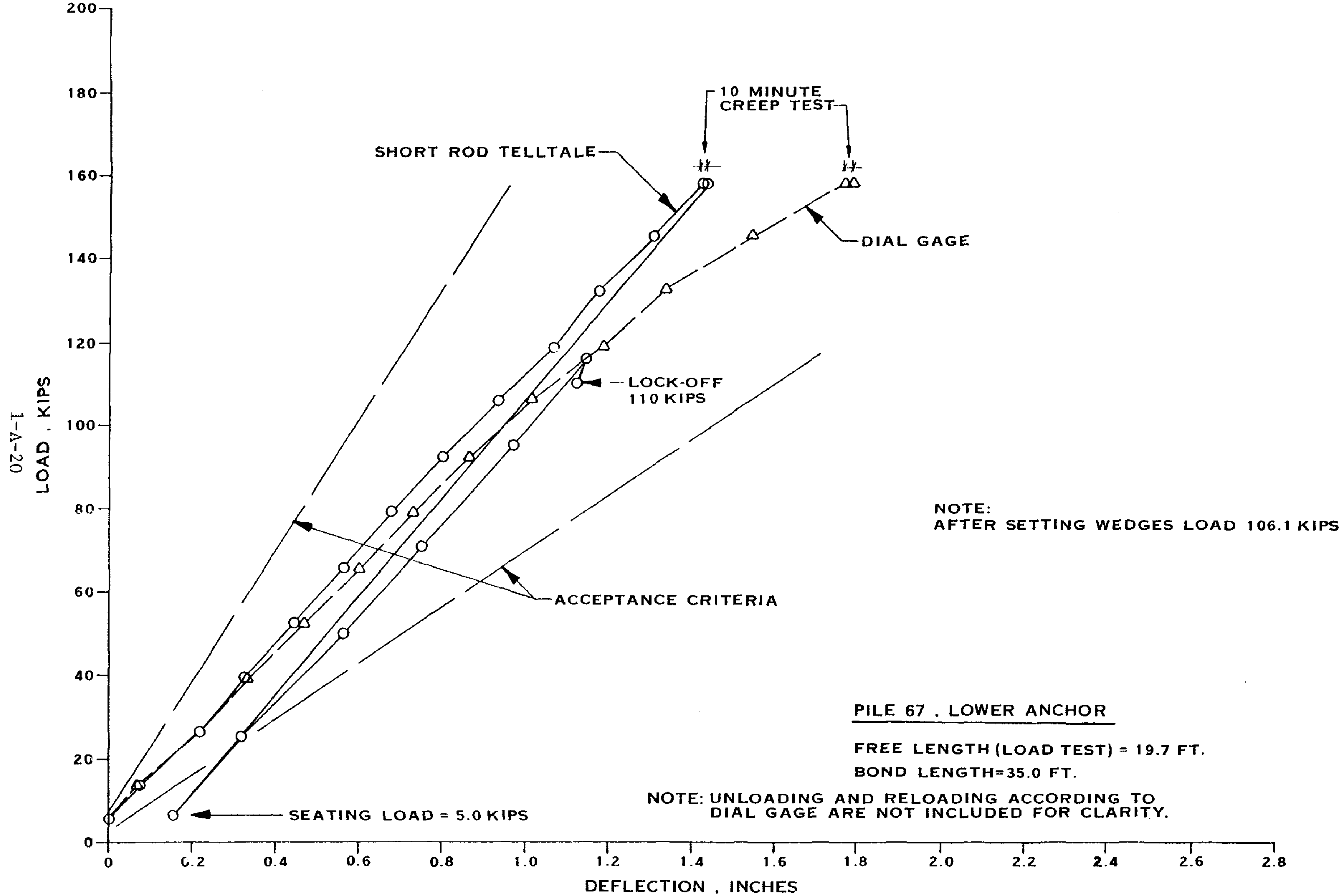
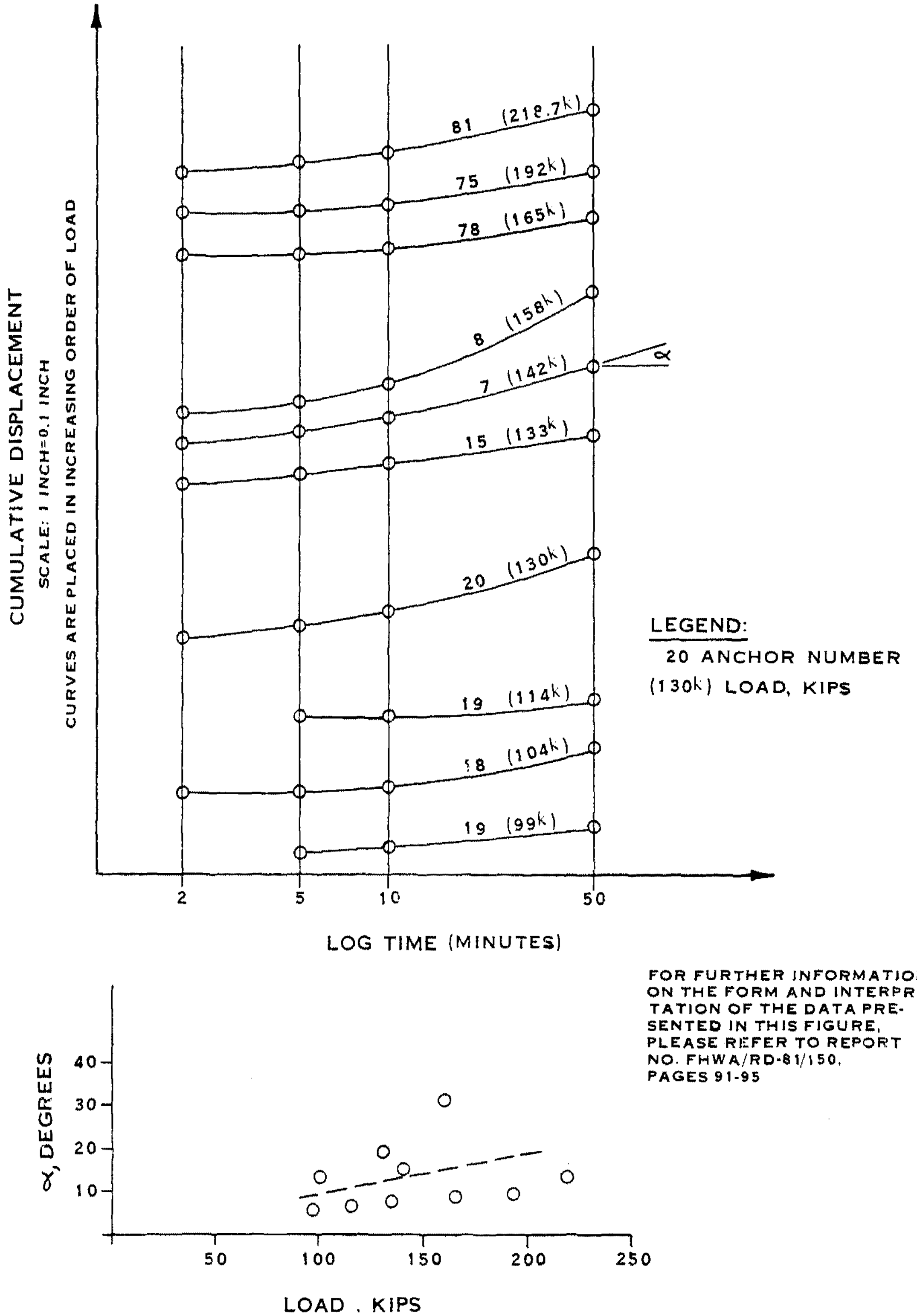


FIGURE 15 - LOAD DISPLACEMENT CURVE - LOAD TEST



FOR FURTHER INFORMATION ON THE FORM AND INTERPRETATION OF THE DATA PRESENTED IN THIS FIGURE, PLEASE REFER TO REPORT NO. FHWA/RD-81/150, PAGES 91-95

FIGURE 16- CREEP CURVES FROM PERFORMANCE TESTS

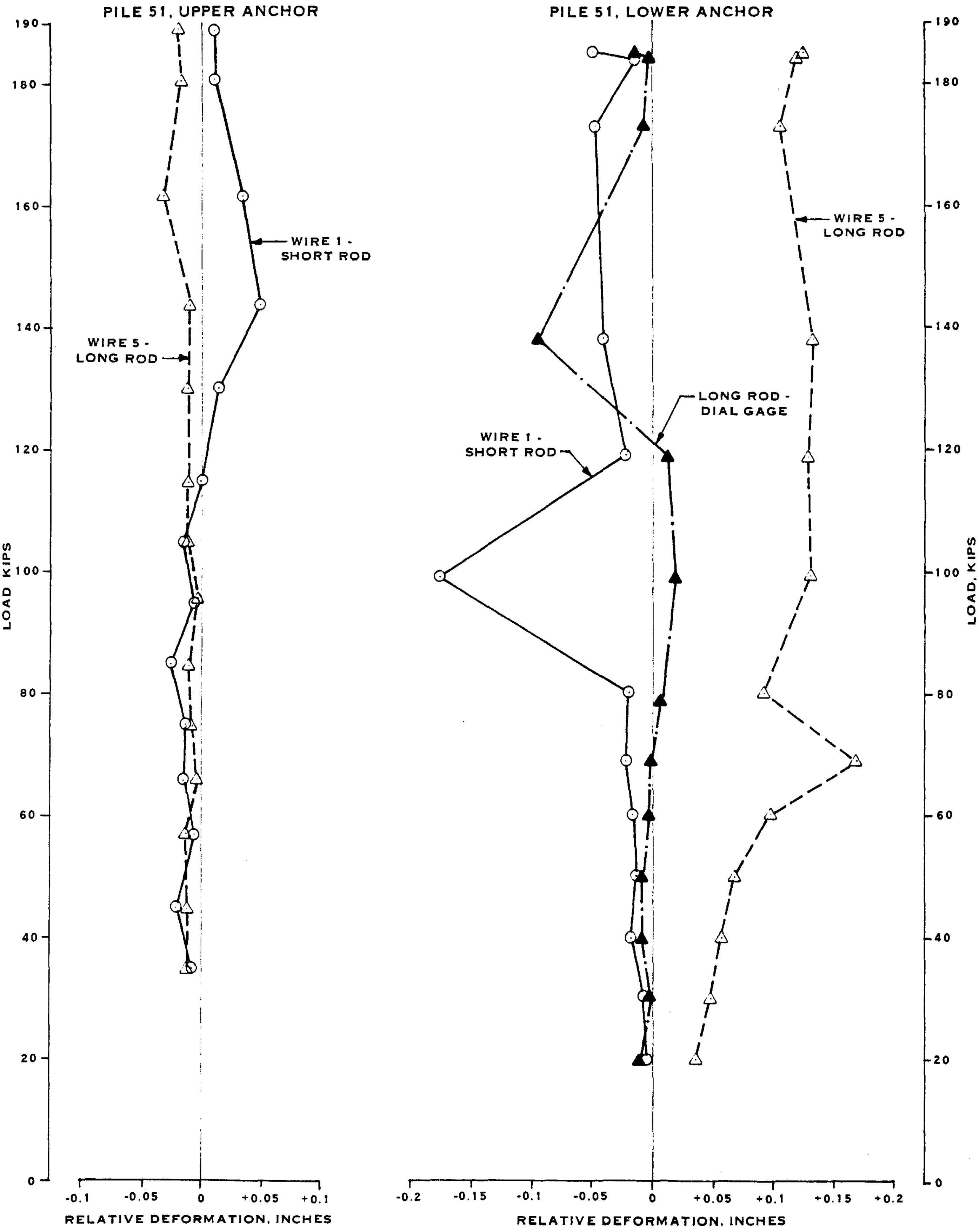


FIGURE 17- COMPARISON OF WIRE TELLTALE VERSUS ROD TELLTALE DURING LOAD TEST

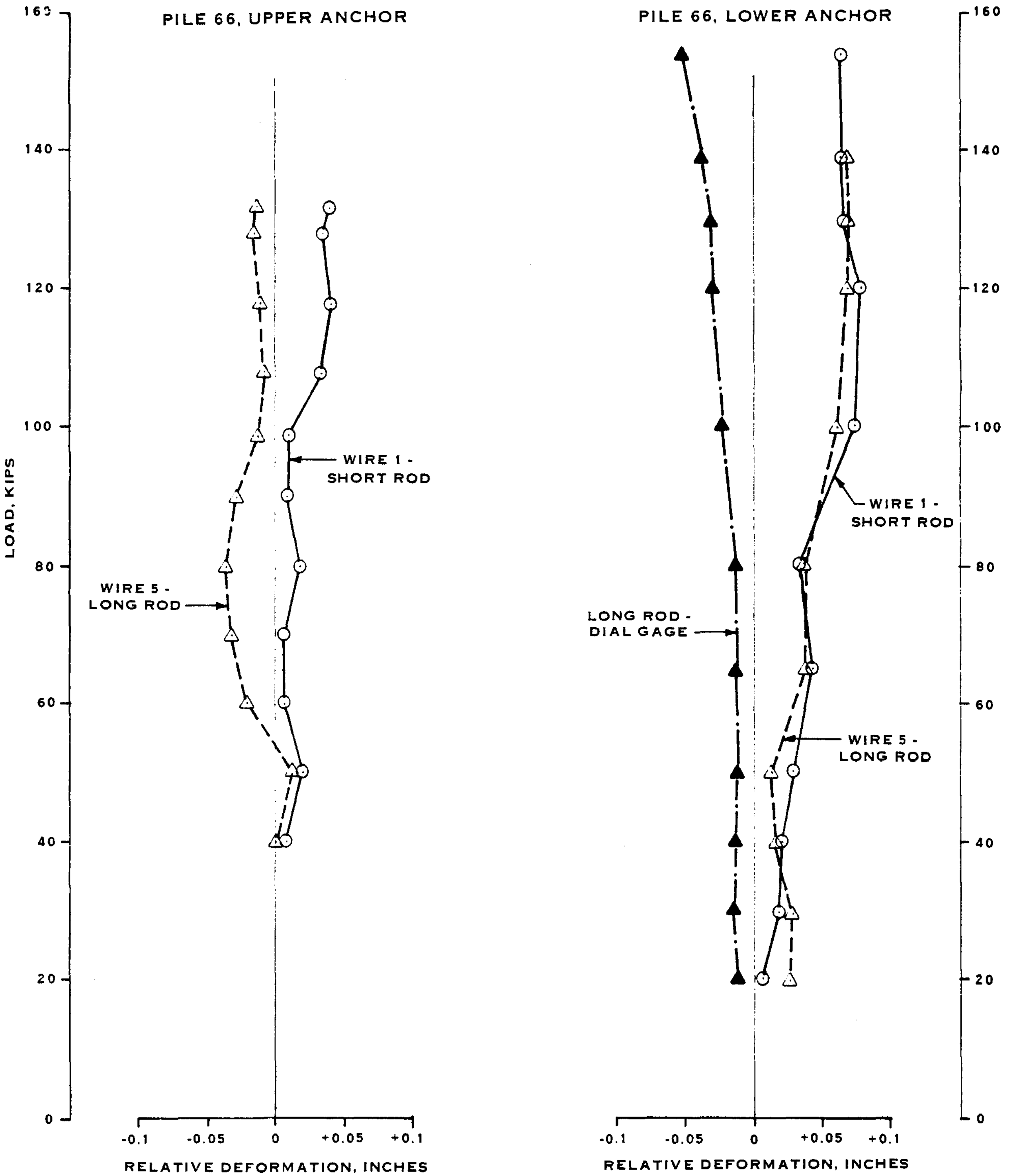


FIGURE 18-COMPARISON OF WIRE TELLTALE VERSUS ROD TELLTALE DURING LOAD TEST

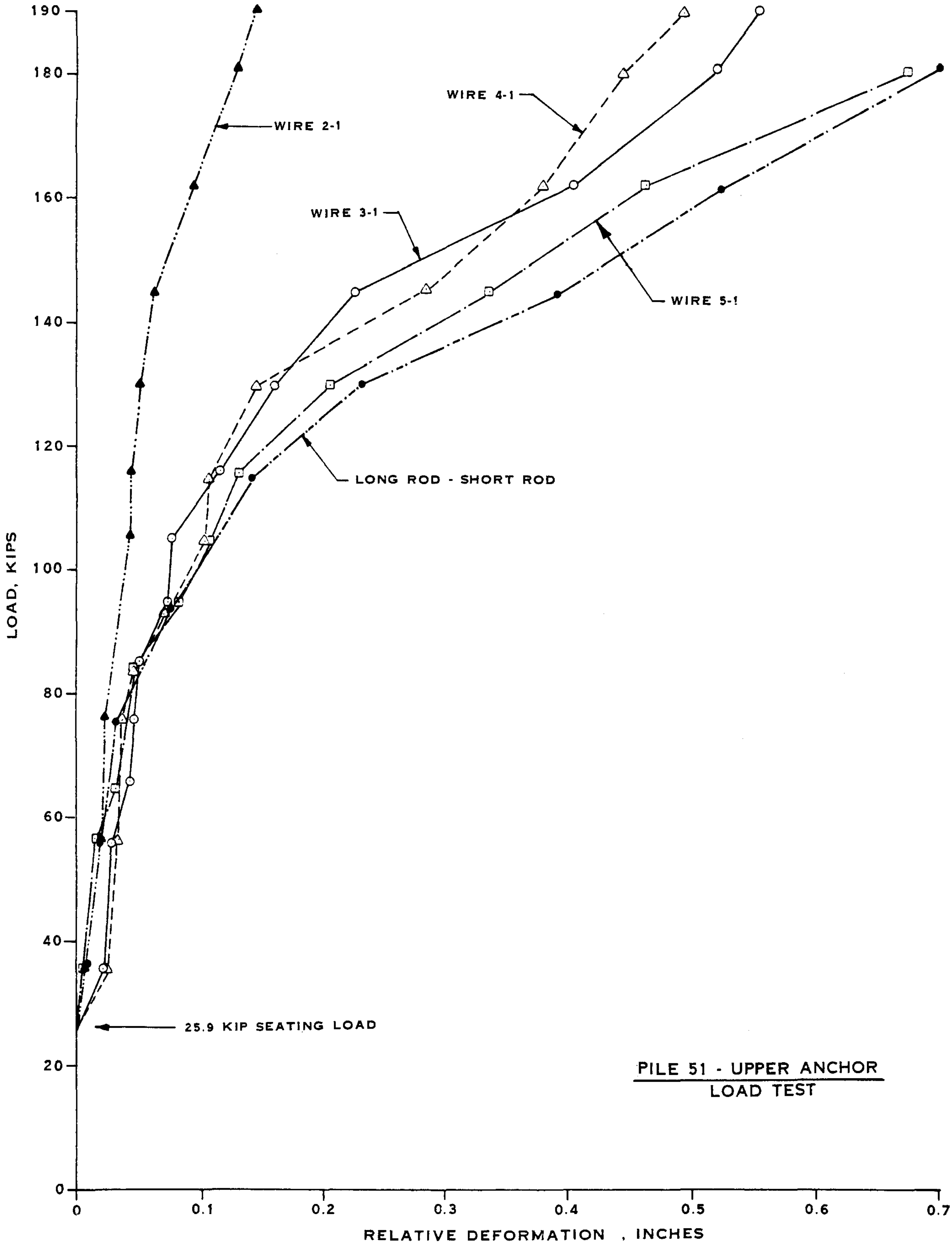


FIGURE 19 - RELATIVE DEFORMATION IN BOND ZONE - LOAD TEST

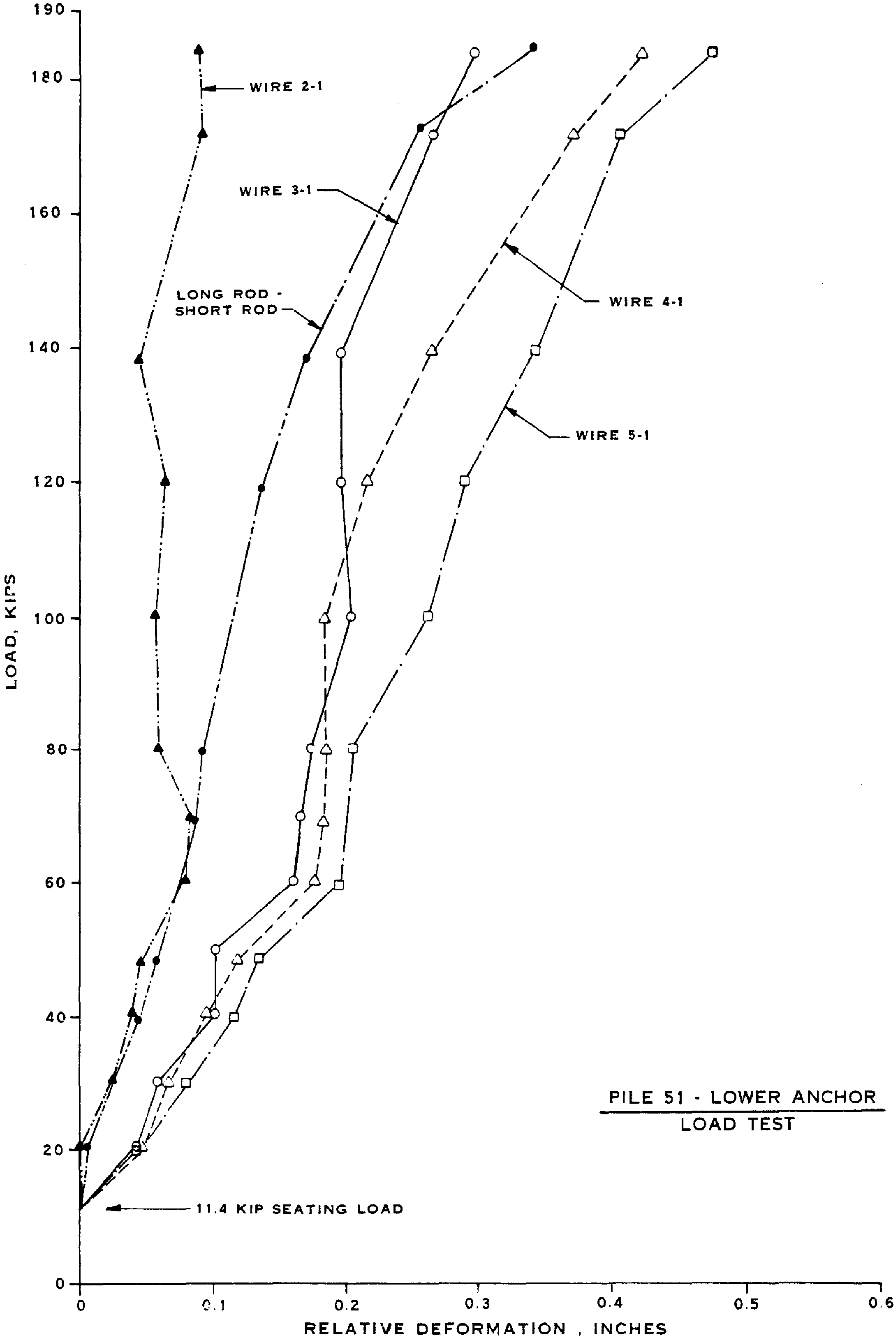


FIGURE 20 - RELATIVE DEFORMATION IN BOND ZONE - LOAD TEST

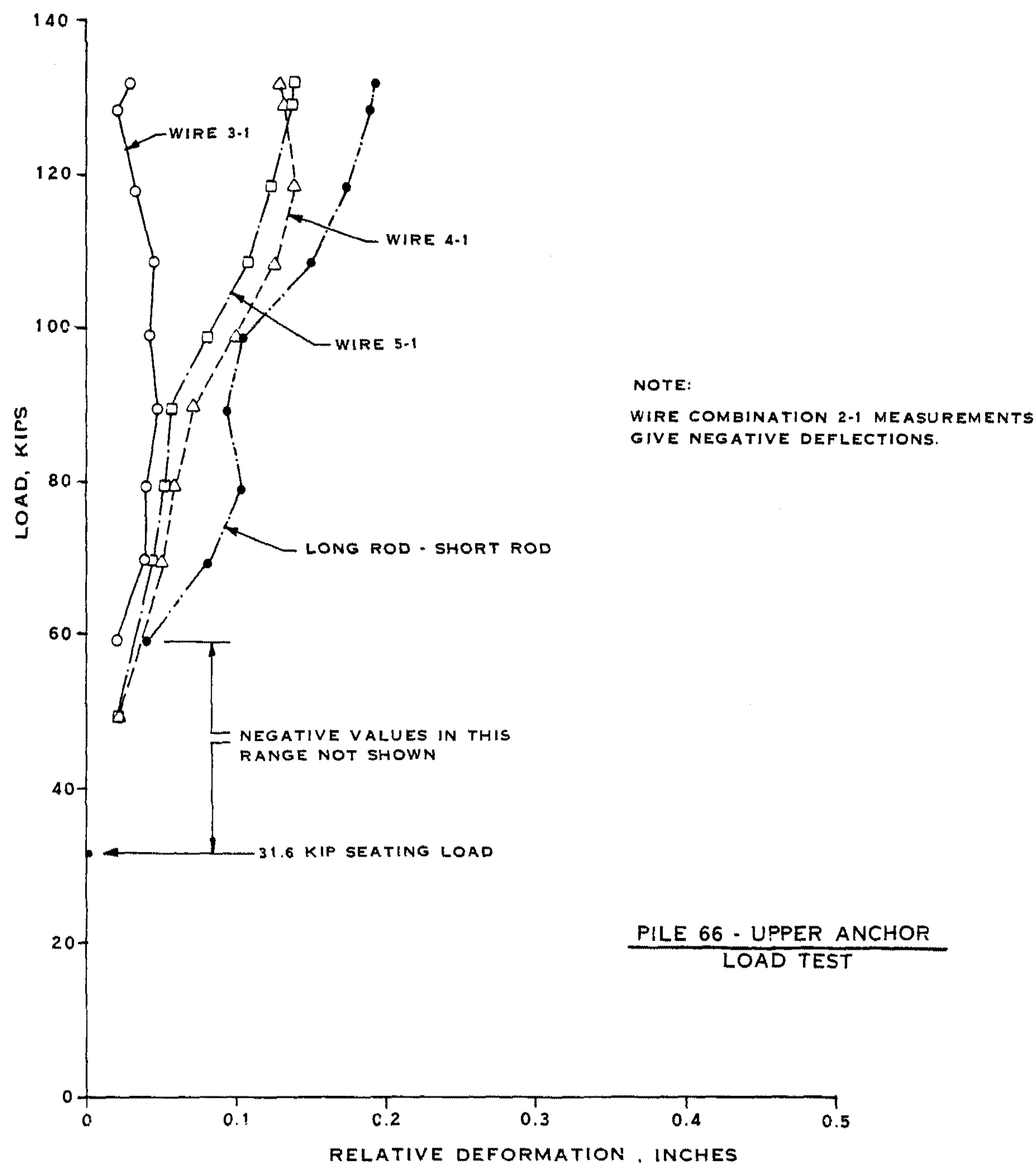


FIGURE 21 - RELATIVE DEFORMATION IN BOND ZONE - LOAD TEST

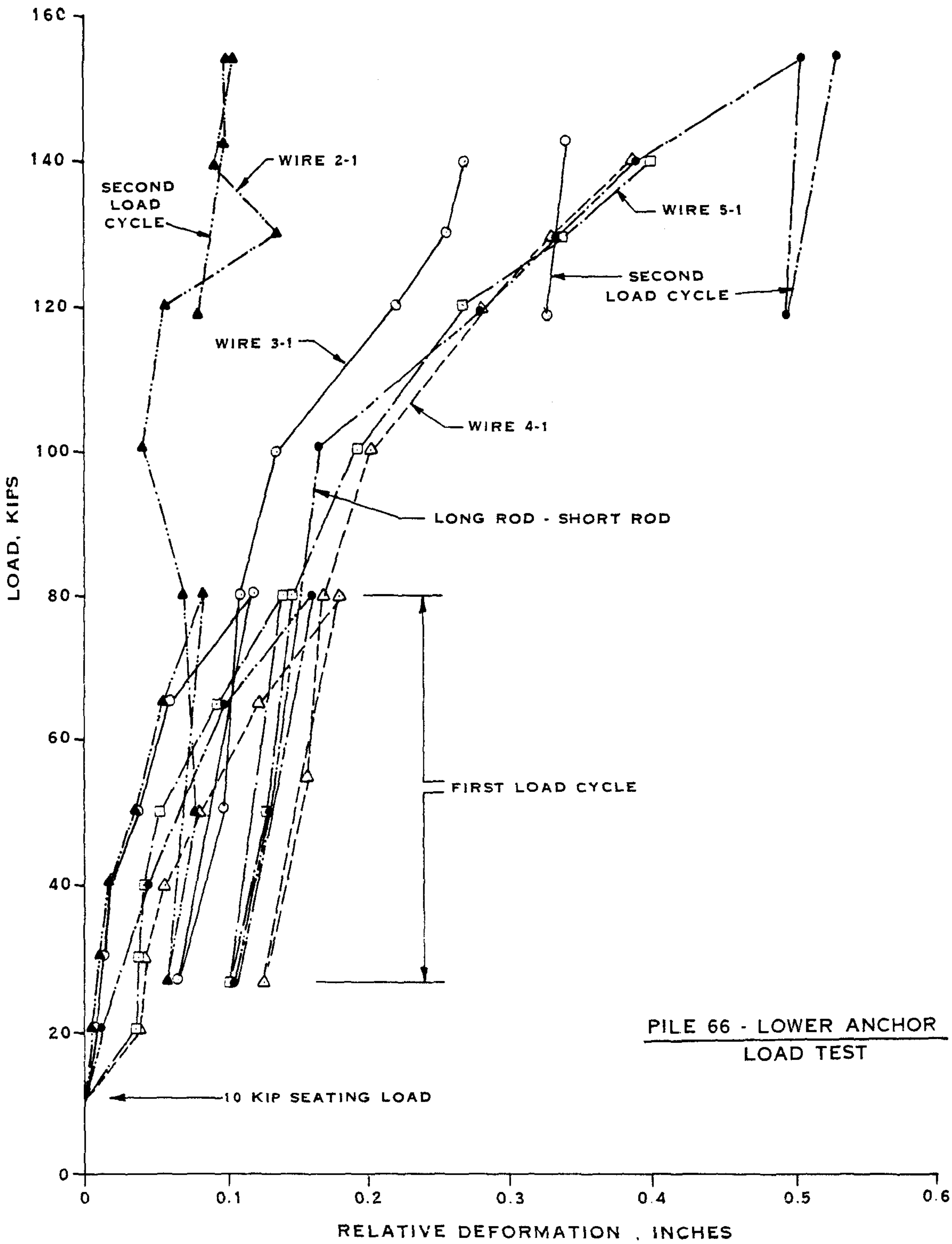
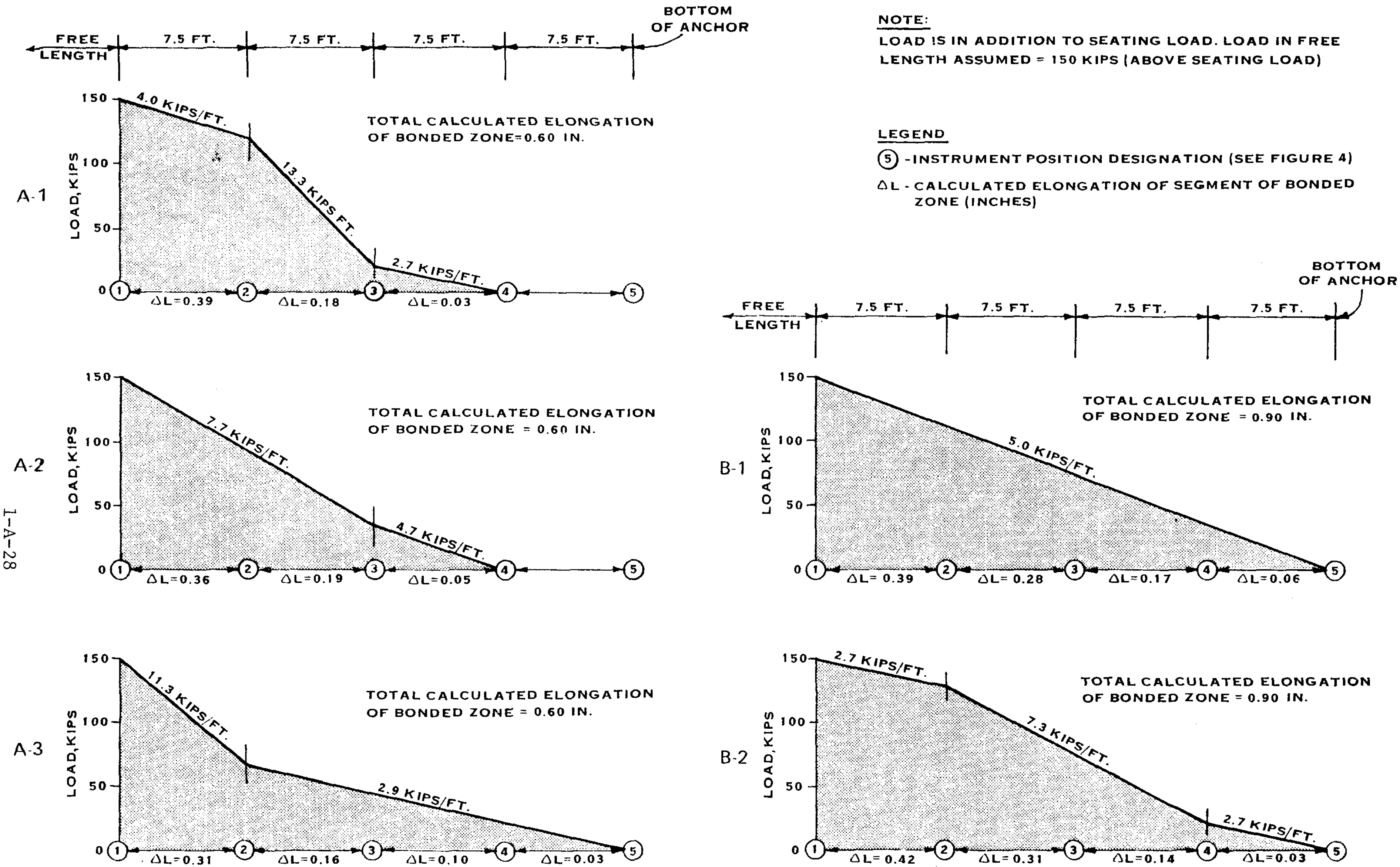


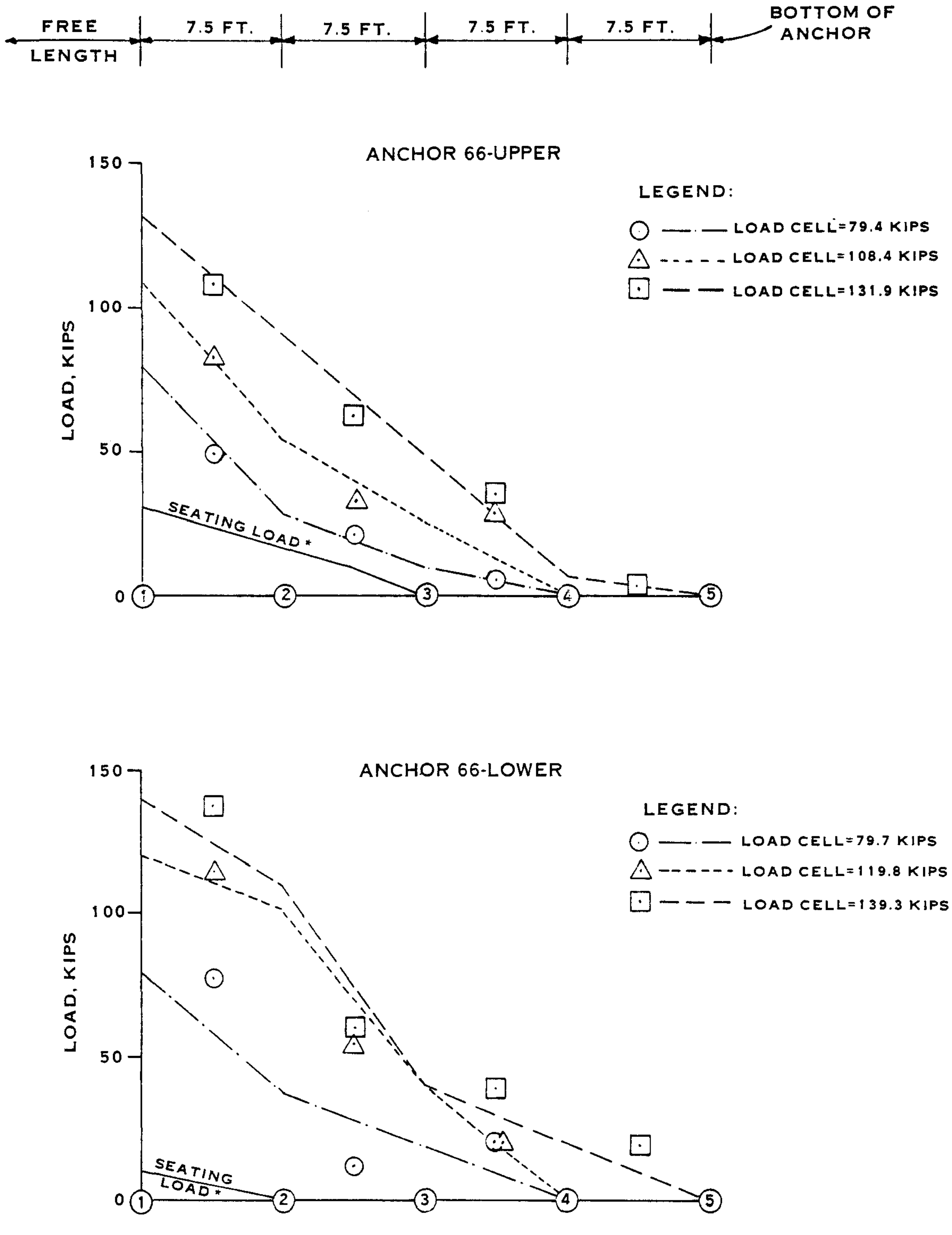
FIGURE 22 - RELATIVE DEFORMATION IN BOND ZONE - LOAD TEST



A. HYPOTHETICAL LOAD TRANSFER DIAGRAMS WITH CALCULATED TOTAL BONDED ZONE ELONGATION = 0.60 INCH

B. HYPOTHETICAL LOAD TRANSFER DIAGRAMS WITH CALCULATED TOTAL BONDED ZONE ELONGATION = 0.90 INCH

FIGURE 23 - HYPOTHETICAL LOAD TRANSFER DIAGRAMS AND CALCULATED TOTAL ELONGATION OF BONDED ZONE



NOTE: POINTS REPRSENT CALCULATED AVERAGE LOAD WITHIN 7.5 FT. SEGMENTS BASED ON RELATIVE DEFLECTIONS AS MEASURED BY WIRE TELLTALES AND ADJUSTED FOR ASSUMED SLIPPAGE OF WIRE #1

CURVES REPRESENT LOAD DISTRIBUTIONS FOR WHICH CALCULATED BONDED ZONE ELONGATION IS EQUAL TO THE MEASURED ELONGATION ADJUSTED FOR ASSUMED SLIPPAGE OF WIRE #1

*DISTRIBUTION AT SEATING LOAD IS ASSUMED

FIGURE 24 - LOAD DISTRIBUTION IN BONDED ZONE (LOAD TEST) ASSUMING SLIPPAGE OF WIRE #1 AND SHORT ROD TELLTALE ANCHORS

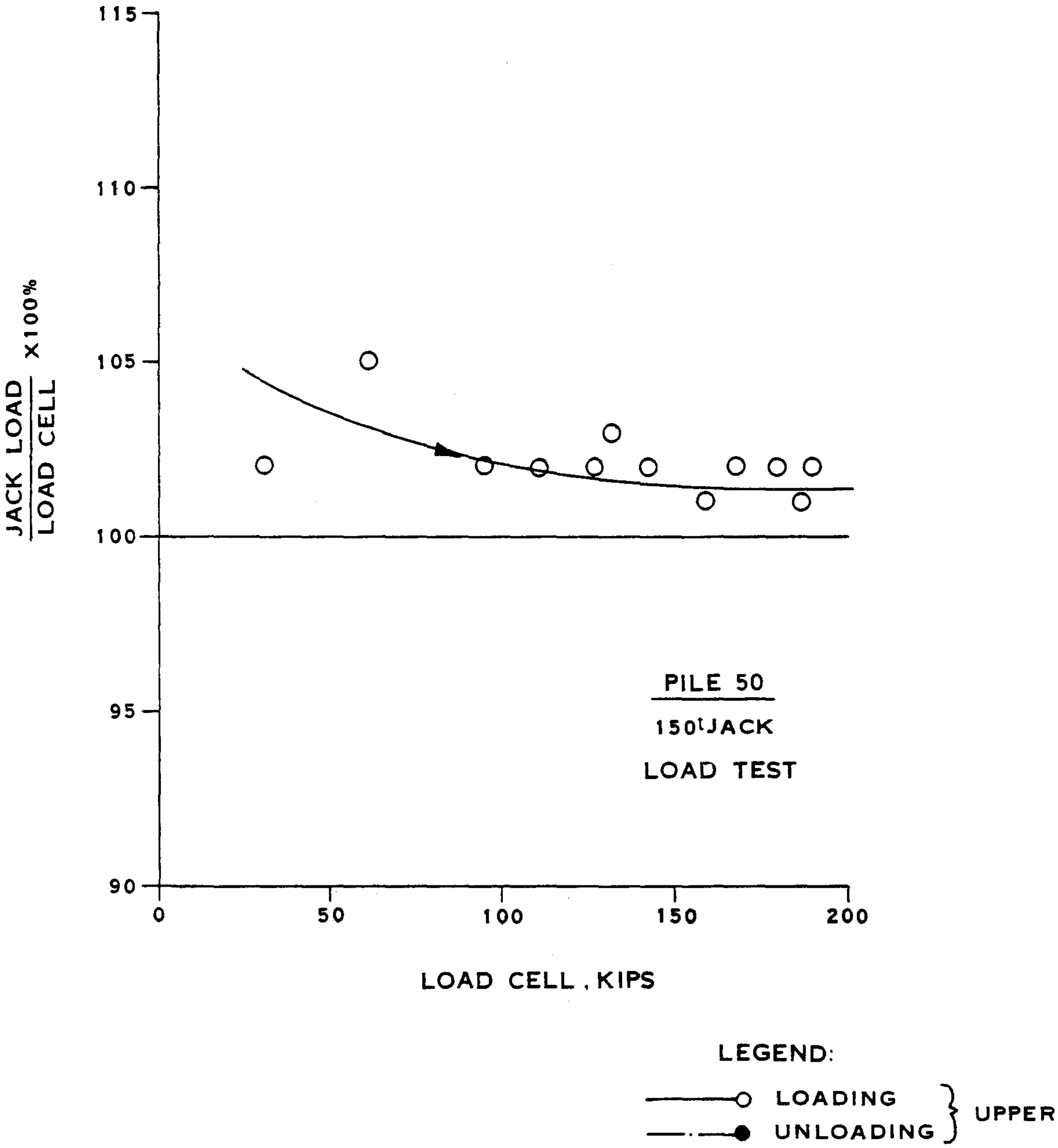


FIGURE 25
COMPARISON OF LOAD CELL AND HYDRAULIC JACK
DURING LOAD TEST

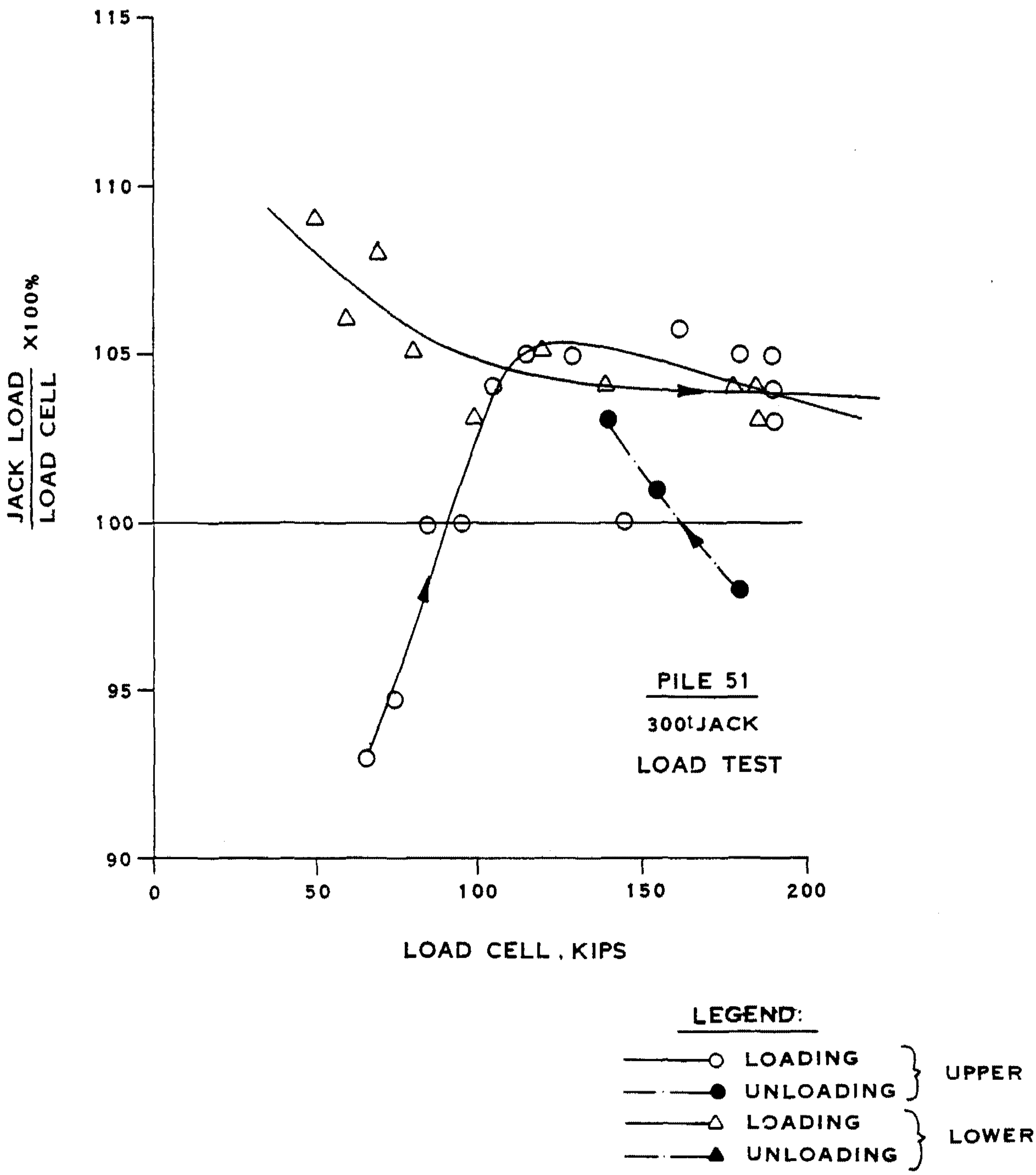


FIGURE 26
COMPARISON OF LOAD CELL AND HYDRAULIC JACK
DURING LOAD TEST

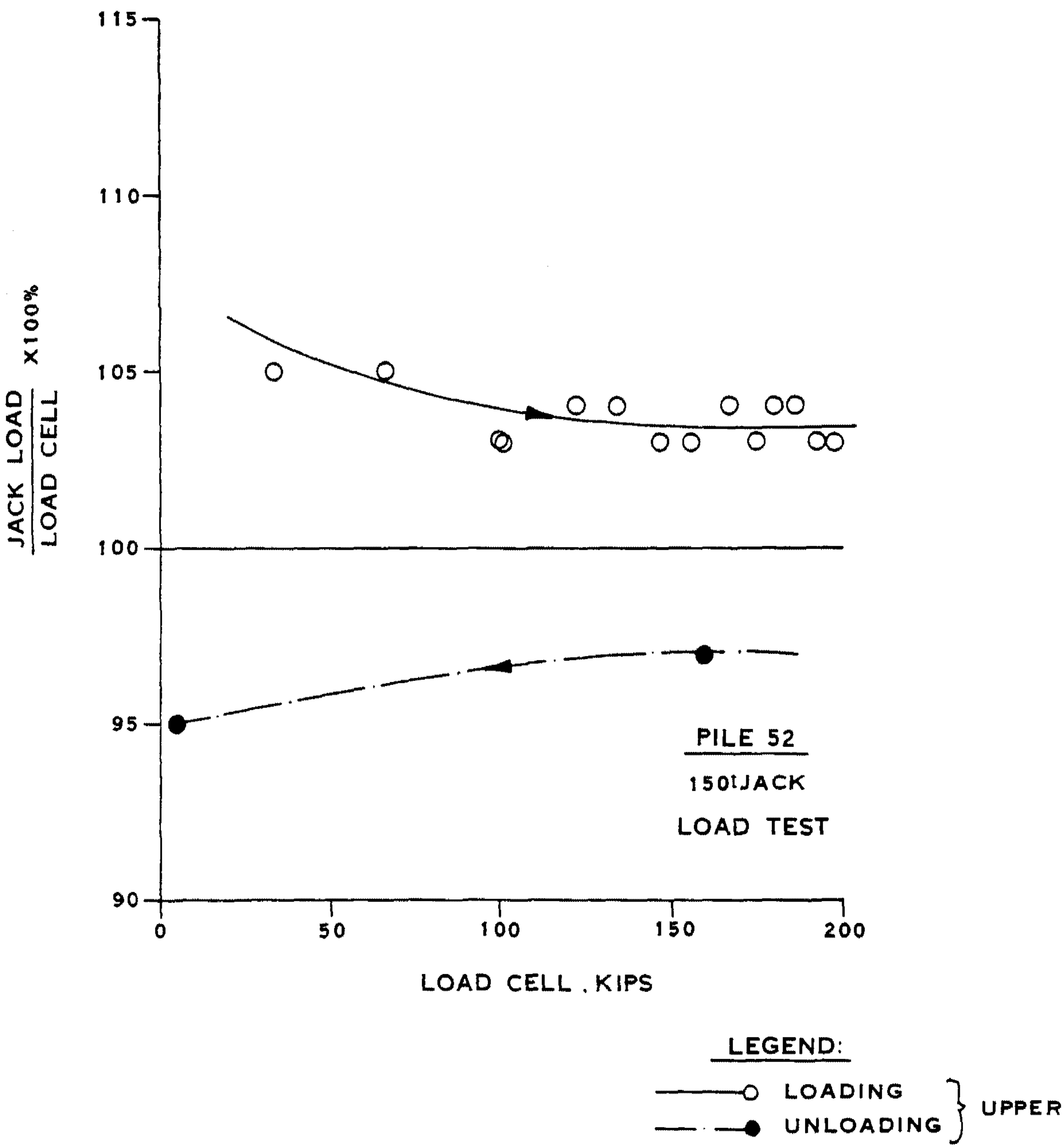


FIGURE 27
COMPARISON OF LOAD CELL AND HYDRAULIC JACK
DURING LOAD TEST

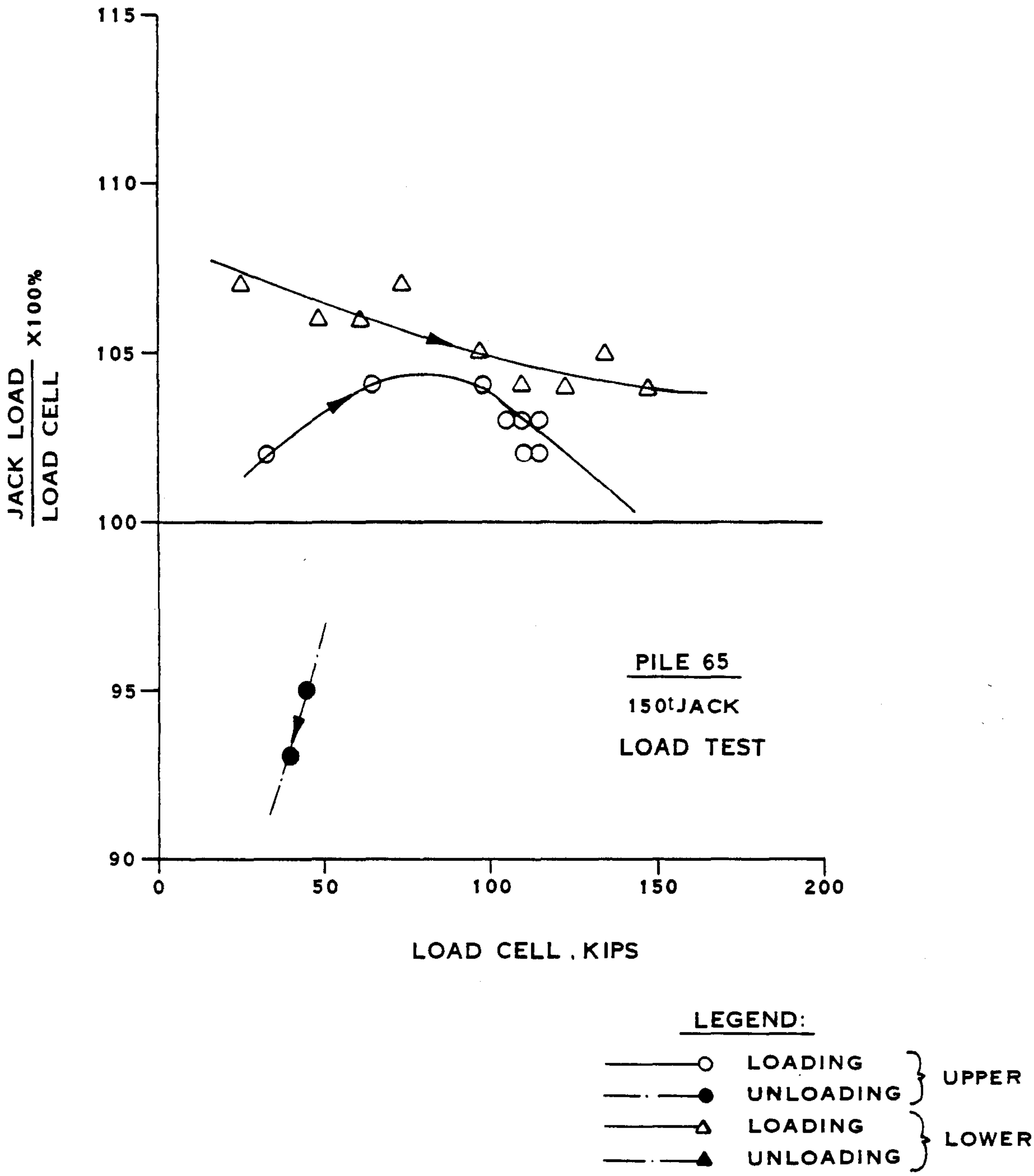


FIGURE 28
COMPARISON OF LOAD CELL AND HYDRAULIC JACK
DURING LOAD TEST

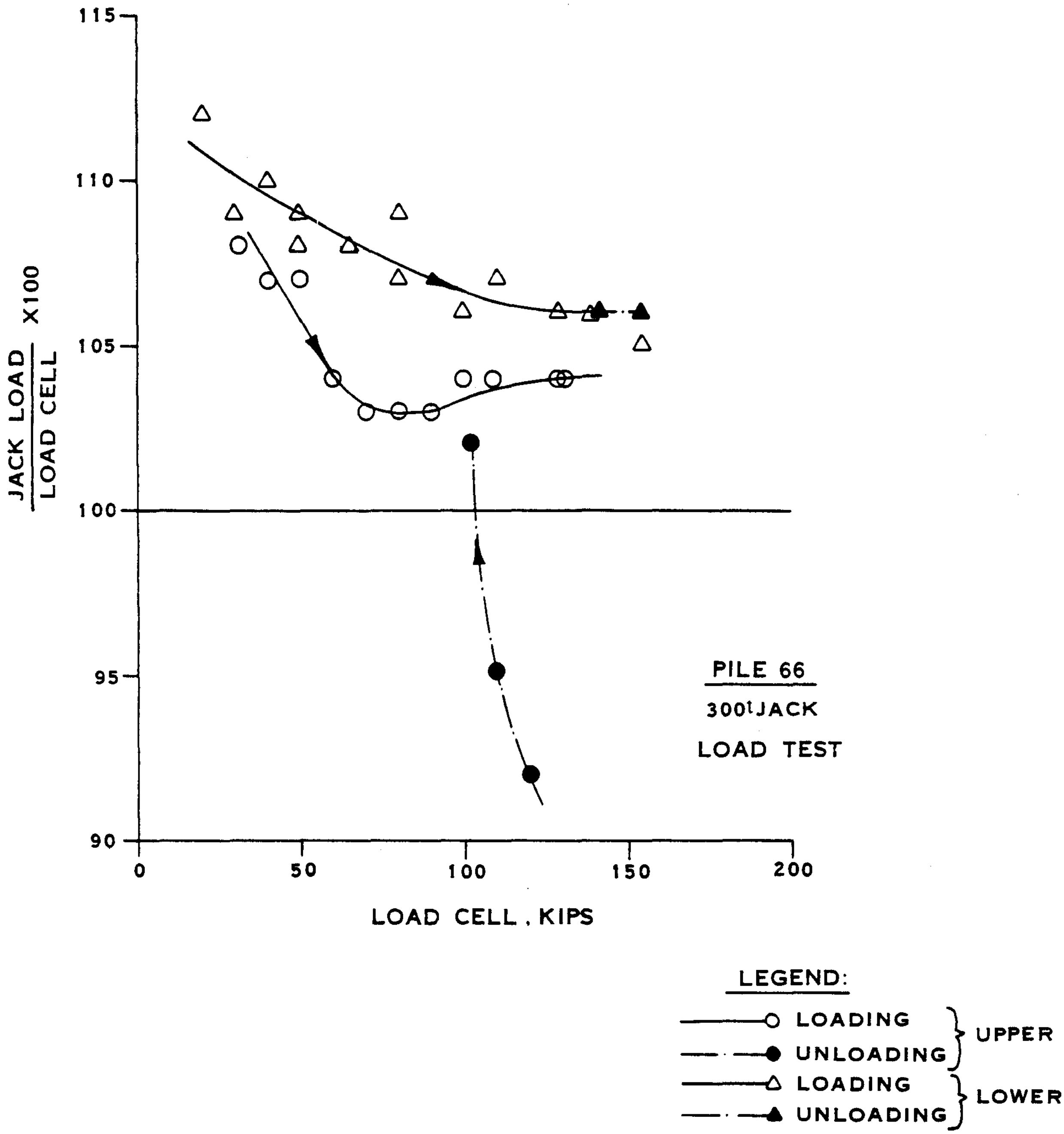


FIGURE 29
COMPARISON OF LOAD CELL AND HYDRAULIC JACK
DURING LOAD TEST

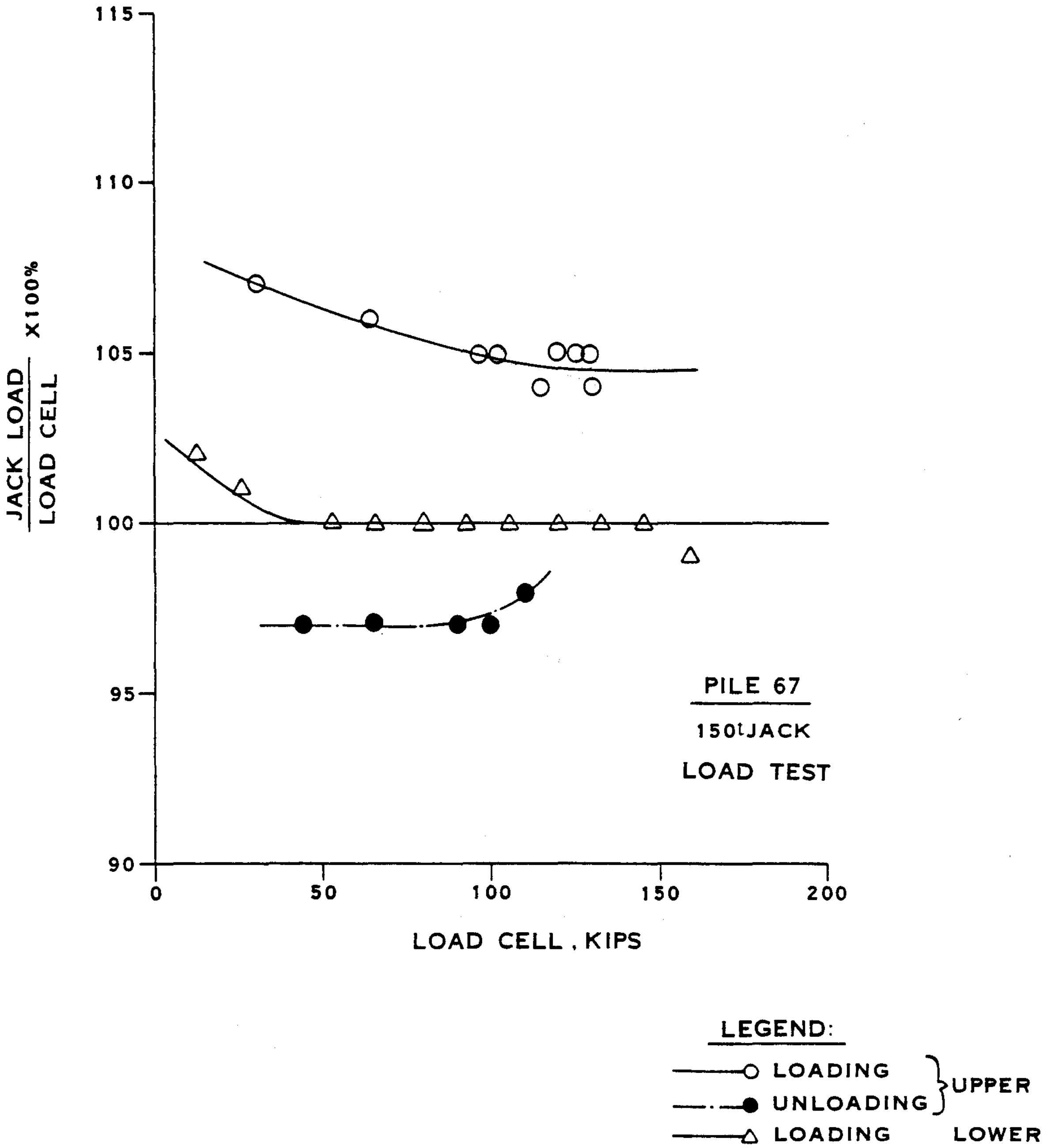
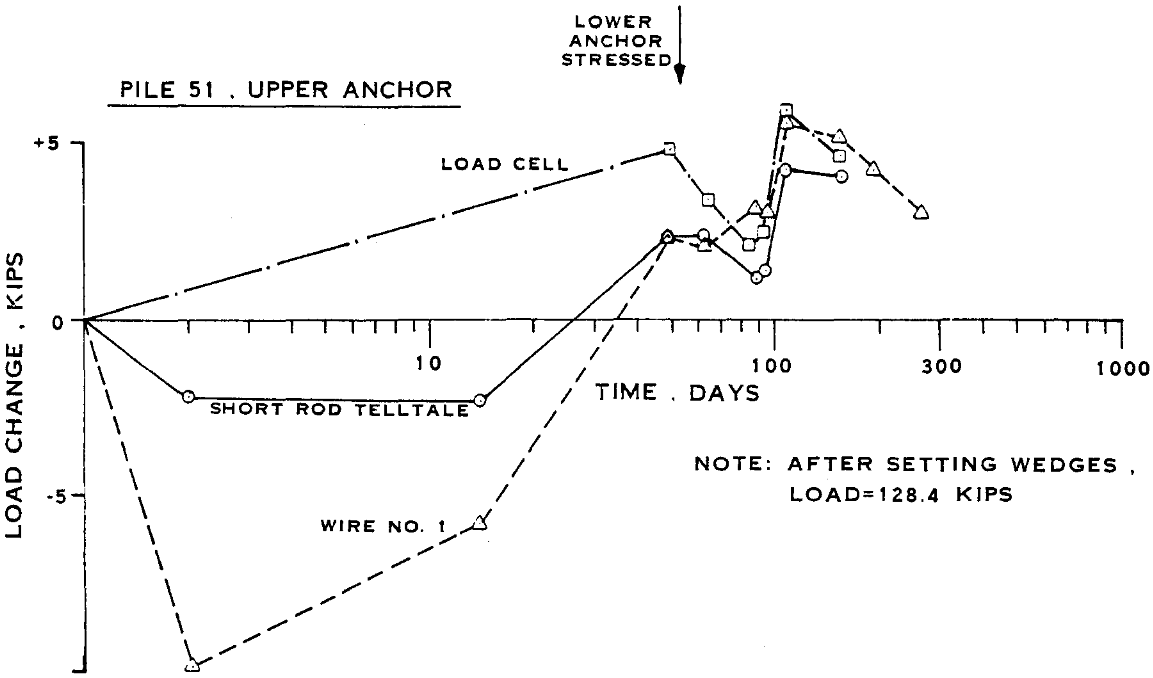


FIGURE 30
COMPARISON OF LOAD CELL AND HYDRAULIC JACK
DURING LOAD TEST



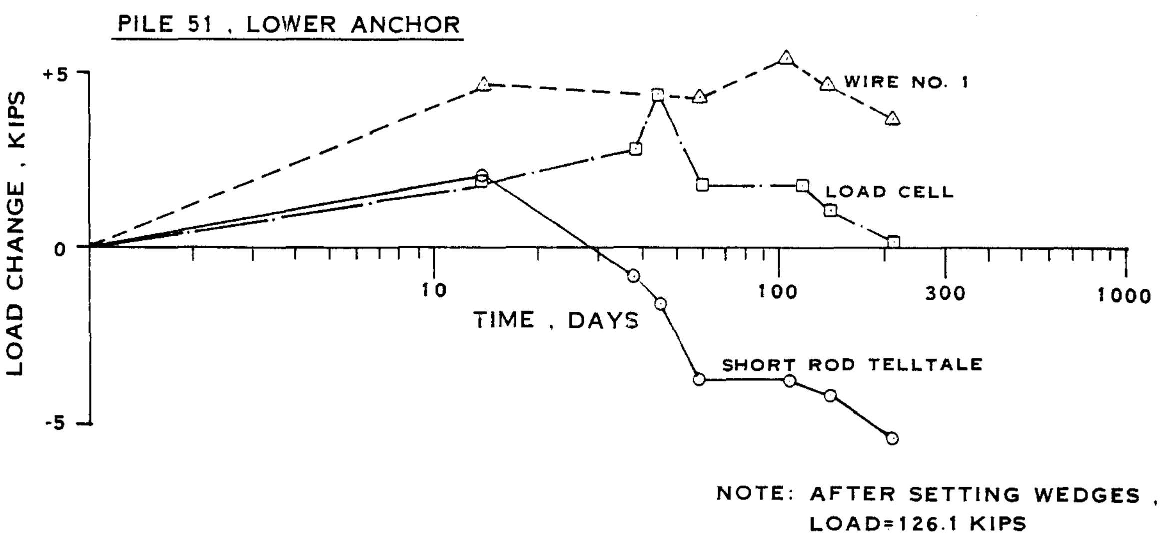
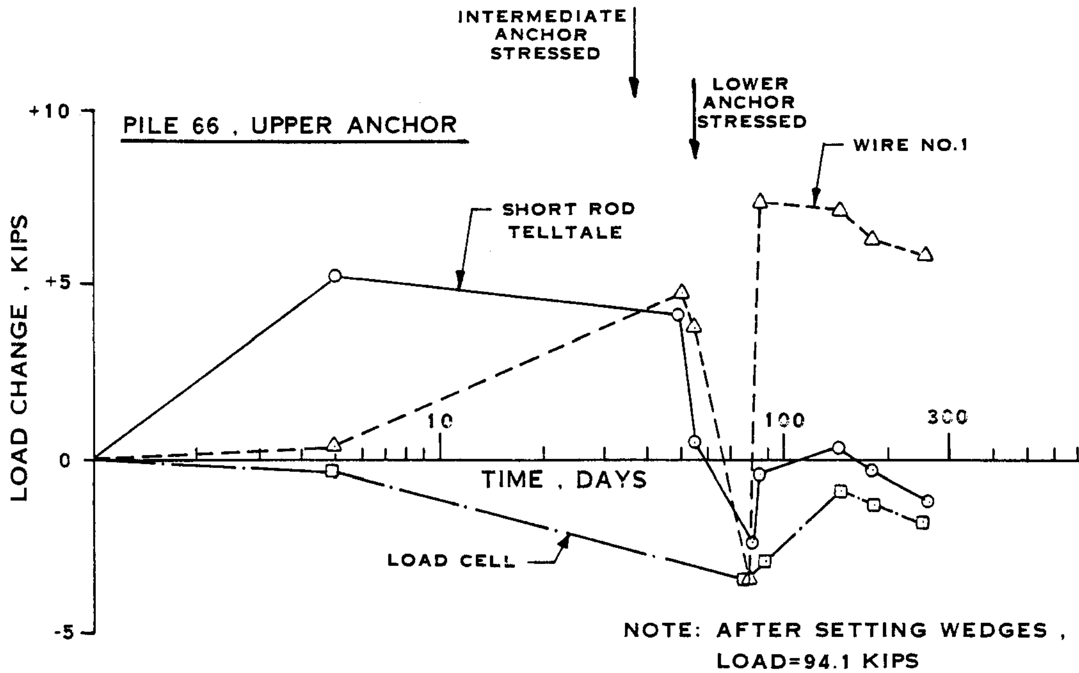
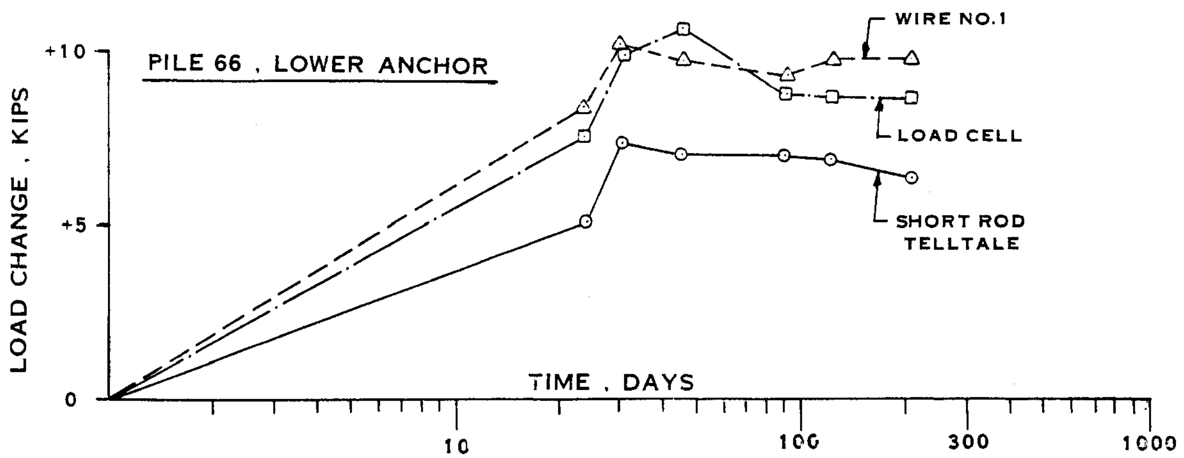


FIGURE 31 - LONG TERM LOAD VARIATION

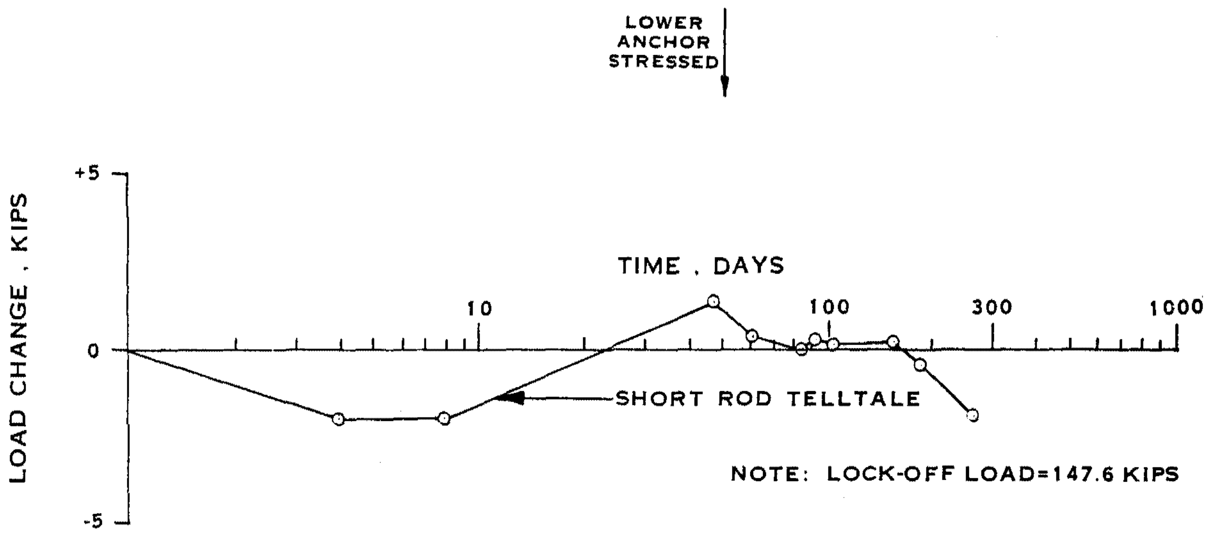




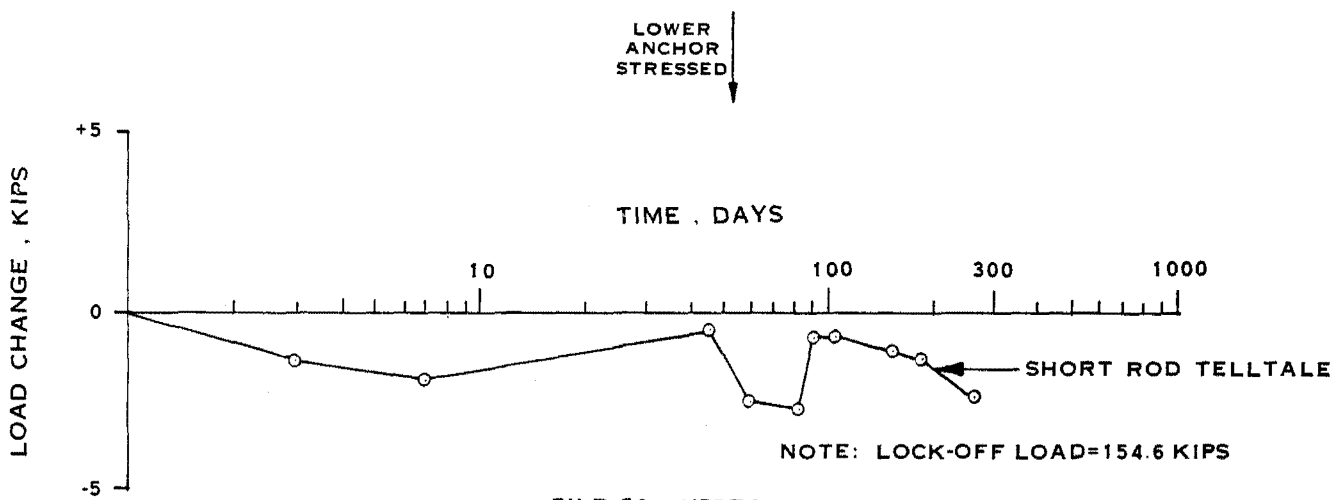
TELLTALE LOAD CELL CHANGES COMPUTED WITH AREA OF FOUR STRANDS ONLY

NOTE: AFTER SETTING WEDGES,
LOAD=99.9 KIPS

FIGURE 32- LONG TERM LOAD VARIATION



PILE 50 . UPPER ANCHOR



PILE 52 , UPPER ANCHOR

FIGURE 33 - LONG TERM LOAD VARIATION

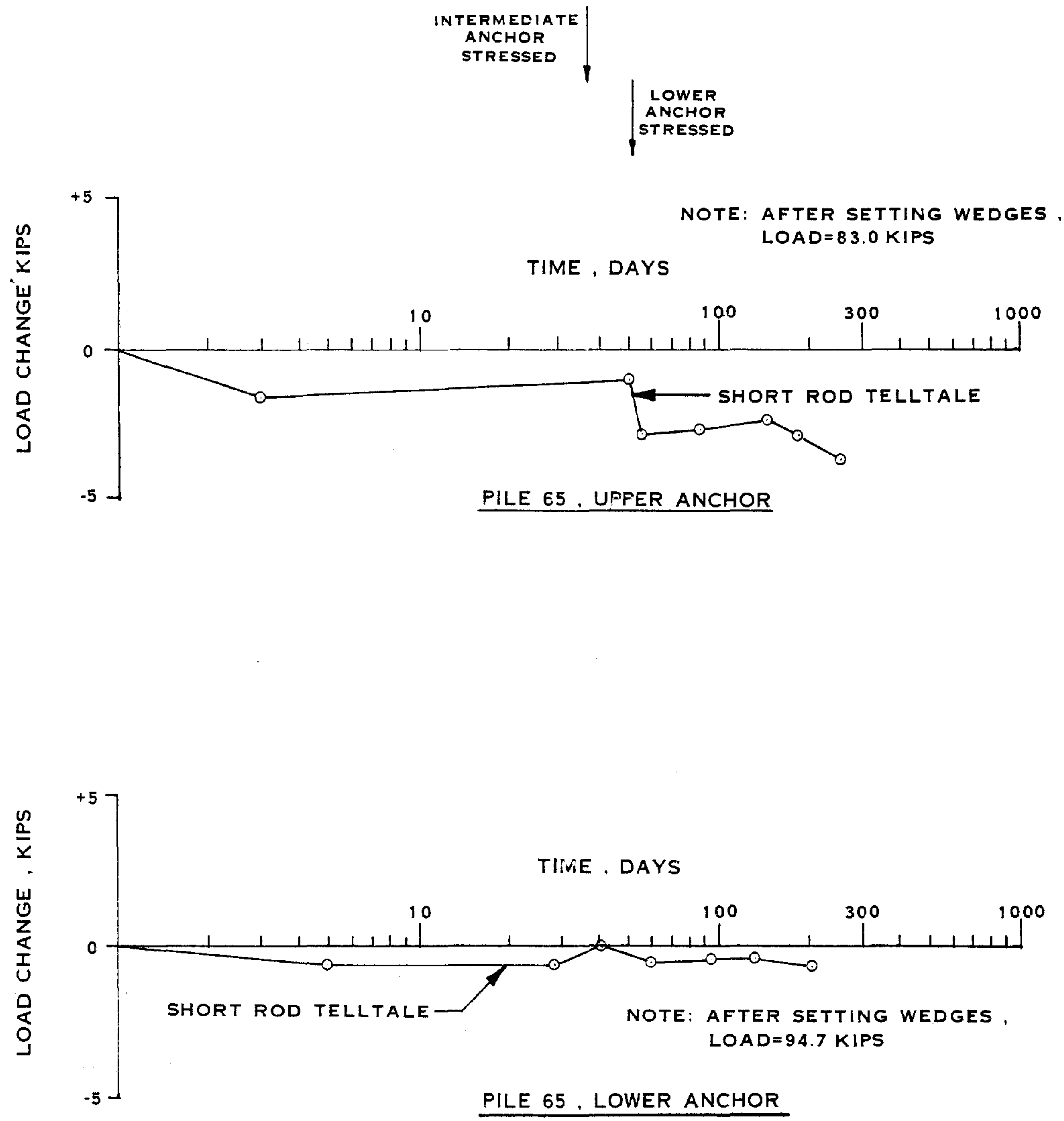
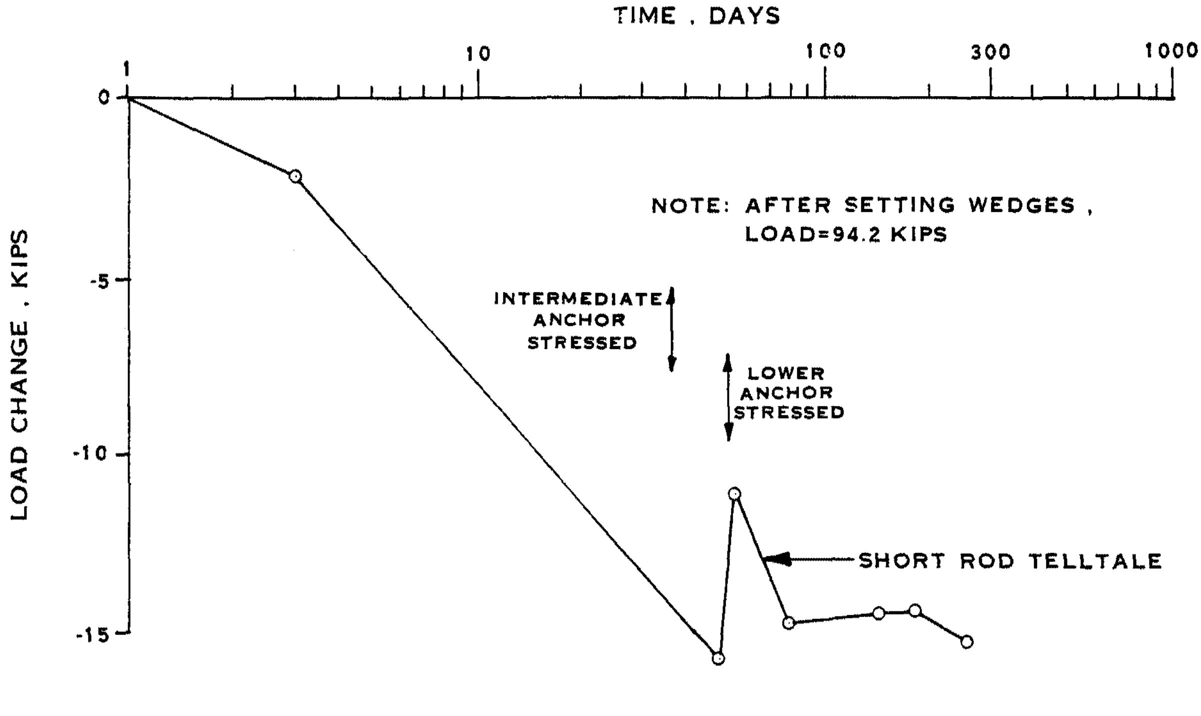
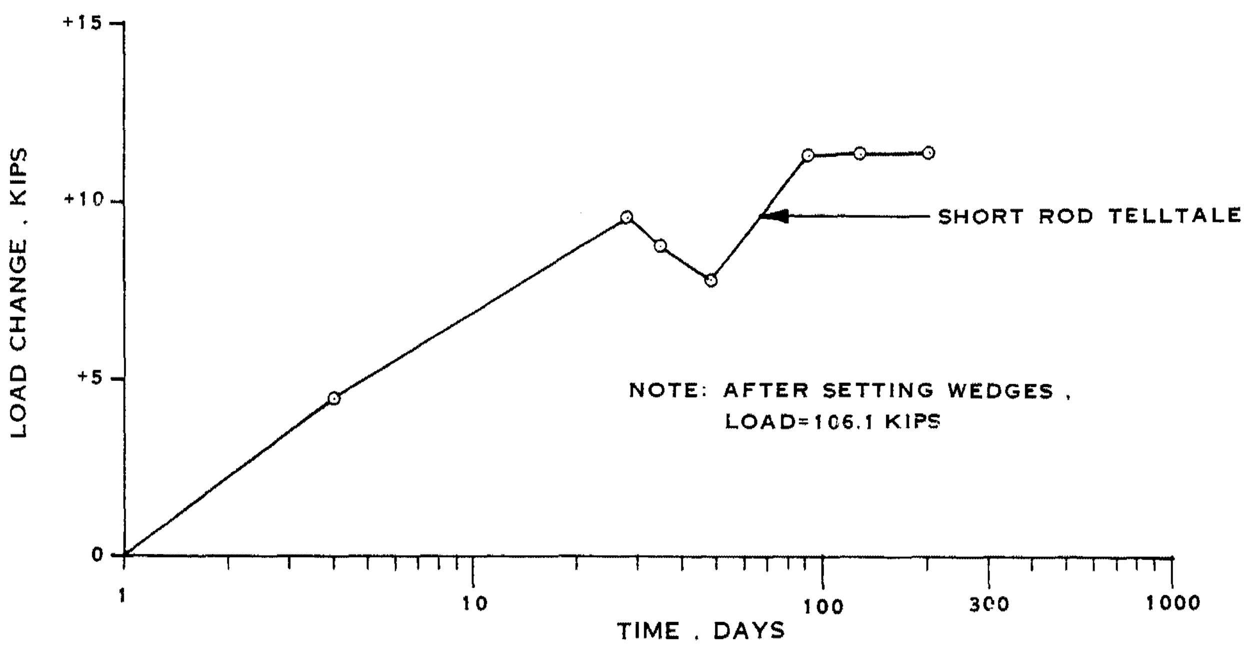


FIGURE 34-LONG TERM LOAD VARIATION



PILE 67 , UPPER ANCHOR



PILE 67 , LOWER ANCHOR

FIGURE 35-LONG TERM LOAD VARIATION

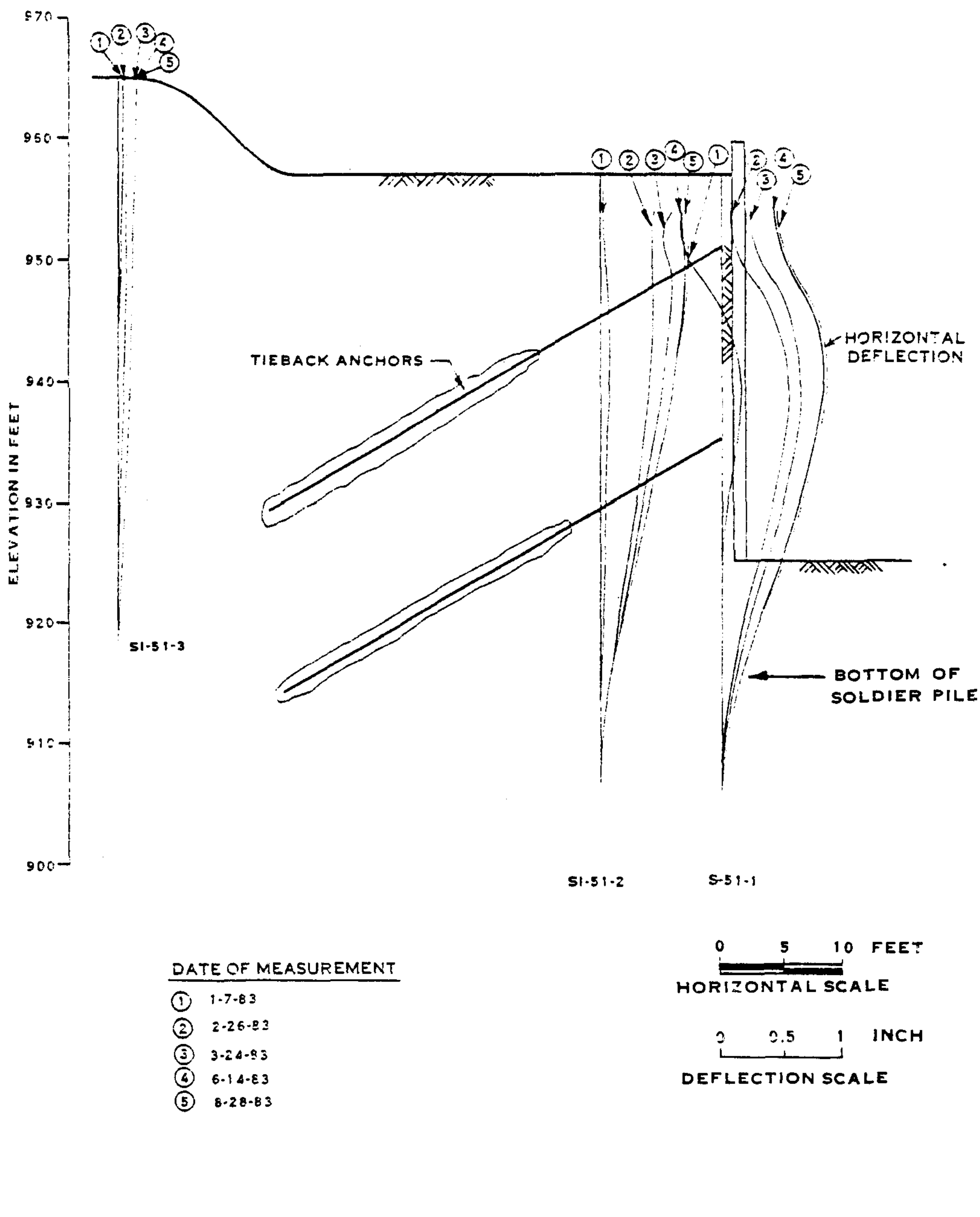


FIGURE 36 - HORIZONTAL DEFLECTION BEHIND PERMANENT TIEBACK WALL (PILE 51)

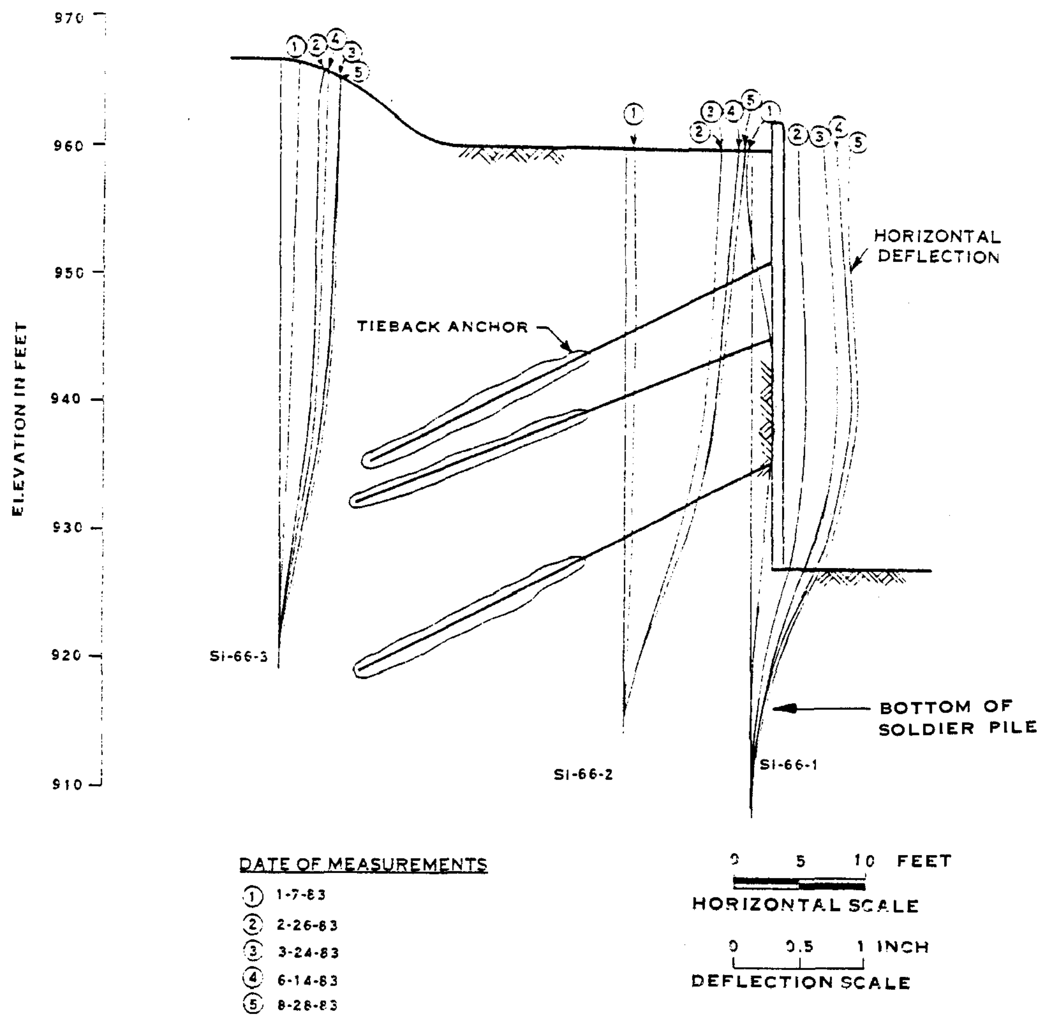
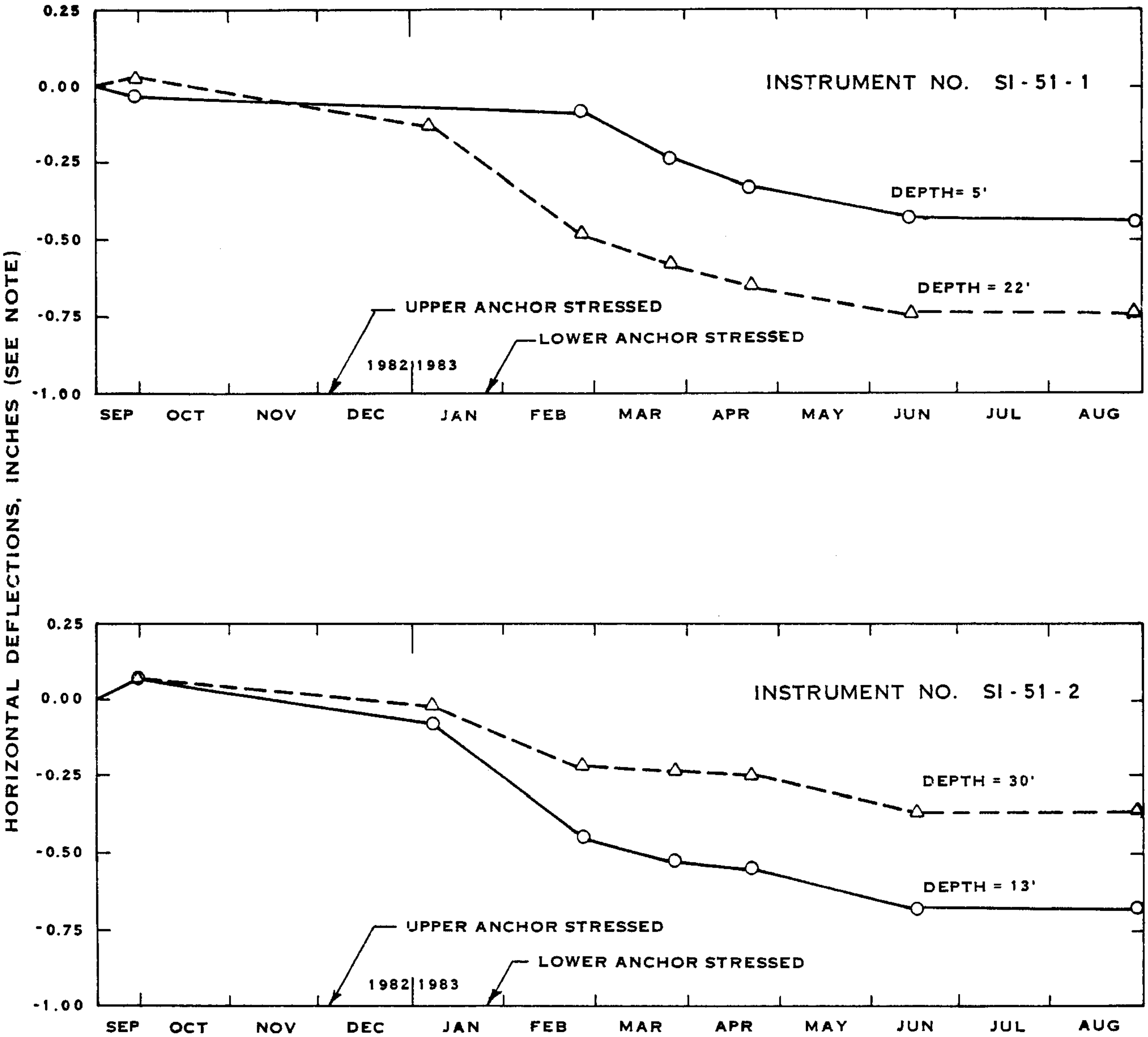
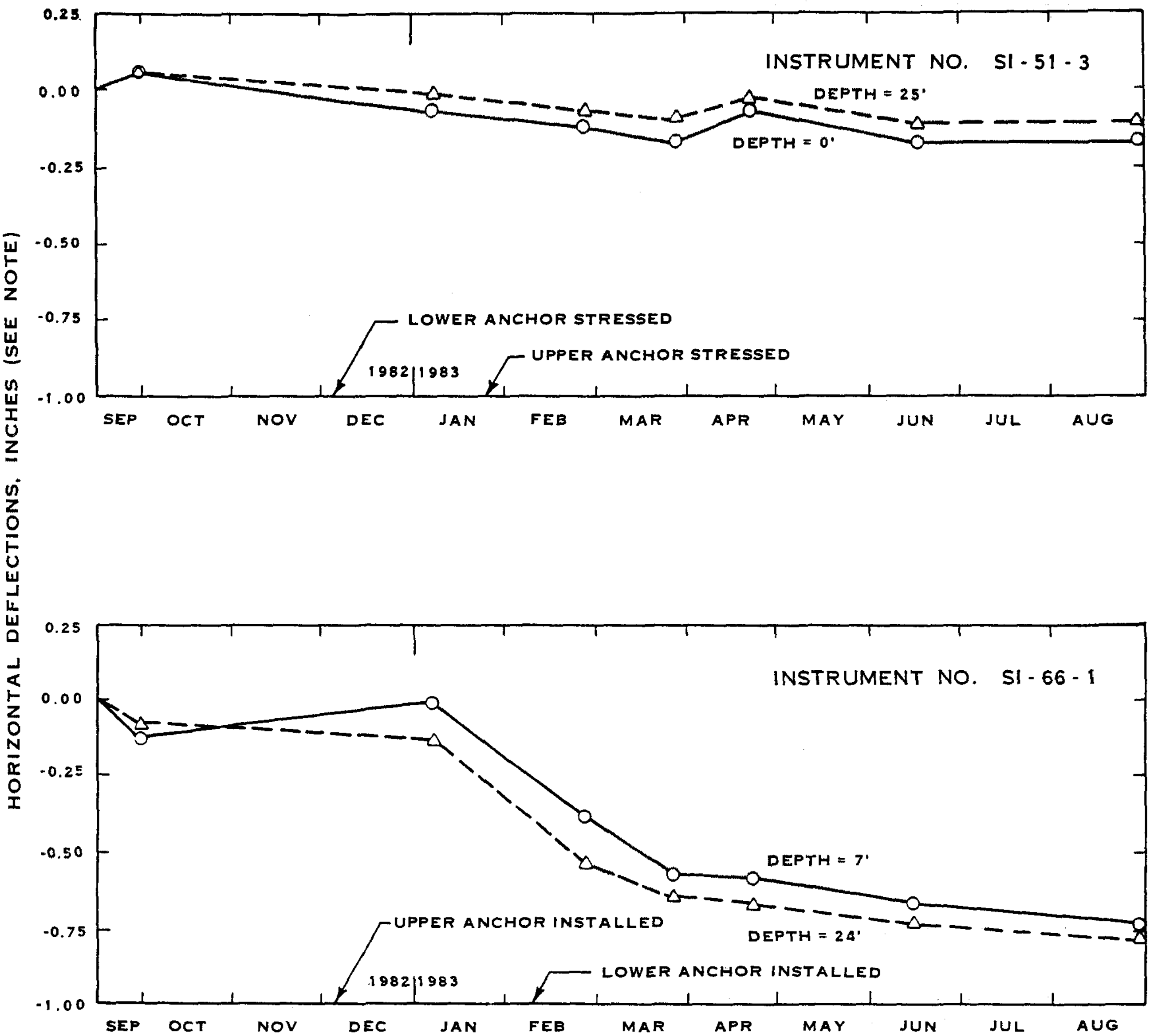


FIGURE 37 - HORIZONTAL DEFLECTION BEHIND PERMANENT TIEBACK WALL (PILE 66)



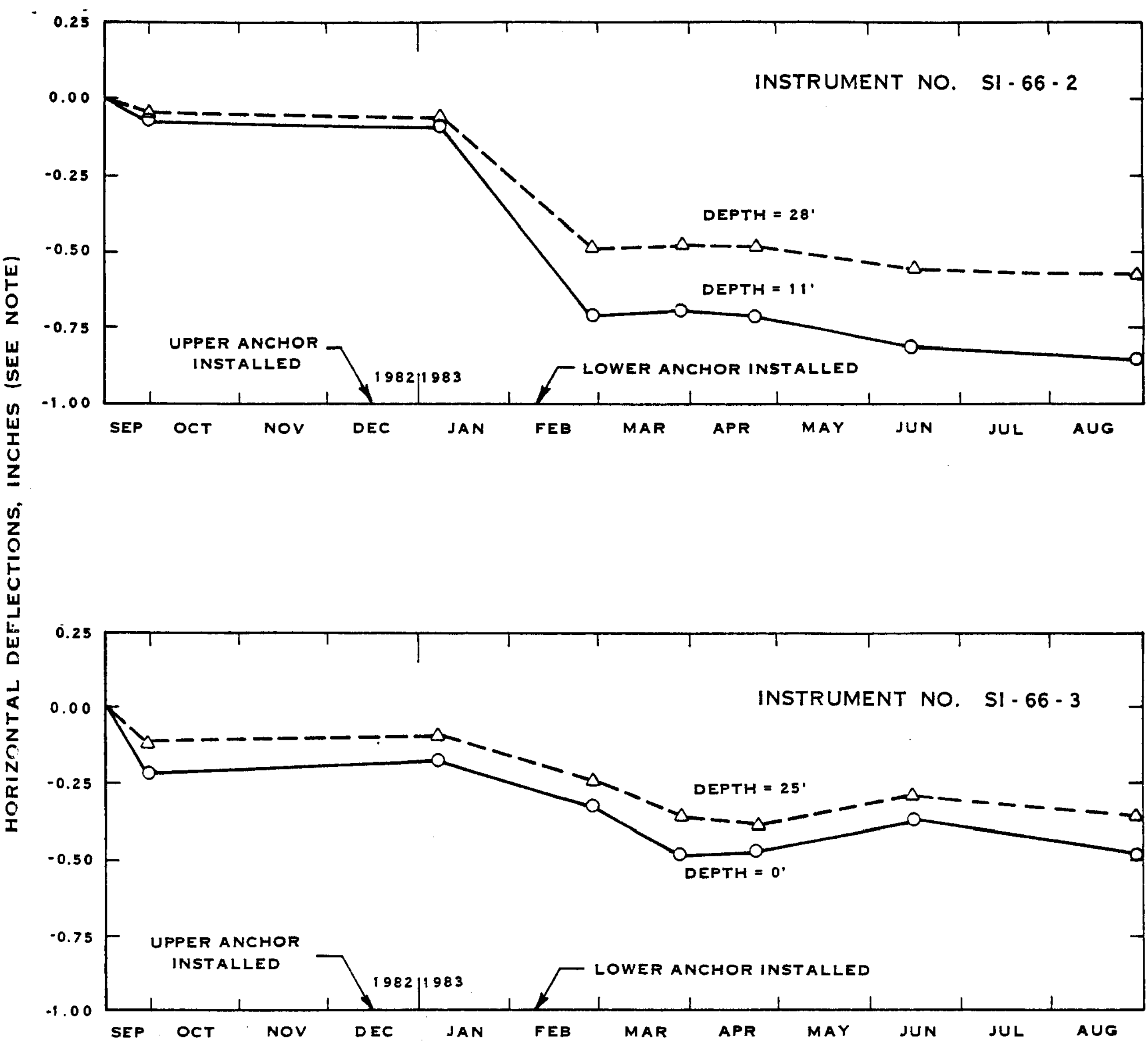
NOTE: (+)MOVEMENT AWAY FROM EXCAVATION
(-)MOVEMENT TOWARD EXCAVATION

FIGURE 38 - INCLINOMETER DEFLECTION VERSUS TIME



NOTE: (+)MOVEMENT AWAY FROM EXCAVATION
(-)MOVEMENT TOWARD EXCAVATION

FIGURE 39 - INCLINOMETER DEFLECTION VERSUS TIME



NOTE: (+)MOVEMENT AWAY FROM EXCAVATION
(-)MOVEMENT TOWARD EXCAVATION

FIGURE 40 - INCLINOMETER DEFLECTION VERSUS TIME

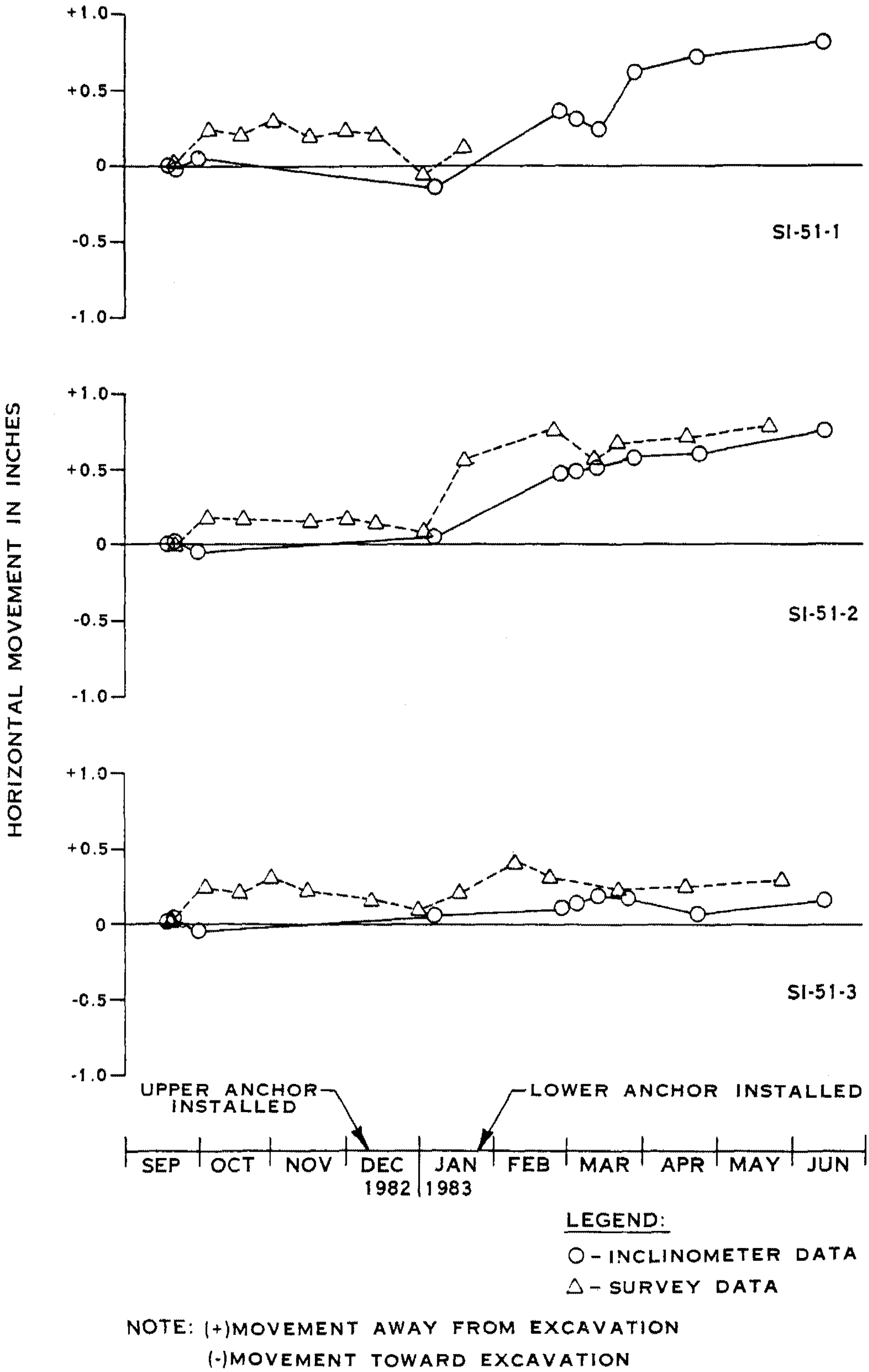
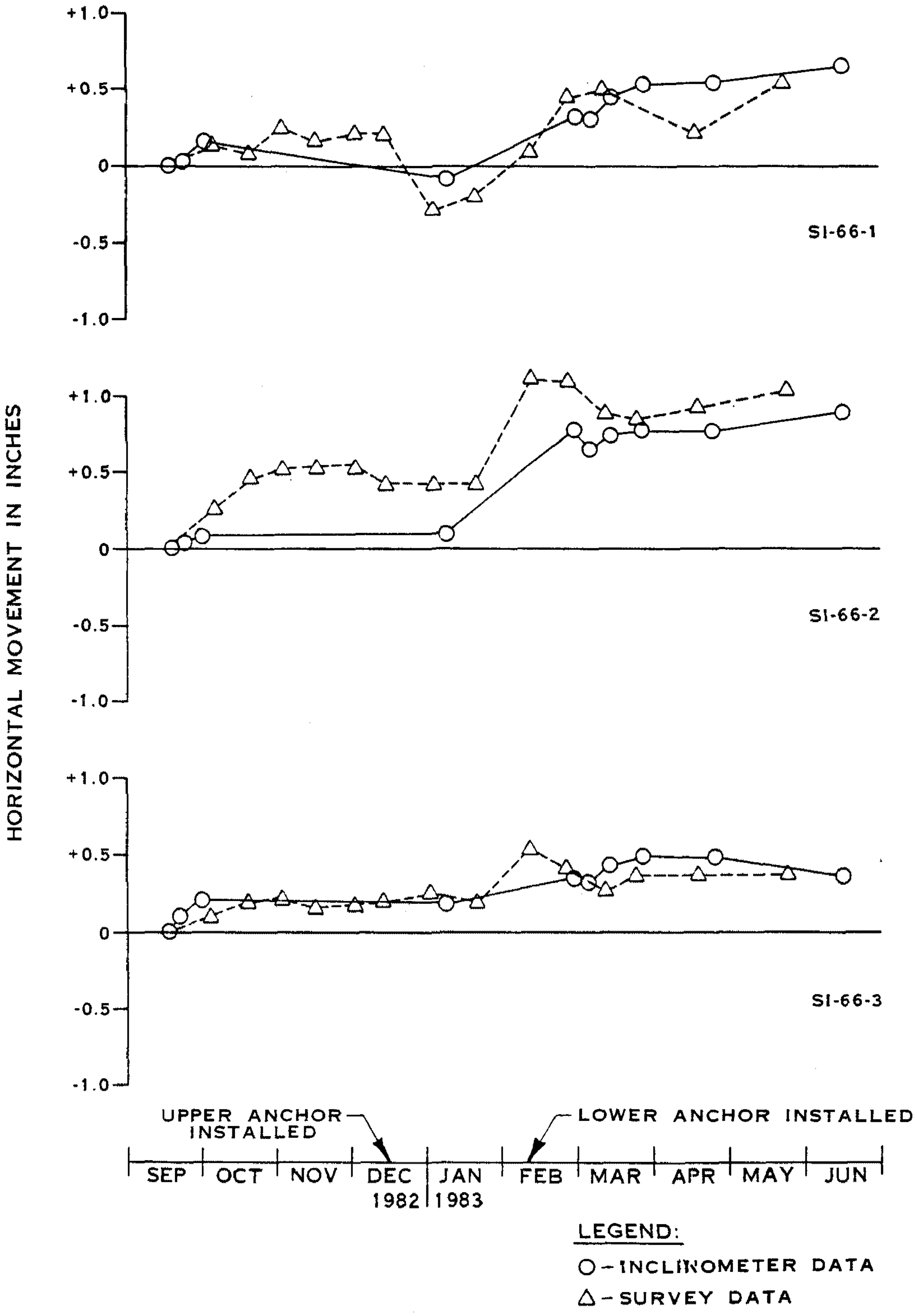
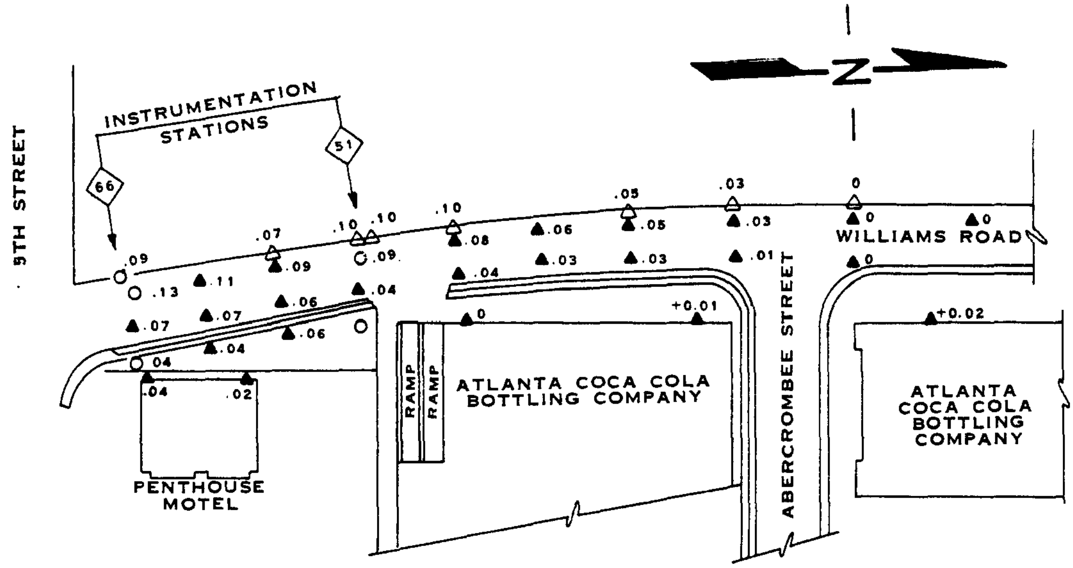


FIGURE 41 - COMPARISON OF INCLINOMETER TOP MOVEMENT SURVEY DATA VERSUS INCLINOMETER DATA

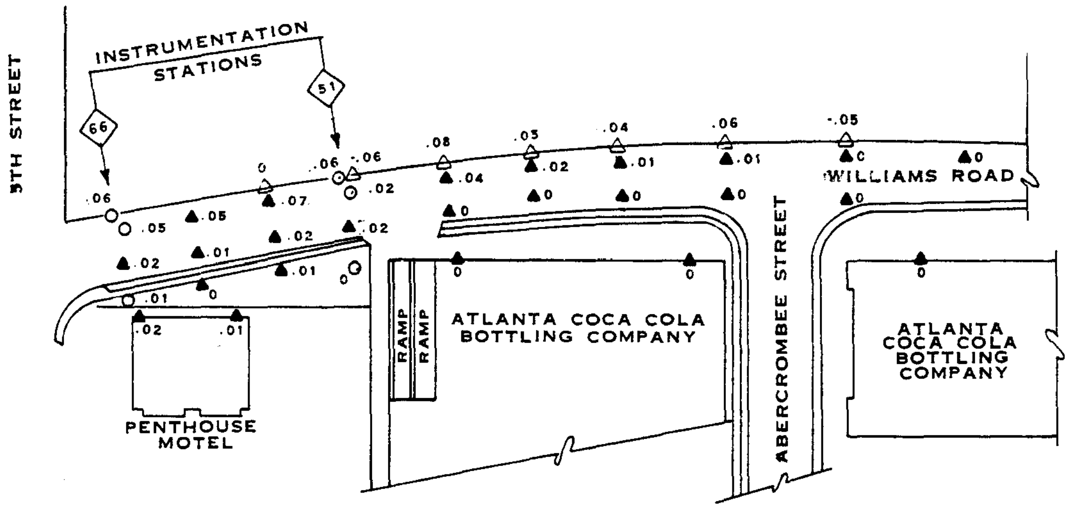


NOTE: (+)MOVEMENT AWAY FROM EXCAVATION
(-)MOVEMENT TOWARD EXCAVATION

FIGURE 42 - COMPARISON OF INCLINOMETER TOP MOVEMENT SURVEY DATA VERSUS INCLINOMETER DATA



HORIZONTAL DEFLECTION, FEET
(ALL DEFLECTIONS ARE OUTWARD)
NO SCALE



VERTICAL DEFLECTION, FEET
(ALL DEFLECTIONS ARE DOWN EXCEPT AS NOTED WITH -)
NO SCALE

LEGEND

△ PILE MONITOR (6 FEET BELOW TOP OF WALL)

▲ GROUND MONITOR OR BUILDING MONITOR

○ SLOPE INDICATOR MONITOR

FIGURE 43 - SUMMARY OF MAXIMUM HORIZONTAL AND VERTICAL SURVEY MEASUREMENTS

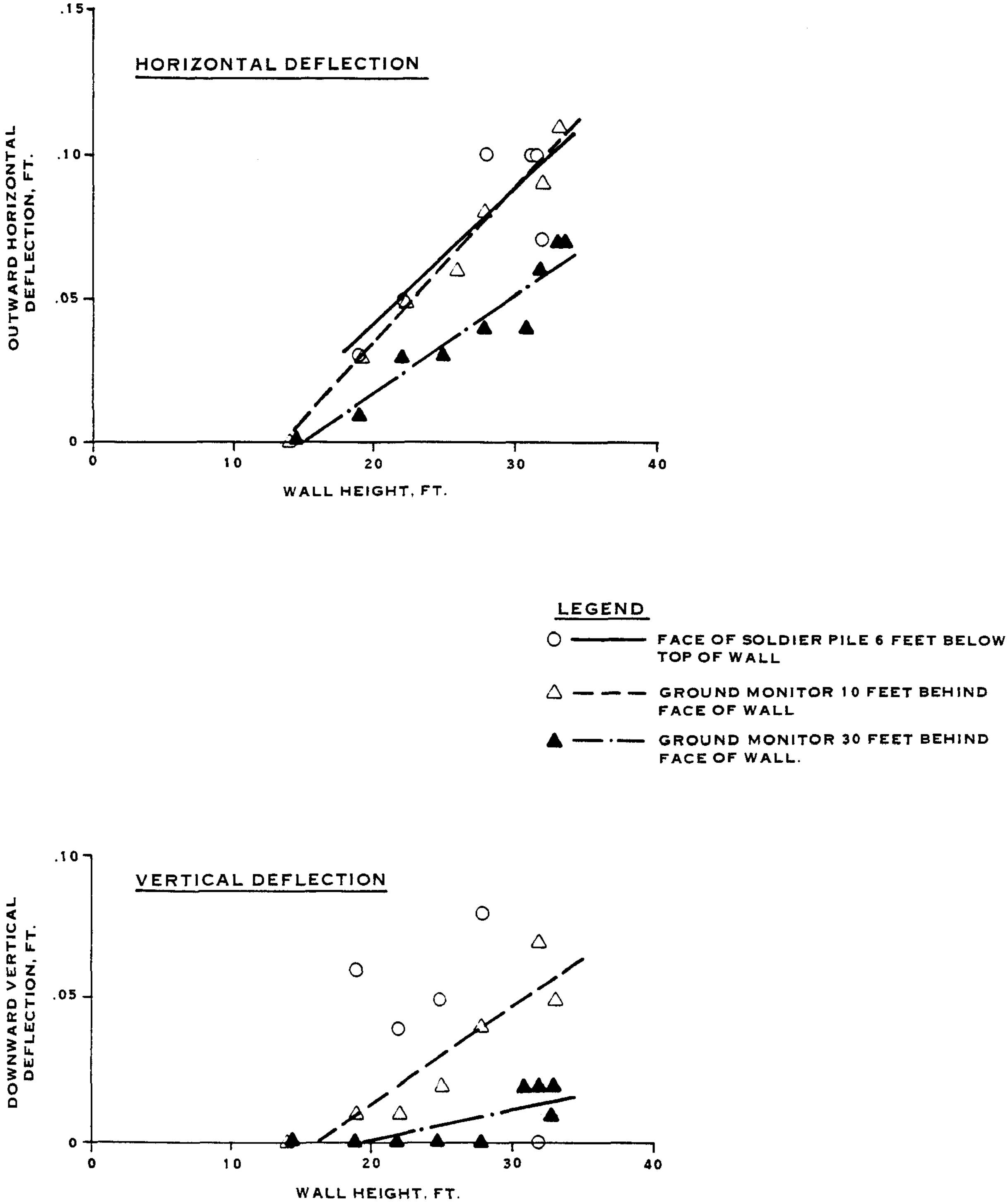
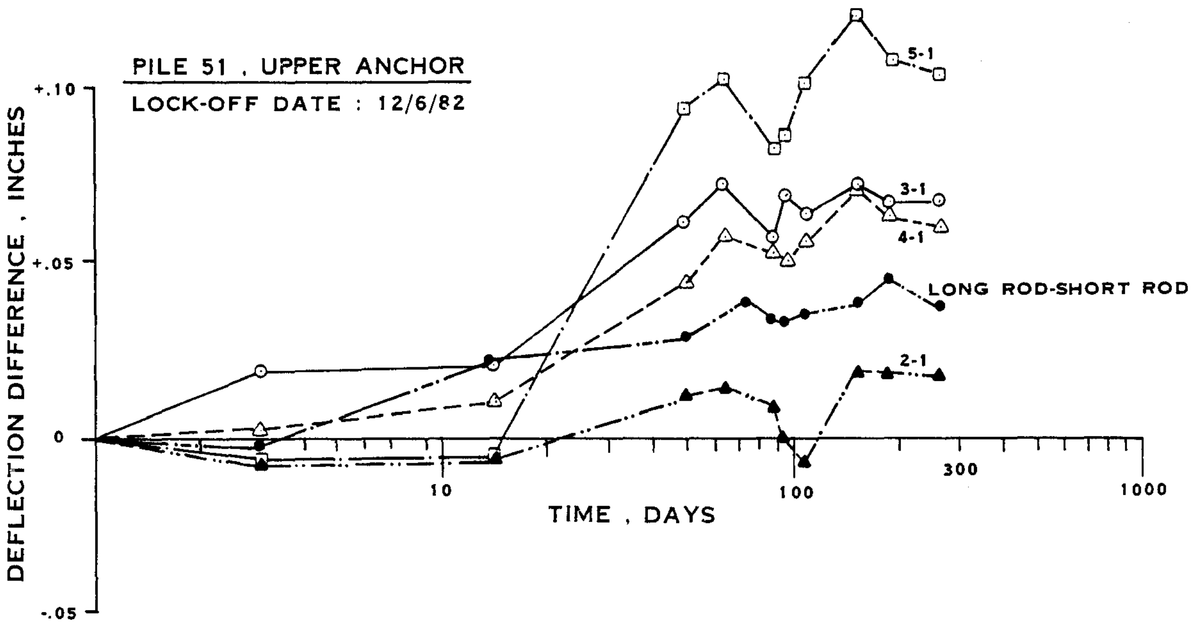


FIGURE 44 - HORIZONTAL AND VERTICAL SURVEY DEFLECTIONS VERSUS WALL HEIGHT



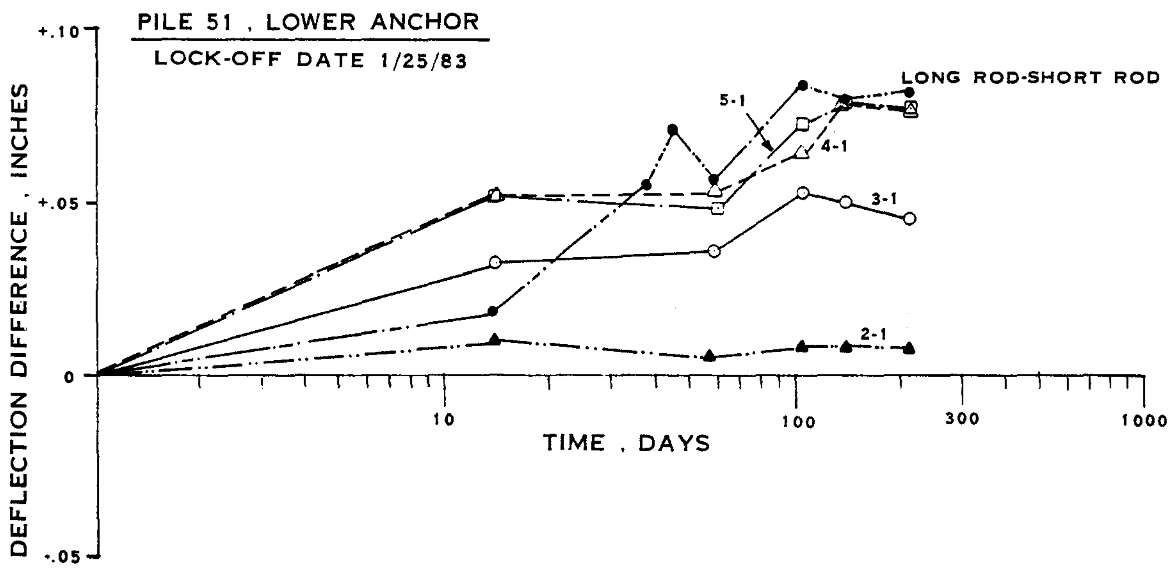
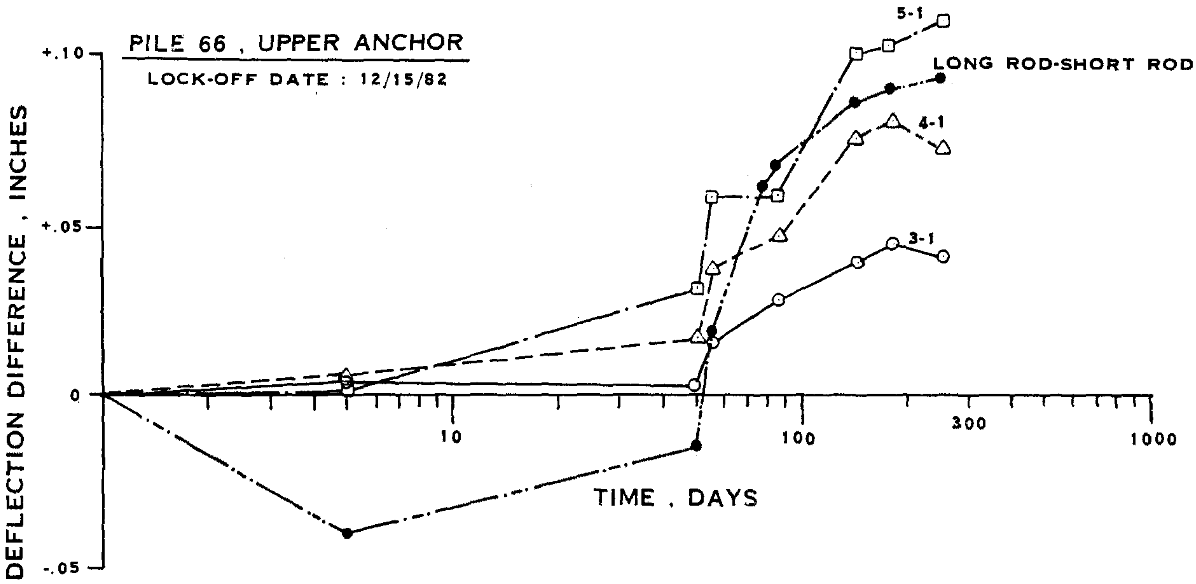


FIGURE 45

LONG TERM RELATIVE DEFLECTION IN BONDED ZONE



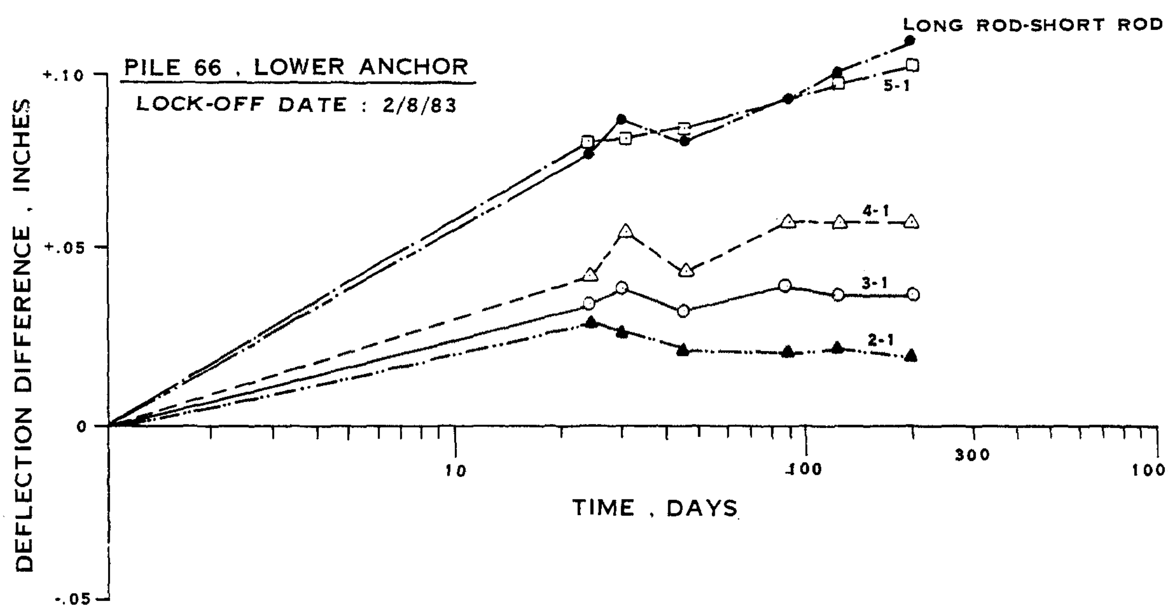


FIGURE 46

LONG TERM RELATIVE DEFLECTION IN BONDED ZONE

APPENDIX B

File #80-25

November 5, 1981

Mr. Kenneth P. Akin, Jr.
Law Engineering Testing Company
401 West Peachtree Street, N.E.
Suite 1840
Atlanta, GA 30308

Re: Fulton County Permanent Tieback Wall

Dear Ken:

This letter will summarize my recommendations for instrumenting the above tieback wall.

1. SUMMARY OF MEASUREMENT METHODS

The primary parameters of interest in evaluating performance are given in Table 1, together with desirable accuracies, recommended instruments, and probable accuracies. Details are discussed in turn below. The two "primary stations" referred to in the table will, as agreed during our October 21 meeting, be selected by you and Georgia DOT.

2. HORIZONTAL DEFORMATION OF WALL FACE

Inclinometers

At the two primary stations, pipe will be tack welded to the full length of the soldier pile (see Figure 1), and plugged at its lower end with a wooden plug. The top of the pipe should be at final grade, i.e., above the soldier pile top. The pipe will be set in place with the soldier pile and, after the porous concrete has set, Georgia DOT will drill a minimum 3-inch diameter hole, through the pipe, to about 10-ft below the bottom of the soldier pile. The actual depth will be selected when geologic cross sections at the primary stations have been drawn. The inclinometer casing will then be installed by grouting through a pipe inside the inclinometer casing, mating with a check valve in the bottom cap. This method is necessitated by space considerations, and the need to minimize the size of the steel pipe from the structural reinforcement standpoint.

The minimum i.d. for the steel pipe is 3-1/2", and Georgia DOT should confirm that they have drill tools that will drill a hole no less than 3" diameter through a 3-1/2" pipe. The new "CPI" inclinometer couplings, discussed during our October 21st meeting, should not be used as they protrude outside the casing o.d. and aggravate the space problem. Presumably Nicholson Construction Co. (NCC) will supply and install the pipe.

Table 2 gives a listing of required materials, on the assumption they will be ordered from Slope Indicator Company.

EDM Surveys on Wall Targets

Presumably all will be handled by Georgia DOT. At each measurement station targets should be set at each anchor head level and at the wall top. Stations should include the two primary stations, the four adjacent soldier piles (locations of telltale load cells), and approximately 8 others, equally spaced along the wall. Arrangements will have to be made for transposing targets from soldier piles to wall face, presumably by setting out targets before the wall is poured.

Conventional Survey on Top of Wall, Using Offsets from a Line of Transit

This is a duplication of EDM measurements on to the wall top, and provides redundancy of a significant parameter. Reference points will need to be set well behind any zone of possible movement, and a line of transit established from these points. The ten measurement stations should be the same as those used for EDM measurements, plus the location of the four soldier piles adjacent to the primary stations. Data will also provide a check on inclinometer data in the two casings at soldier piles.

3. ABSOLUTE DEFORMATION OF ANCHORS

Absolute deformation of anchors at each of the two primary stations will be measured by a combination of inclinometer and telltale readings. Deformation of the soldier pile at each anchor head will be determined using the inclinometer, to which will be added the measured relative deformations between soldier pile and each end of the bond zone. Relative deformations will be determined from two rod telltales installed with each anchor, one attached to a strand at the top of the bond zone, the other to the same strand at the bottom of the bond zone. The upper telltale on each anchor will be the same as used for the "telltale load cells," described later and shown on Fig. 5. Hence these serve both as deformation and load indicators.

4. SETTLEMENT OF WALL

Presumably all will be handled by Georgia DOT, using EDM or conventional optical leveling procedures. Targets should be at the same ten stations used for survey measurements on the wall face pulse the location of the four soldier piles adjacent to the primary stations.

5. HORIZONTAL DEFORMATION AND SETTLEMENT OF SURFACE OF SOIL BEHIND WALL

Presumably all will be handled by Georgia DOT, using EDM or conventional optical survey procedures. Targets should be at the same ten stations used for survey measurements on the wall face (but no need for measurements at the locations of the four soldier piles adjacent to the primary stations). Targets should be set 10 ft, 30 ft, and 60 ft behind the wall, and should consist of a sleeved rod or pipe anchored below the zone of seasonal vertical deformation. Figure 2 shows the standard New York DOT detail, which would be suitable.

At four of the locations the target will be the top of inclinometer casings, described below. At these locations, the horizontal deformation measurements will provide a check on inclinometer data.

6. HORIZONTAL DEFORMATION OF SOIL BEHIND WALL, BELOW SURFACE

Inclinometer casings will be installed at the two primary stations, 10 ft and 50 ft behind the wall. Installation procedure will be the standard Georgia DOT method, using a grout backfill injected down a tremie pipe outside the inclinometer casing. Table 2 indicates required materials, assuming the use of "CPI" casing. These should be ordered as soon as possible so that installations can be made in the near future, before any additional excavation at the site. In planning the installation method, note the coupling o.d. is 3.07". Top arrangements should be designed so that access will eventually be through a removable cover flush with the ground surface. Comprehensive arrangements should be made to protect the installations from damage by construction equipment.

7. LOAD IN ANCHOR STRESSING LENGTH

The primary method of load measurement will be use of load cells. Telltale load cells, as shown schematically in Figure 3, will also be used.

<u>Load Cells</u>

A load cell will be installed on each of the two anchors, at the two primary stations. As discussed under "Load in Anchor Zone" below, a larger anchor head will be required for these four anchors, with an o.d. on the strand pattern of 4.65". Hence the load cell i.d. needs to be about 5". Other load cell design criteria are:

- o 225 kip capacity (to be confirmed when primary station are selected).

- Adequate wall thickness for structural stability.
- length: o.d. ratio not less than 1.4, to minimize errors due to end effects, i.e., a length of about 9".
- Rounded ends, to minimize errors due to end effects.
- Strain concentration at the gage points to maximize sensitivity. This can be accomplished by installing strain gages in side holes as on Figure 4.
- Hermetically sealing strain gages, and verifying seal integrity using a helium leak test.
- Use of a spherical bearing between bearing plate and load cell, to accommodate inevitable imperfect alignment.
- Use of bearing washers above and below the load cell, with Rockwell hardness less than C32.
- A waterproof connector, with dust cap, facing sideways.

I know of only one company capable of supplying cells conforming to the above criteria: Brewer Engineering Laboratories, Inc. (BEL), Marion, Ma. They have many years of experience in field strain gage work, and have helium leak test equipment. This test provides a check on gage hermetic sealing, and essential feature when using resistance strain gages for long term applications. I met with BEL on October 28 to discuss our requirements, and have received from them the budget in Table 3.

While at BEL, I discussed the need for providing a method of removing cells for check calibration at any time. In their experience, given high quality hermetically sealed strain gages, there is very minor likelihood of a significant change in gage calibration over a 15 year period. However, the gage zero can change and, as our required accuracy is very high, they recommended a procedure for checking at any time. They do not believe it is necessary to remove the cells, and are concerned that the split shim arrangement used previously by NCC may create non-uniform loading on the upper end of the cell. They recommend a lift-off method, using a pull-bar, allowing the cell zero reading to be reestablished at any time. By strain gaging the pull-bar or by using a supplementary load cell, the calibration can also be verified during the lift-off tests. I support this recommendation. Details of this arrangement are best worked out by NCC with input from BEL as necessary.

Use of load cells will extend the anchor head length by about 15 inches (cell 9", two bearing washers 1/4" each, spherical bearing 2-1/2", additional bearing plate 2-3/4"). As discussed during our October 21 meeting, the soldier beams at the two primary stations should be installed out-ot-line, further from the wall face, to accommodate the cells. A detail drawing will be required from NCC.

Telltale Load Cells

The principle of a telltale load cell is shown in Fig. 3. Assuming no creep in the strands over the stressing length, telltale load cells will provide backup to the load cell measurements. However, if any creep does occur in the stands over the stressing length, comparison between load cell and telltale load cell data will provide data on creep magnitude.

As described earlier under "Absolute Deformation of Anchors" these telltales will also be used as deformation indicators. A schematic of rod telltale arrangements, both for local and deformation measurements, is shown on Figure 5. Note the location of the telltales at the anchor heads, the position with respect to tendon spacers, and the need for tendon spacers in the stressing length.

Each telltale will consist of a flush-coupled 1/4 inch stainless steel rod within an oil-filled flush-coupled 1/4" Schedule BO PVC pipe (0.540" o.d., 0.302" i.d.). A disconnect will be provided between telltale anchor and telltale rod, so that telltale rods can be installed after anchor grouting and so that free-sliding can be checked at any time. Telltale anchors will be attached to strands using clamps and epoxy. Stainless steel reference plates will be required on the anchor heads.

Installation will generally be as follows:

- o Attach telltale anchor and PVC pipe to strand, feeding pipe through appropriate tendon spacers.
- o Cement cap on upper end of PVC pipe.
- o Insert strands, pipe, etc., into corrugated plastic pipe.
- o Install and grout anchor.
- o Cut off pipe cap.
- o Fill pipe with oil.
- o Insert telltale rod in pipe and lock in place.
- o Conduct anchor proof test and calibrate telltale "change in stickout" versus load, using the BEL load cell.

Note that the final step will require that the last of the 4 primary station anchors must be proof tested **after** all 8 adjacent anchors, so that a BEL load cell can be used for all in-situ calibrations.

Although the telltale principle is very straightforward, many details need to be finalized before components are machined. In my view, the expedient and efficient way if for me to coordinate directly with Geokon Inc., Lebanon, NH (Dr. Barrie Sellers) to finalize details and prepare shop drawings. Components can then readily be machined by Geokon. Clearly there are alternative sources for these materials, but none allowing such close coordination. I have used Geokon for similar work previously, and have found them to be both efficient and competitive. Budget information is included in Table 4. Note the item for engineering time: This is for Barrie Sellers' time to coordinate with me and prepare shop drawings.

8. LOAD IN ANCHOR BOND ZONE

In my October 9, 1981, letter, I identified three possible methods of measuring load in the bond zone: resistance strain gages, vibrating wire strain gages, and telltales. Having studied these options further, I now consider that strain gages are not practicable, for the following reasons:

- o At 225 kips the strands are subjected to about 7000 microstrain. This is nearly three times the range of the miniature vibrating wire strain gages and very large when considering long-term drift-free performance of resistance strain gages.
- o The gage carrier plate must not reinforce the strand significantly. Thus the carrier plate cannot be robust, and would be very subject to damage while installing the anchor.
- o A proven method of clamping the carrier plate to the strand is not available. I've evaluated various options, but all would entail a significant test effort before they could be relied upon.
- o There is a likelihood of torsional strains in the strands as they are tensioned, hence an uncertainty in converting measured strain to axial load.

The alternative of multiple telltales is both practicable and economical. The principle is essentially the same as the telltale load cell, using any pair of telltales to create a gaged length of the bond zone. The system would be installed on the four anchors at the two primary stations. Sketches of the arrangement are shown on Figs. 6 and 7.

Proposed details are:

- o A telltale anchor attached to one strand at 5 points in the bond zone, i.e., at 7' -6" spacing. The anchor surface will be knurled to match the strand outside irregularities, for good bond.

- A wire attached to each anchor.

- A 1/4" oil-filled nylon tube around each wire, connected to a tube fitting threaded into the anchor.

- A 3/4" PVC pipe on the axis of the entire 45 ft anchor, with slots for grout entry and nylon tube entry, through which all nylon tubes and wires will pass to a central hole in the anchor head.

- A small collar attached to the upper end of each wire, above the anchor head.

- A chair for the indicator, attached to the anchor head.

- A mechanical indicator, as shown in Fig. 8.

The system is available from University of Newcastle Tyne, England. I've used the Mark 2 version on two projects requiring long wires, but for this project the Mark 1A (not the reversed version) would be used. A major attraction of this device is the ability to make a deformation reading on an individual wire at more than one standard wire tension, because the difference between the two readings should always be equal to the elastic elongation of the wire, i.e., always a known magnitude and always the same. Hence accuracy can be increased by reading at various tensions and drawing a straight line through the plotted points, and any lack of free wire movement will immediately be discerned.

Installation will generally be as follows:

- Build the anchor around the 3/4" PVC pipe, with appropriate bands and spacers as on Fig. 6. Note that this creates two non-standard features: spacers in the stressing length and a separation between strands at band points in the bond zone. I've checked with NCC whether these features are allowable, and they see no problem. Cut additional slots in the pipe as necessary.

- Pre-assemble anchor/wire/tube/oil assemblies with upper end of tube plugged internally. This will entail use of a wire straightener and oil pump, and an excess length of tube to house the full wire length.

- Feed nylon tubes into the 3/4" pipe, from the anchor locations.

- Attach telltale anchors to a strand. Attach a small block of styrofoam above each anchor so that, if the strand pulls outward with respect to the grout during stressing, the anchor clamps will not slip.

- If necessary (to be determined during detail design), pressurize oil in the tubes by connecting all 5 tubes to a common hydraulic line and locking off, to counterbalance subsequent grout pressure.

- o Insert strands, etc., into corrugated plastic pipe.
- o Install and grout anchor.
- o Assemble indicator chair, wire collars, attach indicator, and conduct proof test.

Wire telltale readings during proof testing, will give data for calculation of stresses in the bond zone, using either the theoretical strand modulus or the modulus determined for the telltale load cell. A significant effort needs to be made to ensure compatibility between proof testing and telltale indicator arrangements at the head. Note that during stressing, it may be necessary to use more than one collar on each wire, as the indicator range is 2". Theoretically, 30 ft of 5 strand anchor loaded to 225 kips elongates about 2-1/2".

As for the rod telltales, many details need to be finalized, and I recommend working with Geokon, Inc. to do this. The indicator and associated items can be procured directly by Georgia DOT from England, but I believe it would be more convenient to procure through Geokon, hence ensuring compatibility of all components. Budget information is included in Table 5.

9. TELLTALE DATA REDUNDANCY

The shortest and longest wire telltale should provide the same data as the two rod telltales (see Fig. 6).

The load at the top of the bond zone will be known from load cell and telltale load cell data. The load at the bottom of the bond zone is, of course, zero. Hence the load transfer curve, plotted from wire telltale data, should be consistent with these two known load values.

10. BUDGET FOR MATERIALS

The materials budget is summarized in Table 6. The $33,075 amount includes a 10% contingency and also $1750 of engineering time by Geokon, Inc.

11. MISCELLANEOUS

(a) A stable benchmark will be required for survey measurements.

(b) A suitable blockout needs to be made through the wall at the location of each instrumented anchor. Figs. 9 and

10 show approximate dimensions. Presumably each blockout will be formed with a steel box and, as discussed during our October 21 meeting, special care needs to be taken to ensure that the covers do not allow water to stain the face of the wall. We need to determine who will design, provide and install the steel boxes and covers. NCC should check that I've left enough room on Figure 9 for the lift-off arrangements.

(c) We need to determine the access needs and arrangements for reading at the 12 instrumented anchor heads, both during and after construction.

12. ACTION ITEMS BY GEORGIA DOT

(a) Review the recommendations in this letter.

(b) Confirm suitability of 3-1/2" min, i.d. for steel pipe on soldier piles.

(c) Order casing, etc., after confirmation of hole depths.

(d) Confirm availability of nearby stable benchmark.

(e) Authorize me to submit a load cell specification to BEL and to request that they submit a quotation to Georgia DOT. If acceptable, an order should be place quickly.

(f) Authorize Geokon, Inc. to proceed with preparing detail designs and shop drawings for rod and wire telltales, and to order appropriate items from England as on Table 5

13. ACTION ITEMS BY LETCO

(a) Review the recommendations in this letter.

(b) Determine required lengths of inclinometer casings.

(c) Confirm load cell capacity is 225 kips (depends on selected primary stations).

14. ACTION ITEMS BY NICHOLSON CONSTRUCTION CO.

These assume acceptance of Nicholson's drawings, and on resolution of any contractual matters resulting from these instrumentation plans.

(a) Make a thorough check on my proposed rod and wire telltale arrangements, installation and reading procedures, to be certain that all is practicable.

(b) Select and supply suitable pipe with wooden plug for inclinometers on two soldier beams. Minimum i.d., 3-1/2". If possible, thinner wall than standard pipe (standard 3-1/2" is 0.226" wall). Include on construction drawings.

(c) Prepare detail drawing showing primary station soldier piles installed out-of-line, with load cell, bearing washers, spherical bearing and additional bearing plate. Coordinate with BEL as necessary. Supply 4 additional bearing plates.

(d) Prepare detail drawing showing load cell lift-off arrangements. We need to discuss:

o How will lift-off tests be made? Who will make them? Who provides what materials?

o Whether to use a supplementary load cell or a strain gaged pull bar. I favor the latter, and recommend that NCC provide a pull bar (and end attachments for calibration) to BEL for strain gaging.

(e) Provide 48 additional tendon spacers for anchor stressing lengths to accommodate rod and wire telltales:

o 4 in each of 4 anchors at primary stations.

o 4 in each of 8 anchors adjacent to primary stations.

(f) Provide 4 anchor heads at primary stations with 9 holes. I understand these are standard, with 2-1/8" diameter threaded central hole and 8 holes 0.7" diameter, with head o.d. 5.95" and o.d. on strand pattern 4.65". Drill and tap heads to receive rod telltale reference surfaces and wire telltale indicator chair. Confirm you will have a jack with adequate center hole diameter for this pattern.

(g) Provide 5" pipe sleeve (instead of 4) at 4 anchors at primary stations.

(h) Provide 8 anchor heads at stations adjacent to primary stations, with approx. 1-1/8" diameter central hole and 6 outer 0.7" diameter holes (5 for strands, one for secondary grouting). o.d. on strand pattern approx. 3.7". Drill and tap heads to receive telltale load cell reference surface.

(i) Assist with ensuring compatibility between wire telltale indicator and proof testing arrangements at the head so that deformation readings can be taken proof testing.

(j) Check adequacy on Figure 9 of room for lift-off arrangements.

As discussed with you, I believe that all recipients of this letter should meet in the near future to resolve outstanding points so that we can proceed with critical items. NCC believes that the first anchor could be installed as early as mid January, and load cell delivery may be 8 weeks. A good date for that meeting would be Thursday November 12. According to my present schedule, I will be out of the country from November 20 through December 15, hence my availability is very limited.

Sincerely,

John Dunnicliff

TABLE 1

SUMMARY OF MEASUREMENT METHODS

Parameter	Desirable Accuracy	Recommended Instrument	Probable Accuracy
Horizontal deformation of wall face.	± 0.1"	Inclinometer attached to soldier pile at two primary stations.	± 0.1"
		EDM survey on to wall targets at approx. 10 stations.	To be assessed by Georgia DOT.
		Conventional survey, measuring offsets from a line of transit, on to targets at top of wall.	± 0.1"
Absolute deformation of anchor.	± 0.1"	Telltales on anchors at two primary stations (also using data from inclinometer attached to soldier beam).	± 0.1"
Settlement of wall.	± 0.1"	Conventional survey or EDM on to targets at top of wall.	± 0.1"
Horizontal deformation and settlement of surface of soil behind wall.	± 0.1"	Conventional survey or EDM on to shallow reference points in soil at approx. 10 stations.	± 0.1"
Horizontal deformation of soil behind wall, below surface.	± 0.1"	Inclinometers at two primary stations.	± 0.1"
Load in anchor stressing length.	+ 0.5 kips for first year. Zero drift not exceeding 5 kips after 15 years.	Load cells on anchors at two primary stations, with arrangements for in-situ recalibration (4 cells).	± 0.5% of range, i.e., ± 1.1 kips for 225 kip cell. In-situ recalibration allows check on drift and calibration any time.
		Telltale load cells on anchors at two primary stations and at four adjacent secondary stations (12 telltales).	± 2 to 5 kips if no creep in strands. Comparison with load cell gives data on any creep in strands over stressing length.
Load in anchor bond zone.	As for stressing length.	Multi-wire telltales on anchors at two primary stations, with mechanical indicator.	± 2 to 5 kips if no creep in strands.

TABLE 2

INCLINOMETER EQUIPMENT TO BE ORDERED FROM
SLOPE INDICATOR COMPANY

Part #	Description	Quantity	Unit Price	Amount
For Casings on Soldier Piles				
51111	2.75" o.d. plastic casing in 10 ft lengths	110 ft (3)	$4.15	$ 456.50
51112	2.75" o.d. plastic couplings	10	1.50	15.00
51115	Protective top cap	2	2.55	5.10
51133-2	Pop rivets	100	.12	12.00
51134-1	Solvent cement, 1/2 pt.	1	2.50	2.50
51136	Grout plug, gasket seal type, for 2.75" plastic casing	2	62.00	124.00
For Casings in Soil				
57511	2.75" o.d. CPI casing in 10 ft lengths	220 ft (3)	4.50	990.00
57512	3.07 o.d. CPI couplings	20	7.50	150.00
57515	CPI bottom cap	4	9.00	36.00
51115	Protective top cap	4	2.55	10.20
57517	Shear wire	50	.25	12.50
57540	'O' ring lubricant	1	2.25	2.25
TOTAL AMOUNT				$1,816.05

Notes

1. Assumes use of LETCO owned Digitilt, cable, indicator, pulley assembly and cable hold.

2. Assumes Georgia DOT already owns assembly tools for standard 2.75" o.d. plastic casing.

3. Assumes bottom of casing is 10 ft below bottom of soldier pile and that one 10 ft length is cut and used for the top length of each installation.

4. Assumes Georgia DOT will plan and provide top protective arrangements.

TABLE 3

LOAD CELL EQUIPMENT

Recommended Supplier: Brewer Engineering Laboratories, Inc.
P.O. Box 288
Marion, MA 02738
(617) 748-0103

ATTN: Mr. Leon Weymouth (or)
Mr. Verne Wallace

Description	Quantity	Unit Price*	Amount*
Load cell, including calibrations, 2 bearing washers and spherical bearing plate	4	$2,690	$10,760
Vishay P-350A strain indicator with connector and 10 ft jumper cable	1	1,195	1,195
Vishay gage installation tester 1300	1	695	695
Gage and calibrate pull-bar supplied by others. Includes cable and connector but excludes any necessary end fittings on pull-bar required during calibration (i.e., assumes they will be provided with pull-bar)	1	1,900	1,900
Instruction manual and data sheets	L.S.	500	500
TOTAL AMOUNT (excluding freight)			$15,050

*Prices for budgeting purposes only. These are not a formal quotation. Prior to obtaining a formal quotation a specification needs to be written, as per the recommendations in this letter. Note that delivery is 6 to 8 weeks, hence arrangements need to be finalized as soon as possible.

TABLE 4

ROD TELLTALE EQUIPMENT

Recommended Supplier: Geokon Inc.
7 Central Avenue
West Lebanon, NH 03784
(603) 298-5064

ATTN: Dr. J. Barrie Sellers

Description	Quantity	Unit Price($)	Amount ($)
1/4" stainless steel flush coupled rod (various lengths)	360 ft	2.00	720.00
1/4" Sch. 80 PVC pipe in 20 ft lengths	240 ft	.50	120.00
Ditto, w/flush threaded couplings	180 ft	2.00	360.00
1/4" PVC pipe caps	16	1.50	24.00
PVC cement, brushes	1 pt	10.00	10.00
Telltale anchor with male and female disconnect, and clamps	16	40.00	640.00
Anchor head reference surface for center hole, w/telltale number	8	15.00	120.00
Ditto for outside hole	8	15.00	120.00
S.S. chair for in-situ calibration	2	10.00	20.00
Ditto for reading	2	10.00	20.00
Micrometer for in-situ calibration	1	165.00	165.00
Ditto for reading	1	165.00	165.00
Oil and filling equipment	L.S.	Included on Table 5	
Installation tools	L.S.	100.00	100.00
Miscellaneous materials	L.S.	250.00	250.00
Engineering time	2 days	350.00	700.00
TOTAL AMOUNT, EXCLUDING FREIGHT*			$3,534.00

*Prices for budgeting purposes only.

TABLE 5

WIRE TELLTALE EQUIPMENT

Recommended Supplier: Geokon Inc.
7 Central Avenue
West Lebanon, NH 03784
(603) 298-5064

ATTN: Dr. J. Barrie Sellers

Description	Quantity	Unit Price ($)	Amount ($)
S.S. anchor, with tube fitting and clamps	20	30.00	600.00
1/4" nylon LM tubing	1,000 ft	.25	250.00
18 SWG steel wire	750 ft	.10	75.00
Tube plugs	20	1.00	20.00
Oil pump and filling equipment (rental)	3 months	50.00	150.00
Tube manifold for pressurizing tubes	1 set	150.00	150.00
Wire straightener (rental)	3 months	50.00	150.00
Mark 1A indicator in carrying case**	1	2,100.00	2,100.00
Mouth-of-hole station**	4	250.00	1,000.00
Adaptation of mouth-of-hole station to create chair for indicator	4	100.00	400.00
Extensometer standardizer in carrying case**	1	800.00	800.00
Wire collar**	40	3.00	120.00
3/4" PVC pipe, with slots and couplings	180 ft	2.00	360.00
Installation tools (including router for cutting slots in 3/4" PVC pipe)	L.S.	200.00	200.00
Miscellaneous materials	L.S.	250.00	250.00
Engineering time	3 days	350.00	1,050.00
TOTAL AMOUNT, EXCLUDING FREIGHT*			$7,675.00

*Prices for budgeting purposes only.

**Materials available from Mining Department, University of Newcastle Upon Tyne, England, ATTN: Dr. Kenneth Dunham. Office tel. (44)-632-328511; home tel. (44)-434-603117.

Note: Six week delivery time, hence an order should be placed as soon as possible.

TABLE 6

MATERIALS BUDGET

Description	Refer to Table	Supplier	Amount
Inclinometers	2	Slope Indicator Co.	$ 1,816.00
Load cells	3	Brewer Engrg. Lab	15,050.00
Rod telltales	4	Geokon	3,534.00
Wire telltales	5	Geokon	7,675.00
Freight	-		1,000.00
Miscellaneous	-	Miscellaneous	1,000.00
		SUB TOTAL	30,075.00
		10% Contingency	3,000.00
		TOTAL	$33,075.00

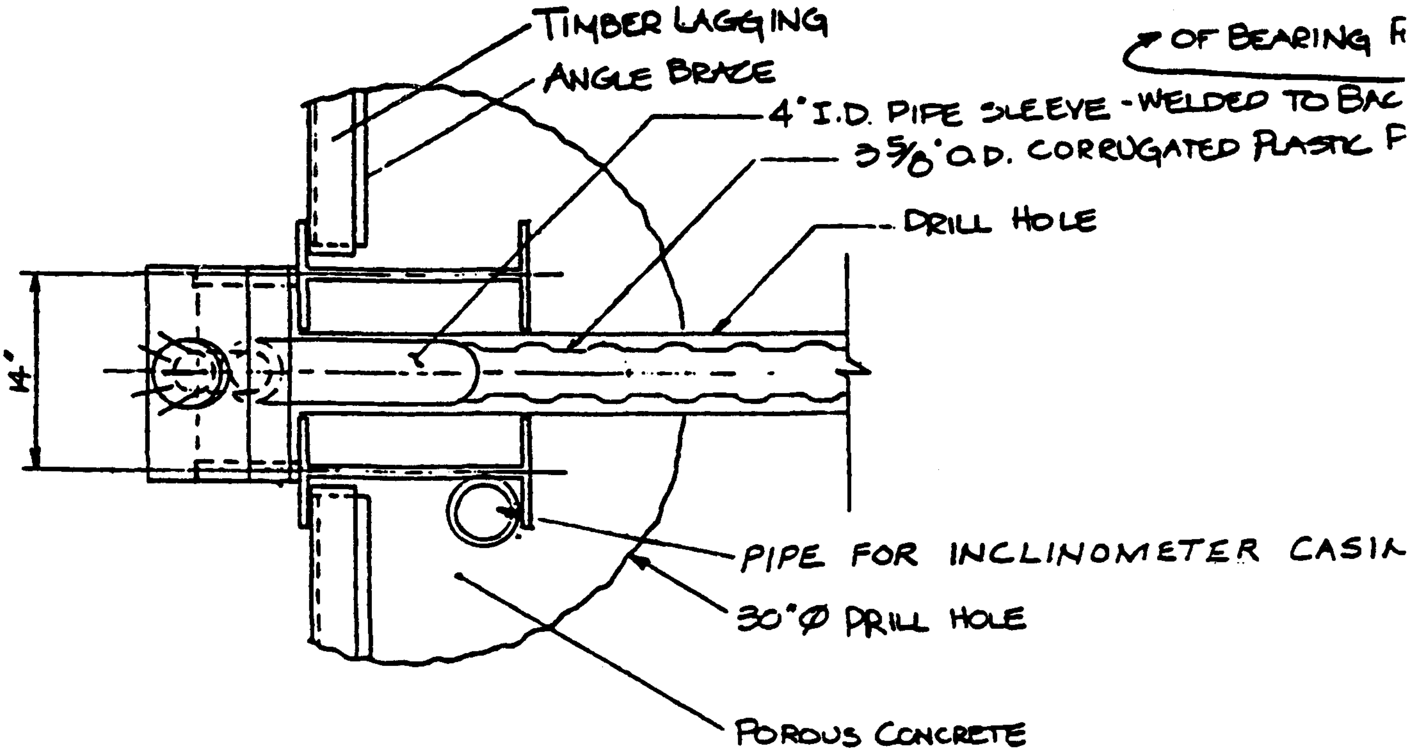


Fig. 1. Pipe For Inclinometer Casing on Soldier Pile

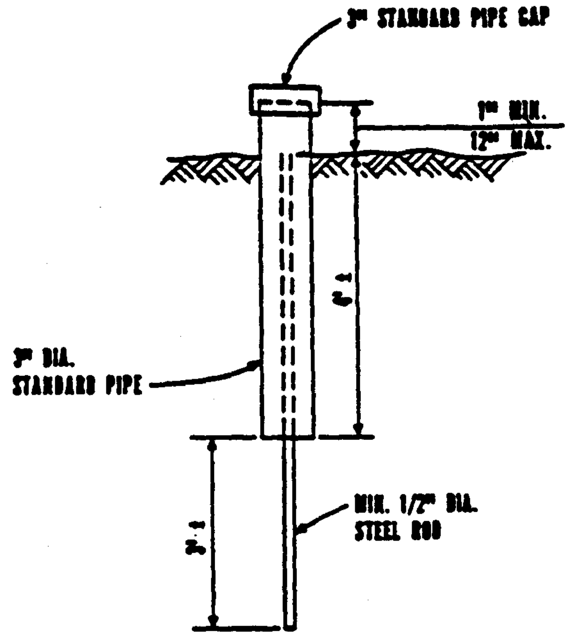
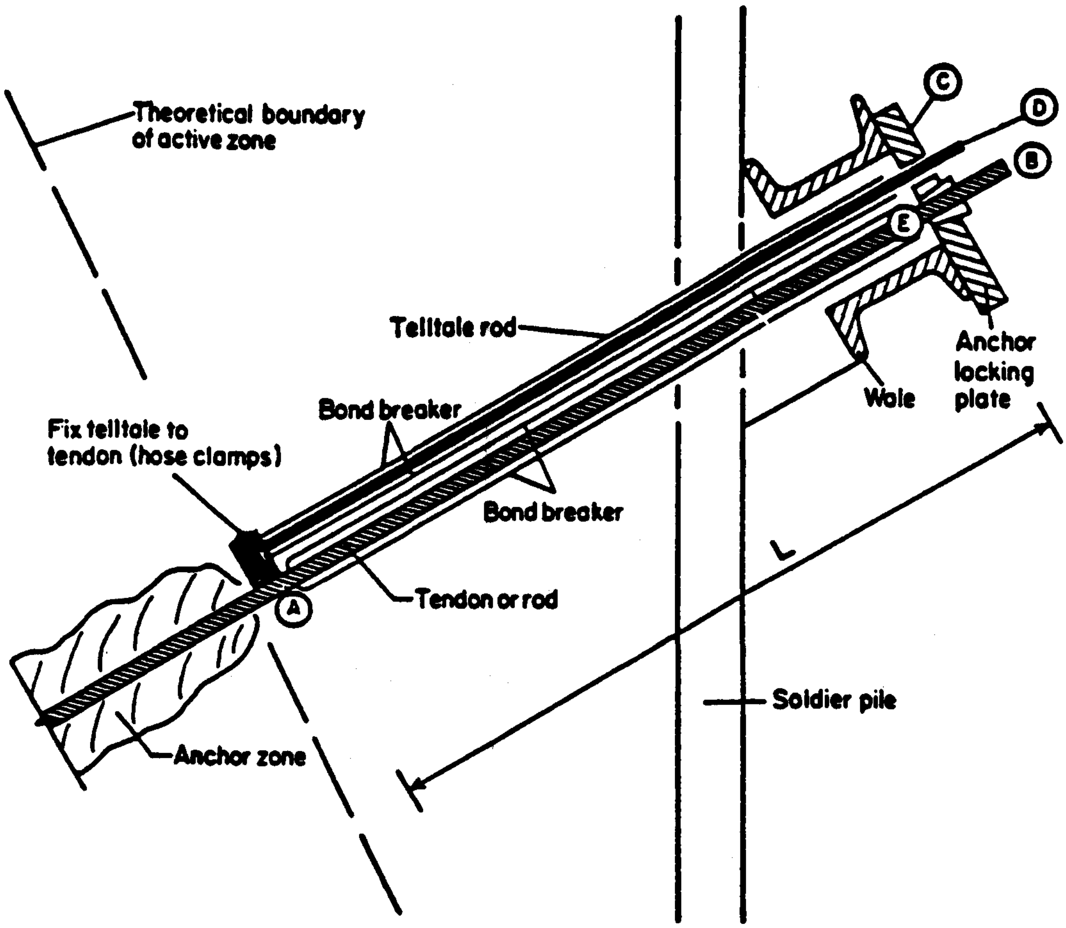


Fig. 2. Surface Settlement Marker (New York State)



Load in tendon at (A) and (E) = P

Change in load in tendon at (A) and (E) = ΔP

Change in length L = ΔL

(D) moves with (A). (E), (B) and (C) move together.

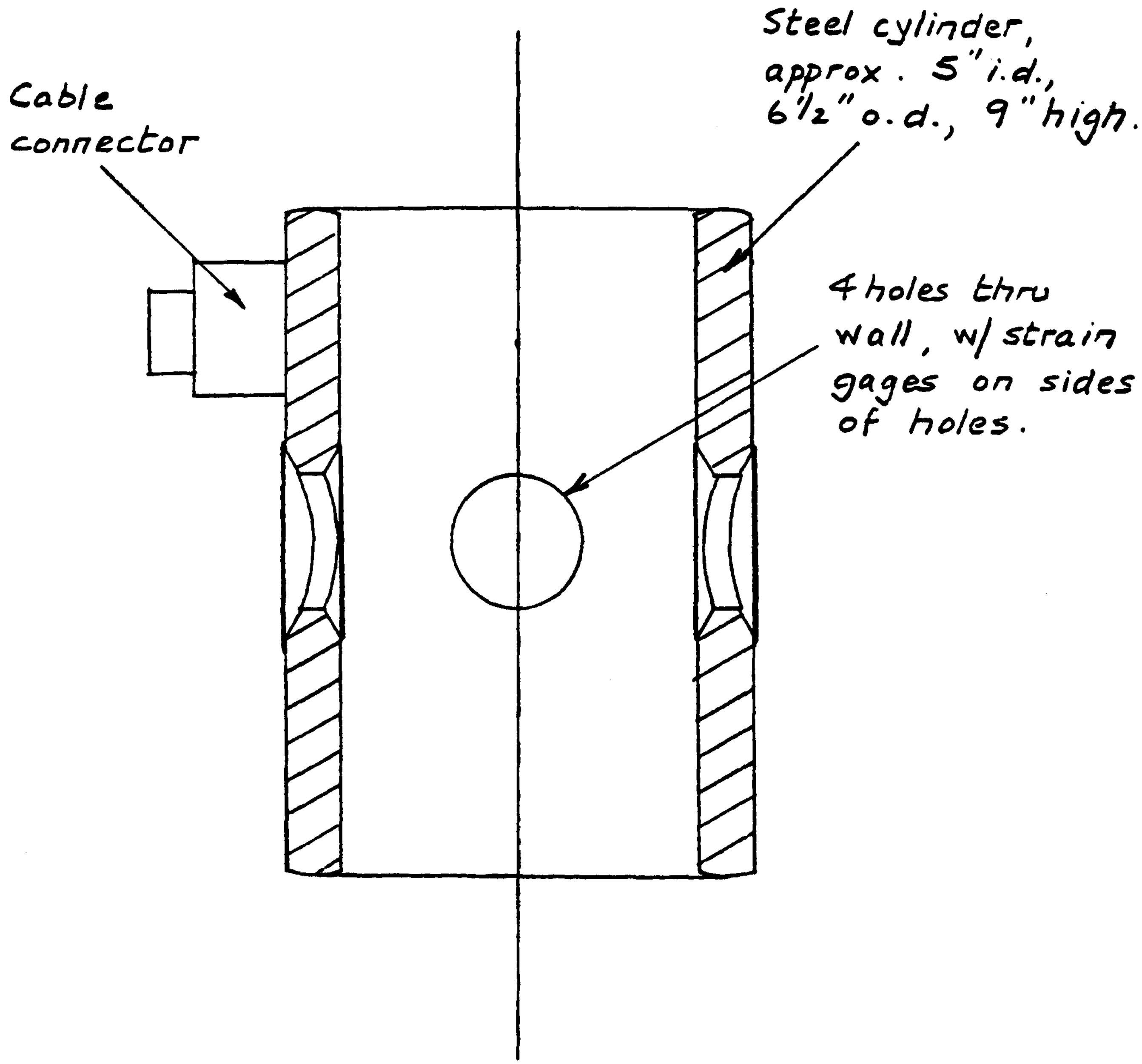
Therefore, change in "stick-out" distance (C) to (D) = ΔL

Hence if $\frac{L}{AE}$ is known, ΔP can be monitored by monitoring change in "stick-out" distance.

This is done with a portable dial micrometer. $P = \Delta L \frac{AE}{L} = \Delta LK$, where K is a constant obtained from field calibration.

Fig. 3. Schematic of Telltale Load Cell

John Dunnicliff, P.E. 11/4/81



Approx. half scale

Fig. 4. Load Cell

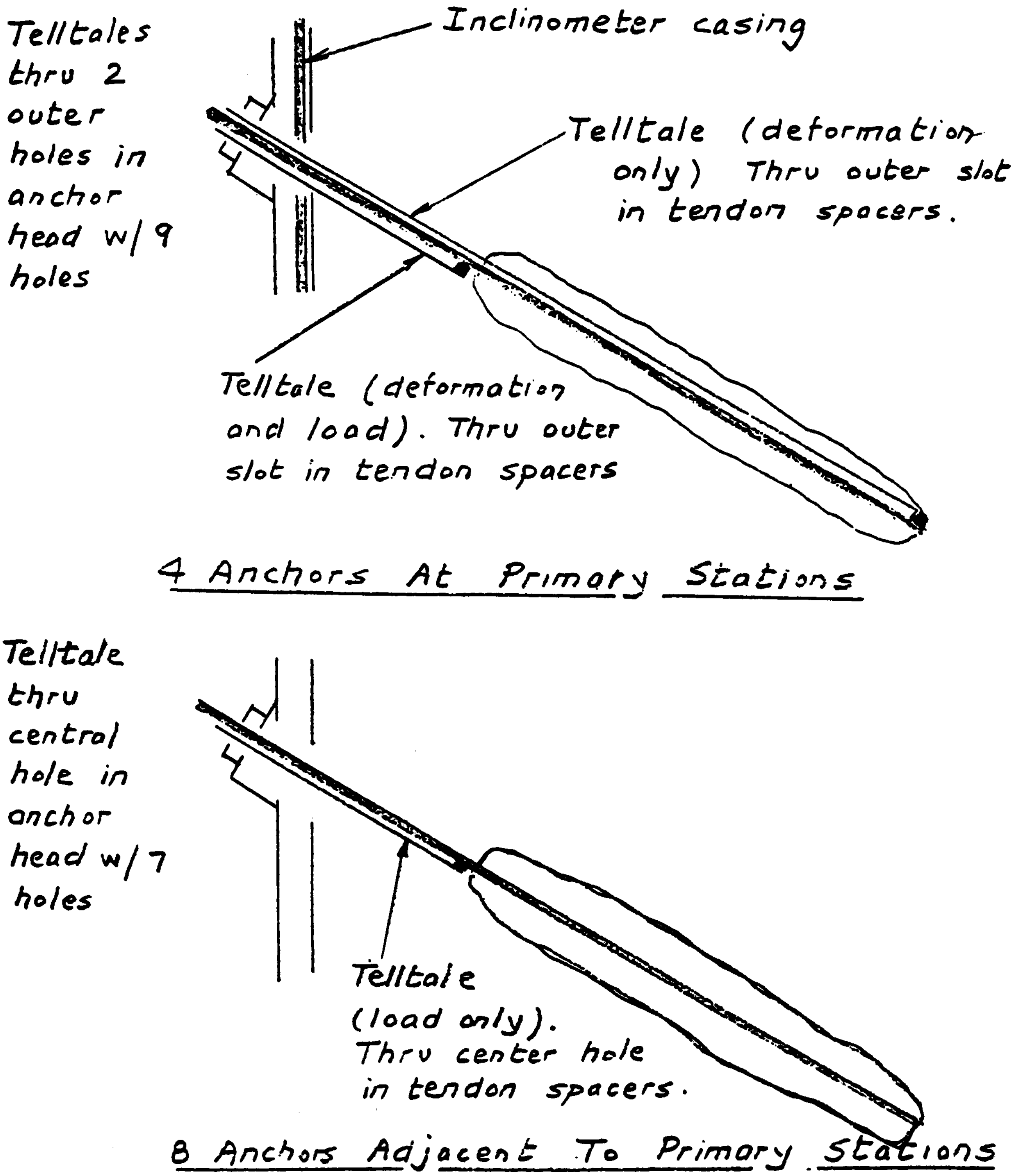


Fig. 5. Schematic of Rod Telltales

John Dunnicliff, P.E. 11/3/81

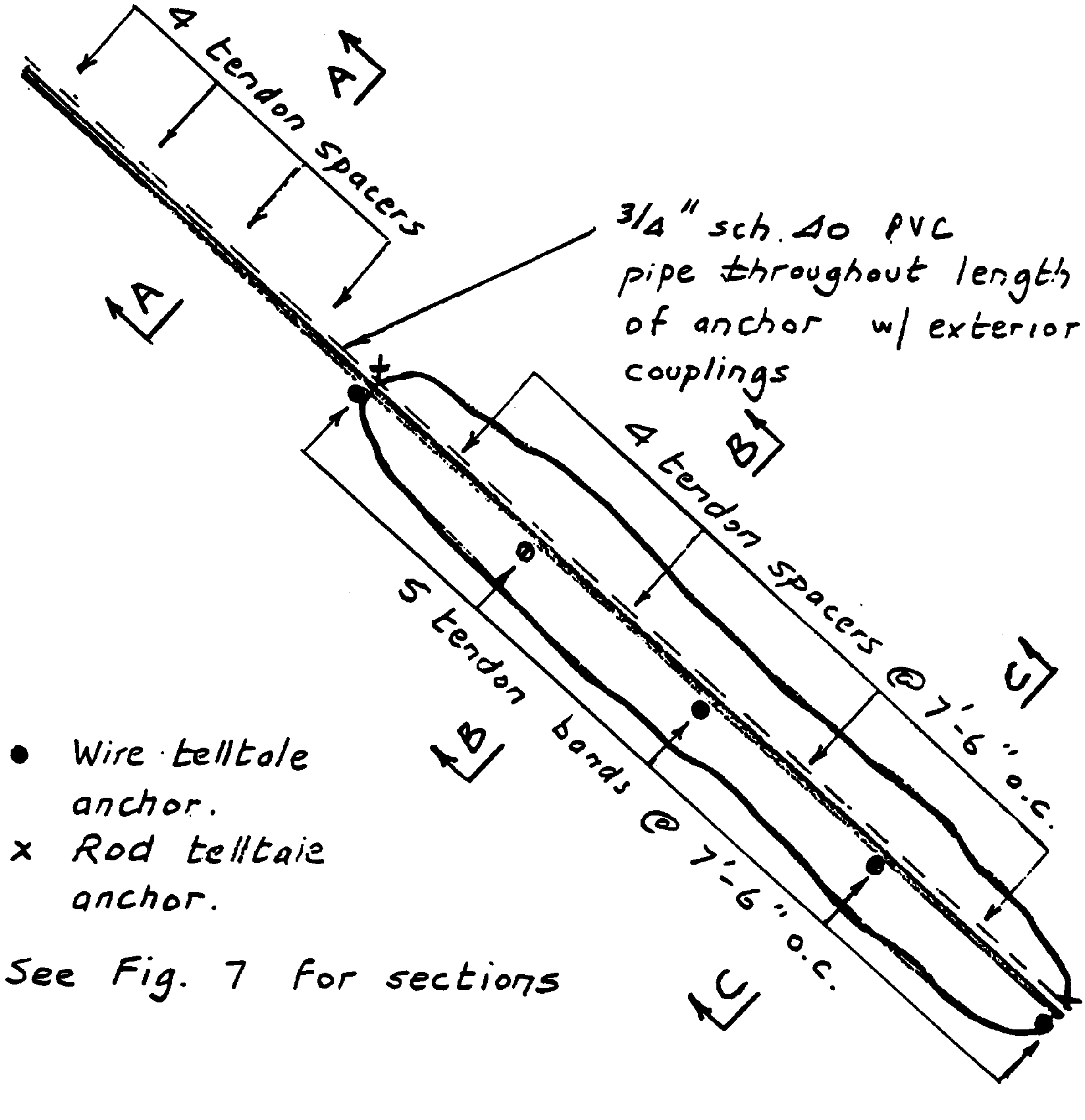


Fig. 6. Schematic of Rod and Wire Telltale Anchor Locations with Spacer and Banding Arrangements

Approx scale
1" = 5'

John Dunnicliff, P.E. 11/3/81

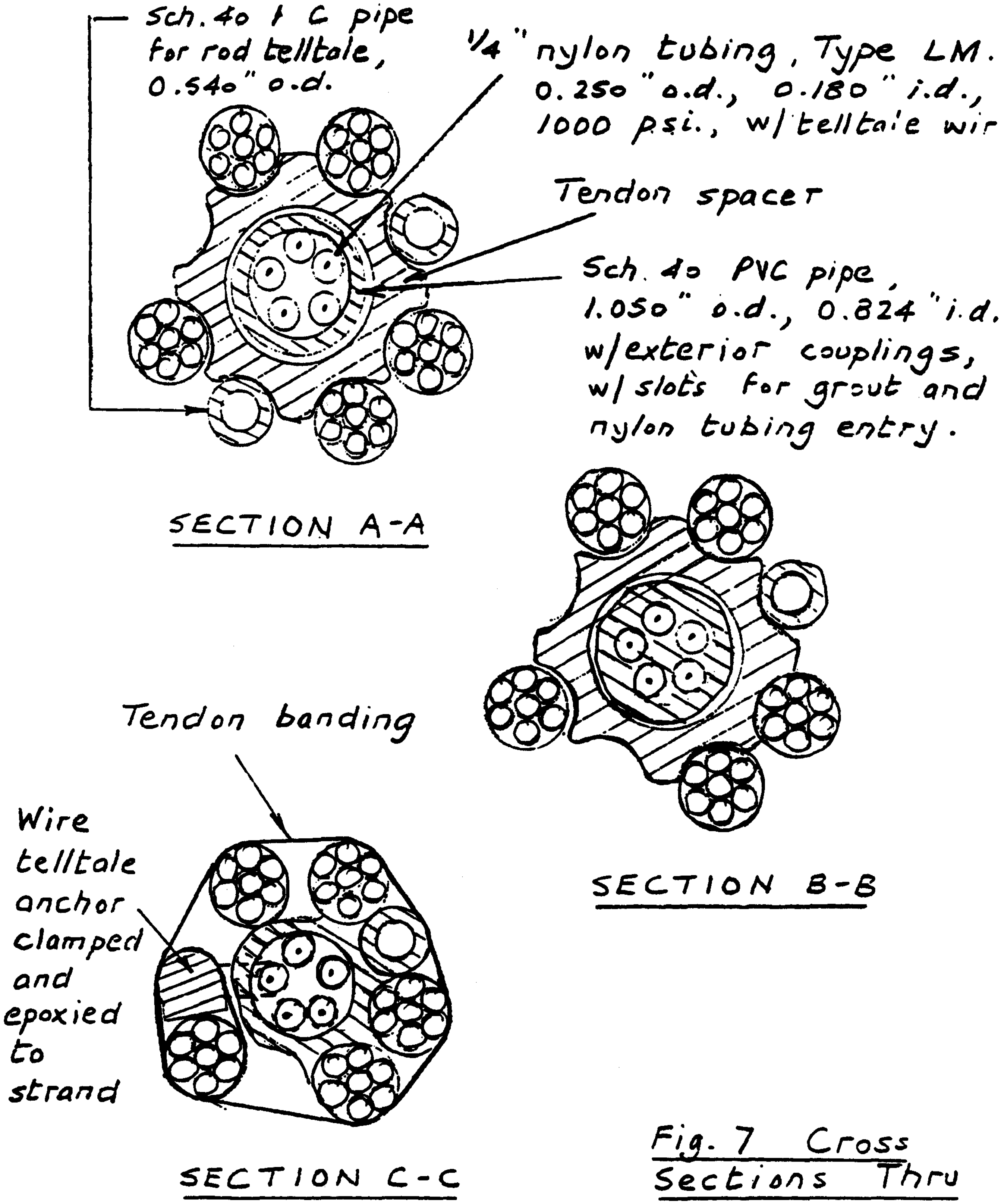


Fig. 7 Cross Sections Thru Telltales

Approx full size

Corrugated plastic pipe not shown.

John Dunnicliff, P.E. 11/3/81

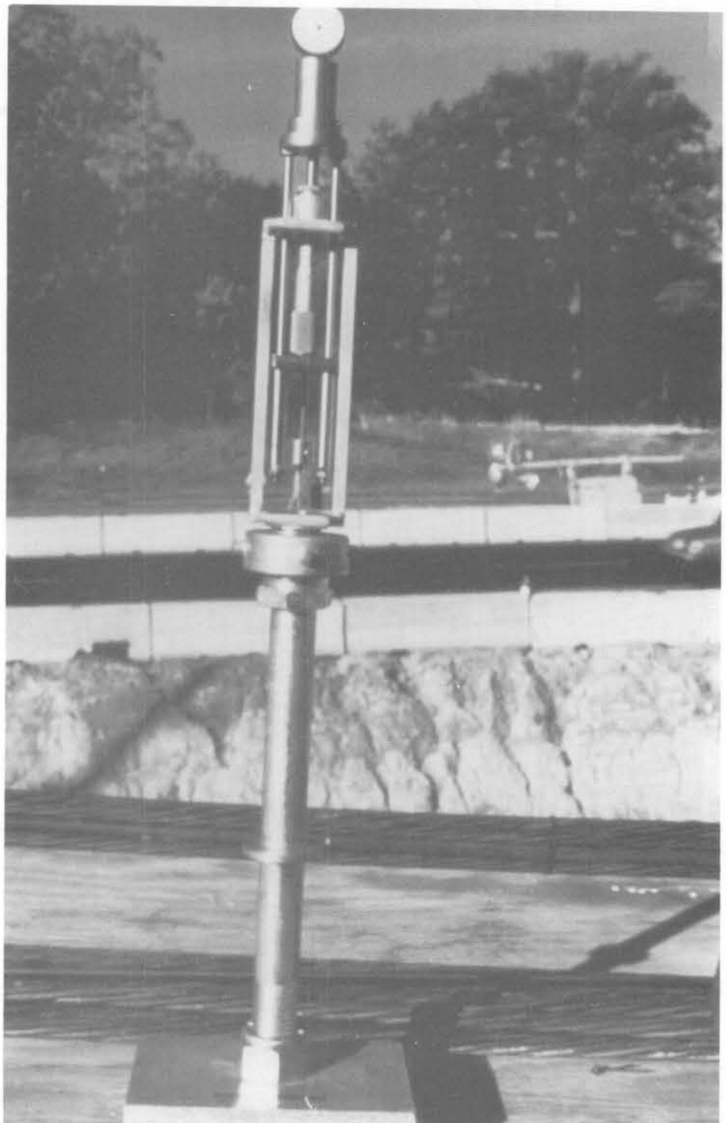


Figure 8. Wire Telltale Indicator

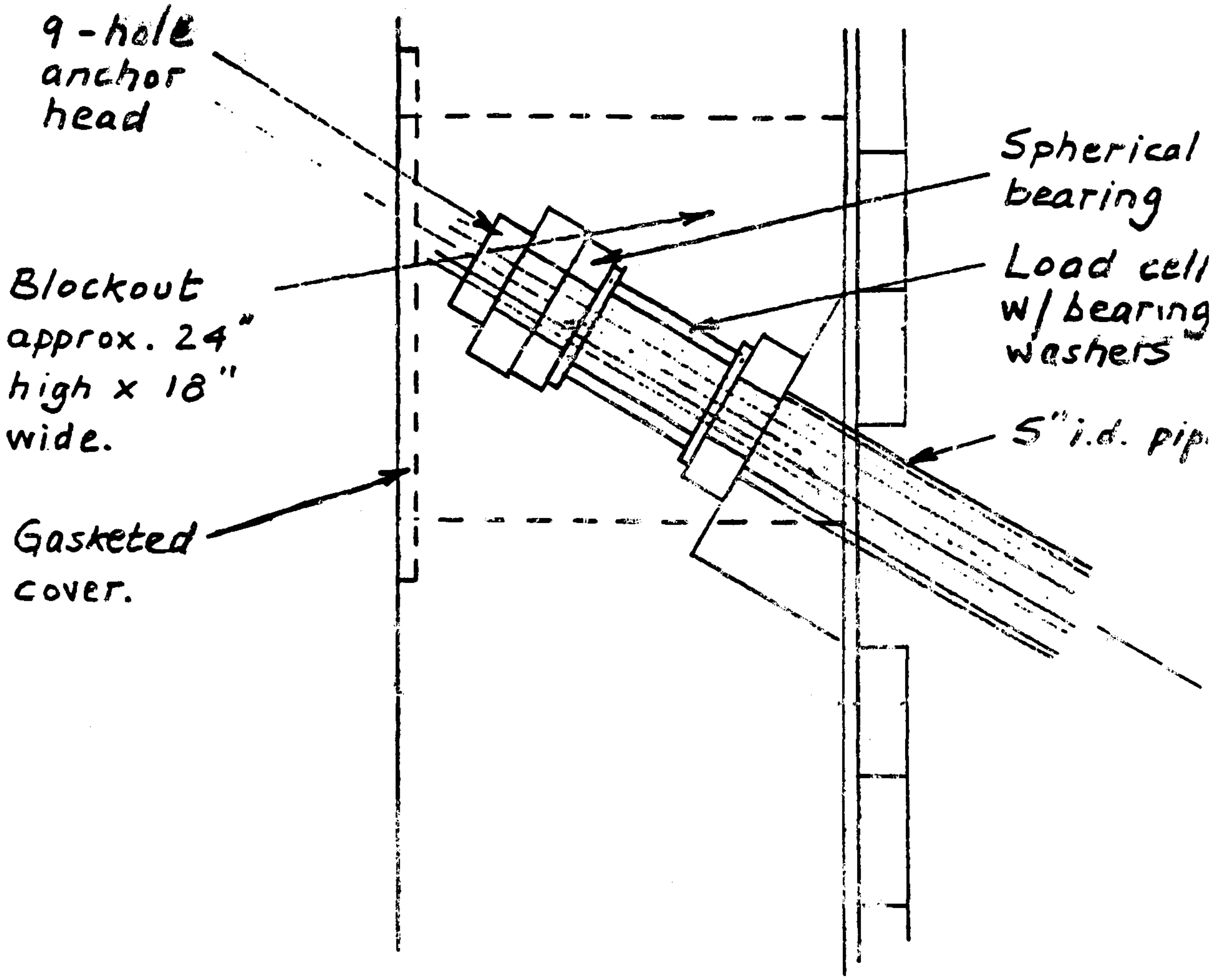


Fig. 9. Schematic of Blockout and Cover at 4 Locations

Approx scale 1/10 full size

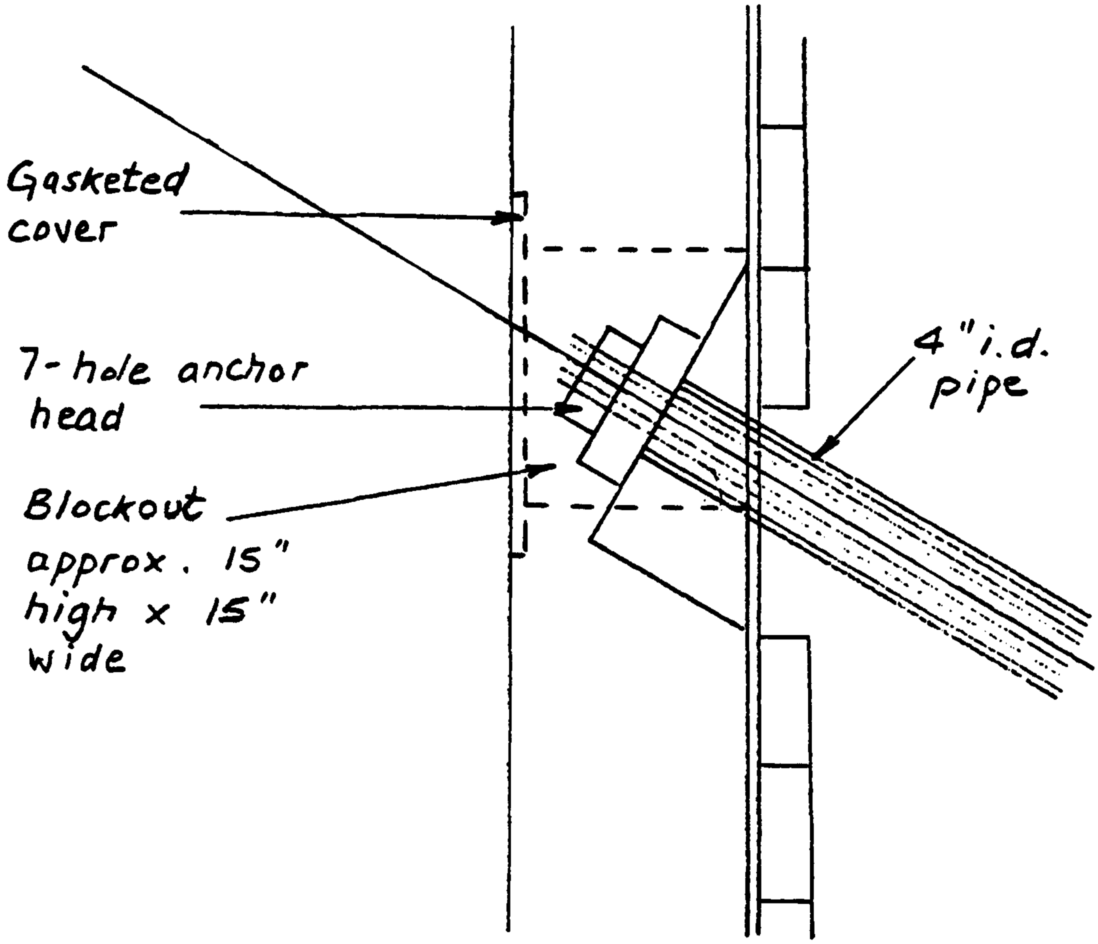


Fig. 10. Schematic of Blockout and Cover at 8 Locations

Approx. scale 1/10 full size

John Dunnicliff, P.E. 11/4/81

REPORT NO. 2

PERFORMANCE OF TWO INSTRUMENTED, PERMANENT POST-GROUTED TIEBACKS

THE MT. CARMEL CEMETERY RETAINING WALL
I-95 - BALTIMORE, MARYLAND

by

Harald P. Ludwig

Report submitted by Schnabel Foundation Company for:

Permanent Ground Anchor Demonstration Project
Federal-Aid Project No. I-95-4(118)57
State Highway Administration No. BC 246-123-815
Baltimore City No. 2641
April 1984

TABLE OF CONTENTS

MT. CARMEL CEMETERY
DEMONSTRATION TIEBACKS

I. Introduction

In January and February of 1982, Schnabel Foundation Company installed and tested two instrumented, permanent post-grouted tiebacks (designated as tieback 15-8 and tieback 18-2) in a stiff-to-very-stiff, red-brown, silty clay at Mt. Carmel Cemetery retaining wall located alongside Interstate 95, north of the Baltimore Harbor Tunnel in Maryland. The two instrumented tiebacks were to be incorporated into the overall plan of stabilizing the failed retaining wall with 98 tiebacks and 4439 linear feet of drains. The purpose of this demonstration project was to study both the short-term and long-term behavior of permanent tiebacks anchored in cohesive soils.

The short-term investigation consisted of conducting one-week long creep tests on each tieback in order to study their load-displacement-time behavior. The long-term investigation consisted of monitoring changes in tieback load, wall deflection, movement of the soil mass, and changes in water table level over a two-year period. The Maryland State Highway Administration Bureau of Soils and Foundations instrumented and monitored the retaining wall and soil mass, while Schnabel Foundation Company was responsible for the instrumentation and monitoring of the tiebacks.

For purposes or organization, the remainder of this report is divided into seven sections. Site conditions are described in Section II. The tieback description and installation procedure are given in Section III. Instrumentation of the tieback is discussed in Section IV, while the test program is described in Section V. Results of the test program and analysis of the performance of both tiebacks form the contents of Section VI. Finally, conclusions and recommendations for further study are given in Section VII and Section VIII, respectively.

II. Site Conditions

The two instrumented tiebacks are located approximately at Station 304+23 and Station 305+19. Cross sectional views of the retaining wall and instrumentation as well as soil profiles at these stations are shown in Figures 1 and 2. The soil profiles indicate that both tiebacks were installed in a stiff-to-very-stiff, red-brown, silty clay. Rebar spacing at the front face of the wall is 12 inches both horizontally and vertically. Rebar spacing at the back face of the wall varies from 6 to 12 inches horizontally and is 12 inches vertically.

III. Tieback Description and Installation Procedure

A. Tieback Description

It is prudent to mention some of the terminology that will be used in the ensuing description of the tieback and in the remainder of this study. Every tieback has an anchor length and an unbonded length. The anchor length is the designed length of the tieback where the tieback force is transferred to the soil. This part of the tieback is commonly referred to as the anchor. The unbonded length of the tieback is the length which is free to elongate elastically. Both tiebacks had an anchor length, la, of 30 feet and a minimum unbonded length, lu, of 27 feet.

Post-grouted tiebacks, known as TMD ("terrain meuble deferment") tiebacks developed by SIF Bachy of France, were used in this demonstration project because:

- they provide corrosion protection for the lifetime of the structure;
- they are capable of developing high capacities in cohesive soils;
- they are capable of being instrumented;
- the installation procedure causes minimal damage to the instrumentation;
- they were to be installed in small diameter holes in the wall; and
- installation equipment did not require a large construction easement along the wall.

The components of the TMD instrumented tiebacks are shown in Figure 3. The TMD tieback used in this study consisted of a 3-inch diameter deformed metal tube, which is grouted to the soil, a 3-1/2-inch diameter polyvinyl chloride (PVC) pipe which served as the bond breaker, and a 60-feet long, 1 3/8-inch diameter threaded Dywidag bar. The inflatable bag shown in Figure 3 is optional and was not used in this study. Its primary purpose is to allow the post-grouting operation to commence immediately after the TMD assembly is inserted in the hole. The threaded Dywidag bar, which is inserted in the deformed metal tube, had heat-shrink tubing and polyethylene bond breaker over its unbonded length.

B. Installation

The installation procedure consisted of four major operations (see Figure 4):

- o punch an 8-1/2-inch diameter hole through the wall;
- o drill hole through soil to desired depth and insert TMD assembly;
- o grout the deformed metal tube to the soil; and
- o grout tendon to the deformed metal tube.

An 8-1/2 inch diameter hole was punched through the wall by using an 8-inch diameter down-the-hole hammer with a button bit attached to its end. Upon encountering the first section of rebar 2-1/2 inches from the front face of the wall, a laborer used a torch to burn off the rebar. The second section of rebar, located near the back face of the wall, was burned out with a lance.

Once the hole had been punched through the wall, the hole was drilled to a final depth of 57 feet using a 6-inch diameter auger with a clay bit attached to its end. From the experience gained during the installation of the 96 non-instrumented tiebacks, it had been decided the hole could be drilled uncased. This greatly simplified the drilling operation. Very soft, soupy soil with running sand was encountered while drilling the hole for tieback 18-2. Fortunately, this presented no problems during the installation of the tieback.

Once the auger had been extracted from the drill hole, the deformed metal tube, with grout valves located every 3.3 feet along its length, and the PVC pipe were placed in the hole. The assembly was centered in the hole by means of plastic centralizers attached to the deformed metal tube. After the TMD assembly had been inserted in the drill hole, a 3-inch diameter double packer was positioned inside the deformed metal tube opposite the bottom grout valve. The packer was inflated and grout subsequently was pumped through the last valve until the grout could be seen exiting from the drill hole. The packer was removed and the inside of the TMD assembly was flushed with water and blown out with compressed air until the effluent was fairly clear. The grout was then allowed to set up overnight.

The post-grouting operation began on the next day. The packer was positioned inside the deformed metal tube opposite the bottom grout valve and inflated. Grout was pumped slowly through the valve until either a maximum grout pressure of 40 bars was reached or one bag of cement had been pumped through the valve. The quantity of grout pumped through each valve was limited, because

REPORT NO. 3

PERMANENT TIEBACK ANCHORS IN COHESIVE SOIL

DEMONSTRATION SITE
SR-90 (SR-5 to CORWIN PLACE)
SEATTLE, WASHINGTON

by

Hart Crowser and Associates, Inc.

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION
MARCH 1984

- a calibrated 150-ton capacity load cell to monitor load at the anchorhead during the short-term investigation;
- a calibrated test pump and jack to load the tieback;
- two dial gauges to monitor movement of the anchorhead relative to a fixed point to the nearest .001 inch;
- a Vishay 220 digital strain readout unit capable of reading and recording the response of 30 strain gauges continuously at a rate of one strain gauge per second;
- a Vishay P-350A strain indicator to monitor load in the load cell; and
- a dummy gauge used to zero the Vishay 220 digital strain readout unit prior to each set of readings over the two-year monitoring period.

In general, the strain gauges performed quite well during the initial two-week test program. Strain gauge failure occurred in less than 17 percent of the strain gauges. However, the performance of the same strain gauges over the two-year monitoring period was less satisfactory. This type of performance is to be expected from electrical resistance strain gauges which are very accurate but, due to electrical aging of components and creep in bonding agents, tend to "drift" with time. Prolonged exposure to a hostile environment over a period of time was probably the reason for the majority of the strain gauge failures.

Since the original intention of the two-year monitoring program was to monitor changes in tieback load, the strain gauges of primary interest were those located in the unbonded length of each tieback. Fortunately, 8 out of the 10 strain gauges located in the unbonded length functioned properly for the entire two-year monitoring period. Therefore, the two-year load change data presented in Section VI is considered reliable and accurate provided that certain limitations associated with the use of strain gauges are recognized. First, the tendency of electrical resistance strain gauges to "drift"" with time influences the absolute magnitude of load measured more so than the trend in load change. Both the strain gauges installed on the two tie backs and the dummy gauge used to zero the Vishay 220 unit were affected by this "drift" phenomenon. Second, the variation in Young's modulus of elasticity of the bar required to convert strain to load introduces error to the absolute magnitude of load measured. The conversion of strain to load required the use of Equation (1):

$$P = eEA \qquad \ldots(1)$$

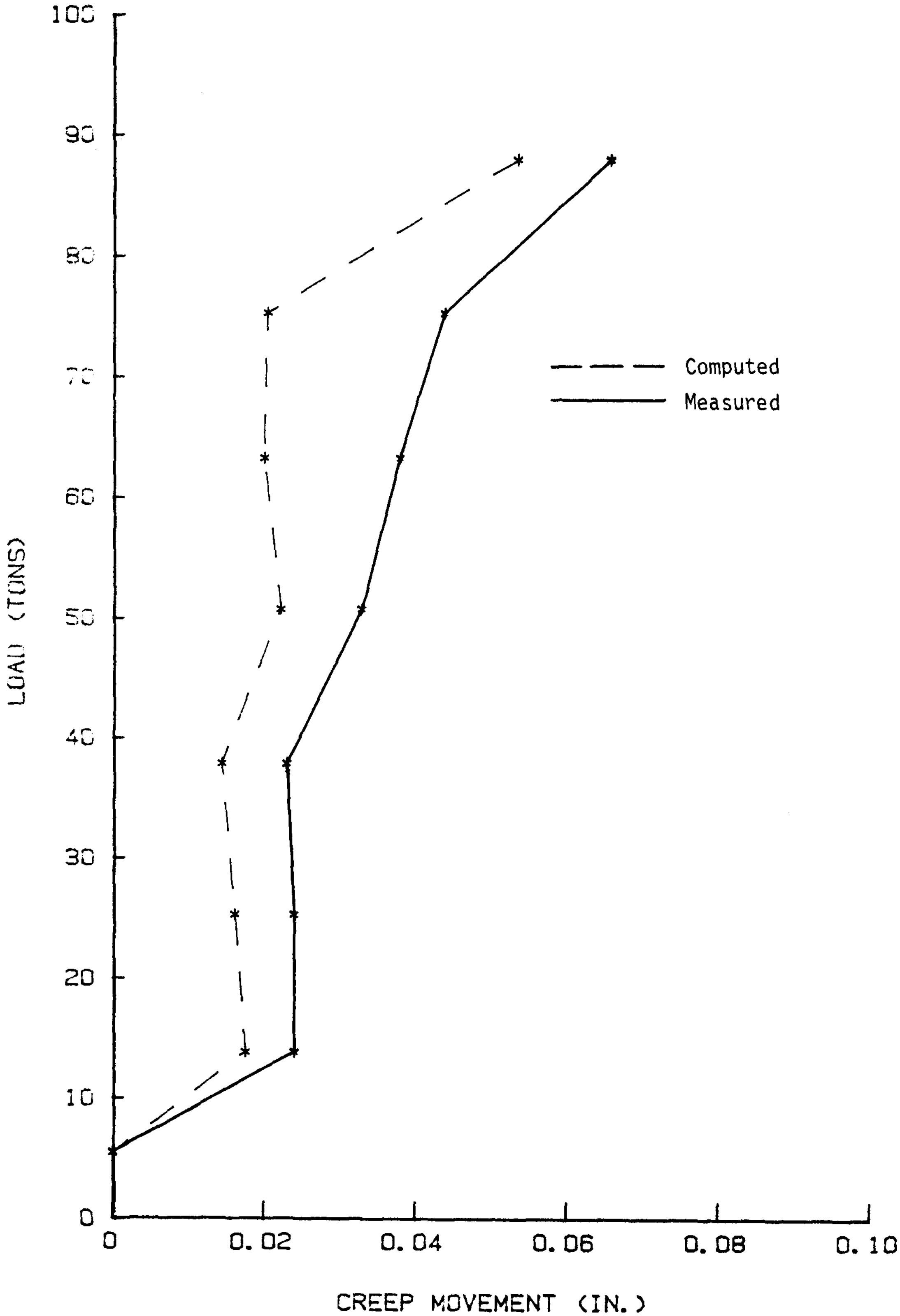
where P = load

e = strain ($\times 10^{-6}$)

E = Young's modulus of elasticity

A = cross-sectional area of the tendon

Figure 31. Measured Versus Computed Creep Movement of Tieback 18-2



In setting up each tieback test, extreme care was exercised to ensure that the bearing plates, load cell, and jack were aligned properly. This was done in order to minimize the effects of friction and eccentric loading due to mis-alignment. Prior to the start of each test, each strain gauge was calibrated internally and initialized to give a "zero" reading to which all subsequent strain gauge readings were referenced. A 5.5-ton alignment load was then placed on the tieback to seat and align the testing hardware. At this time, the dial gauges were set up to measure movement of the anchorhead. All subsequent dial gauge measurements were referenced to the initial "zero" reading at 5.5 tons. The test proceeded by setting the desired load on the Vishay P-350A strain indicator. The jack was then pumped up to the desired load. At this time, the dial gauge and all strain gauges were read. If the next load was to be held constant, the jack was pumped up to the desired load within one minute. An initial reading was taken when the desired load was first reached. The one-minute reading of the load-hold corresponded to the time that had elapsed since the pump was first activated. That is, the time lapse between the initial "zero" reading and one-minute reading of the load-hold was always less than 60 seconds. During the load-hold, it was necessary to adjust the hydraulic pressure in the test jack in order to maintain the load constant. Load was decreased to the alignment load after each load-hold by setting the Vishay P-350A strain indicator to the alignment load and opening up the release valve on the pump. At the end of each test, the load was reduced to zero and all strain gauges were read and checked for internal calibration. Subsequently, each tieback was locked-off at 40 tons by reloading the tieback to 40 tons, tightening a nut against the bearing plate, and releasing the load from the jack.

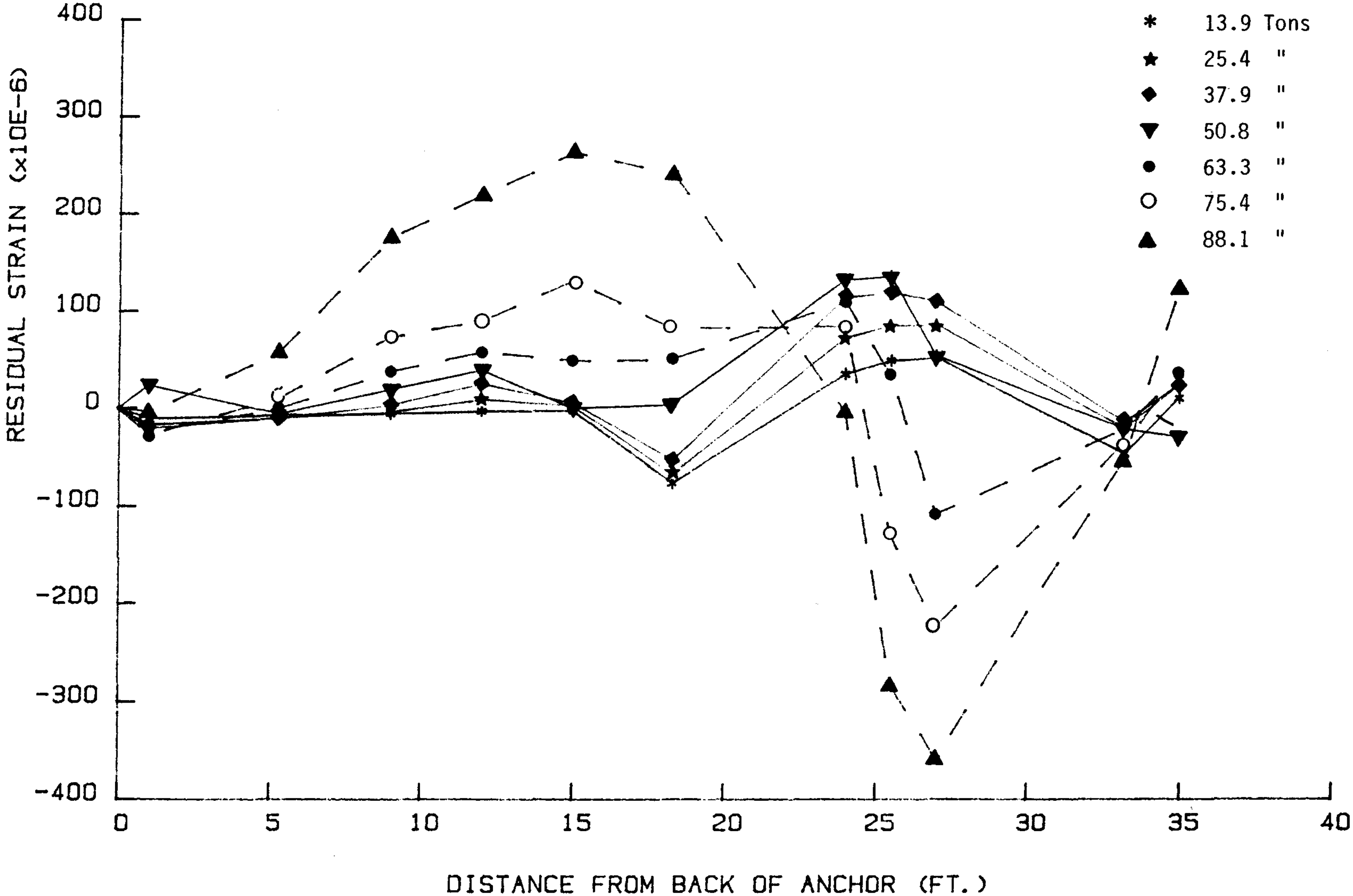
B. Two-year Monitoring Program

The two-year monitoring program consisted of monitoring the change in load in each of the tiebacks at least once every three months during this period of time. The strain gauge readings taken immediately after each of the tiebacks was locked-off at 40 tons served as the initial zero to which all subsequent readings were compared.

The procedure used to take a set of readings is described below. The dummy gauge was used to zero each of the 30 channels on the Vishay 220 readout unit prior to visiting the site. At the site, the dummy gauge was used again to re-zero the 30 channels. Subsequently, the strain gauges on both instrumented tiebacks were read within a period of 60 minutes.

Note that the strain gauge readings taken in June of 1982 are omitted from this report because they were considered unreliable. The reason for their un-reliability was traced to the failure to pot the military connectors which served as the junction between the strain gauges and the Vishay 220 readout unit. Grouting of

Figure 29. Distribution of Residual Strain in the Deformed Metal Tube of Tieback 18-2.



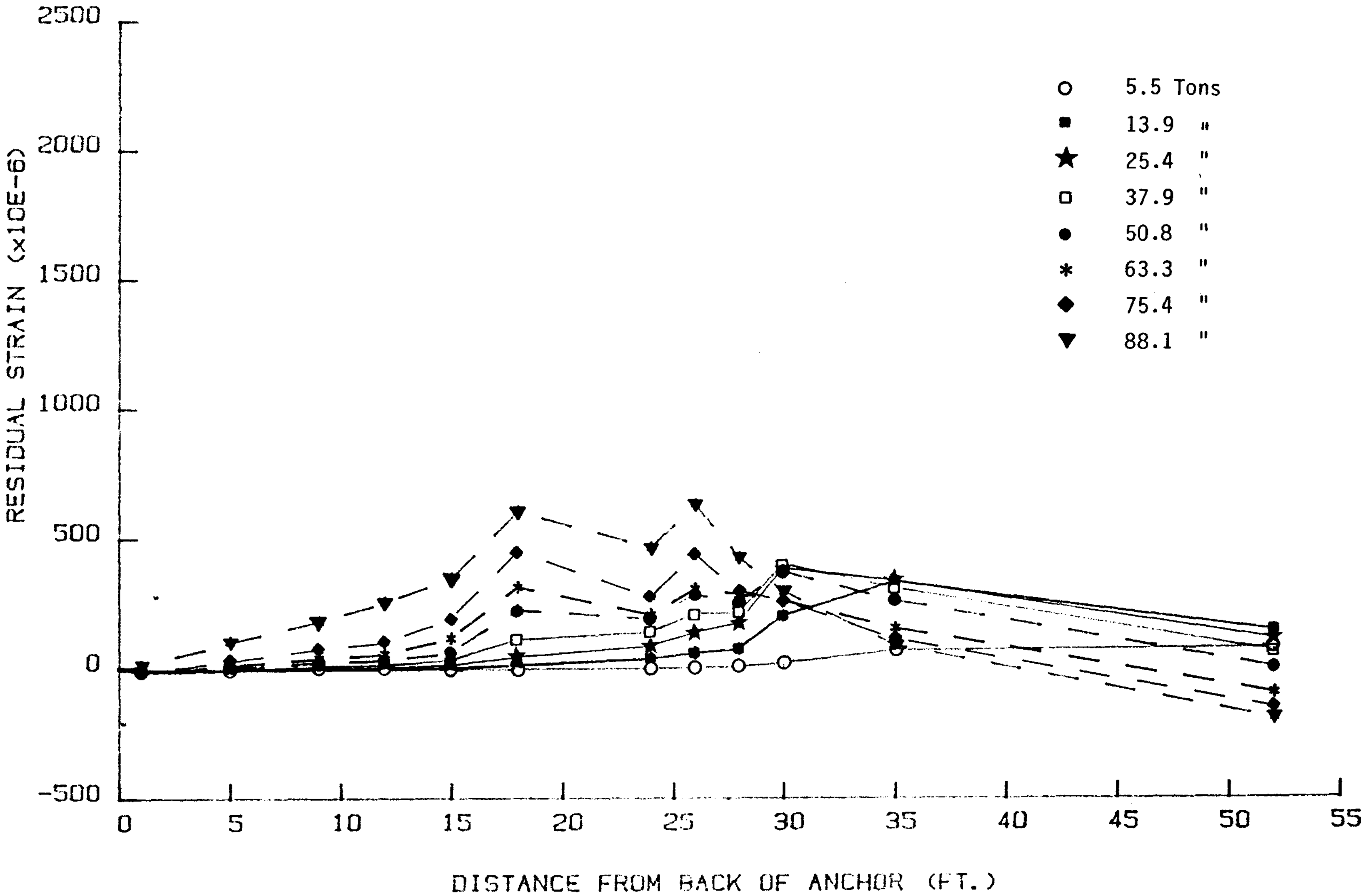
Residual anchor movement is a very useful measure of tieback performance, since it is a function of both the tieback construction technique and the soil in which the tieback is anchored. Reese (1976) highlighted the fact that the displacements required to mobilize ultimate capacity in bearing are much larger than the displacements required to mobilize ultimate capacity in shear in most soils. A large number of tests conducted on tiebacks using extensometers have verified that the anchor moves when loaded and partially rebounds when unloaded, and that the residual anchor movement can be measured reliably in a cyclic test such as the performance test (Schnabel, 1982). Since larger anchor displacements are required to mobilize greater tieback capacity (for loads less than ultimate), residual anchor movement tends to increase as the applied load increases. To date, no acceptance criteria exists to evaluate tieback performance based on residual anchor movement.

Time-dependent tieback displacements at constant load are used to evaluate the long-term performance of a tieback. The time-dependent or creep movements typically are plotted on a semi-logarithmic scale as a function of time (see Figure 8). For each load, the creep movement per logarithmic cycle (or creep rate) is determined by computing the slope of a best fit straight line drawn through the data points. If a straight line cannot be drawn, the creep rate is determined by computing the maximum slope formed by the data points. Creep rate can then be plotted as a function of load. To date, tieback specialists cannot agree on the acceptance criteria to be used to evaluate tieback performance based on time-dependent movements.

The preceding discussion has focused its attention on the measurements used by the engineering profession to evaluate a tieback's performance. Elastic movements, residual anchor movements, and time-dependent movements are used to assess both the short-term and long-term load carrying capability of a tieback. The ensuing discussion presents a step-by-step description of the analysis used herein to evaluate a tieback's performance based on computed performance which can provide insight into the measured performance.

The most obvious contribution of the computed performance of each tieback is the determination of the distribution of strain, and therefore load[1], along the tieback for a given applied load and a given time (see Figure 9). The shaded area represents the total amount of strain that developed during a particular load-hold. The strain distribution in the unbonded length can verify easily the provision of a minimum desired unbonded length--a criteria used by the entire engineering profession to evaluate a tieback's performance. The installation of strain gauges at critical locations along the unbonded length, e.g., outside and inside the face of the tied back structure and at the transition from the unbonded length to the anchor length, can provide valuable information concerning the influence of friction and bending on the distribution of strain in the unbonded length. Typically the

Figure 27. Distribution of Residual Strain in the Tendon of Tieback 18-2

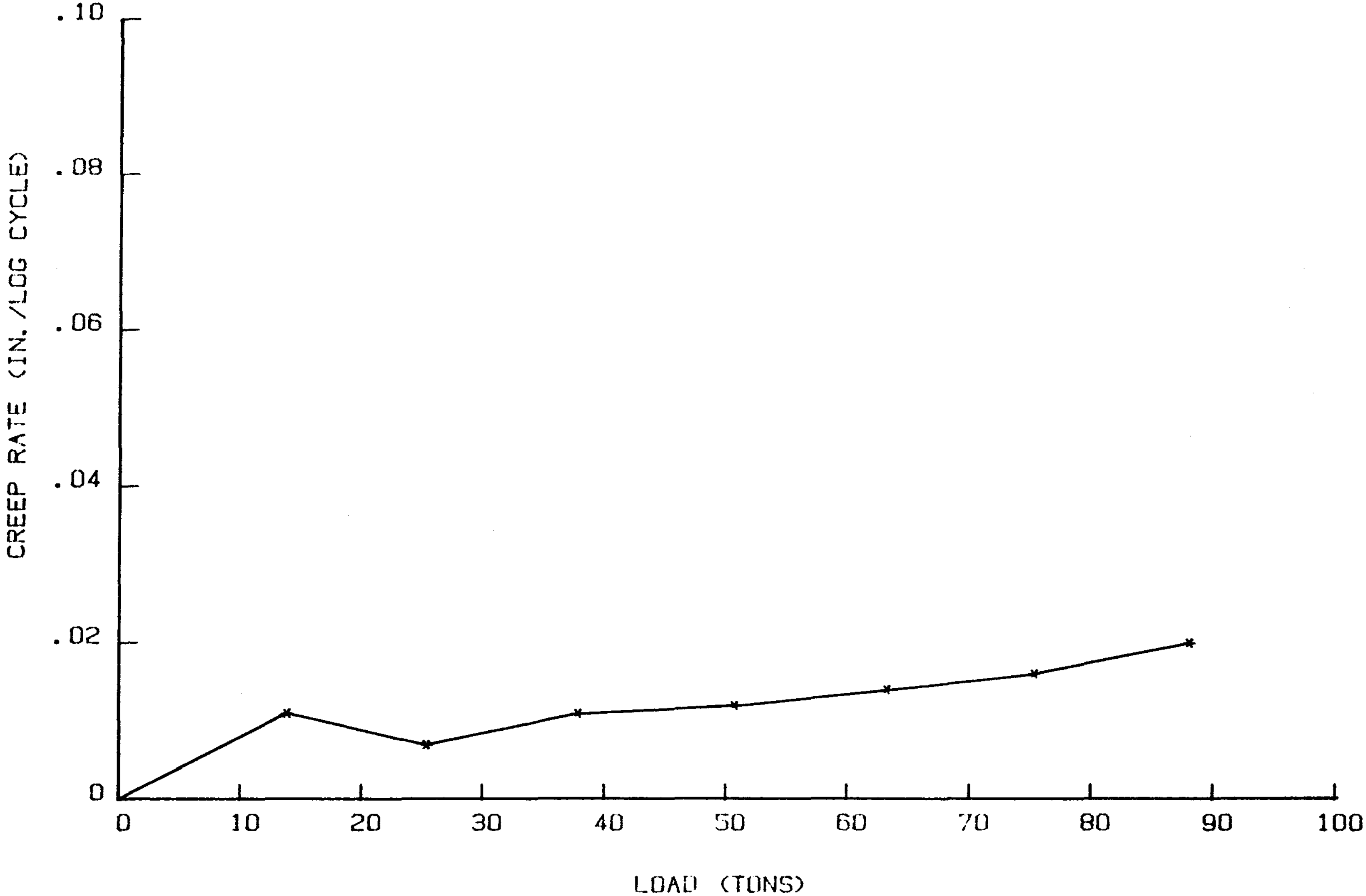


Schnabel Foundation Company on instrumented straight-shafted tiebacks anchored in clay have shown that tieback failure is usually associated with residual anchor movements greater than 0.5 inches. The load-displacement response of tieback 15-8 (see Figure 10) was much more elastic than that of the straight-shafted tieback (see Figure 7). Both the elastic load-displacement response and the relatively small residual anchor movements of tieback 15-8 indicate that additional capacity existed beyond 88.1 tons.

The time-dependent response of tieback 15-8 is presented in Figures 12 and 13. At the maximum test load of 88.1 tons, the creep rate was essentially constant at .043 inches/logarithmic cycle in both the third logarithmic cycle (100 to 1,000 minutes) and the fourth logarithmic cycle (1,000 to 10,000 minutes). However, this creep rate was double the creep rate that occurred in the second logarithmic cycle (10 to 100 minutes). The change in creep rate from the second to the third logarithmic cycle has a plausible explanation. In jacking the load up to the final test load of 88.1 tons, the load was inadvertently maintained constant for a period of four minutes at a load of 86.4 tons. Once the error was discovered, the load was immediately jacked up to 88.1 tons and thereafter maintained constant. It is apparent that the creep movement of the tieback at 86.4 tons, and the effect of the previous loads, influenced the creep performance of the tieback at a load of 88.1 tons by reducing the amount of creep that occurred in the first two logarithmic cycles. The influence of load history on the time-dependent performance of tiebacks anchored in cohesive soils is been noted in other tests on instrumented and non-instrumented tiebacks.

The creep performance of tieback 15-8 showed no signs of imminent failure. The maximum creep rate of 0.043 inches/logarithmic cycle is relatively small for a tieback anchored in cohesive soil. Even at the maximum test load of 88.1 tons, the creep behavior was essentially linear with respect to the logarithm of time after the second logarithmic cycle. Temperature changes can affect the dial gauge support, tendon, and hydraulic fluid in the test jack. Temperature-induced movements may become significant when anchor creep is small. Fluctuations in creep movement observed during the fourth logarithmic cycle at the 88.1 ton load increment are attributable primarily to temperature changes between day and night. While trying to maintain the load constant at 88.1 tons, it was necessary to adjust the jack pressure in response to temperature changes.

Figure 25. Creep Rate Versus Load for Tieback 18-2



The most interesting aspect of the time-dependent distribution of strain in the anchor is the relatively uniform increase in strain at most strain gauge locations for any one load. As a result of this kind of behavior, the slope of the strain distribution curve between most strain gauge locations remained essentially constant during the load-hold. This implies that there was little time-dependent change in the rate of load transfer. Also note that the magnitude of time-dependent strain increase at the maximum test load of 88.1 tons, which was held constant for 10,000 minutes, was not significantly greater than that of any other load-holds which were held constant for much shorter periods of time.

Residual-strain distribution in the tendon is presented in Figure 15. In the unbonded length, residual strain decreased with increasing load. The reason for this behavior was probably due to hysteresis (see Section IV). The residual-strain distribution in the anchor conformed to expected behavior. As the load increased, the residual strain in the anchor increased. The residual-strain distribution indicates that the minimum unbonded length was provided. For all loads, the transition zone between the unbonded length and the anchor length was well-defined and occurred 27 to 30 feet from the back of the anchor.

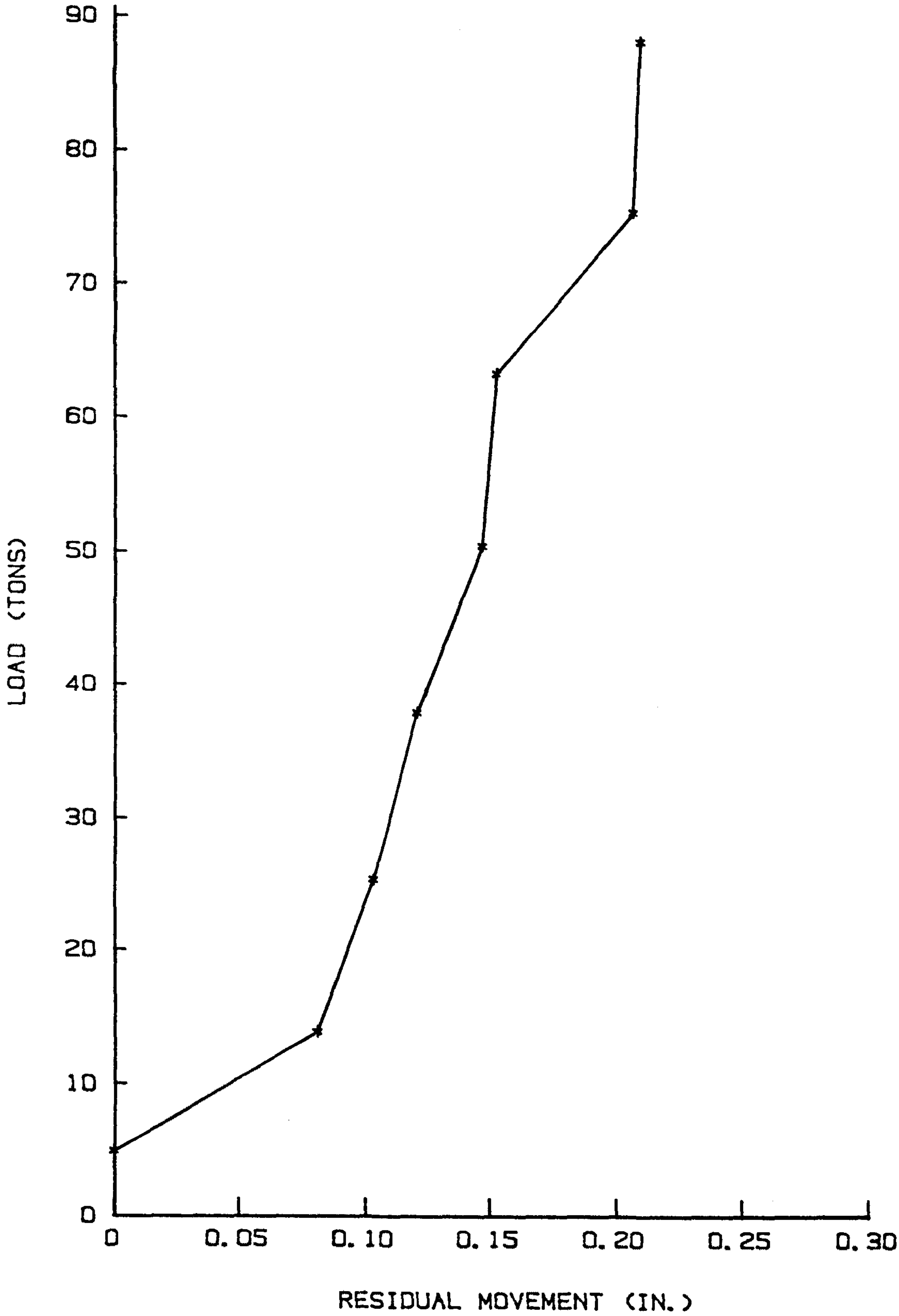
(2) Deformed Metal Tube

The distribution of strain in the deformed metal tube is presented in Figure 16. The most interesting feature of the strain distribution in the deformed metal tube is the presence of compressive strains at the front of the anchor.

This also indicates that the load applied at the anchorhead was transferred down the tendon to the front of the anchor and then transferred to the soil; otherwise, compressive strains could not exist at the front of the anchor. Also, note that the location of the neutral axis (transition point from compression to tension) gradually moved down the anchor as the load increased.

The time-dependent distribution of strain in the deformed metal tube shown in Figure 16 differed significantly from that in the tendon (see Figure 14). During the load-hold at 88.1 tons the deformed metal tube exhibited a significant increase in strain in the back of the anchor. This

Figure 23. Residual Anchor Movement of Tieback 18-2



most loads. However, at the higher loads, the measured creep movements tended to deviate from the computed creep movements. It is highly unlikely that the back of the anchor had begun to move through the soil in light of the load-displacement-time performance and strain distribution characteristics of tieback 15-8. In addition, it should be noted that the maximum difference between the computed and measured creep movement is on the order of 0.06 inch.

2. Tieback 18-2

a. Load-displacement-time Performance

Figure 22 presents a plot of load versus displacement measured at the anchorhead, while Figure 23 displays the residual anchor movement of tieback 18-2. At the maximum test load of 88.1 tons, the residual anchor movement was only 0.210 inches. Both the elastic load-displacement response and the relatively small residual anchor movements of tieback 18-2 indicate that it had additional capacity beyond 88.1 tons.

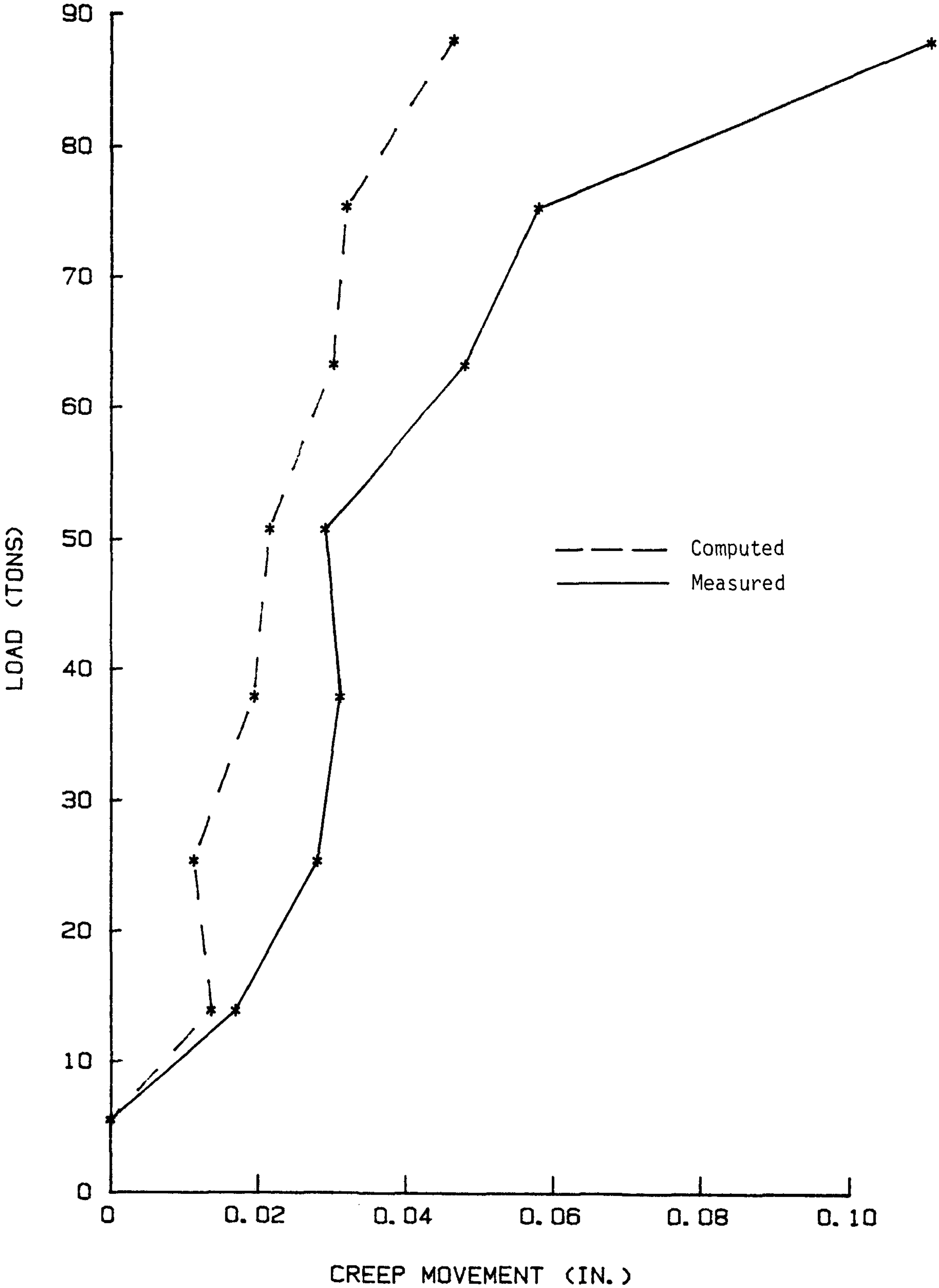
The time-dependent response of tieback 18-2 presented in Figures 24 and 25 showed no signs of tieback failure. At the maximum test load of 88.1 tons, the creep rate was essentially constant at 0.020 inches/logarithmic cycle for the first three logarithmic cycles (1 to 1,000 minutes). The creep rate could not be determined for the last logarithmic cycle, because of the significant decrease in movement throughout this cycle. This behavior is more likely attributable to a shift in the independent reference point from which anchorhead displacement was measured rather than an indication of actual physical performance (after the 1,000-minute mark of the load-hold at 88.1 tons, the weather suddenly turned warm and caused the ice, which partially supported the independent reference point, to thaw). The apparent decrease in creep movement also masked the true residual anchor movement measured at 88.1 tons. The true residual anchor movement was most likely somewhat greater than the measured 0.210 inches.

b. Strain Distribution

(1) Tendon

The distribution of strain in the tendon of tieback 18-2 is presented in Figure 26. There was very little decrease in strain along the unbonded length at most loads. Although bending strains were measured in the unbonded length, paired strain gauges were used to compensate for them. The magnitude of strain measured at the transition from the unbonded to the anchor length is interesting in

Figure 21. Measured Versus Computed Creep Movement of Tieback 15-8



The time-dependent distribution of strain in the anchor shows a relatively uniform increase in strain at most gauge locations--behavior also exhibited in tieback 15-8. This implies that there was little time-dependent change in the rate of load transfer. The magnitude of strain increase at the maximum test load of 88.1 tons, which was held constant for 10,000 minutes, was not significantly greater than that of any other load-holds which were held constant for much shorter periods of time.

Residual-strain distribution in the tendon is presented in Figure 27. In the unbonded length, residual strain decreased with increasing load, as was the case for tieback 15-8. The residual-strain distribution in the anchor length conformed to expected behavior. As the load increased, the residual strain in the tendon increased. The transition zone between the unbonded length and the anchor length was well-defined and occurred 28 to 30 feet from the back of the anchor. This indicates that the minimum unbonded length was provided.

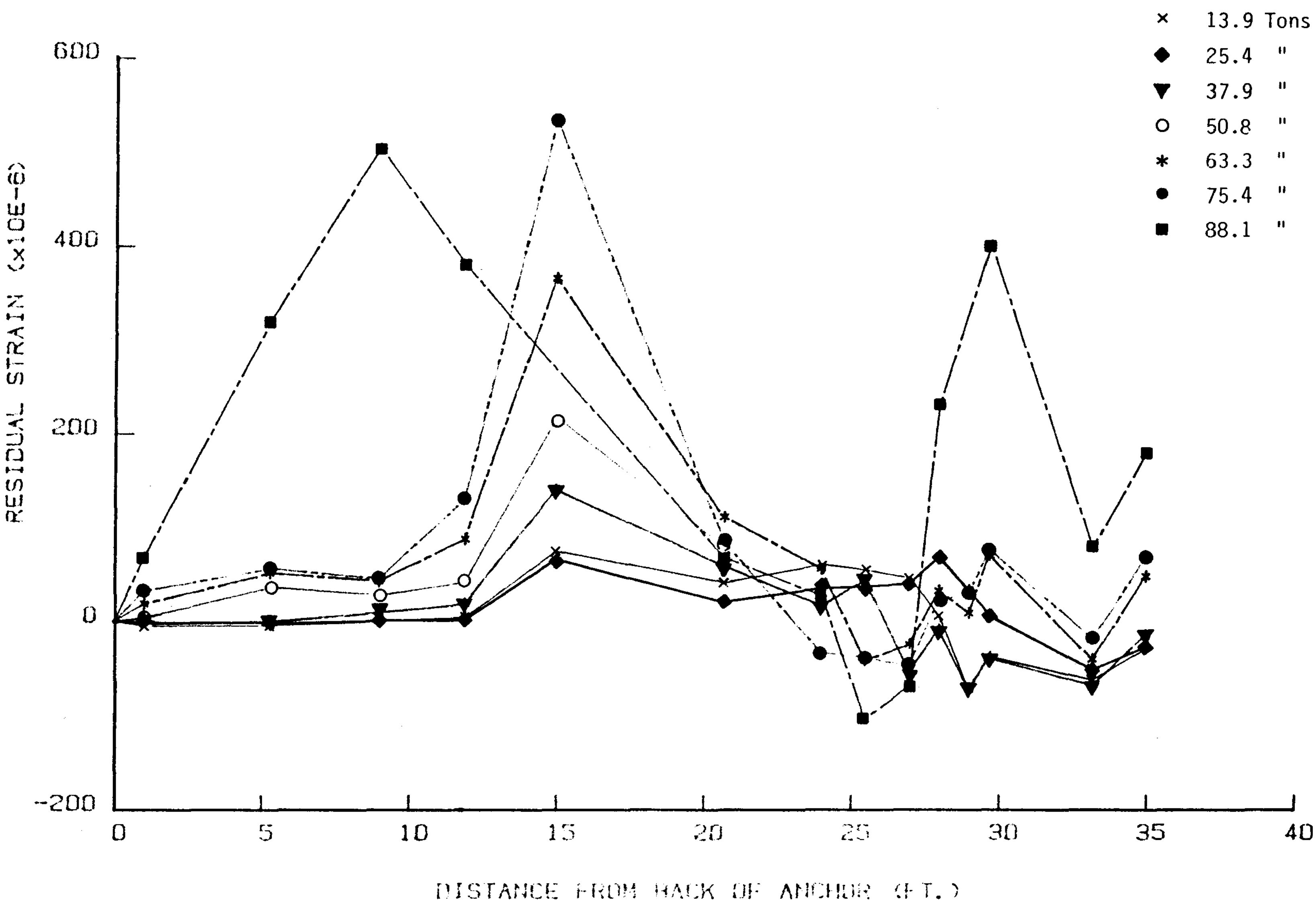
(2) Deformed Metal Tube

The distribution of strain in the deformed metal tube is presented in Figure 28. Strains tended to decrease to zero at the front of the anchor. The time-dependent distribution of strain in the deformed metal tube shown in Figure 28 exhibited a rather uniform increase in strain at each load at most gauge locations. In addition, very little load was transferred to the back of the anchor, even at 88.1 tons. Residual-strain distribution in the deformed metal tube is shown in Figure 29. Significant compressive residual strains were measured at the front of the anchor. The compressive residual strains increased as the load increased. Very little residual strain was measured in the back of the anchor for loads less than 88.1 tons.

c. Computed and Measured Movements

In the ensuing discussion of computed and measured movements, it should be noted that the computed movements of the anchorhead would tend to be less than the measured movements for reasons identical to those presented in the analysis of tieback 15-8 (see Section VI.A.1.c). At 88.1 tons, the measured total movement and creep movement at the 1,000-minute mark were used because of the shift of the reference point. A

Figure 19. Distribution of Residual Strain in the Deformed Metal Tube of Tieback 15-8



After the second logarithmic cycle of the load-hold at 88.1 tons, the creep rate of tieback 15-8 was double that of tieback 18-2. The greater creep rate exhibited by tieback 15-8 appeared to be reflected by the time-dependent changes in strain along the deformed metal tube (see Figures 16-18).

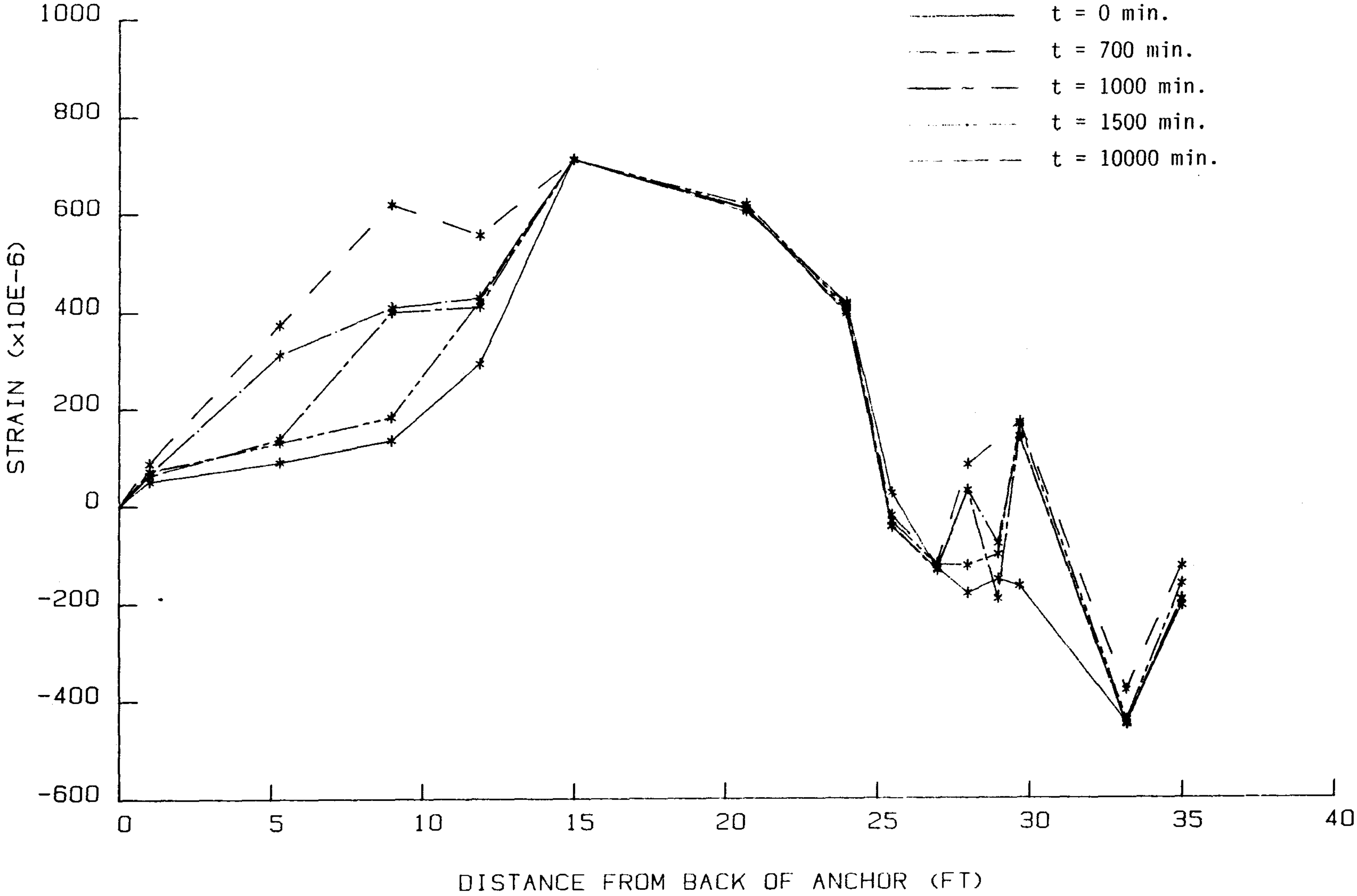
A comparison of the distribution of strain in the tendon of each tieback reveals that, for any given load, tieback 15-8 exhibited slightly higher strains in the unbonded length. This behavior is most likely attributable to the presence of greater friction forces and bending strains in tieback 18-2. Both factors would tend to decrease the magnitude of tensile strain measured in the unbonded length. In the anchor length. The shape of the strain distribution curves of each tieback is different; however, relatively little strain was measured at the back of either tendon, even at the maximum test load of 88.1 tons. The discussion on the mobilization of maximum load transfer indicated that a maximum value of load transfer had not been developed beyond the mid-point of the anchor length of either tendon. Based on the assumption that at failure a maximum value of load transfer is developed at the very back of the anchor, it appears that both tiebacks had additional capacity. A comparison of the distribution of residual strain in the deformed metal tube of each tieback reveals that much more residual strain was developed at the back of the anchor of tieback 15-8.

B. Long-term Investigation

The results of the long-term investigation of the two instrumented tiebacks are presented together with the long-term performance of the retaining wall and soil mass to show that the performance of the tiebacks, and the performance of the retaining wall and soil mass are interrelated.

Figure 32 presents data related to the drop in water table level, wall movement, and tieback load loss at Station 304+23 and Station 305+19. At Station 304+23, the data shows that the major drop in water table level was accompanied by a significant portion of the total wall movement back into the hill in the direction of the cemetery. In turn, the largest loss in tieback load occurred during the same time period. By November of 1982, the water table level has stabilized at an elevation 14 feet lower than its original position. After November of 1982, the wall movement appeared to be at least partially cyclical in nature, while the tieback load loss appeared to have stabilized. A maximum loss in tieback load of 9.2 tons was recorded in December of 1983. At the elevation of the tieback, the maximum movement of the wall in the direction of the cemetery was 0.045 inches in November of 1983. At first glance, a load loss of 9.2 tons appears substantial, representing an approximate 25 percent reduction from the lock-off load of 40 tons. The question arises as to whether this reduction in load was due to creep of the tieback or wall movement. The question can be answered by computing the anticipated loss in load

Figure 17. Time-dependent Redistribution of Strain in the Deformed Metal Tube of Tieback 15-8 at 88.1 Tons



appeared to follow the same pattern observed at Station 304+23. A maximum loss in tieback load of 2.4 tons was recorded in the fall of 1983. At the elevation of the tieback, the maximum movement of the wall in the direction of the cemetery was 0.031 inches. An anticipated loss in tieback load of 2.2 tons was computed using Equation (2). Since the computed and measured tieback load losses are nearly identical, it is concluded that the reduction in tieback load was due to wall movement rather than tieback creep.

A physical interpretation of the data shown in Figure 32 is presented below. A drop in water table level behind the retaining wall resulted in a reduction of the total pressure acting on the wall. A reduction in total pressure resulted in the movement of the wall into the hill until a new point of equilibrium between the unbalanced forces acting on the retaining was reached. Inward movement of the wall was accompanied by an elastic shortening, , of the unbonded length of the tieback and therefore a reduction in load.

VII. Conclusions

Two instrumented, permanent post-grouted tiebacks were installed and tested in a stiff-to-very-stiff, red-brown, silty clay at the Mt. Carmel Cemetery retaining wall in Baltimore, Maryland, in the Winter of 1981-82. Based on the results of the test program, the following conclusions are made:

- The performance of each tieback satisfied the criteria listed in the contract specifications: i.e.,
  - the minimum unbonded length was provided, and
  - the creep movement at 75 tons did not exceed 0.1 inches between 0.5 and 5 minutes.

- Both tiebacks had an ultimate capacity greater than the maximum test load of 88.1 tons.

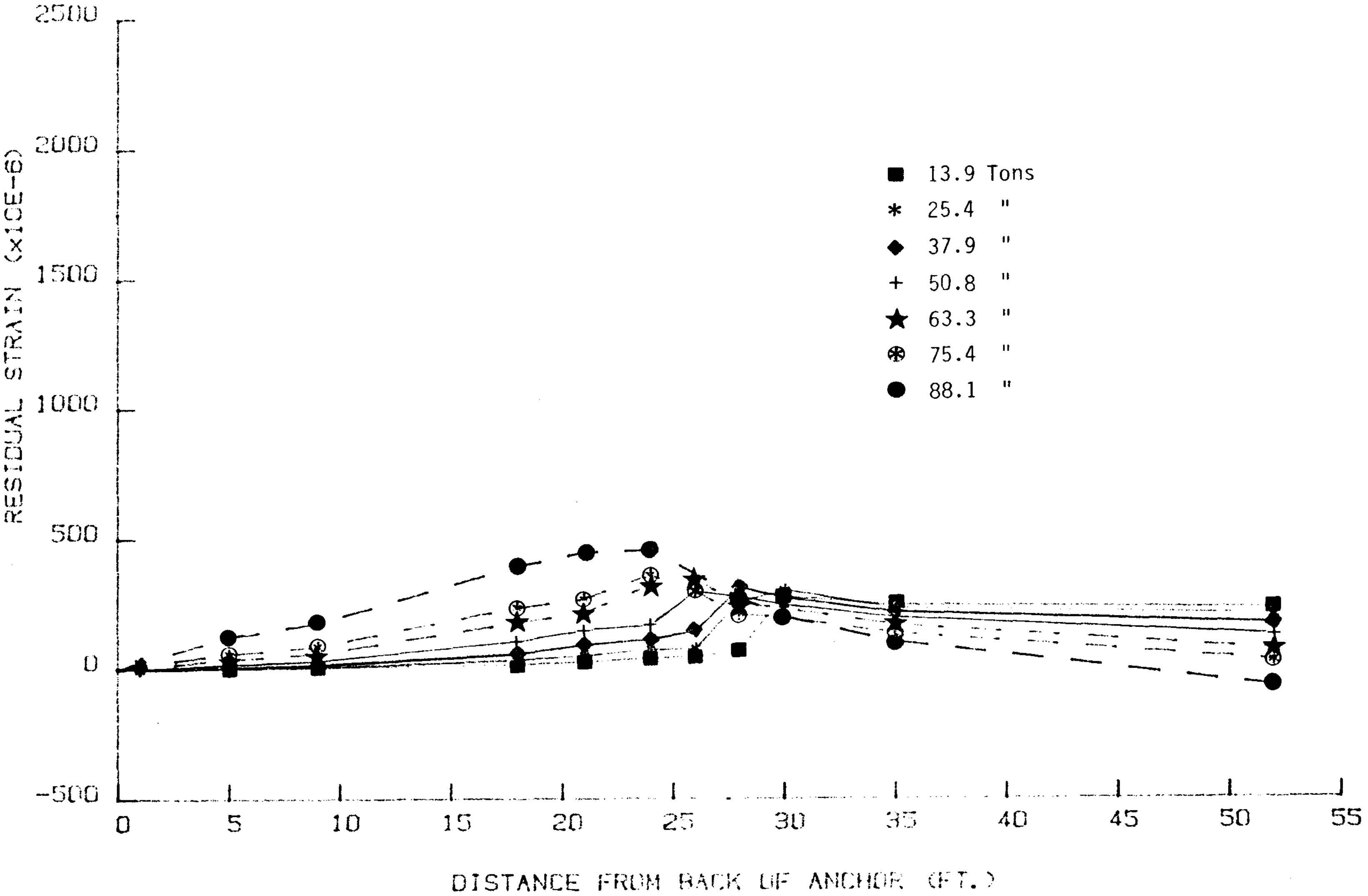
- Frictional forces and/or bending stresses were present in the tendons of both tiebacks and diminished the magnitude of tensile strain measured in the unbonded length at any given load.

- Distribution of strain, and therefore load, in the anchor length of the tendon was non-linear.

- Residual strains measured in the anchor of both tiebacks increased with increasing load. The opposite behavior was observed in the unbonded length and was thought to be due to hysteresis of the stressed tendon.

- Load transfer is a progressive phenomenon from the front of the anchor to the back. A prerequisite for tieback failure may be the development of a maximum value of load transfer at the very back of the anchor.

Figure 15. Distribution of Residual Strain in the Tendon of Tieback 15-8

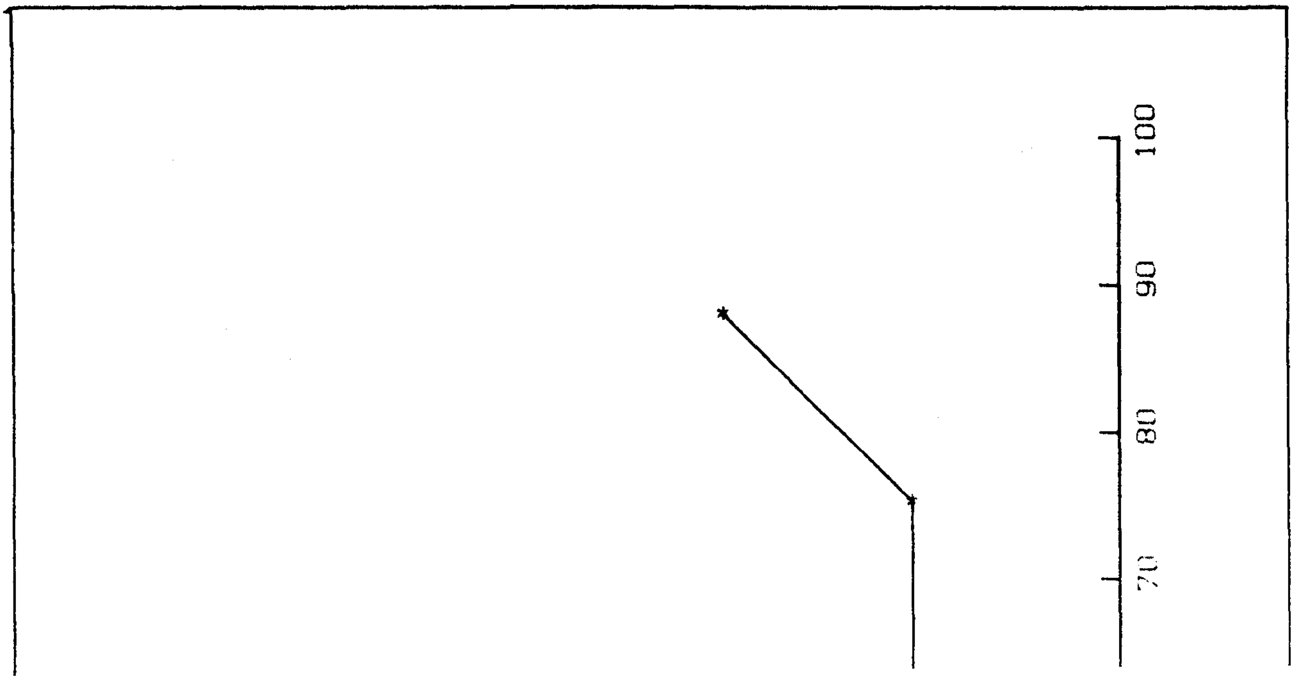


- Piezometers to measure pore water pressures and total pressure cells to measure total pressures can be incorporated into a test program to investigate the stress field around the tieback.

- More test programs involving permanent tiebacks anchored in cohesive soils need to be conducted to evaluate the short-term tieback creep tests currently being employed to predict long-term tieback performance.

- More test programs involving tiebacks anchored in poor soils need to be conducted.

- Load cells should be used in lieu of or in conjunction with strain gauges to measure long-term changes in tieback load.



100
90
80
70

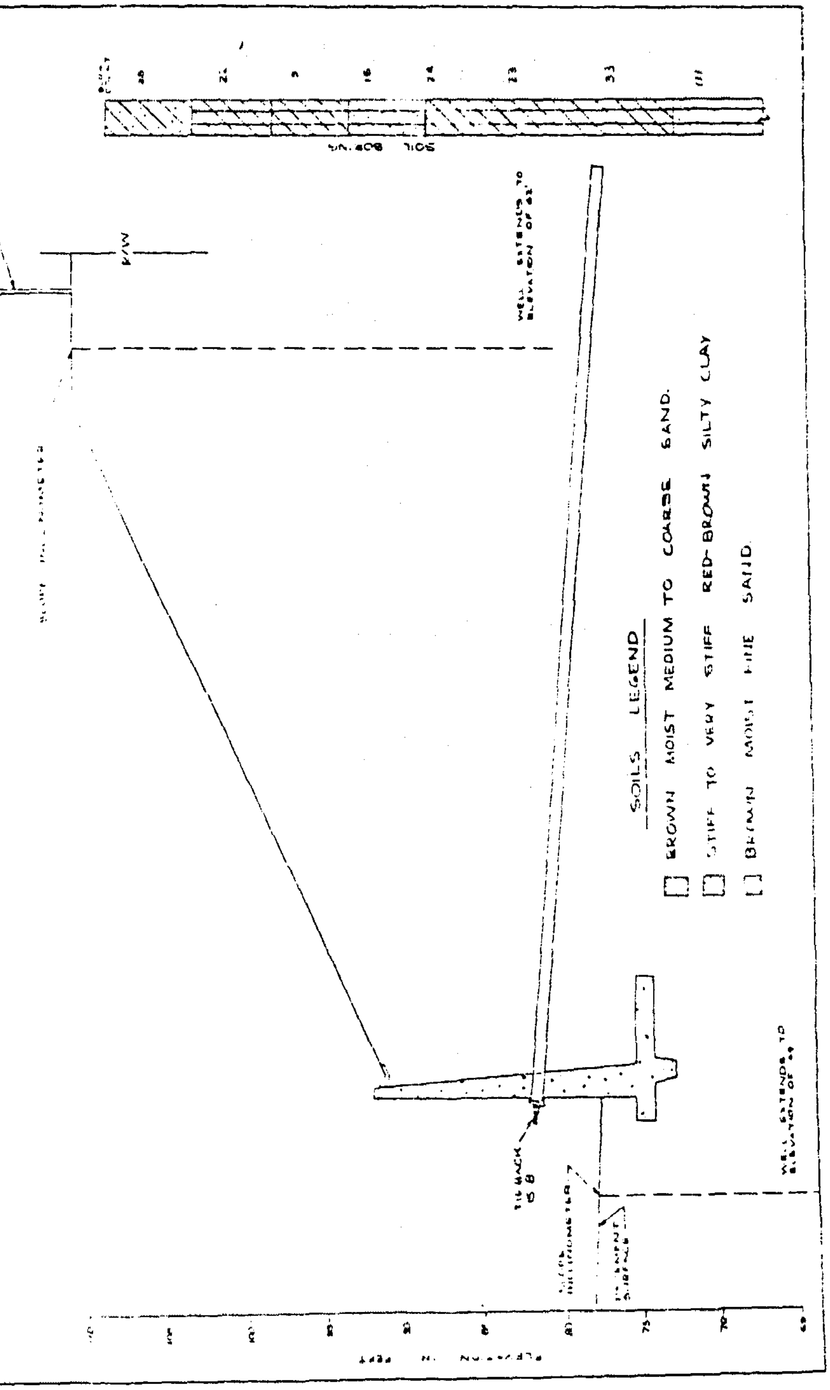
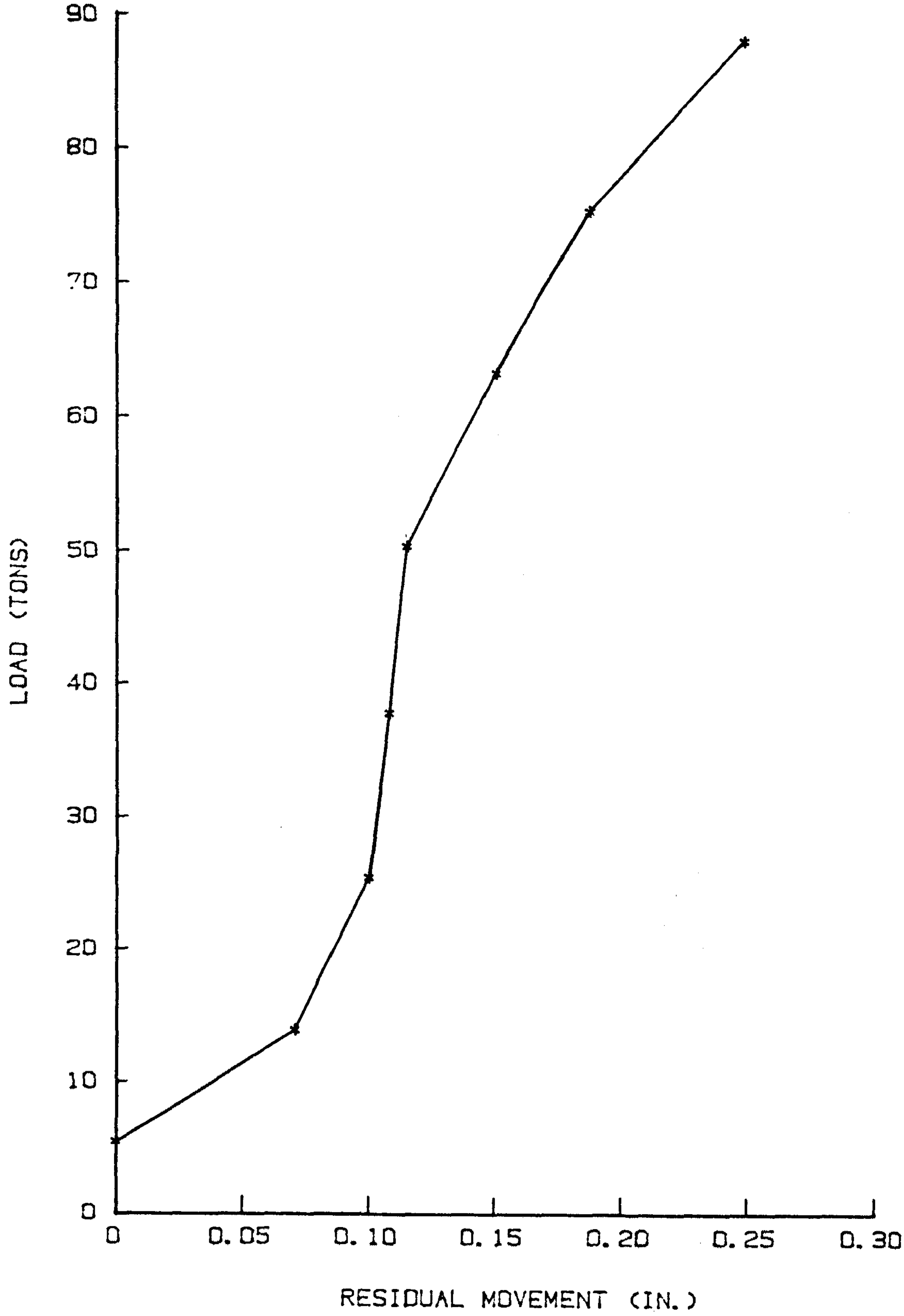
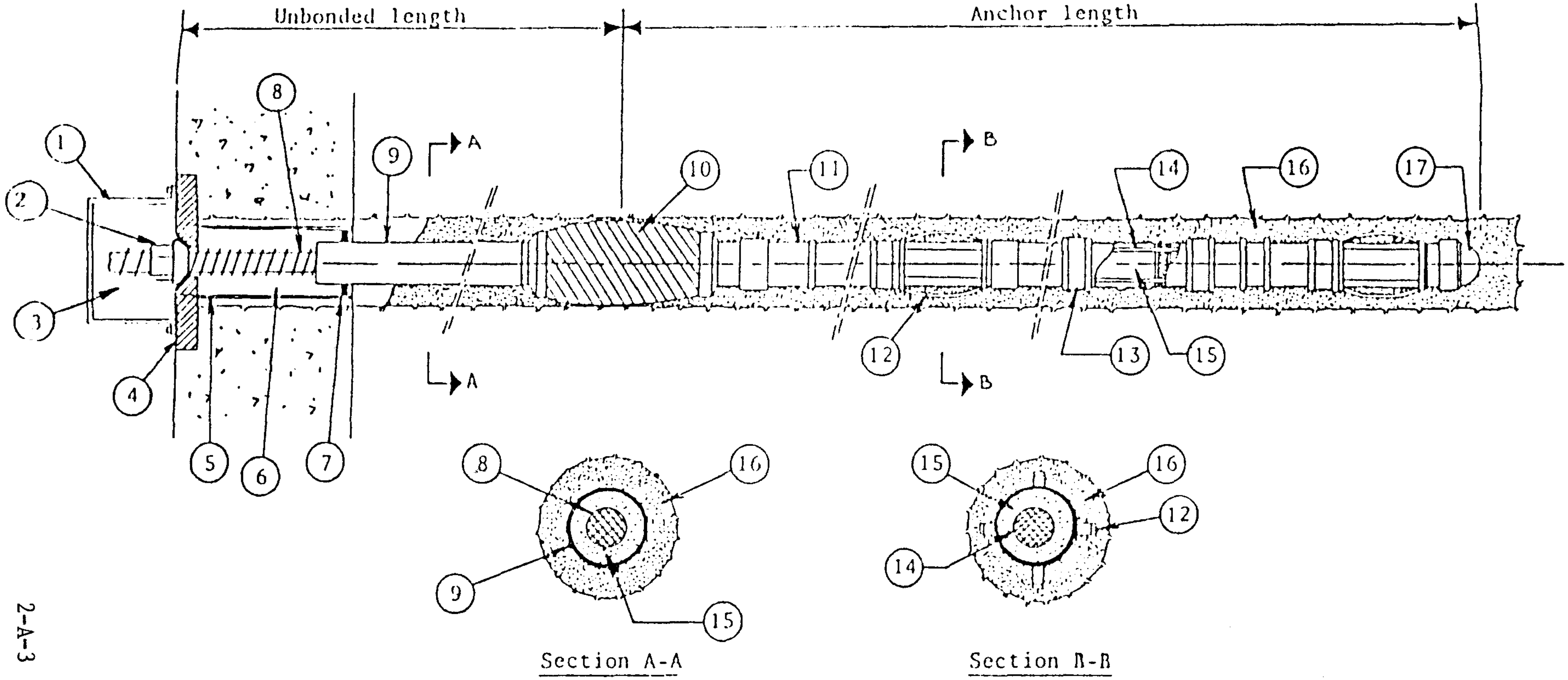


Figure 1. Cross-section and Soil Profile at Station 304+23

Figure 11. Residual Anchor Movement of Tieback 15-8



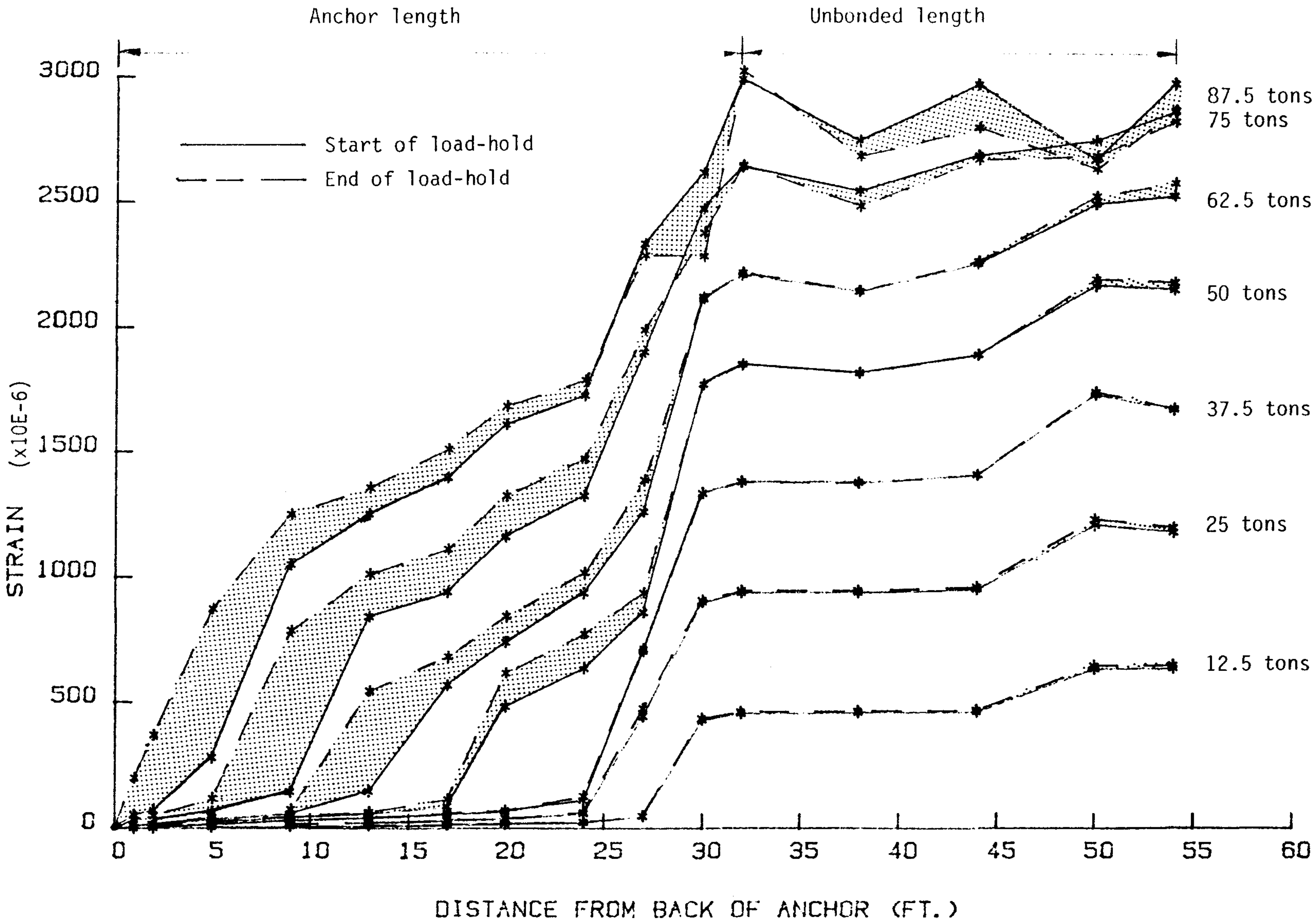


Legend:

1. Anchorage cover
2. Nut
3. Anticorrosion grease or grout
4. Bearing plate
5. Trumpet
6. Anticorrosion grease or grout
7. Seal
8. Bar tendon
9. PVC bond breaker
10. Inflatable bag
11. Deformed metal tube
12. Centralizer
13. Grouting valve
14. Bar tendon
15. Encapsulation grout
16. Anchor grout
17. End cap

Figure 3. Components of the TMD Tieback

Figure 9. Distribution of Strain in the Tendon of a Straight-shafted Tieback Anchored in a Cohesive Soil



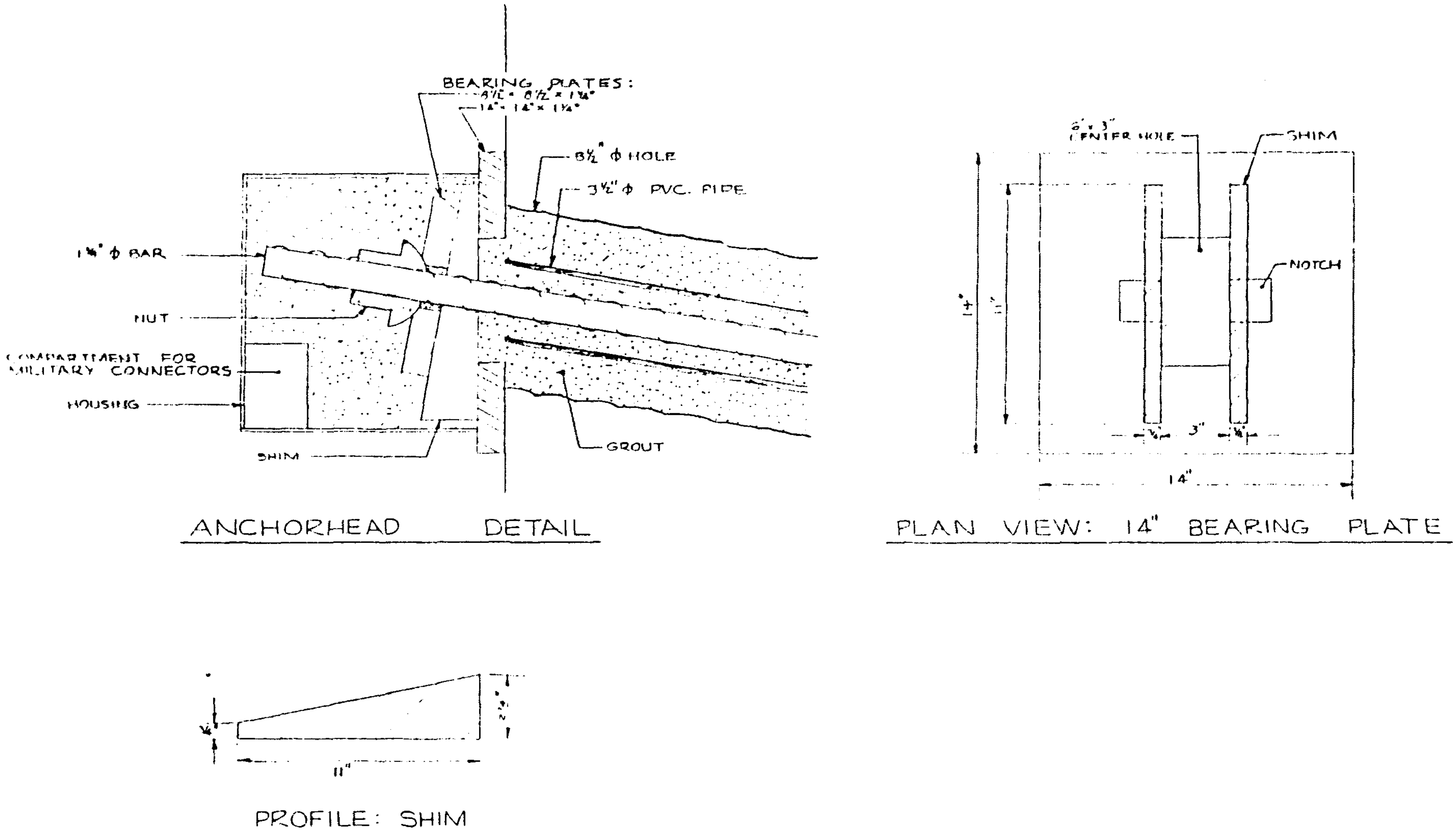


Figure 5. Anchorhead Detail.

Figure 7. Load-displacement Response of a Straight-shafted Tieback Anchored in a Cohesive Soil

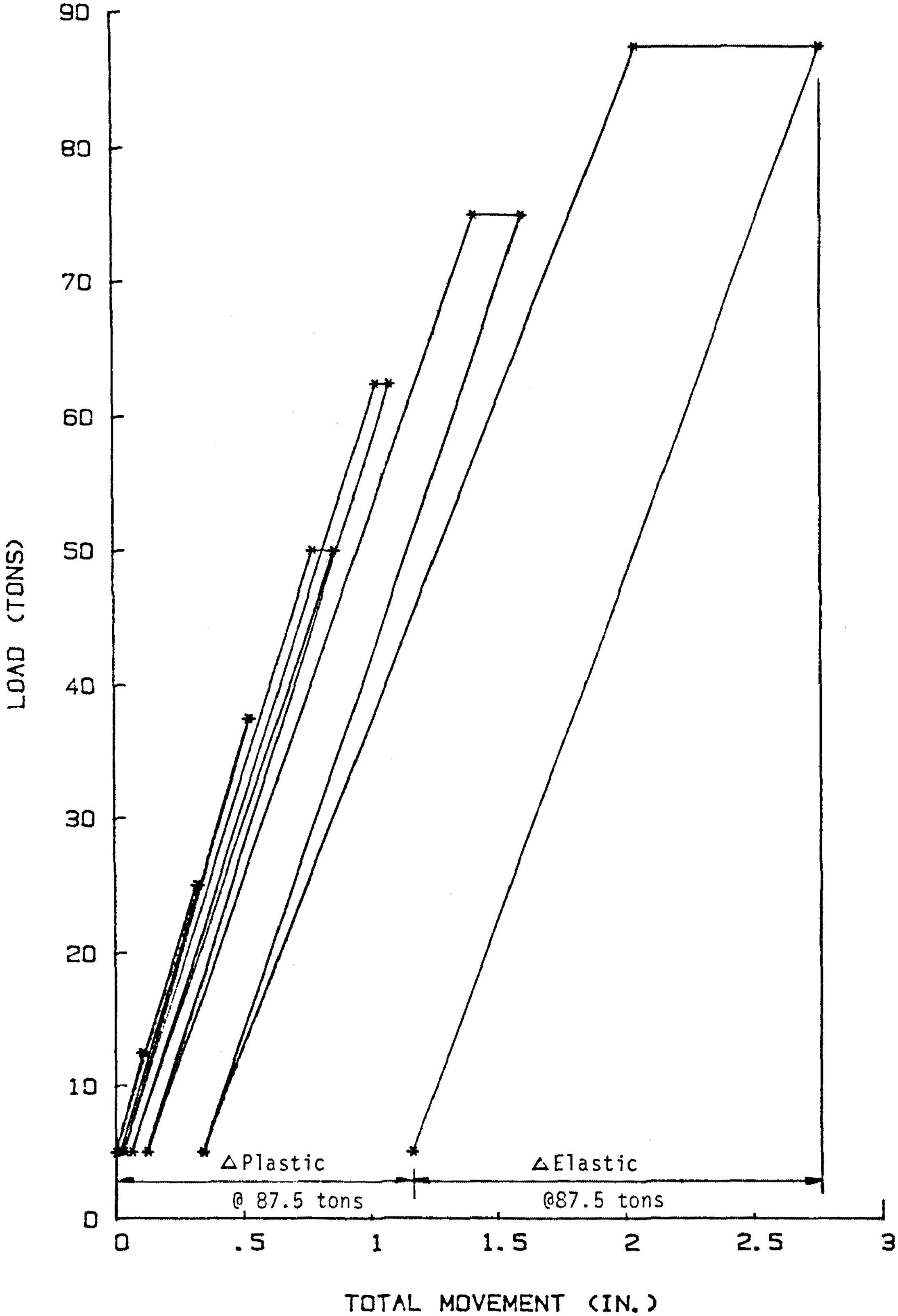
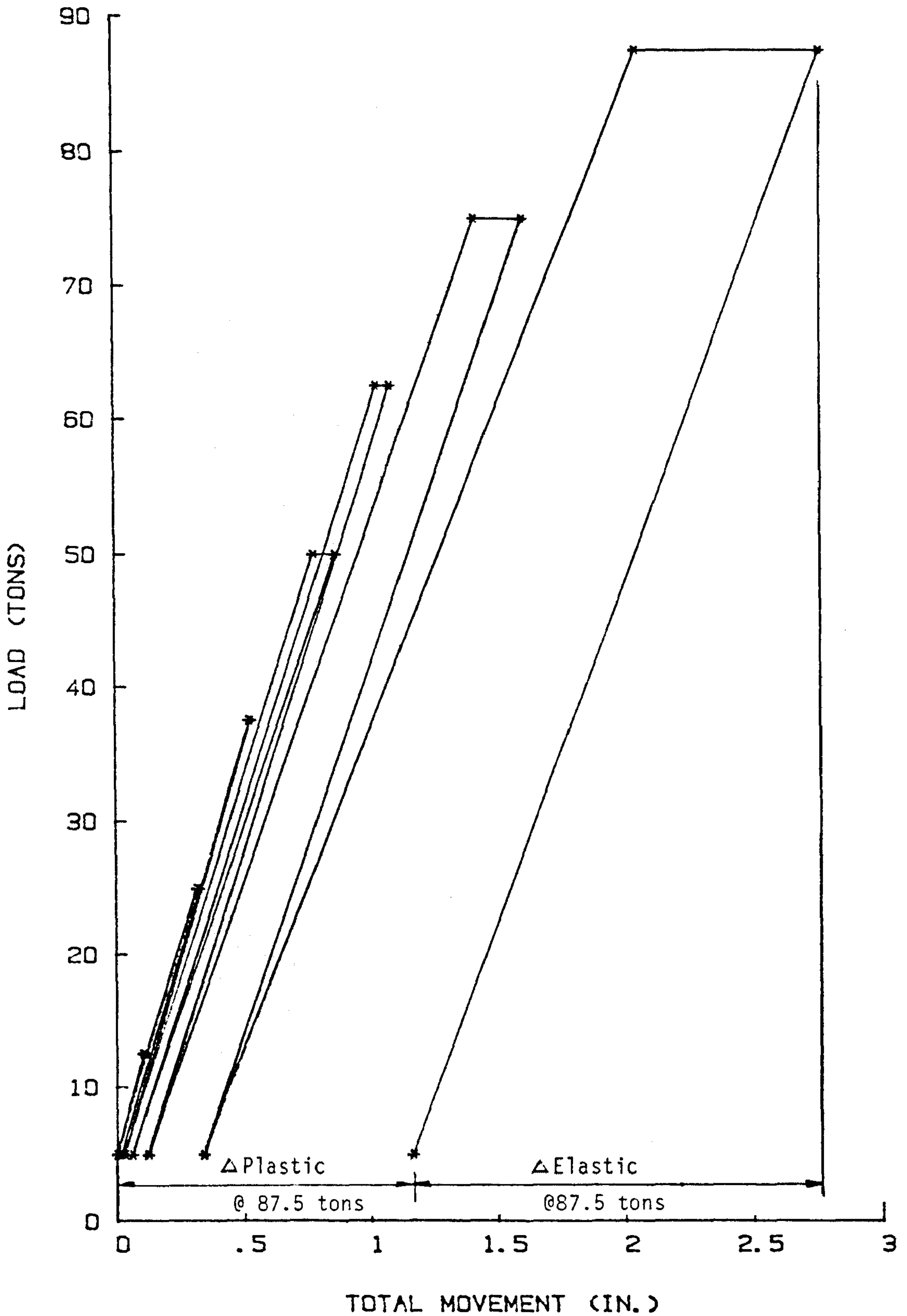


Figure 7. Load-displacement Response of a Straight-shafted Tieback Anchored in a Cohesive Soil



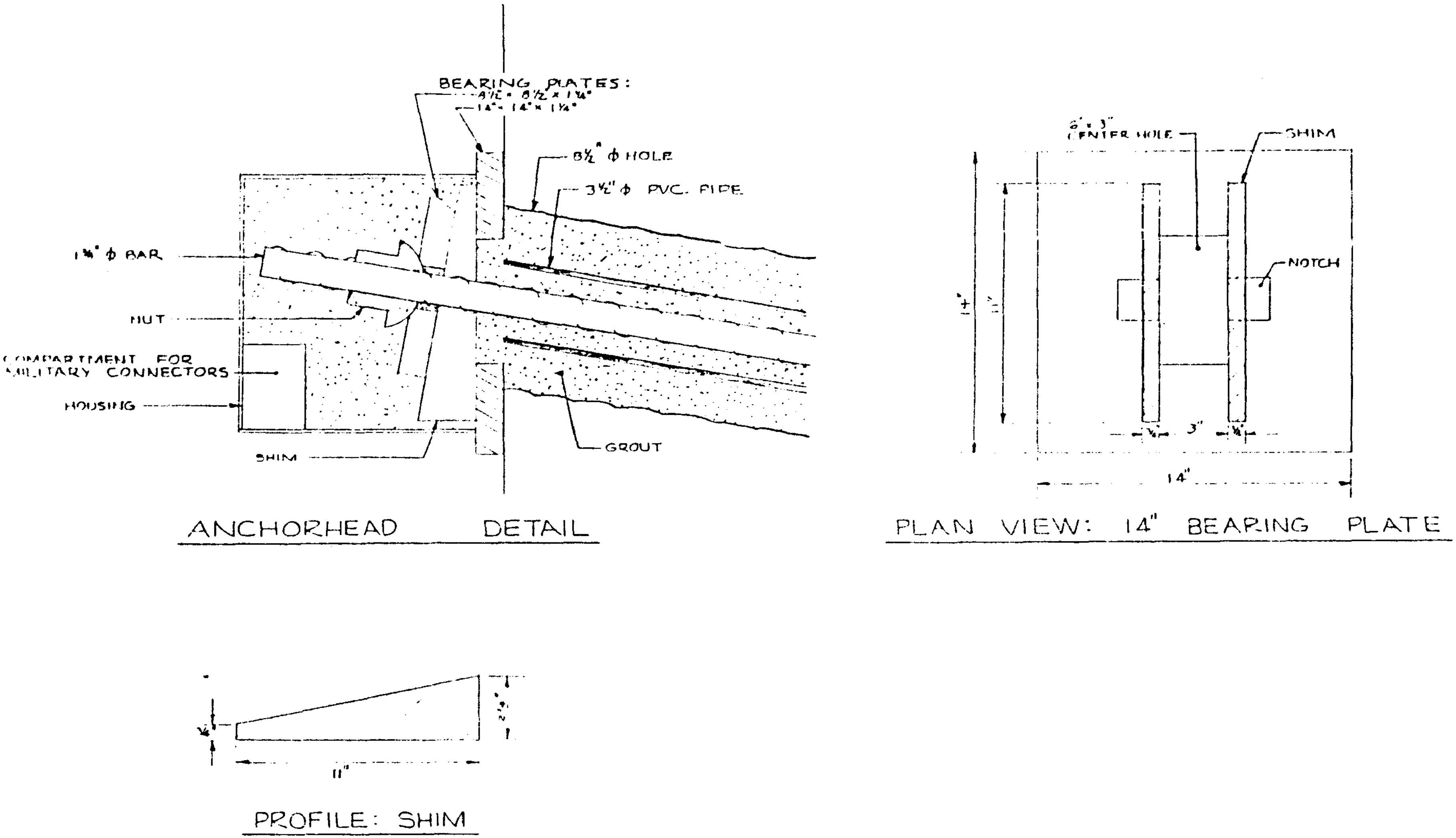
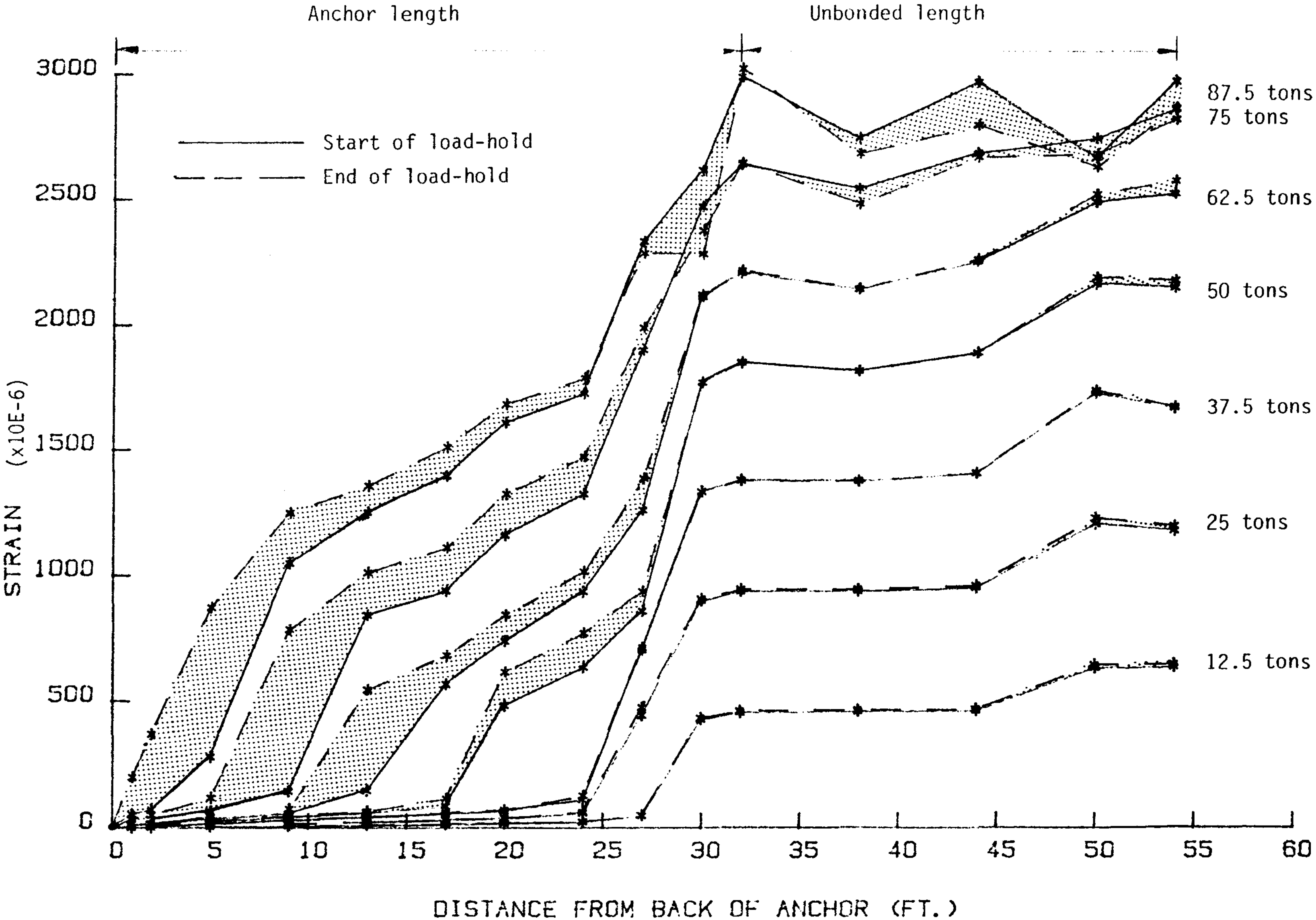
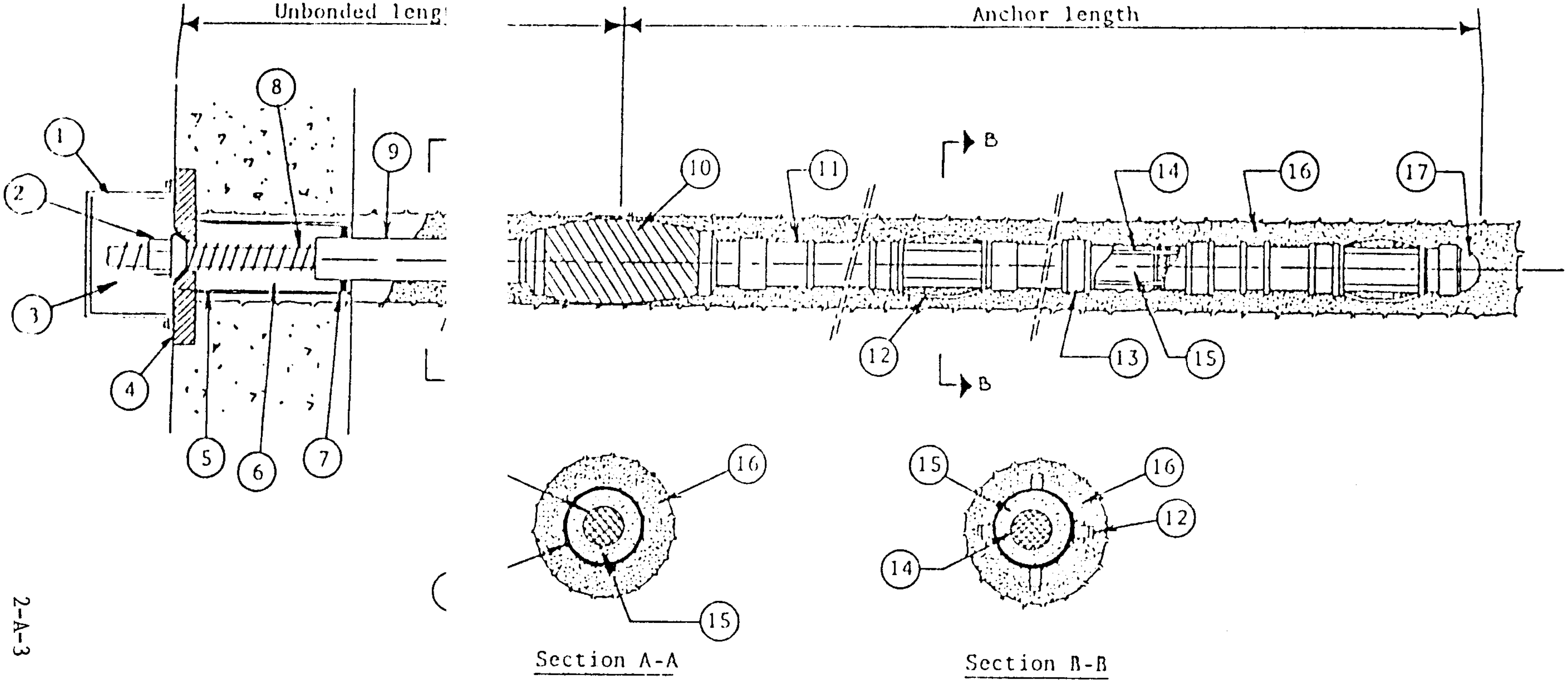


Figure 5. Anchorhead Detail.

Figure 9. Distribution of Strain in the Tendon of a Straight-shafted Tieback Anchored in a Cohesive Soil





Legend:

1. Anchor cover
2. Nut
3. Antico lon grease or grout
4. Bearin ate
5. Trumpe
6. Antico lon grease or grout
7. Seal
8. Bar te
9. PVC bo eaker
10. Inflatable bag
11. Deformed metal tube
12. Centralizer
13. Grouting valve
14. Bar tendon
15. Encapsulation grout
16. Anchor grout
17. End cap

Figure 3. Components ne TMD Tieback

Figure 11. Residual Anchor Movement of Tieback 15-8

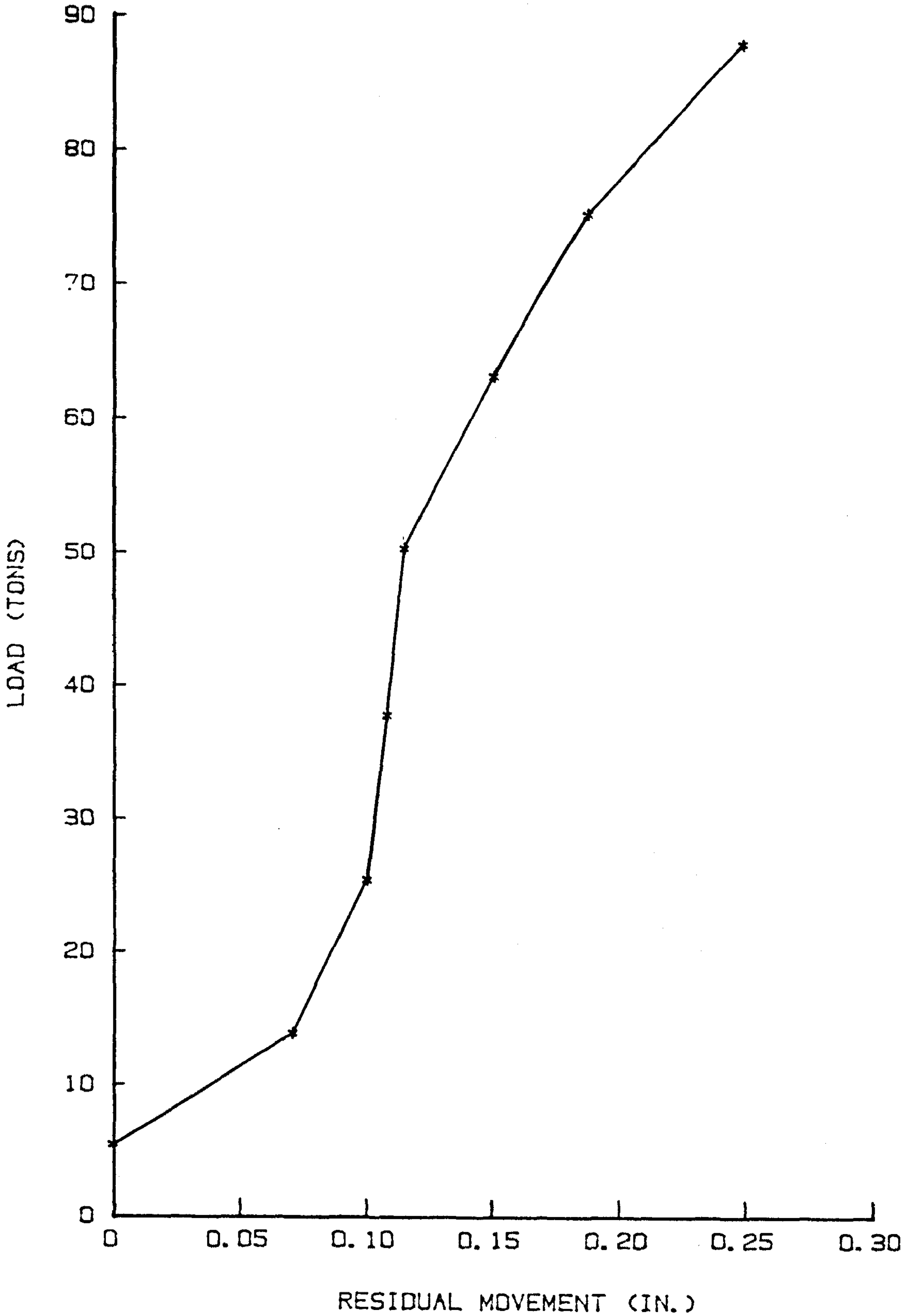


Figure 1. Cross-section and Soil Profile at Station 304+23

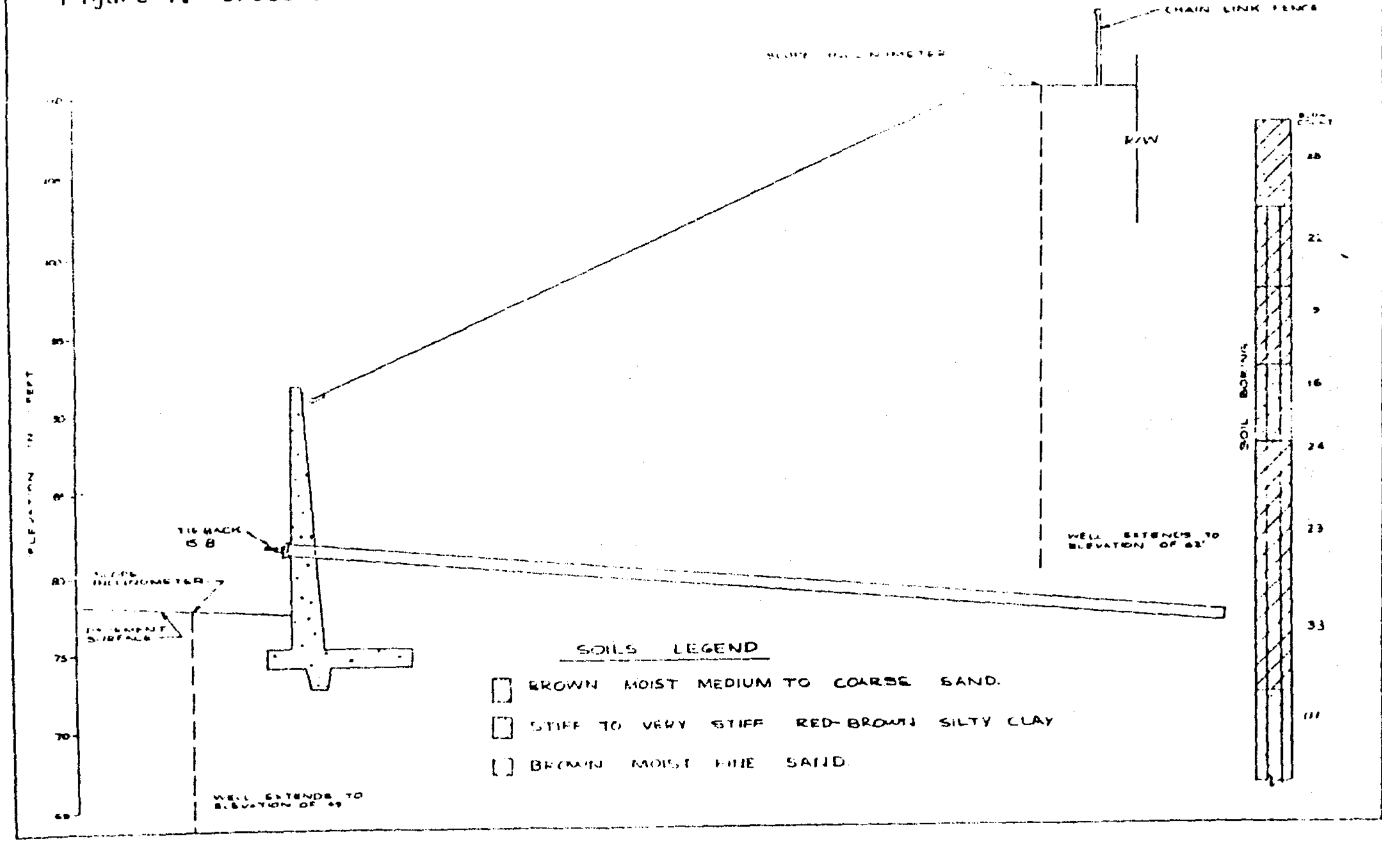
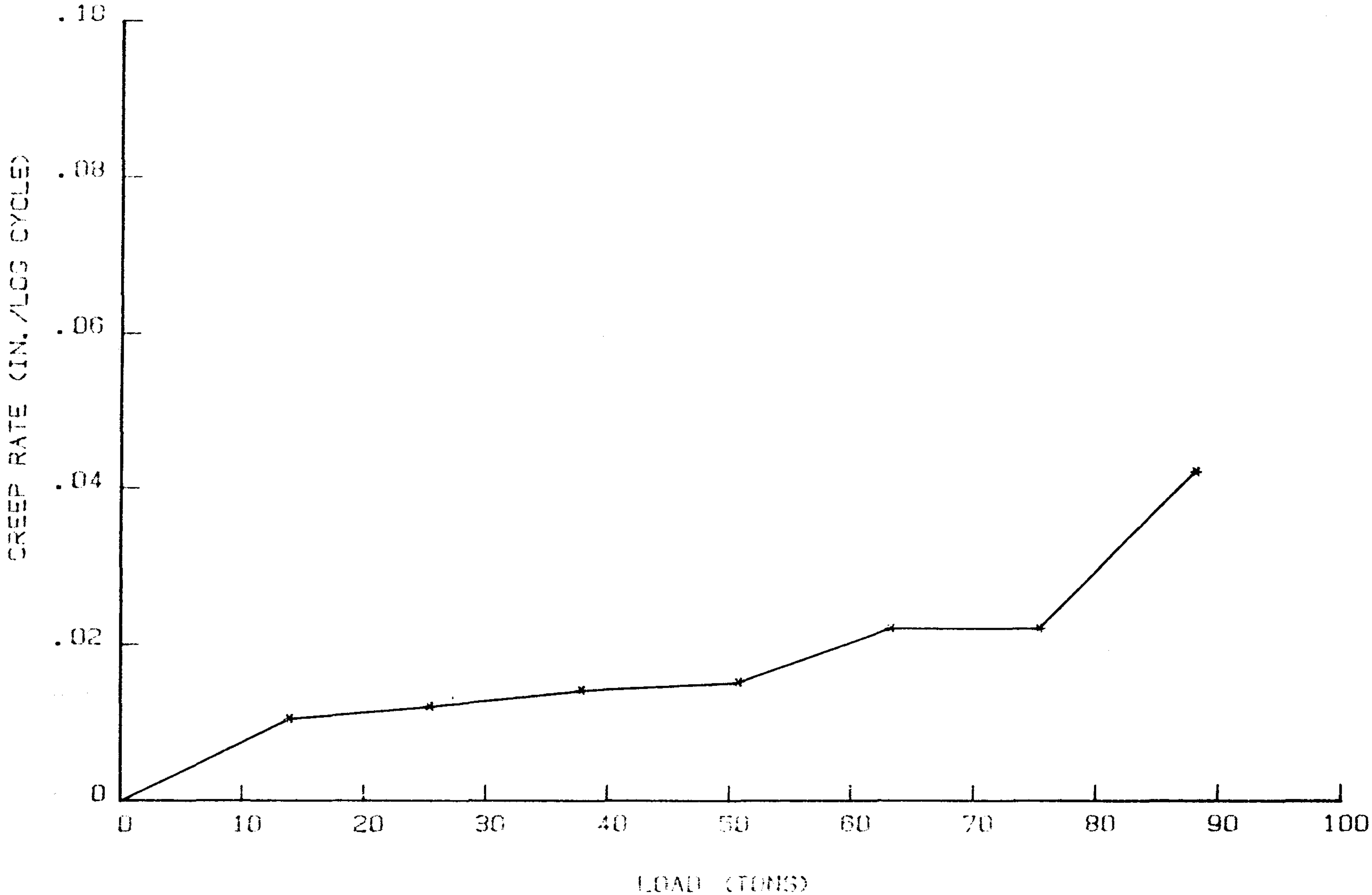


Figure 13. Creep Rate Versus Load for Tieback 15-8



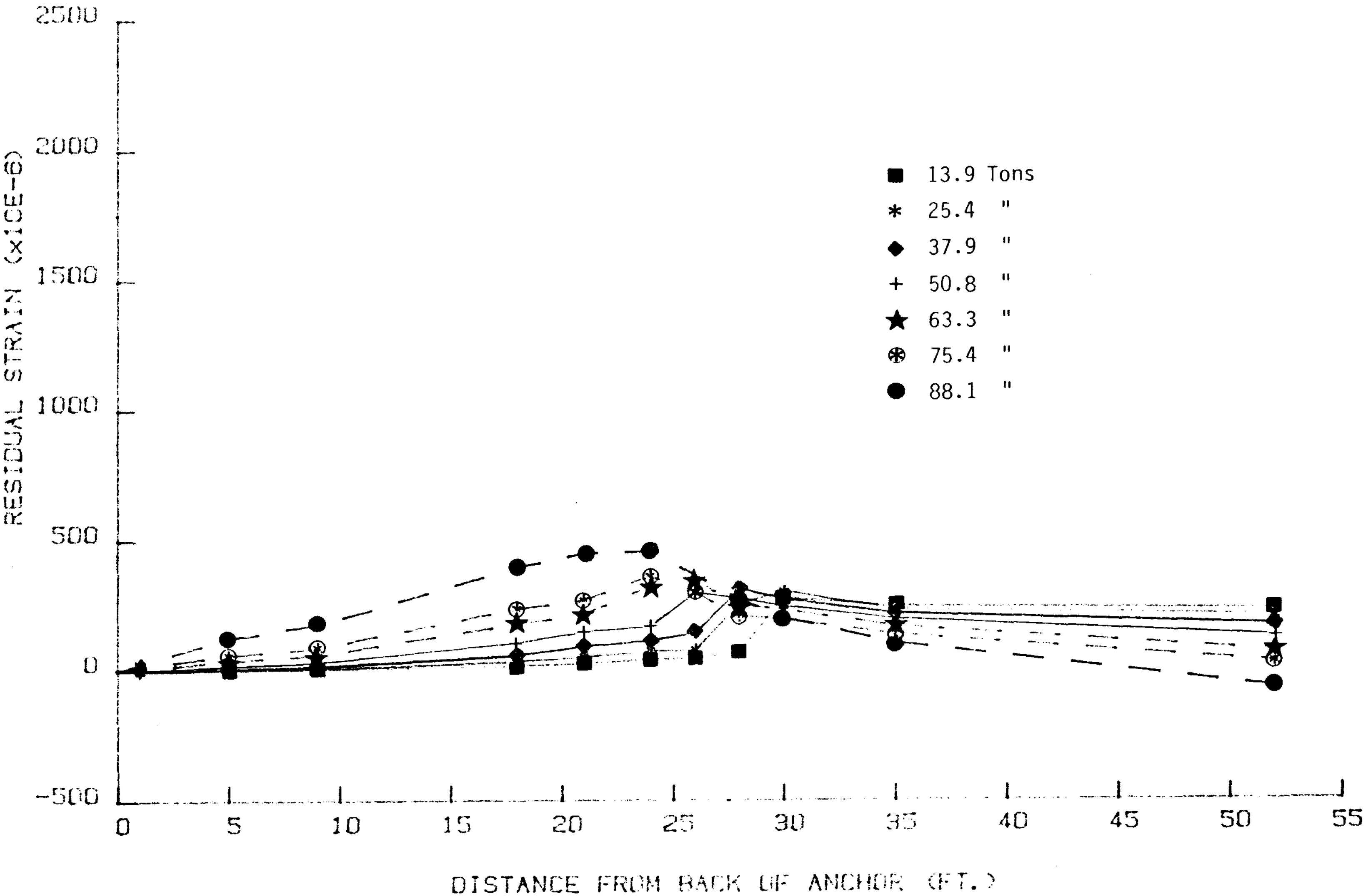
- Piezometers to measure pore water pressures and total pressure cells to measure total pressures can be incorporated into a test program to investigate the stress field around the tieback.

- More test programs involving permanent tiebacks anchored in cohesive soils need to be conducted to evaluate the short-term tieback creep tests currently being employed to predict long-term tieback performance.

- More test programs involving tiebacks anchored in poor soils need to be conducted.

- Load cells should be used in lieu of or in conjunction with strain gauges to measure long-term changes in tieback load.

Figure 15. Distribution of Residual Strain in the Tendon of Tieback 15-8



appeared to follow the same pattern observed at Station 304+23. A maximum loss in tieback load of 2.4 tons was recorded in the fall of 1983. At the elevation of the tieback, the maximum movement of the wall in the direction of the cemetery was 0.031 inches. An anticipated loss in tieback load of 2.2 tons was computed using Equation (2). Since the computed and measured tieback load losses are nearly identical, it is concluded that the reduction in tieback load was due to wall movement rather than tieback creep.

A physical interpretation of the data shown in Figure 32 is presented below. A drop in water table level behind the retaining wall resulted in a reduction of the total pressure acting on the wall. A reduction in total pressure resulted in the movement of the wall into the hill until a new point of equilibrium between the unbalanced forces acting on the retaining was reached. Inward movement of the wall was accompanied by an elastic shortening, , of the unbonded length of the tieback and therefore a reduction in load.

VII. Conclusions

Two instrumented, permanent post-grouted tiebacks were installed and tested in a stiff-to-very-stiff, red-brown, silty clay at the Mt. Carmel Cemetery retaining wall in Baltimore, Maryland, in the Winter of 1981-82. Based on the results of the test program, the following conclusions are made:

- The performance of each tieback satisfied the criteria listed in the contract specifications: i.e.,
  - the minimum unbonded length was provided, and
  - the creep movement at 75 tons did not exceed 0.1 inches between 0.5 and 5 minutes.

- Both tiebacks had an ultimate capacity greater than the maximum test load of 88.1 tons.

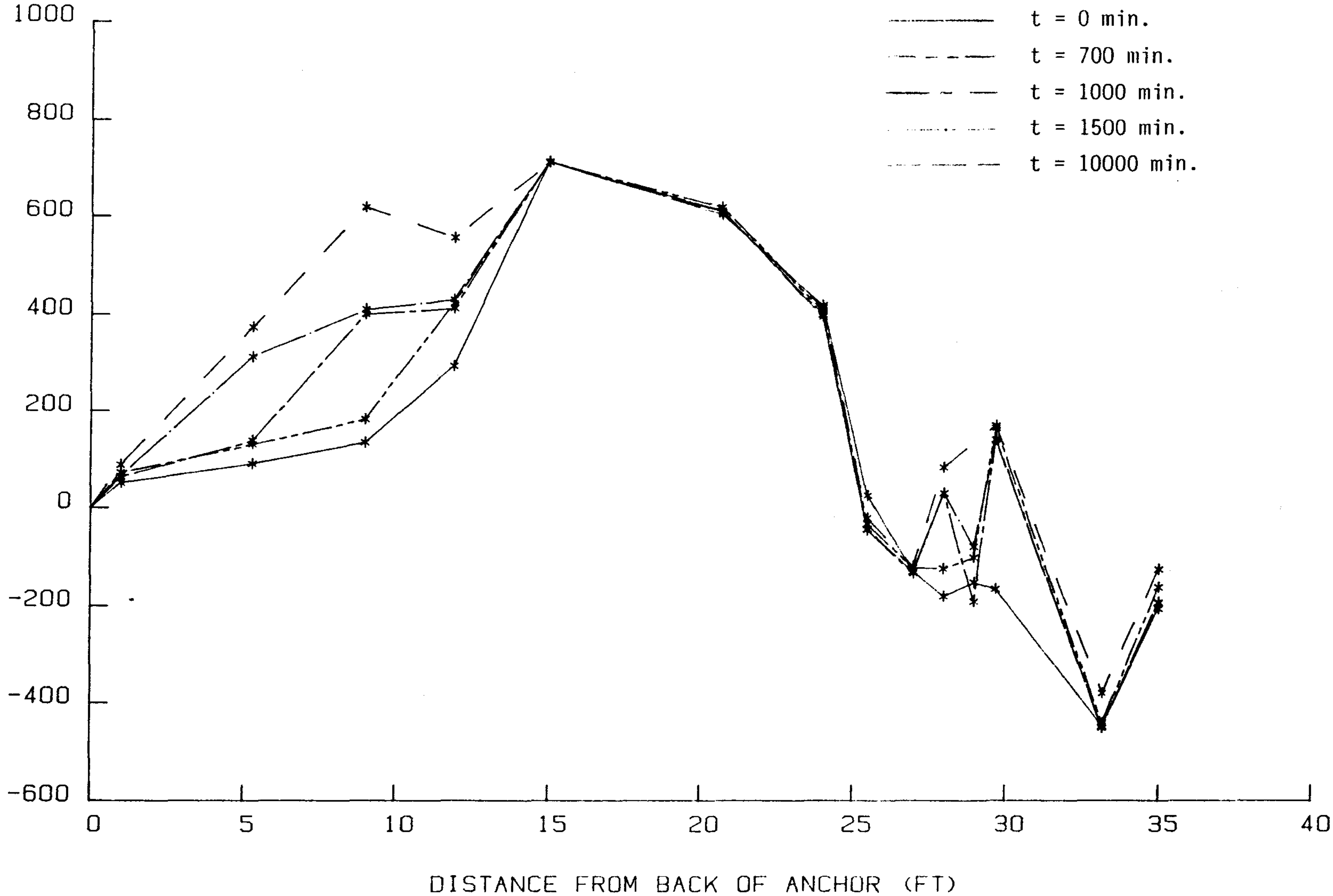
- Frictional forces and/or bending stresses were present in the tendons of both tiebacks and diminished the magnitude of tensile strain measured in the unbonded length at any given load.

- Distribution of strain, and therefore load, in the anchor length of the tendon was non-linear.

- Residual strains measured in the anchor of both tiebacks increased with increasing load. The opposite behavior was observed in the unbonded length and was thought to be due to hysteresis of the stressed tendon.

- Load transfer is a progressive phenomenon from the front of the anchor to the back. A prerequisite for tieback failure may be the development of a maximum value of load transfer at the very back of the anchor.

Figure 17. Time-dependent Redistribution of Strain in the Deformed Metal Tube of Tieback 15-8 at 88.1 Tons



After the second logarithmic cycle of the load-hold at 88.1 tons, the creep rate of tieback 15-8 was double that of tieback 18-2. The greater creep rate exhibited by tieback 15-8 appeared to be reflected by the time-dependent changes in strain along the deformed metal tube (see Figures 16-18).

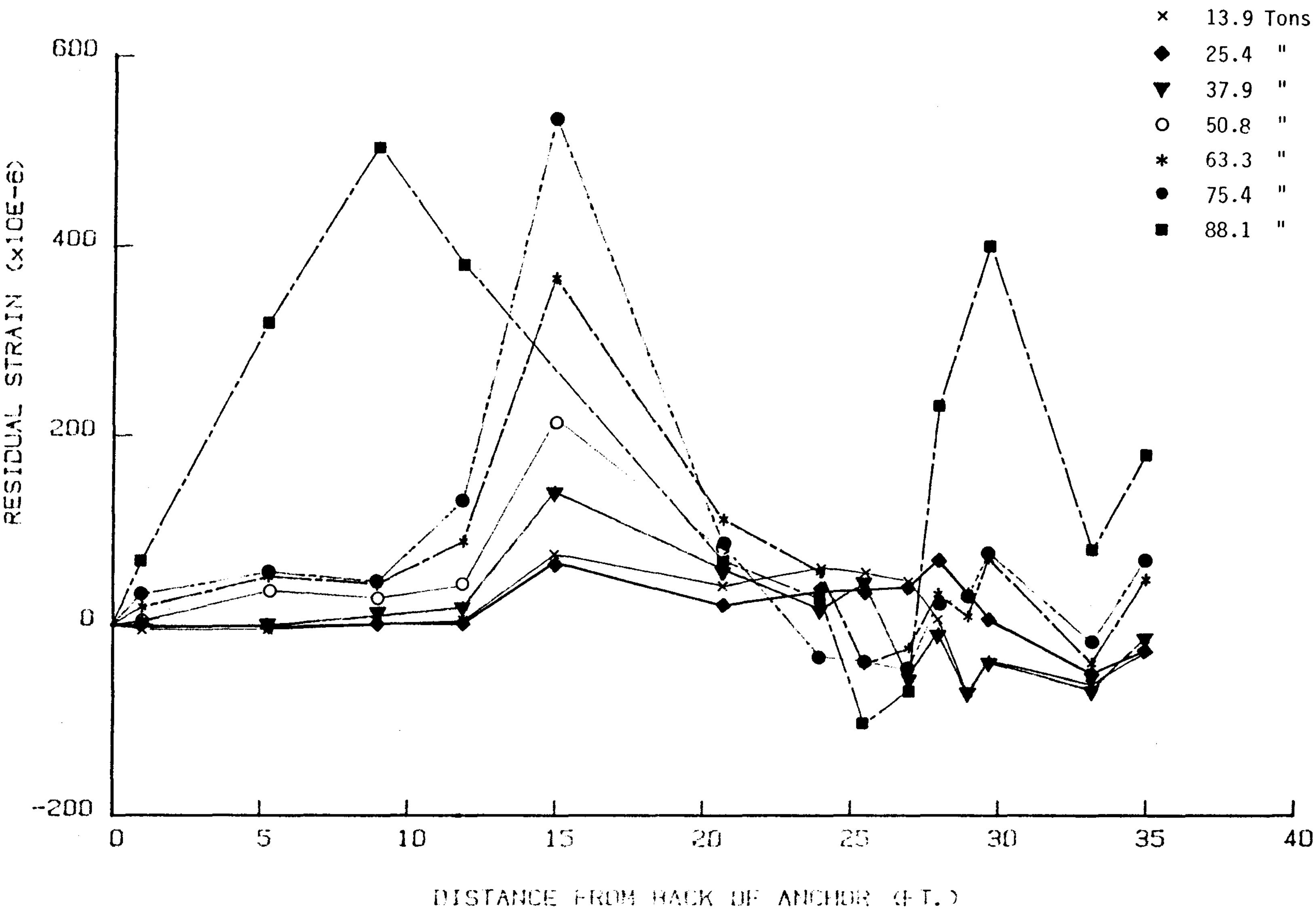
A comparison of the distribution of strain in the tendon of each tieback reveals that, for any given load, tieback 15-8 exhibited slightly higher strains in the unbonded length. This behavior is most likely attributable to the presence of greater friction forces and bending strains in tieback 18-2. Both factors would tend to decrease the magnitude of tensile strain measured in the unbonded length. In the anchor length. The shape of the strain distribution curves of each tieback is different; however, relatively little strain was measured at the back of either tendon, even at the maximum test load of 88.1 tons. The discussion on the mobilization of maximum load transfer indicated that a maximum value of load transfer had not been developed beyond the mid-point of the anchor length of either tendon. Based on the assumption that at failure a maximum value of load transfer is developed at the very back of the anchor, it appears that both tiebacks had additional capacity. A comparison of the distribution of residual strain in the deformed metal tube of each tieback reveals that much more residual strain was developed at the back of the anchor of tieback 15-8.

B. Long-term Investigation

The results of the long-term investigation of the two instrumented tiebacks are presented together with the long-term performance of the retaining wall and soil mass to show that the performance of the tiebacks, and the performance of the retaining wall and soil mass are interrelated.

Figure 32 presents data related to the drop in water table level, wall movement, and tieback load loss at Station 304+23 and Station 305+19. At Station 304+23, the data shows that the major drop in water table level was accompanied by a significant portion of the total wall movement back into the hill in the direction of the cemetery. In turn, the largest loss in tieback load occurred during the same time period. By November of 1982, the water table level has stabilized at an elevation 14 feet lower than its original position. After November of 1982, the wall movement appeared to be at least partially cyclical in nature, while the tieback load loss appeared to have stabilized. A maximum loss in tieback load of 9.2 tons was recorded in December of 1983. At the elevation of the tieback, the maximum movement of the wall in the direction of the cemetery was 0.045 inches in November of 1983. At first glance, a load loss of 9.2 tons appears substantial, representing an approximate 25 percent reduction from the lock-off load of 40 tons. The question arises as to whether this reduction in load was due to creep of the tieback or wall movement. The question can be answered by computing the anticipated loss in load

Figure 19. Distribution of Residual Strain in the Deformed Metal Tube of Tieback 15-8



The time-dependent distribution of strain in the anchor shows a relatively uniform increase in strain at most gauge locations--behavior also exhibited in tieback 15-8. This implies that there was little time-dependent change in the rate of load transfer. The magnitude of strain increase at the maximum test load of 88.1 tons, which was held constant for 10,000 minutes, was not significantly greater than that of any other load-holds which were held constant for much shorter periods of time.

Residual-strain distribution in the tendon is presented in Figure 27. In the unbonded length, residual strain decreased with increasing load, as was the case for tieback 15-8. The residual-strain distribution in the anchor length conformed to expected behavior. As the load increased, the residual strain in the tendon increased. The transition zone between the unbonded length and the anchor length was well-defined and occurred 28 to 30 feet from the back of the anchor. This indicates that the minimum unbonded length was provided.

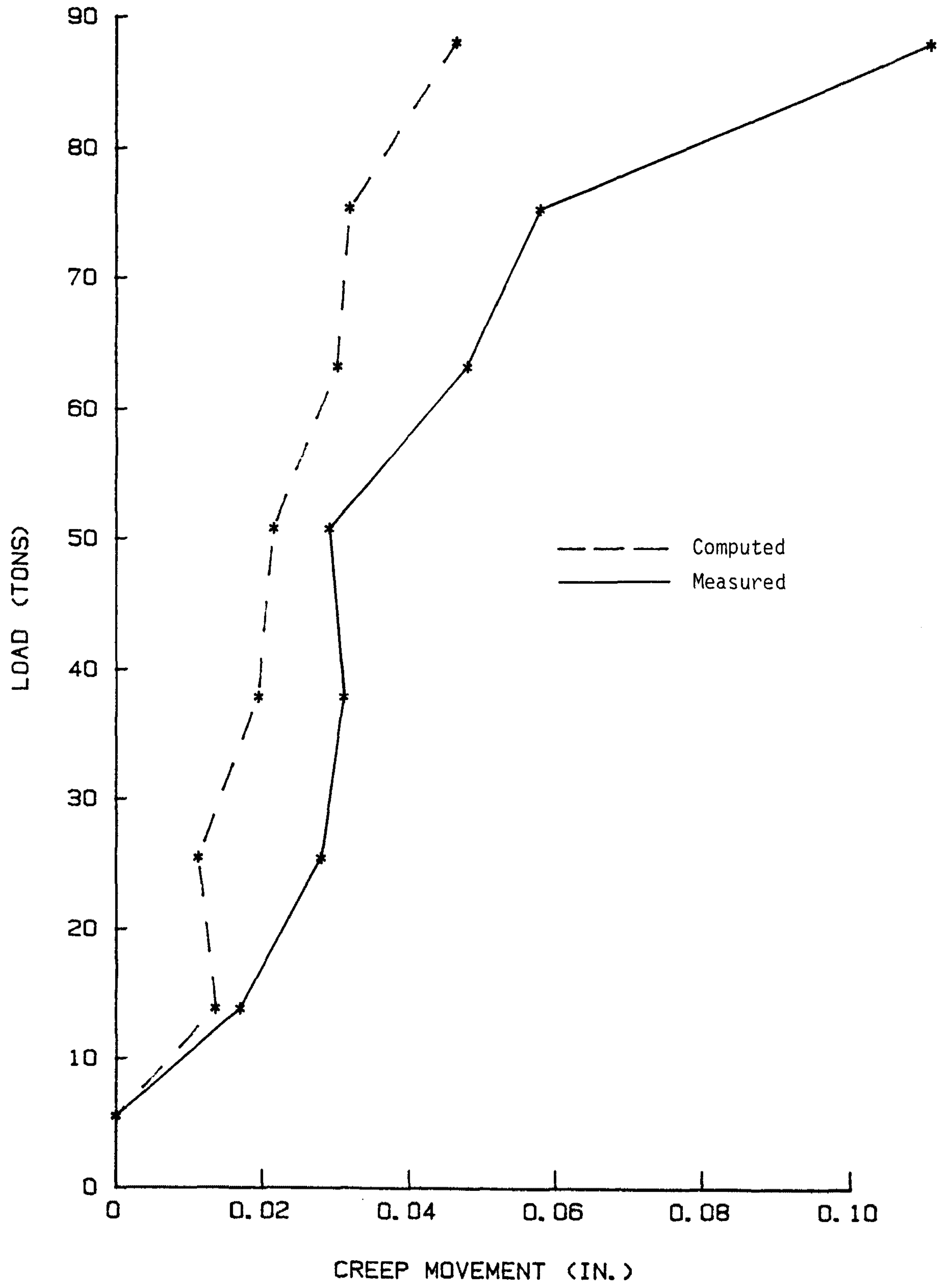
(2) Deformed Metal Tube

The distribution of strain in the deformed metal tube is presented in Figure 28. Strains tended to decrease to zero at the front of the anchor. The time-dependent distribution of strain in the deformed metal tube shown in Figure 28 exhibited a rather uniform increase in strain at each load at most gauge locations. In addition, very little load was transferred to the back of the anchor, even at 88.1 tons. Residual-strain distribution in the deformed metal tube is shown in Figure 29. Significant compressive residual strains were measured at the front of the anchor. The compressive residual strains increased as the load increased. Very little residual strain was measured in the back of the anchor for loads less than 88.1 tons.

c. Computed and Measured Movements

In the ensuing discussion of computed and measured movements, it should be noted that the computed movements of the anchorhead would tend to be less than the measured movements for reasons identical to those presented in the analysis of tieback 15-8 (see Section VI.A.1.c). At 88.1 tons, the measured total movement and creep movement at the 1,000-minute mark were used because of the shift of the reference point. A

Figure 21. Measured Versus Computed Creep Movement of Tieback 15-8



most loads. However, at the higher loads, the measured creep movements tended to deviate from the computed creep movements. It is highly unlikely that the back of the anchor had begun to move through the soil in light of the load-displacement-time performance and strain distribution characteristics of tieback 15-8. In addition, it should be noted that the maximum difference between the computed and measured creep movement is on the order of 0.06 inch.

2. Tieback 18-2

a. Load-displacement-time Performance

Figure 22 presents a plot of load versus displacement measured at the anchorhead, while Figure 23 displays the residual anchor movement of tieback 18-2. At the maximum test load of 88.1 tons, the residual anchor movement was only 0.210 inches. Both the elastic load-displacement response and the relatively small residual anchor movements of tieback 18-2 indicate that it had additional capacity beyond 88.1 tons.

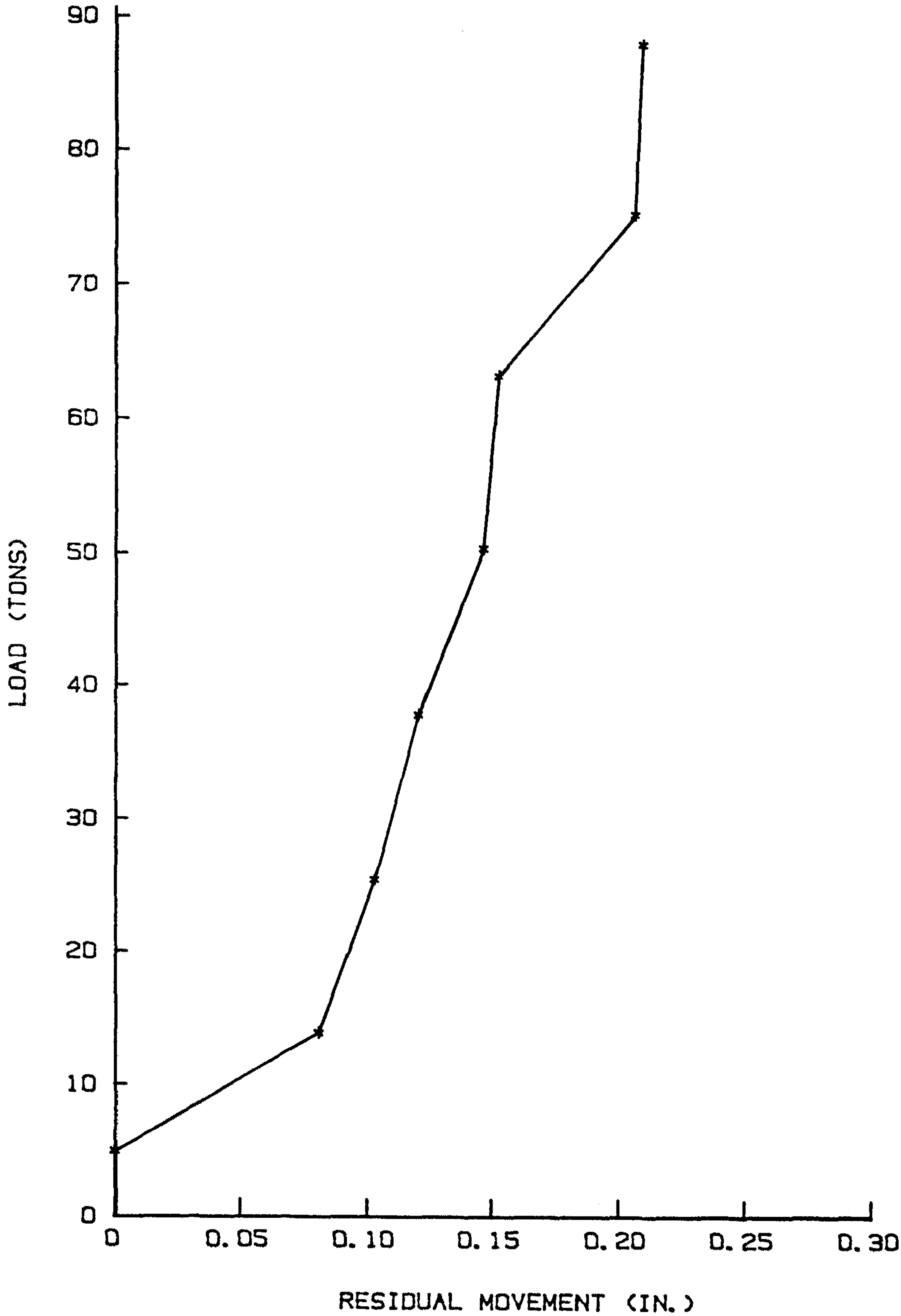
The time-dependent response of tieback 18-2 presented in Figures 24 and 25 showed no signs of tieback failure. At the maximum test load of 88.1 tons, the creep rate was essentially constant at 0.020 inches/logarithmic cycle for the first three logarithmic cycles (1 to 1,000 minutes). The creep rate could not be determined for the last logarithmic cycle, because of the significant decrease in movement throughout this cycle. This behavior is more likely attributable to a shift in the independent reference point from which anchorhead displacement was measured rather than an indication of actual physical performance (after the 1,000-minute mark of the load-hold at 88.1 tons, the weather suddenly turned warm and caused the ice, which partially supported the independent reference point, to thaw). The apparent decrease in creep movement also masked the true residual anchor movement measured at 88.1 tons. The true residual anchor movement was most likely somewhat greater than the measured 0.210 inches.

b. Strain Distribution

(1) Tendon

The distribution of strain in the tendon of tieback 18-2 is presented in Figure 26. There was very little decrease in strain along the unbonded length at most loads. Although bending strains were measured in the unbonded length, paired strain gauges were used to compensate for them. The magnitude of strain measured at the transition from the unbonded to the anchor length is interesting in

Figure 23. Residual Anchor Movement of Tieback 18-2



The most interesting aspect of the time-dependent distribution of strain in the anchor is the relatively uniform increase in strain at most strain gauge locations for any one load. As a result of this kind of behavior, the slope of the strain distribution curve between most strain gauge locations remained essentially constant during the load-hold. This implies that there was little time-dependent change in the rate of load transfer. Also note that the magnitude of time-dependent strain increase at the maximum test load of 88.1 tons, which was held constant for 10,000 minutes, was not significantly greater than that of any other load-holds which were held constant for much shorter periods of time.

Residual-strain distribution in the tendon is presented in Figure 15. In the unbonded length, residual strain decreased with increasing load. The reason for this behavior was probably due to hysteresis (see Section IV). The residual-strain distribution in the anchor conformed to expected behavior. As the load increased, the residual strain in the anchor increased. The residual-strain distribution indicates that the minimum unbonded length was provided. For all loads, the transition zone between the unbonded length and the anchor length was well-defined and occurred 27 to 30 feet from the back of the anchor.

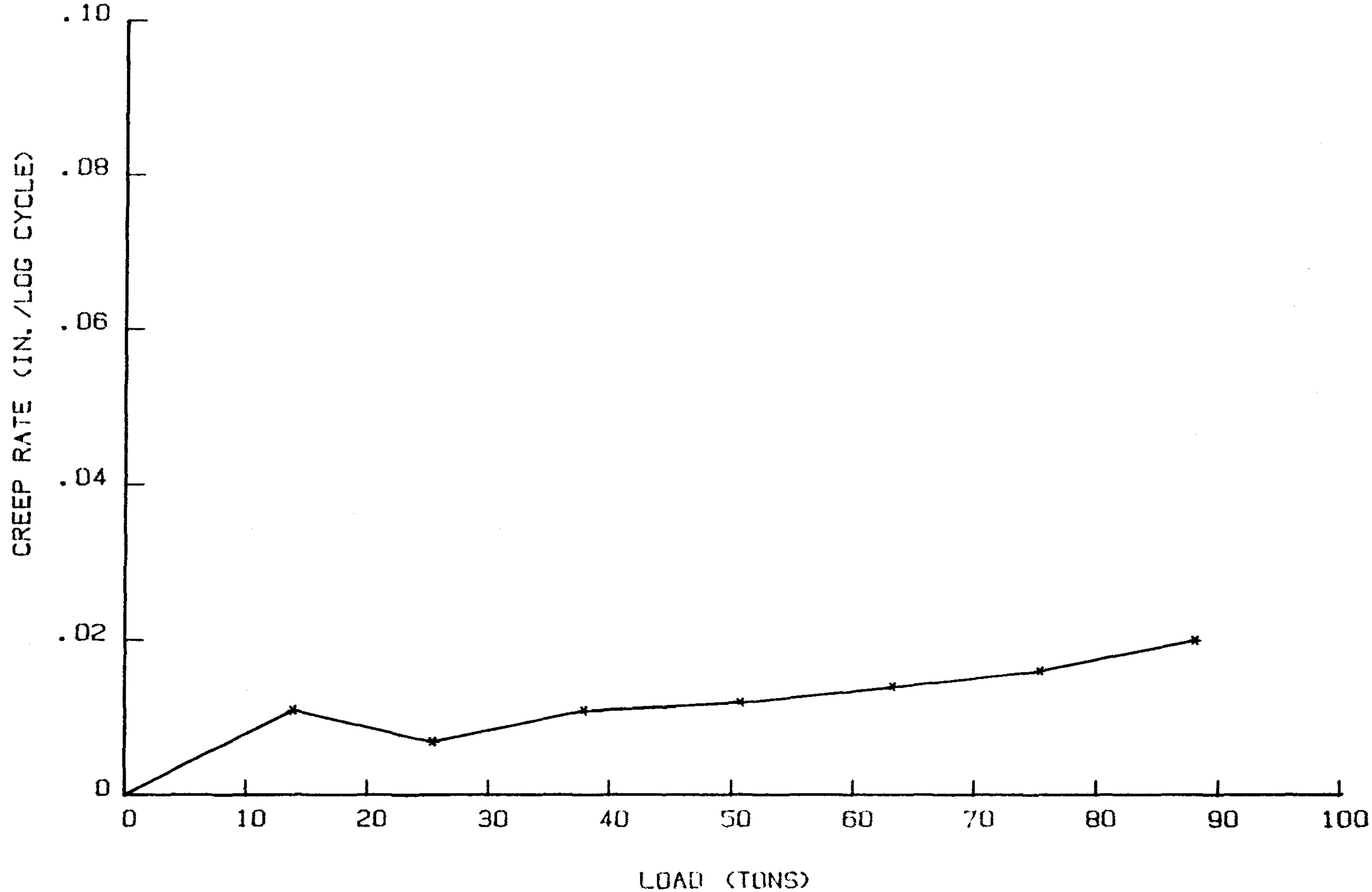
(2) Deformed Metal Tube

The distribution of strain in the deformed metal tube is presented in Figure 16. The most interesting feature of the strain distribution in the deformed metal tube is the presence of compressive strains at the front of the anchor.

This also indicates that the load applied at the anchorhead was transferred down the tendon to the front of the anchor and then transferred to the soil; otherwise, compressive strains could not exist at the front of the anchor. Also, note that the location of the neutral axis (transition point from compression to tension) gradually moved down the anchor as the load increased.

The time-dependent distribution of strain in the deformed metal tube shown in Figure 16 differed significantly from that in the tendon (see Figure 14). During the load-hold at 88.1 tons the deformed metal tube exhibited a significant increase in strain in the back of the anchor. This

Figure 25. Creep Rate Versus Load for Tieback 18-2

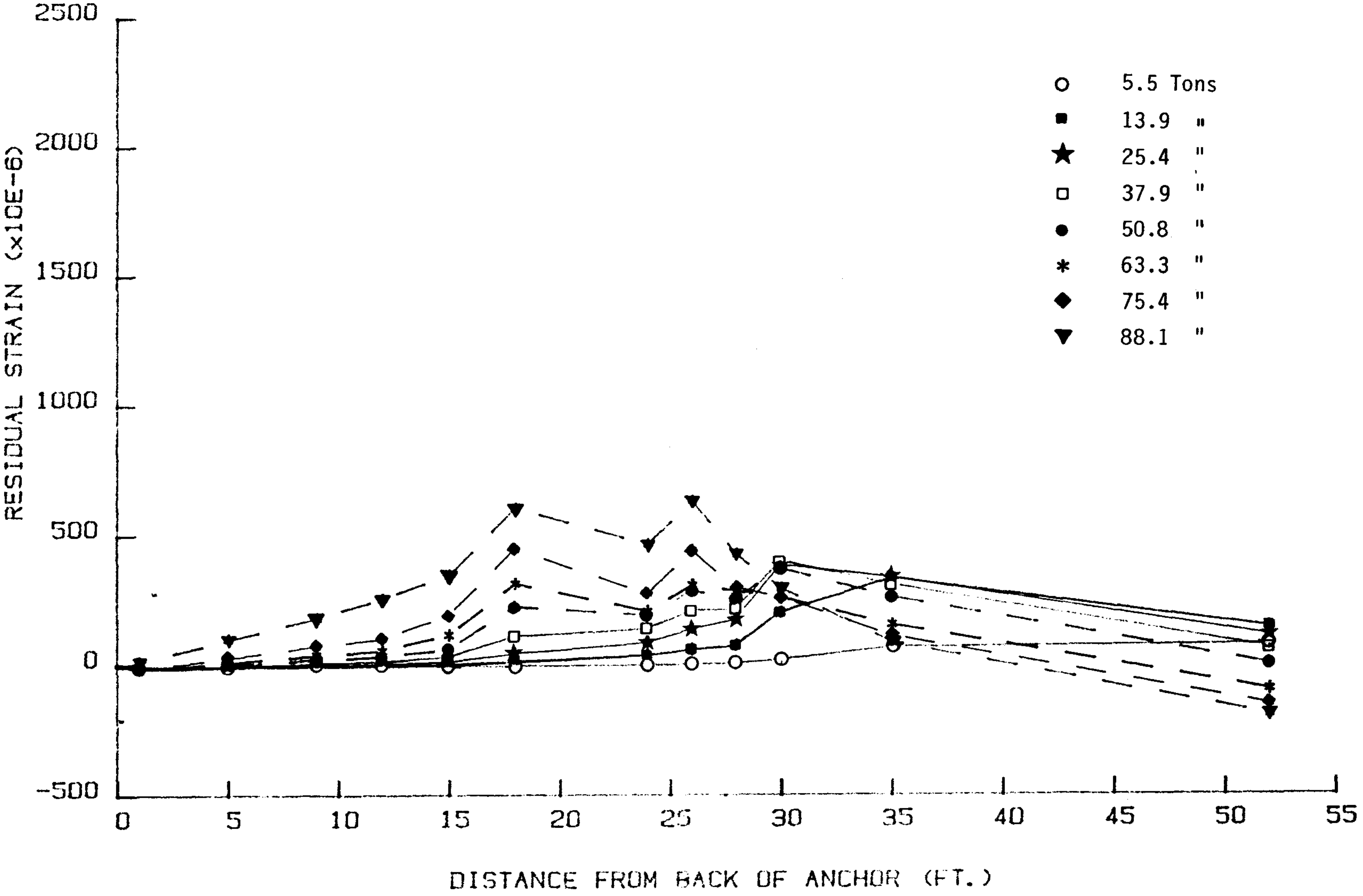


Schnabel Foundation Company on instrumented straight-shafted tiebacks anchored in clay have shown that tieback failure is usually associated with residual anchor movements greater than 0.5 inches. The load-displacement response of tieback 15-8 (see Figure 10) was much more elastic than that of the straight-shafted tieback (see Figure 7). Both the elastic load-displacement response and the relatively small residual anchor movements of tieback 15-8 indicate that additional capacity existed beyond 88.1 tons.

The time-dependent response of tieback 15-8 is presented in Figures 12 and 13. At the maximum test load of 88.1 tons, the creep rate was essentially constant at .043 inches/logarithmic cycle in both the third logarithmic cycle (100 to 1,000 minutes) and the fourth logarithmic cycle (1,000 to 10,000 minutes). However, this creep rate was double the creep rate that occurred in the second logarithmic cycle (10 to 100 minutes). The change in creep rate from the second to the third logarithmic cycle has a plausible explanation. In jacking the load up to the final test load of 88.1 tons, the load was inadvertently maintained constant for a period of four minutes at a load of 86.4 tons. Once the error was discovered, the load was immediately jacked up to 88.1 tons and thereafter maintained constant. It is apparent that the creep movement of the tieback at 86.4 tons, and the effect of the previous loads, influenced the creep performance of the tieback at a load of 88.1 tons by reducing the amount of creep that occurred in the first two logarithmic cycles. The influence of load history on the time-dependent performance of tiebacks anchored in cohesive soils is been noted in other tests on instrumented and non-instrumented tiebacks.

The creep performance of tieback 15-8 showed no signs of imminent failure. The maximum creep rate of 0.043 inches/logarithmic cycle is relatively small for a tieback anchored in cohesive soil. Even at the maximum test load of 88.1 tons, the creep behavior was essentially linear with respect to the logarithm of time after the second logarithmic cycle. Temperature changes can affect the dial gauge support, tendon, and hydraulic fluid in the test jack. Temperature-induced movements may become significant when anchor creep is small. Fluctuations in creep movement observed during the fourth logarithmic cycle at the 88.1 ton load increment are attributable primarily to temperature changes between day and night. While trying to maintain the load constant at 88.1 tons, it was necessary to adjust the jack pressure in response to temperature changes.

Figure 27. Distribution of Residual Strain in the Tendon of Tieback 18-2



2-A-27

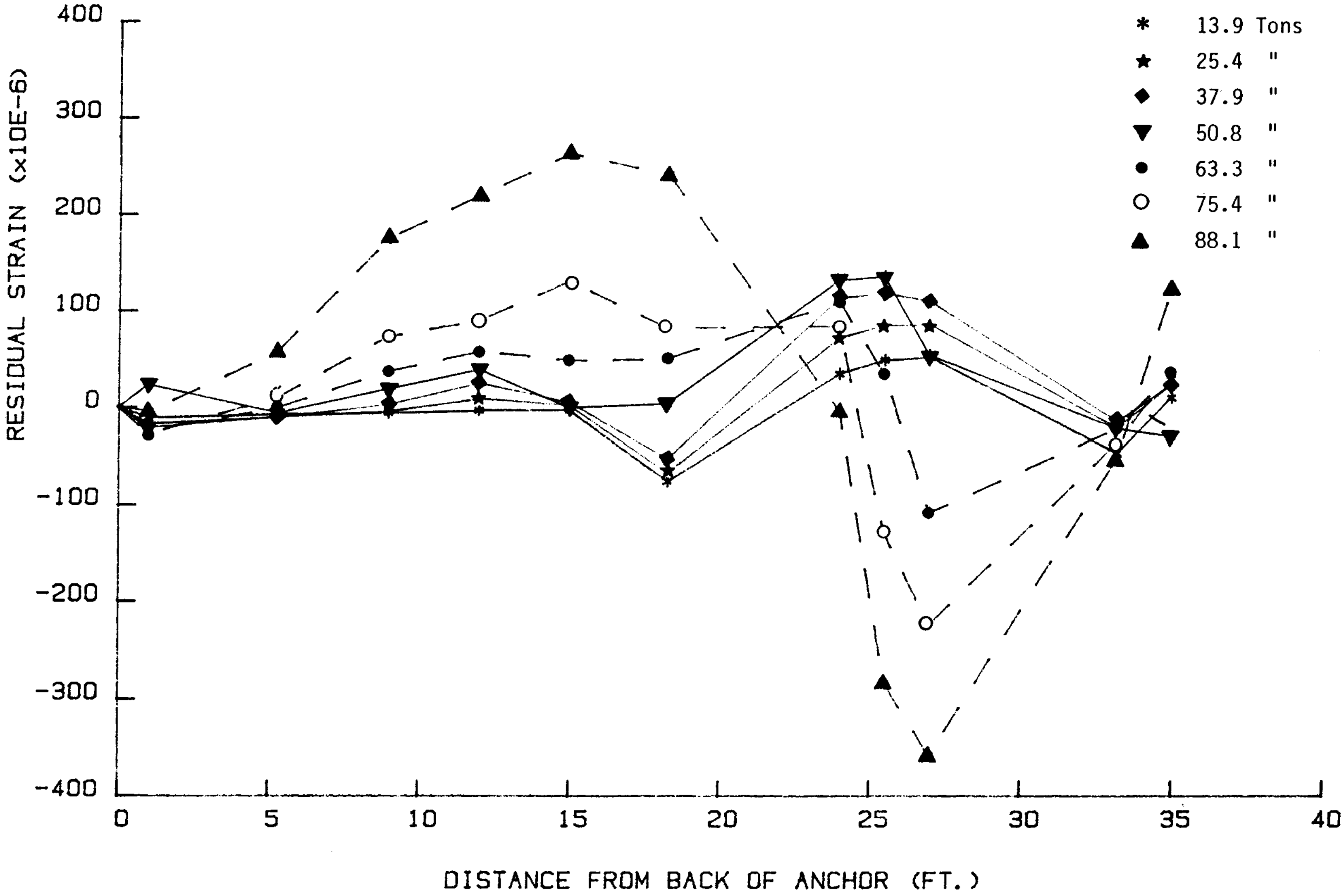
Residual anchor movement is a very useful measure of tieback performance, since it is a function of both the tieback construction technique and the soil in which the tieback is anchored. Reese (1976) highlighted the fact that the displacements required to mobilize ultimate capacity in bearing are much larger than the displacements required to mobilize ultimate capacity in shear in most soils. A large number of tests conducted on tiebacks using extensometers have verified that the anchor moves when loaded and partially rebounds when unloaded, and that the residual anchor movement can be measured reliably in a cyclic test such as the performance test (Schnabel, 1982). Since larger anchor displacements are required to mobilize greater tieback capacity (for loads less than ultimate), residual anchor movement tends to increase as the applied load increases. To date, no acceptance criteria exists to evaluate tieback performance based on residual anchor movement.

Time-dependent tieback displacements at constant load are used to evaluate the long-term performance of a tieback. The time-dependent or creep movements typically are plotted on a semi-logarithmic scale as a function of time (see Figure 8). For each load, the creep movement per logarithmic cycle (or creep rate) is determined by computing the slope of a best fit straight line drawn through the data points. If a straight line cannot be drawn, the creep rate is determined by computing the maximum slope formed by the data points. Creep rate can then be plotted as a function of load. To date, tieback specialists cannot agree on the acceptance criteria to be used to evaluate tieback performance based on time-dependent movements.

The preceding discussion has focused its attention on the measurements used by the engineering profession to evaluate a tieback's performance. Elastic movements, residual anchor movements, and time-dependent movements are used to assess both the short-term and long-term load carrying capability of a tieback. The ensuing discussion presents a step-by-step description of the analysis used herein to evaluate a tieback's performance based on computed performance which can provide insight into the measured performance.

The most obvious contribution of the computed performance of each tieback is the determination of the distribution of strain, and therefore load[1], along the tieback for a given applied load and a given time (see Figure 9). The shaded area represents the total amount of strain that developed during a particular load-hold. The strain distribution in the unbonded length can verify easily the provision of a minimum desired unbonded length--a criteria used by the entire engineering profession to evaluate a tieback's performance. The installation of strain gauges at critical locations along the unbonded length, e.g., outside and inside the face of the tied back structure and at the transition from the unbonded length to the anchor length, can provide valuable information concerning the influence of friction and bending on the distribution of strain in the unbonded length. Typically the

Figure 29. Distribution of Residual Strain in the Deformed Metal Tube of Tieback 18-2.



In setting up each tieback test, extreme care was exercised to ensure that the bearing plates, load cell, and jack were aligned properly. This was done in order to minimize the effects of friction and eccentric loading due to mis-alignment. Prior to the start of each test, each strain gauge was calibrated internally and initialized to give a "zero" reading to which all subsequent strain gauge readings were referenced. A 5.5-ton alignment load was then placed on the tieback to seat and align the testing hardware. At this time, the dial gauges were set up to measure movement of the anchorhead. All subsequent dial gauge measurements were referenced to the initial "zero" reading at 5.5 tons. The test proceeded by setting the desired load on the Vishay P-350A strain indicator. The jack was then pumped up to the desired load. At this time, the dial gauge and all strain gauges were read. If the next load was to be held constant, the jack was pumped up to the desired load within one minute. An initial reading was taken when the desired load was first reached. The one-minute reading of the load-hold corresponded to the time that had elapsed since the pump was first activated. That is, the time lapse between the initial "zero" reading and one-minute reading of the load-hold was always less than 60 seconds. During the load-hold, it was necessary to adjust the hydraulic pressure in the test jack in order to maintain the load constant. Load was decreased to the alignment load after each load-hold by setting the Vishay P-350A strain indicator to the alignment load and opening up the release valve on the pump. At the end of each test, the load was reduced to zero and all strain gauges were read and checked for internal calibration. Subsequently, each tieback was locked-off at 40 tons by reloading the tieback to 40 tons, tightening a nut against the bearing plate, and releasing the load from the jack.

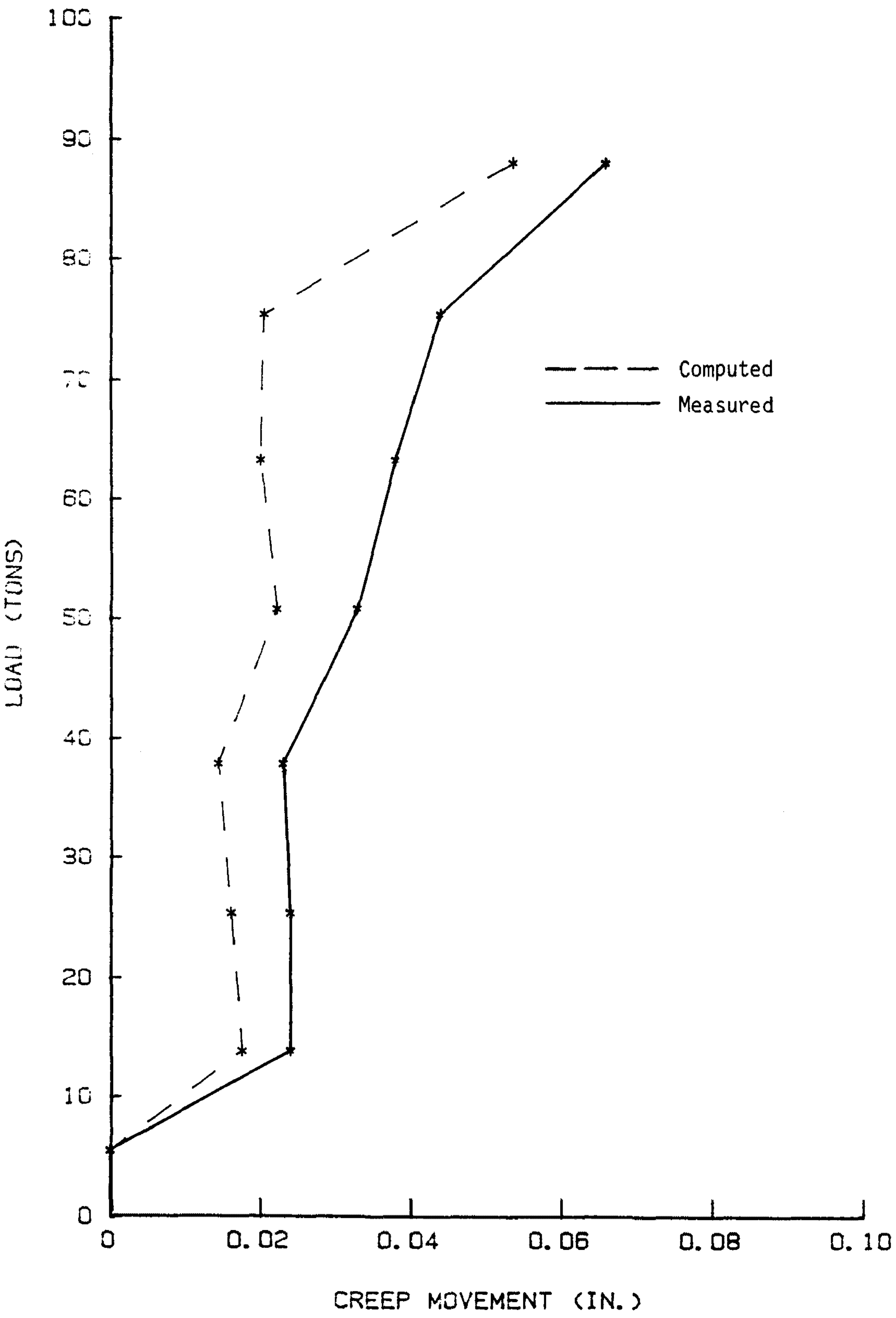
B. Two-year Monitoring Program

The two-year monitoring program consisted of monitoring the change in load in each of the tiebacks at least once every three months during this period of time. The strain gauge readings taken immediately after each of the tiebacks was locked-off at 40 tons served as the initial zero to which all subsequent readings were compared.

The procedure used to take a set of readings is described below. The dummy gauge was used to zero each of the 30 channels on the Vishay 220 readout unit prior to visiting the site. At the site, the dummy gauge was used again to re-zero the 30 channels. Subsequently, the strain gauges on both instrumented tiebacks were read within a period of 60 minutes.

Note that the strain gauge readings taken in June of 1982 are omitted from this report because they were considered unreliable. The reason for their un-reliability was traced to the failure to pot the military connectors which served as the junction between the strain gauges and the Vishay 220 readout unit. Grouting of

Figure 31. Measured Versus Computed Creep Movement of Tieback 18-2



- a calibrated 150-ton capacity load cell to monitor load at the anchorhead during the short-term investigation;
- a calibrated test pump and jack to load the tieback;
- two dial gauges to monitor movement of the anchorhead relative to a fixed point to the nearest .001 inch;
- a Vishay 220 digital strain readout unit capable of reading and recording the response of 30 strain gauges continuously at a rate of one strain gauge per second;
- a Vishay P-350A strain indicator to monitor load in the load cell; and
- a dummy gauge used to zero the Vishay 220 digital strain readout unit prior to each set of readings over the two-year monitoring period.

In general, the strain gauges performed quite well during the initial two-week test program. Strain gauge failure occurred in less than 17 percent of the strain gauges. However, the performance of the same strain gauges over the two-year monitoring period was less satisfactory. This type of performance is to be expected from electrical resistance strain gauges which are very accurate but, due to electrical aging of components and creep in bonding agents, tend to "drift" with time. Prolonged exposure to a hostile environment over a period of time was probably the reason for the majority of the strain gauge failures.

Since the original intention of the two-year monitoring program was to monitor changes in tieback load, the strain gauges of primary interest were those located in the unbonded length of each tieback. Fortunately, 8 out of the 10 strain gauges located in the unbonded length functioned properly for the entire two-year monitoring period. Therefore, the two-year load change data presented in Section VI is considered reliable and accurate provided that certain limitations associated with the use of strain gauges are recognized. First, the tendency of electrical resistance strain gauges to "drift"" with time influences the absolute magnitude of load measured more so than the trend in load change. Both the strain gauges installed on the two tie backs and the dummy gauge used to zero the Vishay 220 unit were affected by this "drift" phenomenon. Second, the variation in Young's modulus of elasticity of the bar required to convert strain to load introduces error to the absolute magnitude of load measured. The conversion of strain to load required the use of Equation (1):

$$P = eEA \qquad \ldots(1)$$

where P = load

e = strain ($\times 10^{-6}$)

E = Young's modulus of elasticity

A = cross-sectional area of the tendon

REPORT NO. 3

PERMANENT TIEBACK ANCHORS IN COHESIVE SOIL

DEMONSTRATION SITE
SR-90 (SR-5 to CORWIN PLACE)
SEATTLE, WASHINGTON

by

Hart Crowser and Associates, Inc.

WASHINGTON STATE
DEPARTMENT OF TRANSPORTATION
MARCH 1984

B. Installation

The installation procedure consisted of four major operations (see Figure 4):

- o punch an 8-1/2-inch diameter hole through the wall;
- o drill hole through soil to desired depth and insert TMD assembly;
- o grout the deformed metal tube to the soil; and
- o grout tendon to the deformed metal tube.

An 8-1/2 inch diameter hole was punched through the wall by using an 8-inch diameter down-the-hole hammer with a button bit attached to its end. Upon encountering the first section of rebar 2-1/2 inches from the front face of the wall, a laborer used a torch to burn off the rebar. The second section of rebar, located near the back face of the wall, was burned out with a lance.

Once the hole had been punched through the wall, the hole was drilled to a final depth of 57 feet using a 6-inch diameter auger with a clay bit attached to its end. From the experience gained during the installation of the 96 non-instrumented tiebacks, it had been decided the hole could be drilled uncased. This greatly simplified the drilling operation. Very soft, soupy soil with running sand was encountered while drilling the hole for tieback 18-2. Fortunately, this presented no problems during the installation of the tieback.

Once the auger had been extracted from the drill hole, the deformed metal tube, with grout valves located every 3.3 feet along its length, and the PVC pipe were placed in the hole. The assembly was centered in the hole by means of plastic centralizers attached to the deformed metal tube. After the TMD assembly had been inserted in the drill hole, a 3-inch diameter double packer was positioned inside the deformed metal tube opposite the bottom grout valve. The packer was inflated and grout subsequently was pumped through the last valve until the grout could be seen exiting from the drill hole. The packer was removed and the inside of the TMD assembly was flushed with water and blown out with compressed air until the effluent was fairly clear. The grout was then allowed to set up overnight.

The post-grouting operation began on the next day. The packer was positioned inside the deformed metal tube opposite the bottom grout valve and inflated. Grout was pumped slowly through the valve until either a maximum grout pressure of 40 bars was reached or one bag of cement had been pumped through the valve. The quantity of grout pumped through each valve was limited, because

TABLE OF CONTENTS

PERMANENT TIEBACK ANCHOR DEMONSTRATION PROGRAM
SR-90 (SR-5 TO CORWIN PLACE)

I. INTRODUCTION

This report provides the results of the first 100 days of data of the SR-90 Permanent Tieback Anchor Demonstration Program. The purpose of the study was to establish creep characteristics of non-pressure grouted tieback anchors installed in the Seattle over-consolidated silts and clays in a predesign testing program. The work included on-site explorations, laboratory testing, installation of nine tieback anchors, and load-creep testing of the anchors. Our work has been accomplished in accordance with our final design submittal authorized in Agreement Y-2573 Supplemental dated June 24, 1983 (FHWA Work Order DTFH 71-83-931-WA-04).

Selection of the tieback anchor for this study was based on the standard of practice throughout the Pacific Northwest. With only minor exceptions, the tieback anchors used throughout the area are non-pressure grouted anchors which typically ranged in diameter from 12 to 18 inches. For this study, 12-inch nominal diameter, 20-foot long, non-pressure grouted anchors were utilized.

The site of the study is located at the western end of the SR-90 project, near the SR-90, SR-5 interchange, as shown on Figure 1. The site was selected to utilize one of the existing cantilevered cylinder pile walls as a reaction to the tieback anchor loads. The 10-foot diameter reinforced concrete cylinder piles provided a very rigid reaction, and for all practical purposes eliminated the influence of backfill loads from the testing program.

The tieback testing for this study was completed in two phases. In the first phase, two anchors were loaded incrementally to pull-out. Each load level was held for up to an hour, while monitoring bar displacement to establish creep versus load characteristics. Based on that data, anchor loads were selected for long term creep monitoring. During the second phase of the test, seven anchors were loaded to and locked off at various percentages of the ultimate anchor capacity (20 to 60 percent). The anchors are currently being monitored to establish long term creep versus time relationships as a function of anchor load.

The body of the report provides a discussion of the various aspects of the project, including subsurface exploration and laboratory testing, tieback installation, ultimate anchor testing, and creep data interpretation.

II. SUMMARY

The following is a summary of the conclusions made within this report. The body of the report should be consulted for discussion of each point. A separate supplemental letter report for the SR-90 project presents our detailed conclusions and recommendations for design of permanent production anchors founded within the over-consolidated silts and clays at the site of this study.

- The two ultimate anchors each pulled out at a tension of approximately 120 kips. This load corresponds to an average adhesion over the nominal surface area of the 12-inch diameter anchor of 1.9 ksf.

- The critical creep tension (Tc′) appears to be in the range of 70 to 80 kips, which is on the order of two-thirds of the ultimate capacity of the anchor. For the purpose of determining lock-off loads, a critical creep tension (Tc′) of 75 kips was assumed.

- The maximum working tension (Tuw) was calculated at 0.8 Tc′, or 0.8 x 75 = 60 kips.

- Seven tieback anchors were locked off at loads of 24 kips, 42 kips, 62 kips and 70 kips, which corresponds to 40 percent, 70 percent, 103 percent, and 117 percent of the calculated maximum working tension(Tuw) of 60 kips.

- Long term monitoring to date indicates displacement creep coefficients of 0.01 inches per log cycle of time, or less, for all of the anchors. This is well below the FHWA maximum acceptable level for these anchors of 0.08 inches per log cycle of time.

- We recommend that monitoring of the anchors continue for a period of at least 2 years from the lock-off date. Based on that criteria, the monitoring would extend to approximately 1 x 106 minutes (2 years) which is approximately one log cycle of time past the current duration of 1.4 x 10^5 minutes (100 days). The recommended completion date for monitoring would be October 1985 or later.

III. SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Two hollow-stem auger borings were advanced at the site, at the locations shown on the Site and Exploration Plan, Figure 2. Samples were obtained using the Standard Penetration Test split-spoon sampler, and using thin wall Shelby tubes. Laboratory testing on the samples included moisture content, Atterberg limits, and triaxial testing. The results of the explorations and laboratory testing are summarized below.

- The subsurface conditions consist predominantly of very stiff to hard, gray clayey silt and silty clay. Slickensides and fracture zones were noted. We have typically classified this soil as the Older Marine unit in our reports for the SR-90 project.

- The Plasticity Index (PI) of the soil ranged from 17 to 39 percent, with a majority of the samples in the range of 24 to 32 percent. The liquidity index ranged from 0.09 to 0.61. Soil with a PI greater than 20 percent generally requires an assessment of creep potential. Moisture contents of samples of drill cuttings from the anchor zones of the tiebacks, taken during tieback drilling, were in the range of 33 to 39 percent.

- The unconsolidated undrained triaxial tests indicated shear strengths in the range of 1600 to 5300 psf. Strain at failure was generally in the range of 2 to 4 percent. The results of strength testing within the Older Marine soils conducted for this and other projects would indicate that a definite trend of increased strength with depth does not necessarily exist. The strength of these glacially consolidated materials will tend to vary from point to point within the soil mass.

IV. TIEBACK INSTALLATION

The anchors were installed on September 20 and 21, 1983. The locations of the anchors are shown on the Site and Exploration Plan, Figure 2. Generalized subsurface profiles at the site are shown on Figures 3 and 4. These figures also show the configuration of the test program in relation to the subsurface conditions disclosed by the explorations. Figure 5, the Typical Tieback Installation, indicates the configuration and dimensions of the tiebacks as installed for this project. The following is a summary of the tieback installation program.

- The anchors were installed using a 12-inch nominal diameter LDH auger rig. The actual drilled hole diameter ranged up to 14 inches, which is typical for the method of installation. The average overall length of each tieback was 55 feet. The grouted anchor length was approximately 20 feet.

- The holes were drilled at an inclination of 25 to 30 degrees from horizontal. Flatter angles of 20 to 25 degrees were not generally achievable due to site geometry limitations (site width, position of concrete footing).

- The tieback bars consisted of 1 3/8-inch diameter Dywidag bars. The bars were installed and anchors grouted by open-hole methods. The anchors were not pressure-grouted.

- The anchors were installed in the Older Marine unit (hard silts and clays). The holes were generally dry with occasional localized saturated zones. Tieback TB-5 encountered a saturated zone, apparently within the unbonded length. Flowing water exited this hole shortly after drilling, and continued to flow following grouting of the anchor. It appears that the presence of this seepage has no discernible affect on the load-creep performance of this anchor.

- Anchor instrumentation consisted of rod extensometers, vibrating wire strain gages, and hydraulic load cells.

V. PHASE I TESTING: PULL-OUT AND SHORT TERM CREEP

The first phase of anchor testing was accomplished on the two ultimate anchors, tiebacks U-1 and U-2, and on tieback TB-6. The purpose of Phase I was to obtain short-term creep data at several load levels, up to pull out (To) of the anchors, as well as establish the pull out load (To) of two anchors. The tieback loads were increased incrementally, with each load level held as described below, and monitored for displacement. The loads on the two ultimate anchors were incrementally increased to pull-out (To). Tieback TB-6 was loaded to the interpreted maximum working load of 60 kips, and held for 72 hours. The ultimate anchors (U-1, U-2) included two Dywidag thread bars. Only one of the bars was loaded during the testing program since the anchor pulled-out prior to reaching the capacity of a single bar. The effect of additional steel (second bar) within the anchor zone on load transfer has not been evaluated. Since both bars were located near the center of the anchor, and since the cross-sectional area of steel at any point along the length of the anchor was constant we do not readily see a potential for differences in trends of development of creep between single bar and double bar anchors. Therefore, in evaluating the test data, the assumption was made that the presence of the second bar in the ultimate anchors had no affect on the results.

A. Ultimate Anchor Testing

The ultimate test anchors (U-1 and U-2) were loaded to pull-out (To) in 10 to 20 kip increments. Each load level was held for up to an hour and monitored for displacement. (The interpretation of the displacement data is discussed below.) The pull-out load (To) was defined as the highest load which could be sustained by the anchor. Once the pull-out load was observed, displacements on the order of several inches were noted along with decreasing anchor loads.

The load displacement curves for the ultimate test anchors, as well as for TB-6, are presented as Figure 6. The pull-out load of 120 kips was observed for both tieback anchors U-1 and U-2. The pull-out load corresponds to an average adhesion of 1.9 ksf as calculated over the nominal surface area of the 12-inch diameter anchor.

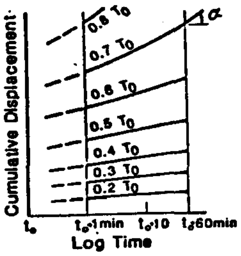
B. Creep Testing

For each load level, a plot of the anchor deflection versus the log of time was made (Creep Curves, See below and Figures 7, 8, 9, and 13). The slope of each curve, designated the creep coefficient (α), was then plotted versus the corresponding load level. The typical shape of the creep coefficient curve in this study is nearly horizontal for loads lower than approximately 50 kips, at values of usually less than 0.03 inches per log cycle of time. At higher loads the value of increases with each load level, with the creep coefficient curve sweeping upward (See Figures 7, 8, and 9).

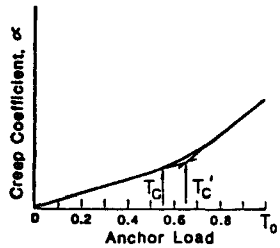
The critical creep tension for each anchor was determined from the creep coefficient plot. Two methods of calculating the critical creep tension were utilized, as follows:

Tc: The load level where the slope of the creep coefficient curve becomes tangent to the initial straight line portion of the curve.

Tc': The load level defined by the intersection of the straight line extended from the initial portion of the curve, with the extension of the straight line extended from the latter portion of the curve.



A. CREEP CURVES



B. CREEP COEFFICIENT PLOT

(Note: Figure 7 also presented following text)

The following is a summary of the sequence of creep testing for the Phase I anchor testing.

- <u>Tieback U-1</u>: Short term creep data was collected while testing ultimate anchor U-1 to pull-out. The creep curves and critical creep tension curve are presented on Figure 8. The data indicates a critical creep tension of : Tc' = 79 kips, Tc=80 kips.

- <u>Tieback U-2</u>: The creep data for the test is presented on Figure 9. The creep coefficient showed a significant rise from the first 70 kip load to the 80 kip load, increasing from 0.02 to 0.06. The data indicated critical creep tension of: Tc'=75 kips, Tc=70 kips. Following the 80 kips short term hold, the load was decreased to 70 kips for the long term hold. The load was maintained for 32 hours, at which time an equipment malfunction resulted in the load increasing to 105 kips. The creep data obtained during the first 32 hours did not plot as a straight line, but rather had an upward curvature on the displacement vs. log-time curve. The curvature may be due to the fact that the anchor was preloaded to 80 kips prior to the 70 kips long term load, or because the load was in excess of the critical creep tension. Sufficient information was not available to establish the cause of the curvature, so a second long term creep test at a reduced load of 60 kips was accomplished on Tieback TB-6. Following the equipment malfunction, tieback U-2 was loaded in 5 to 10 kip increments to a pull-out load of 120 kips.

- Tieback TB-6: The anchor was loaded up to 60 kips in 10 kip increments, with each load held for 1 hour to collect creep data. The 60 kip load was held for 72 hours. The creep data collected during the test is presented on Figure 13. The long term hold at 60 kips plotted as a straight line extension of the initial 1 hour data. The results indicate that 60 kips is below the critical creep tension.

VI. PHASE II TESTING: LOCK-OFF MONITORING AND LONG TERM MONITORING

The purpose of the Phase II testing is to establish the creep characteristics of tieback anchors over the long term (up to 3 years) for comparison to the short term data (1 hour to 72 hours) collected during Phase I. Seven anchors have been locked off at various percentages of the maximum working tension to collect the desired data. The information presented in this report represents the first 100 days of creep data. Data which was collected and analyzed over the next 2 to 3 years by the Washington State Department of Transportation is included in separate report.

A. Lock-Off Loads

The first step in establishing lock-off loads was to determine the maximum working tension (Tuw) of the anchors, The maximum working tension is the highest anchor load which will not result in excessive creep or excessive movement as defined by load/ deflection criteria. Based on the results of the tests it appears that creep criteria governs maximum working tension. The criteria used for selecting Tuw are stated below:

B. Creep Criteria

- Tuw is less than the Tc or Tc′ of the ultimate anchors
- The creep curve of the Tuw load plots as a straight line (not curved upward), with a creep coefficient (α) less than or equal to 0.08 inches per log cycle of time
- Tuw is no greater than: 0.9 Tc or 0.8 Tc′

The results from Phase I (Tiebacks U-1, U-2, see Figures 8 and 9) indicated Tc in the range of 70 to 80 kips, and Tc′ in the range of 75 to 80 kips. The above data indicates a Tuw of 68 to 72 kips based on the 0.9 Tc criteria, and a Tuw of 60 to 64 kips based on the 0.8 Tc' criteria. The creep coefficients (α) were less then 0.08 inches per log cycle of time for all of the loads up to 80 kips. With the exception of the second 70 kip load on Tieback U-2, the creep curves plotted as straight lines on the semilog graphs.

C. Load/Deflection Criteria

Load/Deflection criteria for anchor tests is based on elastic elongation of the tendon or bar during stressing. The criteria for minimum and maximum deflection is described in the draft of the FHWA manual for Demonstration Project Number 68, Permanent Ground Anchors. The minimum deflection of the anchor head is 80 percent of the theoretical elastic elongation of the free, unbonded tendon or bar length. The maximum deflection at the anchor head is the theoretical elastic elongation of the tendon or bar length from the jack to the center of the bond length. The center of gravity of the bond stress should not be beyond the midpoint of the bond length, if the maximum deflection criteria is met.

The anchor test data was evaluated for these criteria based on a steel area of 1.485 square inches for the bar and a modulus of elasticity, E, of the tendons, 29 x 10^6 p.s.i, A free, unbonded bar length of 32 feet, and a length of 42 feet to the center of the bond length were used. The anchor tests all met the minimum deflection criteria. Five of the tests exceeded the maximum deflection criteria by the following percentages. Of the 60 and 70 kip tests, two of five exceeded the criteria, by 10 and 12 percent of the theoretical elongation. Of the 42 kip tests, one of two exceeded the criteria, by 28 percent. Of the 24 to 28 kip tests, two of three exceeded the criteria, by 37 and 65 percent.

A possible explanation of the deflections observed which exceeded the maximum criteria could be inaccuracy of relative deflection measurements due to straightening of the system with increasing loads. Another could be inaccuracy (apparently at low loads) of load measurement. Because the percentage of deflection in excess of acceptance criteria is higher for the lower loads, it appears that the center of gravity of bond stress is not beyond the midpoint, as this would be unlikely for the 24 kip tests.

Further analysis of this phenomena will be made during review of load transfer mechanisms as indicated by additional strain gages installed within the anchor zone of selected anchors. This analysis will be completed and forwarded under separate cover.

The selected value of Tuw for determining lock-off loads was 60 kips. The value was selected following review of the data and discussion with representatives of both FHWA and WSDOT. The actual tieback lock-off loads are presented on the following table.

Table 1 - Tieback Anchor Lock-Off Loads

Tieback Anchor Number	Lock-Off Load in Kips	Lock-Off Load in Percent of Maximum Working Tension (Tuw = 60 Kips)
TB-1	62	103
TB-2	44	73
TB-3	42	70
TB-4 (Initial)	28	47
TB-4 (Final)	70	117
TB-5	25	42
TB-6	63	105
TB-7	25	42

These loads were locked off on the seven demonstration anchors on October 10 to 13, 1983. The lock-off load for Tieback TB-4 was increased from 28 to 70 kips on January 6, 1984, in order to provide additional creep data at higher loads. Each of the anchors was loaded in 10 kip increments to the lock-off load. Each load level was held for up to 1 hour. The creep data collected during the incremental loading is presented on Figures 10, 11, 12, and 13. Figure 14 presents a summary of the creep coefficient ( ) versus tieback tension for the short term tests, the long term tests, and the lock-off tests. The data generally indicates the critical creep tension is below 80 kips and above 60 kips.

The tieback anchors were locked-off by simply tightening the lock nut on the Dywidag bar, removing the jacking system, and allowing the load to transfer directly from the lock nut to the waler. The walers reacted against the relatively rigid, 10-foot diameter cylinder pile wall shown on Figures 1 through 5. It was assumed that the rigid nature of the cylinder pile reaction would eliminate the influence of backfill soil pressures, and wall movements, on the testing program. Movements of the wall were not monitored.

D. Long Term Monitoring

After the load is locked-off, it is no longer possible to measure displacement at the top of the tieback. The waler and the rigid cylinder pile wall on which it rests would not tend to displace significantly with movements of the anchor, since the wall has been designed to stand as a rigid cantilever wall. The testing system was designed to monitor tieback deflection through the use of rod extensometers extending from the anchor zone, along with vibrating wire strain gages on the bar.

The rod extensometers did not provide the desired accuracy. Without direct displacement measurement capabilities, long term monitoring was based on the drop in the tieback load over time and strain measured in the no-load zone.

The load was monitored with the hydraulic load cell at the waler, and with a vibrating wire strain gage spot welded to the Dywidag bar. The vibrating wire strain gage was calibrated on the 1-3/8 inch diameter Dywidag bar. A strain change of 20 microstrain (20 x 10^{-6}), represents a load change in the bar of 1.0 kips. The deflections (Δ) in the no load zone were estimated from the vibrating wire strain gage data as follows:

$$\Delta = \epsilon \times L$$

where ϵ is the strain recorded from the gage, and L is the 32 foot length of the no load zone. Based on this relationship, a deflection (Δ) of 0.01 inches would be represented by a recorded strain (ϵ) of 26 microstrains. That level of strain would also represent a 1.3 kip change in the load of the anchor, based on the above calibration.

Following lock-off, the readings were obtained on a one to two week basis, with the exception of the first week, where data was collected on the first, second, and fourth day. The monitoring to date has been based primarily on the vibrating wire strain gage data.

Based on published information of time-dependent relaxation of prestressing steel presented in FHWA Report Number FHWA/RD-82/047, and on verbal information from Dywidag System International, we have not subtracted an allowance for load relaxation (or "steel creep") from our long-term creep monitoring data. The published information, applicable only to bars stressed to 70 percent of ultimate strength (140 kips for this project), indicates load relaxation would be on the order of 4 percent for the 100-day duration of the monitoring to date. Dywidag representatives inform us, however, that at loads lower than 70 percent of ultimate (bars for this project are stressed at 10 to 30 percent of ultimate), load relaxation would be minimal and probably less than 1 percent. In the absence of published data applicable to this project, we have not considered load relaxation in our data due to its anticipated low magnitudes.

The load dissipation observed during long term monitoring is due to creep of the anchor. Load dissipation to a small extent occurs in the anchors as the anchor creeps under the applied load, and the strain in the bar is reduced. The creep coefficients measured during long term monitoring therefore are affected by the reduction in load that occurs as a result of anchor creep, and are lower than the short term creep coefficients.

To date, load dissipation has been relatively small, and little if any, affect on creep coefficient has been noted. It is not known at what level of load dissipation the coefficients would be significantly affected. For purposes of this program the tiebacks should be restressed up to the initially applied lock-off loads if the tieback load dissipates by more than 10 percent of the lock-off value. If load dissipation on the order of 10 percent is observed, we recommend that one of the tiebacks at each load be allowed to remain at the reduced load, while the other tieback is restressed back to the original load level. This program of monitoring dissipating loads could provide additional information on the effects of load dissipation on creep coefficient. The value of 10 percent is essentially arbitrary based upon the overall accuracy implied in the design and construction of tieback systems.

The plot to date of load versus log-time for all of the anchors is presented on Figure 15. Adjacent to each curve is the calculated creep coefficient α , or both, in terms of displacement (inches/ log cycle of time) and load drop-off (kips/log cycle of time). Because load dissipation is linearly proportional to change in strain (creep) of the bar the two forms of α presented in Figure 15 are simply different expressions of the same phenomenon. The conclusions based on that data are presented below, along with conclusions from other phases of the demonstration program.

VII. CONCLUSIONS

- The long term creep coefficients were generally less than the short term creep coefficients (see Figure 15). If continued long term monitoring shows that the creep coefficients do not increase, this would indicate that the short term tests give an adequate indication of long term anchor performance.

- Short term creep coefficients were measured under essentially constant loads applied by the hydraulic ram system, while long term creep coefficients were measured under loads which dissipate as the anchors creep. SR-90 production anchors will tend to undergo constant load during long-term creep as opposed to the test tiebacks where load diminishes slightly as creep deformations occur. The anticipated load loss for the test tiebacks is anticipated to be small as evidenced by the data to date, Load dissipations observed to date have generally been less than 5 percent of the originally applied loads. In our opinion, the long term creep coefficients are therefore valid for the lower load sustained by the tiebacks. Because these lower loads are generally within 5 percent of the initially applied load, it is our opinion that the affect of the load dissipation on the creep coefficients obtained in this program is relatively small.

- The long term displacement creep coefficients are within the range of 0.001 to 0.012 inches per log cycle of time. These are well below the maximum acceptable creep coefficient (FHWA) of 0.08 inches per log cycle of time.

- The deflection creep coefficient shows a slight increase with increasing load. The 25 to 30 kip anchors have a creep coefficient of 0.001 to 0.004, increasing to 0.006 for the 42 to 44 kip anchors, with a range of 0.004 to 0.012 inches per log cycle of time for the 62 to 63 kip anchors. The 70-kip anchor has a creep coefficient of 0.009 after being locked off about 20 days.

- The creep curves tended to show an initial period of virtually no movement, followed by a downward sloping curve. The break in the curve could be due to the preloading effect from the short term creep test, or other yet undescribed factors.

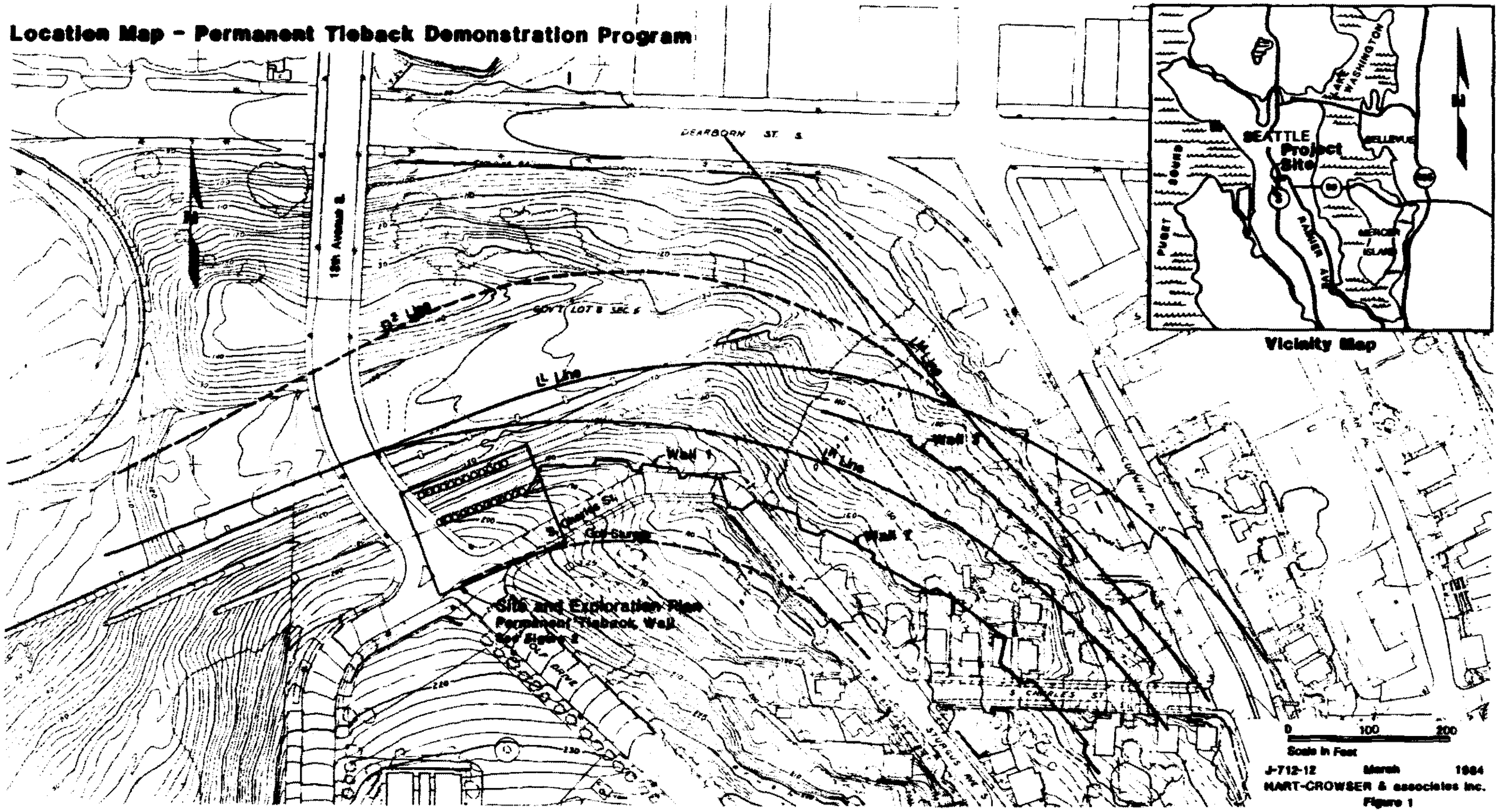
- The coefficients for load drop off tended to range between 0.1 to 0.5 kip per log cycle of time for a 25 kip anchor, and up to 1.5 kips per log cycle of time for the 63 and 70 kip anchors. None of the loads have been re-applied since locking off the anchors.

- The data obtained to date for the anchors exhibits some scatter, and continued monitoring is necessary to evaluate the long-term anchor creep coefficients. We recommend that monitoring continue at least until October, 1985 which would provide 2 years of monitoring since lock-off. As shown on Figure 15, the additional data would include approximately one log cycle of time from the 100 day mark.

- The data collected to date indicates that a 60 kip load on the demonstration anchors satisfied the maximum working tension criteria.

- According to the recent lock-off data from the 70 kip load on TB-4, 70 kips may also satisfy the maximum working tension criteria. However previous data from the ultimate test on Tieback U-2 indicates that 70 kips may not satisfy the maximum working tension criteria. With lack of additional data, our recommendation would be to continue to use 60 kips as the maximum working tension for the project.

Location Map - Permanent Tieback Demonstration Program



J-712-12 March 1984
HART-CROWSER & associates inc.
Figure 1

Site and Exploration Plan

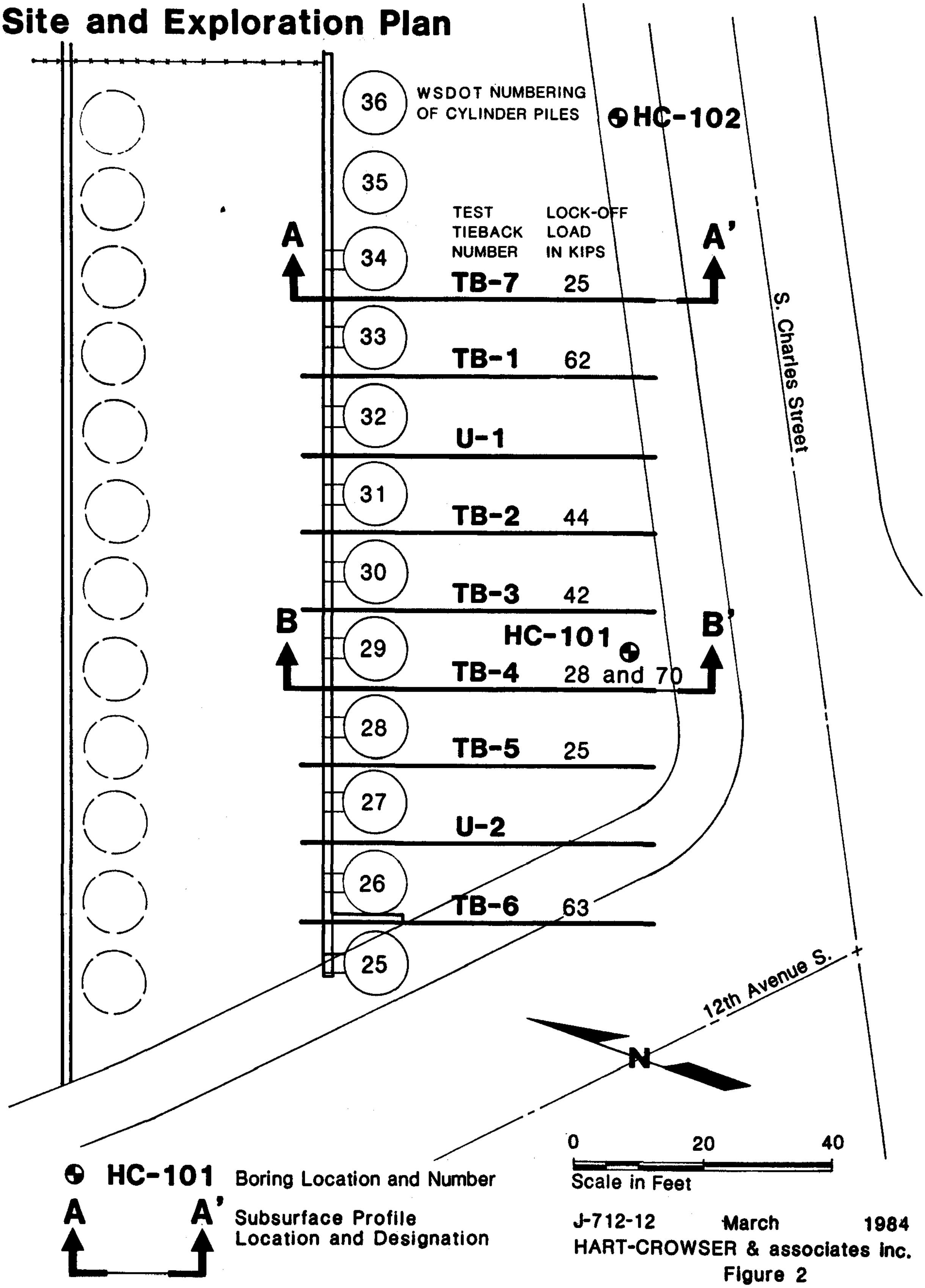


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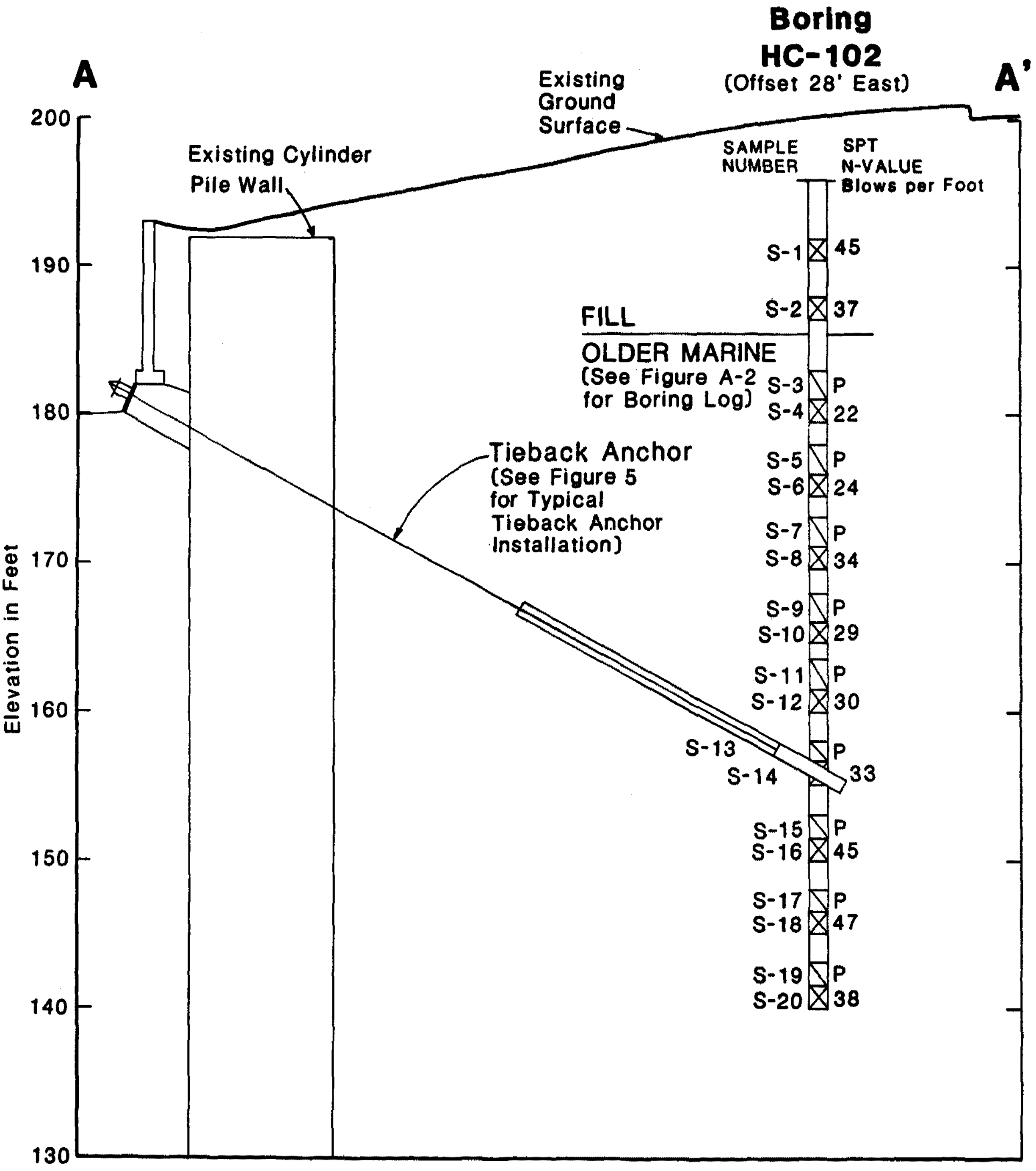
Figure 2

Generalized Subsurface Profile A-A'



☒ Split Spoon Sample Location and Number

⧅ Shelby Tube Sample Location and Number

P Sampler pushed hydraulically, not driven.

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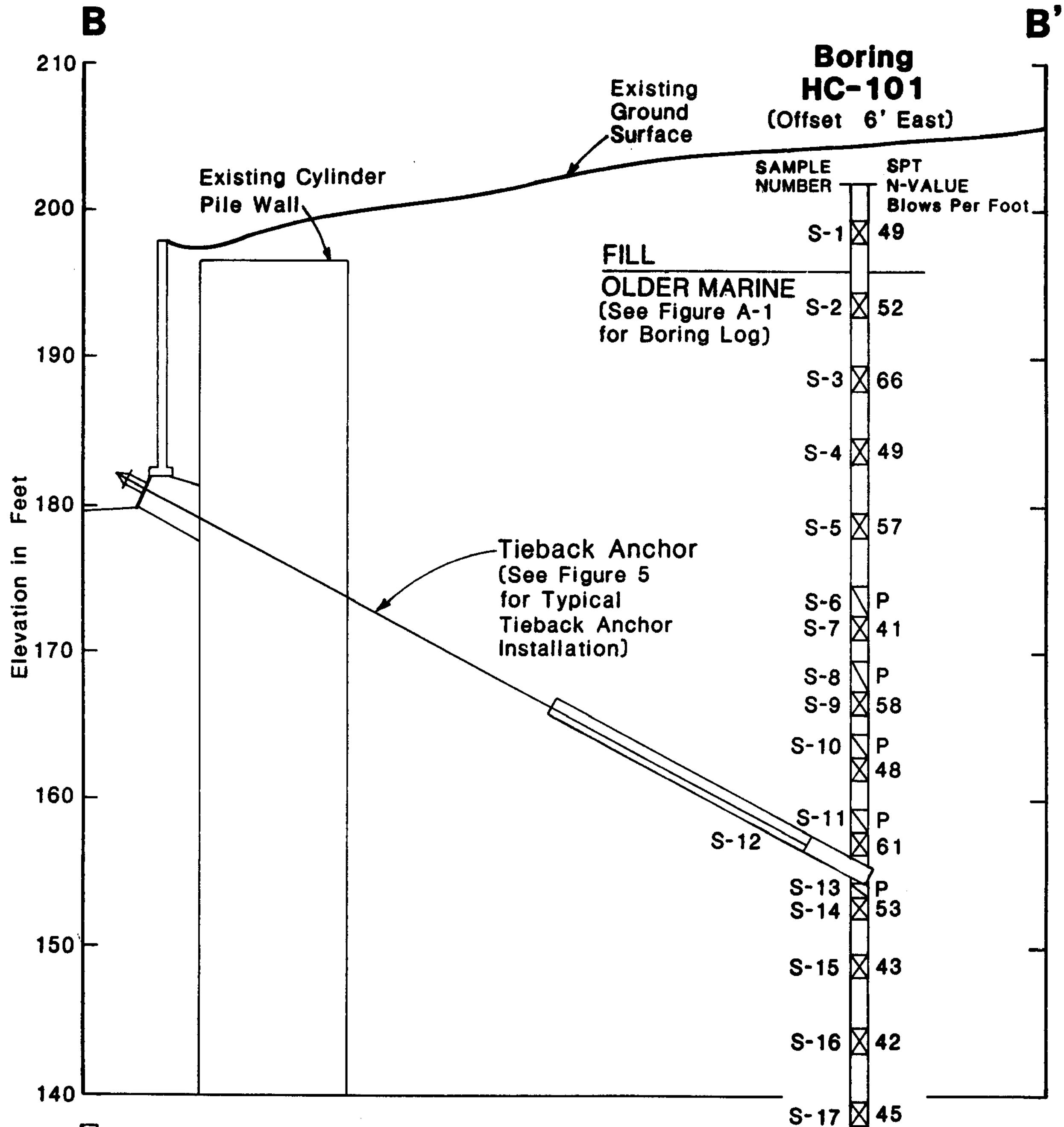
Figure 3

Generalized Subsurface Profile B-B'



☒ Split Spoon Sample Location and Number

◻ Shelby Tube Sample Location and Number

P Sampler pushed hydraulically, not driven.

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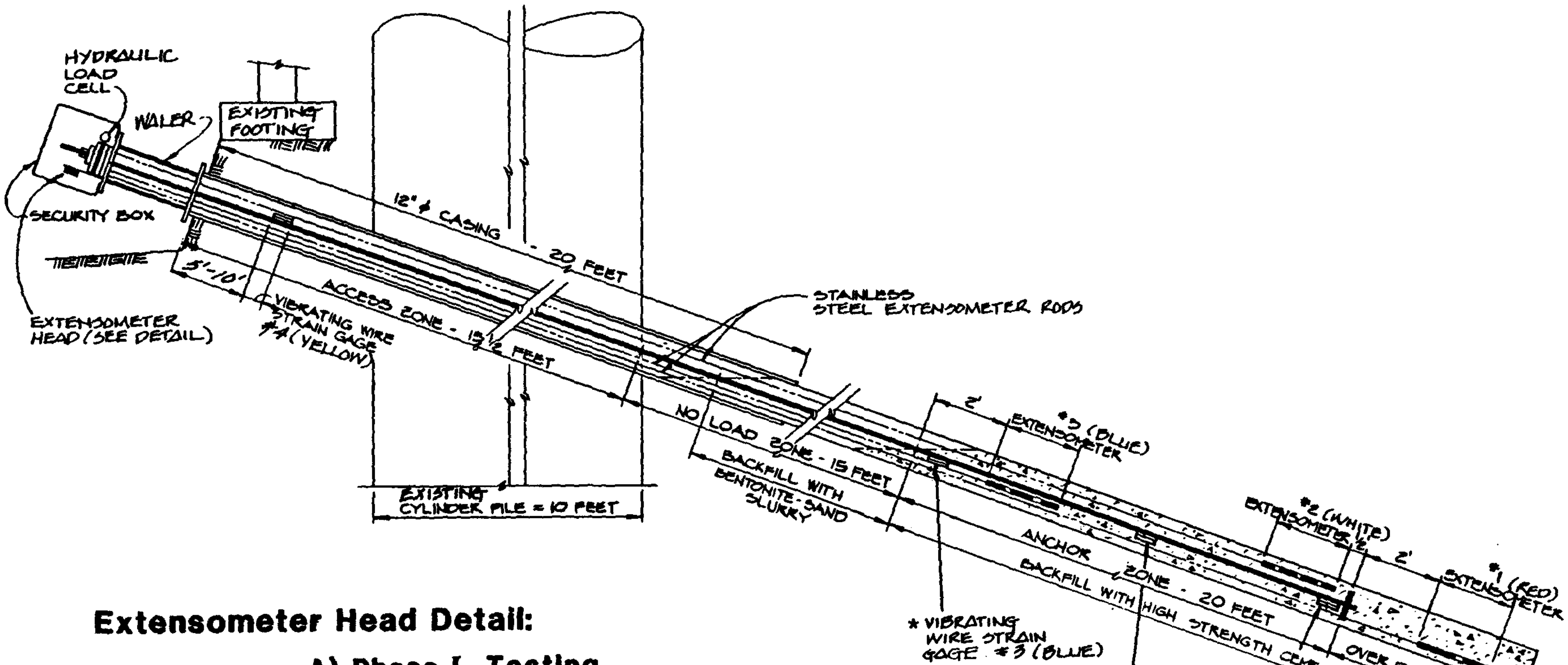
Figure 4

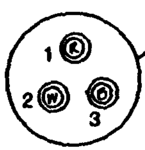
Typical Tieback Installation



* Only installed on TB-6, U-1 and U-2

Extensometer Head Detail:

A) Phase I, Testing

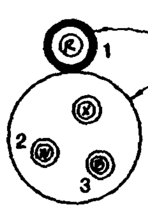


Stainless Steel Extensometer Head

#1 (Red) Bolted to Reference Head

#2 (White), and #3 (Blue) free to slide

B) Phase II, Long Term Monitoring



PVC Sleeve

Stainless Steel Extensometer Head

#1 (Red) free to slide

X Bolted to Reference Head - Anchored behind Waler to the ground

#2 (White), and #3 (Blue) free to slide

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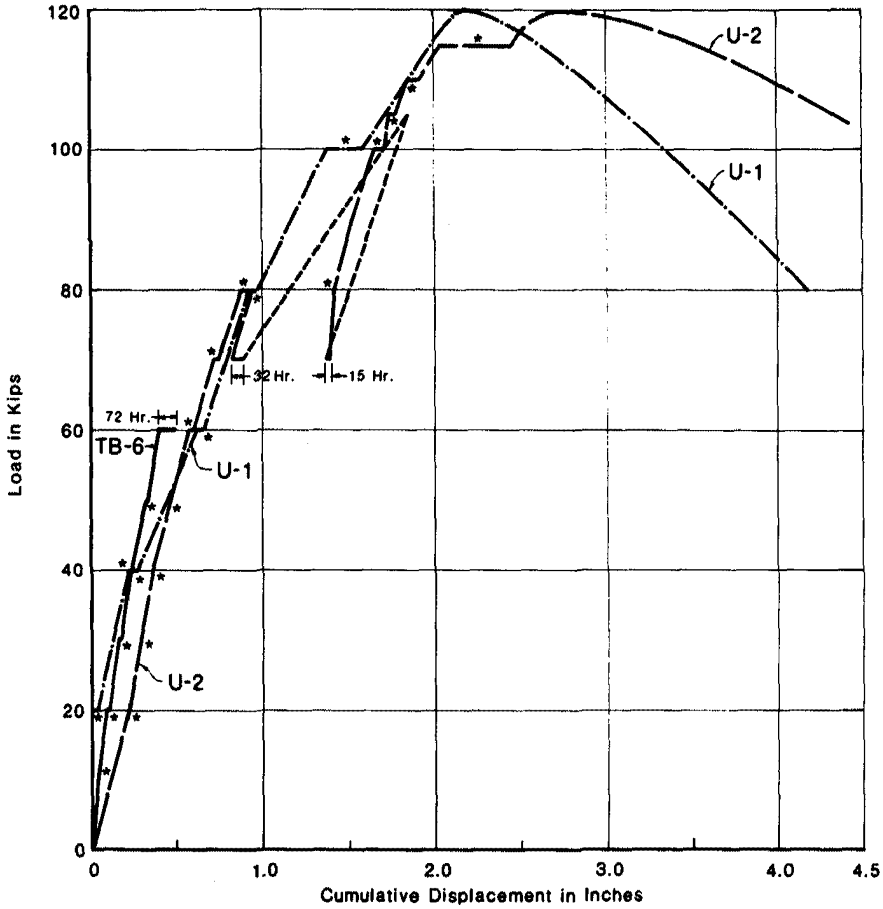
Figure 5

Load Displacement Curves
Tiebacks U-1, U-2 and TB-6



* Short-term Hold (up to 1-hour)

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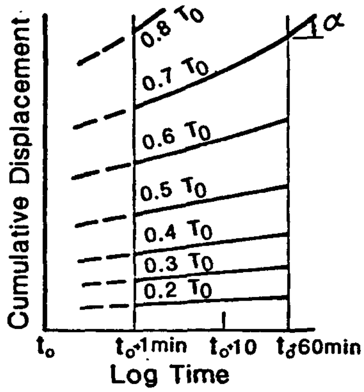
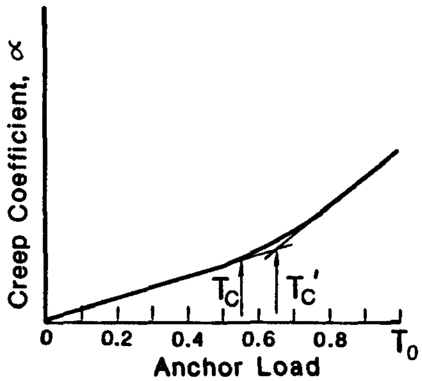
Figure 6

Typical Creep Plots



A. Creep Curves



B. Creep Coefficient Plot

T_0 = Anchor Pull Out Load

Critical Creep Tension:

T_C = The load level where the slope of the creep coefficient curve becomes tangent to the initial straight line portion of the curve.

T_C' = The load level defined by the intersection of the straight line extended from the initial portion of the curve, with the extension of the straight line extended from the latter portion of the curve.

J-712-12 March 1984
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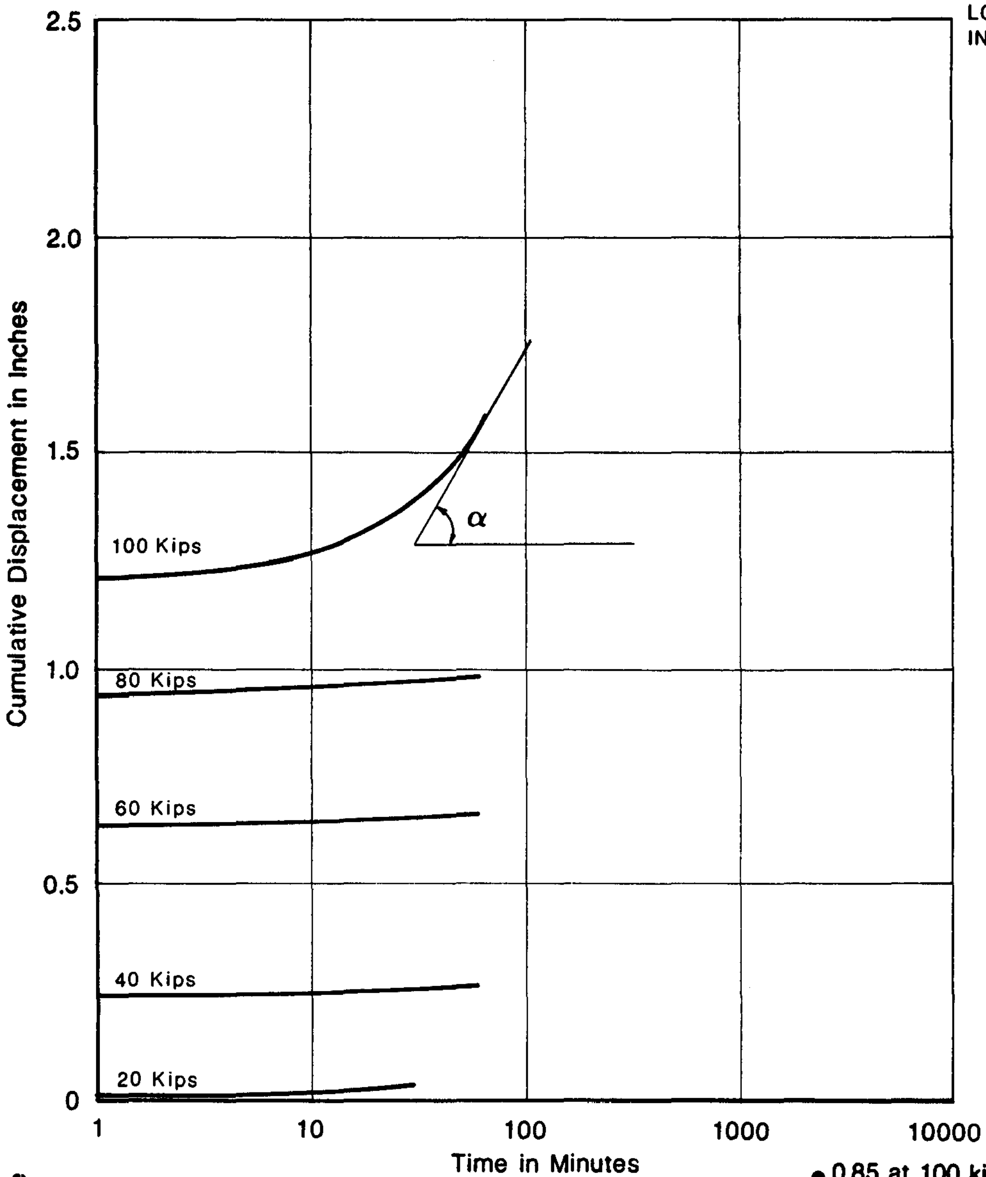
Figure 7

Creep Curves and Creep Coefficient Plot

Tieback U-1



LOAD IN KIPS	α_1 INCHES/LOG CYCLE OF TIME
100	.85
80	.035
60	.02
40	.015
20	.015

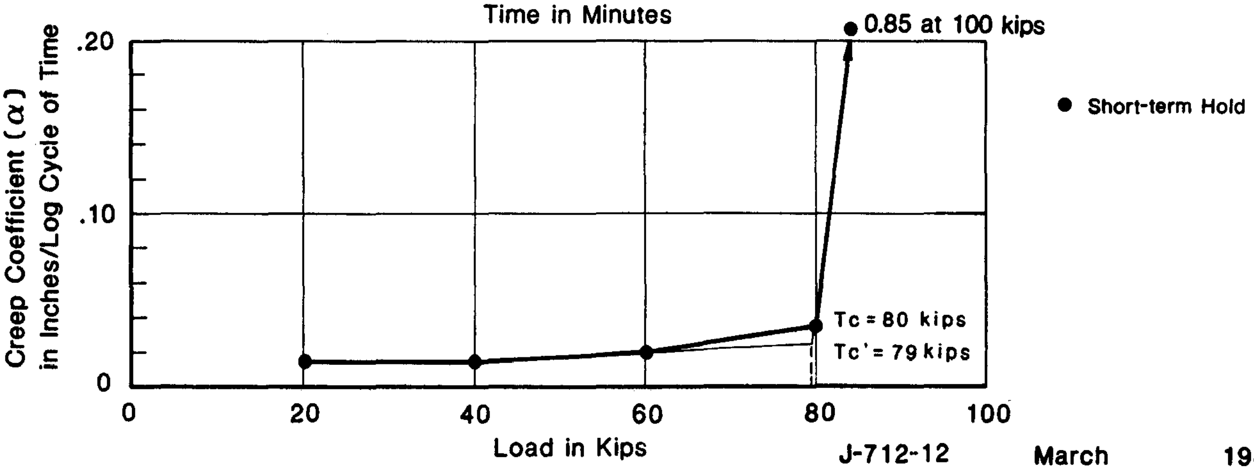


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Figure 8

Creep Curves and Creep Coefficient Plot
Tieback U-2

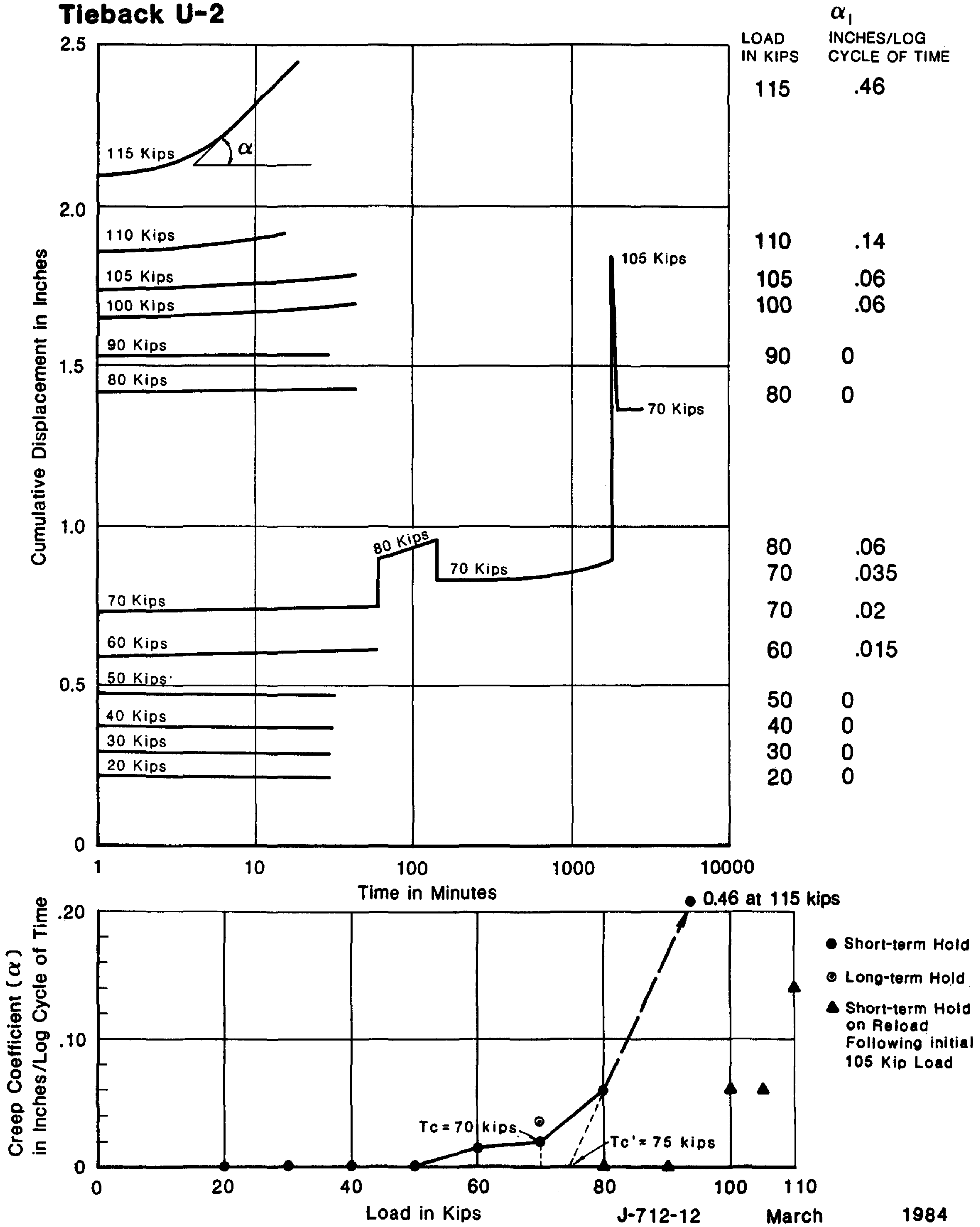

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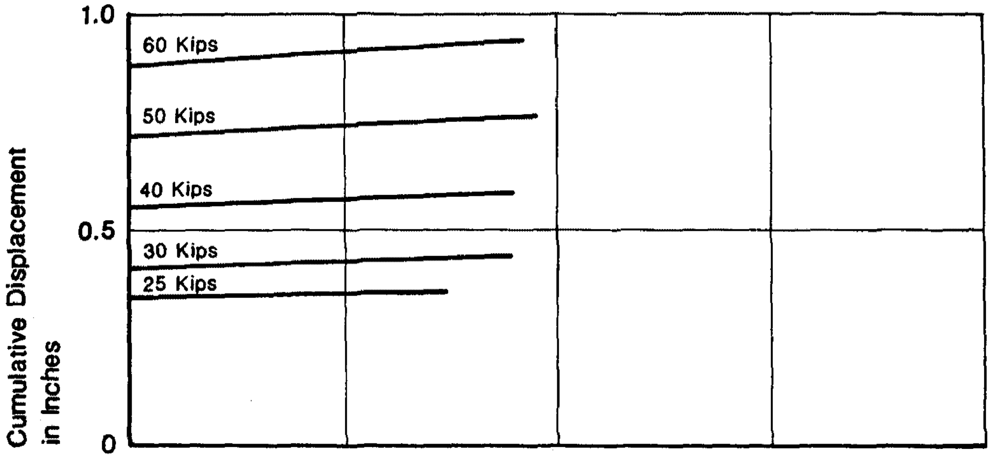
Figure 9

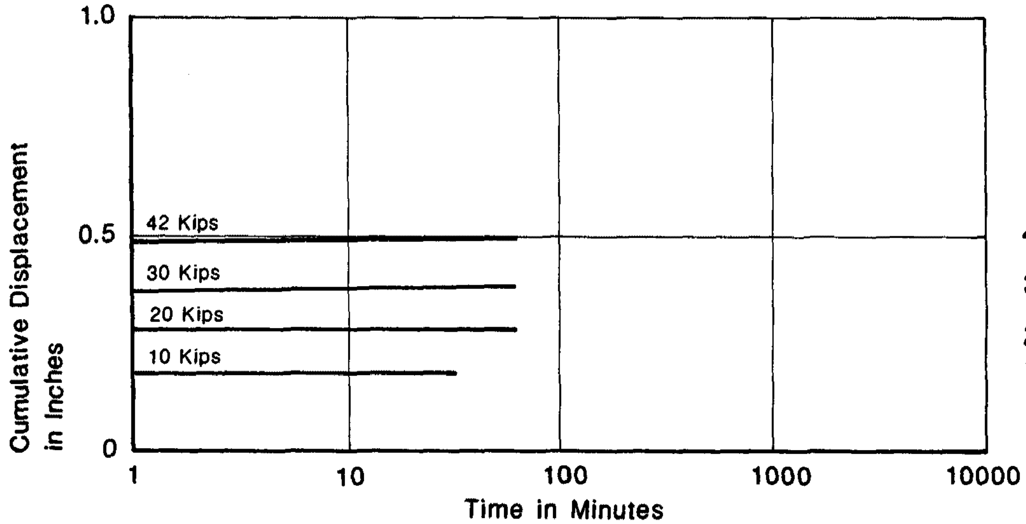
Creep Curves and Creep Coefficient Plot

Tieback TB-1

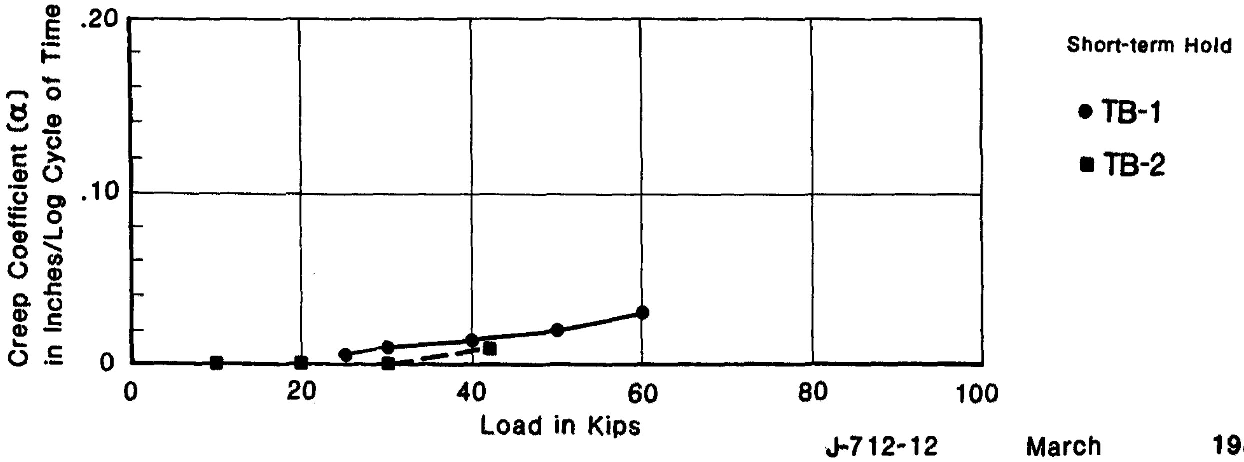


LOAD IN KIPS	α_I INCHES/LOG CYCLE OF TIME
60	.03
50	.02
40	.015
30	.01
25	.005

Tieback TB-2



42	.01
30	.01
20	0
10	0



J-712-12 March 1984

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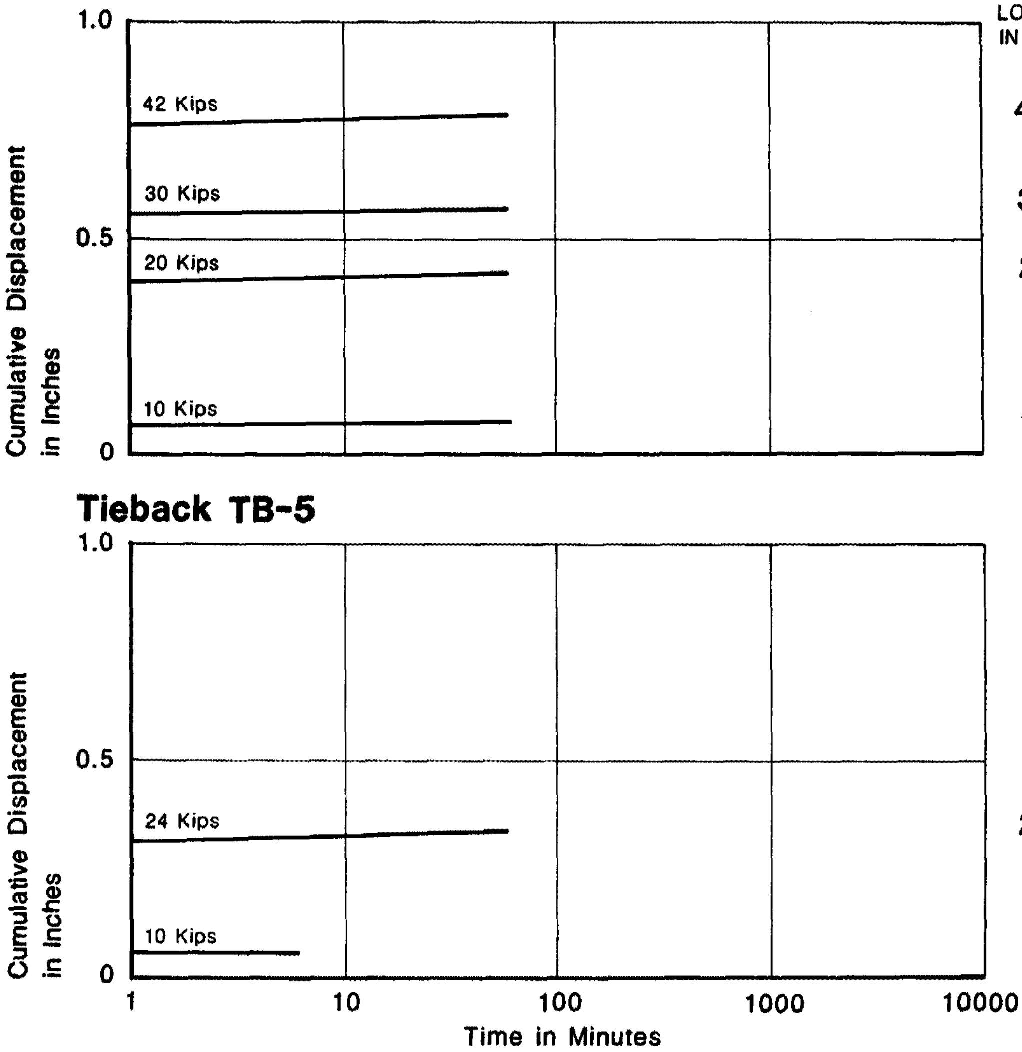
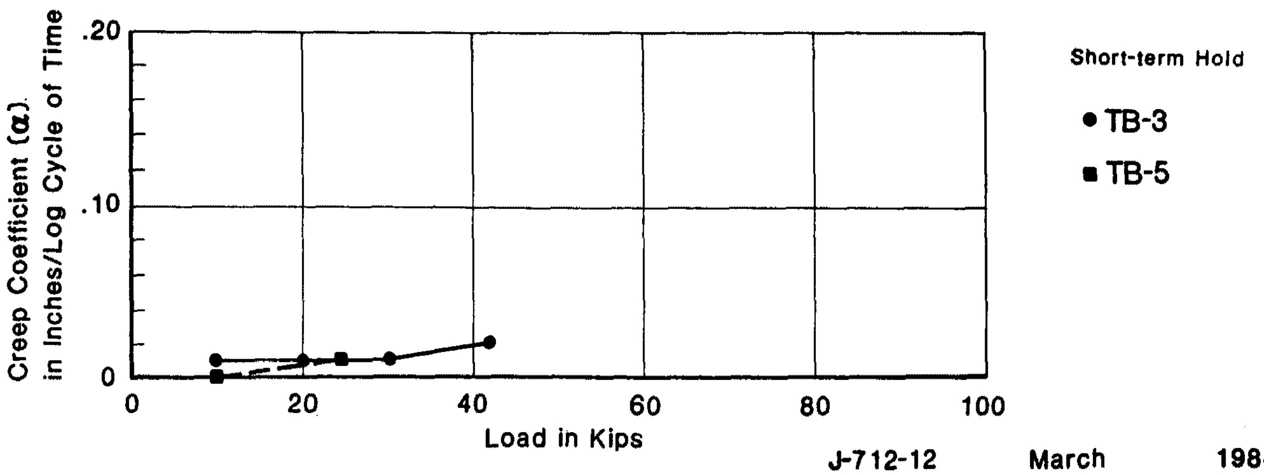
Figure 10

Creep Curves and Creep Coefficient Plot

Tieback TB-3



LOAD IN KIPS	α_l INCHES/LOG CYCLE OF TIME
42	.02
30	.01
20	.01
10	.01
24	.01
10	0



J-712-12 March 1984
HART-CROWSER & associates inc.
Figure 11

Creep Curves and Creep Coefficient Plot

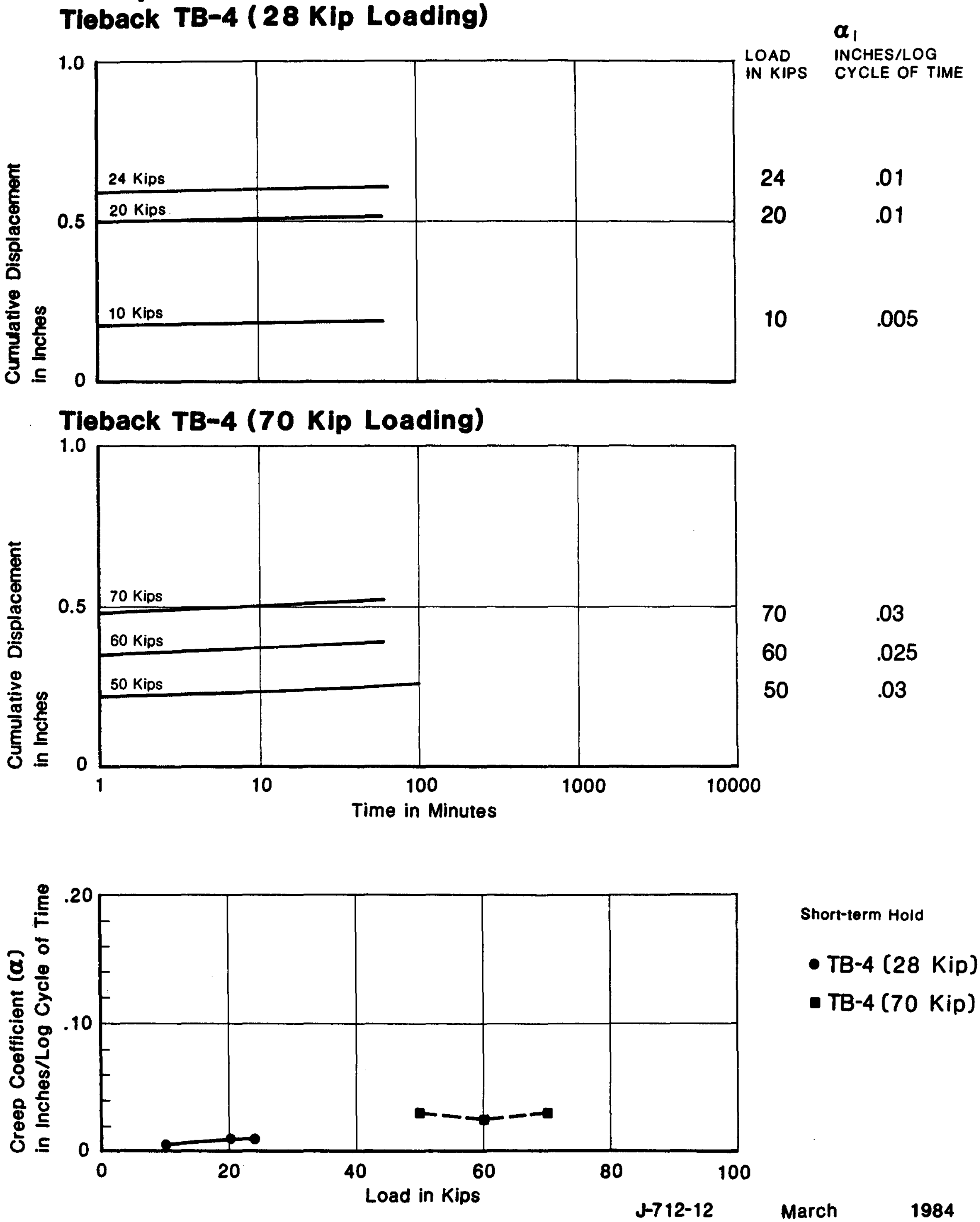


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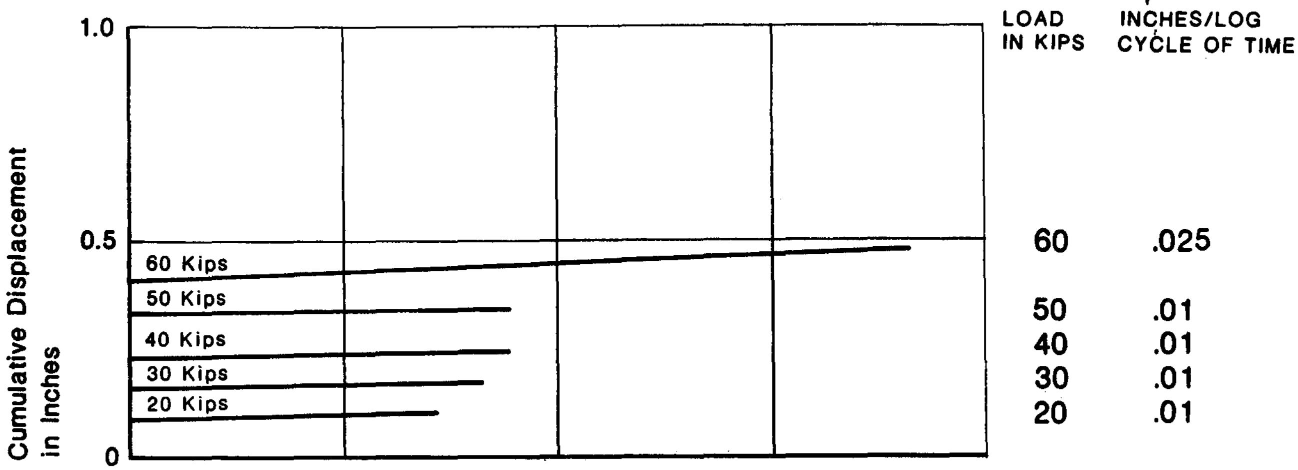
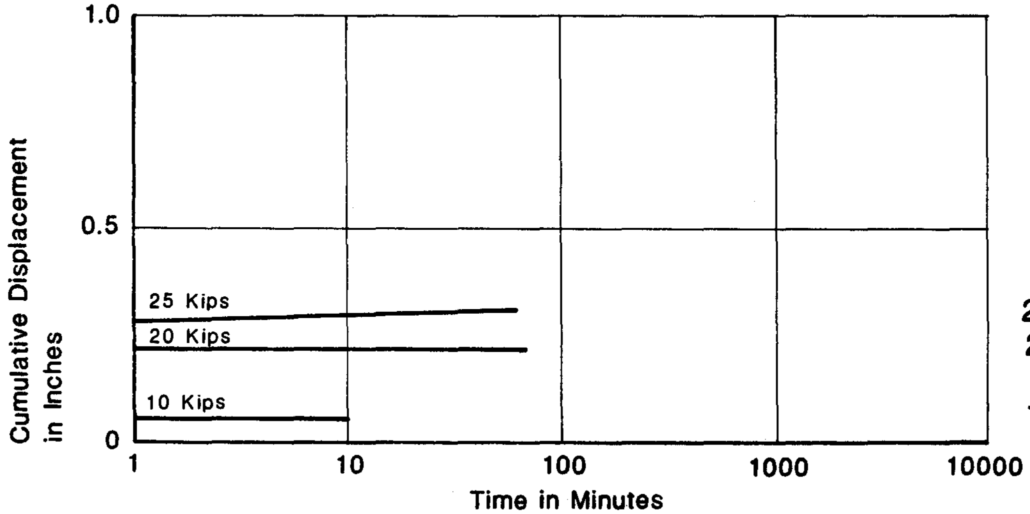
Figure 12

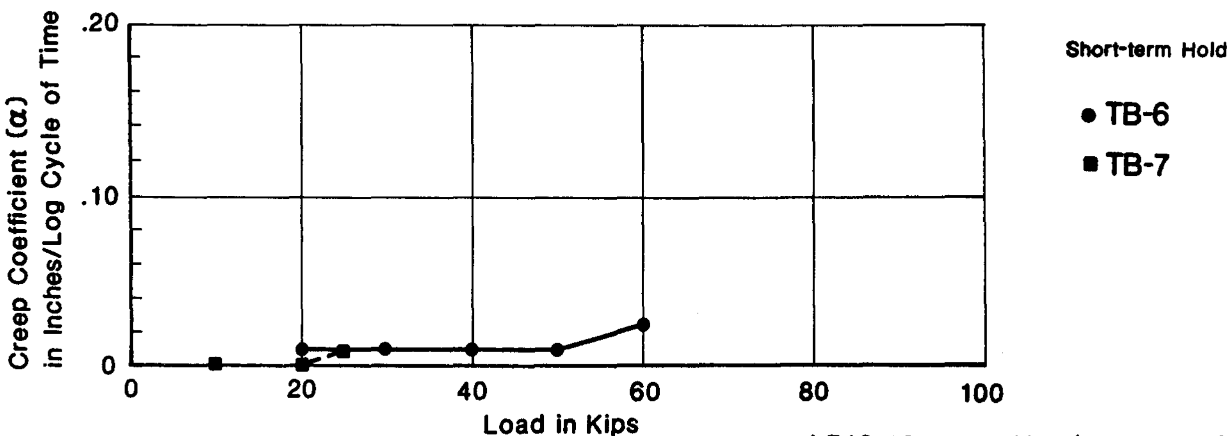
Creep Curves and Creep Coefficient Plot

Tieback TB-6



Tieback TB-7





J-712-12 March 1984
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Figure 13

Creep Coefficient vs Load

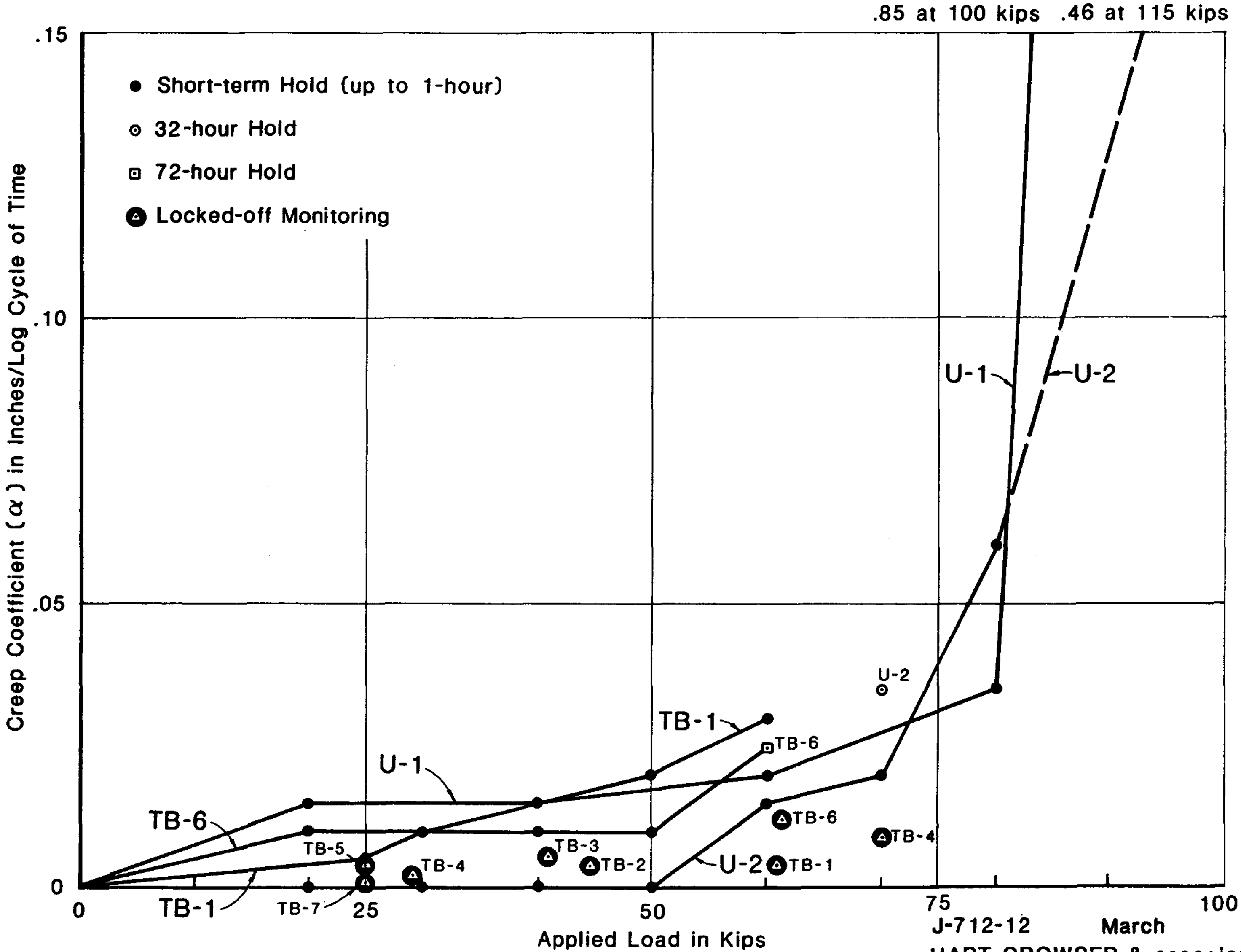

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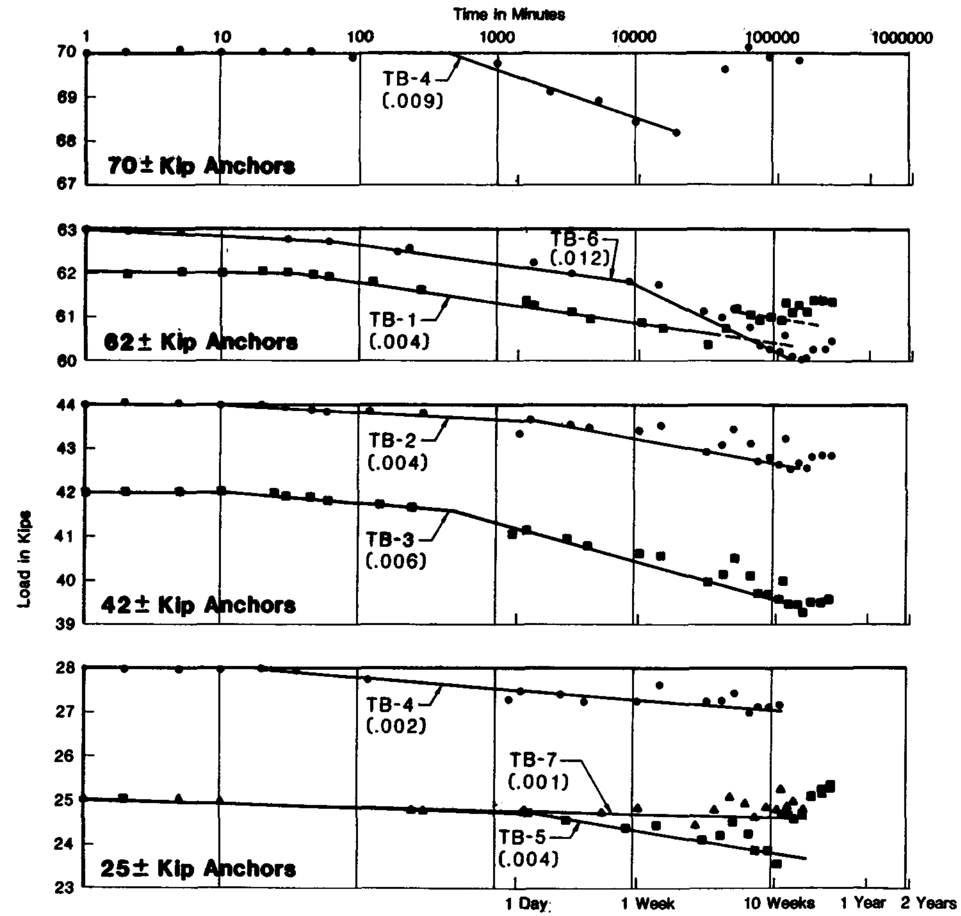
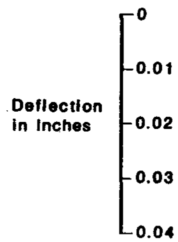
Figure 14

Long Term (Lock-Off) Creep Data



TIEBACK NUMBER	α_l INCHES/LOG CYCLE OF TIME	α_f KIPS/LOG CYCLE OF TIME
• TB-4	.009	1.1
• TB-6	.012	1.5
■ TB-1	.004	0.5
• TB-2	.004	0.5
■ TB-3	.006	0.8
• TB-4	.002	0.3
▲ TB-7	.001	0.1
■ TB-5	.004	0.5

Note:
The Load vs Time plot has the same shape as the Deflection vs Time plot. The time scale remains the same for both plots while the appropriate scale for the deflection axis would be:



J-712-12 March 1984
HART-CROWSER & associates Inc.
Figure 15.

REPORT NO. 4

PERFORMANCE MONITORING OF A HIGHWAY TIEBACK WALL

DEMONSTRATION SITE
I-71, CARROLL COUNTY, KY-227
CORRECTIVE LANDSLIDE MEASURE

by

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Research Investigator
David L. Allen
Chief Research Engineer
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LEXINGTON, KENTUCKY

JULY 1986

TABLE OF CONTENTS

I. PROBLEM STATEMENT

Over the past several years, an unstable cut-and-fill embankment on KY 227 (Carrollton-Worthville Road) in Carroll County had been failing. The problem area is located approximately eight miles southeast of Carrollton between Stations 234+25 and 244+25. This slide caused numerous maintenance problems for the roadway, which was frequently overlaid, and for the nearby railroad track, which had to be periodically realigned. As shown in Figure 1, prior slippage had lowered the shoulder several feet. A view from approximately the same location is shown in Figure 1A. A vicinity map of the slide area, is shown in Figure 2. Figure 2 presents a vicinity map of the slide area.

A. Geology

This site lies in the northern part of the Outer Bluegrass topographic region of Kentucky. It consists of predominately interbedded shales and limestones of the middle and upper series of Ordovician age. Glacial deposits and recent alluvium are present in the river flood plain.

The primary geologic formation involved in the slide is the Kope Formation. In this area, the Kope Formation is a medium gray shale interbedded with a medium gray limestone. Limestone generally comprises about 15 to 30 percent of the formation and usually occurs in even to slightly irregular beds about 12 inches thick. Thin beds of laminated calcareous siltstones are also occasionally found.

B. Subsurface Conditions Prior to Corrective Action

The Division of Materials of the Kentucky Department of Highways conducted a geotechnical exploration of the site with borings located at

Station 236+00 -- 45 feet right of centerline,
Station 236+00 -- 45 feet left of centerline,
Station 236+50 -- 50 feet right of centerline, and
Station 237+00 -- 45 feet left of centerline.

Laboratory tests were conducted on samples obtained at Station 236+50 -- 50 feet right of centerline. Test results indicated the material to be an A-7-6 (19) soil according to the AASHTO classification system and a CL according to the Unified classification system. The natural moisture content was 17 percent at a depth of 10.0 to 11.5 feet. At a depth of 20.0 to 21.5 feet, the AASHTO classification was A-6 (18) and the Unified classification was CL. The natural moisture content was 16 percent.

Slope inclinometers number 3 and 2 were installed at Station 236+50 (50 feet right of centerline) and at Station 236+00 (45 feet right of centerline), respectively. Both slope inclinometers *indicated* deflection rates of 0.2 inch per month. The sliding plane was at elevation 466.0 feet, which was 25 feet below the shoulder.

The two borings left of centerline were used as observation wells. The average water-table depth was 2 feet at Station 236+00 (well 1A) and 4 feet at Station 237+00 (well 1B). A plan view of the site, including structures, natural features, and instrumentation previously discussed, is shown in Figure 3.

The in situ soil strength parameters were estimated by a back analysis iteration. The soil unit weight and cohesion were held constant while the angle of internal friction was varied to arrive at a safety factor of 1.0. The results are

$$w = 128 \text{ pcf},$$
$$c = 0 \text{ psf},$$
$$0 = 18 \text{ degrees}.$$

The cross section used in the existing conditions analysis is shown in Figure 4.

II. REMEDIAL OPTIONS

Several remedial procedures were considered, with an increase of at least 30 percent in the existing safety factor as the critical criterion. Realignment of KY 227 further into the hillside was readily eliminated as impractical. That option would involve disturbing an already marginally stable hillside and require excavation, re-paving, and right-of-way changes for at least one-half mile. Flattening the slope also was eliminated because of adverse effects on the railroad.

Three other remedial procedures were considered; horizontal drains, rail piles, and a tied-back control wall. The horizontal drain option would result in a safety factor of 1.1 to 1.2 at an approximate cost of $62,000. This did not meet the criterion of a 30-percent increase of the safety factor. Rail piles perform best at depths to bedded material of less than 15 to 20 feet. The soil depths at this site exceed 20 feet. The tied-back control wall was chosen as the best alternative. A contract for construction of the wall was awarded on December 18, 1983.

III. STUDY PROPOSAL

The use of tieback walls to control landslide problems is somewhat limited. For that reason, a study was initiated with the objectives of:

(1) documenting construction procedures and obtaining short-term and long-term experimental data on tieback wall performance,

(2) analyzing field behavior using instrumentation installed on and near the wall, and

(3) making recommendations as to the effectiveness and future use of tieback walls constructed with treated wood lagging for correcting highway embankment sliding.

Monitoring of the wall is to continue for a period of five years. Data and observations subsequent to this report will be presented annually in the form of a memorandum.

IV. WALL DESIGN, LAYOUT, AND CONSTRUCTION

The tieback wall was designed and constructed by the Schnabel Foundation Company in compliance with Department of Highways Special Notes. Design personnel were supplied a design restraint force of 33,000 pounds per linear foot of wall where the sliding plane was 25 feet below the top of the wall. A safety factor of 1.5 was assumed. This translates to a normalized uniform loading (p) of:

$$p = 0.0528 \text{ kip/foot} \times h,$$

where h = the height of the wall in feet.

The wall was constructed using a system of steel H-piles, pressure-treated wood lagging, and corrosion-protected tiebacks. A typical wall section is shown in Figure 6. The assumption of a safety factor of 1.5 and testing up to 133 percent of design load (to be discussed in the TIEBACK TESTING section) resulted in test loads of 200 percent of expected loading. This conservative approach was probably the result of a lack of experience with this type structure.

The tied-back wall, as designed, extended from Station 234+25 to Station 244+25. At Station 234+25, the wall was 76 feet right of centerline. The wall gradually approaches centerline, and at Station 235+50 it is 70 feet right. At that point, the wall bends toward the centerline, making it only 40 feet right at Station 236+00. From Station 236+00 to the ending station, the wall is approximately 43 feet right of the centerline. The height of the wall is approximately 15 feet above finished grade. Total cost of the wall and associated efforts was $483,000. A plan view and front view of the wall are shown in Figure 7. Excluding the cost of excavation, cost of the wall was $31.25 per square foot of exposed wall.

A. General Construction

A total of 126 soldier piles were driven on approximately 8-foot centers. Of these, 90 were 10 x 42 H-piles and the remaining 36 were 12 x 53 H-piles. Driving points were specified for penetration of a boulder zone and to insure proper seating. Piles were driven to a resistance of 100 tons or refusal. Refusal was considered to be less than 0.8 inch penetration in 10 hammer blows.

After the piling was driven, the existing embankment was excavated below the elevation of the highest tiebacks -- Figure 8. The piling was then cleaned and stud bolts were welded to the pile flanges. The stud bolts were later used to attach 4-inch by 8-inch treated wood lagging to the pile face. Exposed surfaces of the piling and bolts were protected by an application of "Tapecoat TC Mastic" -- Figure 9.

The lagging was treated southern yellow pine timbers attached to the piling by threaded studs, steel plates, and nuts. There were approximately 2-inch gaps between the lagging through which the threaded studs protruded. The gap was spanned by metal plates and fastened in place with nuts.

Treatment of the lagging consisted of a combination vacuum and pressure. The timber was subjected to a vacuum approaching 27 inches of mercury for 30 minutes. The treatment solution was then pressure injected (140 psi) into the wood for 50 to 60 minutes. The solution consisted of 44.01 percent chromic oxide, 19.27 percent cupric oxide and 36.72 percent arsenic pentoxide. A 3.27 percent concentrate solution was used.

Before the lagging was installed, a drainage pathway was placed between the wall and embankment. The pathway consisted of a layer of AMOCO 4553 fabric placed against the soil embankment and a layer of TENSAR "PWI" grid against the lagging -- Figures 6 and 10. At the bottom of the wall, the pathway ended in a trough made of a cut section of corrugated plastic pipe. A collector system of 8-inch pipe was placed in the trough with outlet lines spaced at approximately 24 feet. The cavity behind the lagging was backfilled with the material previously excavated. The backfill was completed prior to testing, but in many cases failed to support the piling sufficiently during loading. Where the piling deflected too much the soil was removed and replaced with weak concrete. After placement of the drainage pathway, the lagging was installed beginning at the top of the piling.

B. Tiebacks

When the wall had been constructed down to the elevation of a tieback, a hole was drilled into rock and a steel tendon grouted in place. Each tieback was tested when the grout had reached sufficient strength. If the tieback tested acceptably, it was eventually locked off at 75 percent of design load. This lock-off load was chosen to permit some relaxation and movement of the retained embankment. The grout mixture contained Type III portland cement.

Two tiebacks were placed in a bay midway between piles. The tiebacks were stacked vertically with alternating bays tied back. Toward the ends of the wall where the depth to rock decreased, one tieback on alternating bays was used. For a distance of approximately 110 feet (Station 234+80 to Station 23@+90), four tiebacks on alternating bays were used. Where tiebacks failed, additional tiebacks were placed until tests indicated design restraint was achieved.

In Figure 11, completed sections of the wall are shown. Double caps where supplementary tiebacks had to be installed may be noted in the foreground. At the top of the wall, a fence was erected to protect unwary pedestrians.

A soldier beam and tieback schedule is shown in Table 1, page 43.

Tiebacks are high-strength, in this case, multi-stranded, steel tendons anchored in rock at one end, stressed, and then anchored to the wall at the other end. The fixed anchorage or bond length is accomplished by drilling a hole (minimum of 10 feet) into competent rock and grouting the tendon into place. The hole must be clear of deleterious material and centralizers and spacers located so there is a minimum of 1/2 inch grout cover on the tendon. The bond length is calculated by the equation:

$$Lb = P/\ (3.1416)\ (d)\ (t_w)$$

in which

L_b = bond length (not less than 10 feet in solid rock) (feet),
P = design load for tieback (pounds),
d = diameter of the drill hole (inches), and
t_w = bond stress at the interface between rock and grout (psi).

The unbonded length (L_f) is the portion of the tieback free to elongate elastically during stressing. This length is a minimum of 15 feet and is sufficiently long to insure that the bond length is formed in sound competent rock. Tiebacks were installed at angles varying from 10 degrees to 30 degrees from horizontal. A typical tieback cross section is shown in Figure 12.

C. Instrumentation

Several types of instrumentation were installed to monitor

Four permanent load cells were installed near Stations 235+90 (Tieback 21) and 237+@4 (Tiebacks 43-44). At each location, two load cells were installed. One cell was on the upper or first tier tieback and one was on the lower or second tier tieback. These cells were used to monitor the short and long-term stresses on the tendons supporting the wall.

Earth pressure meters were installed between the wall and earthen embankment. These meters are used to monitor pressure on the wall as opposed to the permanent load cells that monitor stress on the tendon supporting the wall. Nine meters were installed. Five were located in the center of the bay or midway between Piles 46 and 47, Station 237+88. These five meters were numbered 1540, 1447, 1541, 1449, and 1659, respectively, from the top of the wall. The other four meters were located as close as possible to Pile 47. They were numbered from the top 1444, 1615, 1542, and 1614, respectively. The top meter on each row was placed approximately two feet below the top of the wall and the remaining meters were spaced at 2.0 to 2.5 foot intervals down the wall. In Figure 13, the top eight meters may be seen with the top two located between the third and fourth lagging from the top of the wall.

As lagging installation proceeded down the wall, pressure meters were installed at the desired locations. The meters were loosely attached to the back side of the lagging with the monitoring cables exiting the open face of the wall. The cables were enclosed in plastic conduit and brought to a common monitoring point. After the meters were in place, the cavity behind the wall was backfilled with the previously excavated material and compacted with gasoline operated hand compactors.

A total of seven slope inclinometers were installed to monitor horizontal earth movement. Five inclinometers were installed behind the wall near Stations 236+00, 237+00, 238+00, 239+00, and 240+00. Location of the inclinometers ranged from 4 to 10 feet behind the wall. Two inclinometers were installed approximately 85 feet right of centerline near Stations 236+00 and 237+00.

Three ground-water observation wells were installed approximately 35 feet right of centerline near Stations 237+50, 238+50, and 239+50. One observation well was installed approximately 30 feet left of centerline at Station 237+00. These wells, in conjunction with the slope inclinometer holes and observation wells installed during the earlier geotechnical investigation, permitted monitoring of the water table at the site.

Tiltmeter plates were installed on the wall near Stations 236+00, 237+00 and 238+50. These plates were installed on the wood lagging. Due to irregularities of the lagging and fluctuations of the characteristics of the wood related to changing moisture conditions, data obtained from these meters were not consistent. Location of instrumentation used to monitor the structure during and after construction of the wall is shown in Figure 14.

V. TIEBACK TESTING

Each tieback was load tested by one of three types of tests. The three test types were creep tests, performance tests and proof tests. Creep tests and performance tests both essentially incrementally loaded and unloaded the tendon to 133 percent of the design load and monitored tendon elongation or movement. These two tests were conducted on a limited number of tiebacks and required a substantial amount of time.

All tiebacks not tested by either creep or performance tests were proof tested. This test is of relatively short duration and consists of loading the tendon to 120 percent of the design load and maintaining that load five minutes. If creep movement during the five minutes is less than 0.03 inch and movement patterns are similar to adjacent tests the tieback is acceptable. If these criteria are not met, but the creep rate over a longer period of time was determined to be less than 0.08 inch per logarithmic cycle of time, the tieback was accepted.

Four tiebacks were creep tested. This test required loading and unloading the tendon through gradually increasing load and time increments until 133 percent of the design load was reached. This load was maintained and movement observed for 300 minutes. An acceptable tieback performance was a creep rate less than 0.08 inch per logarithmic cycle of time.

Five selected tiebacks and 5 percent of all remaining tiebacks were performance tested. This test involved loading and unloading the tendon through gradually increasing load and time increments until 133 percent of the design load was reached. This load was maintained for 10 minutes after which time the test was discontinued if movement was less than 0.04 inch. If movement exceeded 0.04 inch the load was maintained for 60 minutes and the movement recorded Acceptable performance for these tests was

(1) measured elastic movement exceeding 80 percent of the theoretical elongation of the unbonded tendon and

(2) creep movement between 1 and 10 minutes less than 0.04 inches.

Tiebacks failing criterion number 2 were accepted if the creep rate over 60 minutes of maximum loading was less than 0.08 inches per logarithm cycle of time.

Equipment for testing the tiebacks may be seen in Figure 15. A hydraulic jack is affixed to the exposed tendons and load is applied. Resultant stress is monitored by gages not shown in the figure and the deflection is monitored with the dial gage mounted on the tripod.

VI. TEST DATA AND RESULTS

A. Laboratory Data

Laboratory tests were conducted on samples obtained at the slope inclinometer borings. Moisture content of samples tested ranged from 12.6 to 23.5 percent and averaged 17.7 percent. Specific gravity ranged from 2.67 to 2.81 with an average of 2.70. The material classified as A-6 or A-7-6 by the AASHTO system or CL by the Unified system. Results of index tests are contained in Table 2.

A series of consolidated-undrained triaxial tests was performed. Results of these tests indicate an internal friction angle of 37 degrees and a cohesion of 0.

B. Lateral Movements

Slope inclinometers are identified as numbers 1, 4, 6, 8, 10, 11, and 12. The locations of Inclinometers 1 through 10 are behind the wall, approximately 40 feet right of the centerline, in order of ascending Stations 236+00, 237+00, 238+00, 239+00, and 240+00, respectively. Inclinometers 11 and 12 are located approximately 85 feet right of the centerline at Stations 236++00 and 237+00, respectively. Inclinometers 1, 6, 8, and 10 were installed in the first week of March 1984, which was before excavation for wall construction began. Inclinometer 4 was installed May 5, 1984, after the wall was essentially complete. Inclinometers 11 and 12 were installed June 18 and 11, respectively.

Data obtained at Inclinometers 1 through 10 indicate little movement at depths greater than 4 or 5 feet. The exception to this is Inclinometer 1. This is approximately the location of monitoring instrumentation in place prior to corrective action. The sliding plane then was located approximately 25 feet below the surface. Data from Inclinometer 1 indicate a displacement of 0.5 inch 47 days after installation of the inclinometer. This movement took place primarily along the existing sliding plane. Records indicate that final tiebacks near that location were stressed and locked off on May 11, 1984. After that date, little movement has been observed at that location -- Figure 16.

All inclinometers near the wall indicate the greatest movement within 4 to 5 feet of the surface -- Figures 16 through 20. This is probably due to sloughing of the embankment after the material below the piling was excavated. The magnitude of this movement ranged from 0.5 inch to 1.5 inches. In Figures 21 through 23, movements at selected depths are plotted versus time and the dates of tieback lock off are noted. The top of the embankment was pulled back toward its original position when the tiebacks were stressed. In Figure 21, the movement toward the original position is illustrated for each inclinometer at approximately 125 to 150 days. This coincides with the locking off of tiebacks. At depths greater than 5 feet, embankment movement (with the exception of Inclinometer 1) was less than 0.3 inch.

Inclinometers 11 and 12 indicate maximum movements of 0.2 and 0.8 inch, respectively. Movement at Inclinometer 11 is of such small magnitude that it is probably insignificant (Figure 24). It probably is erratic because it is approaching the limit of resolution of the instrument. At Inclinometer 12, movement is occurring throughout the entire depth of soil. As of March 1986 0.8 inch of movement had occurred, but approximately 0.5 inch of movement occurred from February to November of 1985. Figure 25 shows movement is continuing at this location, but the rate has decreased.

As noted earlier, instrumentation intended to monitor tilt of the wall did not function properly. However, optical surveys were conducted to establish the initial position of the wall. Elevation of the top of the wall and plumb of the vertical face were established. No measurable changes have been observed.

VII. PRESSURE DATA

Earth pressure on the wall and retaining stress of the tiebacks were monitored with earth pressure meters behind the wall and permanent load cells on the tiebacks. The earth pressure meters used were high range meters (200 psi). Sensitivity of these meters was such that, except during testing and tieback lock off, pressure on the wall was too low to be monitored accurately. Initial readings were obtained prior to the backfilling operation. These readings were used as zero readings and subsequent readings were compared to them.

During testing and lock off, pressure on the wall ranged from 0.0 to approximately 10.0 psi at some locations. Pressure data obtained from earth pressure meters are shown in Figures 26 and 27. As seen in these figures, long-term pressure on the wall appears to be insignificant. The apparent negative pressure readings are a result of the low sensitivity of the 200 psi meters at the low existing pressure conditions.

Permanent load cells were installed on Tiebacks 21 and 43-44. At both locations, the higher (first tier) and lower (second tier) tiebacks were instrumented. Tieback 21 is approximately located at Station 235+90 in a bay where four tiebacks were used. Tieback 43-44 is approximately located at Station 237+64. At this location, only two tiebacks per bay were used.

Lock-off loads on the tendons ranged from 68.8 kips to 132.0 kips. Loads on all tendons decreased with time and eventually ranged from 53.1 to 83.0 kips. Load cell data are plotted versus time in Figure 28.

A. Water Table

Water-table elevations were monitored with observation wells located at Stations 236+00 (45 feet left), 237+00 (30 feet left), 237+50, and 238+50 and 239+50 (27 to 35 feet right) (see Figure 14). The water table fluctuated during construction of the wall, but gradually rose after the wall was complete. Water-table depths are shown in Figure 29, with completion of the wall occurring at approximately 200 days.

B. Durability

Durability of the structure, primarily the exposed wood lagging and metal surfaces, was a major concern. Visual inspections and soundness checks indicated these wall components have not noticeably deteriorated during the first two years.

VIII. PROBLEMS

The most common problem associated with this project involved the inability of tiebacks to withstand test loading. Some of the reasons for tieback failure were the following:

1. "Slick holes" resulting from drilling an anchorage shaft in the presence of water. Native rock at this site (Kope Formation shale) weathers rapidly in the presence of water. Bond between the grout and rock would be reduced in this case, thus permitting slipping of the tieback.

2. Broken strands of the tendons accounted for several tieback failures. Of 121 tiebacks designed for the project, 25 failed and were replaced or supplemented with additional tiebacks.

Several tieback tests were discontinued due to excessive movement of the wall. In some cases, the piling deflected more than 4 inches at only 50 percent of design load or approximately 75 percent of the required resisting force. The solution to this problem was to excavate behind the piles and backfill with a weak concrete mix.

Temporary delays were caused by failure of the welds on the threaded studs affixing the lagging to the piling and re-calibration of the jack used to load the tiebacks.

IX. CONCLUSIONS

The tied-back wall, to the present, has performed well. Lock-off loads on the tiebacks were 75 percent of design load. Present loads are considerably less than lock-off. Pressure on the lagging appears to be insignificant. Using the previously noted loading ($p = 0.0528$ kip/foot x h), the wall at the pressure meter location was designed to support 7.33 pounds per square inch. Initial pressure on the lagging was about 50 percent of the design restraint with two meters briefly exceeding that valve (Figures 25 and 26). This has since dropped to nearly zero at all metered locations. Pressure meters having better resolution would have been desirable for this application. However, the resolution of the meters is such that any significant pressures would have been measured.

Earth movement behind the wall has been controlled. The sliding failure existing prior to wall construction appears to have stabilized. In the vicinity of Station 236+00, movement along the sliding plane was observed until the tiebacks were stressed. Since that time, movement has been minimal. Below the wall at Slope Inclinometer 12 (85 feet right of Station 237+00), approximately 0.8 inch of lateral movement, Figure 25, was observed from October 10, 1984, to November 18, 1985. Movement is still occurring, but its rate has decreased. Surface slumping, due to excavation, was stabilized when the tiebacks were stressed. Since its completion, the wall has not moved or tilted. Optical surveys of the walls initial and more recent positions verify its stability.

Perhaps the reduction of tieback stress and earth pressures may be explained by a combination of soil cohesion and relaxation of the wall components. Before the tiebacks were stressed, the embankment continued to slide and to slump where the soil had been excavated. When the tiebacks were stressed, those movements ceased.

After stressing of the tiebacks the tieback tendons, piling, and lagging began to relax. This reduced the pressure on the wall components but restrained the embankment sufficiently to prevent it from moving along the sliding plane again. The cohesional component of shear strength (cementation) of the embankment may have prevented it from relaxing to the point of maintaining or increasing the original pressure on the wall.

While the retained embankment is apparently stabilized,, the material below or in front of the wall may not be stabilized. The driving force on the embankment below the wall has been reduced, but movement continues. The rate of movement has decreased from 0.05 inch per month to 0.009 inch per month, but monitoring will continue. If embankment movement should continue, alignment of the railroad and possibly the stability of the retained embankment could be adversely affected. The movement below the wall, Figure 25, apparently justifies the assumption, Figure 5, of no soil resistance in front of the wall.

In general, the methods and materials used in construction of the wall appear acceptable. Problems, such as unacceptable pile deflection during testing, jack re-calibration, and poor stud bolt welding, caused relatively short delays. More significant delays resulted from "slick holes" and broken tendon strands. This led to placement of additional tiebacks. Overall, the construction progressed satisfactorily.

It is anticipated that future design of similar structures will be less conservative. Due tieback wall has been constructed in Kentucky since the completion of the study structure. The second structures was designed with a safety factor of 1.0 and tested to 133 percent of design load.

Figure 1. Ky. 227 Slide Site Prior to Remedial Action.

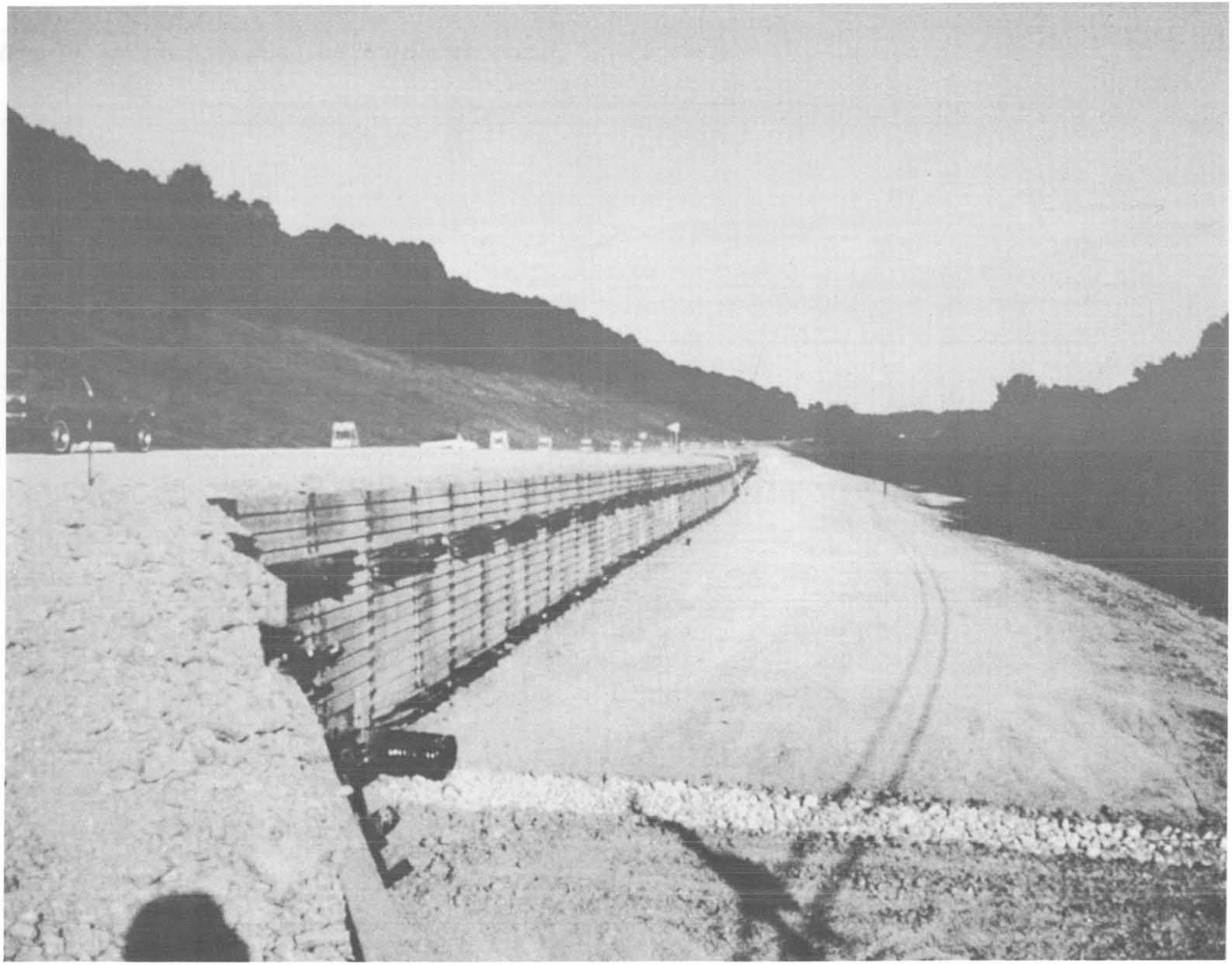


Figure 1A. Ky. 227 Slide Site When Tieback Wall is Nearly Complete.

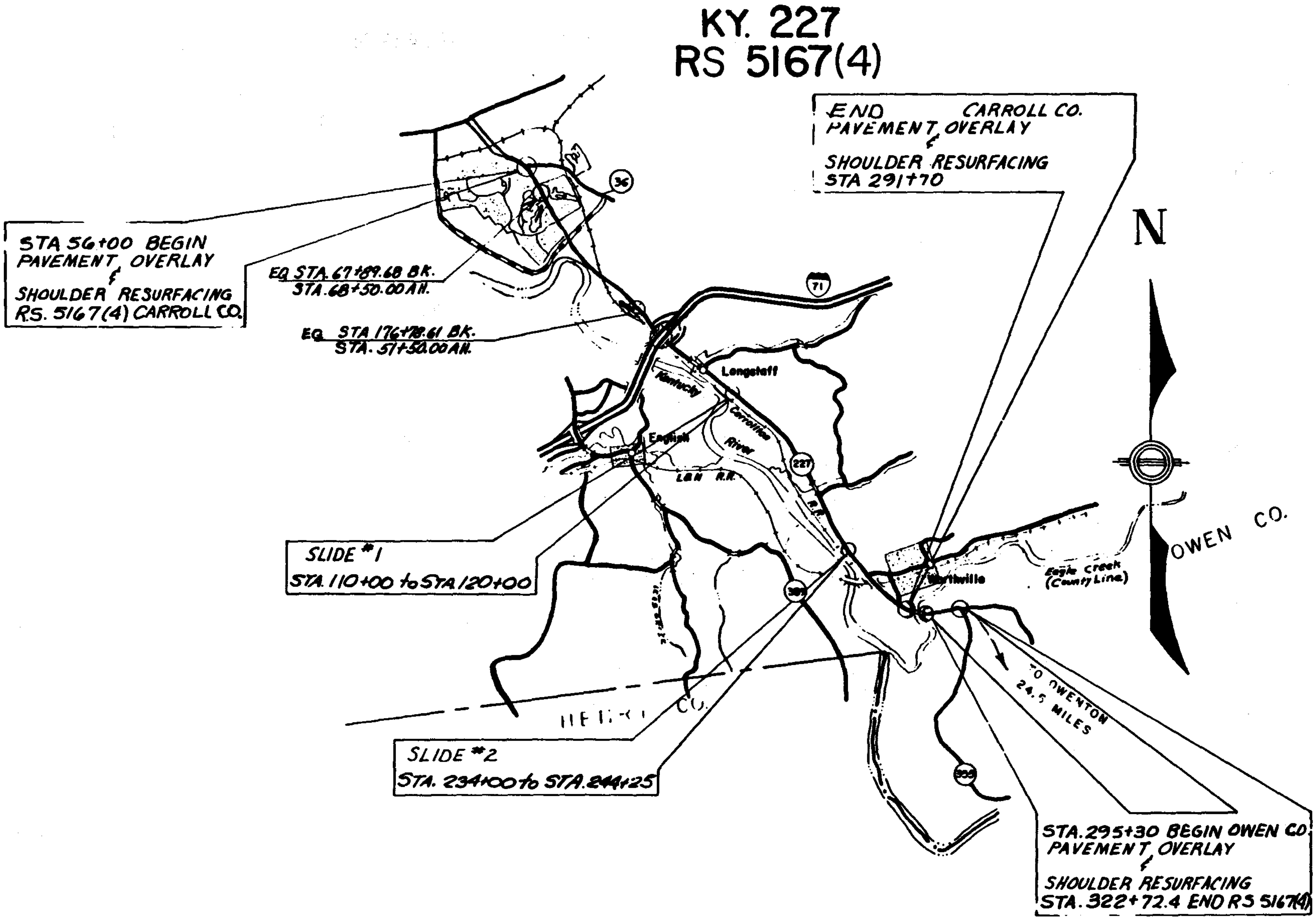


Figure 2. Vicinity Map of Site.

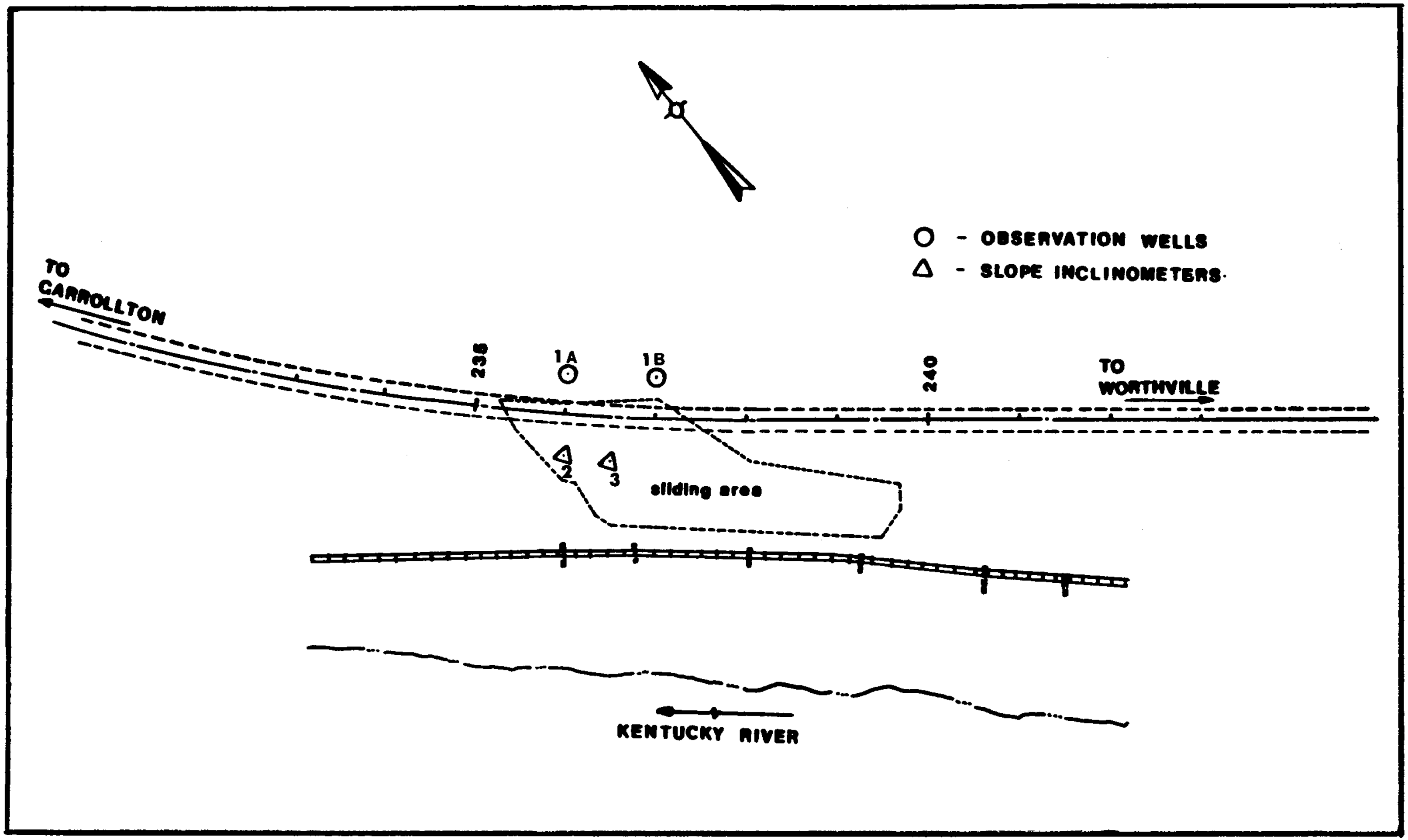


Figure 3. Plan View with Original Instrumentation and Slide Area Located.

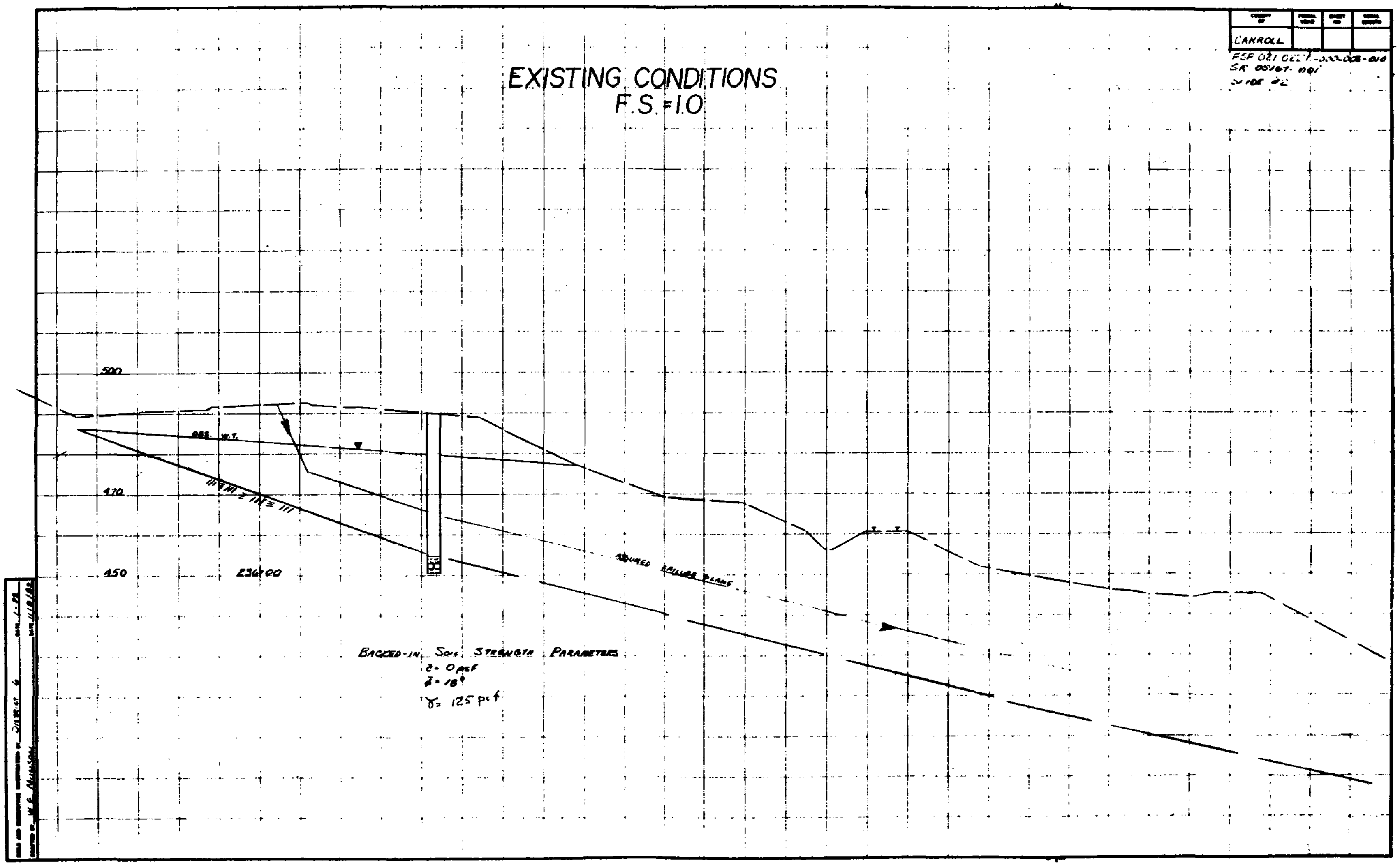
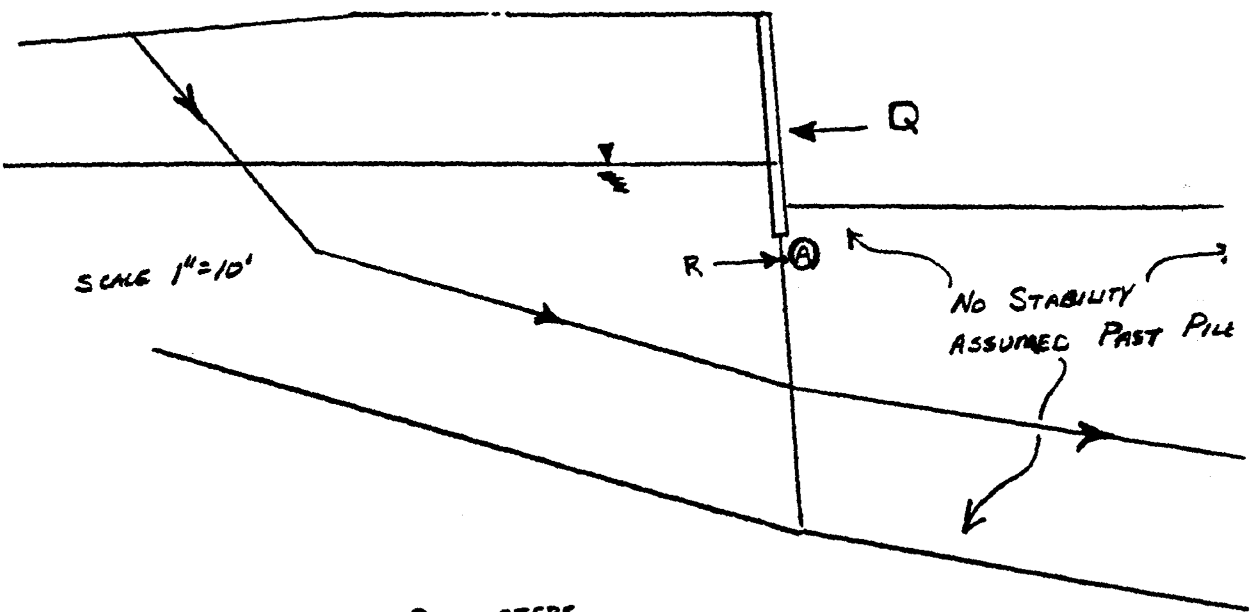


Figure 4. Cross Section Used for Stability Analysis Prior to Remedial Action.

TYPICAL SECTION OF WALL



SOIL STRENGTH PARAMETERS

$c' = 0$ psf

$\phi' = 18°$

R - THE RESULTANT OF SLIDE FORCES

Q - THE MAGNITUDE OF RESTRAINT FORCES + SOIL FRICTION

Q - THE MAGNITUDE OF RESULTANT FORCES

A - ESTIMATED POSITION OF RESULTANT FORCES

R = 34.62 K/FT   SOIL FRICTION = 18.75 K/FT

FOR F.S. = 1.5, FIND Q

$$(1.5)(34.62) = Q + 18.75$$

$$33 \text{ k/ft} = Q$$

Figure 5. Section Used for Calculation of Design Criteria.

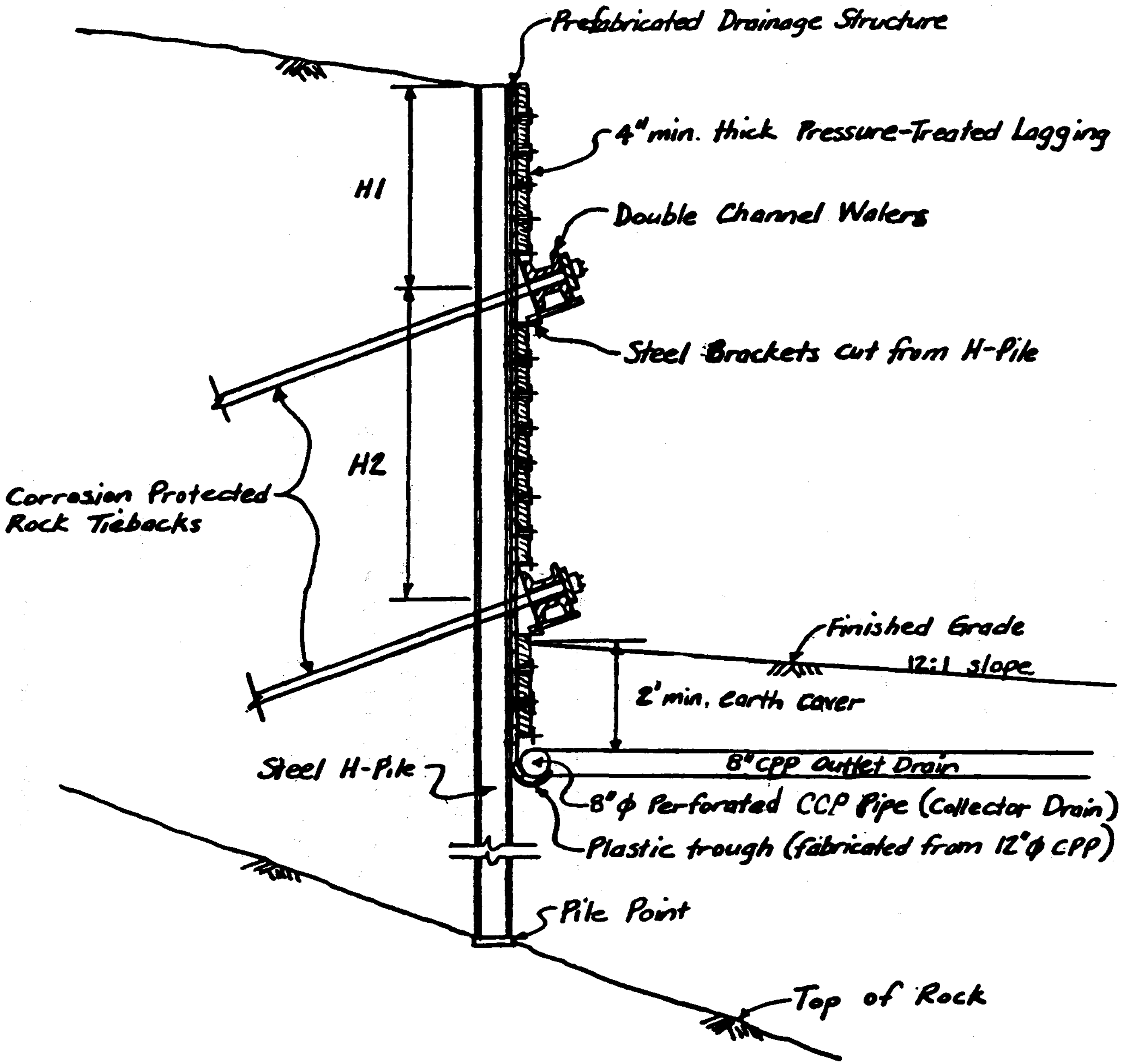


Figure 6. Typical Tieback Wall Section.

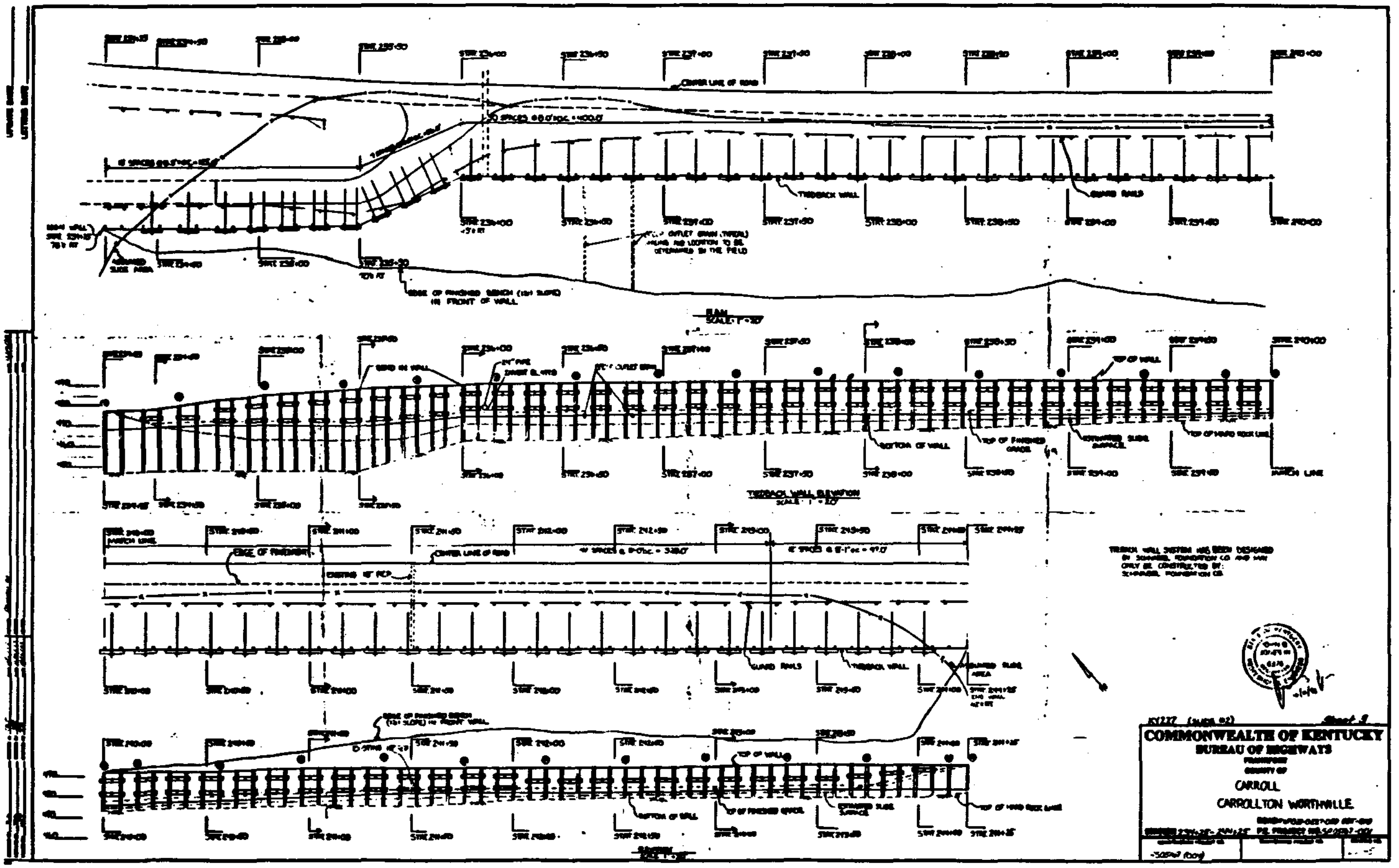


Figure 7. Plan and Front View of Wall with Stationing, Pile Numbers, and Tiebacks Located.

Figure 8. Piling in Place and Excavation of Embankment.

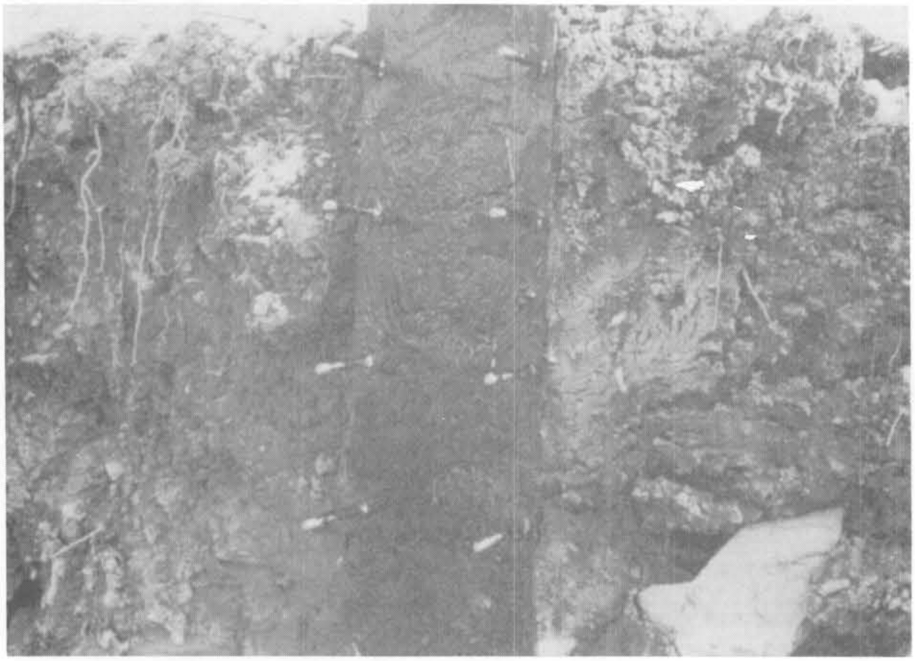


Figure 9. Stud Bolts Welded to Piling with Protective Coat of "Tapecoat TC Mastic".

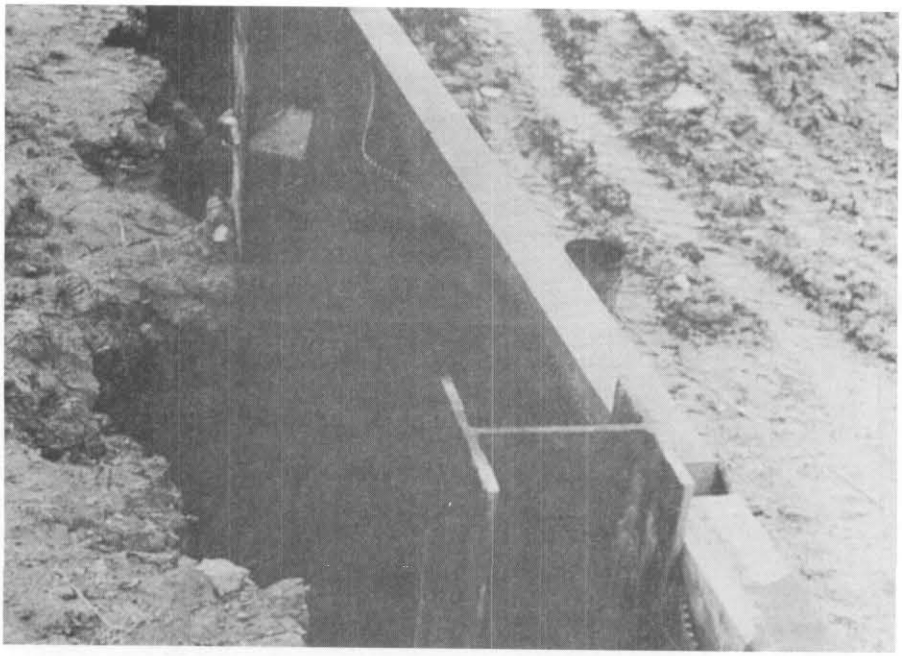


Figure 10. Drainage Pathway of AMOCO 4553 Fabric against The Earth and TENSAR "PWI" Grid against The Lagging.

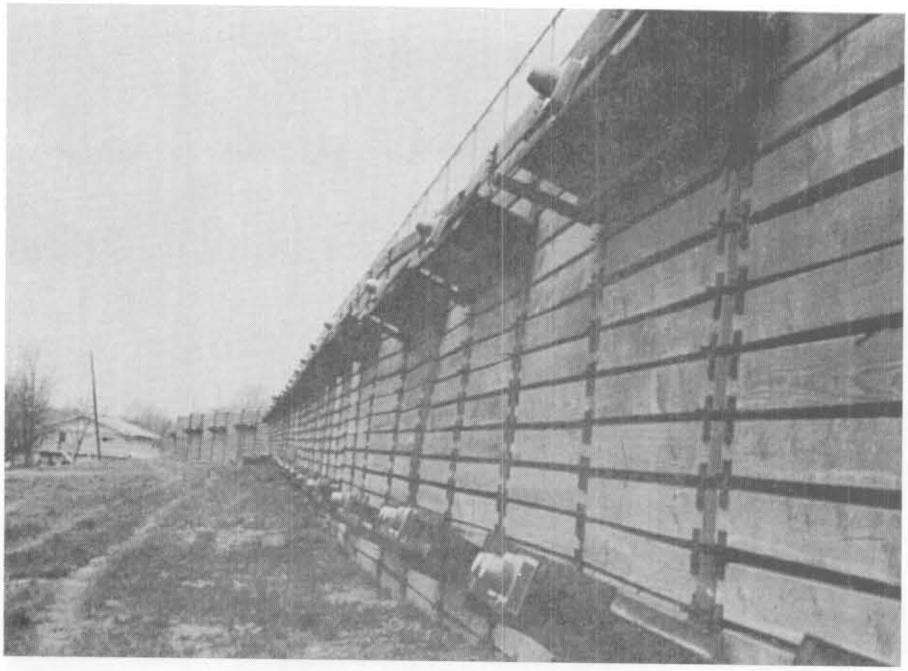
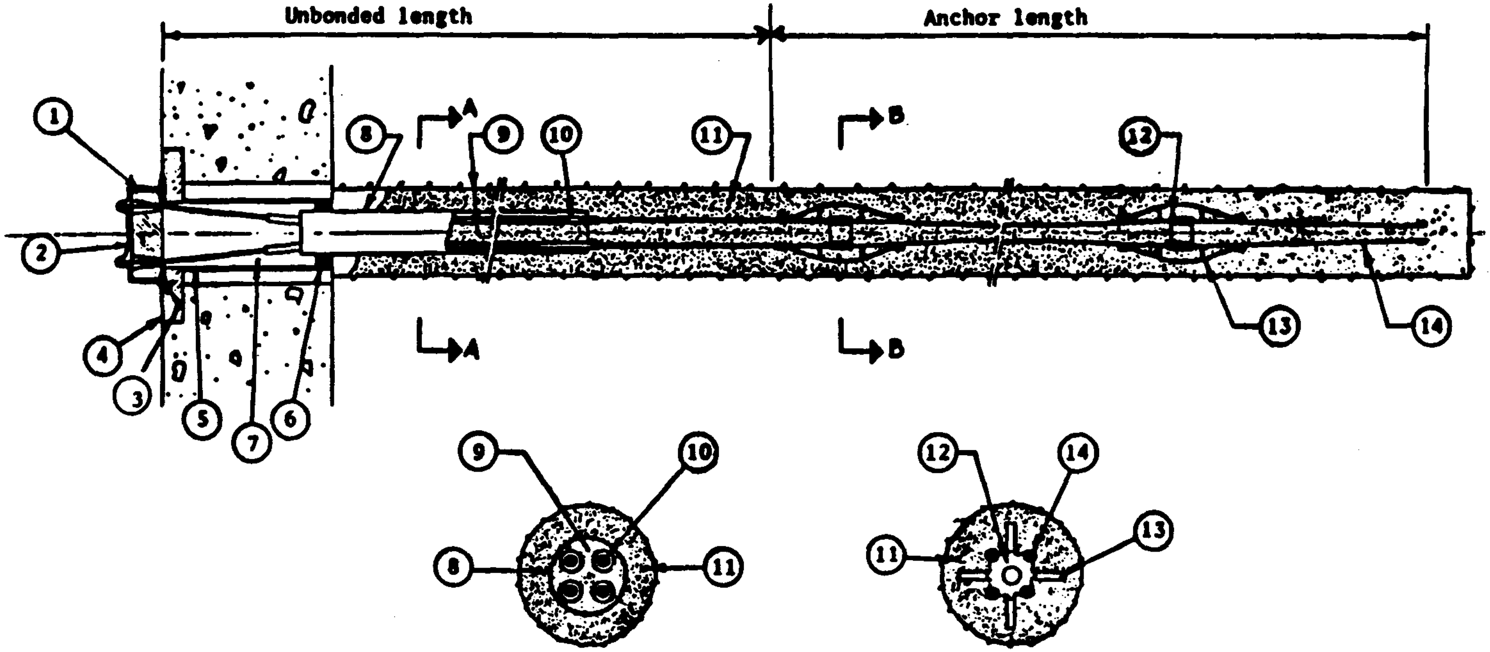


Figure 11. Completed Tieback Control Wall.



Section A-A

Section B-B

Legend:

1. ~~Insulating~~ grout filled or heat ~~shrinkable cover~~.
2. Anchor head and wedges
3. Bearing Plate Insulation
4. Bearing plate
5. Trumpet
6. Seal
7. Anticorrosion grease or grout
8. Steel, PVC or polyethylene tube
9. Grout
10. Individually greased & sheathed strands
11. Anchor grout
12. Spacer
13. Centralizer
14. Tendon

Simple corrosion protected strand tieback.

Figure 12. Typical Tieback Cross Section.

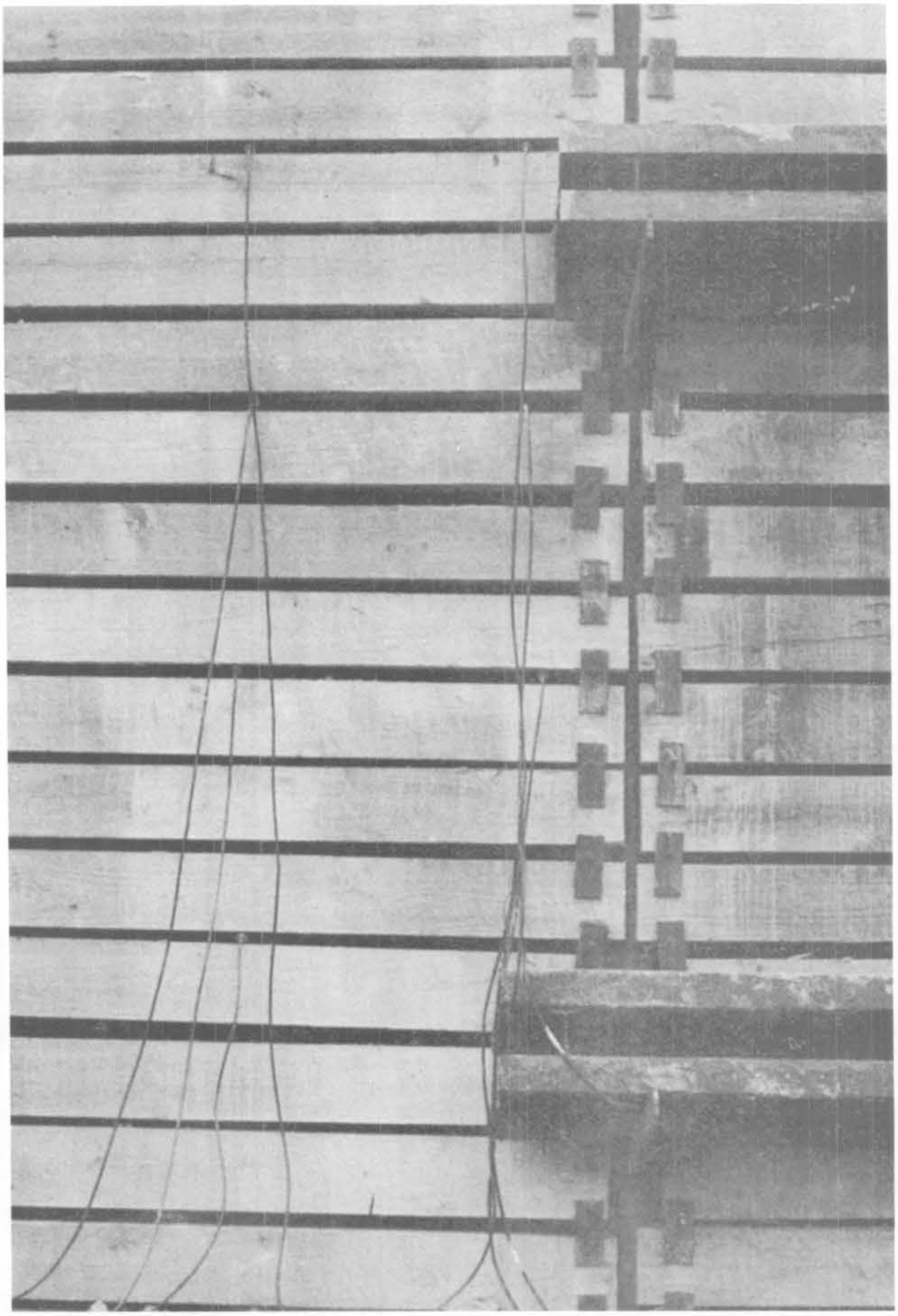


Figure 13. Earth Pressure Meter Locations

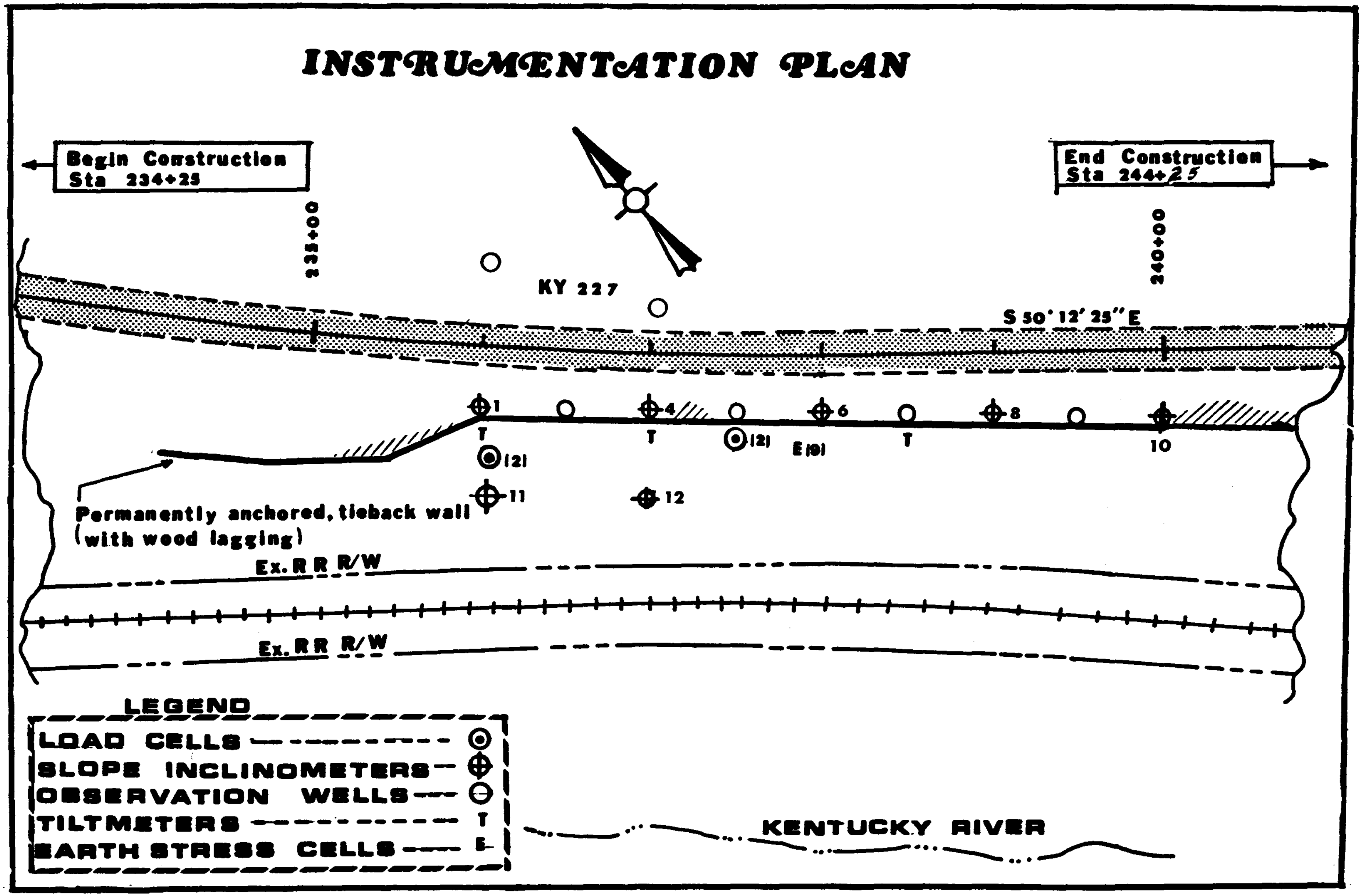


Figure 14. Plan View of Site with Instrumentation Locations.

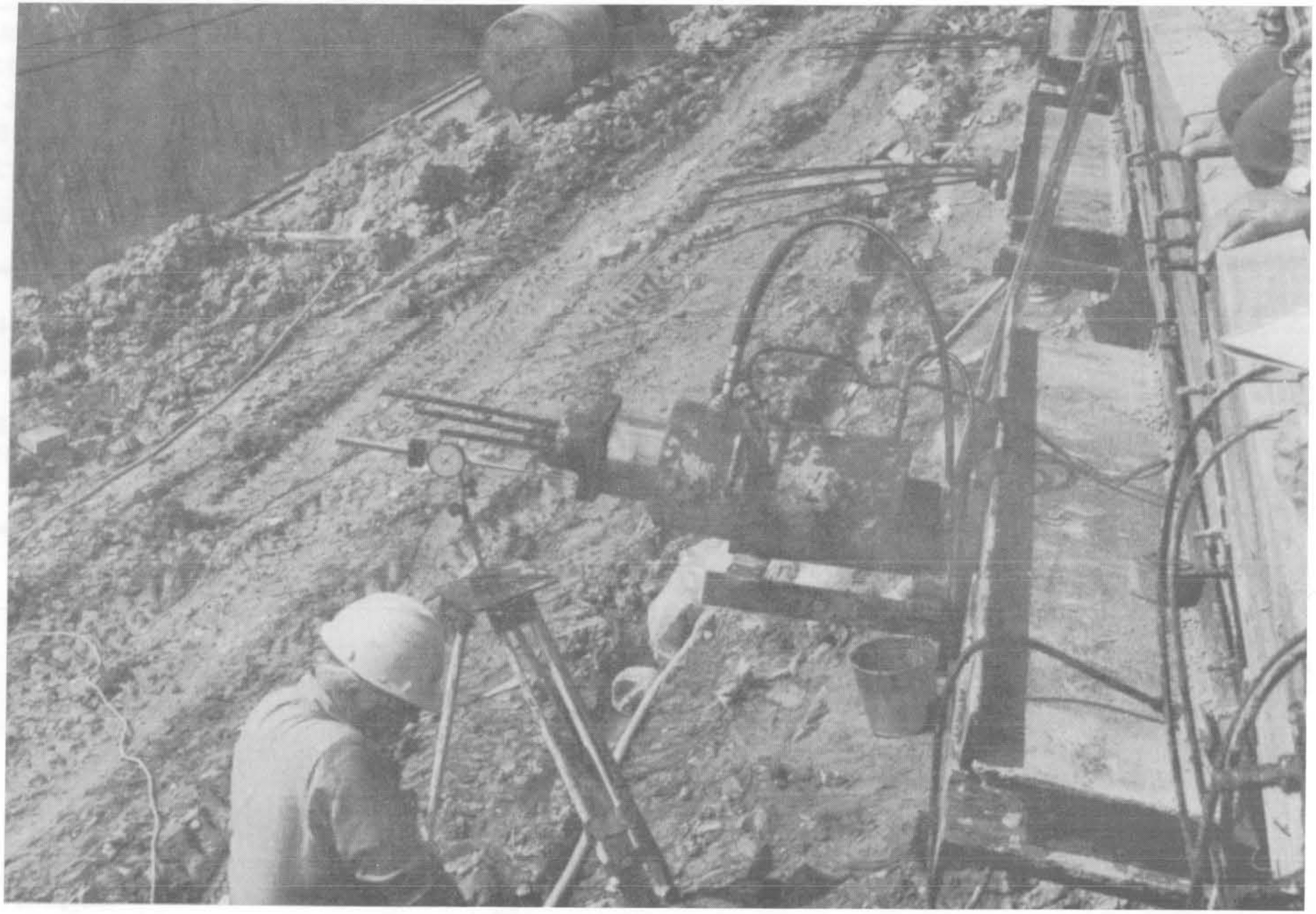


Figure 15. Tieback Testing.

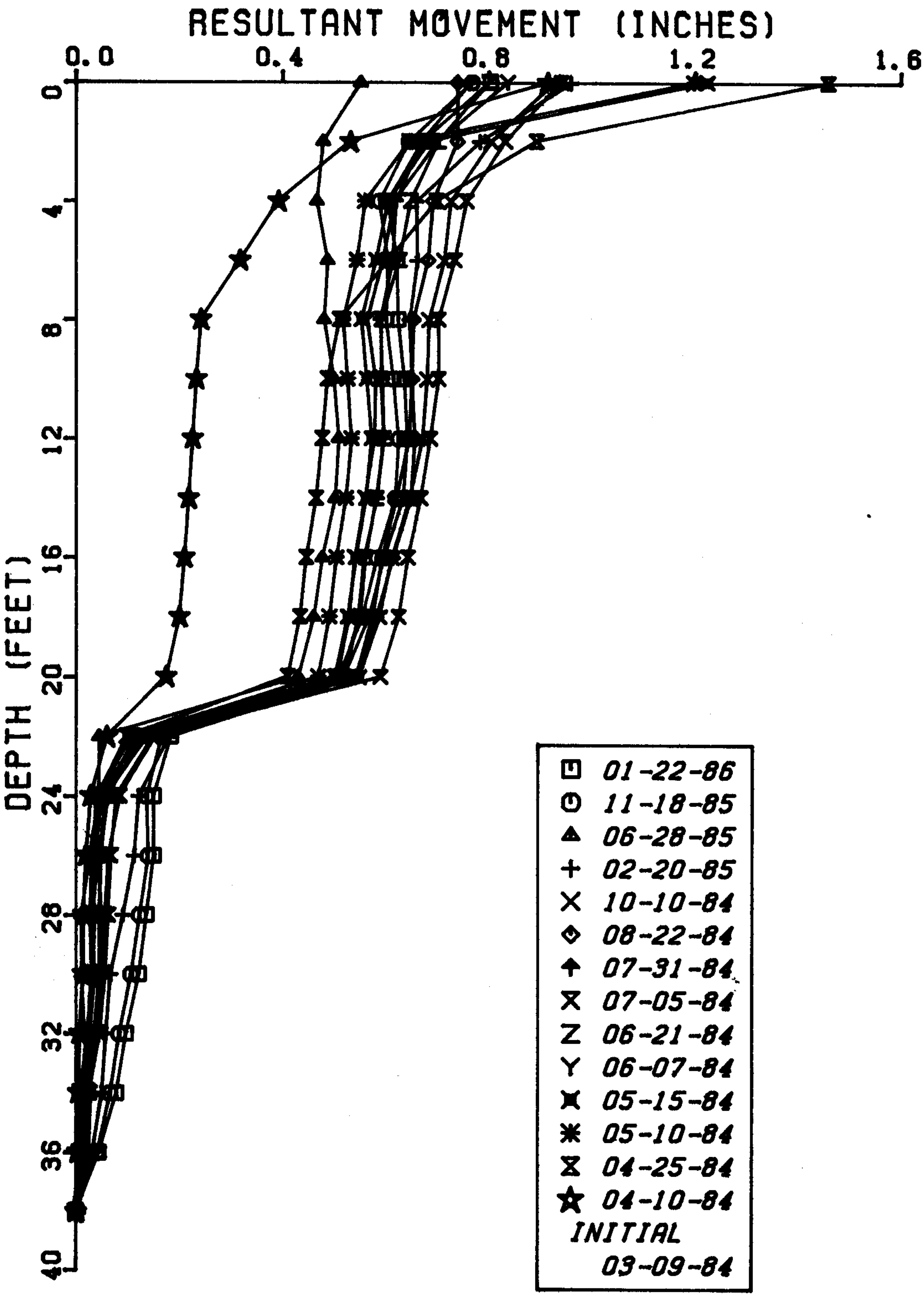


FIGURE 16. SLOPE INCLINOMETER 1 - LATERAL MOVEMENT AT STATION 236 + 00, 35 FEET RIGHT.

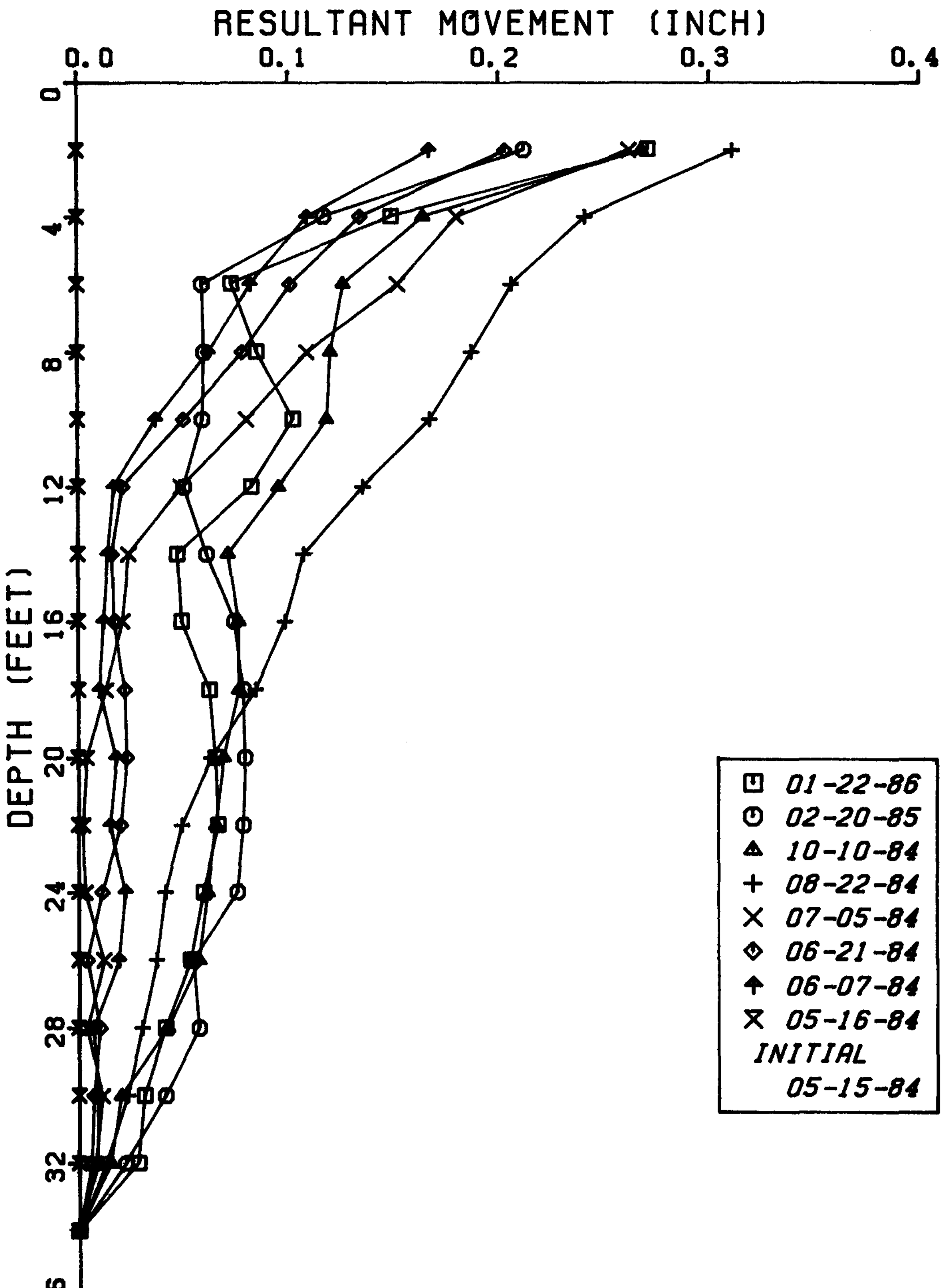


FIGURE 17. SLOPE INCLINOMETER 4 - LATERAL MOVEMENT AT STATION 237 + 00, 30 FEET RIGHT.

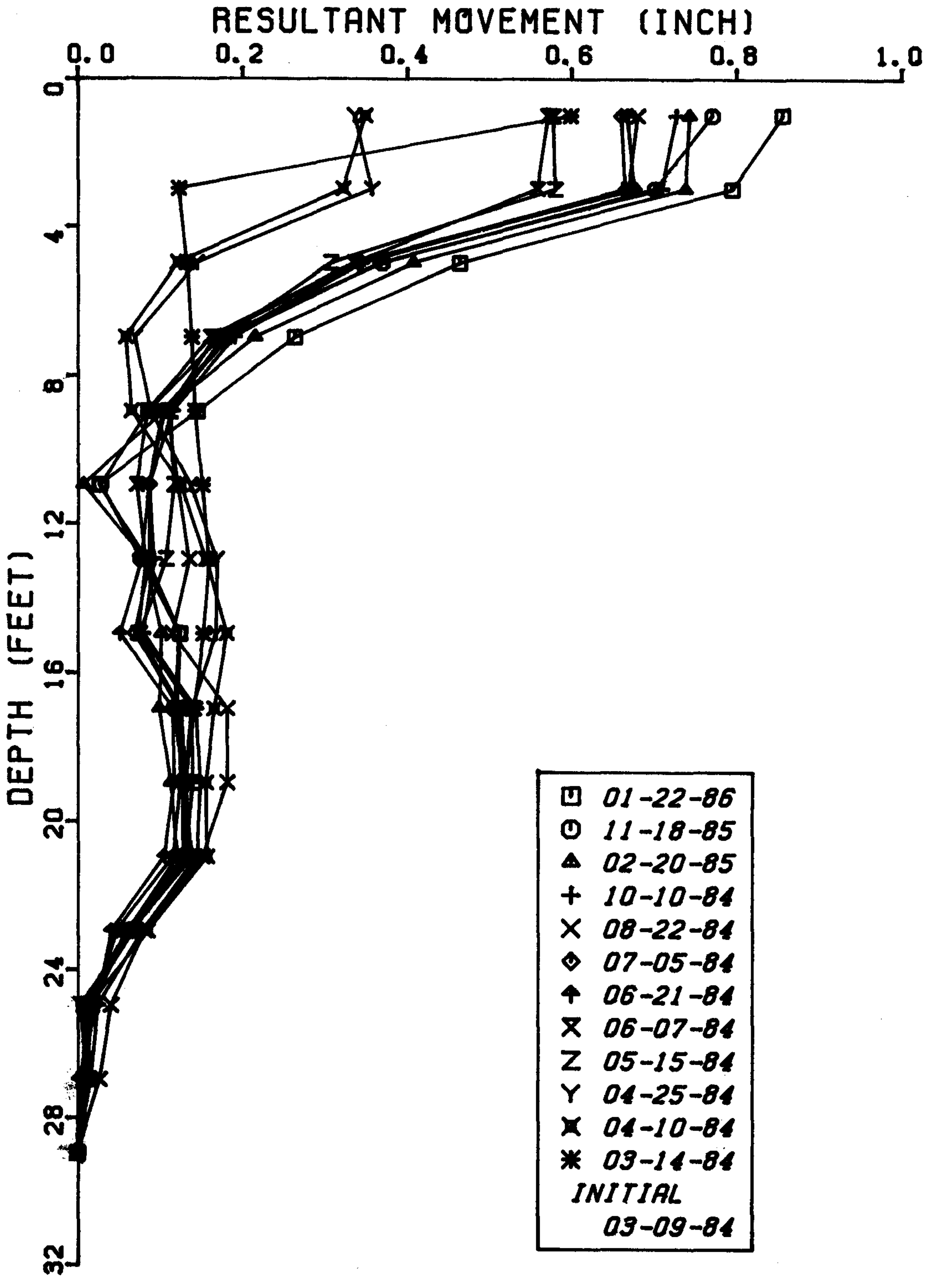


FIGURE 18. SLOPE INCLINOMETER 6 - LATERAL MOVEMENT AT STATION 238 + 00, 30 FEET RIGHT.

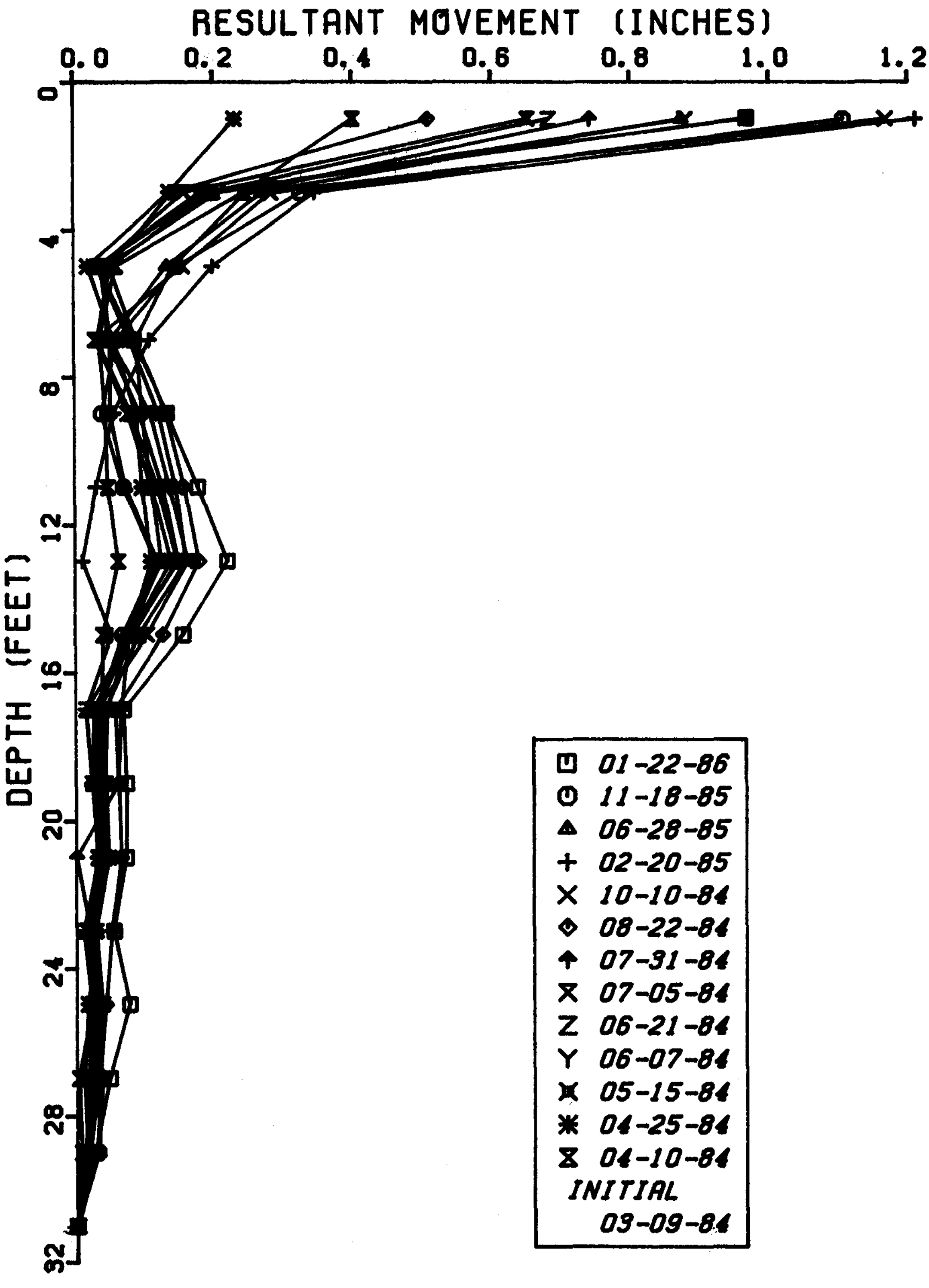


FIGURE 19. SLOPE INCLINOMETER 8 - LATERAL MOVEMENT AT STATION 239 + 00, 30 FEET RIGHT.

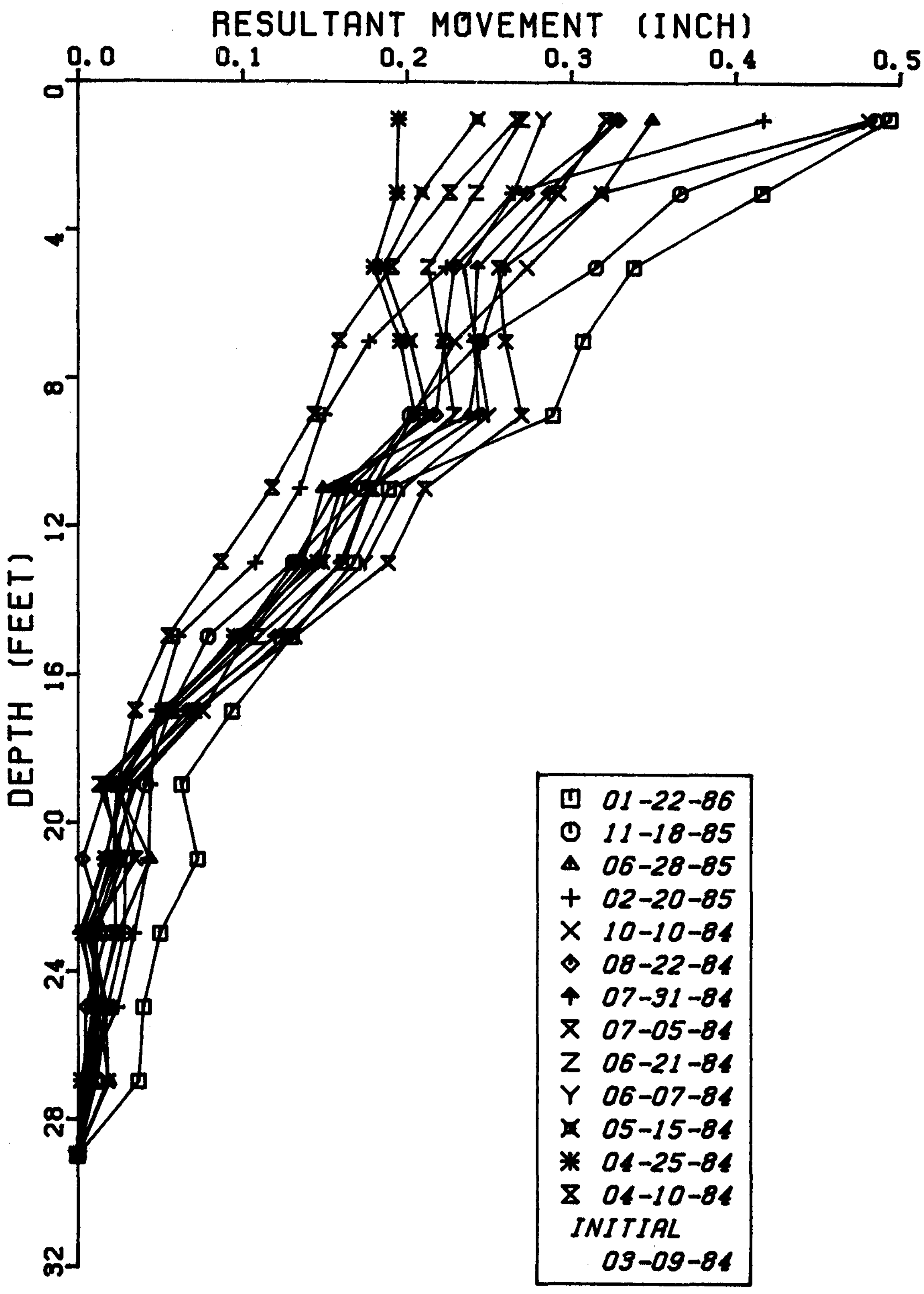


FIGURE 20. SLOPE INCLINOMETER 10 - LATERAL MOVEMENT AT STATION 240 + 00, 30 FEET RIGHT.

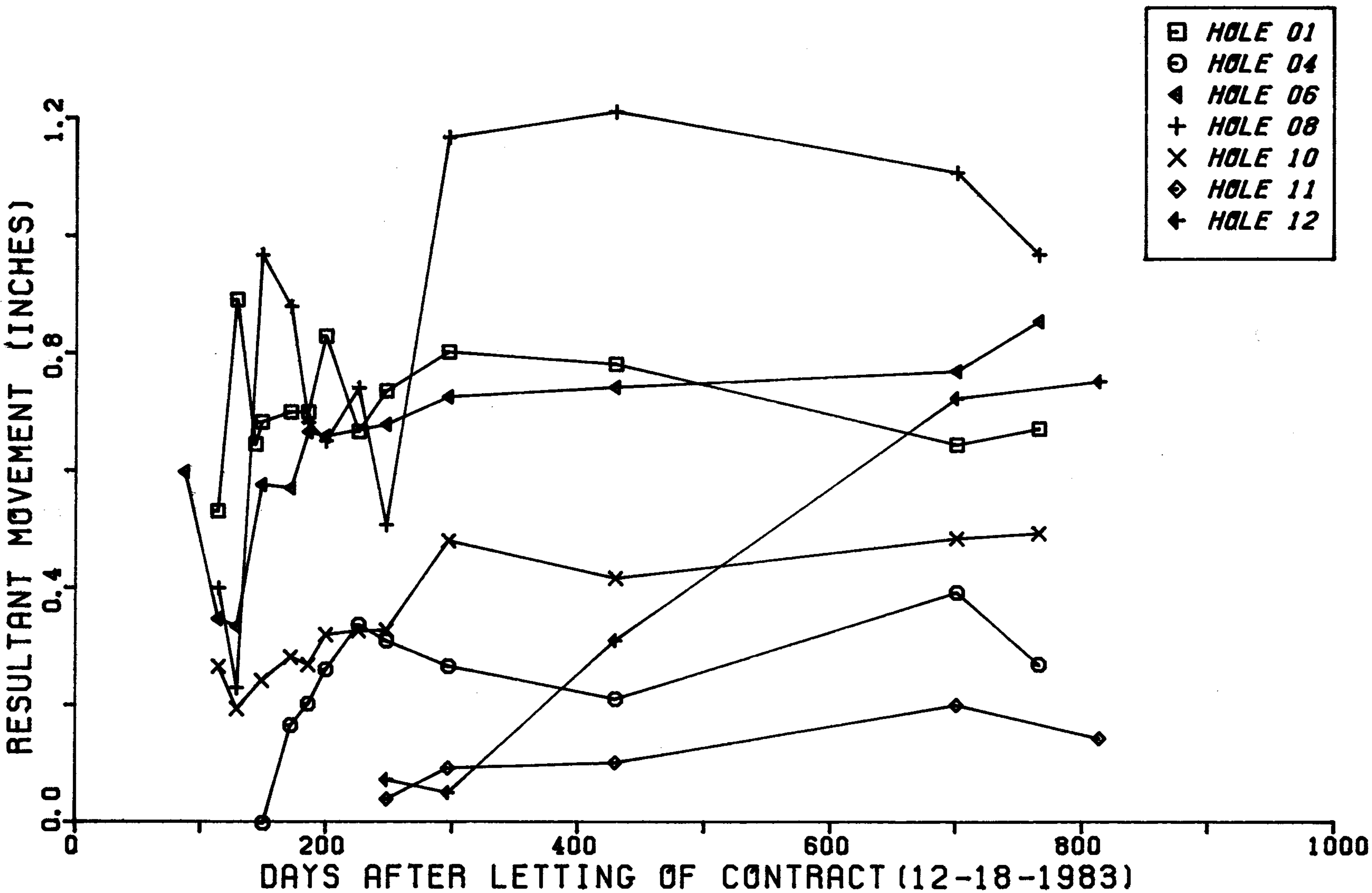


FIGURE 21. MOVEMENT OF TIEBACK WALL SLOPE INCLINOMETERS AT A DEPTH OF 1-2 FEET.

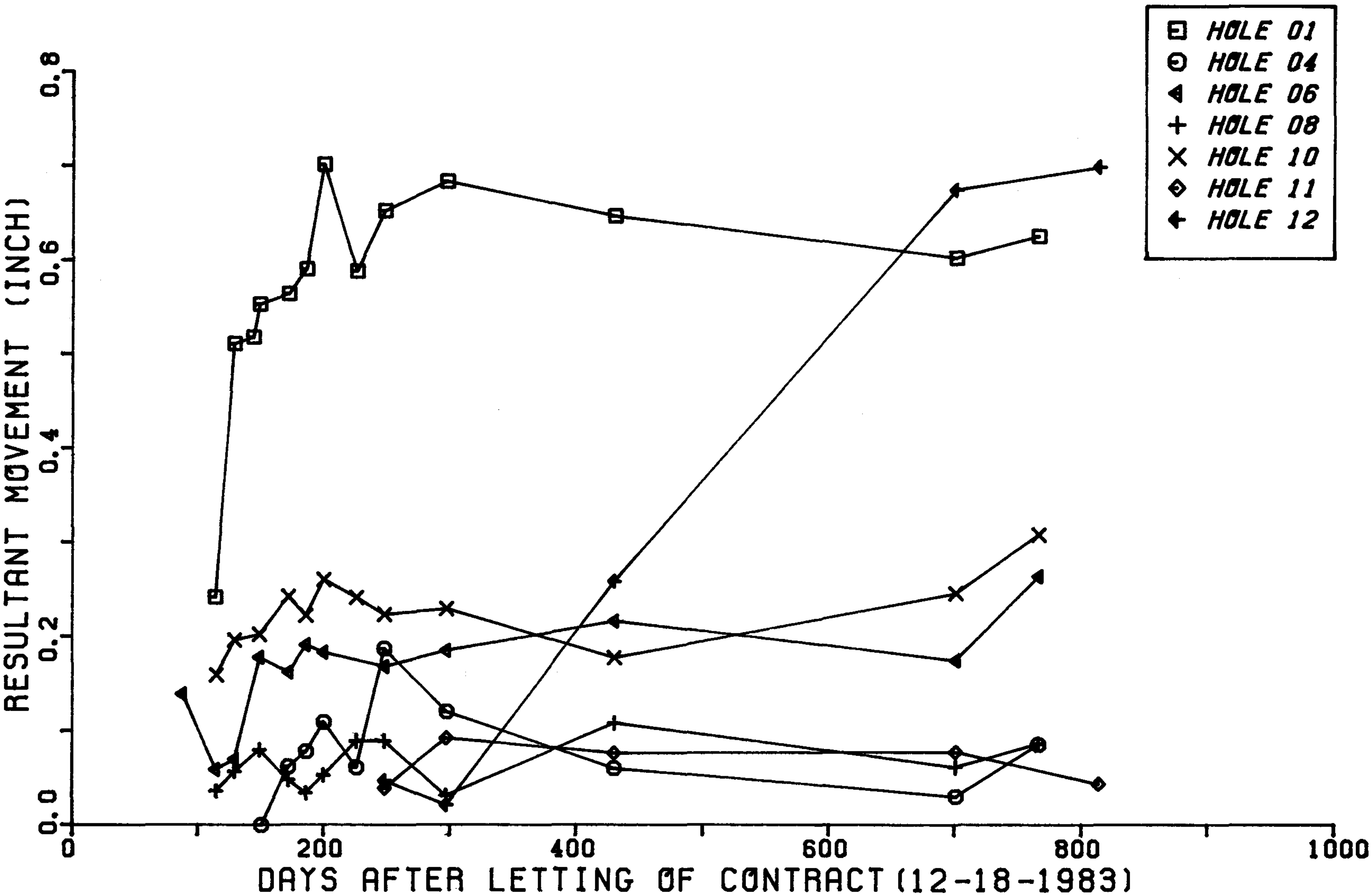


FIGURE 22. MOVEMENT OF TIEBACK WALL SLOPE INCLINOMETERS AT A DEPTH OF 7-8 FEET.

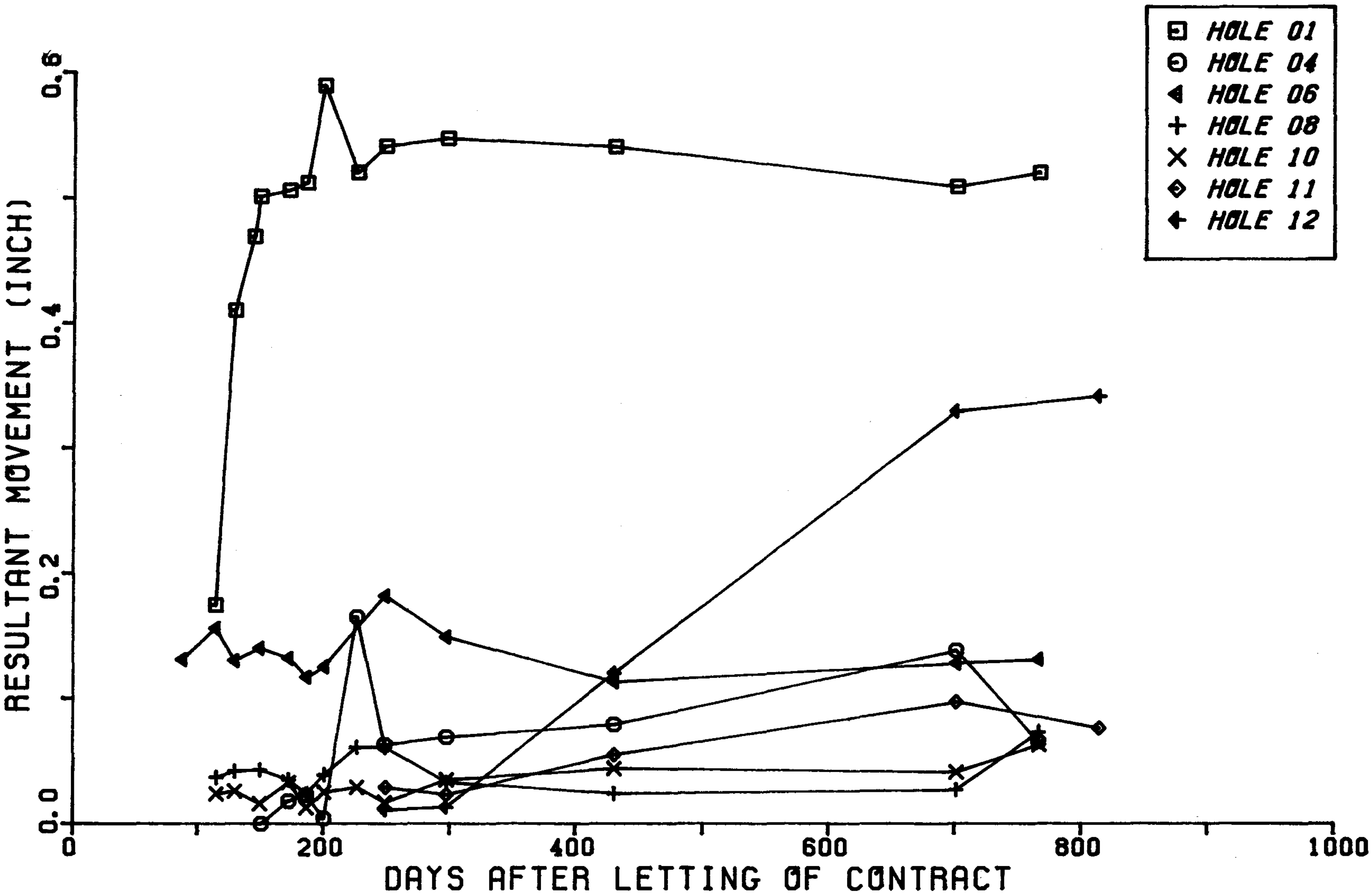


FIGURE 23. MOVEMENT OF TIEBACK WALL SLOPE INCLINOMETERS AT A DEPTH OF 19-20 FEET.

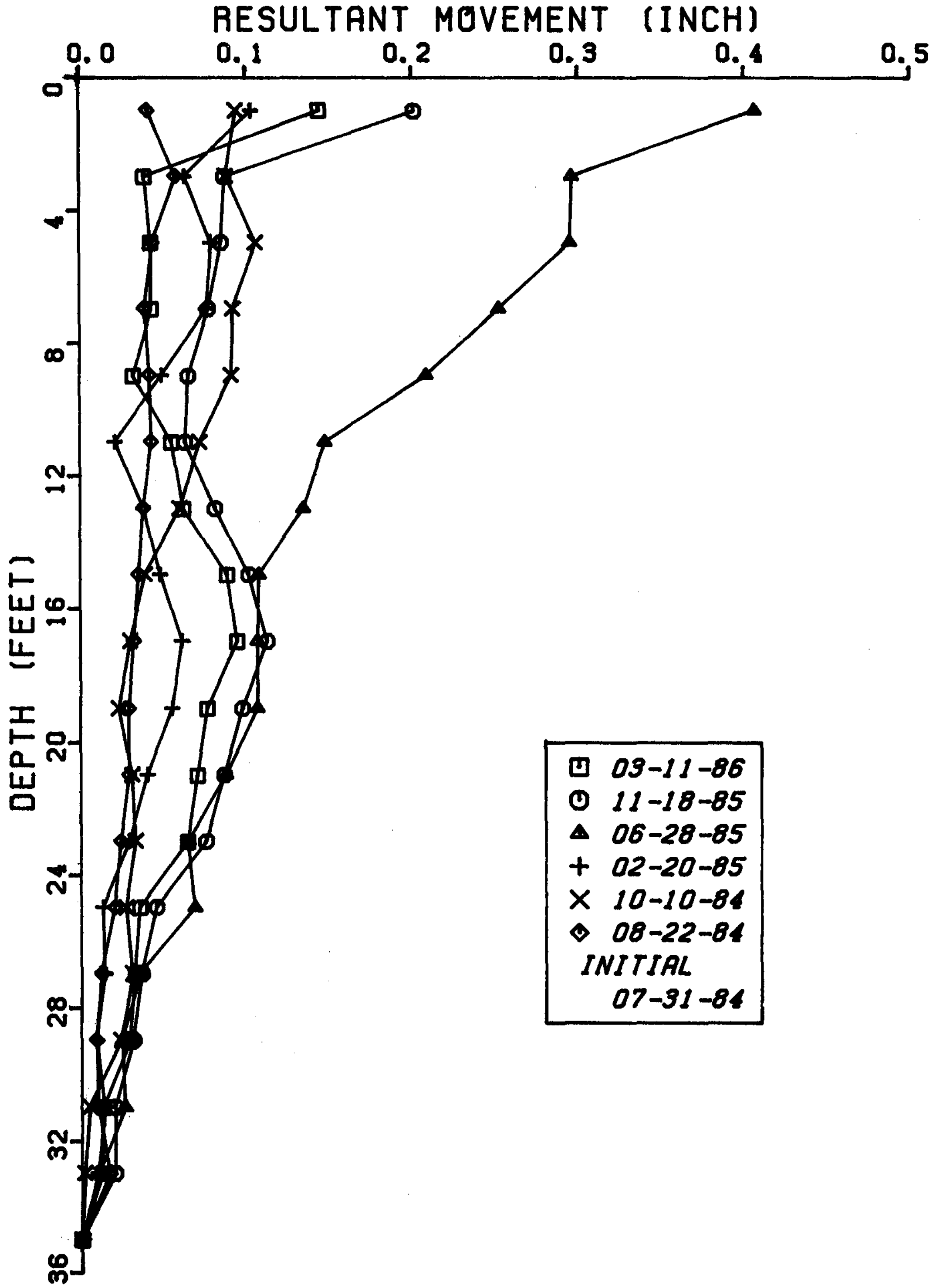


FIGURE 24. SLOPE INCLINOMETER 11 - LATERAL MOVEMENT AT STATION 236 + 00, 85 FEET RIGHT.

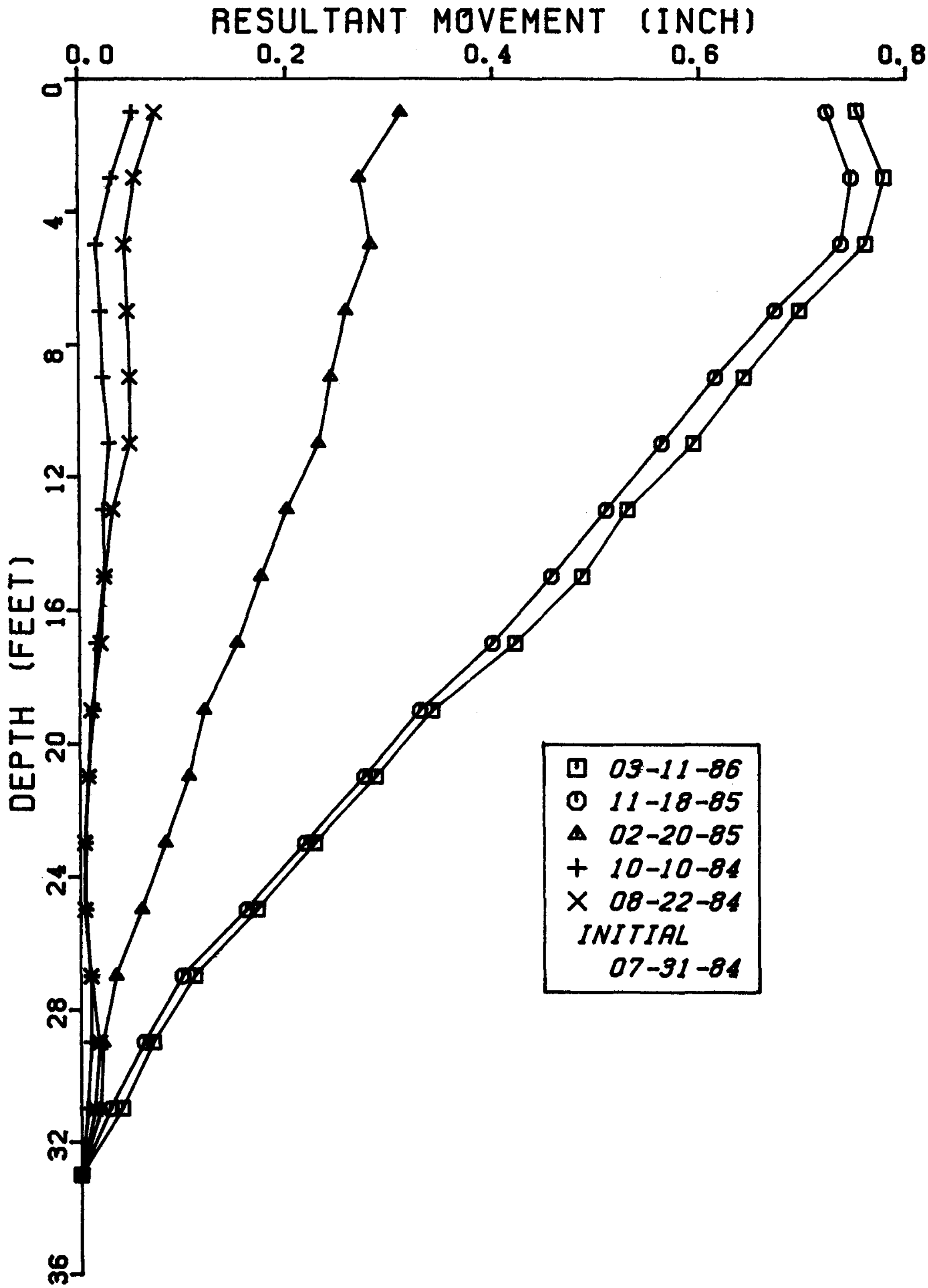


FIGURE 25. SLOPE INCLINOMETER 12 - LATERAL MOVEMENT AT STATION 237 + 00, 85 FEET RIGHT.

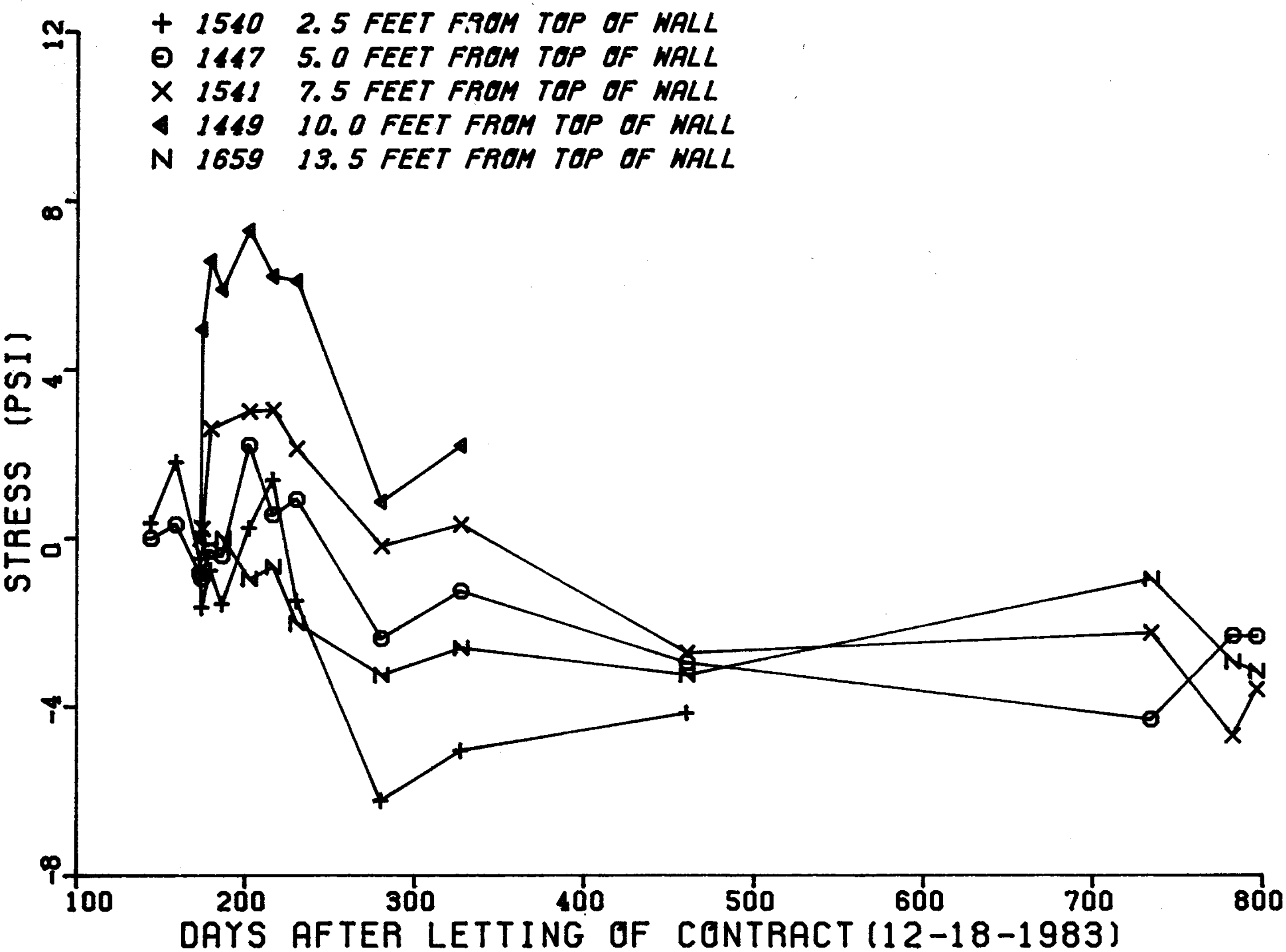


FIGURE 26. EARTH PRESSURE - BAY CENTER, BETWEEN PILES 46 AND 47.

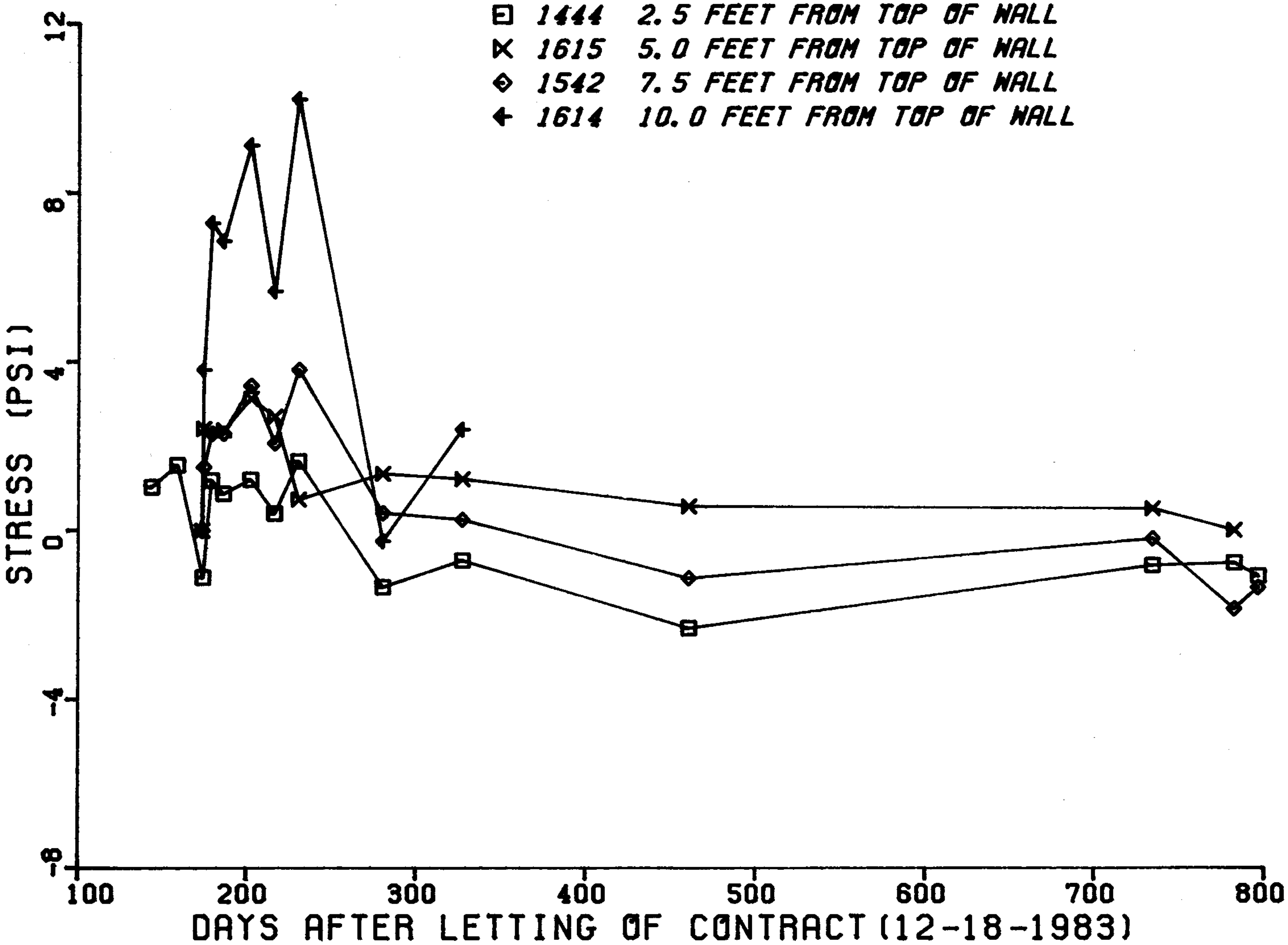


FIGURE 27. EARTH PRESSURE - NEAR PILE NUMBER 47.

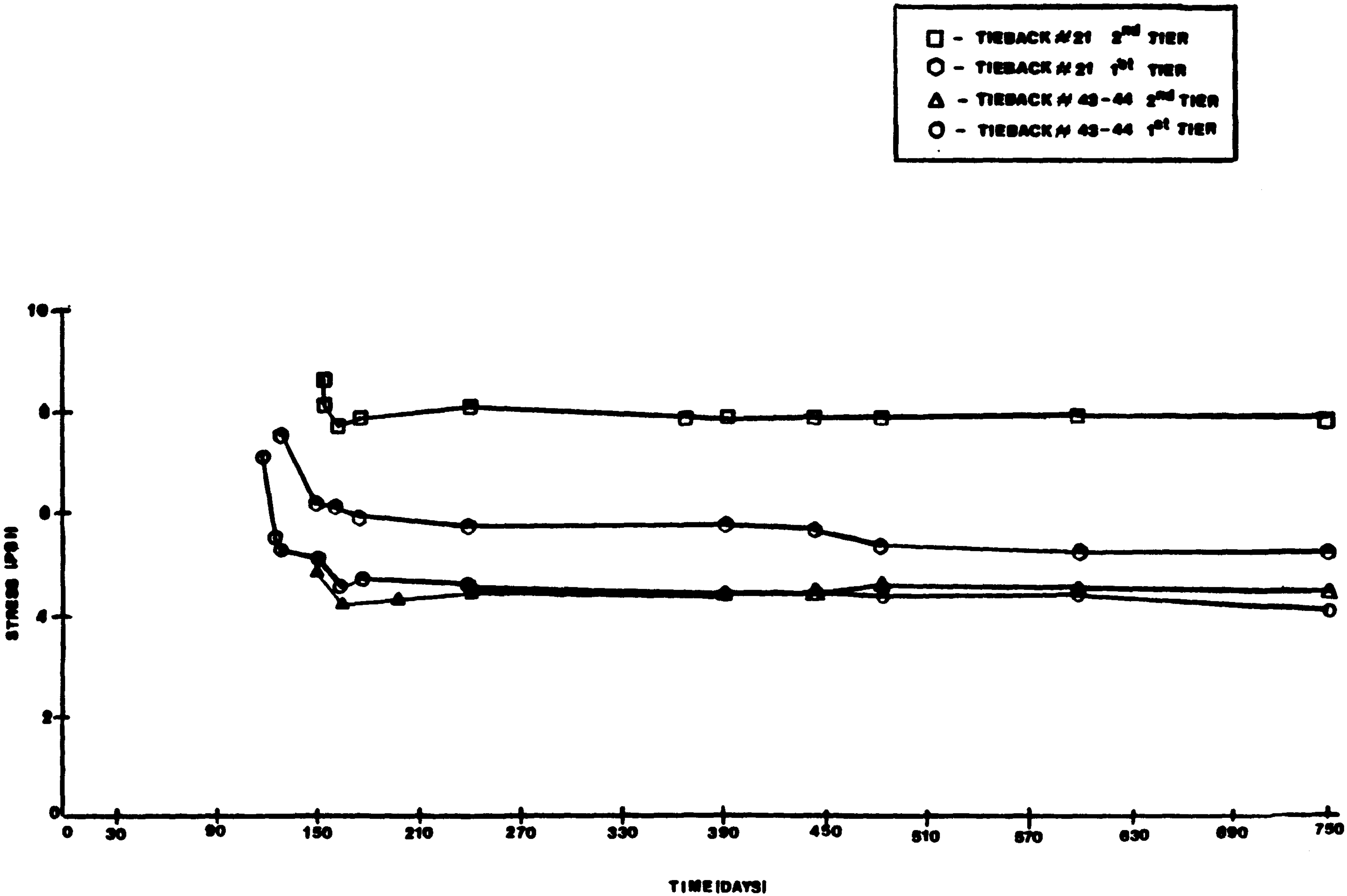


Figure 28. Permanent Load Cells — Tieback Stress.

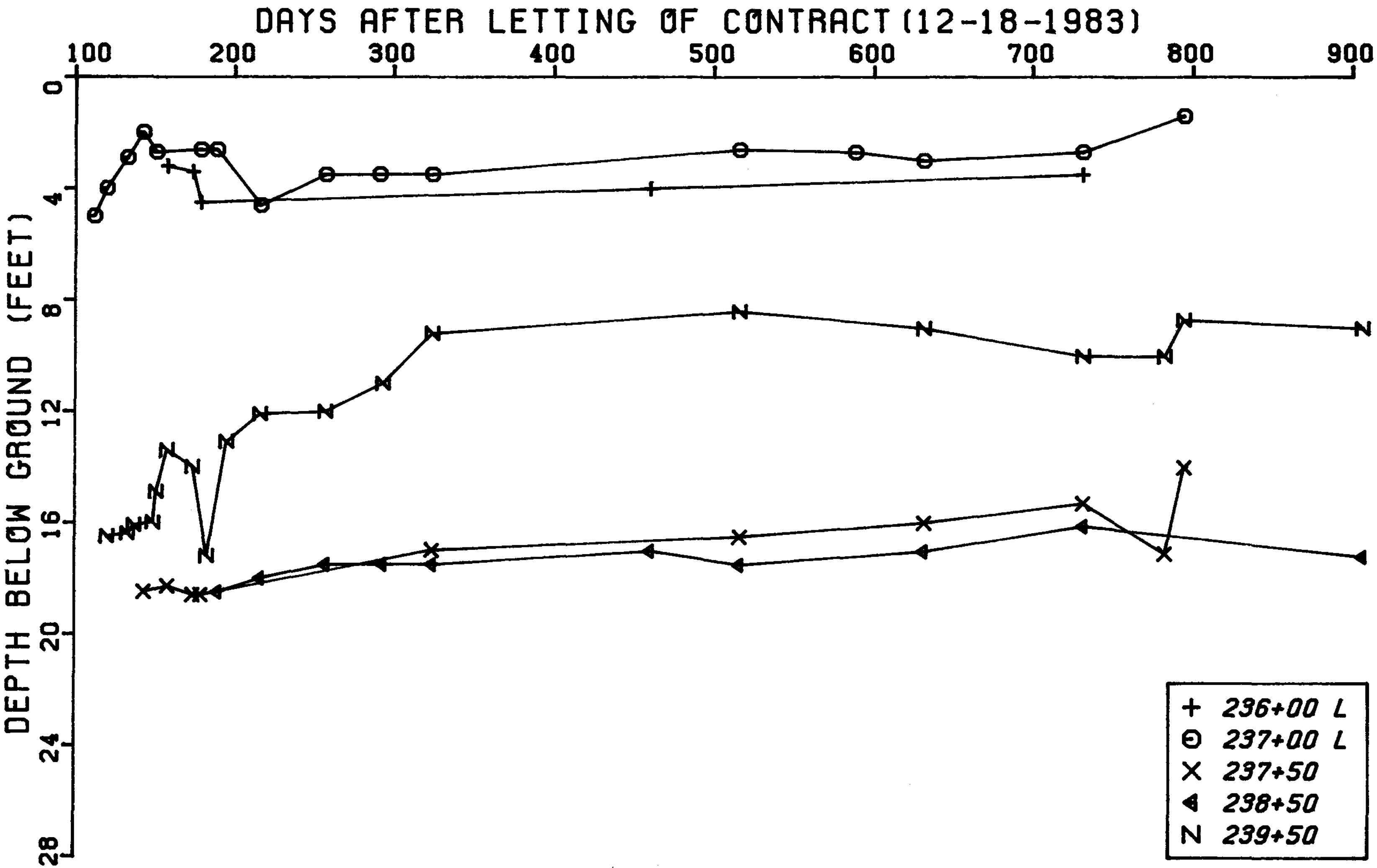


FIGURE 29. WATER TABLE LOCATION.

REPORT NO. 6

PROGRESS REPORT
RETENTION SYSTEM MONITORING

DEMONSTRATION PROJECT NO. 68
NORTH STREET GRADE SEPARATION
LIMA, OHIO

For

OHIO DEPARTMENT OF TRANSPORTATION

INVESTIGATION BY
THE H.C. NUTTING COMPANY
CINCINNATI, OHIO

1988

TABLE OF CONTENTS

I. DEMONSTRATION PROJECT NO. 68

A. PROJECT LOCATION

The project (389-86) is located within the City of Lima, Allen County, Ohio. The project, ALL-S.R. 81-16.83 North Street Grade Separation, provides construction of an underpass below the railway serving the C & O/B & O Railroad Company and the N & W Railway Company. Reference is made to the location map attached with this report.

B. SITE GEOLOGY

The soils within the project site were deposited as glacial ground moraine during both the Illinoian and Wisconsin glaciation. The thickness of the soil cover above the bedrock varied from approximately 42 to 52 feet. The underlying bedrock is dolomite of the Tymochtee Member of the Monroe Group.

The project soils are comprised of unsorted, unstratified mix of clay, silt and sand with a minor but variable percentage of gravel size material. The profile includes erratic seams and layers of sand and sand and gravel, discontinuous both horizontally and vertically. Generally, the sand and gravel is waterbearing as a result of surface water infiltration and accumulation in the more permeable granular zones. Typically, the soil possesses a very stiff to hard consistency, whereas the granular more permeable zones demonstrated a compactness varying between medium dense and very dense.

C. TIEDBACK RETAINING WALL PERFORMANCE STUDY

The North Street Grade Separation project in Lima, Ohio features the use of permanent soil anchors, soldier beams, and reinforced concrete facing wall in lieu of conventional cantilever retaining walls. Permanent tieback retention systems eliminate the need for temporary retention in congested areas thereby providing an economical alternative to the construction of standard cantilever walls.

This study involves the monitoring of instrumentation to develop useful data for evaluating the long-term performance of the permanent tiedback retaining walls. The data provides documented magnitudes of stress within the components of the wall system which can be related to construction stages, field related conditions and design and construction effectiveness.

D. DESCRIPTION OF PERMANENT TIEDBACK RETAINING WALL COMPONENTS

Reference is made to shop drawings prepared by The Schnabel Foundation Company as well as the project plans prepared by the State of Ohio, Department of Transportation. Relevant drawings and plan sheets have been included in the appendix.

The instrumentation and monitoring was performed at wall unit No. 71, soldier beam No. 93 and wall unit No. 22, soldier beam No. 24. Thirty-inch diameter drilled shafts were installed on 6 ft. centers. Between the bottom of the drilled shaft and the bottom of the wall footing the drilled shaft concrete was ODOT Class C mix which has a design compressive strength of 4000 psi. From the bottom of wall footing up to the top of the shaft, a lean concrete mix was utilized. The 7-day minimum compressive strength for the lean concrete was 1000 psi. Drilled shaft soldier beams included double channel C-15x33.9 with a clear interior spacing between channels of 14". The double channels were to be placed at plan location within the 30" diameter drilled shaft. The steel channels were ASTM Designation A-36 Grade Steel.

As the excavation progressed, wood lagging was installed behind the steel beam flanges after removing drilled shaft lean concrete at the exterior face flanges. The lagging consisted of nominal 3" thick rough cut mixed hard woods which were untreated.

The permanent tiebacks were installed at various stages of the excavation in accordance with plan requirements. The drill holes for the anchors were 12" diameter and drilled using a hollow stem auger rig. The tieback tendon was placed within the hollow stem and carried to the end of the drill hole by means of a closure device at the lower end of the drill stem. The closure device was disengaged and the auger stem withdrawn during the grouting process. The grout composition consisted of a 9 bag mix (portland cement Type I), sand and fly ash.

The tiebacks included a double corrosion protection system. The tiebacks were 1-1/4" diameter Dywidag thread bars encapsulated within a corrugated sheathing as supplied by the manufacturer. The 1-1/4" diameter Dywidag bars are 150 ksi grade steel conforming to ASTM A-722.

After the basic retention system was completed, a reinforced concrete facing wall was constructed. The facing wall was anchored to the flanges of the double channel soldier beams by studs welded to the channels. A 2 ft. wide strip footing was constructed at the base of the facing wall.

E. TYPE AND LOCATION OF INSTRUMENTATION

The instrumentation program included the use of digitilt inclinometer casing, load cells and vibrating wire strain gages. The instrumentation was installed at two wall sections; one at soldier beam No. 93, wall unit No. 71 and at soldier beam No. 24, wall unit No. 22.

The instrumentation at soldier beam No. 24 included digitilt inclinometer casing full length within the soldier beam. The casing was attached on the interior side of the south channel flange. Instrumentation at soldier beam No. 24 also included load cells on both the first and second level anchors at the anchor head. Load cells were installed after the individual anchors were load tested.

Instrumentation at soldier beam No. 24 also included strain gages installed directly on the two 1-1/4" diameter Dywidag bar tiebacks. Gages were placed at the midpoint of the unbonded anchor length and at four locations within the bonded anchor length. The gages installed within the unbonded anchor length were 15 ft. beyond the anchor head. The gages within the bonded anchor length were installed at distances of 2 ft., 6 ft., 10 ft. and 18 ft. beyond the beginning of the bonded anchor length. Figure 4 illustrates the typical strain gage and load cell locations for the instrumented tiebacks. As noted, a 30 ft. unbonded anchor length and a 20 ft. bonded anchor length were typical for instrumented tieback anchor installations.

The instrumentation at soldier beam No. 93 was similar to that for soldier beam No. 24, however, additional instrumentation included strain gages placed on the double channel soldier beams (2xC15x33.9). Four horizontal sections through the soldier beams were chosen for strain gage instrumentation. Gages were placed at the same elevation as the level one tieback. This section included four strain gages, one each on the exterior flanges of the soldier beam channels. The second section was chosen at a location midway between the second level tieback and the bottom of facing wall footing. Section 2 included four strain gages, one each on the exterior face of the channel flanges. Section 3 was chosen at a location corresponding to the bottom of the facing wall footing elevation. In addition to the four strain gages on the channel flanges, two strain gages were placed on the center line of each channel web. These gages were placed on the face of the web between the protruding flanges. The instrumentation at Section 4 was located 5 ft. above the bottom of the soldier beam. The instrumentation at this section included two strain gages on the centerline of each channel web similar to the installation at Section 3. This section did not include strain gage instrumentation on the channel flanges.

Figures 1 thru 4 illustrate strain gage, load cell and digitilt inclinometer location details as well as the construction details of the tiedback wall.

II. PURPOSE OF INSTRUMENTATION

The primary purpose of the instrumentation program is monitoring the long-term performance of the permanent soil tiebacks. A secondary benefit of the instrumentation program is the monitoring and evaluation of the overall tiedback wall performance on a short-term and long-term basis.

The primary components of the tieback retention system constructed for the North Street Grade Separation project are the individual soldier beams and tiebacks. The evaluation of tieback system performance relative to these components addressed four key elements:

1. The rate of soldier beam deflection and tieback load changes,

2. The magnitude of soldier beam deflection and tieback loads,

3. The direction of soldier beam deflection, and

4. The specific location and magnitude of deflection along the length of the soldier beam and tieback load development along the anchor length.

Digitilt inclinometers, load cells, and strain gages were utilized to obtain data for evaluation of the four items listed above.

Strain gage installation on the soldier beams permits analysis of both axial and bending stress at various locations along the soldier beam length. This instrumentation, at least to some extent, can be used to evaluate the design performance of the soldier beam components . To a lesser extent, this data can be correlated with the other tiedback wall component data to permit a more thorough evaluation of the overall tiedback retaining wall system performance.

The relationship between the various instrumentation provides some refinement in overall performance evaluation, especially with respect to the load cell data and the data from the strain gages mounted on the tiebacks. Further, the deflection data obtained from the digitilt inclinometer monitoring permits interpretation of tieback load changes. Some additional evaluation benefit is derived through the relationship between the strain gage data on the soldier beam channels and the other instrumentation data.

A. PROJECT INSTRUMENTATION

The following is a list of instrumentation and monitoring equipment utilized for this study.

<u>SINCO</u>: Slope Indicator Company

Digitilt Inclinometer Sensor Probe Model 50325

Digitilt Inclinometer Indicator Model 50306

Digitilt Inclinometer Casing - ABS Plastic
1.90" O.D.x1.50" I.D.

IRAD GAGE:

Vibrating Wire Hollow Load Cell, Model VH-150

Spot-Welded Vibrating Wire Strain Gages, Model SM-2W with thermistors.

Vibrating Wire Manual Switching Stations, Model TB-12S and Model TB-24S

IRAD/KLEIN Smart reader ™ Model SR-11

B. INSTRUMENTATION INSTALLATION

The digitilt inclinometer casing was installed by the specialty contractor, Schnabel Foundation Company, by temporarily fastening the casing with metal straps to the web and flange of the soldier beam channel. After installation of the soldier beam channels into the drilled shaft, the drilled shaft concrete secured the casing permanently into place within the soldier beam. The digitilt inclinometer casing extended full depth within each of the monitored soldier beam locations. Figures 1 thru 3 illustrate the location of the digitilt inclinometer casing and the reference elevations for the subsequent monitoring readings.

Strain gage installation on the 1-1/4" Dywidag thread bars was performed by a representative of IRAD GAGE. The installation was performed at the Dywidag Systems International USA, Inc. production plant in LeMont, Illinois. Installation of the strain gages at the plant was necessary due to the fact that the tiebacks were required to have a double corrosion protection system. The gages, therefore, had to be installed prior to encapsulating the Dywidag bars within the corrugated sheathing.

The strain gages were installed on the tiebacks at the locations illustrated in Figure 4. The strain gages were spot-welded to the Dywidag bar after standard surface preparation was completed. The strain gages were protected by a standard coil/magnet cover. Each of the strain gages included a thermistor for monitoring temperature at selected gage locations. Since the tiebacks are tension elements, the initial strain gage wire tension was adjusted in order to increase the tensile strain range of the gage.

In order to facilitate installation of the instrumented tiebacks during construction, it was necessary to cut each of the strain gage read-out cables so that the cables did not extend beyond the head end of the tieback within the hollow stem augers. Each of the cables was labeled with a corresponding reference to a gage location. After installation of the tiebacks, it was necessary to splice each of the read-out cables so that there was sufficient length of cable to reach the final location of the switching stations.

Strain gage installation on the double channel beams for soldier beam No. 93 were performed by a representative of IRAD GAGE. This installation was made at the job site within a secured area. Again, the gages and coil/magnet covers were spot-welded to the channel beams after standard surface preparation. Figures 2 and 3 illustrate the locations of the strain gages installed on the channel beams.

Efforts were made to protect the instrumentation on the channel beams from potential damage during installation and concrete placement. Small steel plate sections were welded over each strain gage cover. The read-out cables from the strain gages were placed within 3/4" diameter PVC pipe which was extended to the top of the soldier beams. The PVC pipe and limited sections of exposed read-out cable were secured with liquid adhesive.

The load cell was installed on each of the tiebacks after the load testing at each location had been completed. The load cell was sandwiched between two anchor bearing plates, the tieback tensioned to lock-off load and the anchor nut then secured in place. The load cells were installed by the specialty contractor. The lead cables from the load cells were protected in 3/4" diameter PVC pipe which was attached to the wood lagging to inhibit damage during construction.

During the later stages of wall construction, the digitilt inclinometer casing which protruded above the wall beam cap was protected within a permanent steel casing with a lockable hinged cap. The vibrating wire switching stations which were utilized to facilitate strain gage readings were placed in standard concrete electrical pull boxes immediately behind the wall location of soldier beam Nos. 24 and 93. The pull boxes and the steel casing over the digitilt inclinometer casing provides reasonable protection against vandalism and accidental damage.

C. INSTRUMENTATION DAMAGE DURING TIEBACK WALL CONSTRUCTION

During the early stages of tieback wall construction, two gages which were located on channel flanges were damaged. These two gage locations were at Section 1 on the south flanges of the channel beams facing the excavation. These gages were damaged during the removal of the drilled shaft lean concrete along the channel flanges in order to provide clearance for the lagging installation and reinforced concrete facing wall construction.

Eight of the read-out cables for the strain gage and load cell instrumentation were damaged by the excavation equipment at soldier beam No. 93. However, the read-out cables were repaired by splicing additional length of cable after removing the damaged section of cable.

As indicated in Tables I thru V, a number of the strain gages were non-functioning at some point in time after initial readings were made. The non-functioning gages include gage Nos. 20, 35, 42,45 and 48. Gage No. 43 could have been over-stressed during tieback anchor stressing to lock-off load, however, this is not certain. The reason that the other gages became non-functional was not able to be determined.

D. INSTRUMENTATION MONITORING

The instrumentation monitoring commenced July 16, 1987. The soldier beam installation had been completed at soldier beam location Nos. 24 and 93. initial excavation along the wall had also been made and the wood lagging operations were in progress. The initial monitoring of the instrumentation established the base readings for referencing subsequent instrumentation data.

For all gages other than the load cells, initial instrumentation readings were not possible until after field installation of the tieback and soldier beam components. As an example, the instrumented tiebacks and channel beams were subjected to different strains in the installed position of these elements as compared to their pre-installation strains.

The instrumentation monitoring reflected various stages of in situ moisture content with respect to the difference between the liquid limit and the plastic limit of the soil. The test results indicated a consistency index varying between 1 and 1.3. This range exceeds 0.9, therefore, indicating non-creep susceptible cohesive soils. Other generally accepted criteria requires that in situ moisture content be at or below the plastic limit of the soil. Project soils satisfy this criteria as well. Unconfined compressive strengths vary between 2.35 and 4.22 tons per sq. ft. which exceeds the minimum 1 ton per sq. ft. criteria for non-creep susceptible cohesive soils. Effective strength parameters of the soil were determined by performing consolidated undrained triaxial compression tests with pore pressure measurements. The testing indicated an angle of internal friction of 35° and a cohesion value of 340 lbs. per sq. ft.

B. DIGITILT INCLINOMETERS

The digitilt inclinometer determined deflection perpendicular to the wall (north-south primary orientation) as well as parallel to the wall (east-west secondary orientation) at soldier beam Nos. 24 and 93. On two occasions during the monitoring process, it was not possible to determine secondary deflection due to malfunctioning of the sensors within the probe.

Figures 6 thru 11 provide graphic plots of the deflection data. The plots also include the locations of the two tieback levels, the bottom of wall footing and other relevant data. It should be noted that the top of casing elevation shown on the plots represents the elevation at the cable clip on the inclinometer assembly. This elevation also serves as the zero depth basis. According to the equipment manufacturer, the digitilt inclinometer data has an overall accuracy of 0.3" per 100 ft., of inclinometer casing length. Based on the inclinometer casing lengths for this project, overall accuracy is on the order of 0.1 to 0.15".

The inclinometer data obtained in the primary orientation at soldier beam No. 24 indicates essentially no deflection occurred below the level of the first tieback anchor subsequent to the first two readings. Reading No. 1 was made after tensioning of the first level tieback and the data illustrates that the top of the soldier beam was pulled toward the retained soil approximately 0.25". As as results, the point of soldier beam rotation relative to the base readings occurred at a depth of 13 ft. or approximately elevation 863.5. The second reading was made after the second level tieback had been tensioned. The excavation locally adjacent to the wall had also been taken to the bottom of wall footing elevation. This and subsequent readings indicated that below the first tieback level deflection did not exceed approximately 0.1". The portion of soldier beam above the first tieback level behaved similarly to a cantilever beam as demonstrated by deflection data from the subsequent readings. The subsequent readings indicated that the top of soldier beam deflected toward the excavation on the order of 0.3" relative to data developed during reading No. 1. As clearly demonstrated by the later readings, the upper portion of the soldier beam has rotated about a point corresponding to the first tieback level.

Secondary orientation inclinometer data for soldier beam No. 24 indicated initial movement of the soldier beam in an eastward direction as reaction to the tensioning of the first level tieback. The upper half of the soldier beam deflected approximately 0.1" to the east. Following lock-off of the second level tieback, the soldier beam returned essentially to its initial location. Subsequent to August 5, 1987, there was a stabilization of soldier beam deflection in an east-west direction. The upper half of the soldier beam experienced deflections within the range of zero to 0.1" west of its initial location. It is probable that completion of the concrete facing wall by August 21, 1987, accounted for stabilization of movement in the east-west direction.

Data developed at soldier beam No. 93 in the primary axis indicated deflection similar to that at soldier beam No. 24. Data from reading No. 1 showed that the top of soldier beam was pulled into the retained soil approximately 0.15" as a direct result of tensioning the first level tieback. Reading No. 2 was made immediately following tensioning of the second level tieback. This reading clearly indicated the impact of tensioning the second level tieback relative to soldier beam deflection. Immediate soldier beam deflection was on the order of 0.05" within the local area of the second level tieback with magnitude of deflection diminishing toward the top and bottom of the soldier beam. The first two readings indicated that the point of rotation was at an approximate depth of 12 ft. or elevation 865. Reading No. 3 was taken following local excavation to the wall footing level. In response, the soldier beam deflected toward the excavation between 0.05" and 0.1". Subsequent readings indicated a slight increase in deflection toward the excavation with a maximum value near the level of the second tieback of 0.2". The soldier beam deflection demonstrated a broad parabolic deflection distribution. It would also appear that the soldier beam deflection has stabilized since the end of September 1987. Data at soldier beam No. 93 in the secondary axis illustrated a distinct reverse curvature of the soldier beam at a depth of approximately 18 ft. It is probable that this phenomenon is the result of excavation equipment contact during excavation. Reading 2 provided deflection data immediately following tensioning of the second level tieback. Tensioning of the second level tieback reduced the localized protrusion of the soldier beam eastward at the 18 ft. depth level. At this point, the maximum eastward deflection was reduced to 0.15" with deflection decreasing to initial soldier beam location at the top and bottom. Reading No. 3 demonstrated the effects of excavation to the bottom of wall footing elevation. Deflection increased to the east as much as 0.15" within the portion of soldier beam below the level of the first tieback. Subsequent readings indicated a slight soldier beam shift to the west. Further, it is apparent that the portion of soldier beam above the first level tieback has behaved as a cantilever beam. Completion of the reinforced concrete facing wall for the retaining system stabilized east-west deflection, as would be expected.

Comparison of the digitilt inclinometer data developed at soldier beam No. 24 and 93 within the primary orientation revealed some differences in the deflected shapes of the soldier beams. The later readings at soldier beam No. 24 indicated an S-shaped deflection distribution. Maximum deflection toward the excavation occurred between the second level tieback and the bottom of footing. Magnitude of deflection was on the order of 0.1". The upper section of the soldier beam behaved similarly to a cantilever beam bending about the first level tieback. The data developed at soldier beam No. 93 indicated a parabolic shaped deflection distribution. At this location, maximum deflection toward the excavation occurred within the vicinity of the second level tieback having a magnitude of 0.2". At this location, there was no reverse curvature in the deflected shape of the soldier beam at the first tieback level.

The digitilt inclinometer readings to date have revealed relatively minor soldier beam deflection. The magnitude of soldier beam deflection is consistent with or slightly less than what would generally be expected for large diameter, straight shaft permanent soil anchors in cohesive soils.

C. TIEBACK LOAD DISTRIBUTION

Instrumentation of the tiebacks included a load cell installed at the anchor head for both tieback levels at soldier beam Nos. 24 and 93. Each of these tiebacks also included vibrating wire strain gage instrumentation at five locations along the anchor length. The strain gage locations are shown on Figure 4. The strain gage data was converted to stress utilizing the theory of elasticity. The resulting stress was used to determine the tieback loads at the specific strain gage locations. The load cell data was used to determine tieback loads at the anchor head utilizing calibration charts and equations provided by the load cell manufacturer.

The load cell and tieback forces are shown graphically with respect to time since initial tieback lock-off on Graphs 1 thru 4.

The design and lock-off load for these tiebacks was 80 kips. The data indicates that the tiebacks have been locked-off at a force less than 80 kips. Initial tieback loads after lock-off varied between 53 and 70 kips. The low lock-off loads are more than likely attributable to the lock-off load operation using the stressing jack, limited accuracy of the pressure gage on the stressing jack, seating of the anchorage at the bearing plate and other construction related factors. Data obtained immediately following lock-off of the second level tieback anchor at soldier beam No. 93 showed that the design load was not achieved at lock-off. The initial reading was 70 kips as compared to the 80 kip design load.

Data from three of the four load cells indicated tieback load loss of 1% or less between the initial readings taken late July or early August, 1987 through the early part of December 1987. These percentages are based on the difference in load over the monitoring period to date as compared to initial anchor loads.

The load cell at soldier beam No. 24, first level tieback, indicated an approximate 6% loss of tieback anchor load within the monitoring period to date. A substantial loss of load was recorded between the readings taken August 14 and 21, 1987, approximately 7 kips. A good portion of this load loss was recovered based on readings taken the following week. Beyond this point the load cell indicated stabilization of tieback load. Less than 2% load loss occurred within the final 3-1/2 months of the monitoring period through December 1987. It would appear that the 7 kip load loss in August represented shift of anchor bond capacity further back on the anchor length.

In addition to the load cells, the strain gages mounted on the tieback bars were utilized to evaluate load transfer with time along the length of the anchor. Figure 4 illustrates the locations of the strain gages along the anchor length. The strain gages located at the approximate mid-point of the unbonded length should theoretically yield loads which agree with the tieback loads indicated by the load cells at the anchor head. At the beginning of the monitoring period, the strain gage readings on the tiebacks revealed some difference in tieback load as compared to the load cell data. There was a gradual increase in anchor load at the No. 1 strain gages over the length of the monitoring period. The December 1987 readings disclosed an approximate 1 kip or less load difference between the data developed from the load cells and No. 1 strain gages. As noted at soldier beam No. 24, first level tieback, the No. 1 strain gage was not functioning.

It is probable that there was some initial friction between the tieback anchor bars and the smooth sheaths in the unbonded anchor length. With time, the unbonded zone was fully mobilized. Tie-back loads at the No. 1 strain gage locations increased 3.4 to 10.9 kips over the first 4-1/2 months of monitoring.

Strain gage Nos. 2 thru 5 on each of the instrumented tiebacks are located within the bond length of these anchors. It was intended that the strain gage data provide tieback anchor load distribution along the anchor length as well as anchor load transfer with time. The gages did show distribution of anchor load within the bonded anchor length of the tieback. A significant portion of the anchor load, however, was developed within the anchor zone preceding the bond length. Undoubtedly, the anchor grout immediately preceding the bond zone is in compression and provides significant load capacity as a result of the anchor grout/soil bond. To illustrate this point, the No. 2 gages revealed initial anchor loads of approximately 21 to 36% of the tieback load indicated by the corresponding load cells. The December 1987 readings at the No. 2 gage locations demonstrated a similar load range, about 31 to 33% of the tieback loads indicated by the load cells. As noted, the No. 2 gages are located only 2 ft. beyond the beginning of the bonded anchor length. Therefore, more than 67% of the anchor load capacity is developed within the length of anchor preceding the No. 2 gage locations.

It is interesting to note that No. 2 strain gage on the first level tieback of soldier beam No. 93 decreased 4.9 kips upon tensioning of the second level tieback. This response illustrates the inter-dependence of the retaining system components in an effort to reach stress equilibrium.

The data developed at strain gage locations 3, 4, and 5 were also evaluated relative to percentage of tieback load indicated by the load cells. At the No. 3 gage locations, initial readings suggested 3.8 to 5.2% of anchor load distribution. The December 1987 gage readings increased to approximately 15% of anchor load distribution. At the No. 4 gage locations, initial readings indicated 1.2 to 6.7% of anchor load distribution as compared to the range of 7.7 to 13.6% of anchor load distribution for the December 1987 readings. The No. 5 strain gage locations indicated initial tieback load percentages of 0.5 to 2.4%. The December 1987 readings indicated 5.4 to 5.7% of anchor load distribution.

It should be noted that there were a number of non-functioning strain gages on the instrumented tiebacks. Further gage Nos. 2,3, and 5, at the second anchor level of soldier beam No. 24, yielded peculiar data. In fact, gages Nos. 3 and 5 suggested compression loads. Anchor tensioning may have introduced local bending strains within the bar tieback which misrepresents actual tieback loads. At low axial strains, gages mounted on one side of the tiebacks may reflect compression bending strain. If the data is reviewed in terms of increase in anchor load, the strain gage data at these locations have some value.

The instrumentation data shows tieback load has remained relatively stable at the anchor head. However, the tiebacks have demonstrated some increase in anchor load distribution at the various locations along the unbonded and bonded anchor length. Generally, the first 3 to 5 weeks after tieback lock-off has demonstrated most of the load increase at the strain gage locations. The increase in stress along the bonded anchor length is probably a function of many factors including excavation adjacent to the wall, tieback tensioning at adjacent and lower level anchors, creep between the grout anchor zone and the surrounding soil and the time dependent stress relaxation of the prestressed steel tiebacks. Literature suggests a 3 to 4% tieback load loss at the anchor head can be expected i the 4-1/2 month monitoring period due entirely to the stress relaxation of the prestressed steel.

D. SOLDIER BEAM NO. 93 AXIAL LOADS

The strain gage instrumentation placed on the double channel beams (soldier beam No. 93) provided data to determine axial and bending stress in these members at specific locations along the soldier beam length. Axial loads were determined for both the steel channels as well as the composite section of steel beams and concrete of the surrounding drilled pier. Sections 3 and 4 included the full drilled pier concrete area, whereas Section 2 analysis accounted for the smaller concrete area as a result of concrete removal to install the reinforced concrete facing wall and wood lagging. Reference is made to Graphs 5 thru 11; all present the results of strain gage instrumentation on soldier beam No. 93.

Readings taken early in the monitoring program indicated that gage 3.2 was yielding erroneous data. It is probable that the excavation equipment or the concrete removal process at the face of the channel flanges locally deformed the channel flange which resulted in distortion of the data. The data has been presented in two ways. The first considers the actual data from gage 3.2. The other procedure provides a theoretical correction of gage 3.2. The theoretical correction of gage 3.2 was determined by developing a linear strain distribution north to south through the east channel beam. Gages 3.4 and 3.6 were utilized for this purpose.

As noted previously, the two strain gages on the south flanges of the channel beams at Section 1 of soldier beam No. 93 were damaged during concrete removal for lagging installation. Therefore, the strain gage data developed at Section 1 consisted of only the north flanges of the channel beams. These gages indicated there were low compression strains in these flanges during the early monitoring period. After the middle of August 1987, there was a transition to tensile strain. Obviously, the data manifests bending of the upper portion of the soldier beam as a result of progressive mobilization of earth pressure acting on the wall. The developed stress within the channel beams is on the order of 1% of the allowable steel stress.

Strain gage instrumentation at Section 2 indicated an average stress within the steel channel beams between 3 and 4% of allowable within the later part of the monitoring period. Converting strains to axial load for both the steel channel beams and the composite section revealed low initial axial loads with dramatic increase of axial loads within the first month of monitoring. A 6.2 kip increase in axial load occurred within the composite section as a result of tensioning the second level tieback on July 28, 1987. Another large increase in axial load occurred during the following week as the excavation was extended to bottom of wall footing elevation. The axial load on the composite section showed a 10.5 kip increase due to the reduction in drilled pier shaft friction related to the excavation. Monitoring readings taken August 14, 1987, revealed the greatest jump in soldier beam axial load, approximately 27.5 kips. It is probable that this jump occurred due to concrete removal at the south flanges of the steel channel beams relative to lagging installation. A hoe ram and pneumatic hammer were used to remove the concrete. This operation produced considerable vibration of the soldier beam and permitted axial load transfer to greater depths on the soldier beam.

There was some continued increase in soldier beam load at Section 2 until the middle of September and then a gradual decrease in axial load through the end of the monitoring period to date. Maximum axial loads in the composite section reached 55.4 kips. The vertical component of axial load due to the tieback tensioning at the first and second level amounts to 46 kips. The weight of the pier above Section 2 is approximately 13 kips. A total vertical load of 59 kips is theoretically developed. This does not include the weight of the reinforced concrete facing wall which at least partially is carried by the soldier beam. Therefore, most of the load produced by the various wall components has been carried to the level of the Section 2 gages. The data also suggests there is an instantaneous as well as time dependent response to axial loads produced by tieback anchor tensioning, dead load of the wall system components, excavation adjacent to the wall, and general construction procedures.

Discussion related to the Section 3 strain gages will be limited to the data based on the rectification of gage 3.2. The calculated stress within the steel channel beams at this section was on the order of 1/2 to 1% of allowable stress and on the same order of magnitude as Sections 1 and 2. The Section 3 data did not indicate the instantaneous response to various construction operations as demonstrated at Section 2. There was a time dependent transfer of soldier beam axial load to the Section 3 location. However, axial load transfer was considerably more gradual than encountered at Section 2. Further, the composite section axial loads were on the order of 30% of the loads developed at Section 2 during the later stages of instrumentation monitoring. In fact, there was only an approximate 10 kip increase in axial load on the composite section between the initial readings and the December 1987 readings.

The maximum axial load was approximately 17 kips in December as compared to a computed 63 kips of axial load. The 63 kips includes the vertical component of the tieback loads and weight of the soldier beam pier but excludes any partial load developed due to the facing wall. Therefore, the majority of the soldier beam axial load was supported through drilled pier shaft friction within the 5 ft. length of soldier beam between the Section 2 and

Section 3 gages. It should be noted that the finished pavement grade is approximately elevation 854 adjacent to the wall or approximately 1 ft. below the Section 2 gages. The reinforced concrete facing wall footing probably supports some of the axial load transmitted to the soldier beam considering the integral fabrication of the reinforced concrete facing wall to the soldier beam units.

Section 4 strain gage instrumentation included placement of strain gages on the centerline of the webs on each channel beam. The developed strains were extremely low. The data indicates that gage 4.6 experienced a much greater strain rate change than gage 4.5. This difference may be due to concentration of local bending strains at the location of Section 4. Averaging the strain data at Section 4 yields composite section axial loads up to a maximum of 3 kips. The computed axial load at Section 4 is 70 kips and is the summation of the vertical component of the tiebacks and the weight of the soldier beam concrete and steel. It is interesting to note that essentially all of the strain at Section 4 has been carried by the east channel beam, especially as indicated by the data recorded in November and December 1987.

Most of the axial load on the soldier beam has been transmitted to Section 2, just above final grade elevation adjacent to the wall. The net axial load quickly diminished with increased depth of soldier beam embedment below finished grade. The stress developed in the steel channel beams is less than 4% of allowable based on the strain gage readings. Additionally, if the maximum vertical load as recorded at Section 2 (55.4 kips) was carried entirely by the channel beams, neglecting the surrounding concrete, stress levels would only be approximately 12% of allowable.

E. SOLDIER BEAM NO. 93 BENDING MOMENTS

The strain gage instrumentation at soldier beam No. 93, Sections 2 and 3, was used to determine bending moments in the soldier beam. At Section 2 the bending moments were determined for the steel channel beams only. The complications in determining composite area bending moments at this section were prohibitive considering the fact that a portion of the concrete had been removed for lagging installation and accommodation of the facing wall. Analysis of Section 3 data permitted determination of bending moment in the steel channel beams as well as the composite section considering an uncracked concrete section. Reference is made to Graphs 12 thru 16.

Similar to the determination of axial soldier beam loads, gage 3.2 was theoretically adjusted as described in the previous section of this report. Discussion of bending moments at Section 3 has been limited to the data which considers the adjustment made to gage 3.2.

At both Sections 2 and 3, the bending moments were higher in the Y axis (direction parallel to the wall) than bending moments in the X axis (direction perpendicular to the wall). Another peculiar results indicated by the data is that the individual channel beam bending moments in the X axis were in opposite directions. In other words, the west channel beam indicated compression strains in the south flange while the east channel beam indicated tensile strains in the south channel flange. It is apparent that soldier beam torsion has considerably affected the strain gage readings. Another possible factor contributing to the peculiar strain gage data is that the bond between the steel channel beams and the surrounding concrete is questionable due to the ram hoe and pneumatic hammer operation during concrete removal along the south flanges of soldier beam No. 93.

The data suggests that bending in the Y axis became stabilized after the reinforced concrete facing wall was completed following the August 21, 1987, readings.

The developed strain gage data indicates that bending moments within the soldier beam were extremely low relative to the structural capability of the steel channel beams and composite soldier beam section.

F. TEMPERATURE READINGS AT SELECTED GAGE LOCATIONS

Thermistors on selected gages were used to monitor temperature changes. Temperature readings were obtained at two locations on the steel channel beams at soldier beam No. 93, gage 1.3 and 3.1, as well as the first strain gage on each of the instrumented tiebacks. Reference is made to Graphs 17 and 18 which depict the temperature readings over the course of the monitoring period to date.

As expected, the thermistors on the steel channel beams showed a direct response to ambient temperatures, once excavation progressed below the location of the gages. There is only minimum thermal separation between the ambient temperature and the location of the thermistor, the concrete surrounding the steel channel beams and the facing wall concrete.

Temperature response was most clearly demonstrated by gage 3.1 within the first 7-weeks of monitoring. This gage is located on the south flange at Section 3 of soldier beam No. 93. This location corresponds to the level at the bottom of wall footing elevation. The first two temperature readings at this location (67° and 60°F) reflected the cooling influence of the surrounding soil prior to excavation below the second level tieback location. Upon excavation to the bottom of wall footing elevation, temperature readings at the gage (3-weeks consecutively) were 77°F. This represents a 17°F jump due to more direct gage exposure during excavation. This illustrates the effect of soldier beam exposure to ambient temperatures. Subsequent wall footing construction and backfilling produced a 7° drop in temperature, again illustrating the thermal influence of the surrounding soil.

The thermistors located at the approximate mid-point of the unbonded length of the tieback anchors showed relatively stable temperature readings. As would be expected, temperature fluctuations were much narrower then readings at the gages located on the soldier beam. Further, temperature readings at the tieback anchor locations, 58° to 68°F, yielded results expected for soil thermal conditions at considerable depth below surface grades.

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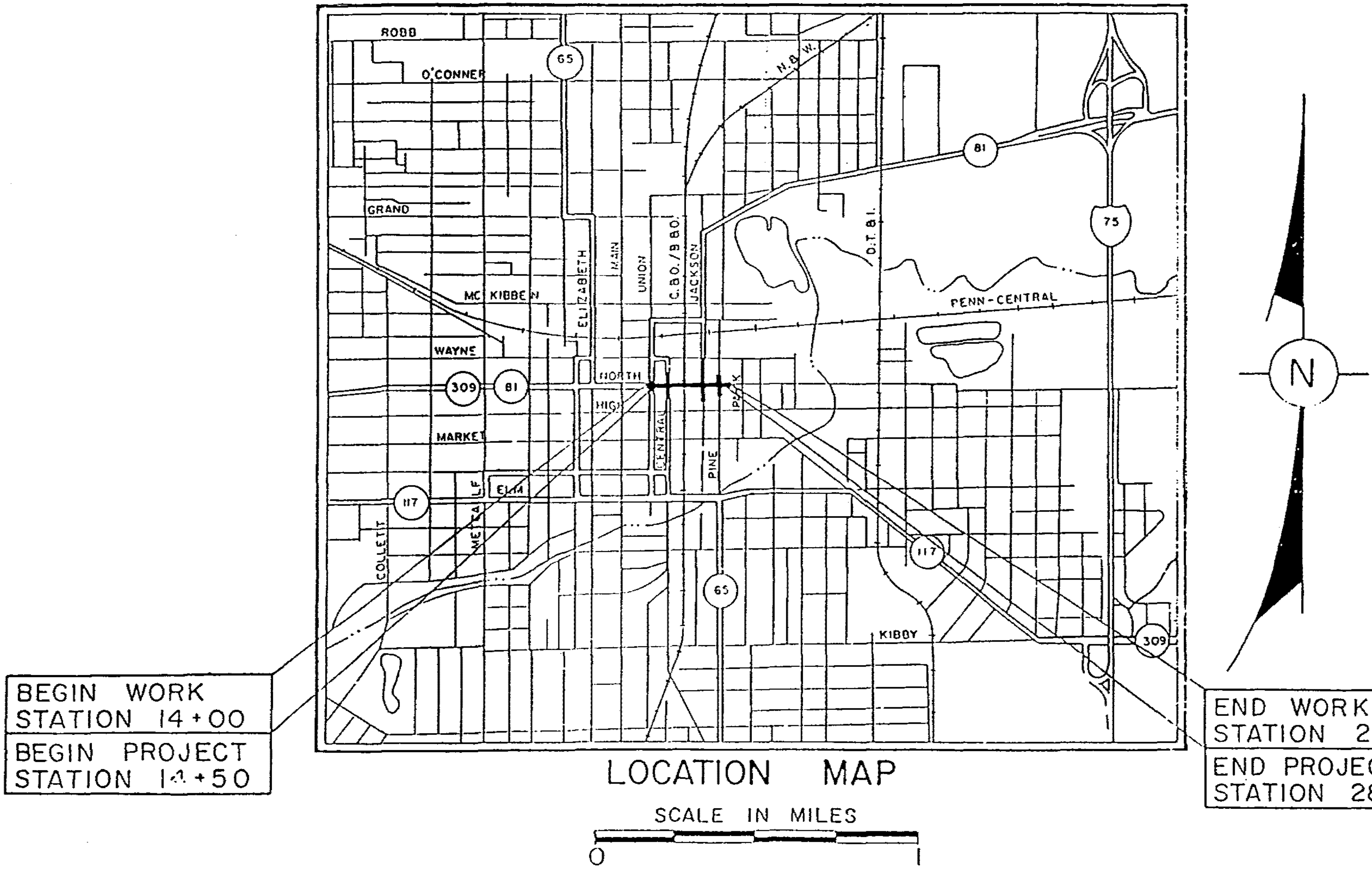
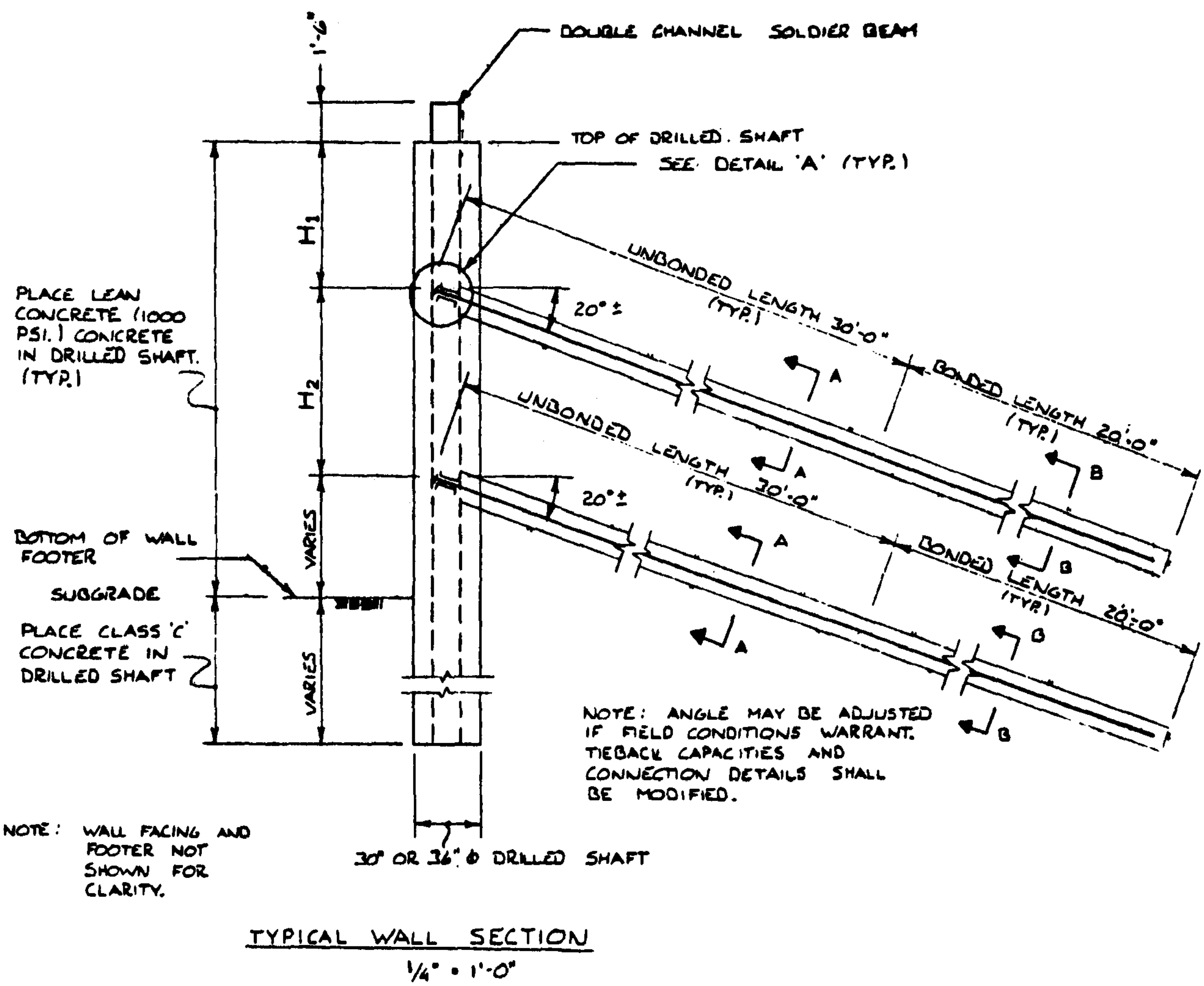


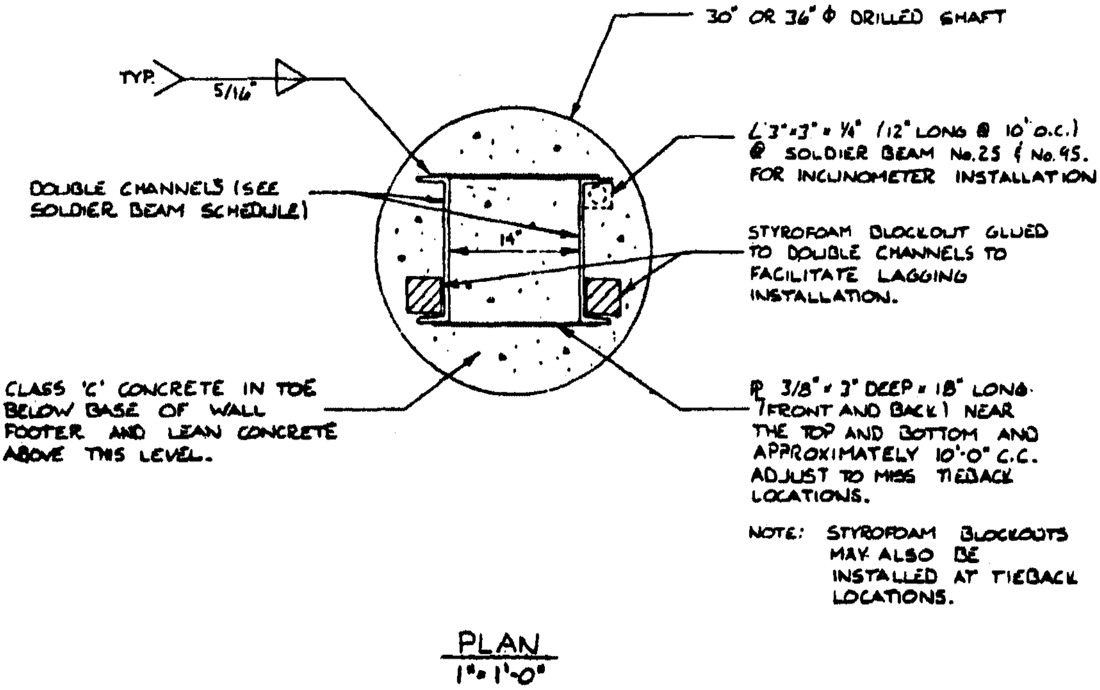
LOCATION MAP

6-A-1

APPENDIX A



Tieback Wall Details.

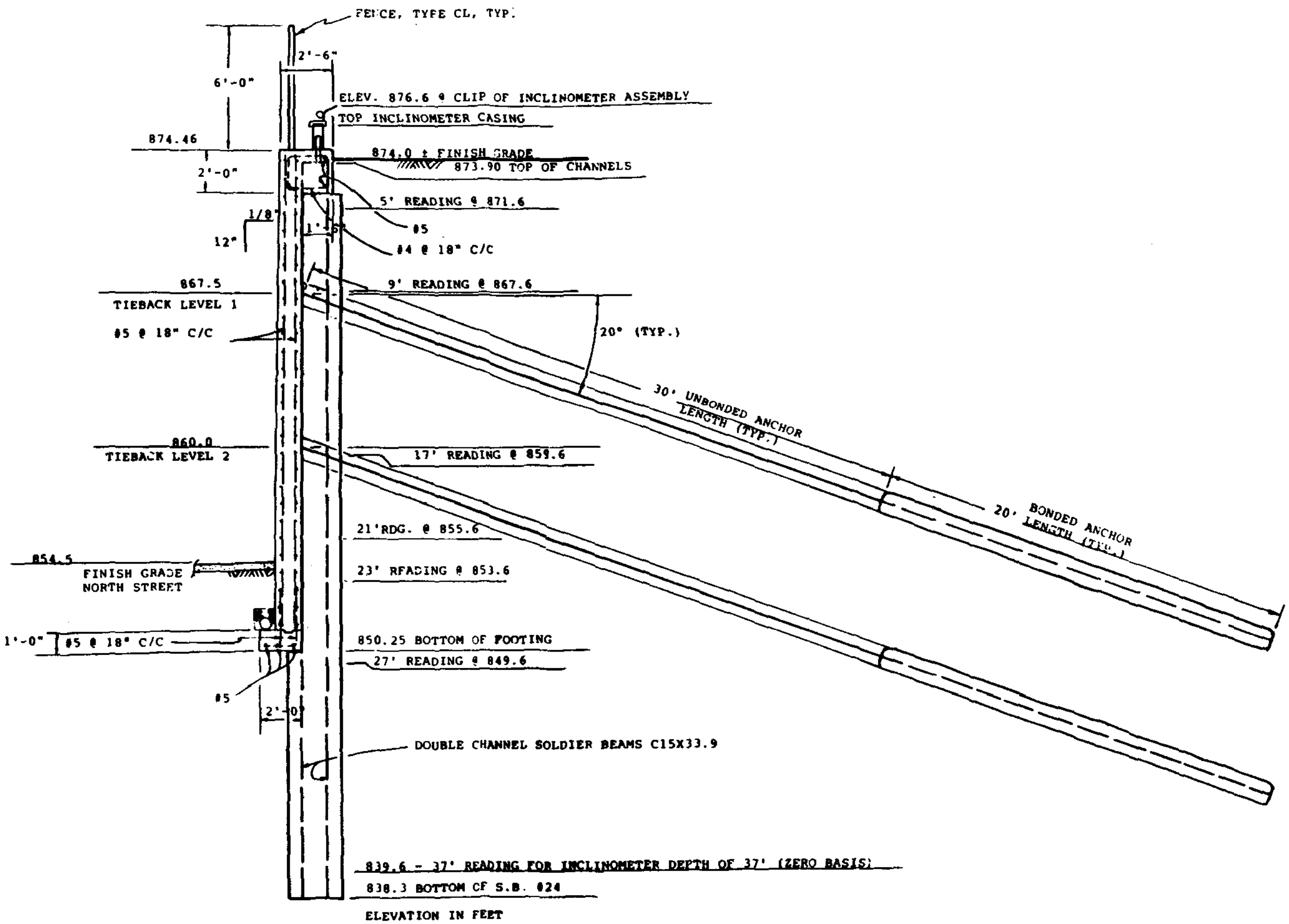


Tieback Wall Details.

SOLDIER BEAM & TIEBACK SCHEDULE

SOLDIER BEAM NO.	DOUBLE CHANNEL SIZE	TOP ELEVATION DRILLED SHAFT	BOTTOM ELEVATION LEAN CONCRETE BACKFILL	BOTTOM ELEVATION DRILLED SHAFT	LENGTH OF DOUBLE CHANNELS	H 1 (ft.)	H 2 (ft.)
1	C 15 x 33.9	868.2	851.0	839.0	30.7	3.7	---
2		868.4			30.9	3.9	---
3		868.6			31.1	4.1	---
4		868.8			31.3	4.3	---
5		869.0	850.5	838.5	32.0	4.5	---
6		869.3			32.3	4.8	---
7		869.5			32.5	5.0	---
8		869.7			32.7	5.2	---
9	C 15 x 40	869.9	850.00	837.0	34.4	4.9	---
10		870.1			34.6	5.1	---
11		870.3			34.8	5.3	---
12		870.5			35.0	5.5	---
13		870.7			35.2	5.7	---
14		870.8			35.3	5.8	---
15		871.0			35.5	6.0	---
16	C 15 x 33.9	872.0	849.8	837.8	35.7	4.5	7.5
17		872.1			35.8	4.6	
18		872.1			35.8	4.6	
19		872.2			35.9	4.7	
20		872.2	850.0	838.0	35.7	4.7	
21		872.3			35.8	4.8	
22		872.3			35.8	4.8	
23		872.4			35.9	4.9	
24		872.4	850.3	838.3	35.6	4.9	
25		872.5			35.7	5.0	
26		872.6			35.8	5.1	
27		872.6			35.8	5.1	
28		872.7	850.8	838.8	35.4	4.7	
29		872.7			35.4	4.7	
30		872.8			35.5	4.8	
31		872.8			35.5	4.8	
32	C 15 x 33.9	872.9	851.3	839.3	35.1	4.9	
33		872.9			35.1	4.9	
34		873.0			35.2	5.0	
35		873.0			35.2	5.0	
36		873.1	852.0	840.0	34.6	4.6	
37		873.1			34.6	4.6	
38		873.2			34.7	4.7	
39		873.3			34.8	4.8	
40		873.3	852.8	840.8	34.0	4.8	
41		873.4			34.1	4.9	
42		873.4			34.1	4.9	
43		873.5			34.2	5.0	
44		873.5	853.8	841.8	33.2	4.5	
45		873.6			33.3	4.6	
46		873.6			33.3	4.6	
47		873.7			33.4	4.7	
48		873.8	854.8	842.8	32.5	4.8	
49		873.8			32.5	4.8	
50		873.9			32.6	4.9	
51		873.9			32.6	4.9	
52		874.0			32.7	5.0	
53	C 12 x 25	866.2	856.5	848.5	19.2	3.9	---
54		866.5			19.5	4.2	---
55		866.8			19.8	4.5	---
56		867.2			20.2	4.9	---
57		867.3	855.3	846.3	22.5	5.0	---
58		867.5			22.7	5.2	---
59		867.7			22.9	5.4	---
60		867.9			23.1	5.6	---
61	C 15 x 33.9	868.0	854.0	843.0	26.5	4.3	---
62		868.2			26.7	4.5	---
63		868.3			26.8	4.6	---
64		868.5			27.0	4.8	---
65		868.7	853.0	842.0	28.2	5.0	---
66		868.9			28.4	5.2	---
67		869.1			28.6	5.4	---
68		869.3			28.8	5.6	---
69		869.5			29.0	5.8	---
70		869.6	852.0	841.0	30.1	5.1	---
71		869.7			30.2	5.2	---
72		869.7			30.2	5.2	---
73		869.8			30.3	5.3	---
74		869.9	851.3	839.3	32.1	5.4	---
75		870.0			32.2	5.5	---
76		870.1			32.3	5.6	---
77		870.2			32.4	5.7	---
78		870.2	850.5	838.5	33.2	5.7	---
79		870.3			33.3	5.8	---
80		870.4			33.4	5.9	---
81		870.5			33.4	6.0	---
82	C 15 x 40	870.6	850.0	837.0	35.1	5.6	---
83		870.6			35.1	5.6	---
84		870.7			35.2	5.7	---
85		870.8			35.3	5.8	---
86		870.8	849.8	835.8	36.5	5.8	---
87		870.9			36.6	5.9	---
88		870.9			36.6	5.9	---
89	C 15 x 33.9	872.6	849.5	834.5	39.6	4.6	8.0
90		872.6			39.6	4.6	
91		872.7	850.0	835.0	39.2	4.7	
92		872.8			39.3	4.8	
93		872.9			39.4	4.9	
94		873.0	850.3	835.3	39.2	5.0	
95		873.0			39.2	5.0	
96		873.1	850.8	835.8	38.8	5.1	
97		873.2			38.9	5.2	
98		873.3			39.0	5.3	
99		873.4	851.3	836.3	38.6	5.4	
100		873.5			38.7	5.5	
101		873.6			38.8	5.6	
102		873.7	852.0	837.0	38.2	5.7	
103		873.8			38.3	5.8	
104		873.9			38.4	5.9	
105		873.9	853.0	838.0	37.4	4.9	
106		874.0			37.5	5.0	
107		874.1			37.6	5.1	
108		874.2	854.3	839.3	36.4	5.2	
109		874.3			36.5	5.3	
110		874.4			36.6	5.4	
111		874.4	855.5	840.5	35.4	5.4	
112		874.5			35.5	5.5	
113		874.6			35.6	5.6	

Tieback Wall Details.



CROSS SECTION

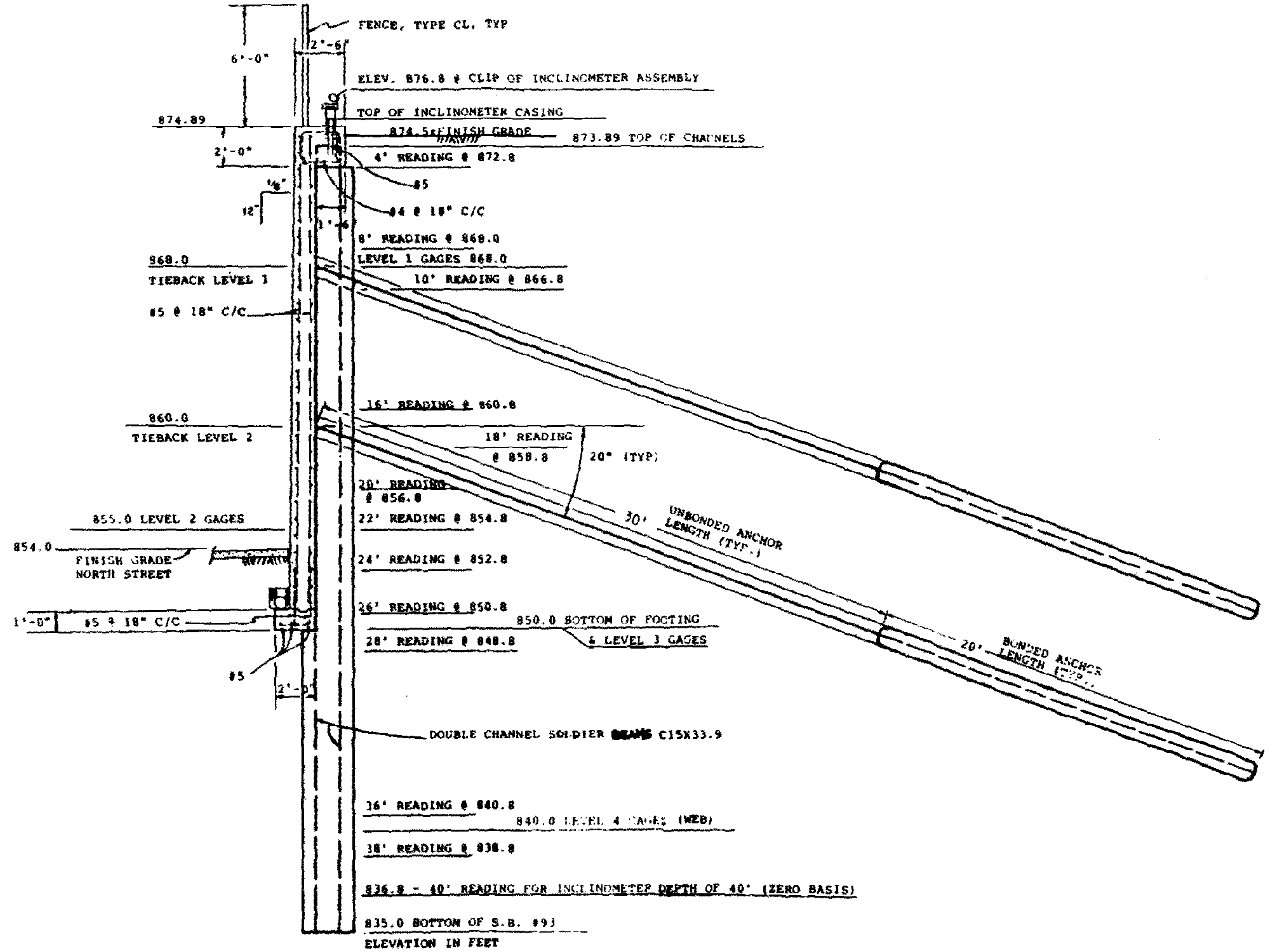
SOLDIER BEAM #24 (WALL UNIT #22)

THE H. C. NUTTING COMPANY
GEOTECHNICAL AND TESTING ENGINEERS SINCE 1921

INSTRUMENTED TIEBACKS AND SOLDIER BEAMS CROSS SECTION

RETAINING WALL UNIT 22, S.B. #24
NORTH STREET GRADE SEPARATION
CITY OF LIMA, OHIO
ALLEN COUNTY

SCALE: 1/4"=1'-0"	DATE: JAN., 1988	FIGURE NO. 1



CROSS SECTION

SOLDIER BEAM #93 (WALL UNIT #71)

THE H. C. NUTTING COMPANY

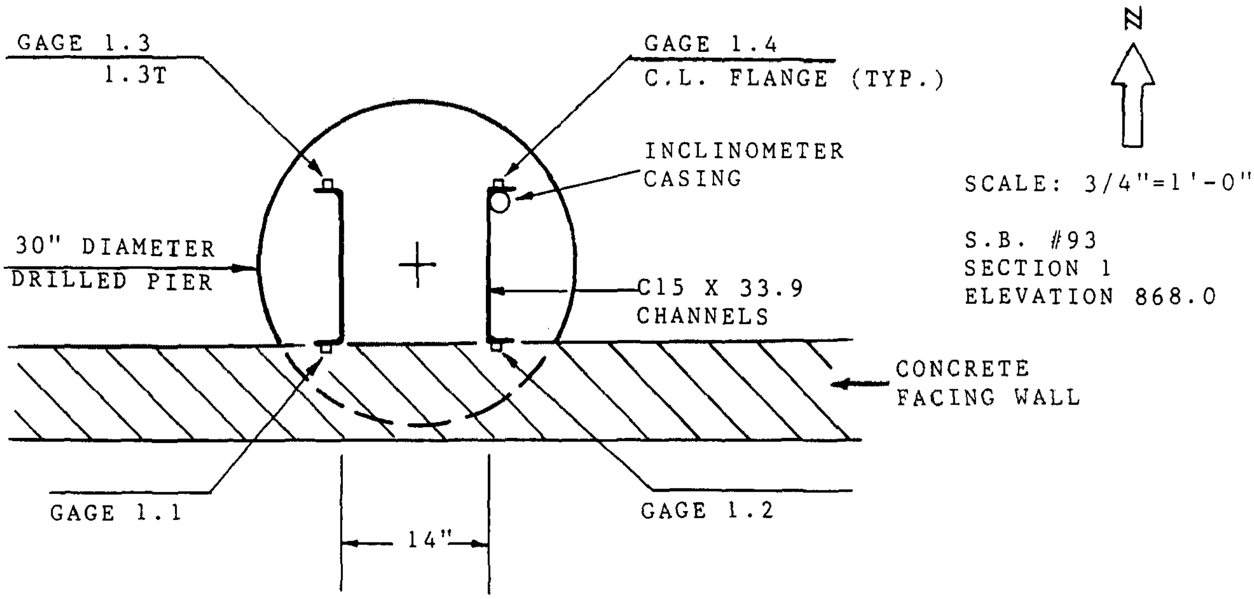
GEOTECHNICAL AND TESTING ENGINEERS — SINCE 1921

4120 AIRPORT ROAD • CINCINNATI OHIO 45226 • TEL 513-321-5816
912 MORRIS STREET • CHARLESTON W VA 25301 • TEL 304-344-0821
BOX NUMBER 11 • HIGHLAND HEIGHTS KY 41076 • TEL 606-781-3042

INSTRUMENTED TIEBACKS AND SOLDIER BEAMS CROSS SECTION

RETAINING WALL UNIT 71, S.B. #93
NORTH STREET GRADE SEPARATION
CITY OF LIMA, OHIO
ALLEN COUNTY

SCALE: 1/4"=1'-0"	DATE: JAN., 1988	FIGURE NO. 2



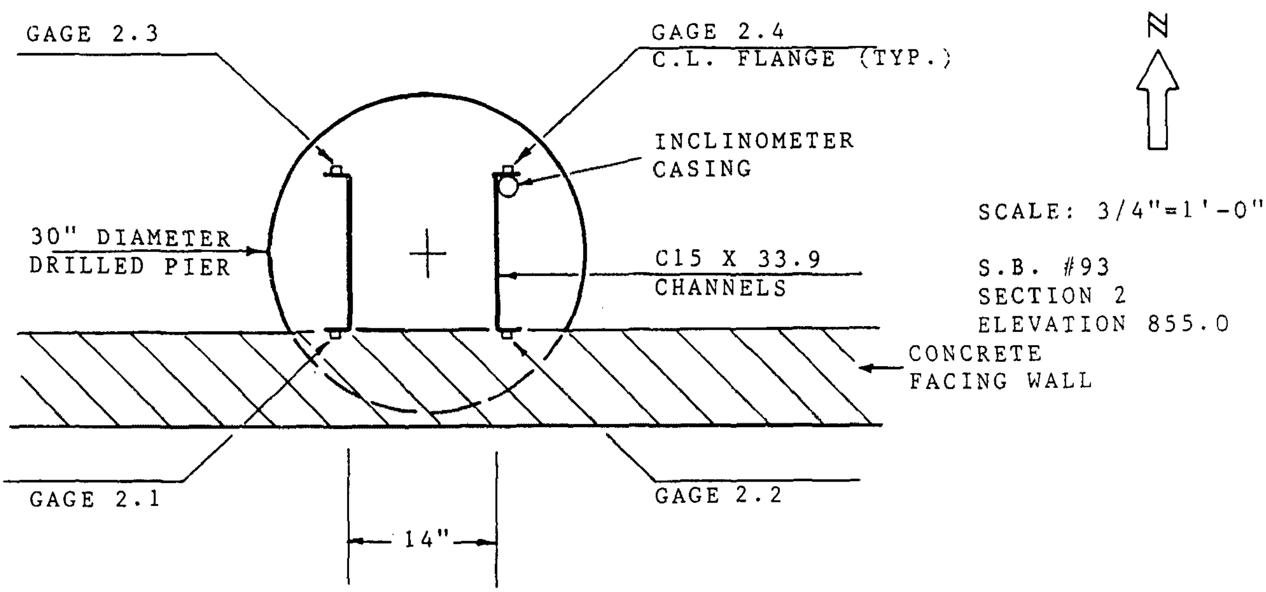
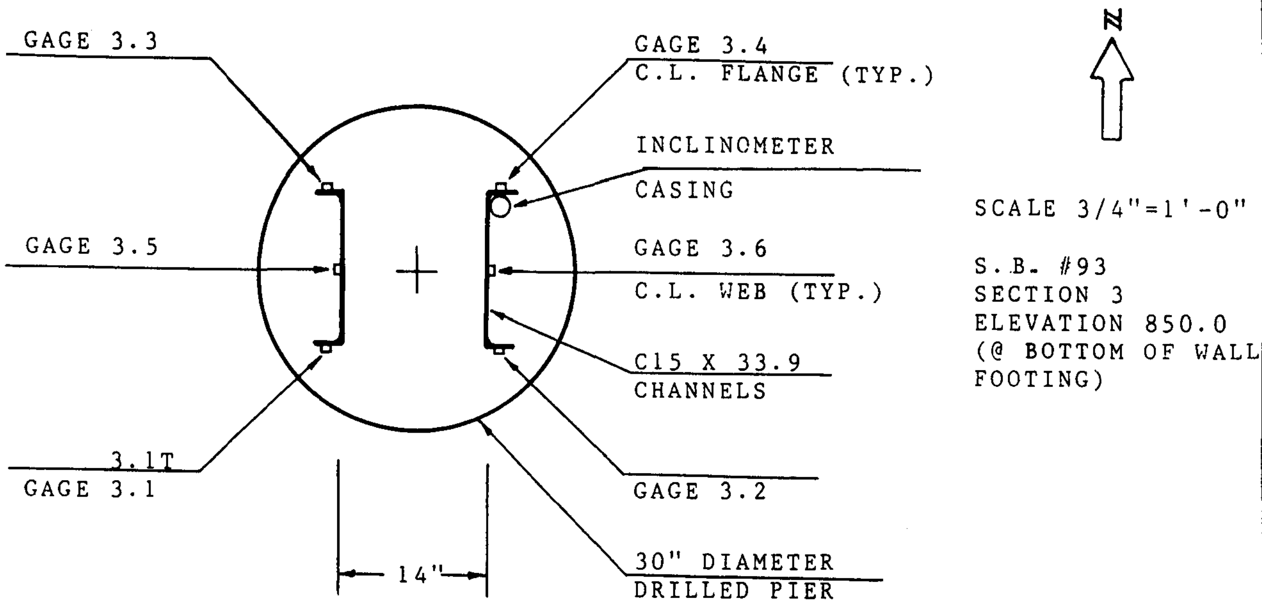


FIGURE 3A. STRAIN GAGE LOCATIONS
SOLDIER BEAM #93



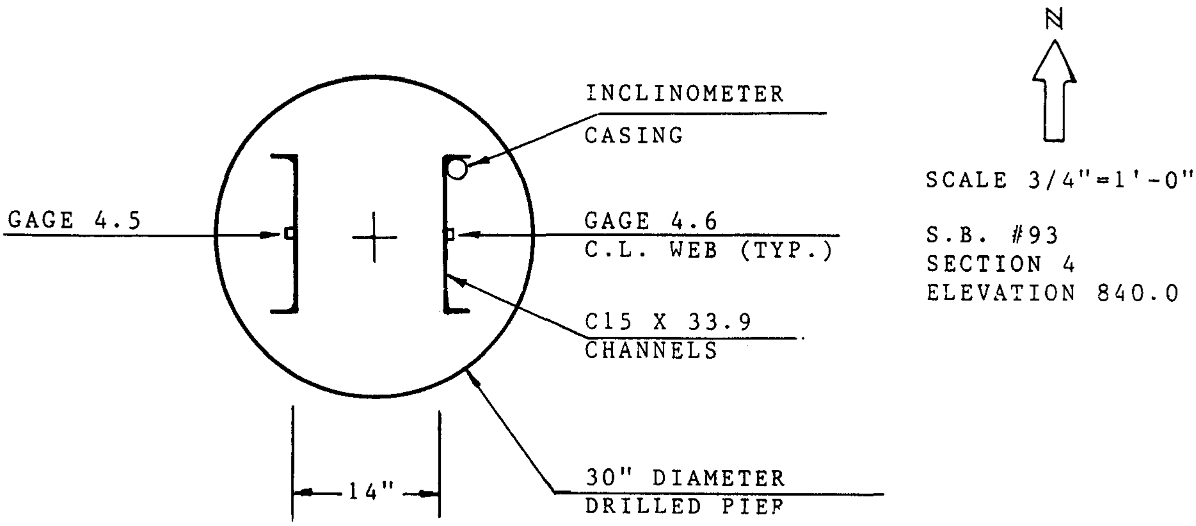


FIGURE 3B. STRAIN GAGE LOCATIONS
SOLDIER BEAM #93

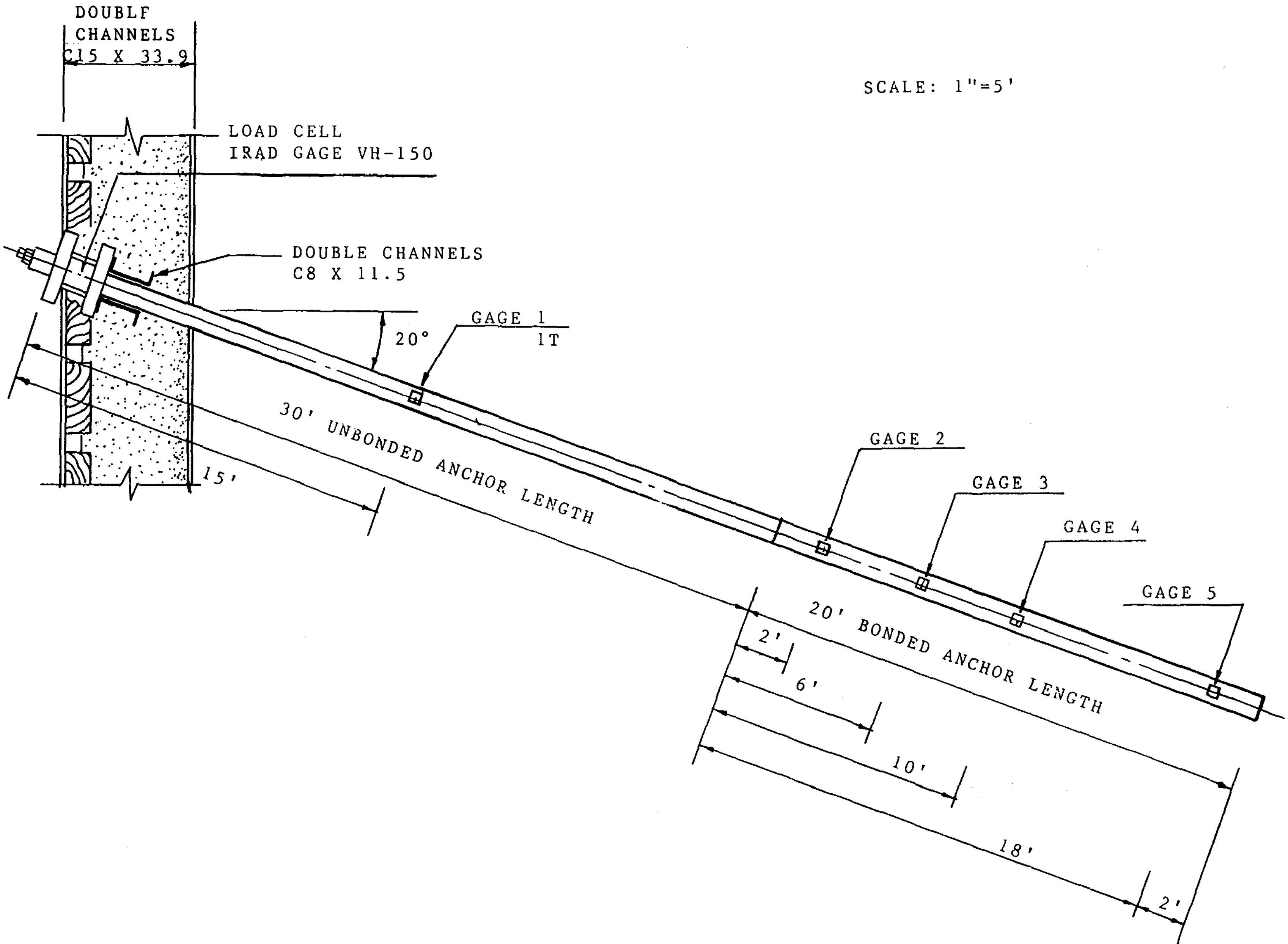


FIGURE 4. STRAIN GAGE & LOAD CELL LOCATIONS
TYPICAL FOR INSTRUMENTED TIEBACKS

THE H. C. NUTTING COMPANY 4120 AIRPORT ROAD CINCINNATI, OHIO 45226	OHIO DEPARTMENT OF TRANSPORTATION DEMONSTRATION PROJECT NO. 68, ALL-S.R. 81-16.83, NORTH STREET GRADE SEPARATION, LIMA, OHIO

DATE	SOLDIER BEAM LOCATION	
7-16-87	@ 24 & 93	- Excavation level @ 1' below 1st tieback elevation (elev. - 867 @ 93, 866.5 @ 24) - Tiebacks not tensioned - Readings (background on SB 93 (beam gages) & 1st tieback @ 93 & 24, load cells @ 1st tieback level & inclinometers
7-28-87	@ 24 & 93	- Excavation level @ 2' below 2nd tieback elev. (Elev. 858 @ 24 & 93) - Tiebacks at 1st level tensioned Tieback elevation @ 24 - 867.5 @ 93 - 868.0 - Readings (background) taken on 2nd level Tiebacks & load cells @ 24 & 93 prior to tensioning - Readings on remainder of gages also taken - Top of steel soldier beams 873.89 @ 93 (Provided by ODOT) 873.90 @ 24 - 2.9' top of beam to top of clip @ 93, 2.7' @ 24, elev. 876.8 @ clip (93) elev. 876.6 @ clip (24) - A second reading was taken 7-28-87 on all gages and load cells and inclinometer @ 93 after second tieback level tensioned - 2nd level tieback elev. 860 (24 & 93) - 2nd level tieback @ 24 not tensioned this date
8-05-87	@ 24 & 93	- Excavation level @ 10'± below 2nd tieback anchor levels 24 & 93 (Elev. 850.25) approximately same elev. as bottom of wall ftg. - All tiebacks tensioned - 2nd level tiebacks @ 24 tensioned subsequent to previous readings. - Readings taken all gages & load cells & inclinometers
8-14-87	@ 24 & 93	Same conditions as 8-05-87
8-21-87	@ 24 & 93	Wall footing in place - bottom of footing 850.25 @ 24, 850.0 @ 93 - Facing wall @ 24 in place
8-28-87	@ 24 & 93	- Facing wall in at 93. - Subgrade for roadway complete adjacent to 24 & 93, @ 24 - 854 elev. ±, @ 93 - 853.5 elev. ±
9-4-87	@ 24 & 93	- Wall beam cap in place, top/cap 874.89 @ 93 874.46 @ 24

Figure 5. Log of field conditions on dates of instrumentation readings

THE H. C. NUTTING COMPANY
4120 AIRPORT ROAD
CINCINNATI, OHIO 45226

OHIO DEPARTMENT OF TRANSPORATION
DEMONSTRATION PROJECT NO. 68,
ALL-S.R. 81-16.83, NORTH STREET
GRADE SEPARATION,
LIMA, OHIO

DATE	SOLDIER BEAM LOCATION	
9-18-87	@ 24 & 93	- Road surfacing essentially complete Finish Grade @ 24 - 854.5 elev. @ 93 - 854.0 elev.
9-30-87	@ 24 & 93	- Storm sewer excavation made immediately behind wall @ 93, 2'± behind soldier beam pier, 4'± deep, bottom of trench @ elev. 870.5± - Grade at back of wall @ 93 - 874.5±, @ 24 - 874.0±
10-16-87	@ 24 & 93	- Construction essentially completed - This & subsequent readings - routine
11-02-87		- Routine Reading - 2 week interval
12-08-87		- Routine Reading - begins 1 month reading intervals
1-20-88		- Routine Reading
2-22-88		- Routine Reading - begin three month reading intervals
5-27-88		- Routine Reading
8-27-88		- Routine Reading

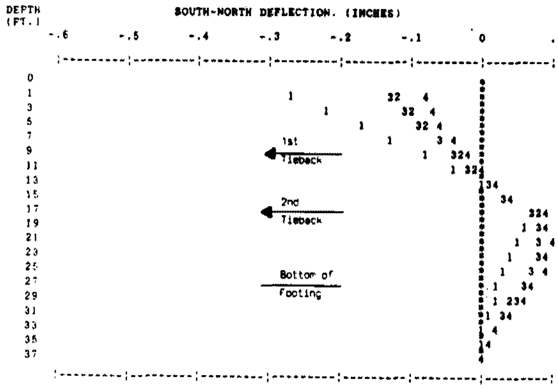
Figure 5. (Continued)

THE H. C. NUTTING CO.
DIGITILT INCLINOMETER SUMMARY SHEET

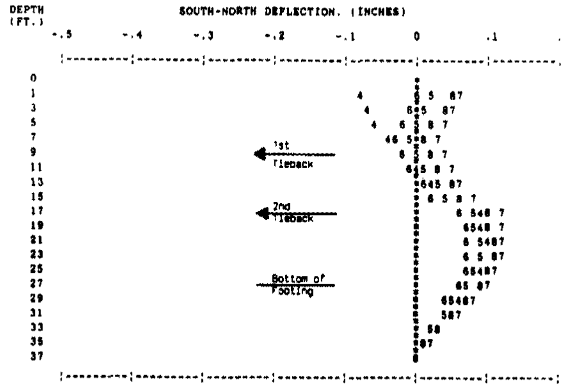
NORTH STREET GRADE SEPARATION
PROJECT NO. 8069.005

INCLINOMETER LOCATION NO. 24 HORIZONTAL SCALE: 1 INCH = .1 INCH
TOP OF CASING ELEV. 876.6 VERTICAL SCALE: 1 INCH = 10 FT.

DATE OF REFERENCE READING: 7-16-87
LATEST READING: READING NO. 11 DATE: 12-08-87 FILE: LIMA25.11

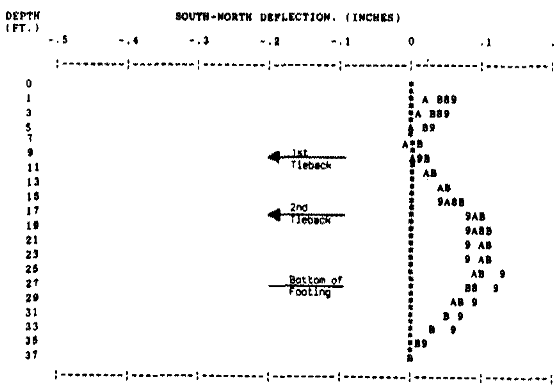


LEGEND
READING NO.1 DATE: 7-28-87
READING NO.2 DATE: 8-05-87
READING NO.3 DATE: 8-14-87
READING NO.4 DATE: 8-21-87



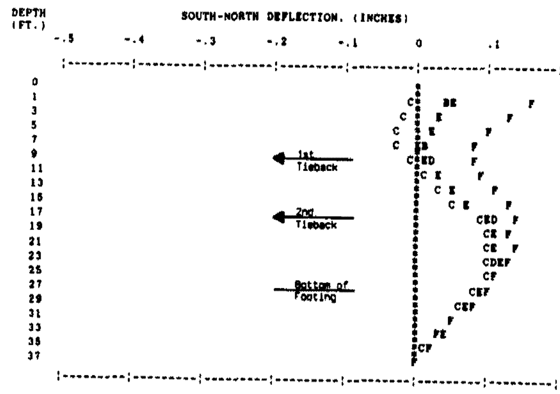
LEGEND
READING NO.4 DATE: 8-21-87
READING NO.5 DATE: 8-28-87
READING NO.6 DATE: 9-04-87
READING NO.7 DATE: 9-18-87
READING NO.8 DATE: 9-30-87

FIGURE 7



LEGEND
READING NO.8 DATE: 9-30-87
READING NO.9 DATE: 10-16-87
READING NO.A DATE: 11-02-87
READING NO.B DATE: 12-08-87

FIGURE 8



LEGEND
READING NO.B DATE: 12-08-87
READING NO.C DATE: 1-20-88
READING NO.D DATE: 2-22-88
READING NO.E DATE: 5-27-88
READING NO.F DATE: 8-27-88

FIGURE 8B

THE H. C. NUTTING CO.
DIGITILT INCLINOMETER SUMMARY SHEET

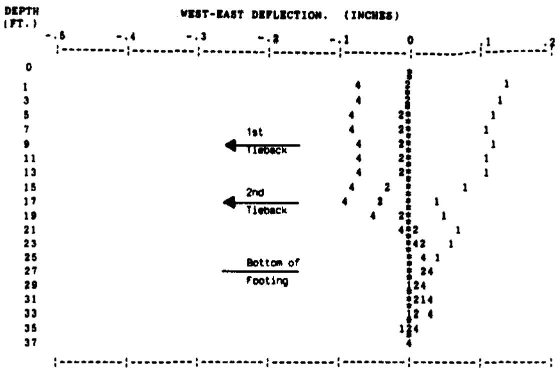
NORTH STREET GRADE SEPARATION

PROJECT NO. 8069.005

INCLINOMETER LOCATION NO. 24
TOP OF CASING ELEV. 876.6

HORIZONTAL SCALE: 1 INCH = .1 INCH
VERTICAL SCALE: 1 INCH = 10 FT.

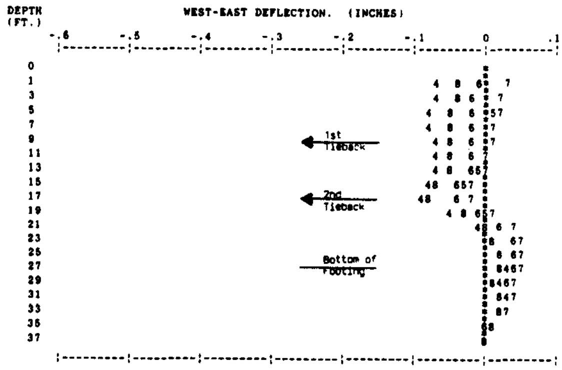
DATE OF REFERENCE READING: 7-16-87
LATEST READING: READING NO. 4 DATE: 8-21-87 FILE: LIMA25.4



LEGEND

READING NO.1 DATE: 7-28-87
READING NO.2 DATE: 8-05-87
READING NO.4 DATE: 8-21-87

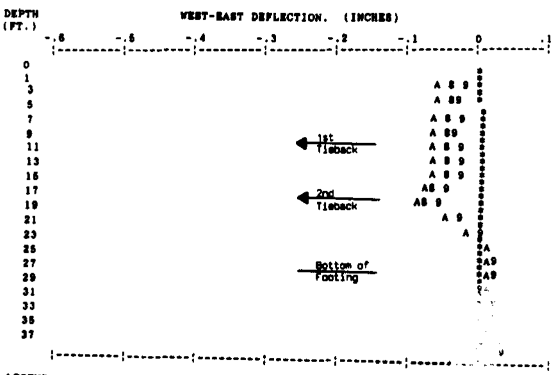
FIGURE 6A



LEGEND

READING NO.4 DATE: 8-21-87
READING NO.5 DATE: 8-28-87
READING NO.6 DATE: 9-04-87
READING NO.7 DATE: 9-18-87
READING NO.8 DATE: 9-30-87

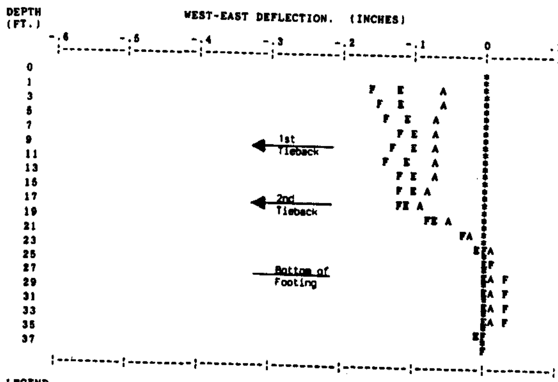
FIGURE 7A



LEGEND

READING NO.8 DATE: 9-30-87
READING NO.9 DATE: 10-16-87
READING NO.A DATE: 11-02-87

FIGURE 8A



LEGEND

READING NO.A DATE: 11-02-87
READING NO.E DATE: 5-27-88
READING NO.F DATE: 8-27-88

FIGURE 8C

THE H. C. NUTTING CO.
DIGITILT INCLINOMETER SUMMARY SHEET

NORTH STREET GRADE SEPARATION
PROJECT NO. 8069.005

INCLINOMETER LOCATION NO. 93 — HORIZONTAL SCALE: 1 INCH = .1 INCH
TOP OF CASING ELEV. 876.8 — VERTICAL SCALE: 1 INCH = 10 FT.

DATE OF REFERENCE READING: 7-16-87
LATEST READING: READING NO. 5 DATE: 8-21-87 FILE: LIMA93.5

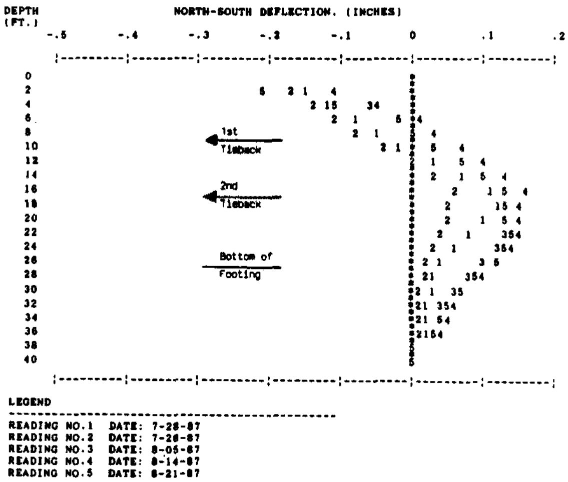


FIGURE 9

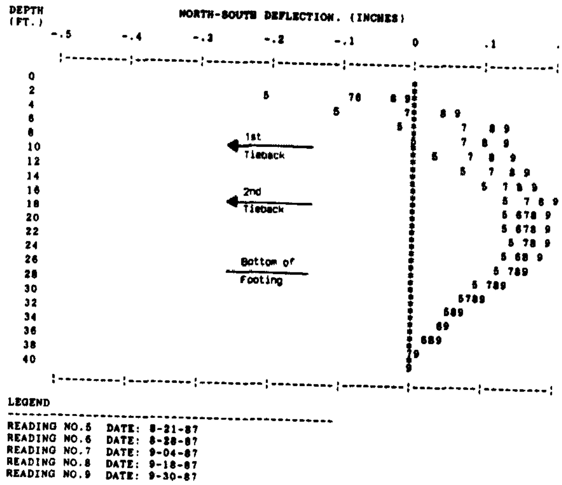


FIGURE 10

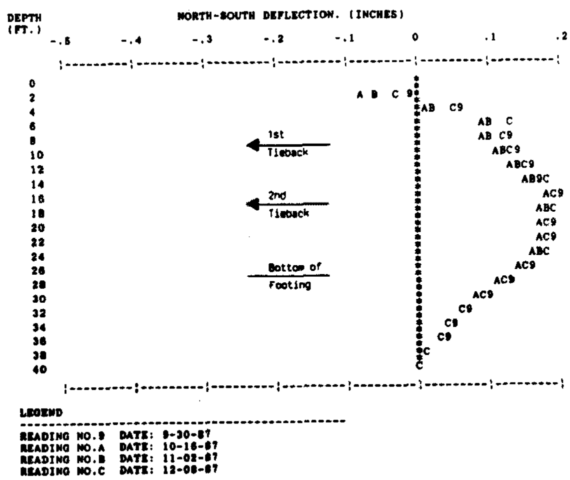


FIGURE 11

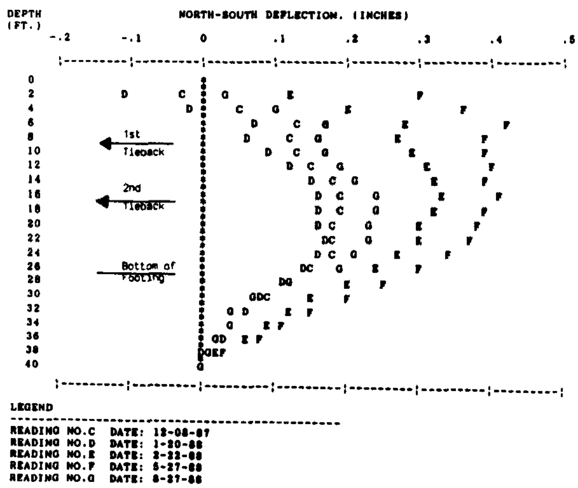


FIGURE 11B

THE H. C. NUTTING CO.
DIGITILT INCLINOMETER SUMMARY SHEET

NORTH STREET GRADE SEPARATION

PROJECT NO. 8069.005

INCLINOMETER LOCATION NO. 93 HORIZONTAL SCALE: 1 INCH = .1 INCH
TOP OF CASING ELEV. 876.8 VERTICAL SCALE: 1 INCH = 10 FT.

DATE OF REFERENCE READING: 7-16-87
LATEST READING: READING NO. 5 DATE: 8-21-87 FILE: LIMA93.5

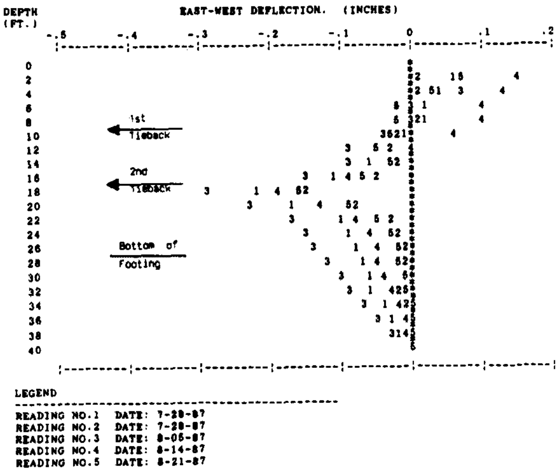


FIGURE 9A

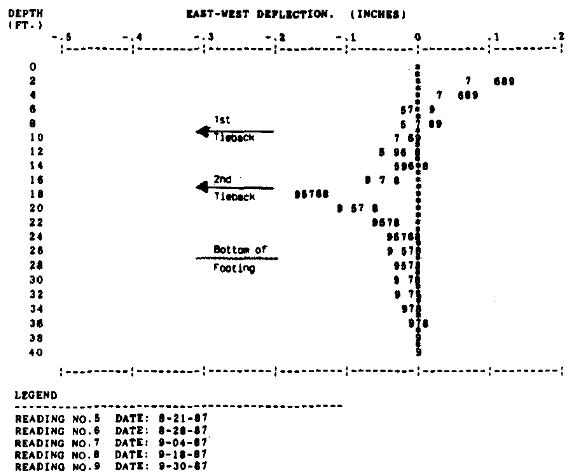


FIGURE 10A

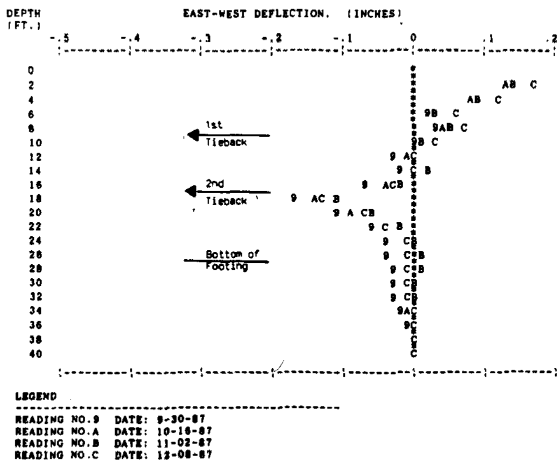


FIGURE 11A

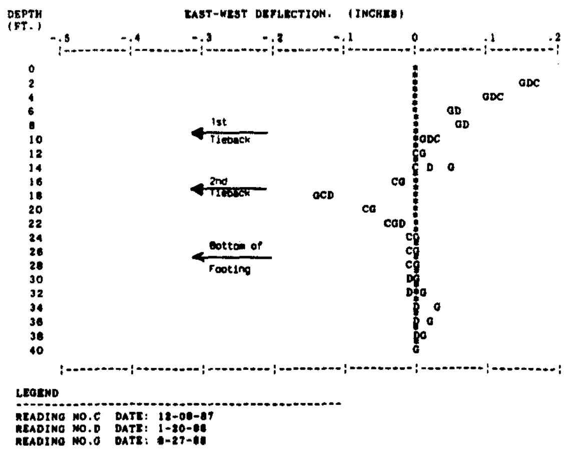
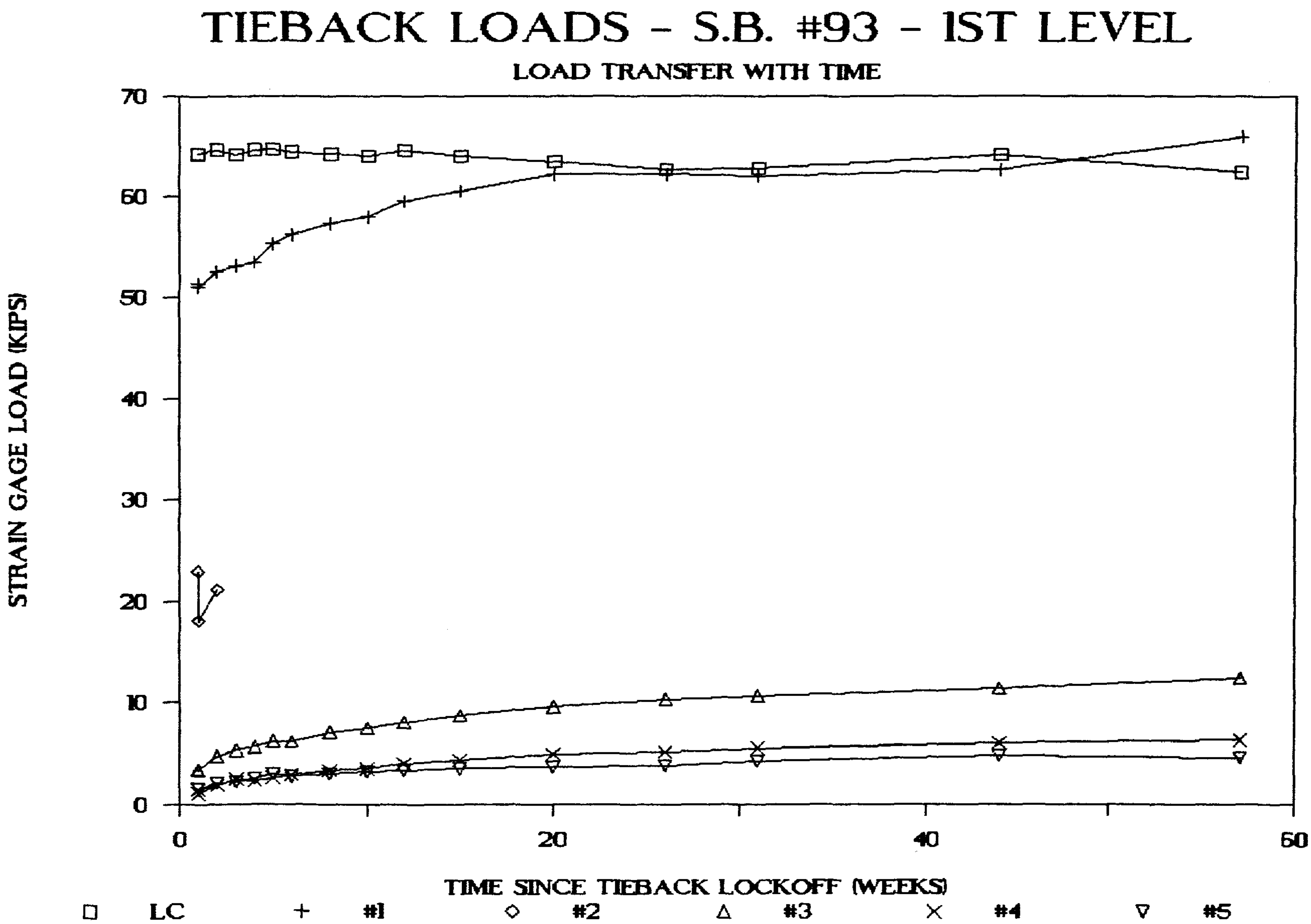


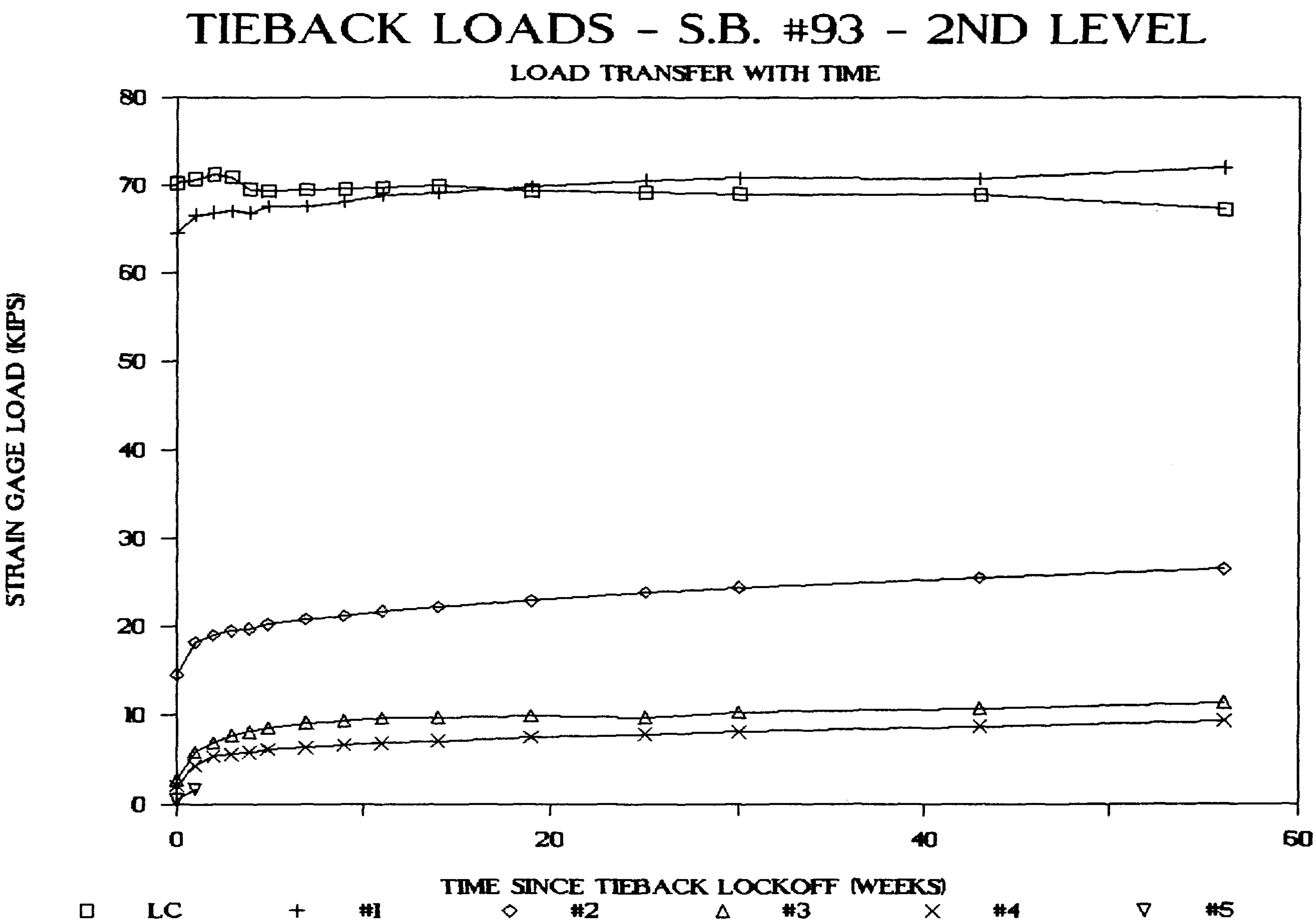
FIGURE 11C



GRAPH 1

6-B-1

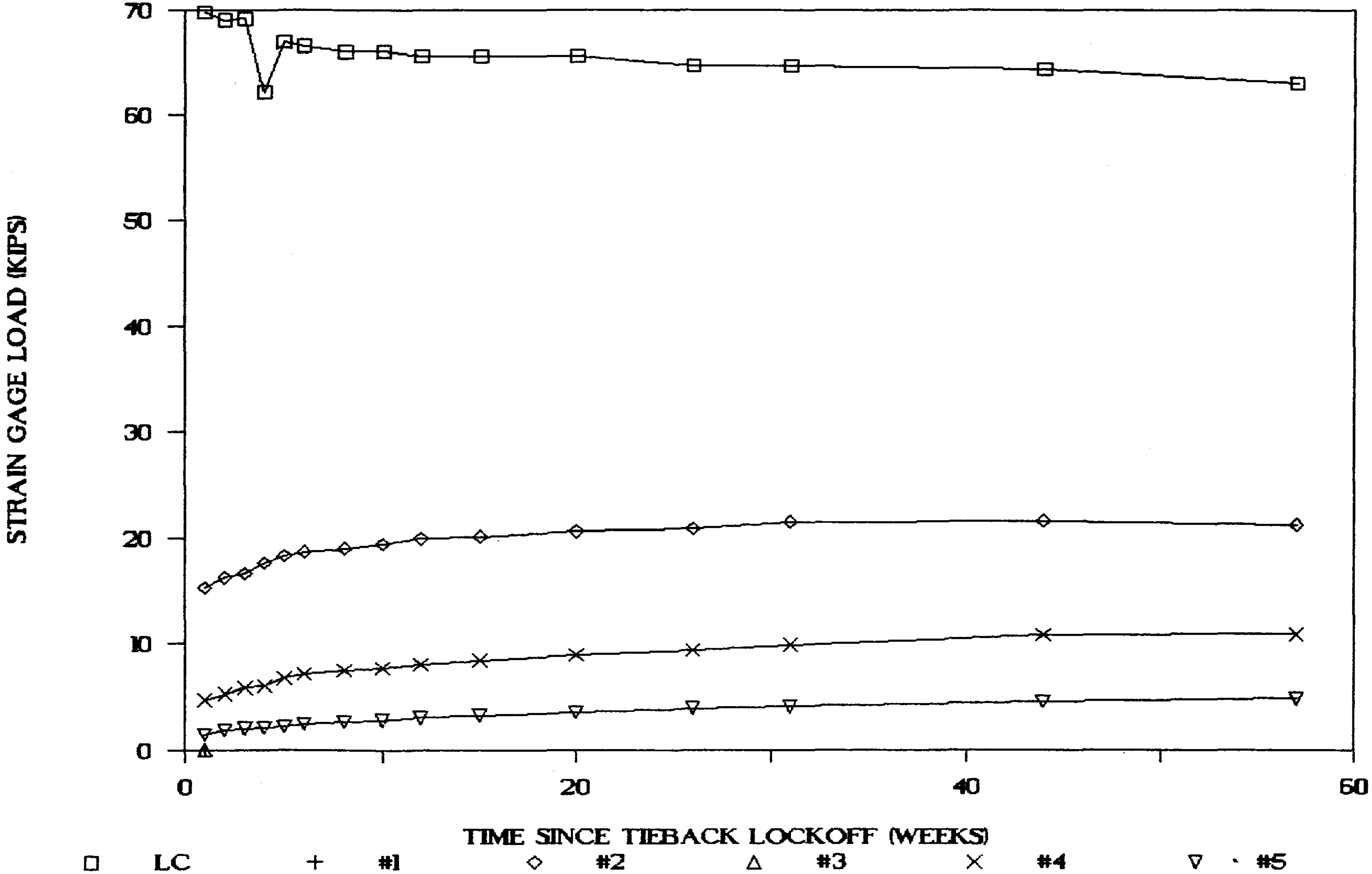
APPENDIX B


TIEBACK LOADS - S.B. #93 - 2ND LEVEL
LOAD TRANSFER WITH TIME
STRAIN GAGE LOAD (KIPS)
80
70
60
50
40
30
20
10
0
0
20
40
60
TIME SINCE TIEBACK LOCKOFF (WEEKS)
LC
#1
#2
#3
#4
#5

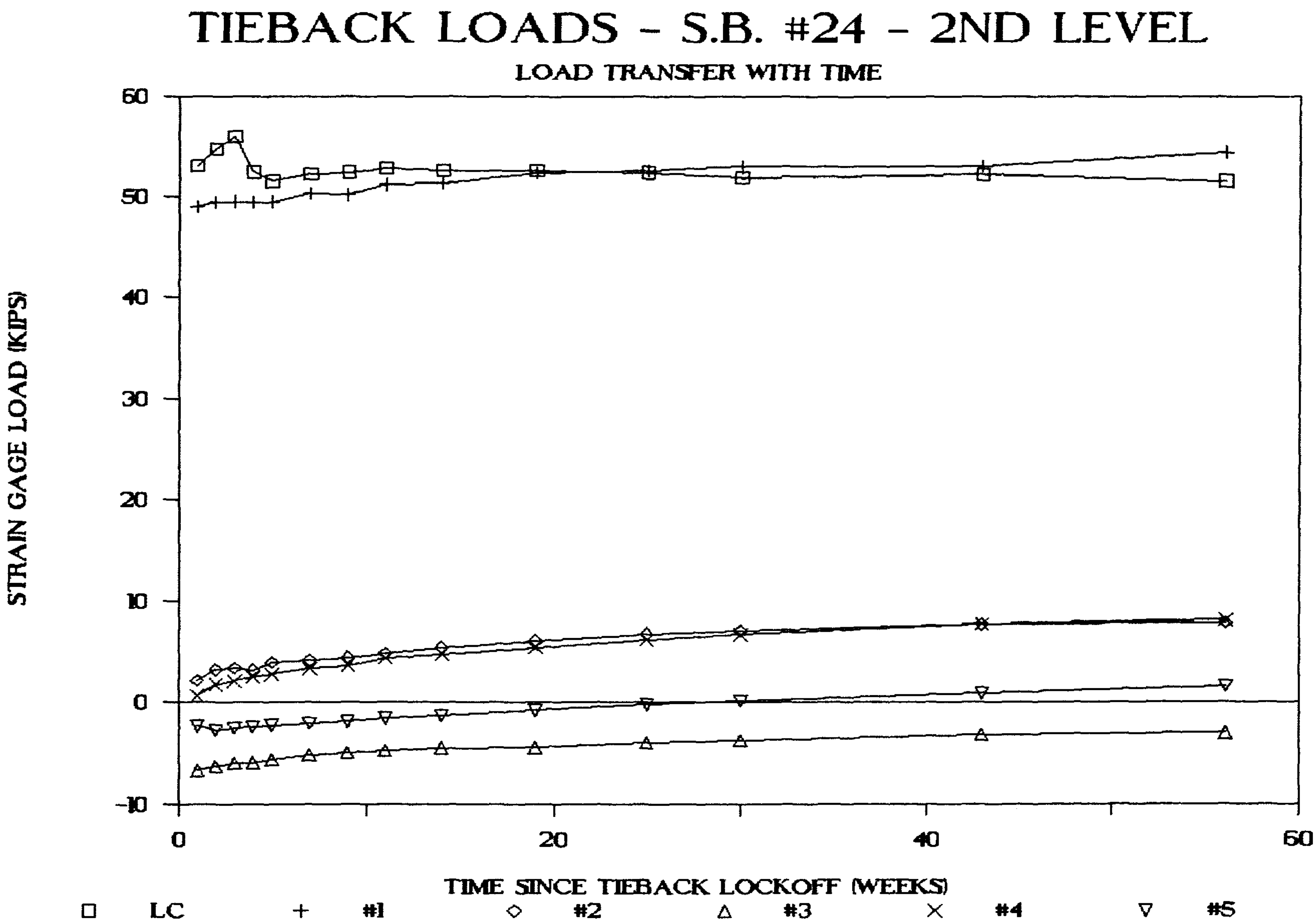
GRAPH 2

TIEBACK LOADS - S.B. #24 - 1ST LEVEL

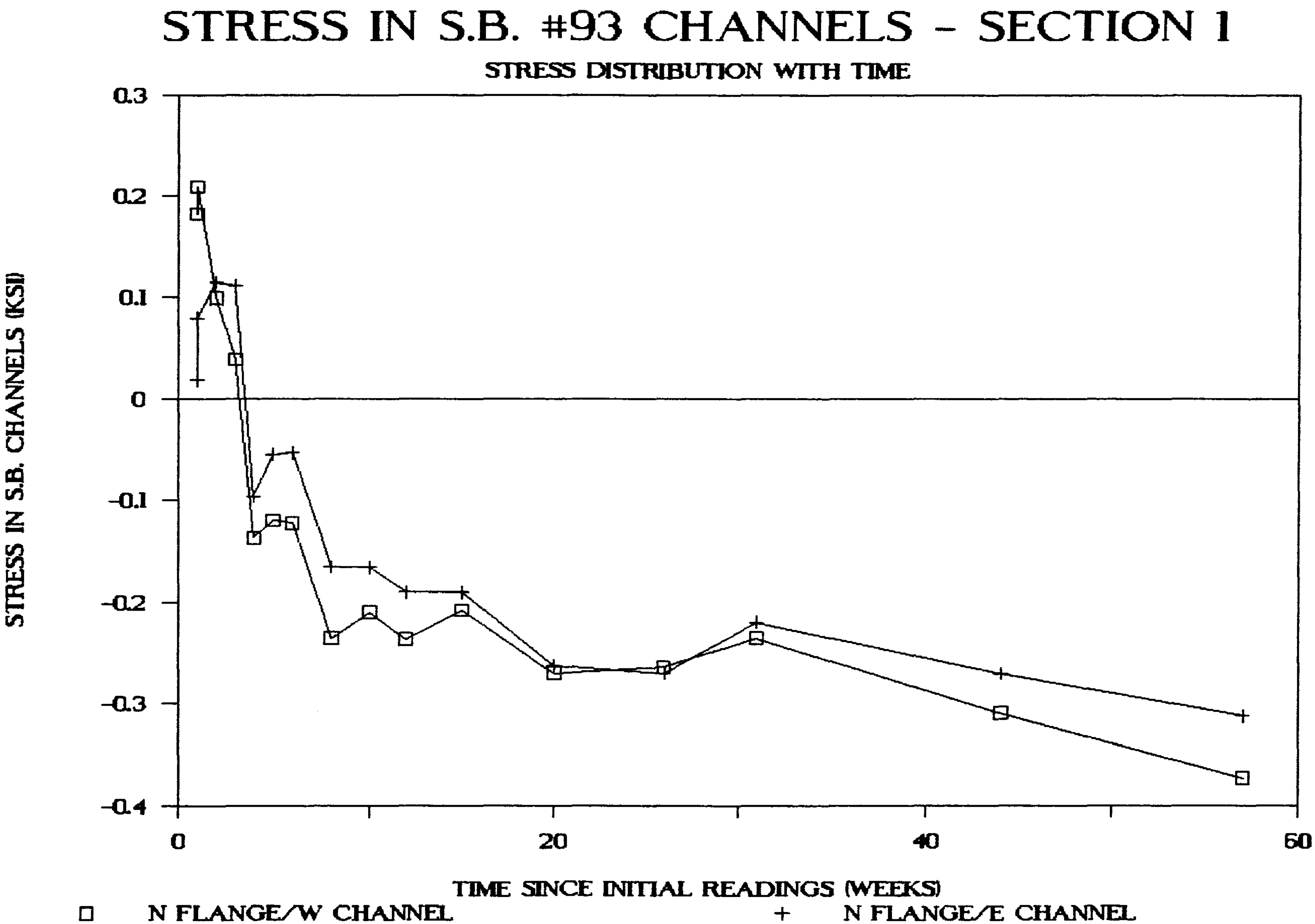
LOAD TRANSFER WITH TIME



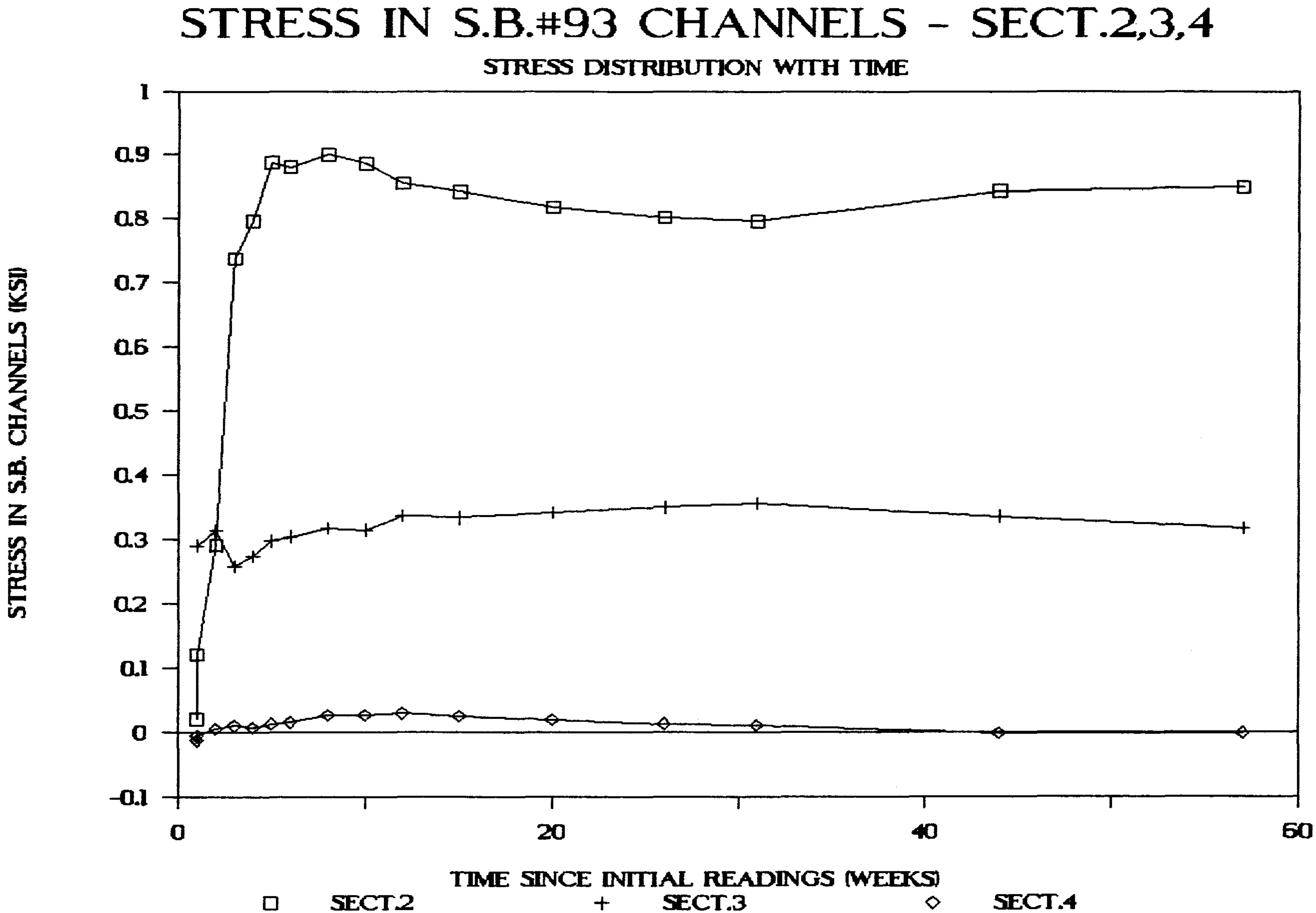
GRAPH 3


TIEBACK LOADS - S.B. #24 - 2ND LEVEL
LOAD TRANSFER WITH TIME
STRAIN GAGE LOAD (KIPS)
60
50
40
30
20
10
0
-10
0
20
40
60
TIME SINCE TIEBACK LOCKOFF (WEEKS)
LC
#1
#2
#3
#4
#5

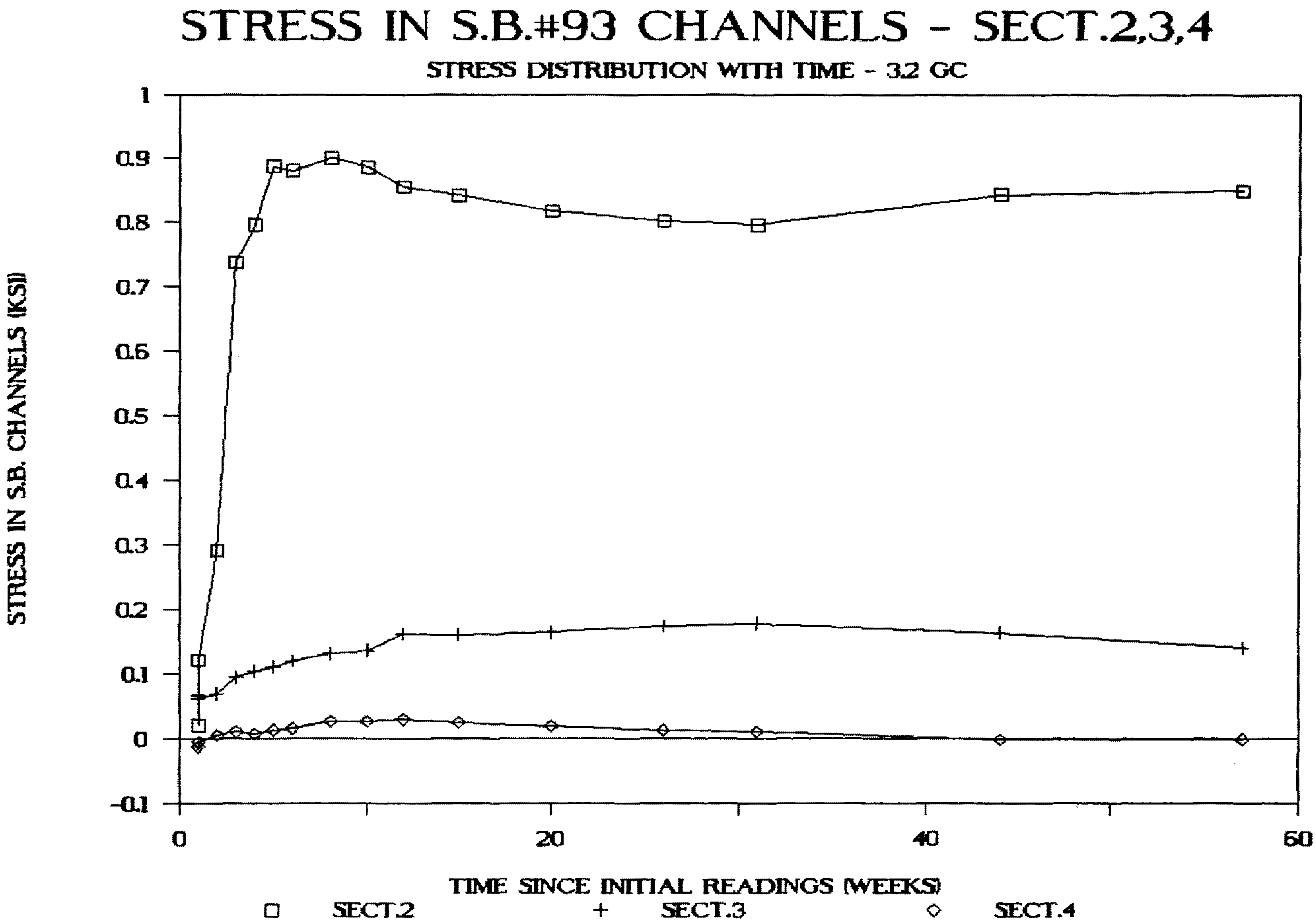
GRAPH 4


STRESS IN S.B. #93 CHANNELS - SECTION 1
STRESS DISTRIBUTION WITH TIME
STRESS IN S.B. CHANNELS (KSI)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
N FLANGE/W CHANNEL
N FLANGE/E CHANNEL

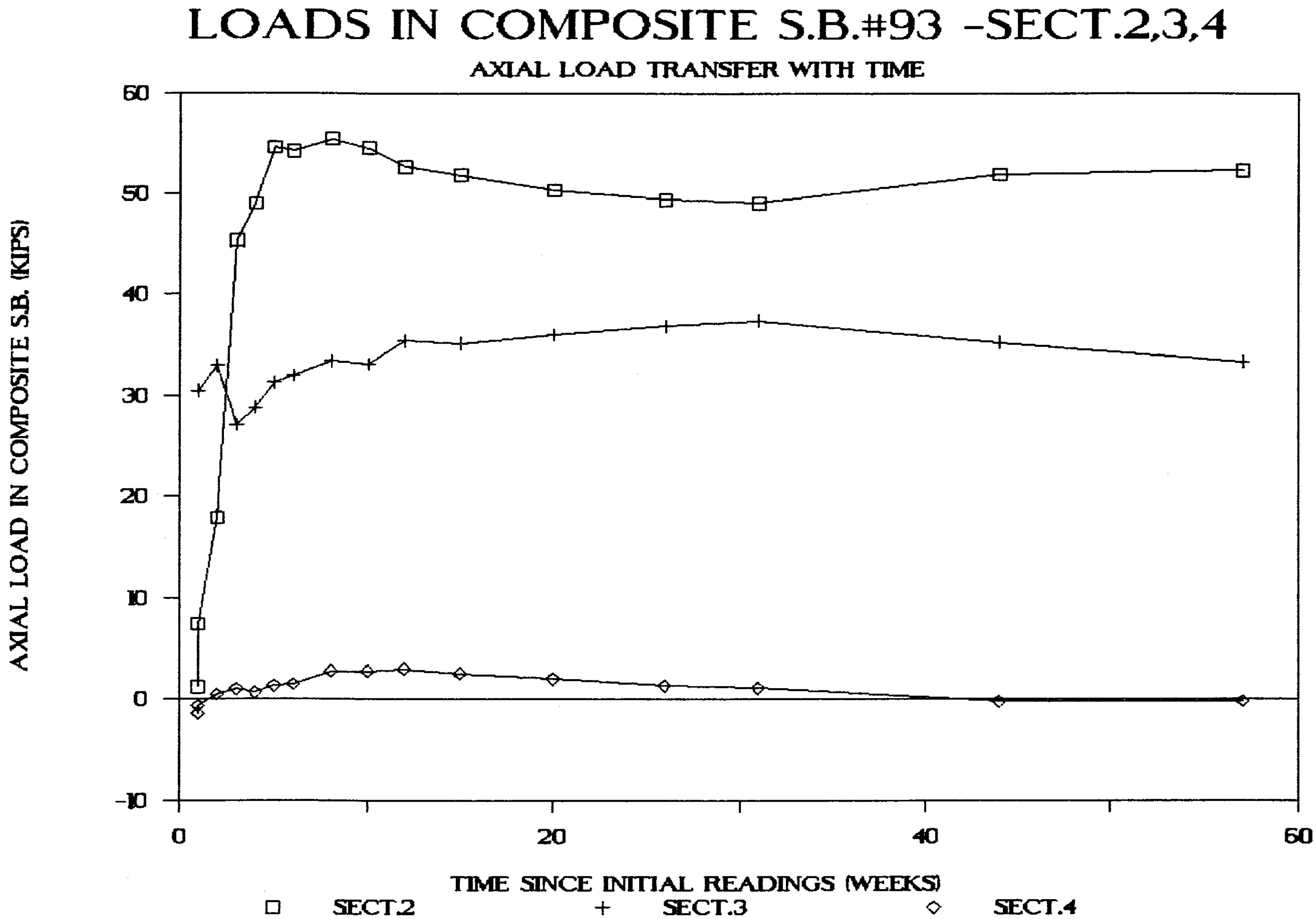
GRAPH 5


STRESS IN S.B.#93 CHANNELS - SECT.2,3,4
STRESS DISTRIBUTION WITH TIME
STRESS IN S.B. CHANNELS (KSI)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
SECT.2
SECT.3
SECT.4

GRAPH 6


STRESS IN S.B.#93 CHANNELS - SECT.2,3,4
STRESS DISTRIBUTION WITH TIME - 3.2 GC
STRESS IN S.B. CHANNELS (KSI)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
SECT.2
SECT.3
SECT.4

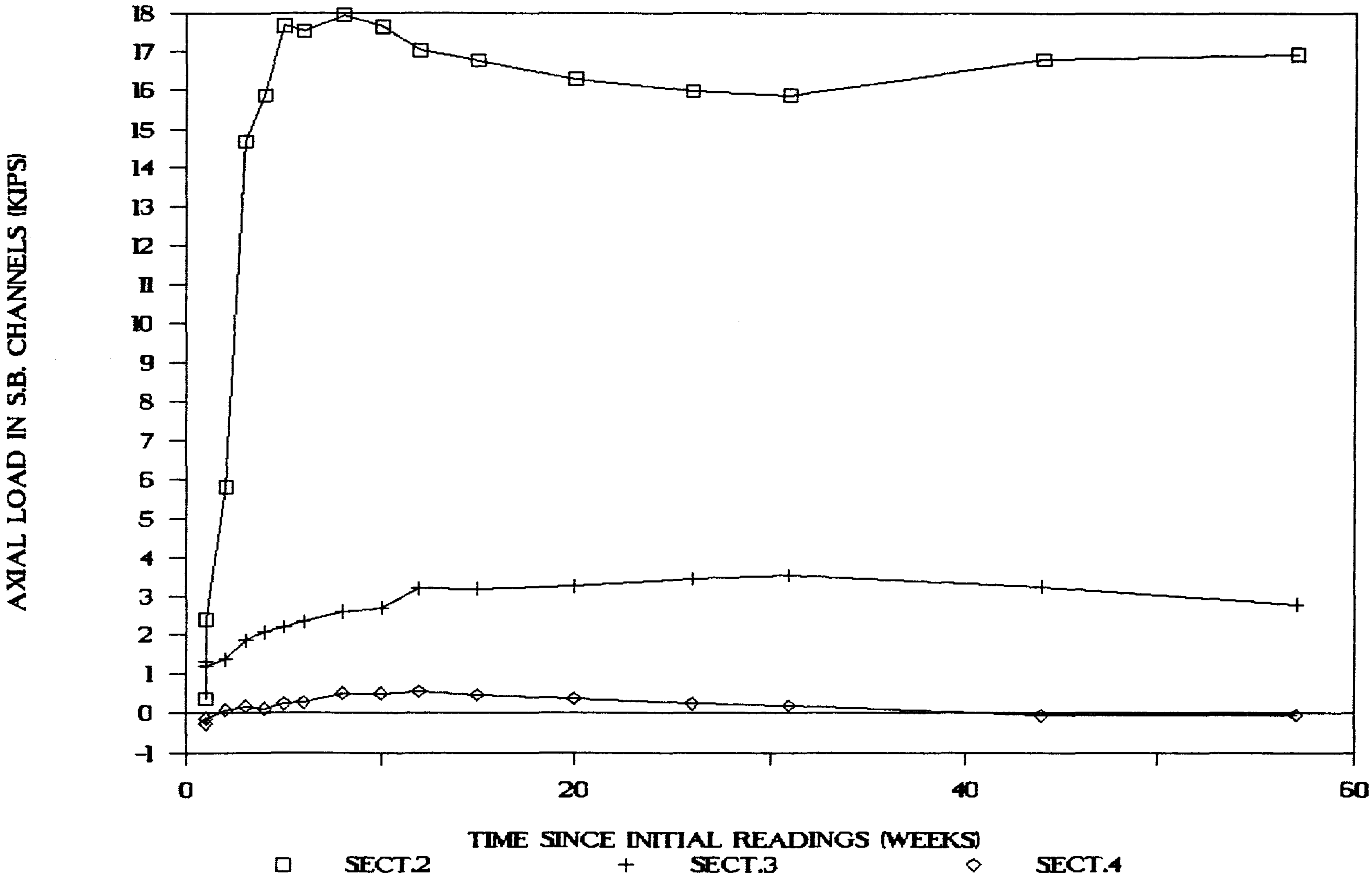
GRAPH 7


LOADS IN COMPOSITE S.B.#93 -SECT.2,3,4
AXIAL LOAD TRANSFER WITH TIME
AXIAL LOAD IN COMPOSITE S.B. (KIPS)
60
50
40
30
20
10
0
-10
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
SECT.2
SECT.3
SECT.4

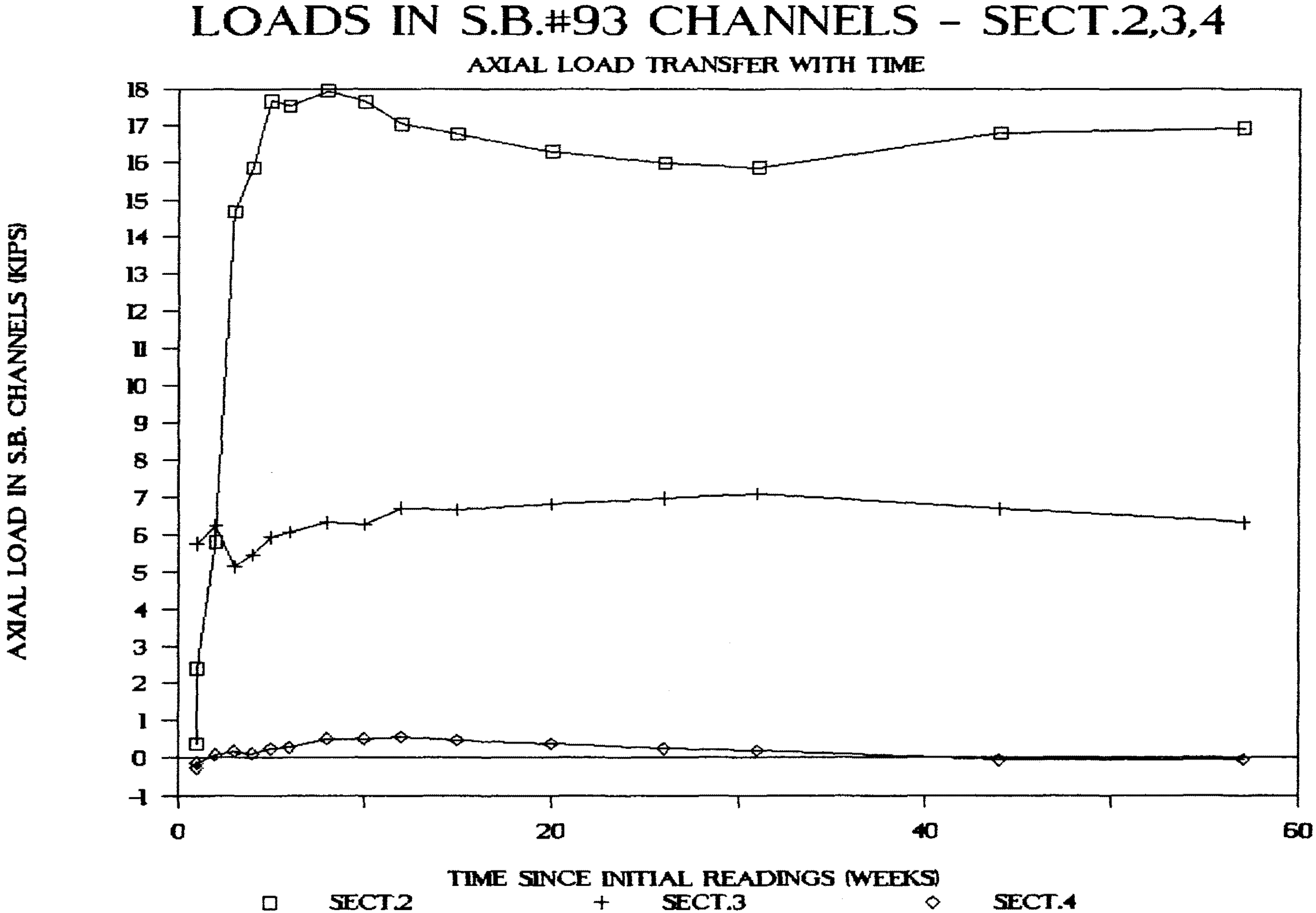
GRAPH 8

LOADS IN S.B.#93 CHANNELS - SECT.2,3,4

AXIAL LOAD TRANSFER WITH TIME - 3.2 GC

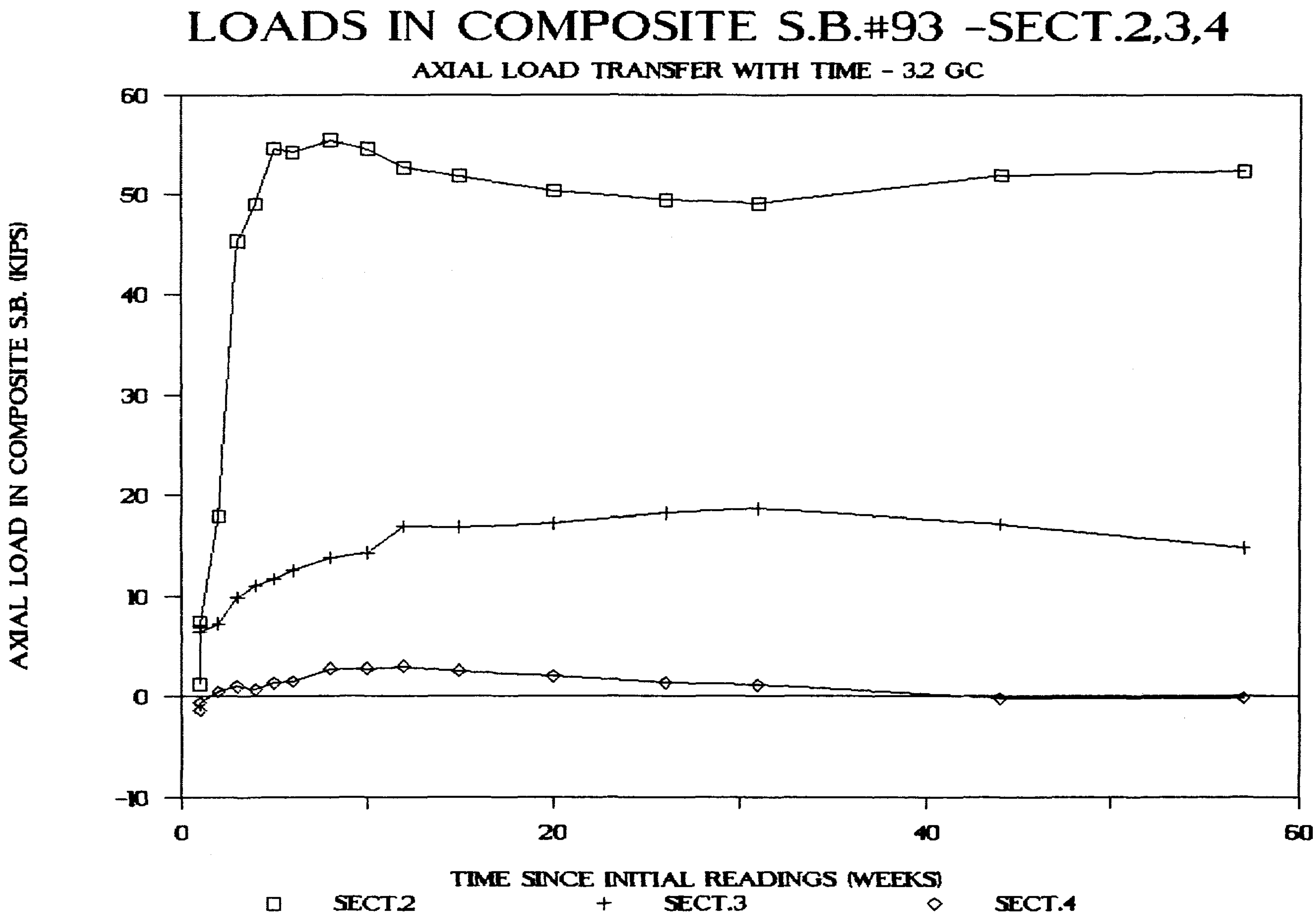


GRAPH 9



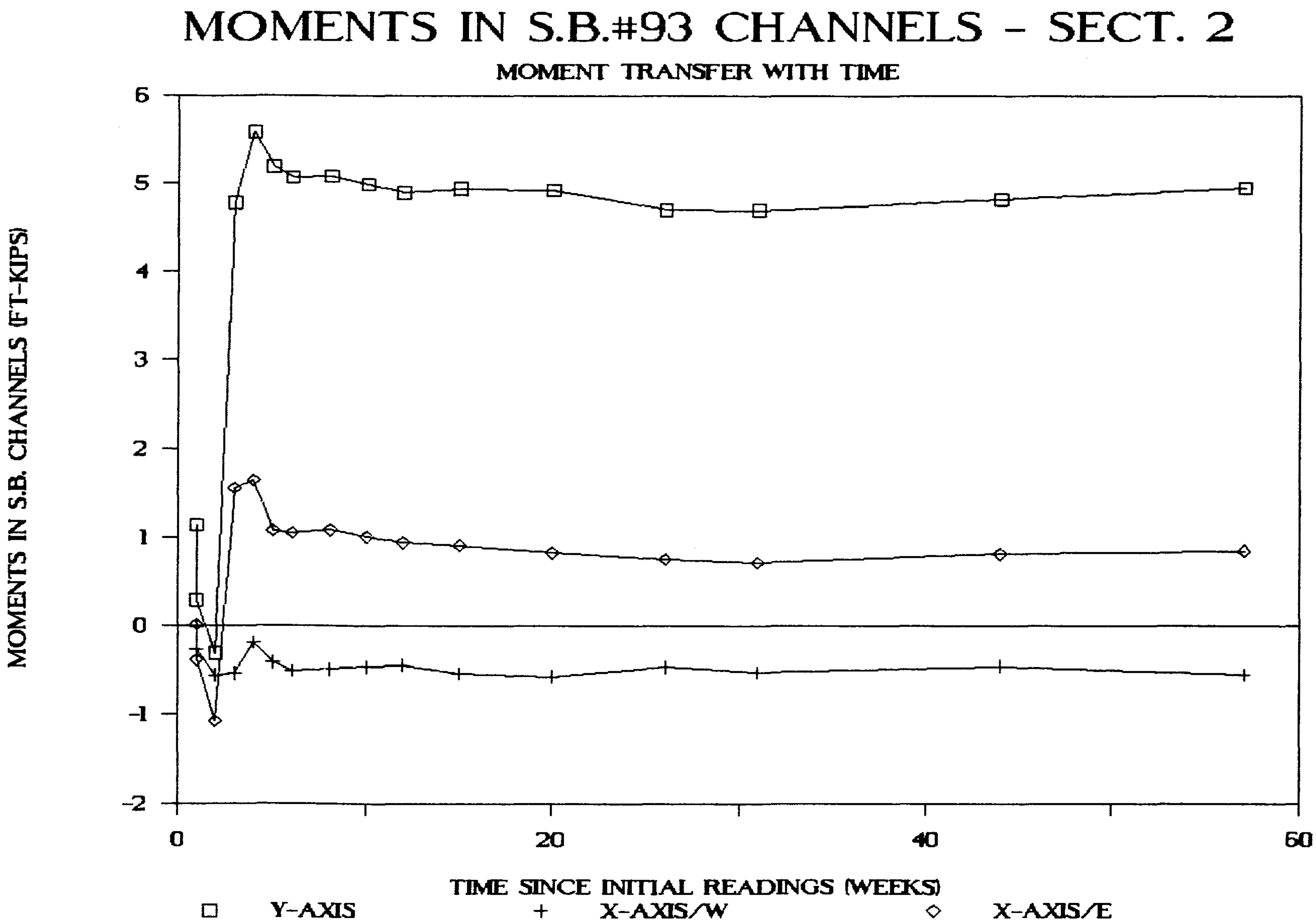
LOADS IN S.B.#93 CHANNELS - SECT.2,3,4
AXIAL LOAD TRANSFER WITH TIME
AXIAL LOAD IN S.B. CHANNELS (KIPS)
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
SECT.2
SECT.3
SECT.4

GRAPH 10

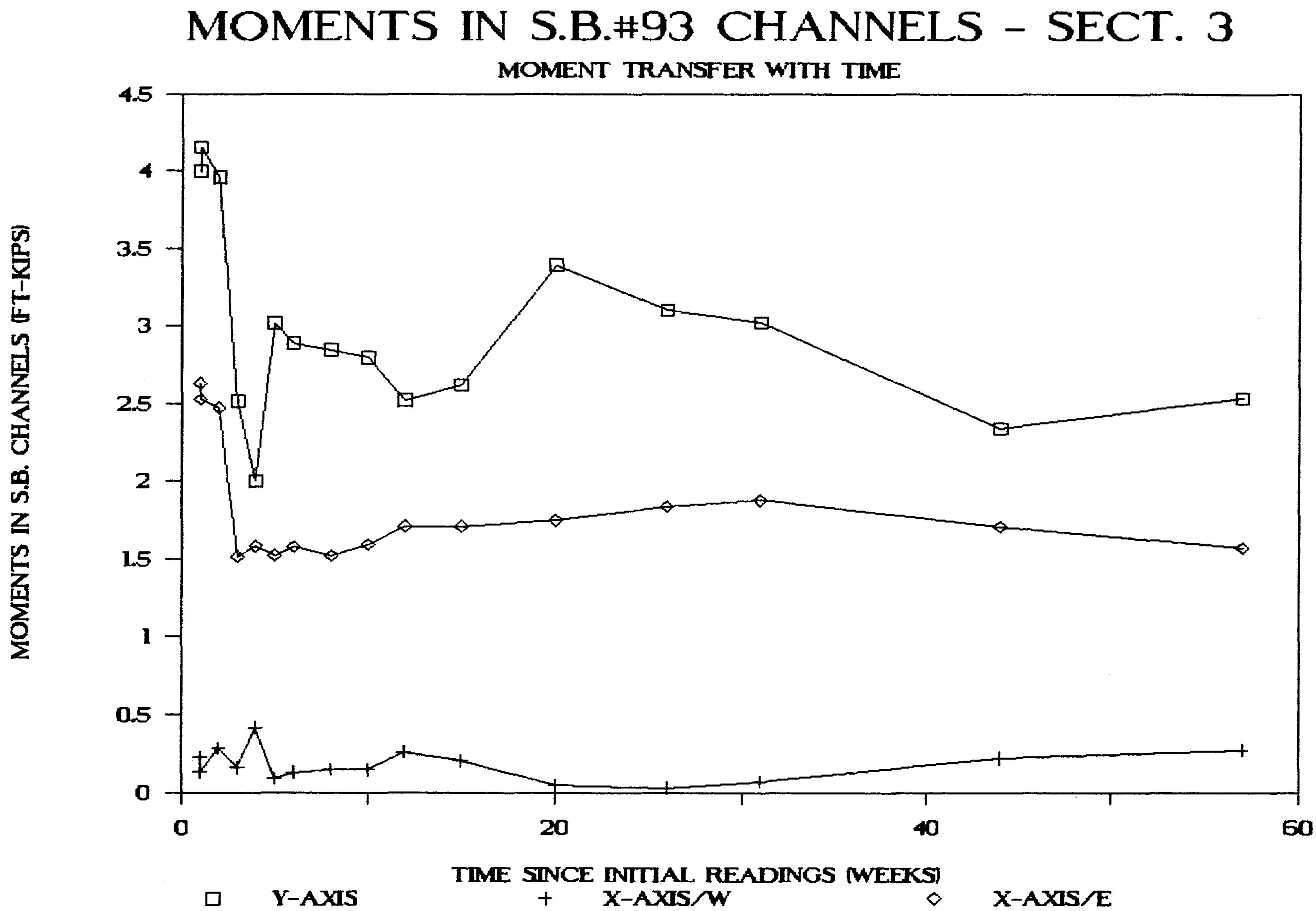


LOADS IN COMPOSITE S.B.#93 -SECT.2,3,4
AXIAL LOAD TRANSFER WITH TIME - 3.2 GC
AXIAL LOAD IN COMPOSITE S.B. (KIPS)
60
50
40
30
20
10
0
-10
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
SECT.2
SECT.3
SECT.4

GRAPH 11


MOMENTS IN S.B.#93 CHANNELS - SECT. 2
MOMENT TRANSFER WITH TIME
MOMENTS IN S.B. CHANNELS (FT-KIPS)
6
5
4
3
2
1
0
-1
-2
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
Y-AXIS
X-AXIS/W
X-AXIS/E

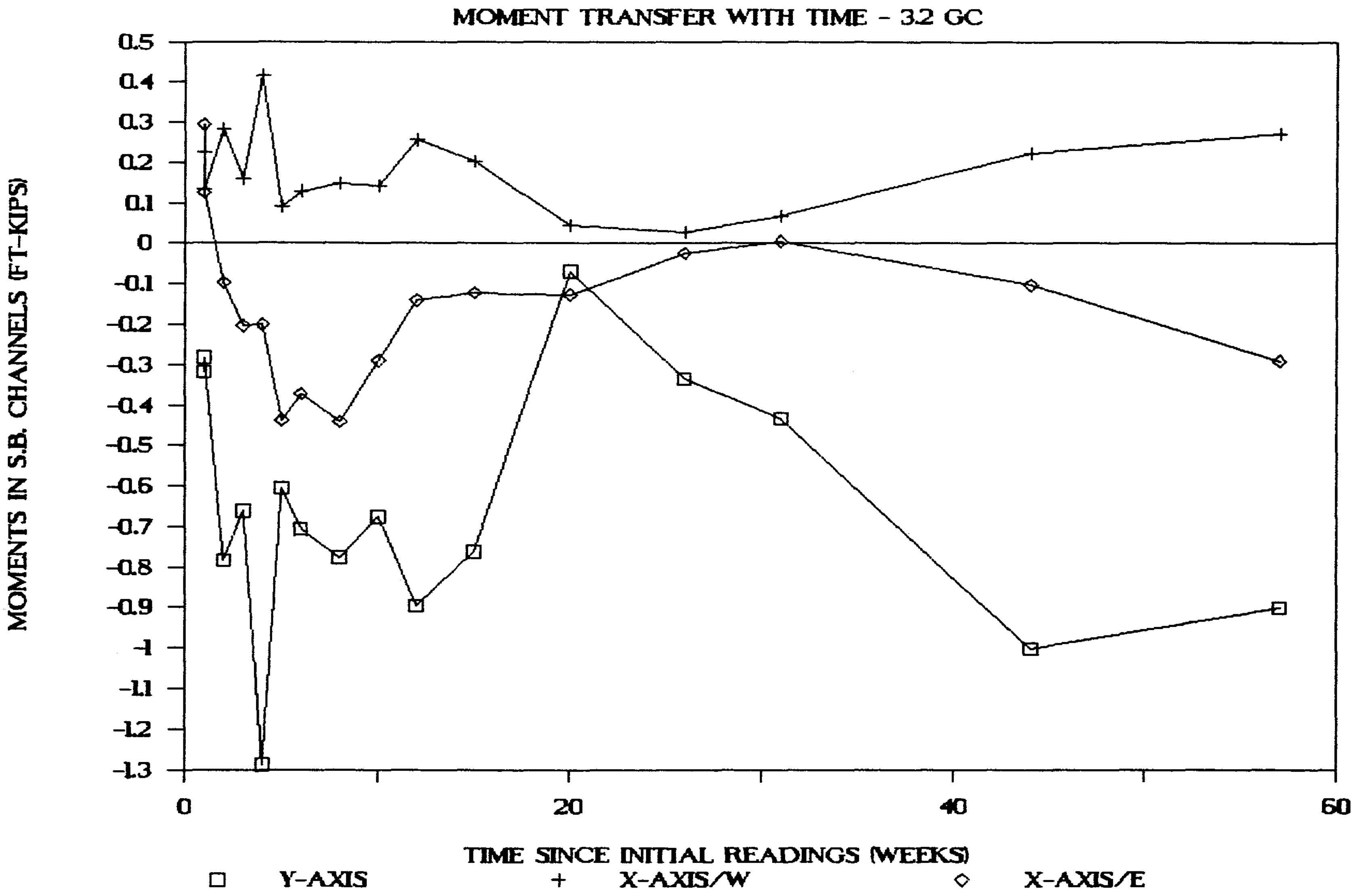
GRAPH 12



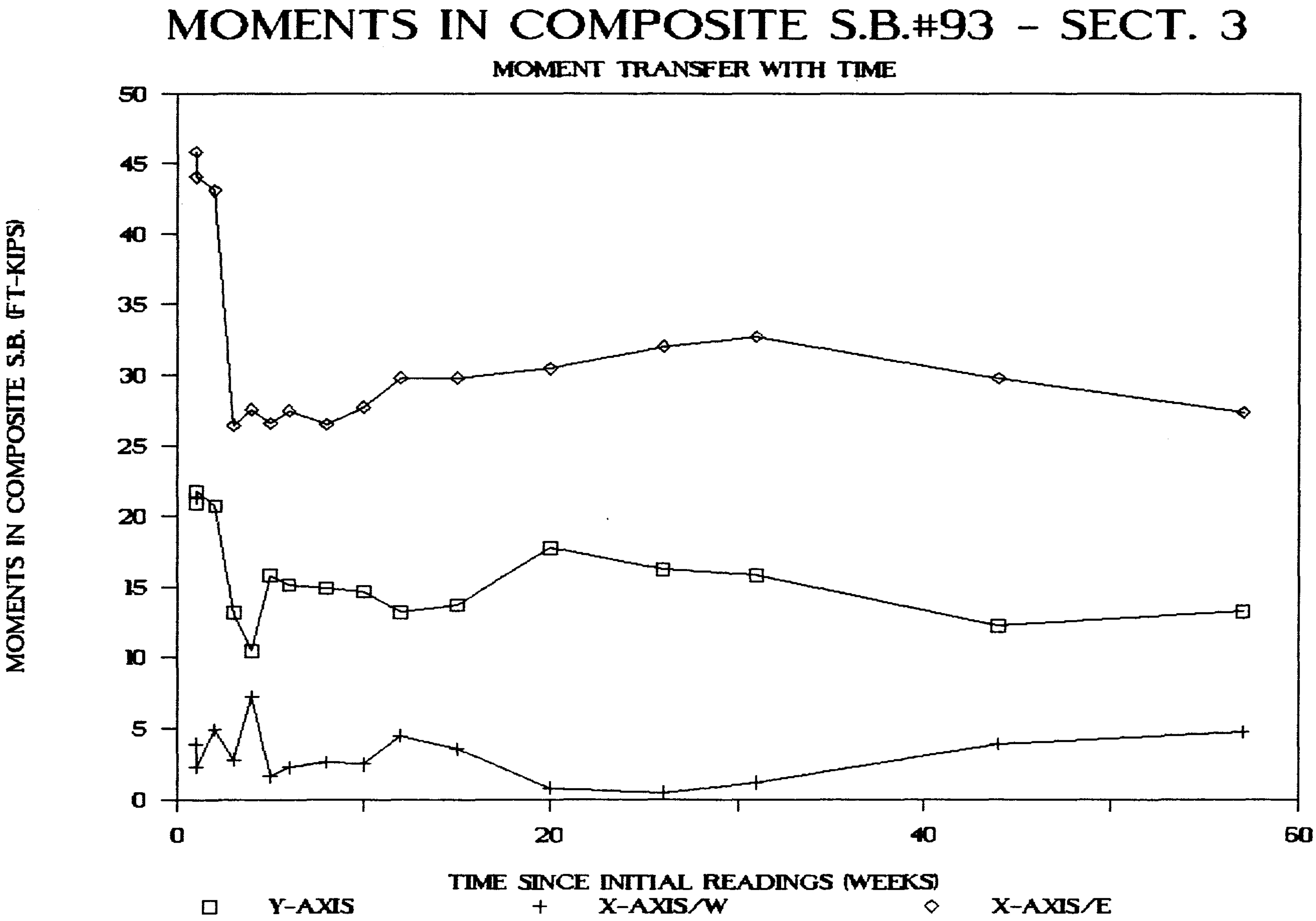
MOMENTS IN S.B.#93 CHANNELS - SECT. 3
MOMENT TRANSFER WITH TIME
MOMENTS IN S.B. CHANNELS (FT-KIPS)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
Y-AXIS
X-AXIS/W
X-AXIS/E

GRAPH 13

MOMENTS IN S.B.#93 CHANNELS - SECT. 3



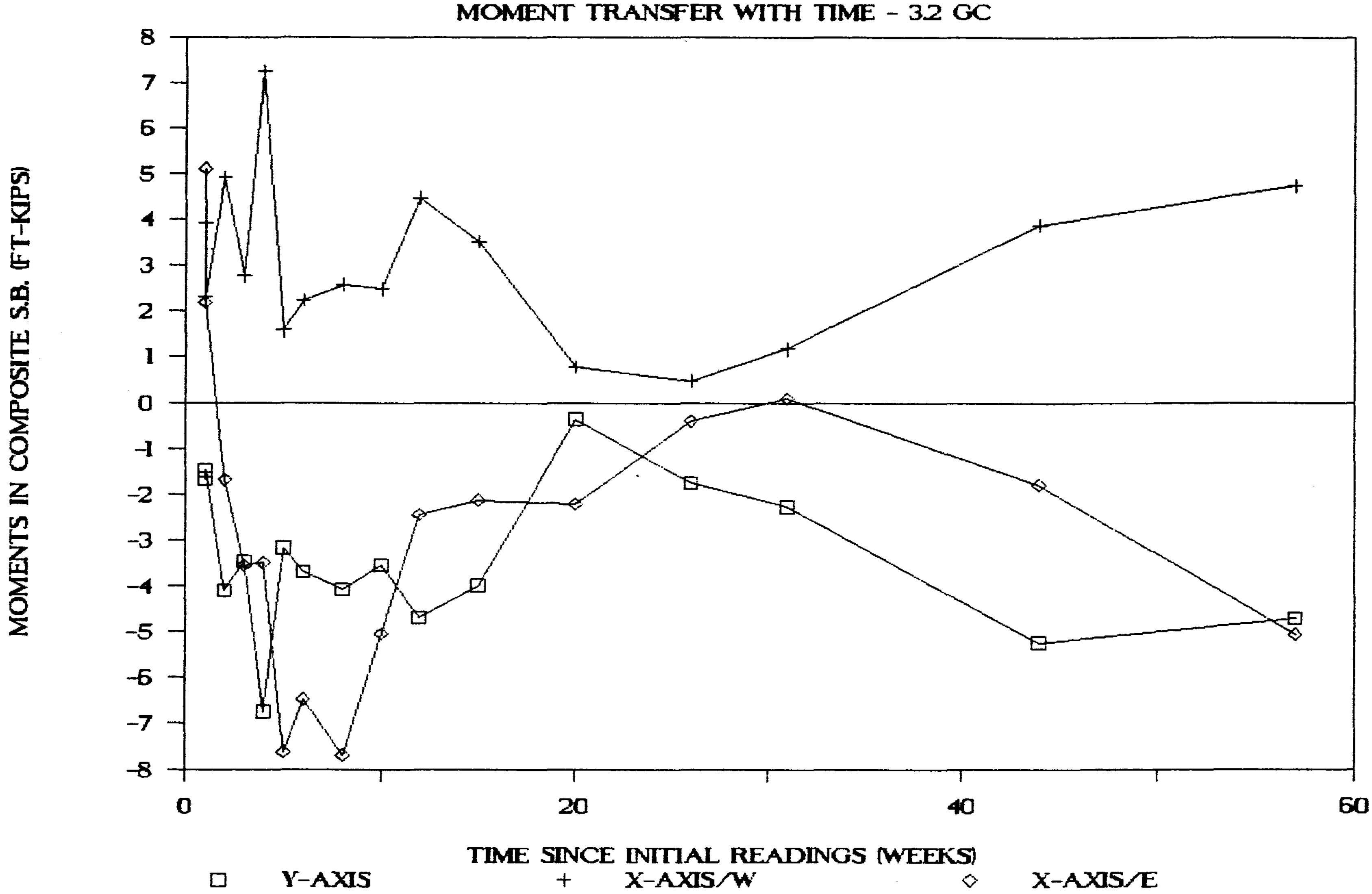
GRAPH 14



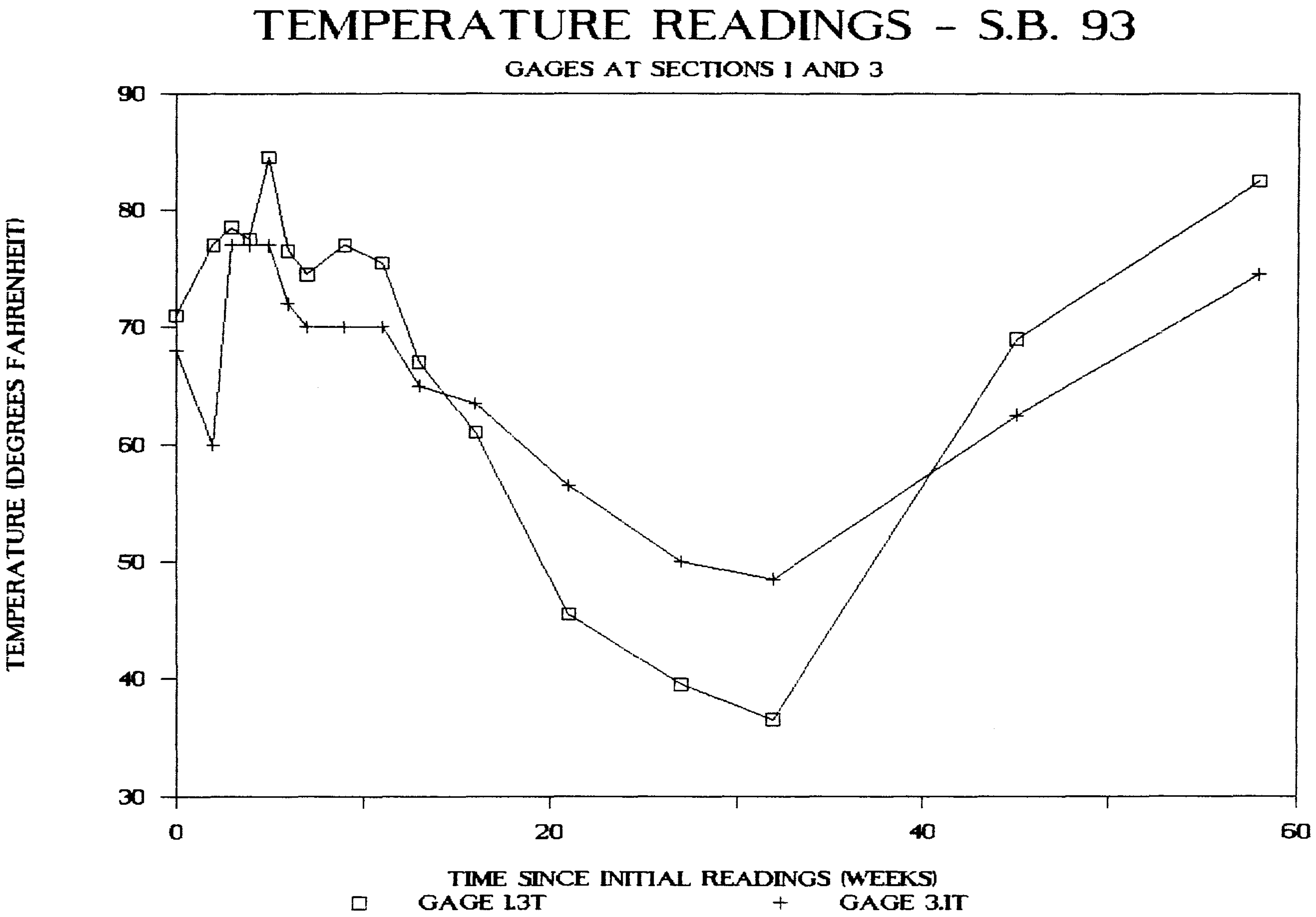
6-B-15

GRAPH 15

MOMENTS IN COMPOSITE S.B.#93 - SECT. 3

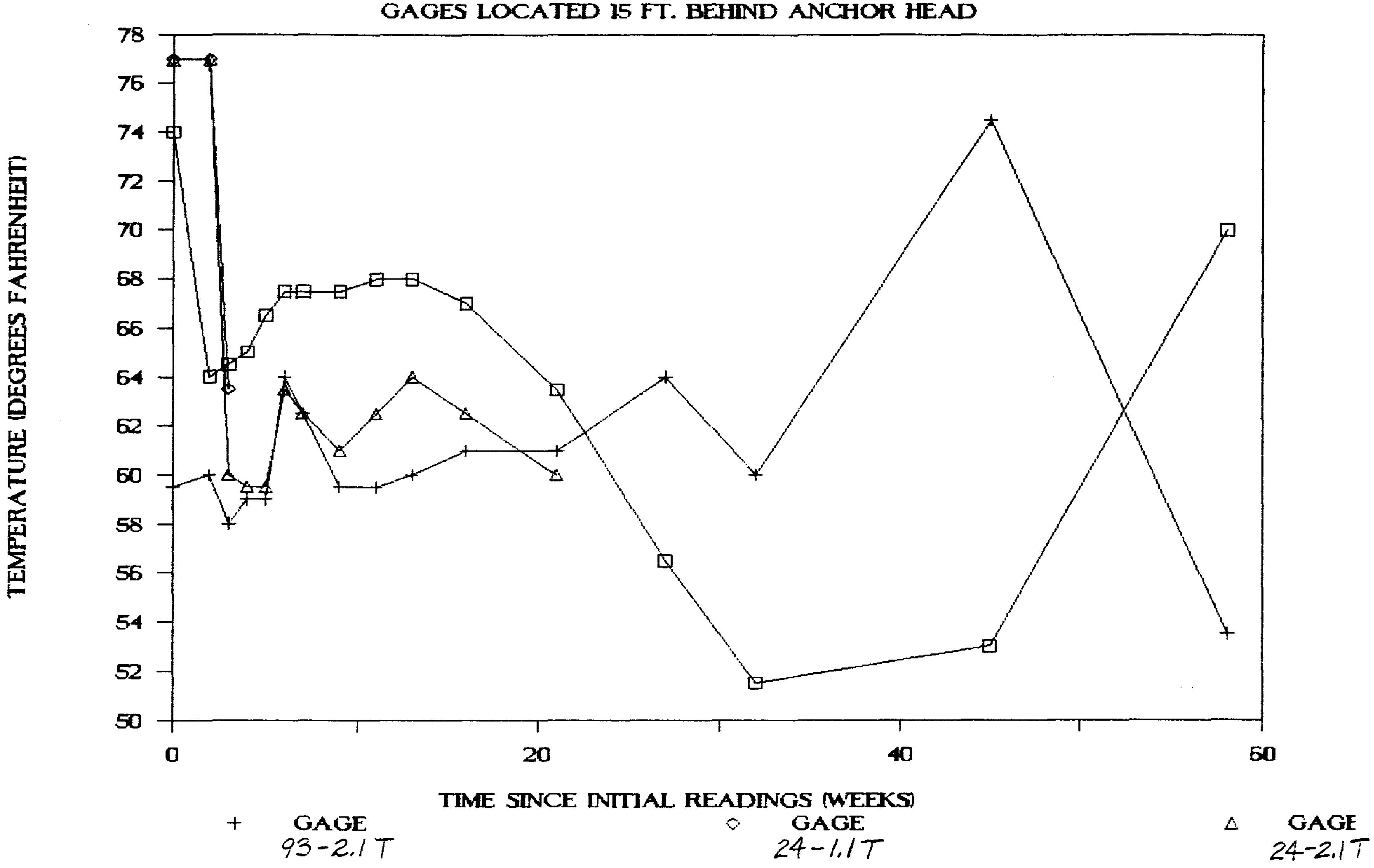


GRAPH 16


TEMPERATURE READINGS - S.B. 93
GAGES AT SECTIONS 1 AND 3
TEMPERATURE (DEGREES FAHRENHEIT)
90
80
70
60
50
40
30
0
20
40
60
TIME SINCE INITIAL READINGS (WEEKS)
GAGE 1.3T
GAGE 3.1T

GRAPH 17

TEMPERATURE READINGS - TIEBACK GAGES



GRAPH 18

REPORT NO. 5

PERMANENT GROUND ANCHORS FOR LATERAL SUPPORT OF BRIDGE END SLOPES

DEMONSTRATION SITE
DIMOND BOULEVARD UNDERPASS STRUCTURE
PROJECT NO. FM-0520(1)

SUBMITTED BY:

ALASKA DEPARTMENT OF TRANSPORTATION
TO THE FHWA DEMONSTRATION PROJECTS DIVISION
FOR INCLUSION IN
FHWA DEMONSTRATION PROJECT NO. 68
PERMANENT GROUND ANCHORS
ANCHORAGE, ALASKA

NOVEMBER 1986

TABLE OF CONTENTS

I. INTRODUCTION

A permanent ground-anchored retaining wall has been constructed on the Dimond Boulevard Project. The widening of Dimond Boulevard beneath the new Seward Highway overpass structure required the removal of a portion of the existing bridge end fill slope. This slope supported the abutments which were founded on spread footings. The only retaining wall that was possible to construct without a detrimental effect to the overpass structure was a soldier pile wall using permanent soil anchors. To our knowledge, this was only the second time that permanent ground anchors were used for a retaining wall supporting a bridge structure founded on spread footings in the United States. The other case was in Washington State.

The primary purpose of this project is to:

1. Evaluate the performance of permanent ground-anchored retaining walls supporting and existing bridge structure founded on spread footings.

2. Measure the bridge settlement that takes place during and after the installation of the retaining wall.

3. Provide data for incorporation into the Federal Highway Administration's Permanent Ground Anchor Demonstration Project No. 68.

This progress report summarizes the installation of the instrumentation, initial data analysis, and bridge performance during the first year after construction. The final report scheduled for January 1989 will present all results and conclusions of the test program.

II. RETAINING WALL DESIGN

The New Seward Highway overpass was originally constructed in 1976. This structure is a 110 foot long, single span, prestressed concrete girder structure supported on spread footings founded in approach fills. In 1986, Dimond Boulevard was widened from 66 feet to approximately 96 feet. To increase the headroom clearance, the Dimond Boulevard grade was also lowered two feet, which required an eight foot subcut below the existing grade to remove pockets of buried peat.

The grade and template changes to Dimond Boulevard required the construction of two permanent retaining walls. A 2124 square foot retaining wall supported each abutment. The maximum height of the permanent wall is 12 feet; however, the wall was constructed to a height of 18 feet to support the walls of the sub-excavation. This increased the total wall area to approximately 3100 square feet for each wall.

The retaining wall was designed by the Alaska Department of Transportation and Public Facilities. A performance specification was used for the permanent ground anchors. Based on this specification, the contractor was required to:

1. meet an experience-based pre-qualification requirement,

2. design the ground anchors including method of installation and bond length,

3. proof test each anchor to 150 percent of the design load, and

4. conduct performance tests on the first four anchors to 150 percent of the design load.

The soil conditions at the location of the soldier piles were primarily dense sandy gravel to gravelly silty sand with occasional cobbles (backfill material) overlying gravelly silty sand (glacial till). This area was previously a peat bog which had been excavated during the initial construction of the overpass. Pockets of peat, which were located at the edge of the existing structure, were also removed as part of this project. The water table was 7 feet below the ground surface.

The soils in the vicinity of the bond zone of the ground anchors were primarily very dense sandy gravel to gravelly sand (backfill) overlying silty sandy gravel (glacial till).

III. RETAINING WALL CONSTRUCTION

The HP 12 x 84 soldier piles were installed through holes drilled in the bridge deck. The soldier piles were spaced at 7 feet, 6 inches, and embedded below ground using an ICS 812 vibratory hammer instead of pre-boring the holes. The piles were driven 30 feet below original ground, or 22 feet below the bottom of the subcut. A template consisted of a 12 inch tubular steel frame supporting angle clips which restrained the pile flanges was used to maintain alignment.

Alignment of the driven piles was true within 1/2 inch. Some difficulty in obtaining exact alignment and elevation required that piles be occasionally extracted part way and re-driven. This was easily accomplished with the vibratory hammer. The only problem encountered was friction heating of the angle clips due to passage of the pile flange. The vibration peak particle velocity measured at the footings ranged from .06 to .12 inch per second. The ground anchors were inclined at 15 degrees below horizontal. The free (no bond) zone was 20 feet long and the bond zone was 25.5 feet long (Figure 1). The design load of the anchors was 105 kips. The anchors had single level corrosion protection, i.e. the grout was the only corrosion protection. The anchor rod was protected from corrosion in the bond zone by grout encapsulation. The free zone was protected by a grease-filled PVC sheath.

The ground anchors were installed with a rotary air percussion drill. In most cases, the 5-1/2 inch diameter casing was driven with a conical tip. The casing was then disengaged from the tip and extracted by a hydraulic jack during the grouting operation. In some cases, the casing was driven open-ended and drilled out.

Dywidag thread bars, 1-3/8 inch in diameter, were placed in the casing. (The contractor had the option of using steel strands or steel bars.) No coupler connections were needed for the bars. A 1/2 inch PVC sheath tubing was attached to the bars for secondary grouting. Two holes drilled in the tubing and covered with a rubber compound performed as valves in the bond zone. A 20-foot long PVC sheath was placed around the bar in the free zone to prevent bonding between the grout and the bar.

The grout was mixed at a ratio of 7 gallons of water per bag of grout. The grout was pumped through the grout tube at 100 psi pressure as the casing was extracted. After 24 hours, an attempt was made to inject additional grout at 500 to 600 psi. Only 39 percent of the anchors accepted secondary grouting. The volume for secondary grouting ranged from 10 to 50 gallons. Some of the grout tubes were damaged during installation or blocked by concrete shards inadvertently allowed into the tube.

The first four anchors were performance tested to 150 percent of the design load. The remaining anchors were proof tested to 150 percent of the design load. All of the anchors were tested from 3 to 14 days after primary grouting. All anchors passed their test on the first try. Due to problems with the load cells, the test loads and the lockoff loads were based on the "calibrated" jack readings.

The timber lagging was a dense Douglas Fir which was pressure treated with creosote. The lagging was cut in the field to fit between the piles. The ends were field treated with creosote. The excavation was not allowed to proceed beyond 18 inches below the bottom of previously placed lagging before placing a new lag.

In late August, a severe rainstorm struck the project. Water funneled through the deck holes, washed material from behind the retaining wall. This resulted in 1 to 2-1/2 inches of settlement in the overpass footings. The settlement occurred when the excavation was only 6 feet below the footing prior to the installation of the ground anchors. To prevent additional settlement, grout was pumped behind the walls to fill the voids. This grouting operation continued as the excavation and wall installation proceeded.

Precast concrete panels were attached by four hangers welded to the soldier piles. The panels were difficult to install due to the small hangers, uneven deflection in piles from the anchor loads and insufficient head room to use a boom to lift the heavy panels. The project personnel suggested that an adjustable double angle hanger would ease the installation.

The average total cost of the retaining wall without the precast concrete wall panels was approximately $69 per square foot. The precast panels for the exposed portion of the retaining wall cost an additional $18 per square foot installed.

The contract prices were:

Item	Quantity	Unit Price	Amount
Wall Anchors	38 ea.	$2,500	$ 95,000
Deck Holes	26 ea.	2,000	52,000
Precast Wall Panels	4,248 sf.	15	63,270
Aesthetic Facia	4,248 sf.	3	12,744
Steel Piles	1,944 lf.	100	194,400
Treated Timber	48.74 MBM	1,500	73,110
		TOTAL	$490,524

IV. RETAINING WALL INSTRUMENTATION

A SINCO Model 513510 electric load cell was installed at the head of one anchor on each wall. A SINCO Model 51309 electric load cell readout device was purchased to monitor the load cells.

Vibrating wire strain gauges (SINCO Model 52621 with pickup sensor Model 52622) were installed to measure load transfer along the anchor shaft (bond zone) and to measure strain in the no load zone. Three sets of two gauges each were installed on the two anchors, one set near the anchor head; two sets approximately 4.9 feet and 13.2 feet from the end of the bond zone. A vibrating wire strain indicator readout divide (SINCO Model 52669) was used to monitor the strain gauges. Of the 12 strain gauges installed only the two strain gauges located in the middle of the bond zone on Anchor #16 failed to work.

The proposed slope inclinometer casing could not be installed since the soldier piles were driven and not placed in pre-bored holes.

V. DATA ANALYSIS

Based on the data collected during the first year after installation, the following observations can be made:

1. There is a 10-15 kip difference in the measured load between the load cell and the strain gauges in the no bond zone on Anchor #16. This may be due to either miscalibration of the strain gauges, bearing plates adjacent to the load cell may be too thin to transfer load without bending, or the grout has extended into the no bond zone which permits some load transfer to the no bond zone.

2. Anchor #35 was originally tensioned to 105 kips on 9/11/85 and de-tensioned on the same day. The anchor was tensioned again to 105 kips on 9/12/86; however, six days later the anchor head was damaged. The anchor was repaired and re-tensioned a third time. The initial strain gauge readings prior to anchor stressing were different each time. The lowest set (second) of readings were used as initial readings for this report.

3. The load cell and strain gauges in the unbonded zone on Anchor #16 indicate an apparent load reduction rate of 5 to 6 kips per log cycle of time from the period to 100 days to 300 days. This equates to a load reduction of 10 to 12 kips in thirty years. The load cell and strain gauges in the unbonded zone of Anchor #35 indicate an apparent load reduction of 1.25 to 1.4 kips per log cycle time.

   The load reduction may be caused by movement of the wall into the backslope, due to the applied anchor load exceeding the at-rest pressure behind the wall. If this is the case, the load reduction should cease when the anchor load equals the at-rest pressure.

4. The difference in anchor load measured by the load cells and strain gauges with the load measured by the "calibrated" jack may be due to inaccurate load measurement of the anchor loads by the jack.

5. The strain gauges located 4.9 feet into the bond zone indicate a load reduction while the strain gauges located 13.2 feet into the bond zone indicates an increase in load. This is due to load transfer down the bond zone over time.

VI. ADDITIONAL WORK

The following tasks will be performed during the next two years:

1. Continue monitoring the load cells and strain gauges for the next two years.

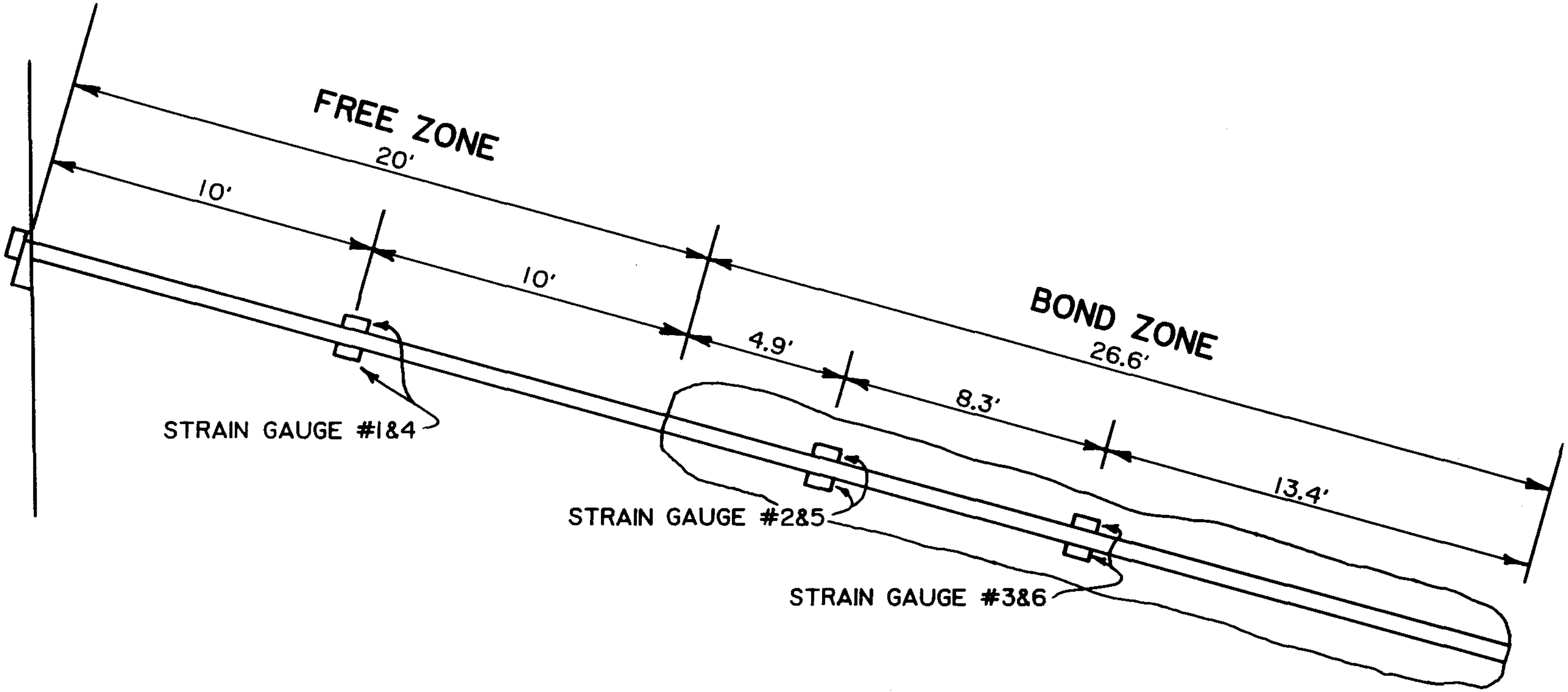
2. Retension the two instrumented ground anchors to determine the cause for the difference in load measured between the strain gauges in the no bond zone and the load cell.

3. Establish and monitor survey control points on the soldier piles to determine lateral movement.

4. Reestablish and monitor survey control points on the overpass structure to measure settlement.

5. Prepare final report to include data interpretation, conclusions, and recommendations.

PERMANENT GROUND ANCHOR



STATE OF ALASKA

DEPARTMENT OF TRANSPORTATION
AND
PUBLIC FACILITIES

PERMANENT GROUND ANCHOR
DIMOND BLVD

Fig. 1

ANCHOR #16 (NORTH)

STRAIN GUAGE #1 & #4

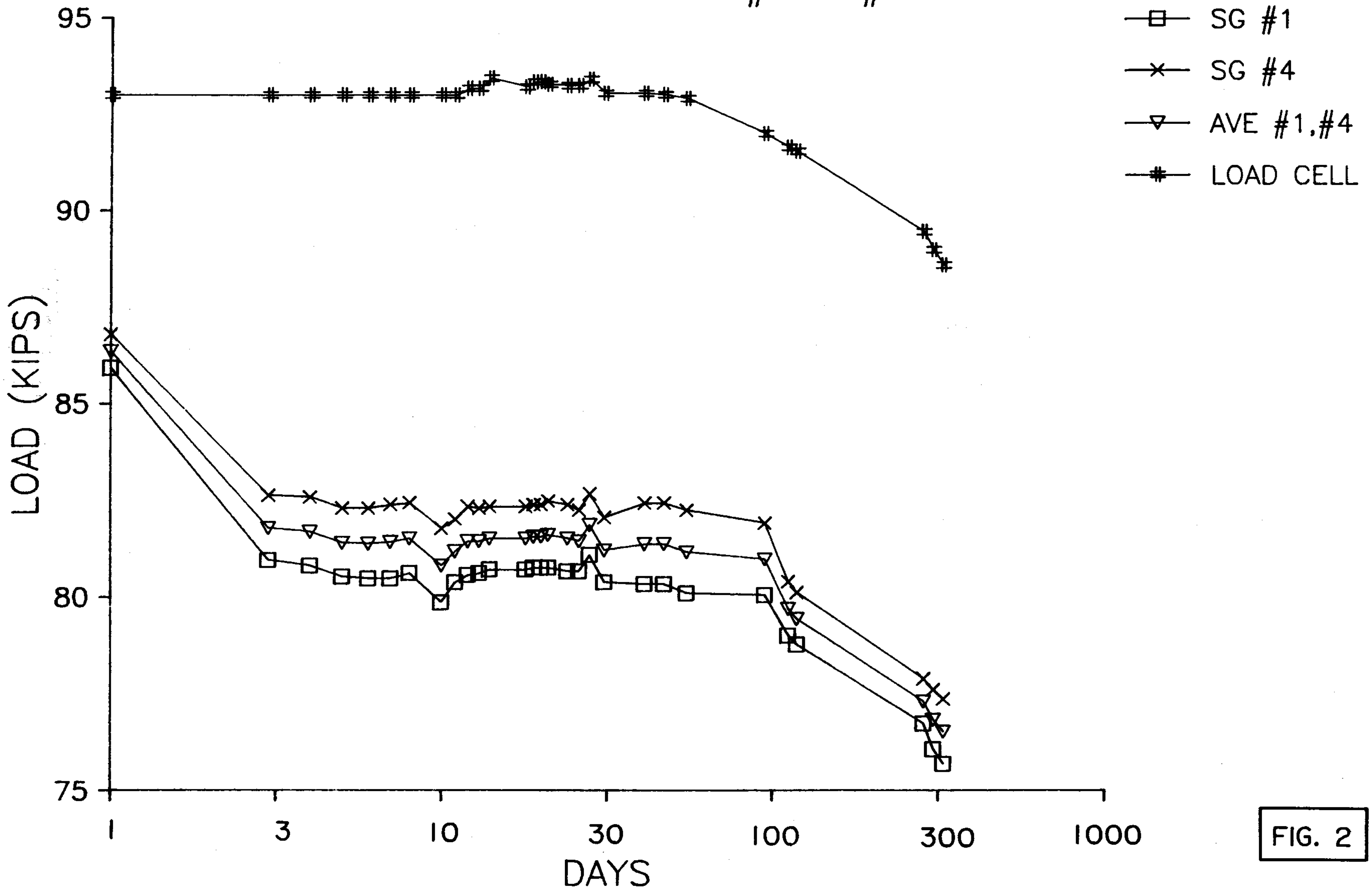


FIG. 2

ANCHOR #16 (NORTH)
STRAIN GUAGE #2 & #5

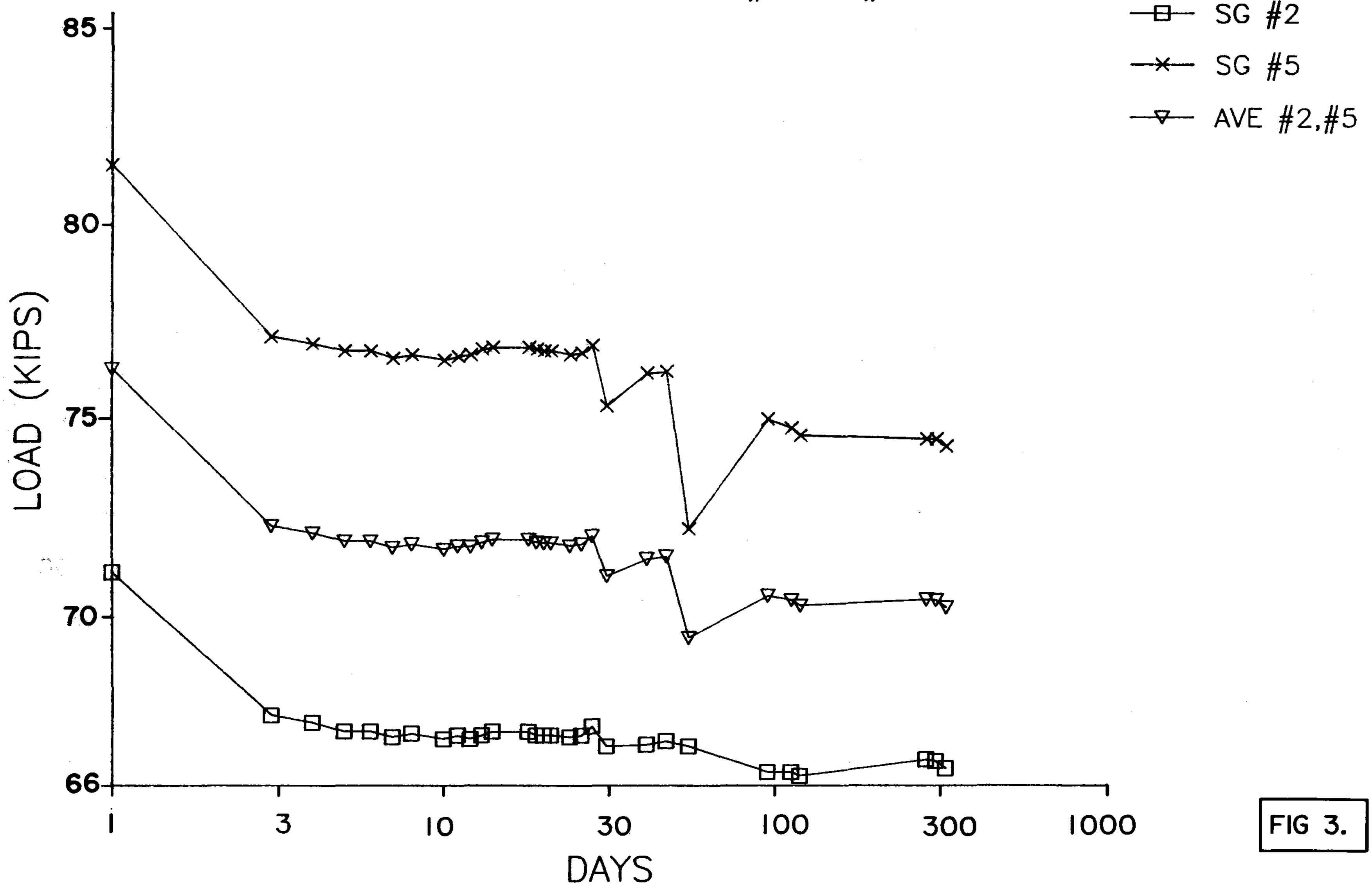


FIG 3.

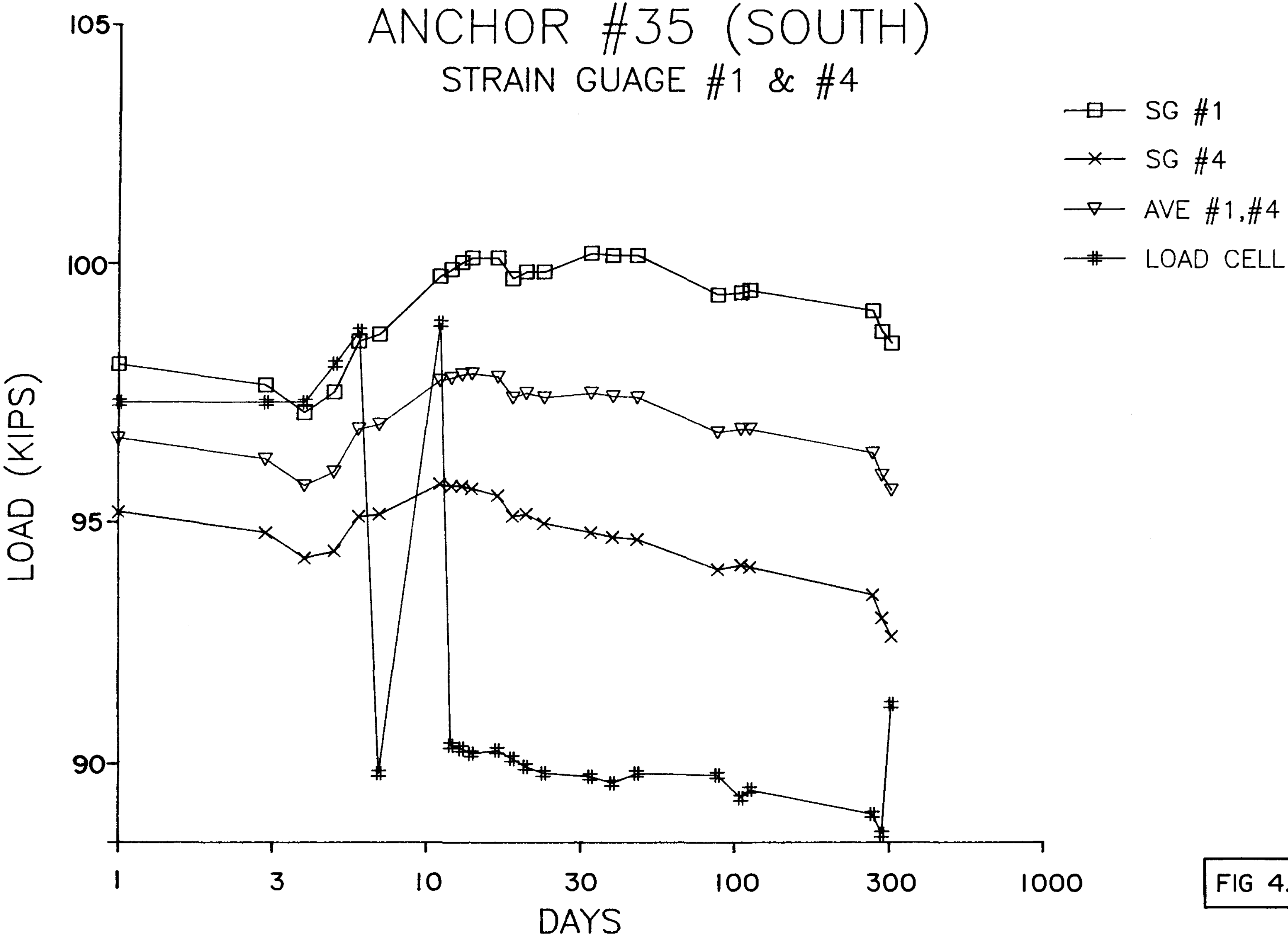

ANCHOR #35 (SOUTH)
STRAIN GUAGE #1 & #4
SG #1
SG #4
AVE #1,#4
LOAD CELL
LOAD (KIPS)
105
100
95
90
DAYS
1
3
10
30
100
300
1000

FIG 4.

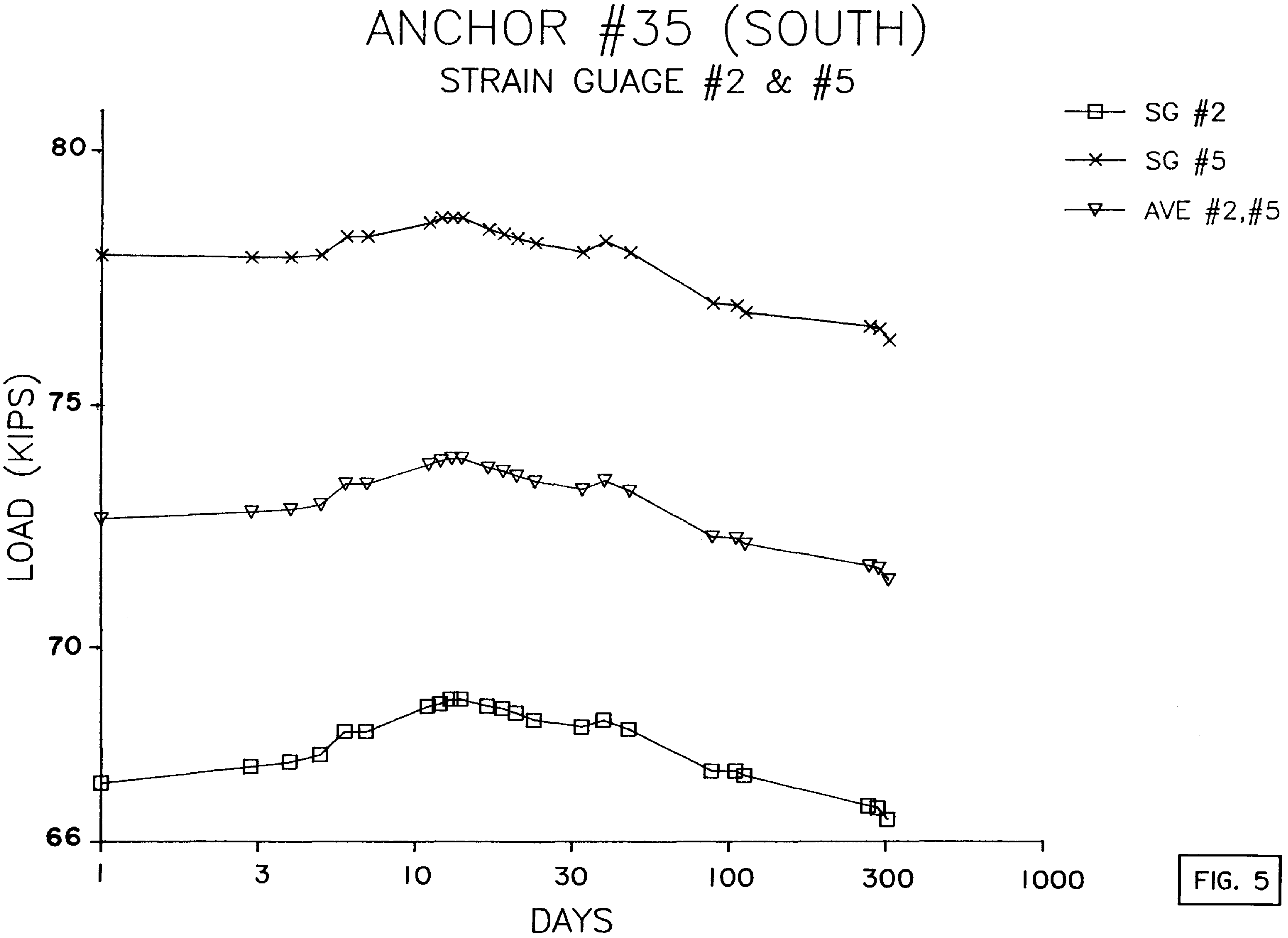

ANCHOR #35 (SOUTH)
STRAIN GUAGE #2 & #5
SG #2
SG #5
AVE #2,#5
LOAD (KIPS)
80
75
70
66
1
3
10
30
100
300
1000
DAYS

FIG. 5

5-A-5

ANCHOR #35 (SOUTH)

STRAIN GUAGE #3 & #6

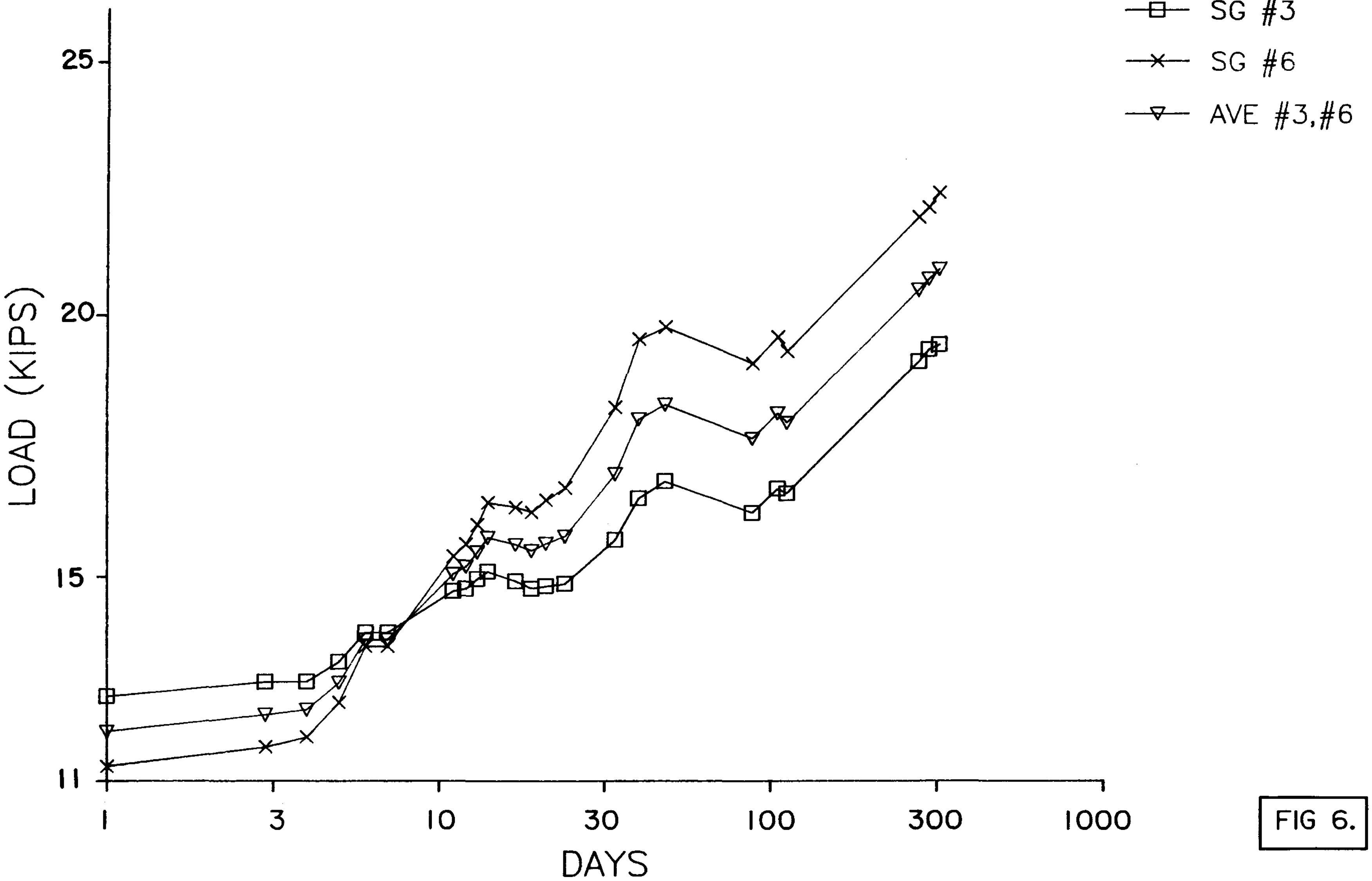


FIG 6.

5-A-6

LOAD TRANSFER
ANCHOR #16

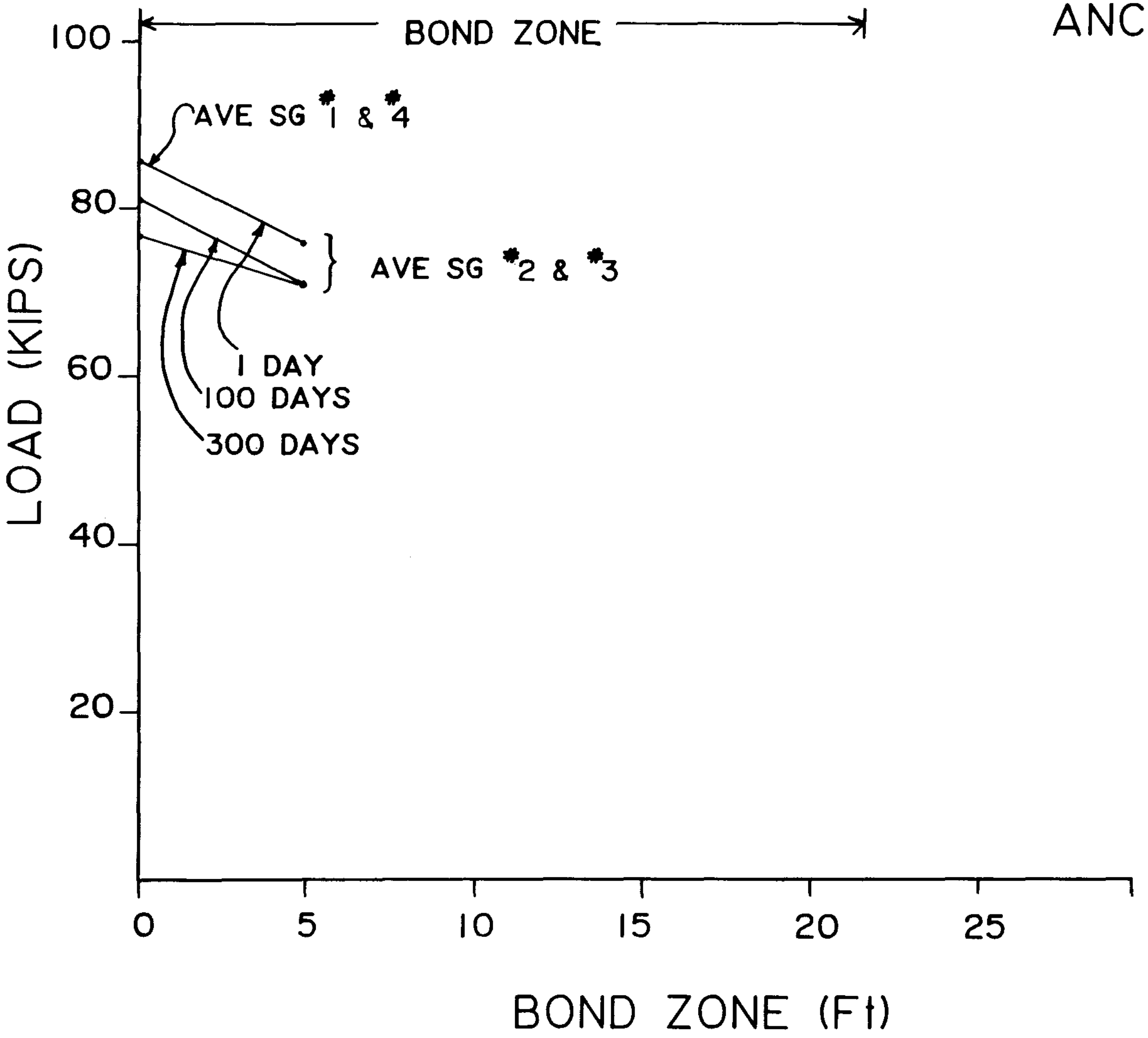


FIG 7.

LOAD TRANSFER
ANCHOR # 35

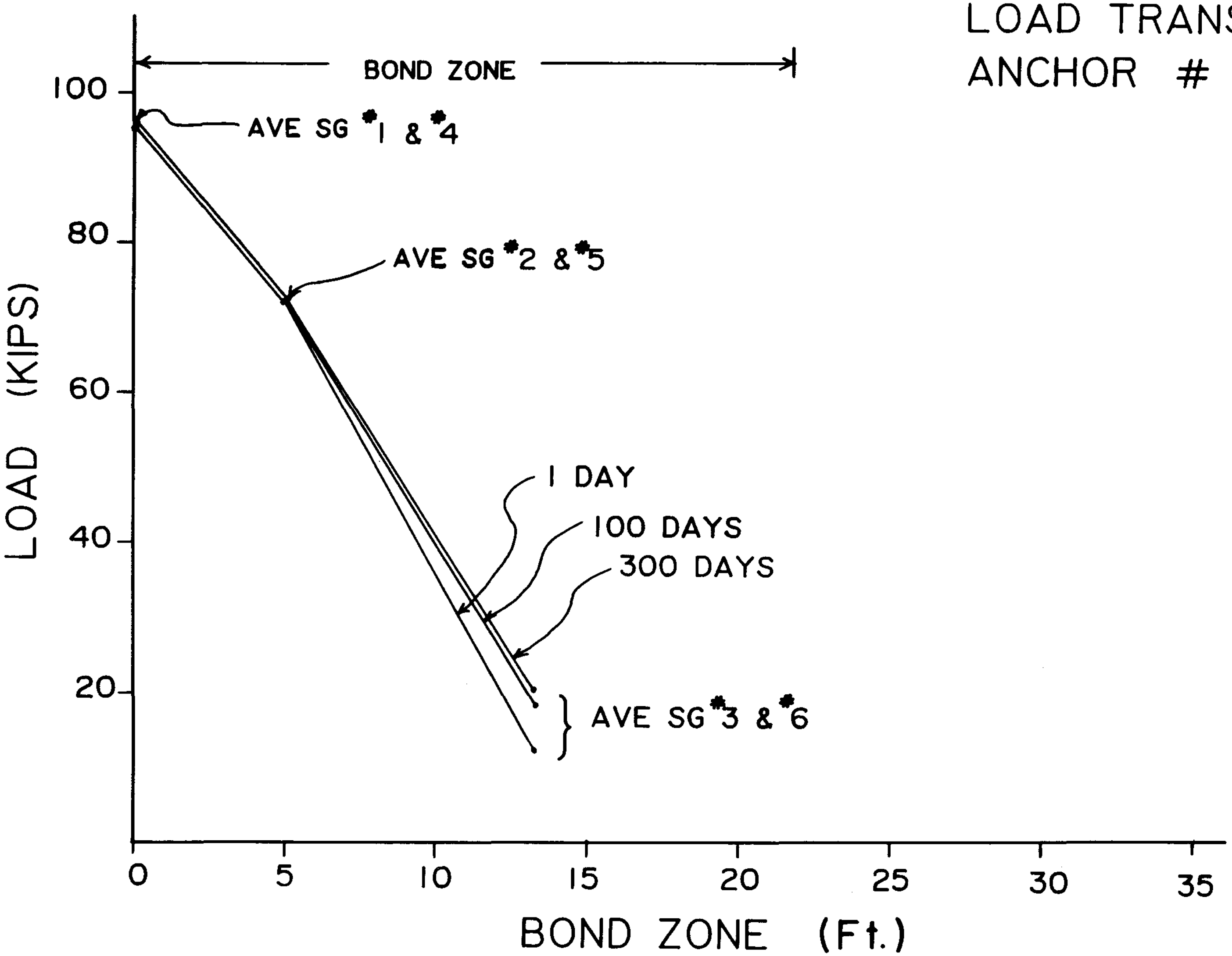


FIG 8.