Research Report
UKTRP-88-15

SOIL-BRIDGE ABUTMENT INTERACTION

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Research Investigator

and

David L. Allen
Chief Research Engineer

August 1988

Kentucky Transportation Research Program
College of Engineering • University of Kentucky
Transportation Research Building
Lexington, Kentucky 40506-0043

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in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

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16. Abstract

Presented herein are analyses of 6 bridge approaches. Lateral movement and settlement of the foundations and embankments are discussed. Results of slope stability and finite element analyses are related to measured movements. A graph for predicting approach pavement settlement is included.

A theoretical approach model was used to run an extensive series of finite element analyses. Movements resulting from these analyses are presented with several variable model conditions. Variable conditions include embankment and foundation configuration, soil cohesion, and soil friction angle.

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COMMONWEALTH OF KENTUCKY
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MILO D. BRYANT
SECRETARY
AND
COMMISSIONER OF HIGHWAYS

WALLACE G. WILKINSON
GOVERNOR

March 2, 1989

Mr. Robert E. Johnson
Division Administrator
Federal Highway Administration
330 West Broadway
Frankfort, Kentucky 40602

Subject: IMPLEMENTATION STATEMENT
KYHPR 72-69, "Soil Bridge Abutment Interaction"

Dear Mr. Johnson:

Data from this and other studies show the necessity of paying special attention to bridge approaches. Already implemented as a result of these studies are: heavier compaction for embankments, select granular backfill for abutments (Standard Drawings RGX-100-01 and RGX-105-01), and wick drains for foundations.

Also implemented as a result of this study are heavily reinforced bridge approach slabs and a method for calculating permissible horizontal resistance per pile. This method is based upon the amount of cohesion the soil has and is found as a Revised Item in the Division of Bridges Guidance Manual, Section 66-05.0045 and Exhibit No. 66-05-02.

Charts in this report will be used to estimate settlement and lateral squeeze.

Sincerely,

O. G. Newman, P. E.
State Highway Engineer

PREVIOUSLY ISSUED REPORTS

UKTRP NO.	TITLE	DATE
84-33	"A Review and Analysis of Pile Design"	December 1984
85-10	"Analysis of Movements and Forces on Bridge Approaches: A Case Study (Bridge over Chesapeake Avenue on Interstate 471 in Campbell County, Kentucky"	April 1985
85-25	"A Survey of the States on Problems Related to Bridge Approaches"	October 1985

REPORTS TO BE ISSUED

NONE

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Figure 91. Near Surface Settlement (2-3 Feet) of Embankment (190 Feet) in Laurel County, Kentucky.

INTRODUCTION

Settlement of bridge approaches and movements of bridge abutments has been a major problem for designers and maintenance personnel. Allen reported that 27 states consider this a major problem(1). Several bridge approaches in Kentucky have been reconstructed in attempts to correct problems associated with continuous movements. One bridge was lengthened due to the severity of settlement and movement of an abutment. Patching is a continuing maintenance expense at nearly all bridge approaches. In January 1964, the Kentucky Department of Highways initiated a study entitled "Settlement of Highway Bridge Approaches and Embankment Foundations." That study concentrated largely on settlement and results were reported by Hopkins(2). Partly as a result of that study, it was decided to study all the forces and movements at bridge approaches and abutments.

Five of the six case histories reported by Hopkins have been reanalyzed for study. One additional bridge site was instrumented under this study and results were reported by Allen et al (3). The objectives of this study were:

1. provide an experimental analysis of lateral forces exerted upon piling used for support of a bridge end-bent,

2. analyze present design procedures related to lateral loads on piles and recommend changes if necessary,

3. measure magnitude of settlements in fill and foundation,

4. measure and analyze forces exerted on the end-bent, and translational and rotational movement of the end-bent, and

5. measure downdrag forces on piles.

Objectives 1, 3, and 4 were analyzed and reported by Allen et al (3). Objective 2 was studied and reported by Allen (4). Objective 5 was not accomplished because of time and funding restrictions. The purpose of this final report is to analyze and summarize information gained from this study and the information reported by Hopkins (2). Conclusions and recommendations are included herein.

CASE HISTORIES

The case histories reanalyzed under this study follow. A brief summary is given for each site; however, detailed information is contained in the previously cited references (2, 3). Available site geology, instrumentation, geotechnical information and observed earth movements are included for each case history. These sites are:

Site 1 - KY 4 over Parkers Mill Road in Fayette County, Kentucky.

Site 2 - I 64 over Bull Fork Creek in Rowan County, Kentucky.

Site 3 - I 71 over Kentucky River in Carroll County, Kentucky.

Site 4 - I 64 over Slate Creek in Bath County, Kentucky.

Site 5 - I 471 over Chesapeake Avenue in Campbell County, Kentucky and

Site 6 - I 24 over Eddy Creek in Lyon County, Kentucky.

Those locations are shown in Figure 1. Abutments at each of those sites are founded on point bearing piles driven to rock.

KY 4 over Parkers Mill Road

The KY 4 Parkers Mill overpass is located approximately two miles southwest of downtown Lexington on the Lexington Peneplain (Figure 2). At this site, the approach embankments are approximately 20 feet in height and the foundation depth is approximately 12 feet. The embankment and foundation soils are silty clays developed primarily from limestone and shales. Those soils are relatively plastic but, due to bedrock conditions and fragmentary structure, they are highly permeable.

Soil samples were obtained approximately seven years after completion of construction. Consolidated-undrained triaxial tests with pore pressure measurements were performed. The effective stress parameters o' and c' of the embankment soils were 28.5 degrees and 606 pounds per square foot, respectively. The effective stress parameters o' and c' of the foundation soils were 28.8 degrees and 214 pounds per square foot, respectively.

Embankment construction began in May 1966, and was completed in September 1966. The embankment material was placed in one-foot lifts with good compaction. Approach pavements were placed in November 1966. One single-point settlement gage was installed on the foundation of the northwestern approach prior to embankment placement (Figure 3).

Foundation settlement after approximately 800 days was one inch with 80 percent of the settlement occurring by the time the approach pavement was constructed (Figure 4). Approach pavement settlement was 0.3 to 0.5 inch with a settlement cradle from 75 to 100 feet (Figures 5 and 6).

No significant embankment erosion was visible at the site.

I 64 over Bull Fork Creek

I 64 crosses Bull Fork Creek and Bull Fork Road on twin bridges in Rowan County (Figures 7 and 8). Construction of the western embankment began in January 1967 and was completed in July 1967. Construction of the eastern embankment began in August 1967 and was completed in October 1967. Approach pavements were constructed in October 1968.

The eastern and western approach embankments are 75 and 65 feet high, respectively. The approach foundations are approximately 18 feet thick. Foundation material is mostly alluvium with soft locations. Average natural moisture content of the foundation was 30.0 percent. The average liquid limit was 30.0 percent and the plasticity index was 6.8 percent.

Embankment material was generally a greenish shale and sandstone obtained from the Waverly Formation. Natural moisture content of the embankment ranged from 8 to 40 percent. The liquid limit ranged from 25 to 31 percent and the plasticity index ranged from 4 to 11 percent. Consolidated-undrained triaxial tests with pore pressure measurement indicated the effective stress parameters ϕ' and c' were 29.8 degrees and 69 pounds per square foot, respectively. The embankment was placed in 2- to 3-foot lifts with poor compactive effort.

Instrumentation at this site (Figures 9 and 10) included a settlement gage on the foundation of each approach, three piezometers in the western foundation, and a slope inclinometer at the top of the eastern embankment slope. The slope inclinometer was installed in 1970, about three years after embankment construction was completed. Points for monitoring movement of the foundation at the toe of the embankment were established prior to the beginning of construction.

The western and eastern approach foundations settled 10.0 and 17.0 inches, respectively (Figures 11 and 12). In both cases, virtually all of the settlement had occurred by the time embankment construction was complete. Monitoring the points established at the toe of the embankment revealed no movement during or after construction.

The eastern approach pavement settled 4.9 inches in 3.1 years with the settlement cradle extending 295 feet from the bridge (Figures 13, 14, 15, and 16). Pavement settlement of the western approach was 3.5 inches in 3.1 years with a settlement cradle of approximately 200 feet (Figures 17, 18, 19, and 20). Pavement settlement at this site is apparently the result of embankment subsidence.

As noted earlier, there was very little foundation movement at the toe of the embankment but the slope inclinometer installed two years later revealed significant lateral movement of the embankment. Lateral movement between 1970 and 1980 near the bottom of the embankment was 1.5 inches. Lateral movement near the top of the embankment was roughly 7.0 inches (Figure 21). Within 2 to 4 feet of the surface, lateral movement was as great as 10 inches.

Pore pressure measurements in the western approach foundation indicated pressure increasing to near critical levels as embankment elevations increased. Significant embankment erosion near the abutment has occurred.

I 71 over the Kentucky River.

I 71 crosses the Kentucky River on twin bridges in Carroll County. (Figures 22 and 23). The southwestern approach embankment was begun on August 26, 1966 and completed in October 1966. The southwestern approach has a foundation depth of approximately 100 feet and an embankment height of 35 to 55 feet. The approach pavement was placed in September 1968.

The foundation material is composed of alluvial deposits of clay, sand, and silts. Natural water contents ranged from 17.5 to 31 percent. Liquid limits ranged from 23 to 36 percent for the more clayey zones and the sandier zones ranged from non-plastic to 20 percent. Plasticity indices ranged from non-plastic to 14 percent overall. Triaxial compression tests indicate an average friction angle of 28 degrees and degrees and an average cohesion of 187 pounds per square foot. The average unconfined compressive strength was approximately 2,300 pounds per square foot.

Soil samples were obtained in 1970 while drilling for a slope inclinometer installation. Consolidated-undrained triaxial tests performed on those samples indicated effective stress parameters Ø' and c' of the embankment soil to be 24.8 degrees and 250 pounds per square foot, respectively. Embankment and foundation configuration and soil properties are shown in Figure 24. Embankment material was placed in one foot lifts with medium to good compaction.

Instrumentation included six piezometers at various locations, two settlement gages and two slope inclinometers (Figure 24). The piezometers and settlement gages were installed in and on the foundation during construction. Slope inclinometers were installed in 1970, approximately four years after the embankment was completed.

As seen in Figure 24, the embankment cross section is somewhat irregular. Foundation settlement under the higher fill (55') reached approximately 23 inches (Figure 25) while settlement under the lower fill reached approximately 19 inches (Figure 26). At both locations, nearly 85 percent of total measured foundation settlement had occurred by the time the embankment was complete.

Pavement settlement was monitored from October 1968 to June 1972. Pavement settlement over the more shallow fill was approximately 1.5 inches with the settlement cradle extending 310 feet (Figures 27 and 28). Pavement settlement over the higher fill was 2.8 inches and the settlement cradle was 300 feet long (Figures 29 and 30). Projected foundation settlement, when compared to measured pavement settlement, would indicate that most pavement settlement is due to secondary consolidation at the foundation.

Two slope inclinometers were installed in the southwest approach in July of 1970, some four years after completion of the embankment. Both inclinometers were installed near the bridge abutments with inclinometer 1 located near the westbound lanes and inclinometer 2 located near the eastbound lanes. Slope inclinometer data indicate no significant lateral movement of the approach since the inclinometers were installed. During the 14 years of monitoring, lateral movement was generally less than one inch (Figures 31 and 32), except at the top two to four feet where up to two inches of movement occurred.

From stability analyses, it was determined that a pore pressure increase equivalent to a static water table elevation of 466 feet would be the maximum acceptable pressure increase. Monitoring of the piezometers indicated a pressure increase at piezometer 6 equivalent to a water table elevation of 468 feet. Construction was halted until the pore pressure decreased.

I 64 - Slate Creek

I 64 crosses Slate Creek and Kendall Springs Road over twin bridges in Bath County (Figures 33 and 34). The eastern approach embankment is 55 feet high and the approach foundation is approximately 12 feet thick. The embankment was constructed of greenish nondurable shale with limestone rock. Lift thicknesses were two to three feet with poor compaction.

Foundation material is an alluvial sandy silty clay. Natural moisture contents ranged from 22.6 to 24.1 percent. The friction angle for this material was 28 degrees and cohesion was 0. Liquid limits ranged from 27 to 34.5 percent and plasticity indices ranged from 6.3 to 9.2 percent. Those soils classified as A-4 by the AASHTO System and M2 by the Unified System.

Construction of the embankment began on November 23, 1965 and was completed in June of 1966. The approach pavement was placed in July of 1967. A single point settlement gage was placed on the foundation of the eastern approach. In August 1970, a slope inclinometer was installed between the abutments of the eastern embankment, Station 1622+80. Instrumentation locations are shown in Figure 35.

The settlement gage was only sporadically readable. Settlement data indicate a final foundation settlement of 4 inches with virtually all of that occurring by the time the embankment was complete (Figure 36). By May 1970, the east approach pavement had been raised six to seven inches by mudjacking.

Approach pavements were monitored from August 1967 to May 1970. The eastbound pavement (Figure 37) settled a maximum of 4.8 inches at the shoulder or outside edge of the pavement. The settlement cradle was 200 feet. Settlement at the centerline was 3.0 inches with a cradle of 75 feet. At the median or inside pavement edge, settlement was 2.4 inches and the cradle was 100 feet. The westbound approach pavement settled approximately 2.0 inches across the pavement with the cradles being 100 feet (Figure 38).

The slope inclinometer installed in 1970 was monitored through 1979. Maximum lateral movement during that time was 1.4 inches (Figure 39). There is visual evidence of significant erosion of the embankment near the abutment at this site.

I 471 over Chesapeake Avenue

I 471 crosses Chesapeake Avenue in Campbell County (Figure 40). At this site, the southern bridge approach was instrumented and monitored. The approach fills are 45 feet and 60 feet high for the south and north approaches, respectively. The foundation for the south approach is irregular and on a steeply sloping rock line. Near the location of the south end-bent, the foundation depth is approximately 30 feet (Figure 41).

Bedrock at this site is the Kope Formation of the Eden Group. This formation is comprised of 75 to 80 percent shale with interbedded limestone. This shale weathers rapidly and is known as having poor engineering performance. Samples obtained from this foundation material indicated natural moisture contents ranging from 15 to 30 percent. Liquid limits ranged from 22 to 54 percent, and the plasticity indices ranged from 2.4 to 29 percent. This material is generally classified as CL with an internal friction angle of 26.0 degrees and cohesion of 288 pounds per square foot.

Construction of the south approach embankment began in June 1978. In September 1978, the south embankment failed and the north embankment was begun in an attempt to stabilize the failure. Both embankments were completed by October 1979. Embankment material was placed in 2- to 3-foot lifts with poor compaction. Approach pavements were placed in mid 1981.

Instrumentation at this site included settlement gages placed on the foundation and at various elevations in the embankment. Slope inclinometers were placed around the toe of the embankment, through the embankment, and on the end-bent piling. A settlement platform was placed on the foundation and earth pressure meters were placed on the end-bent faces. Approach pavement elevations were also monitored for settlement. Instrumentation locations are shown in Figures 42 and 43.

Foundation settlement (Gage 1) was approximately 11.5 inches in April 1980 with the rate of settlement slowing at that time (Figure 44). Gage s ,approximately 28 feet higher in the embankment, indicated settlement of 16 inches with 11 to 12 inches occurring by April 1980 (Figure 45). The top level of gages (Gages 7 and 8), approximately 40 feet above the foundation, settled 14 to 16 inches with approximately 8 inches occurring in the material (foundation and embankment) below Gage 3 (Figure 46).

The settlement platform indicated settlement of 9.7 inches in April 1980 which is nearly identical to a nearby point on the foundation settlement gage (Figure 47).

Slope inclinometer data indicate maximum movement at the toe of embankment of 1.4 inches with most of the movement within 5 feet of the surface. Lateral movement at the top of the slope was 2.5 inches with the entire embankment moving to some extent. Slope inclinometer data are shown in Figures 48 through 52. Locations of those inclinometers are shown in Figure 42.

Earth pressure meter data suggest a possible end-bent rotation due to embankment movement toward the bridge. Slope inclinometer data tend to substantiate that possibility. Pressure distribution, as indicated by pressure meter data, on the vertical face of the abutment is shown in Figure 53.

The approach pavement settled approximately one foot from September 1981 through June 1986. The settlement cradle was approximately 100 feet long (Figure 54). The south approach was mudjacked approximately two years after the road was opened to traffic.

I 24 over Eddy Creek

The bridge approaches on I 24 over Eddy Creek in Lyon County were instrumented and monitored (Figure 55). That site differs from the previously discussed sites in that the approach pavements were placed approximately 8 years after the embankments were complete and no pavement settlement data are available. Foundation depths range from 20.0 to 40.0 feet and is a clayey - silty alluvial deposit. Natural moisture contents of the foundation material ranged from 17 to 63 percent. The average effective friction angle, 0', was 31 degrees, and the average effective cohesion, c', was 120 pounds per square foot.

Embankment height was 35 feet (Figure 56).

Embankment construction was begun in May 1968 and both approaches were complete by December 1968. Approach pavements were not placed until 1976.

Settlement platforms placed on the foundation indicated total foundation settlement over a three-year period to be 18.0 to 19.0 inches with more than 90 percent occurring by the time the embankment was complete. Visual observation in 1985 indicated that pavement settlement was slight, probably less than 0.5 inch.

STABILITY ANALYSES

Slope stability analyses were conducted for each of the discussed sites. The analyses were perfomed using HOPK-1 (5), a stability model and computer program developed by Hopkins. Post construction conditions and soil parameters were used for the analyses. Safety factors ranged from 1.12 to 2.80 and are listed in Table 1. The safety factor of 1.18 at I 471 over Chesapeake Avenue was for conditions after stabilization of the failure. At the time the analysis was conducted, the water table was rising significantly. If the rise continued at the rate indicated by the most recent data, the safety factor is approaching 1.00. This is a possible explanation of the extreme pavement movements occurring at that site. Stability analyses sections for all sites are shown in Figures 57 through 62.

Five of the sites, Bull Fork Creek, Slate Creek, Kentucky River 34.5 feet and 55 feet fills, and Parkers Mill Road were analyzed for varied soil strength parameters. Soil weight per unit volume (density) was chosen to be the controlling variable. It was chosen because density is easily monitored and controlled in the field. The cohesion versus wet density of a typical shale, Crab Orchard, was determined from data published by Hopkins (6). Data for cohesion of 550 to 1,200 pounds per square foot were available. Data above and below these values were extrapolated and are shown in Figure 63. Safety factors for the various sites and cohesion values are shown in Figure 64. The safety factor at all sites increased with an increase of cohesion with the sharpest safety factor increase being from a cohesion of 0 psf to a cohesion of 500 psf. At 0 psf, cohesion safety factors ranged from 1.05 to 1.37. At 500 psf, cohesion safety factors ranged from 1.55 to 2.44. This illustrates the need for good compaction during embankment construction.

FINITE ELEMENT ANALYSES

Several analyses were performed on the actual study sites and a variety of theoretical models. A finite element model was used for the analyses. The finite element program used, ISBILD, was developed by Ozawa and Duncan (7). The objective was to compare measured and predicted movements and to develop a method of predicting embankment movement. This, in turn, would facilitate prediction of pavement settlement for use in future bridge and bridge approach design.

Six study sites were modeled and analyzed with finite elements. Several theoretical embankment models were analyzed. In the theoretical models, the foundation depth, embankment height, percent slope, and soil strength parameters were varied. Actual site models were analyzed for existing conditions, as well as could be determined, and results were compared to measured soil movements. Actual site models were also run with varied soil strength parameters in an effort to determine conditions at which unacceptable embankment movements would occur. Soil strength parameters were varied in the same manner as described in the Stability Analyses section, where a series of density values are assumed and other parameters are related to density. While model element stresses are an output of ISBILD, the primary interest was directed toward element strain. Strain was represented by x (horizontal), y (vertical), and total (resultant) component.

Actual site models and existing conditions are presented in Appendix along with the final strain conditions of the model components. ISBILD does not consider long term creep or secondary consolidation; therefore, elements at the top of the model do not indicate strain. This is obviously not the case and ISBILD is not a practical tool for directly predicting surface movement.

Because it is important to evaluate the various elements contributing to surface movement, ISBILD predicted vertical strain at the foundation - embankment interface. That strain was compared to observed settlement at similar locations. These data are shown in Table 1 and are graphically displayed in Figure 65. In four of six cases, ISBILD predicted more settlement than was measured in the field. ISBILD strain values for the series of actual site models are plotted in Figures 66 through 75.

Series of theoretical models for varied foundation depths, fill heights, percent slope, and soil strength parameters were analyzed. Soil strength parameters were varied as described in the Stability Analyses section. Seventy five sets of conditions were analyzed. In Table 2, the variables and constants for each analysis are listed. Initial analyses were conducted on full and symmetrical approach cross sections. Strain on each side of centerline was equal; therefore, a half cross section was used to reduce computer time. The finite element grid used for all theoretical models is shown in Figure 76.

Embankment Strain

Maximum vertical strains (settlements) for a series of theoretical embankments are shown in Figure 77. These models use constant soil strength parameters (foundation cohesion equals 250 pounds per square foot and friction angle equals 24 degrees). The embankment cohesion equals 900 pounds per square foot and the friction angle equals 29 degrees. The variables in this series are foundation depth (5, 10, 15, 20, and 30 feet), embankment height (40, 100, and 200 feet), and side slope (2:1, 2.5:1, and 3:1). Embankment settlement increases with increasing foundation depths and with increasing slope steepness, but the more significant settlement is related to embankment height. Maximum settlement for embankments of 40 feet is approximately one third (.33) of the settlement for 100-foot embankments and approximately one-eighth (.125) of the settlement for 200-foot embankments. Maximum settlement consistently occurred near the top of the embankment at node 78 (see Figure 76).

Lateral strain for that series of models was not so clearly related to embankment height. The greatest lateral strain was on 200-foot embankments (2:1 and 2.5:1) but lateral strains for that series were scattered as shown in Figure 78. Maximum lateral strain for this series occurred near the toe of the slope at node 62 or 67 for 2:1 slopes, and at nodes 72 or 77, near midslope, for 2.5:1 and 3:1 slopes (see Figure 76).

Series of analyses were conducted with a constant foundation depth (20 feet) and embankment height (100 feet). In those series the embankment cohesion and soil unit weight were varied. Maximum vertical and lateral strains are shown in Figures 79 and 80 respectively. As shown in Figure 79, vertical strain decreases by approximately 25 percent as cohesion increases from 0 to 2,000 pounds per square foot. Lateral strain is more noticeably affected by changes in cohesion with strain for all slopes decreasing by approximately 62 percent over the range of cohesion change. On slopes of 2.5:1 and 3:1, most lateral strain occurred between 0 and 500 pounds per square foot. On the 2:1 slope, lateral strain decreased significantly up to a cohesion of 1,000 pounds per square foot.

Series of analyses were conducted with the friction angle and side slope as the variables and the constants being a foundation of 20 feet, embankment of 100 feet, and embankment cohesion of 900 pounds per square foot. The friction angle was varied from 21 to 33 degrees and the side slopes were 2:1, 2.5:1, and 3:1. Maximum vertical strain was not greatly affected by the side slope. Vertical strain for a 3:1 slope at any friction angle was approximately 84 percent of the strain for a 2:1 slope. Vertical strain decreased with increased friction angles with approximately 31 percent reduction in strain from friction angles of 21 to 33 degrees.

Lateral strain for any side slope decreased approximately 37 percent with an increase in friction angle from 21 to 33 degrees. The most striking change in lateral strain in this series is related to side slope. Lateral strain at any friction angle decreased by approximately 31 percent from a 2:1 slope to a 2.5:1 slope and by approximately 50 percent from a 2:1 slope to a 3:1 slope. Strain values for this series are shown in Figures 81 and 82.

Foundation Strain

A phenomenon of interest to approach designers is the lateral strain or lateral squeeze in the foundation. Lateral squeeze was computed in the analysis series with foundation depths, embankment heights, and side slopes as variables. In this series, the foundation soil strength parameters were constant at a cohesion of 250 pounds per square foot and a friction angle of 24 degrees. Finite element foundation strain data are probably more accurate than embankment data.

Lateral squeeze, in most cases, increases with increasing foundation depth, embankment height, or side slope. The exceptions to this are cases having very high embankments (200 feet) and shallow foundations (5 or 10 feet). In those cases, lateral squeeze decreased as foundation depth increased up to 10 or 15 feet in depth. Lateral squeeze ranged from 0.35 foot to 5.8 feet. High embankments (200 feet) having steep slopes (2:1 or 2.5:1) produced the greatest lateral squeeze (Figures 83, 84, and 85). In most cases, the maximum lateral squeeze occurred at the toe of the embankment (node 55) or at the adjacent node (node 56). See Figure 76 for node locations.

Foundation vertical strains, settlements, for this series of theoretical models are presented in Figures 86 through 88. Settlement generally increases with increasing foundation depth and embankment height. As seen in those figures, side slope has little effect on foundation settlement. In Figure 89, foundation settlements for the different foundations are plotted and a line fitted to the points representing each embankment height at each side slope.

SUMMARY OF FIELD DATA AND OBSERVATIONS

All sites, except I 24 over Eddy Creek, have required pavement maintenance in the form of patching and/or mud-jacking. At the I 24 over Eddy Creek site, the embankment was completed approximately 8 years prior to pavement construction. Pavement settlements, excluding the Eddy Creek site, ranged from 0.5 inch to approximately one foot. Pavement settlement at I 24 over Eddy Creek was negligible.

The primary factor contributing to pavement settlement appears to be embankment subsidence with secondary consolidation of the foundation and embankment erosion contributing to a lesser degree. A large part of embankment subsidence probably occurs as a result of poor compaction or too steep slopes. At sites where the lift thickness was 2 to 3 feet and compaction was not good, considerable pavement settlement occurred. Considering the height of the embankment alone does not provide a good indicator of pavement settlement. The embankment height along with the safety factor calculated from the slope stability analysis seem to provide a better settlement predictor. In Figure 90, the height of the embankment divided by the square of the safety factor is plotted versus pavement settlement for each site. As seen in that figure, a good correlation between pavement settlement and the variables exists. Data for one site, I 471 over Chesapeake Avenue, do not fit this relationship very well. The low safety factor at that site along with the previous failure and a water table that has been rising could produce a near failure condition.

An embankment phenomenon noted in an earlier report by Allen et al (8) possibly contributes significantly to pavement settlement. Embankments, especially higher ones, tend to slump with more of the movement occurring near the sloped surfaces. Slumping is illustrated by plotting data from a high embankment (190 feet). In Figure 91, settlement of a point near the embankment surface is plotted versus time. It may be seen that a large part of the slumping occurs in less than two years. In reviewing slope inclinometer and pavement settlement data from the bridge approach sites, it appears the slope inclinometers were not installed soon enough to reveal the initial slumping. At every site, most of the pavement settlement occurred less than two years after placement of the pavement. Nearly all slope inclinometer data indicate a near-surface movement.

The erosion most often contributing to pavement settlement probably occurs around the bridge abutment. There is apparently a tendency for surface water to drain along the abutment surfaces and remove embankment material. This contributes to embankment subsidence very near the abutment.

RECOMMENDATIONS

Future approach construction directed toward minimizing embankment subsidence should include:

1. Construction of thinner embankment layers or lifts. Compacted lift thickness for fine or coarse grained soils would range from 6 to 12 inches as defined in NAVFAC DM-7.2, May 1982 (9). Durable rock, excluding shale, should have a compacted lift thickness of 2 feet. Durable shale, SDI greater than 95 percent from KM-64-513(79), should have lifts not greater than 1.5 foot. Shales with SDI between 95 and 60 percent should have loose lift thicknesses not greater than 12 inches. Shales with SDI less than 60 percent should have loose lifts not greater than 8 inches (2).

2. More compactive effort during construction (2). Shales should be brought to optimum moisture (ASTM D 698), disked, and compacted with static and vibratory rollers to 98 percent maximum dry density (ASTM D 698).

3. Flattened slopes (3:1),

4. Surcharging the embankment, and

5. Select material and drainage facilities around the abutment, and allowing as much time as possible between completion of the embankment and placement of the pavement. Efforts to enhance compaction and configuration of the embankment would increase the safety factor which would decrease pavement settlement.

6. It is recommended that newly designed bridge approaches be designed with self-supporting approach slabs. One end of the slab would be anchored to the bridge and constructed in a manner that would permit rotation about an axis perpendicular to the bridge. The end of the slab farthest from the bridge would be constructed on a footer that could move as the embankment settled or slumped. These approach slabs are particularly important in the western portion of Kentucky where there is a high risk from earthquakes. Experience in California has shown many approach embankments settle during an earthquake. An approach slab similar to the one described above would remain in service if the embankment settled several inches.

Secondary consolidation of the foundation generally contributes a lesser, but significant, portion of pavement settlement. This portion of pavement settlement often requires 10 to 20 years to become insignificant. Sites where foundation settlement would contribute most to pavement settlement would be deep foundations of predominately clay with slow draining natures. These sites could be improved by use of sand drains or wick drains. Surcharging the embankment would also decrease the amount of foundation settlement occurring after placement of the pavement.

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1. Allen,D.L.; "A Survey of the States on Problems Related to Bridge Approaches," Research Report UKTRP-85-25, University of Kentucky Transportation Research Program, October 1985.

2. Hopkins,T.C.; "Long-Term Movements of Highway Bridge Approach Embankments and Pavements," Research Report UKTRP-85-12, University of Kentucky Transportation Research Program, April 1985.

3. Allen,D.L., Meade,B.W., and Hopkins,T.C.; "Analysis of Movements and Forces on Bridge Approaches: A Case Study (Bridge over Chesapeake Avenue on Interstate 471 in Campbell County, Kentucky), " Research Report UKTRP-85-10, University of Kentucky Transportation Research Program, April 1985.

4. Allen,D.L.; "A Review and Analysis of Pile Design," Research Report UKTRP-84-33, University of Kentucky Transportation Research Program, December 1984.

5. Hopkins,T.C.; "A Generalized Slope Stability Computer Program: User's Guide for HOPK-1," Research report UKTRP-86-2, University of Kentucky Transportation Research Program, July 1985.

6. Bishop,C.S., Armour,D.W.,Jr., and Hopkins,T.C.; "Design of Highway Embankments on Unstable Natural Slopes," Research Report UKTRP-86-22, University of Kentucky Transportation Research Program, September 1986.

7. Ozawa,Y., and Duncan,J.M.; "A Computer Program for Analysis of Static Stresses and Movements in Embankments," Research Report No. TE-73-4, University of California, Berkeley, California, December, 1973.

8. Allen,D.L., and Meade,B.W.; "Analysis of Loads and Settlements for Reinforced Concrete Culverts," Research report UKTRP-84-22, University of Kentucky, Transportation Research Program, August 1984.

Site 1 - KY 4, Fayette County

Site 2 - I 64, Rowan County

Site 3 - I 71, Carroll County

Site 4 - I 64, Bath County

Site 5 - I 471, Campbell County

Site 6 - I 24, Lyon County

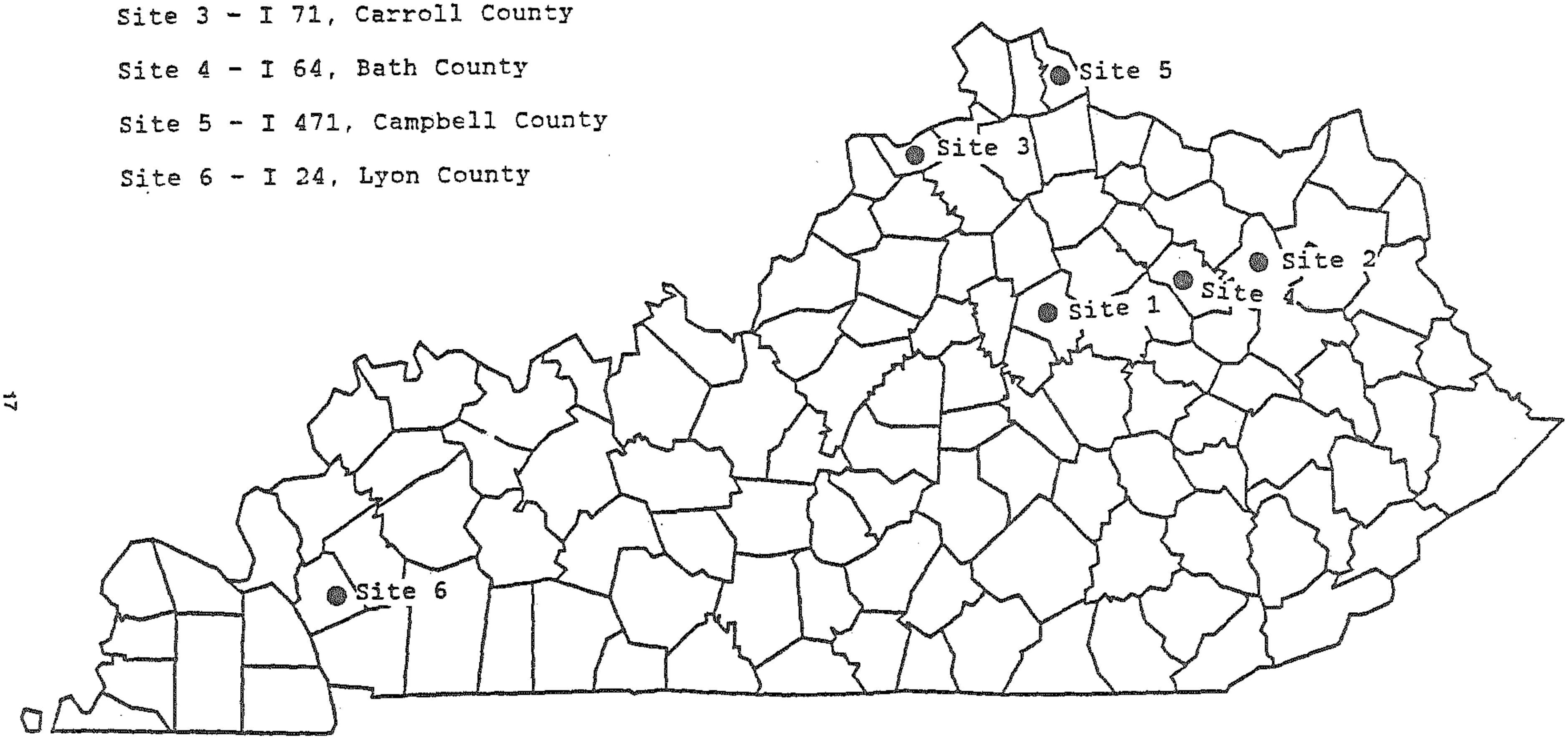


Figure 1. Location of Study Sites.

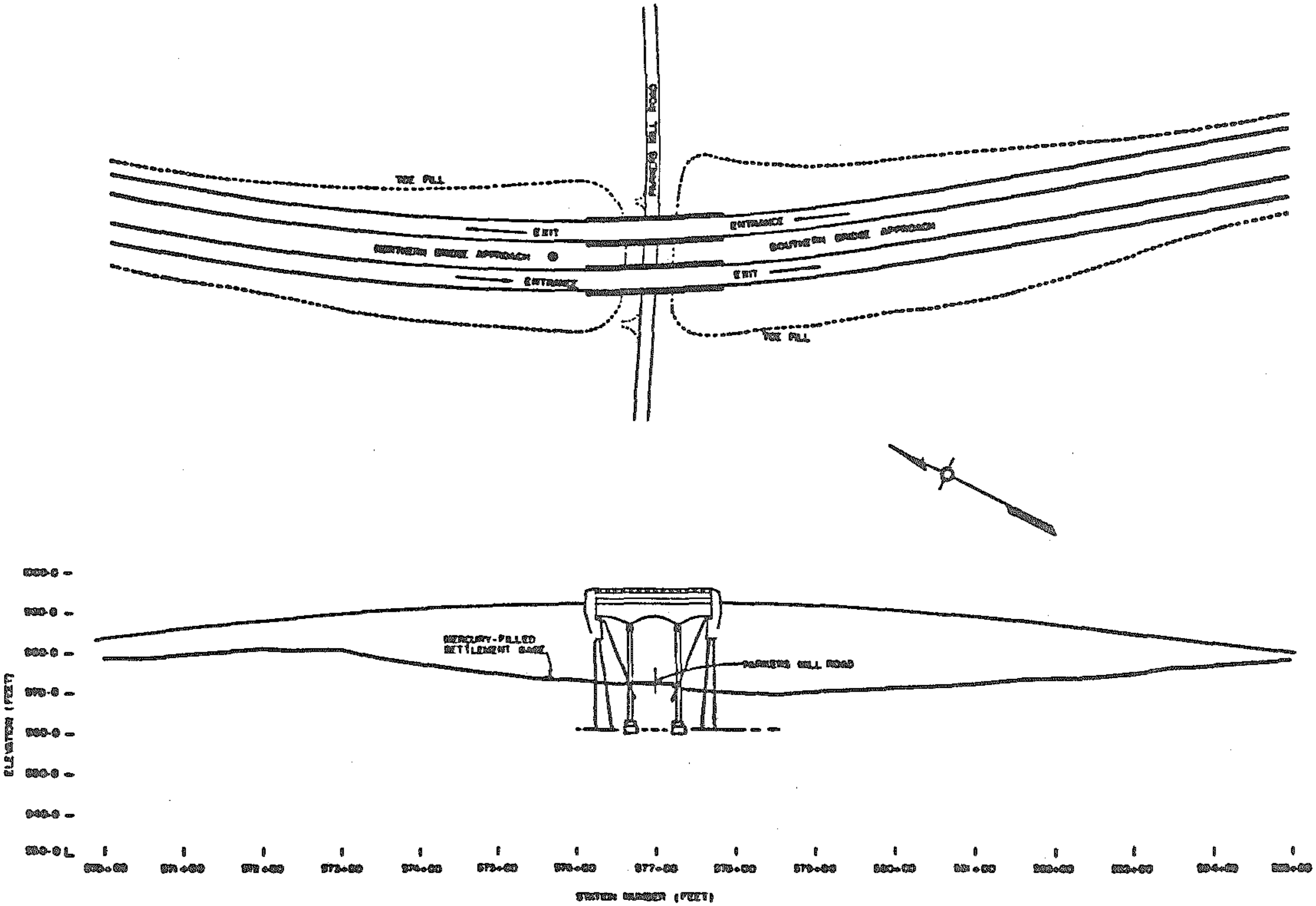


Figure 2. Lexington Relief Route Bridge Site (KY 4 - Parkers Mill Road).

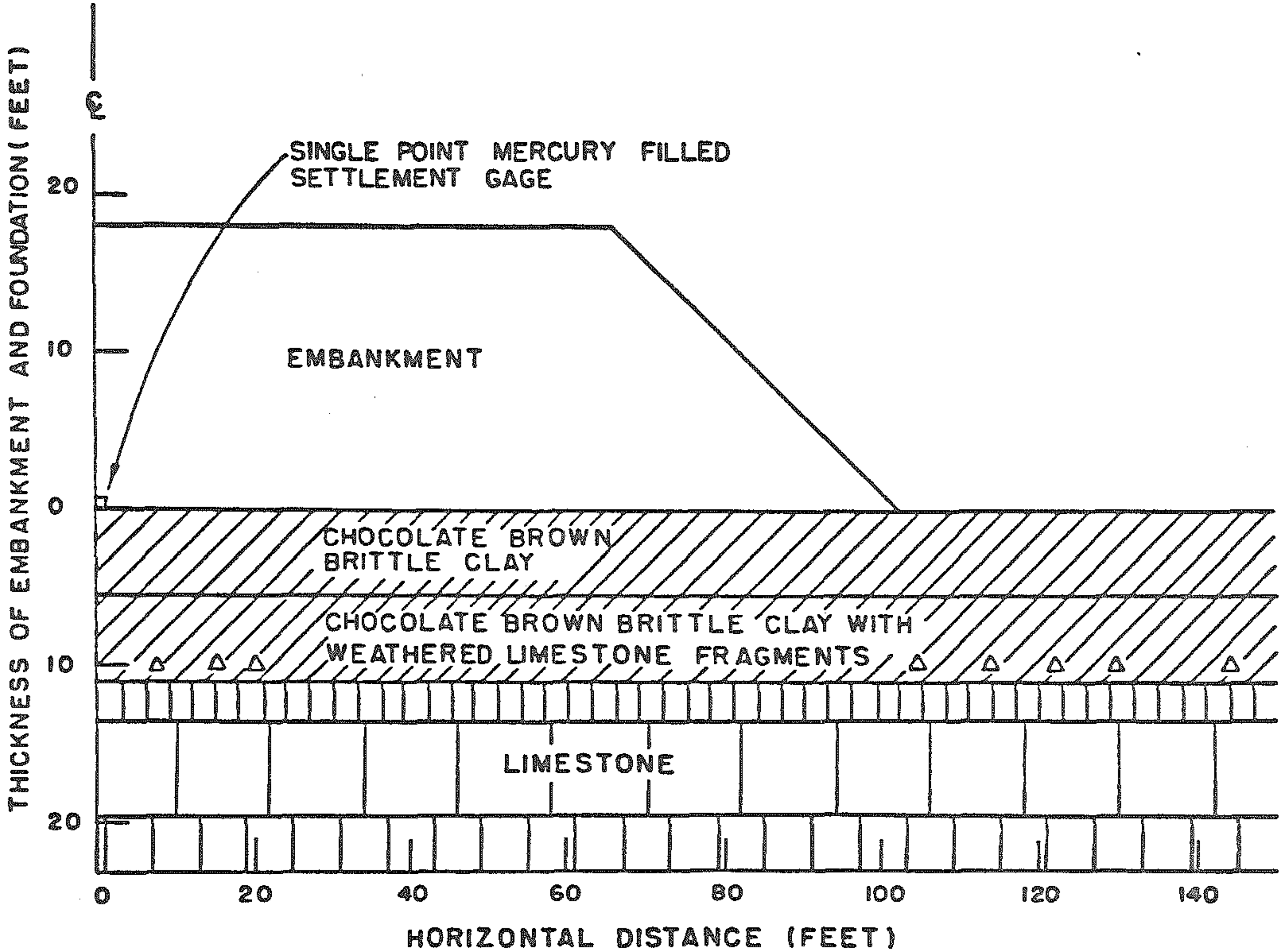


Figure 3. Typical Cross Section of Embankment and Foundation, Station 575+65, Parkers Mill Overpass.

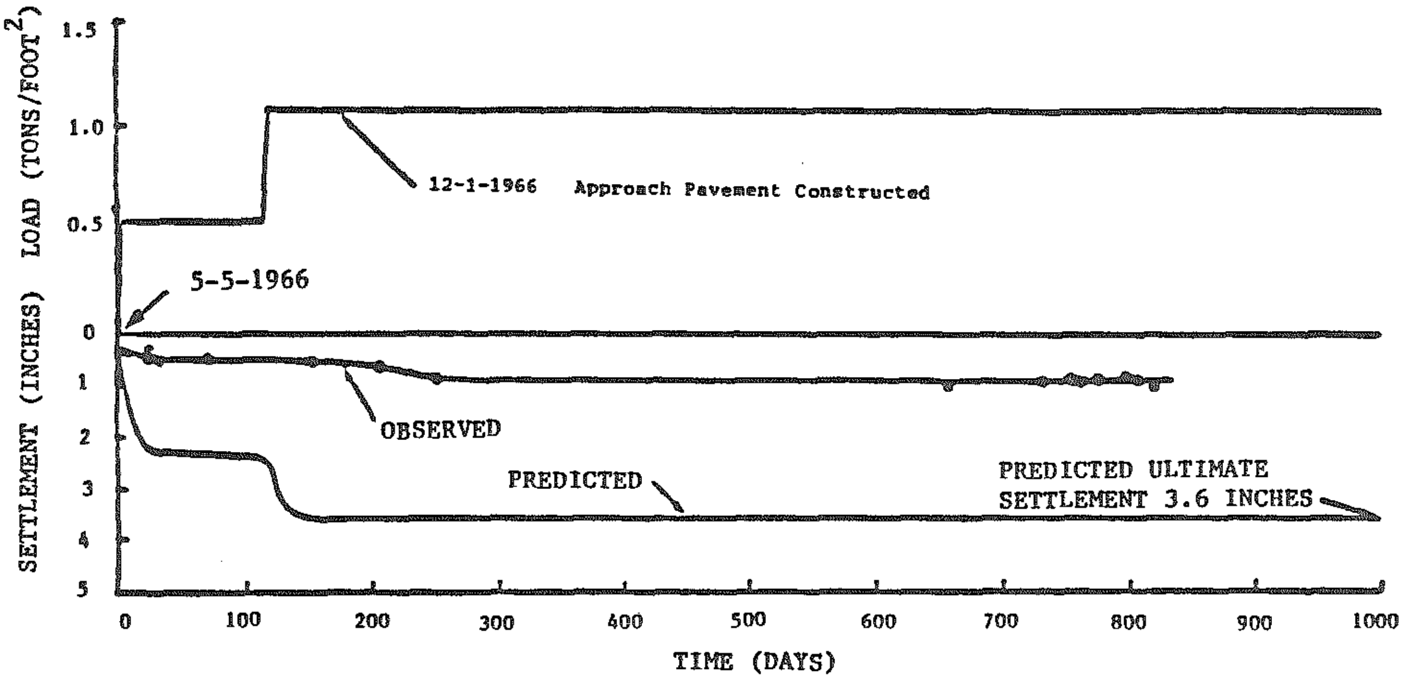


Figure 4. Observed and Predicted Time-Settlement Curves and Time-Loading Curves, Parkers Mill Road Overpass.

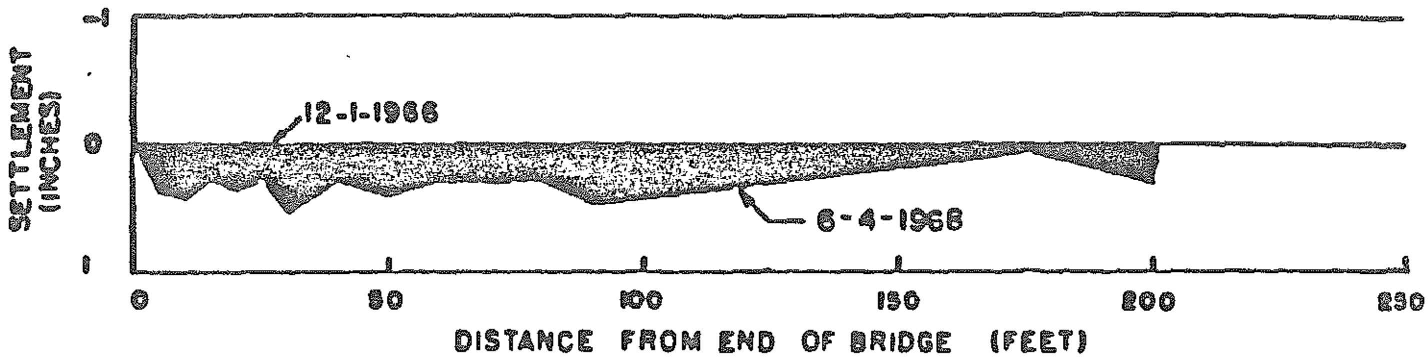


Figure 5. Settlement of the Entrance Approach of the Southbound Bridge, Parkers Mill Road Overpass.

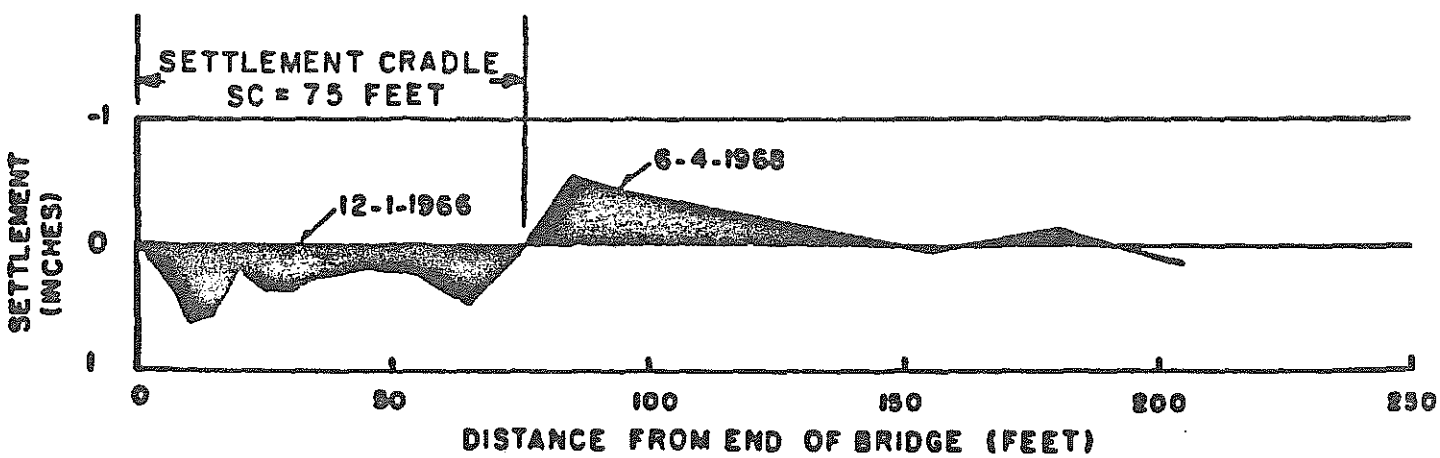


Figure 6. Settlement of the Exit Approach of the Northbound Bridge, Parkers Mill Road Overpass.

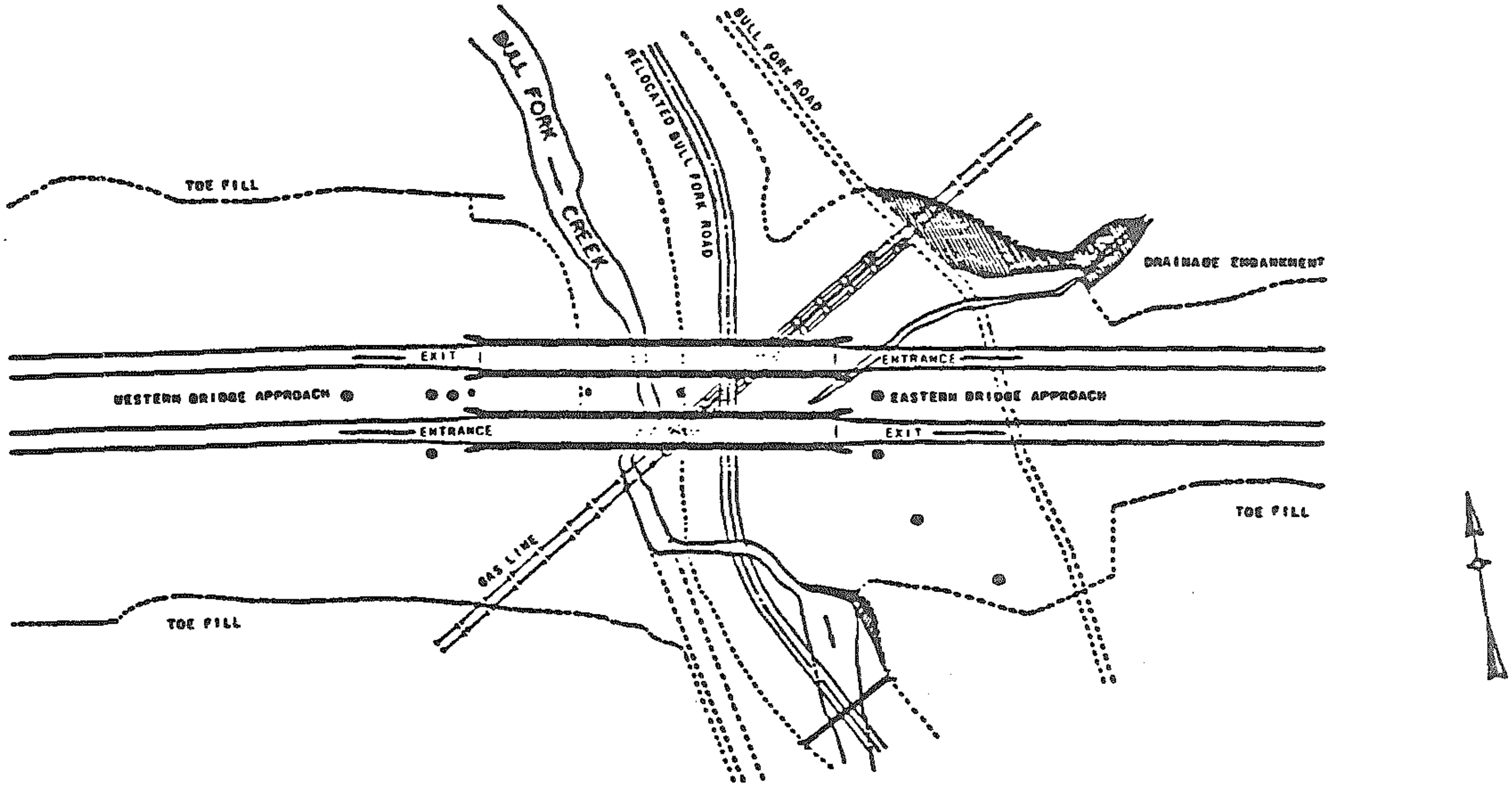


Figure 7. I 64 over Bull Fork Creek.

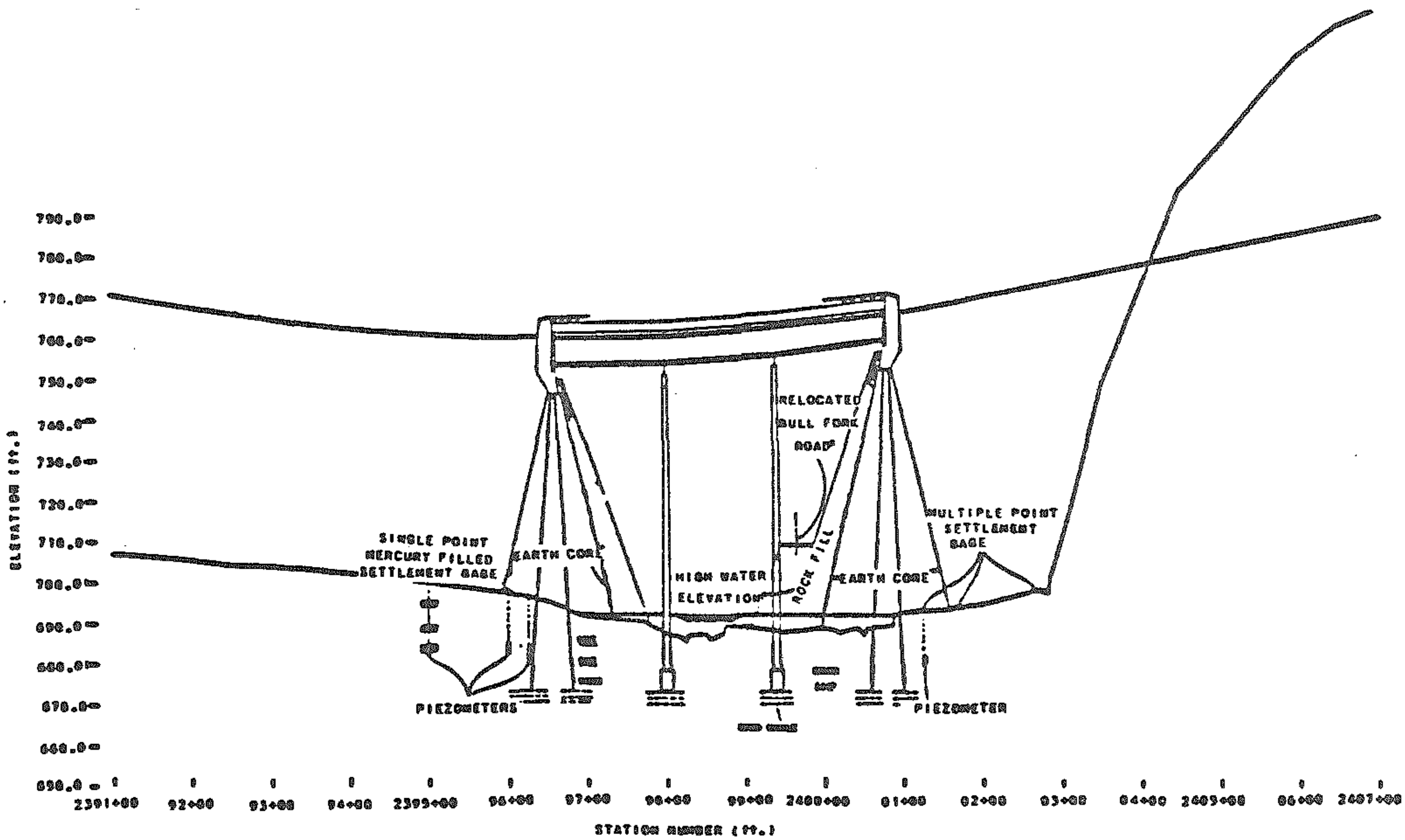


Figure 8. Centerline Section of Bull Fork Creek Site.

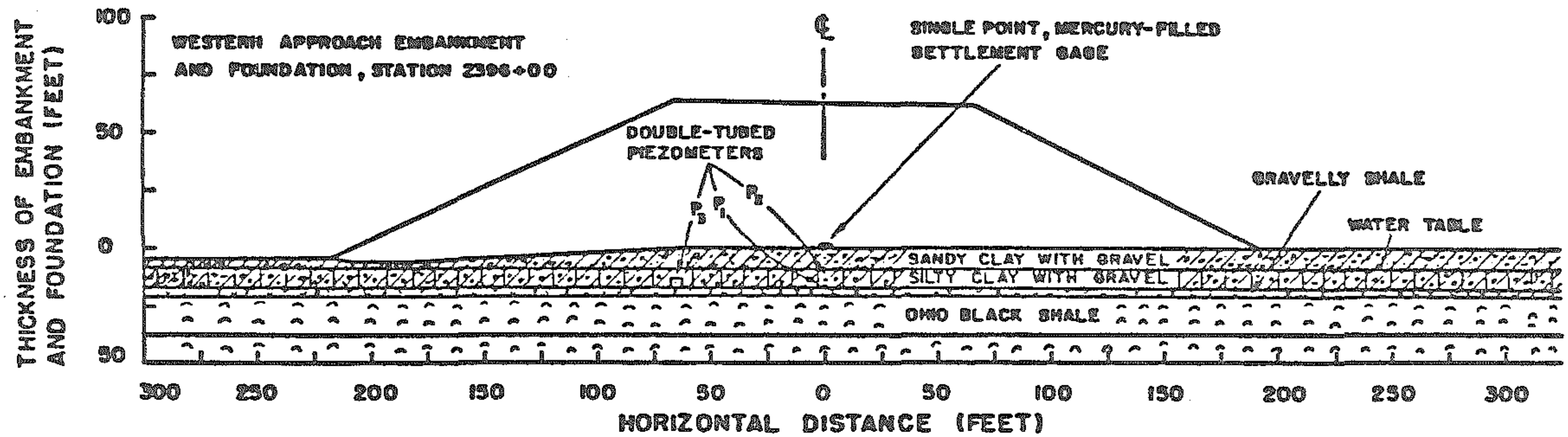


Figure 9. Western Approach of Bull Fork Creek Site with Instrumentation Locations.

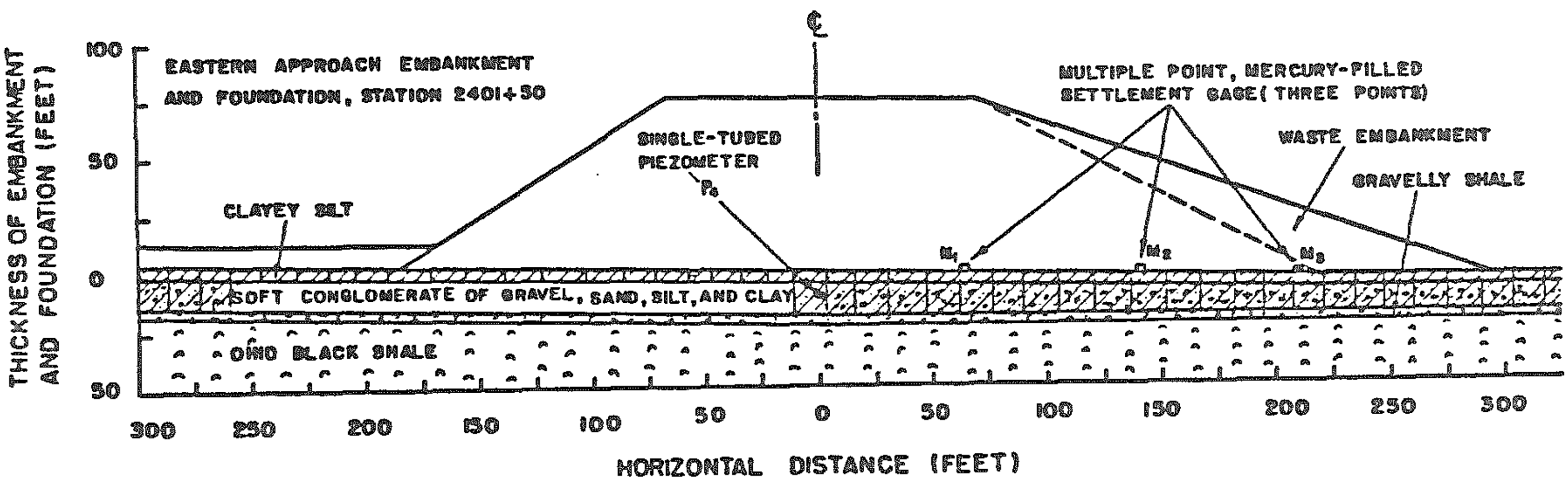


Figure 10. Eastern Approach of Bull Fork Creek Site with Instrumentation Location.

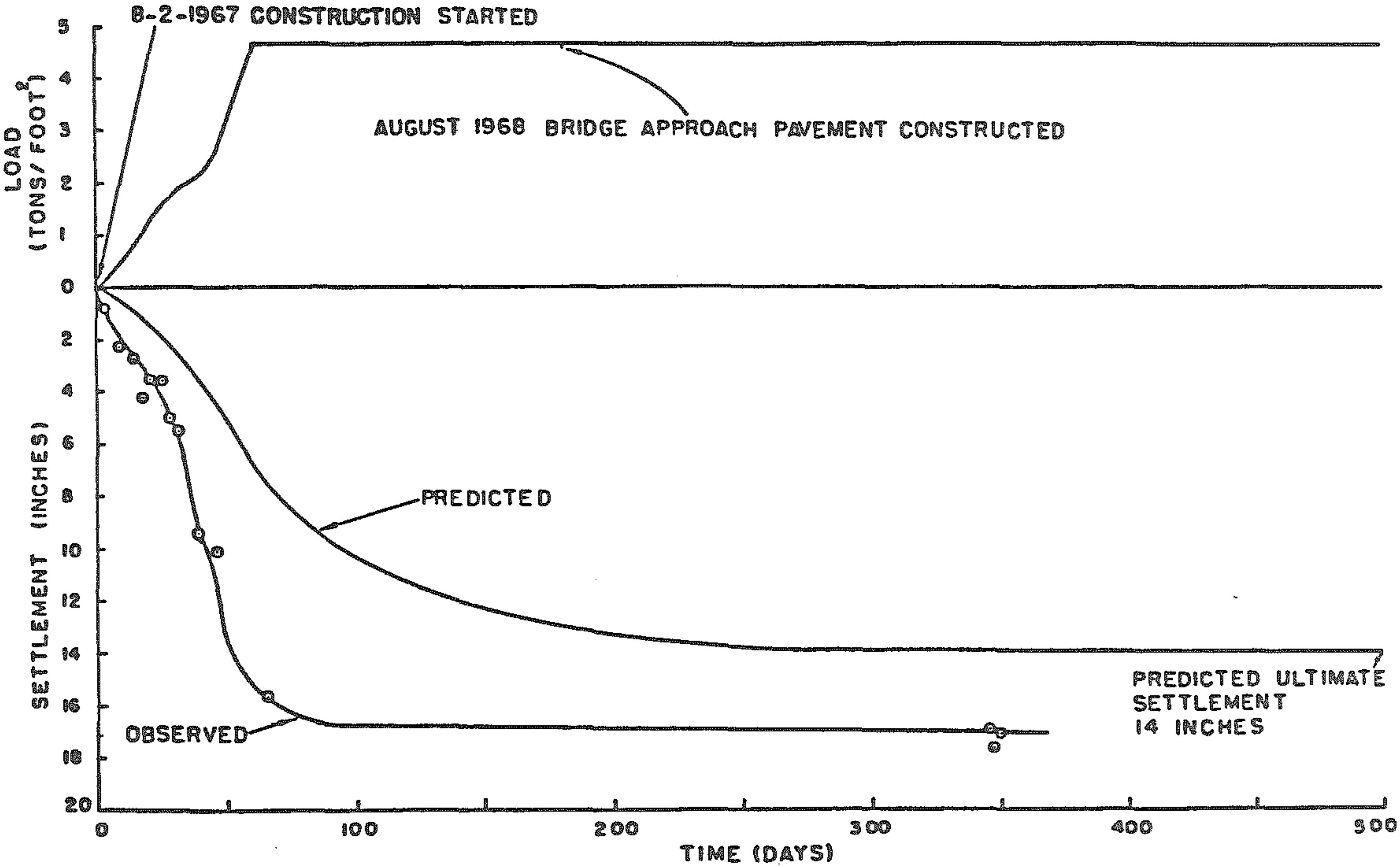


Figure 11. Observed and Predicted Time-Settlement Curves, Station 2396+00 West Approach, Centerline, I 64 over Bull Fork Creek, Rowan County.

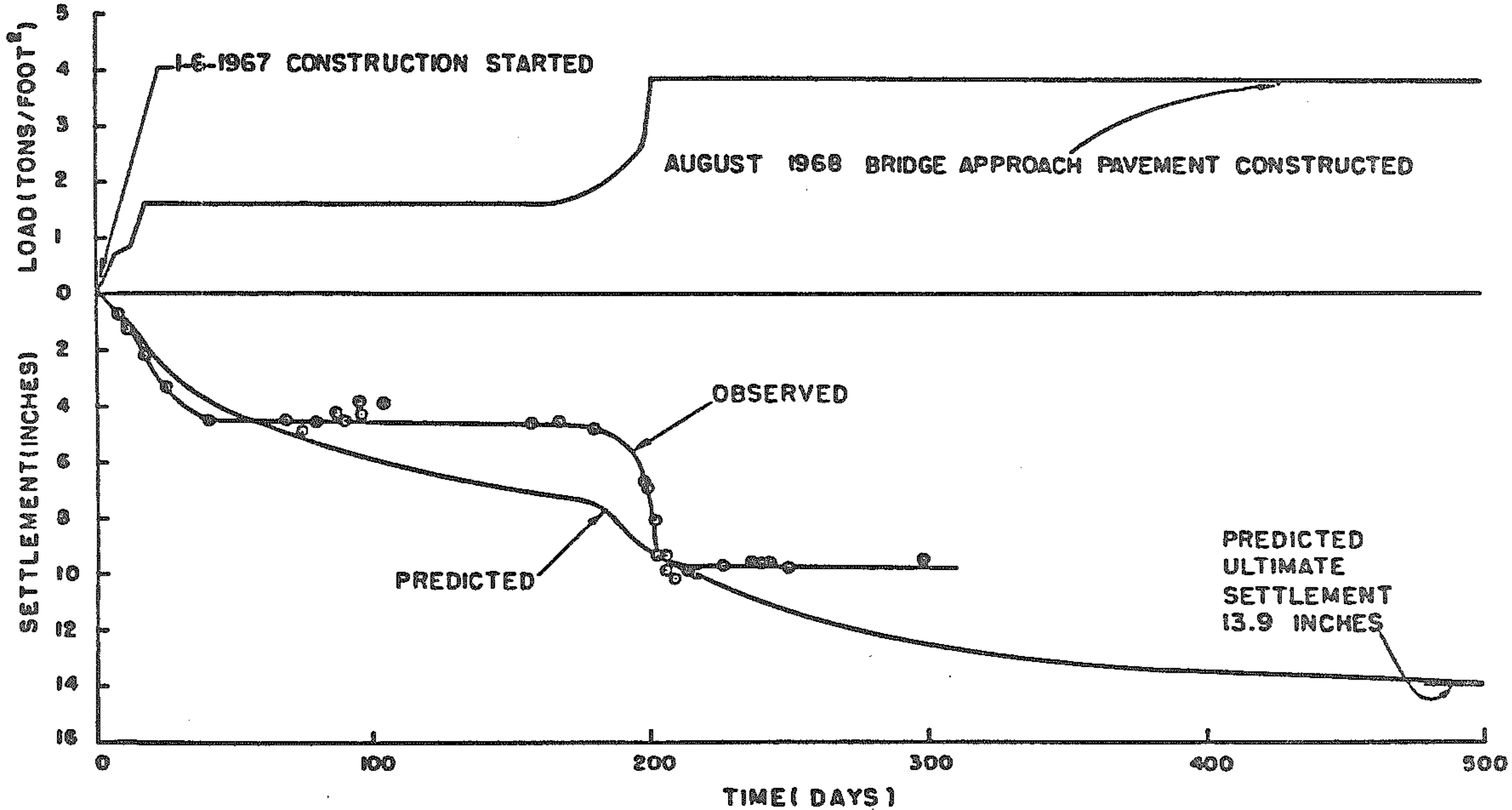


Figure 12. Observed and Predicted Time-Settlement Curves, Point 1, Station 2401+25 East Approach, 66 feet right of Centerline, I 64 over Bull Fork Creek, Rowan County.

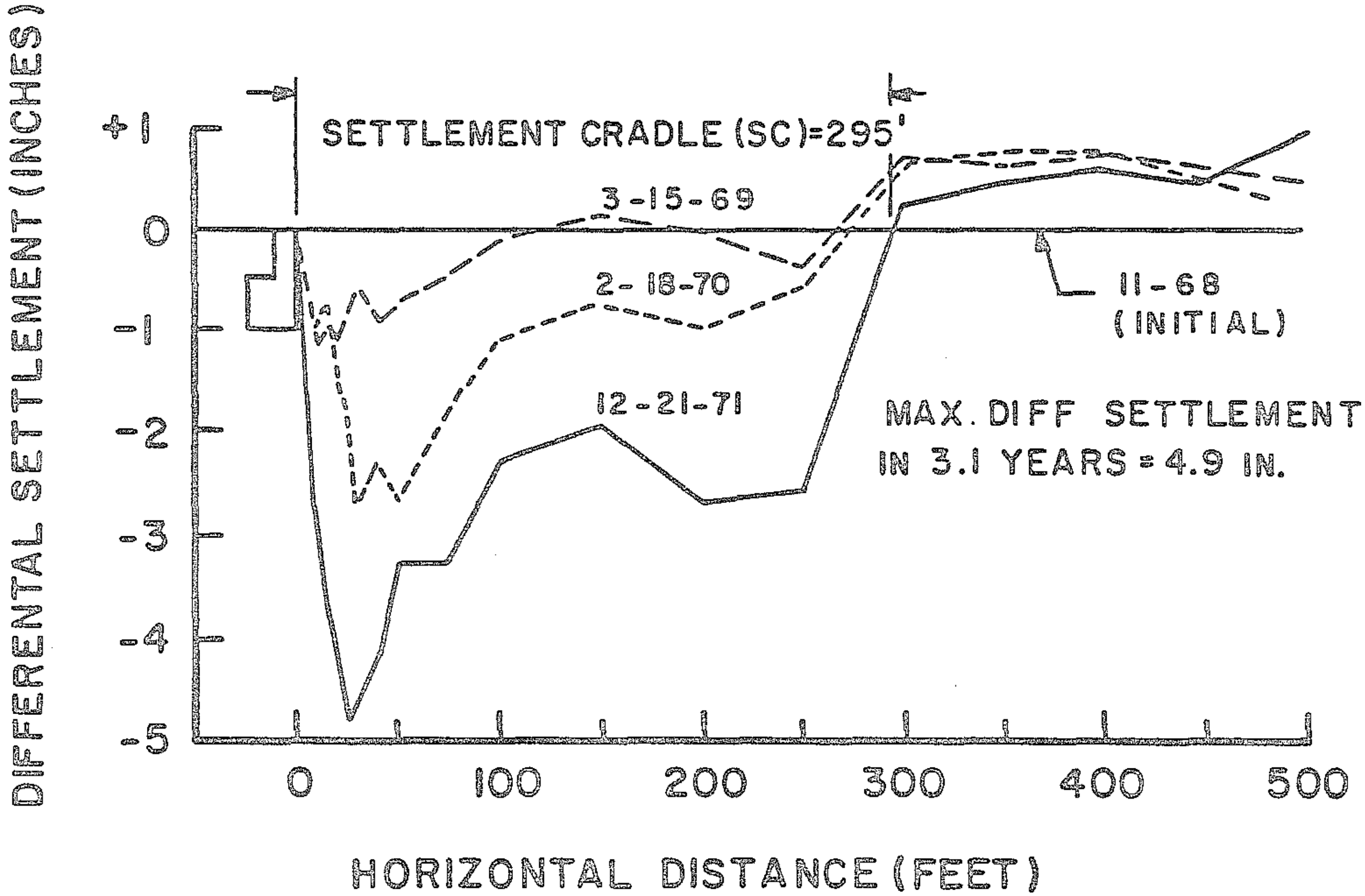


Figure 13. Pavement Profile of East Approach (Outside Edge), Westbound Lanes, Bull Fork Creek.

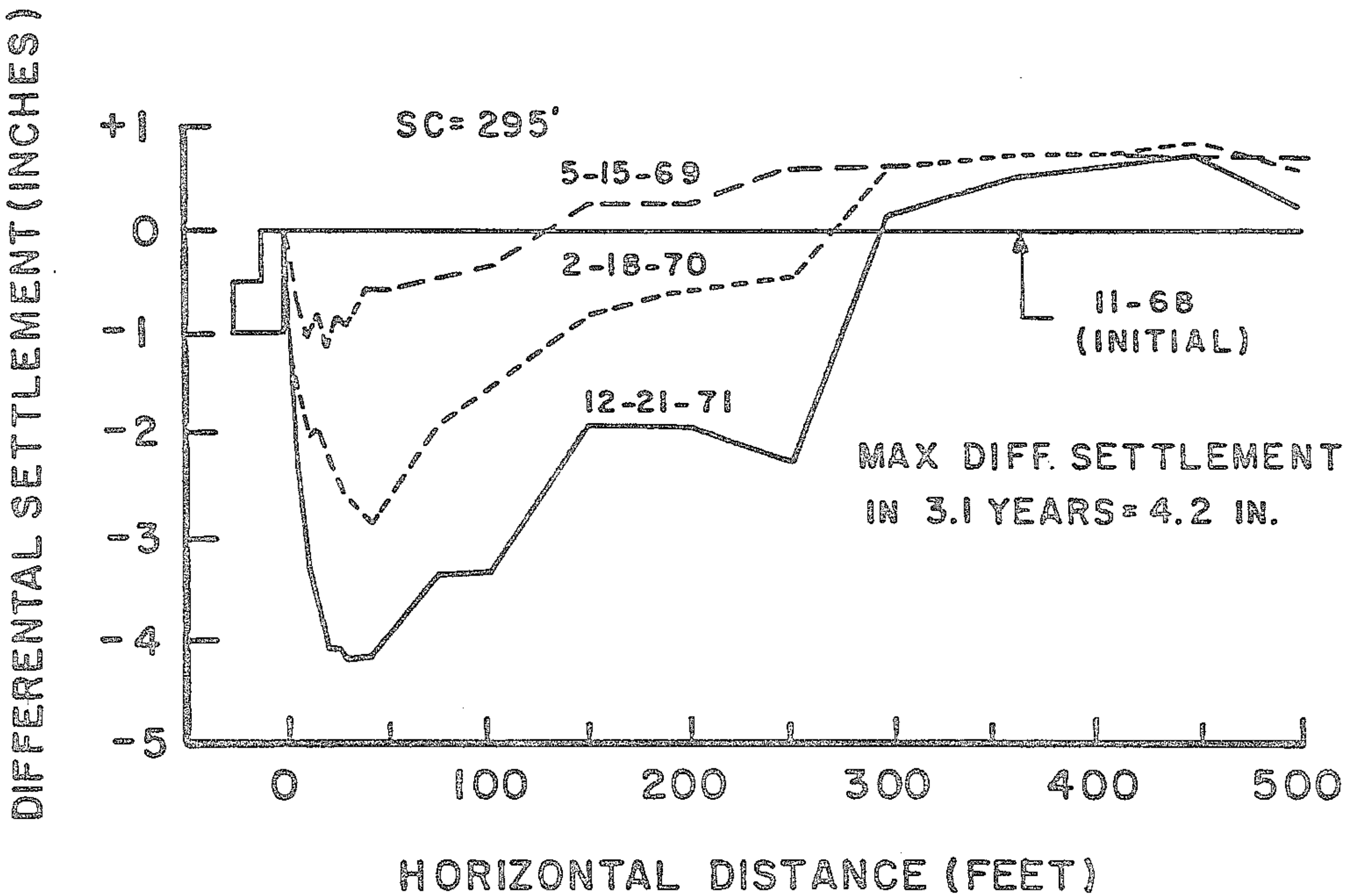


Figure 14. Pavement Profile of East Approach (Inside Edge), Westbound Lanes, Bull Fork Creek.

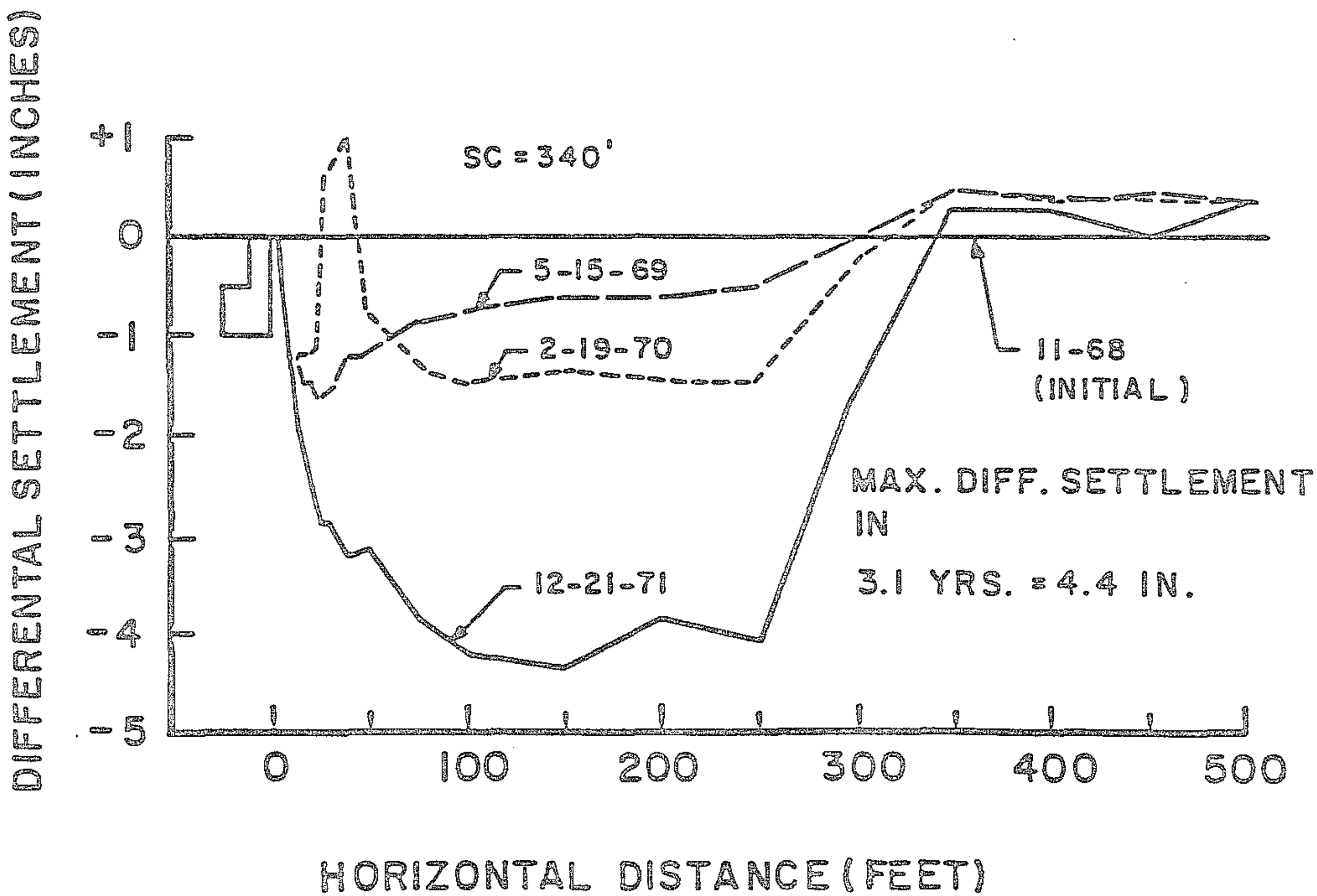


Figure 15. Pavement Profile of East Approach, Eastbound Lanes, (Outside Edge), Bull Fork Creek.

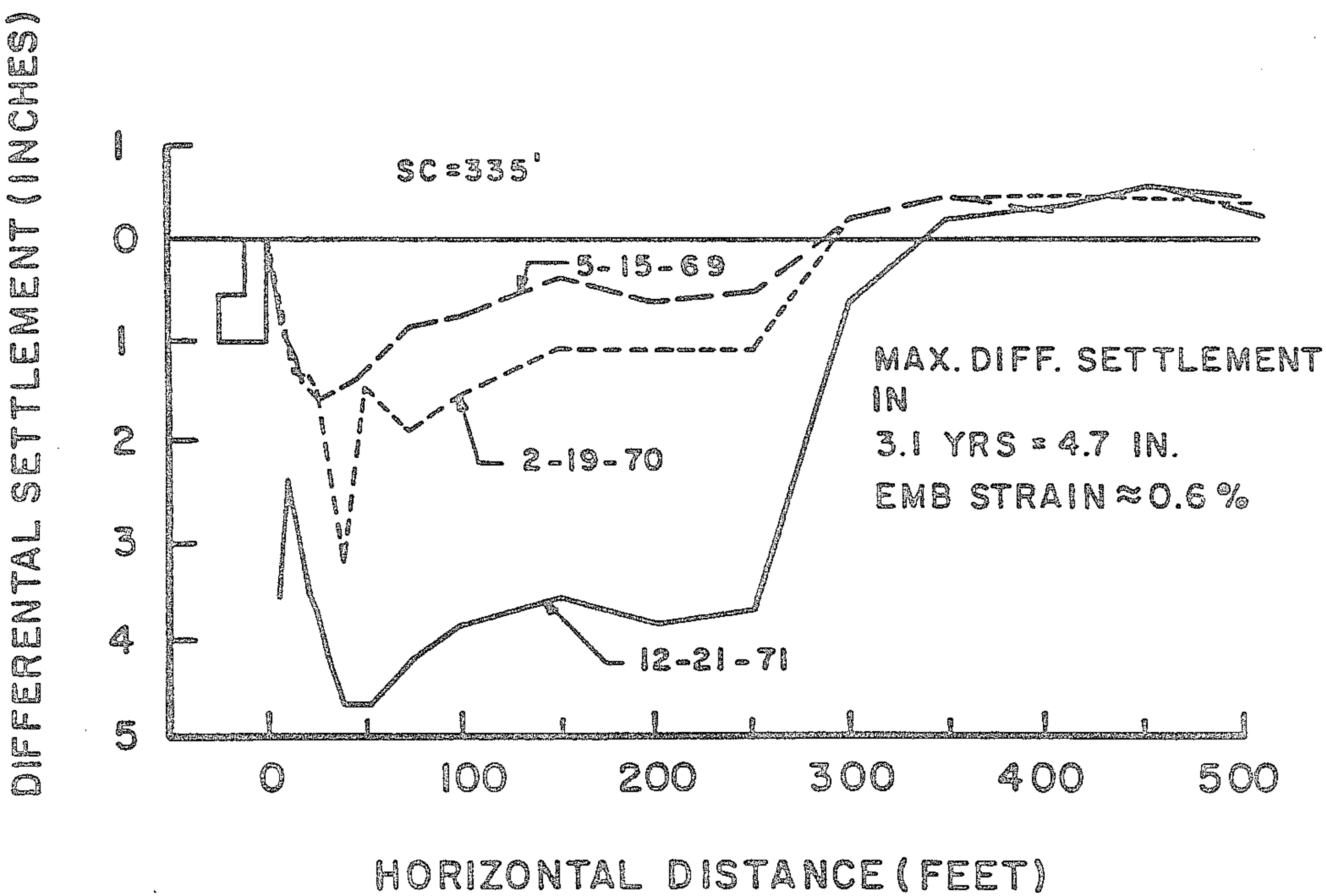


Figure 16. Pavement Profile of East Approach, Eastbound Lanes, (Inside Edge), Bull Fork Creek.

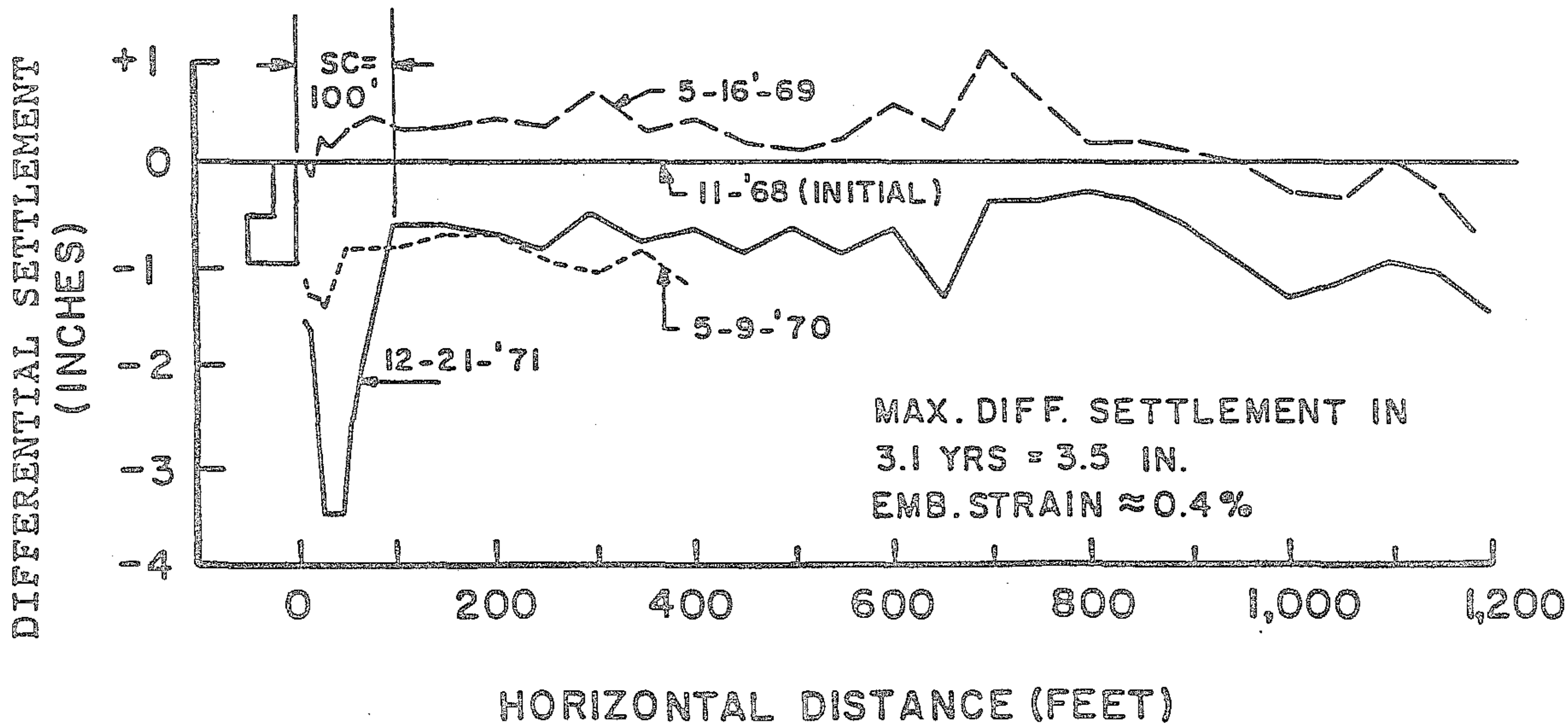


Figure 17. Pavement Profile of West Approach, Eastbound Lanes, (Outside Edge), Bull Fork Creek.

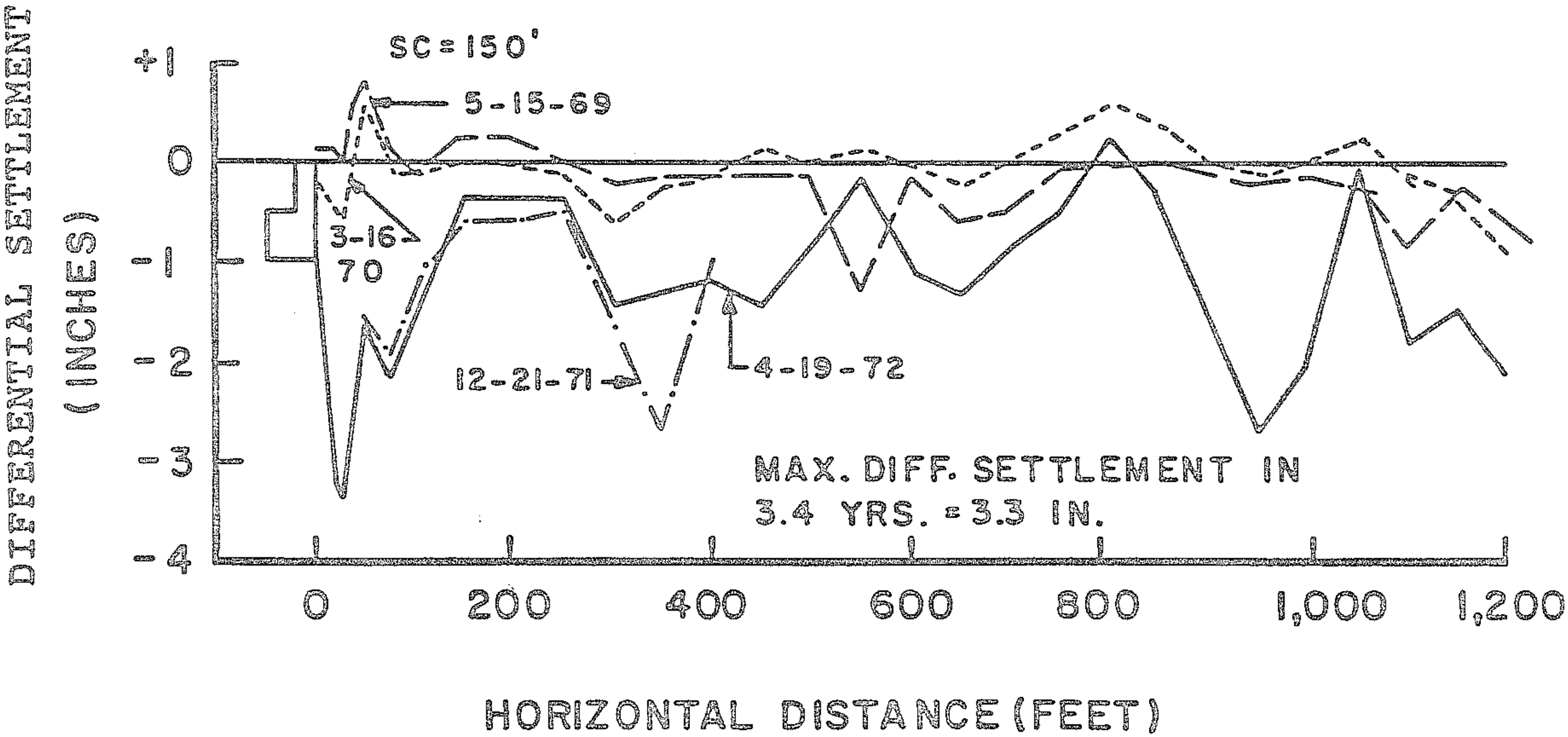


Figure 19. Pavement Profile of West Approach, Westbound Lanes, (Outside Edge), Bull Fork Creek.

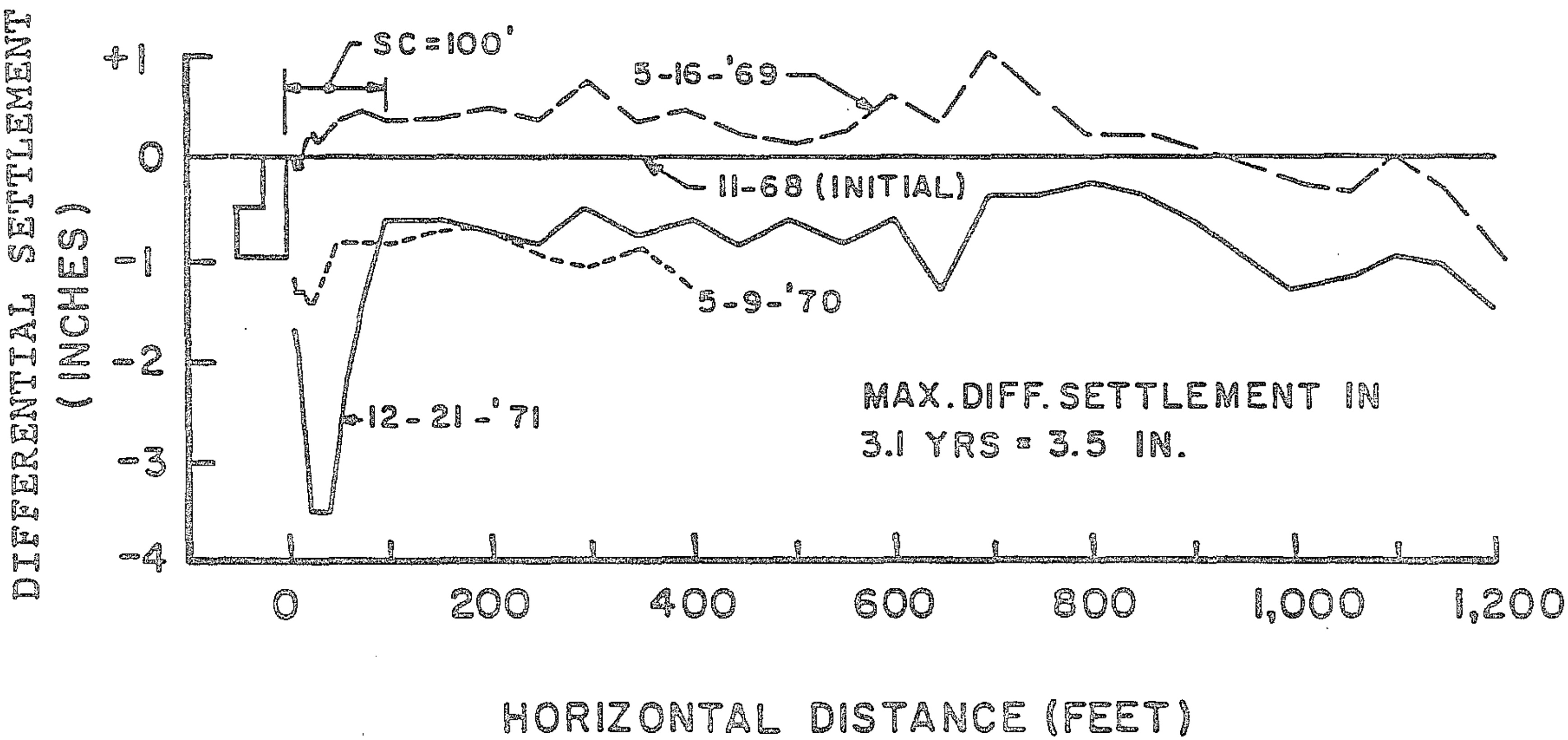


Figure 18. Pavement Profile of West Approach, Eastbound Lanes, (Inside Edge), Bull Fork Creek.

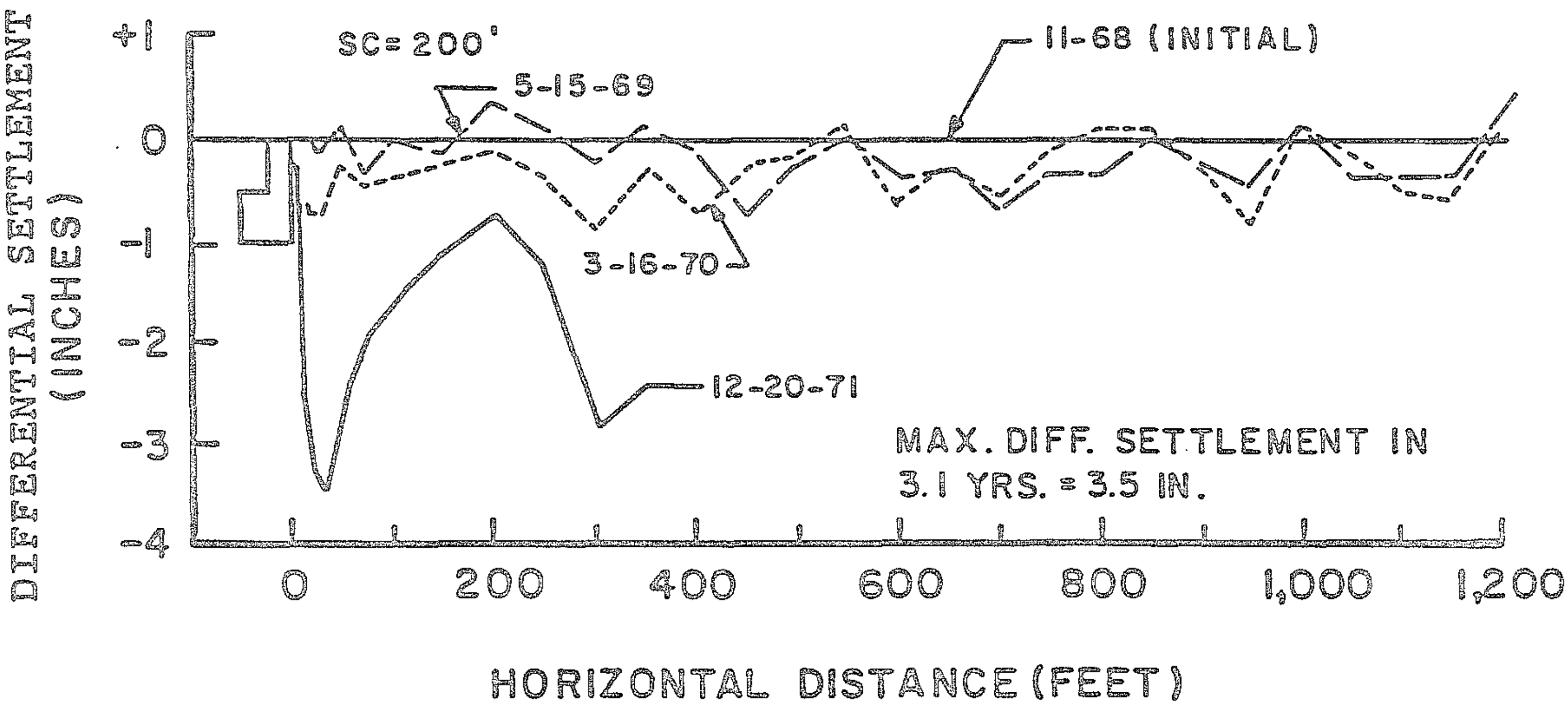


Figure 20. Pavement Profile of West Approach, Westbound Lanes, (Inside Edge), Bull Fork Creek.

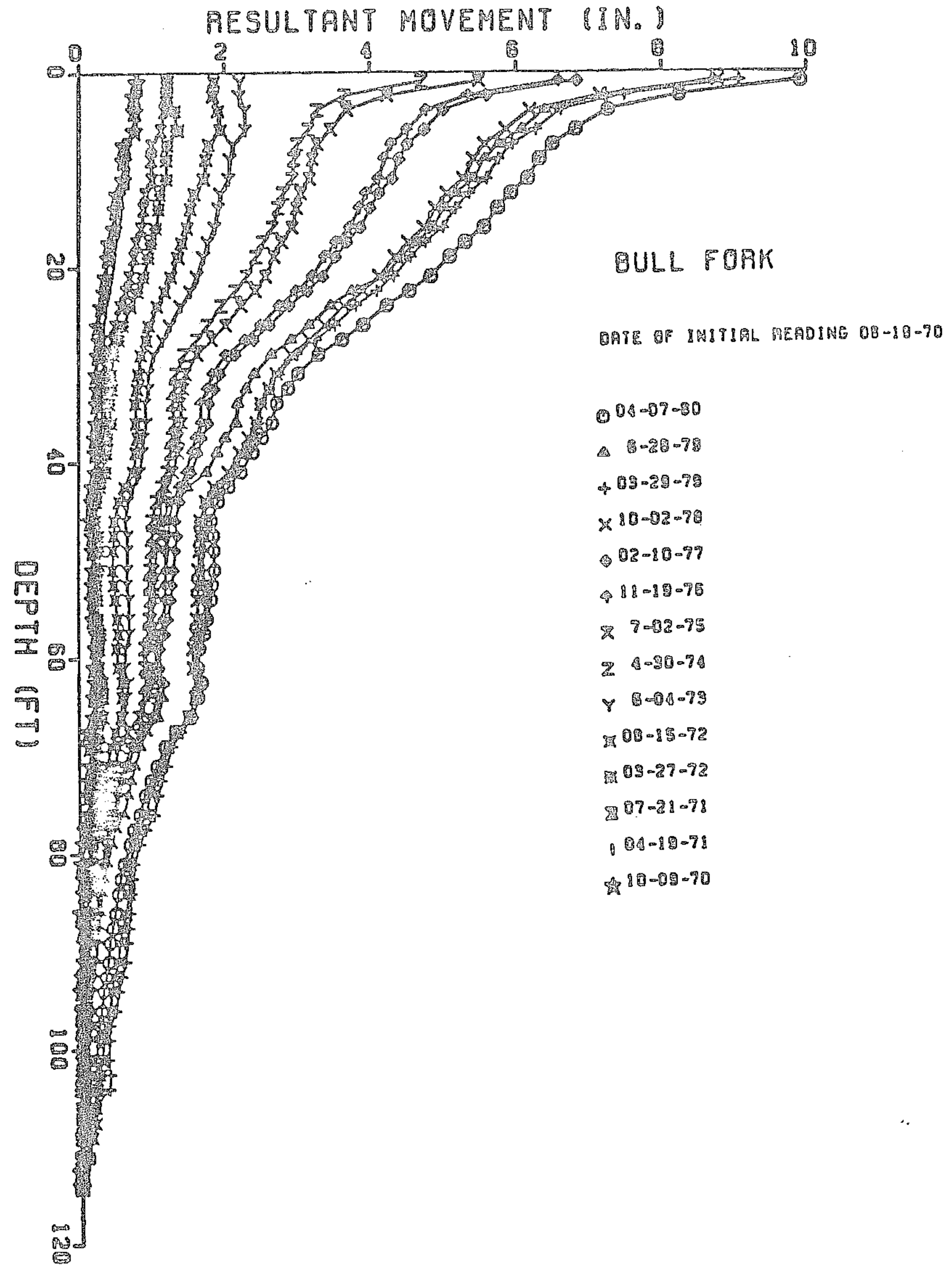


Figure 21. Lateral Movement of the East Approach, Bull Fork Creek.

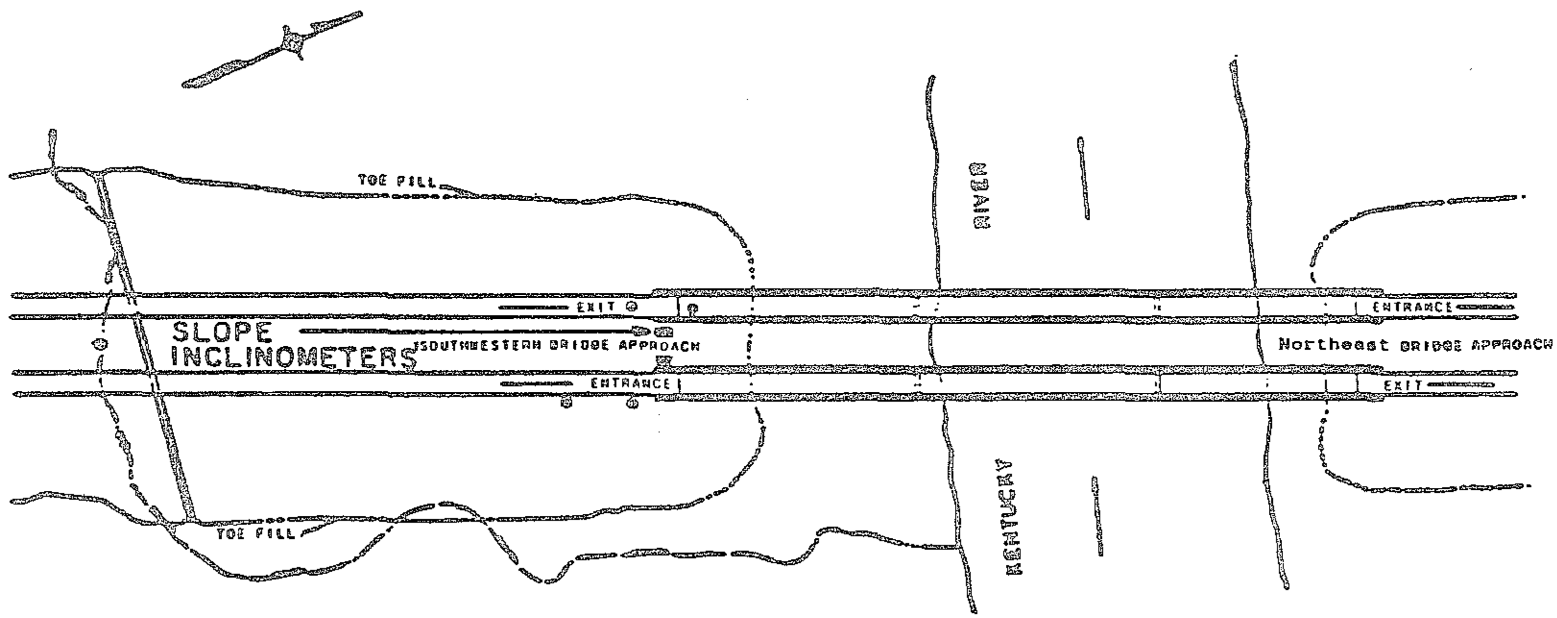


Figure 22. I 71 over the Kentucky River.

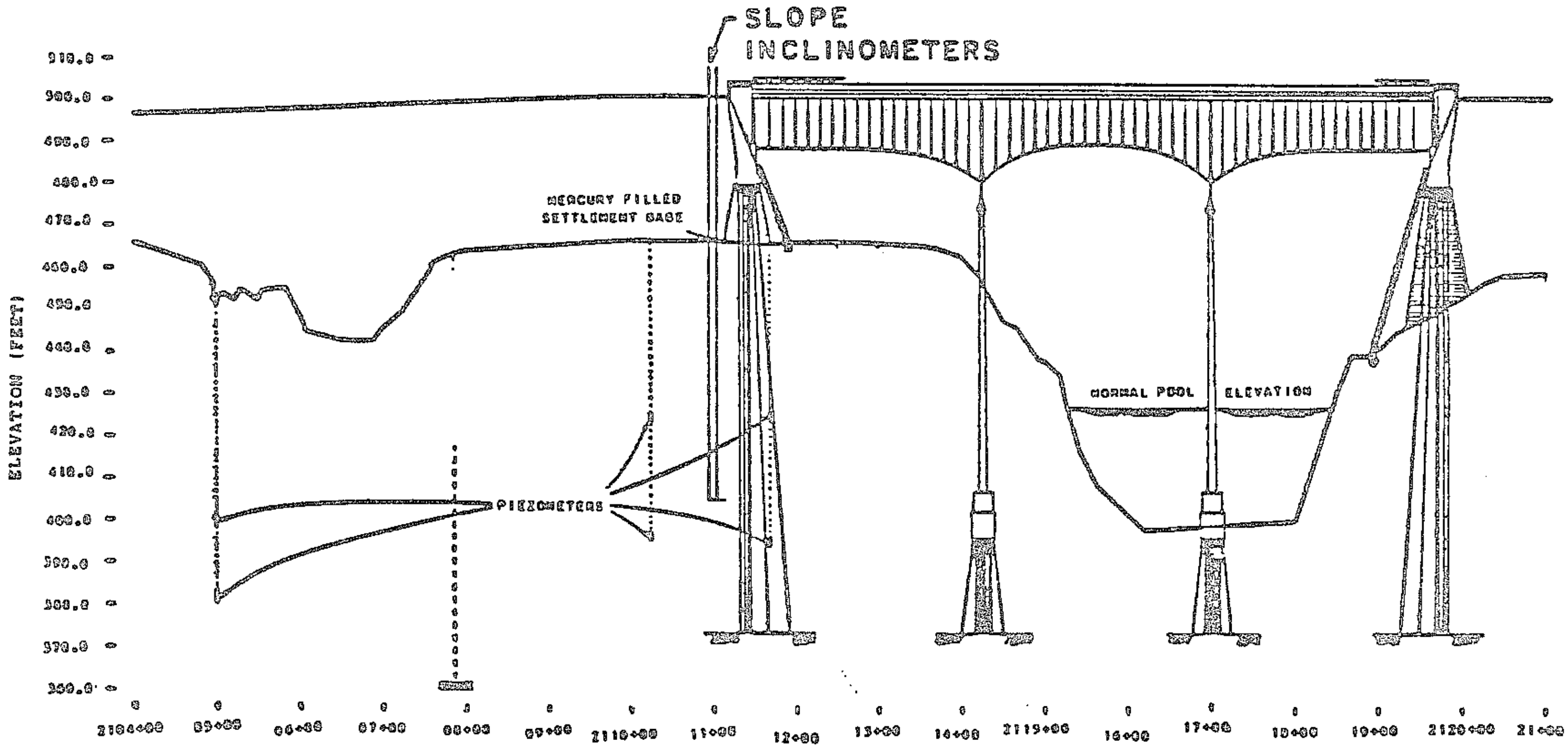


Figure 23. Centerline Section of I 71 over the Kentucky River Site.

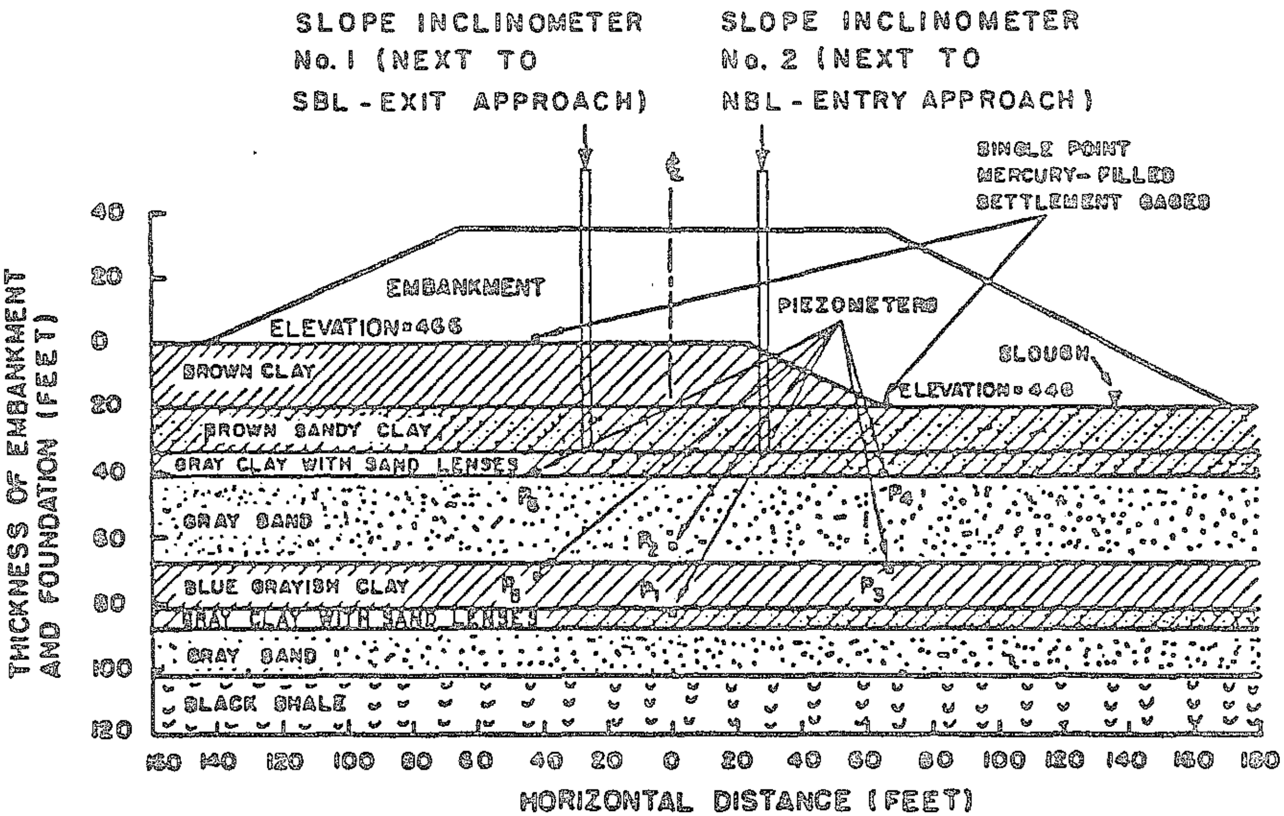


Figure 24. Typical Cross Section of Embankment and Foundation, Near Station 2111+50, I 71 over the Kentucky River, Carroll County.

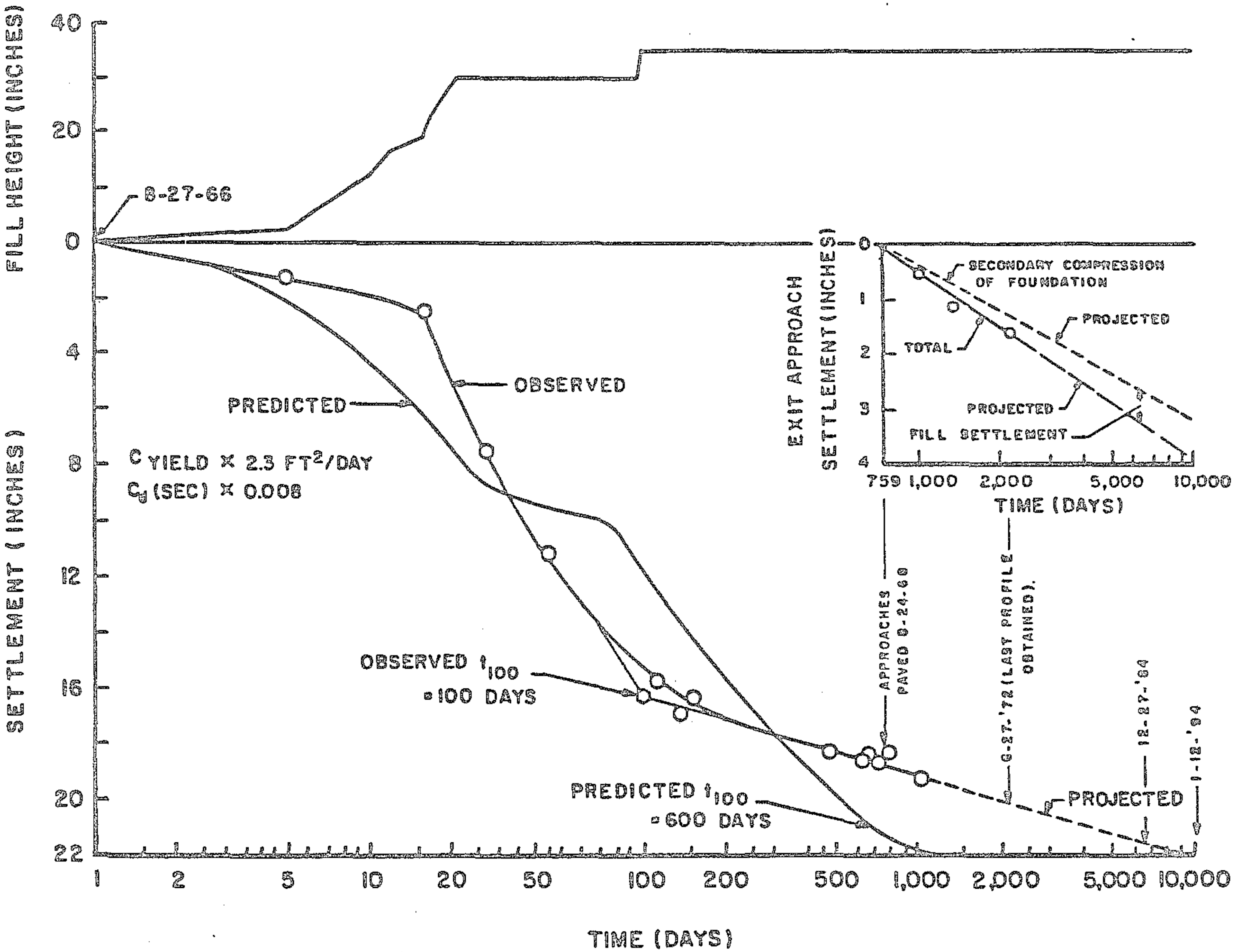


Figure 25. Observed and Predicted Foundation Settlement at Station 214+50, 42 feet left, I 71 over the Kentucky River.

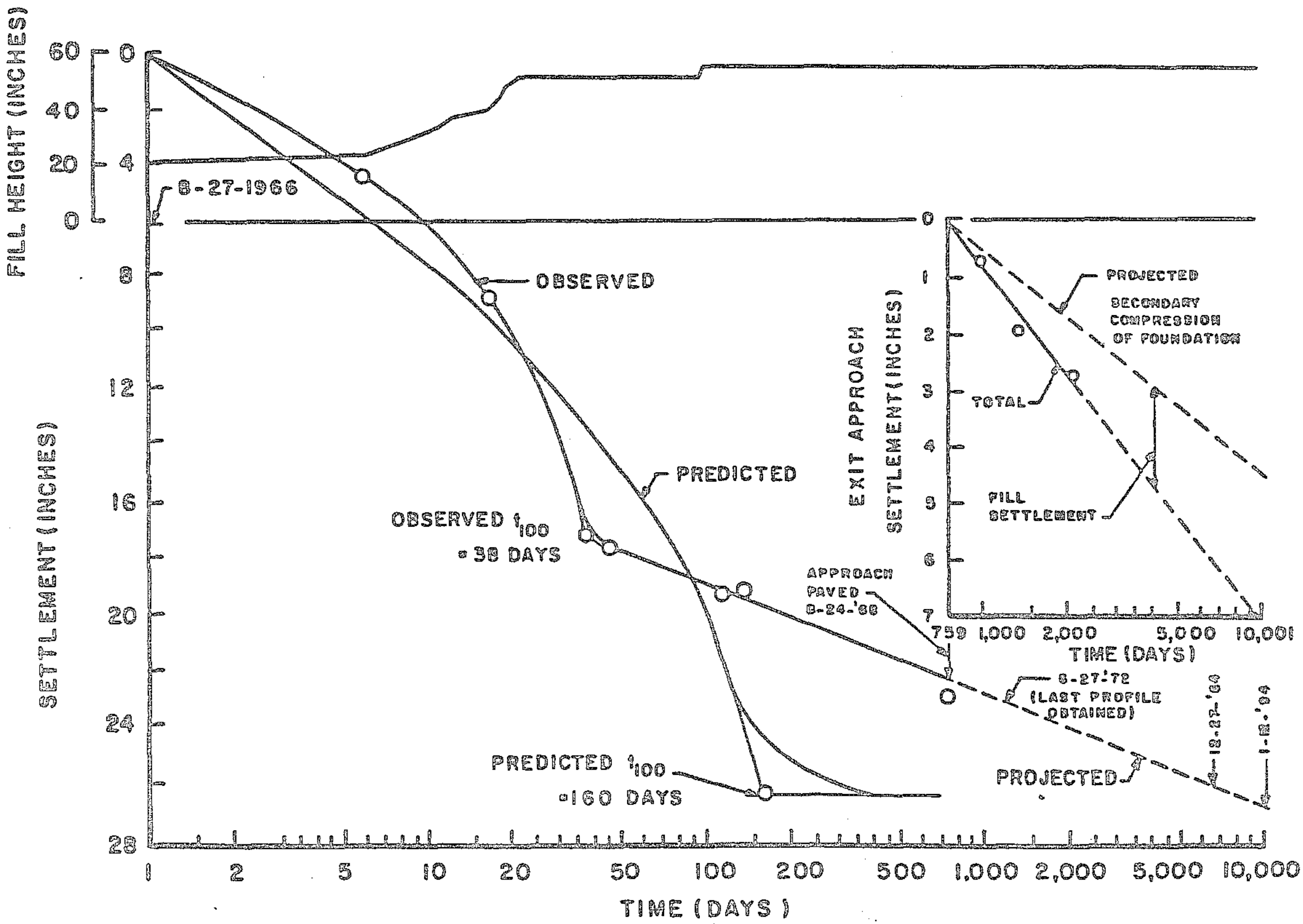


Figure 26. Observed and Predicted Foundation Settlement at Station 2111+50, 65 feet right, I 71 over the Kentucky River.

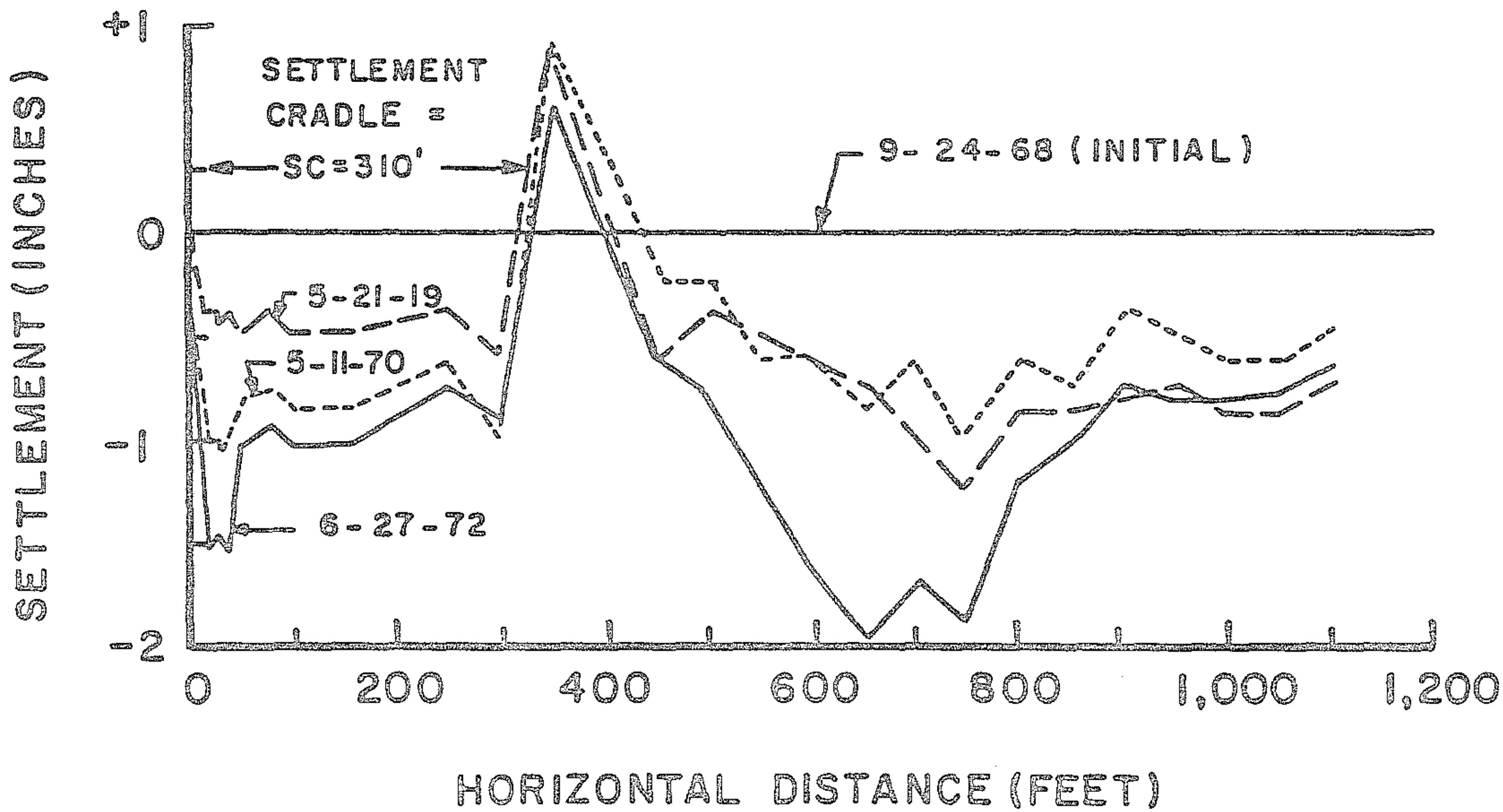


Figure 27. Pavement Settlement at Outside Edge of Westbound Lanes, I 71 over the Kentucky River.

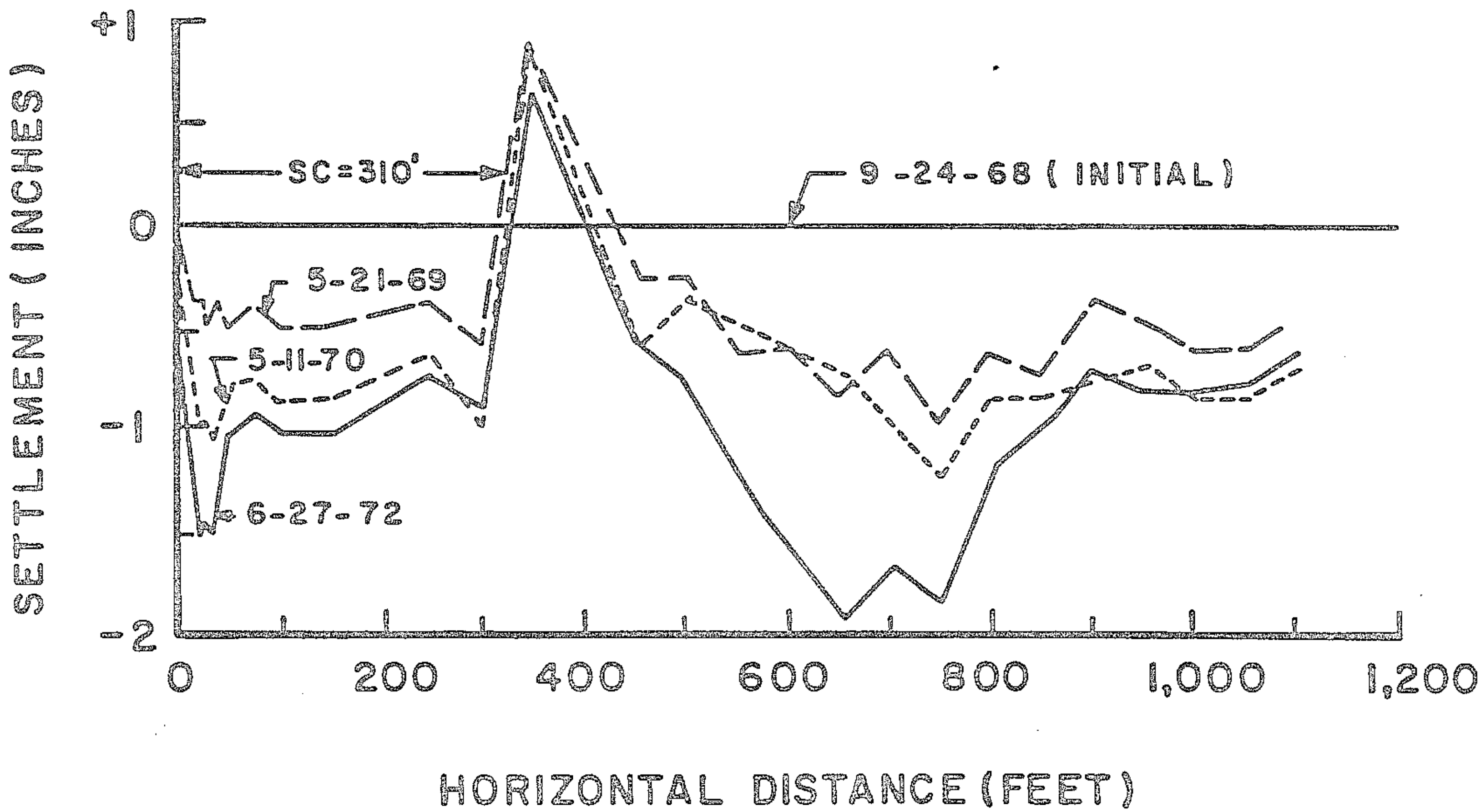


Figure 28. Pavement Settlement at Inside Edge of Westbound Lanes, I 71 over the Kentucky River.

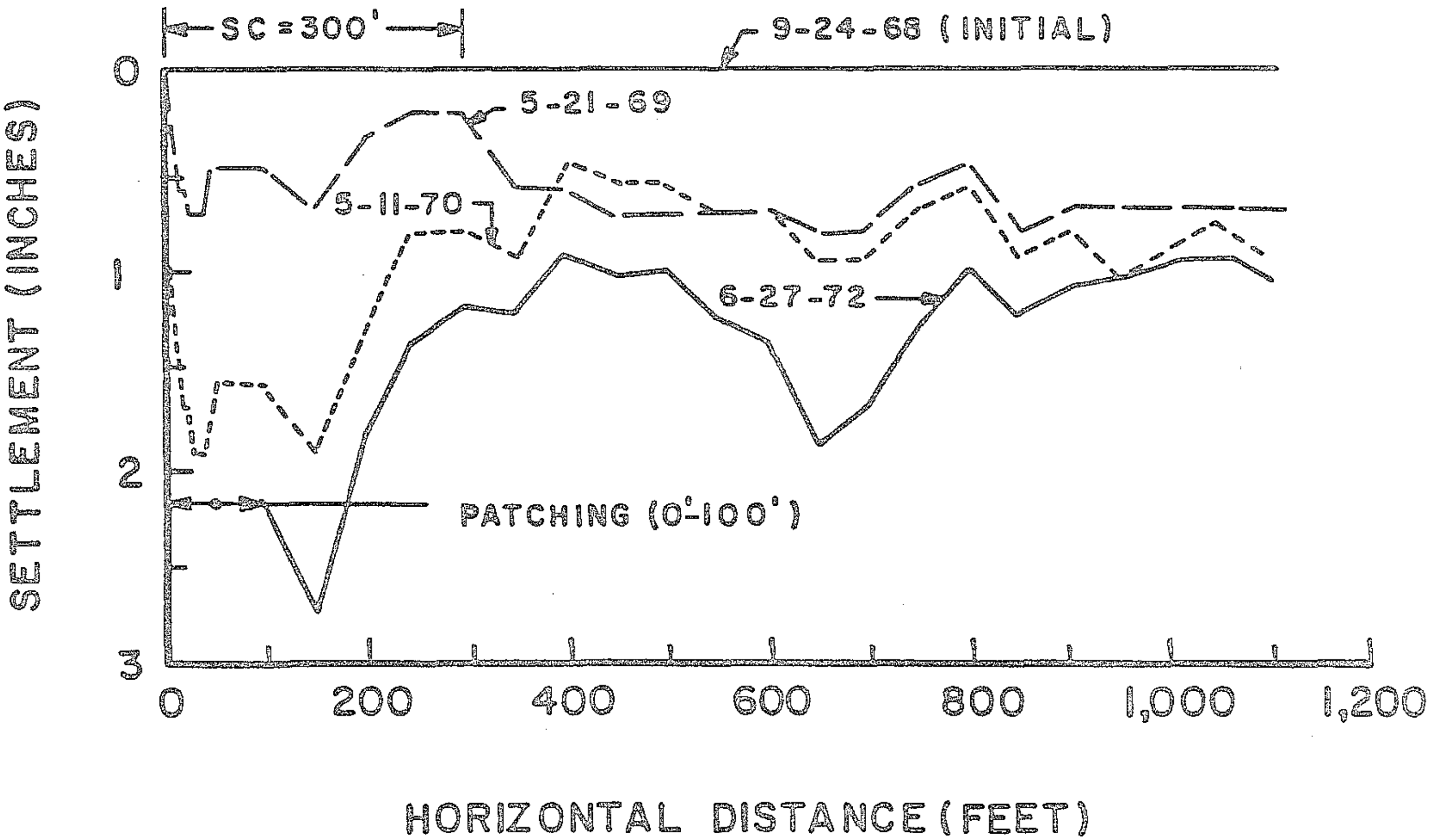


Figure 29. Pavement Settlement at Outside Edge of Eastbound Lanes, I 71 over the Kentucky River.

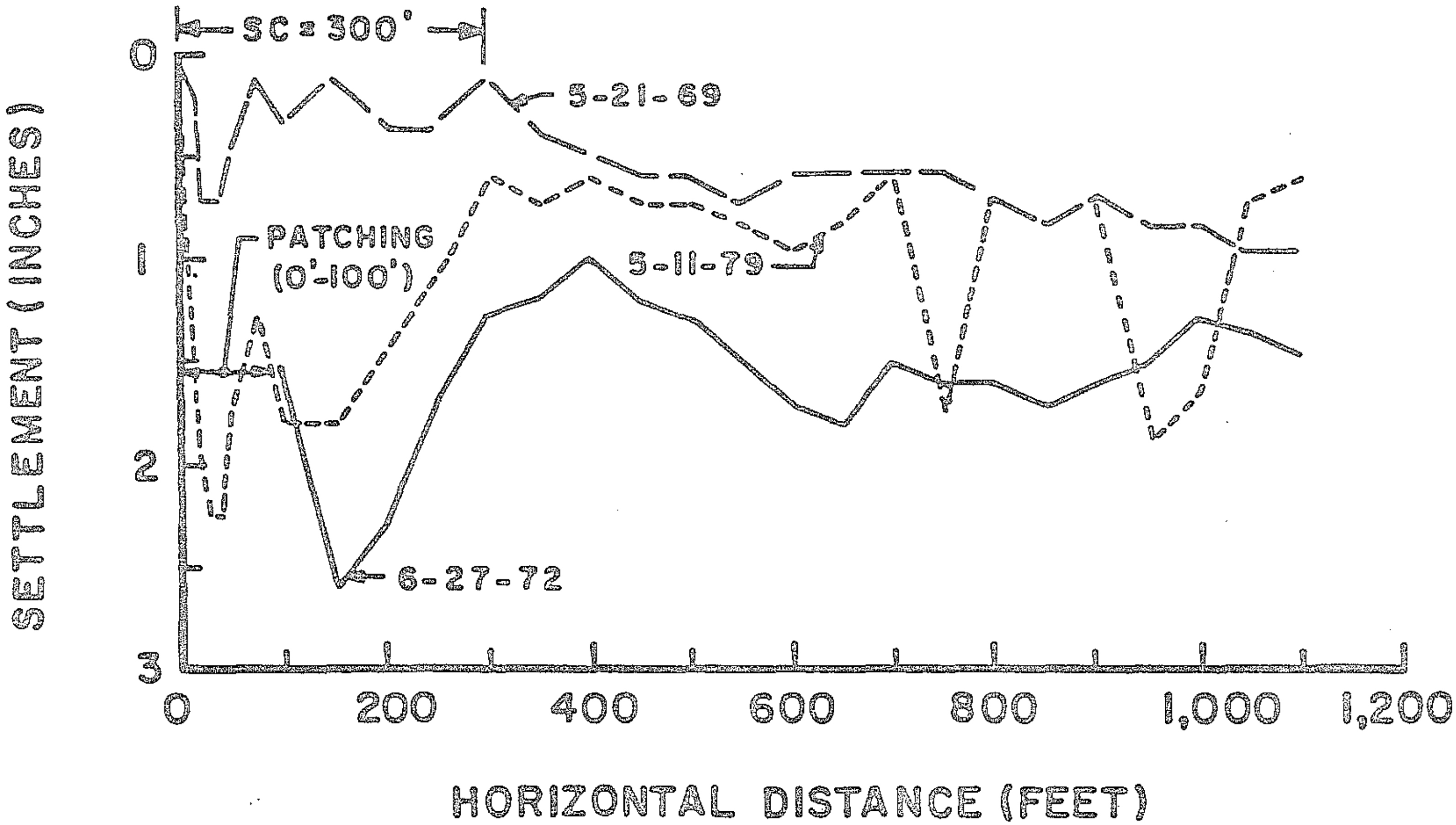


Figure 30. Pavement Settlement at Inside Edge of Eastbound Lanes, I 71 over the Kentucky River.

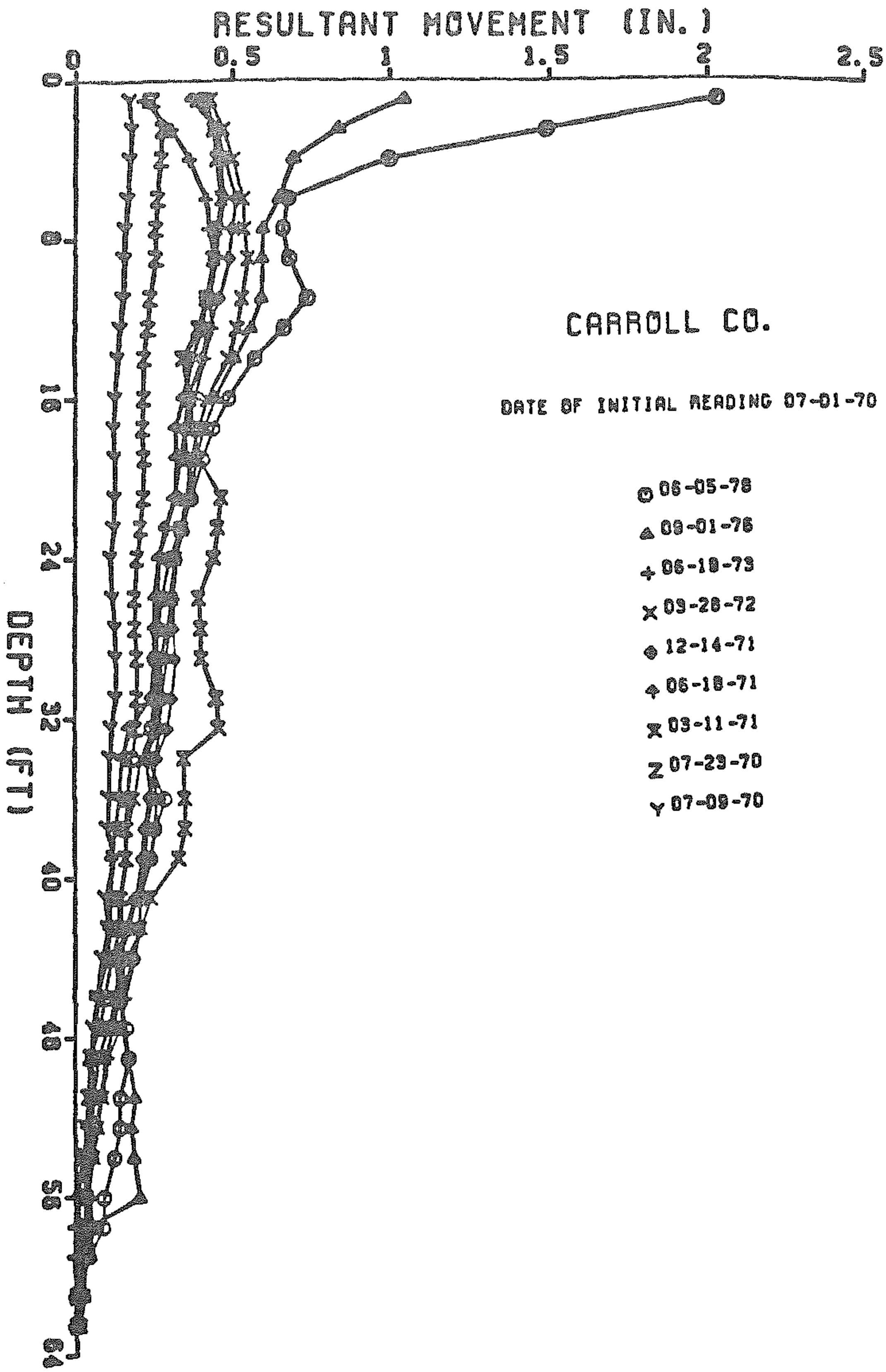


Figure 31. Slope Inclinometer 1, I 71 over the Kentucky River.

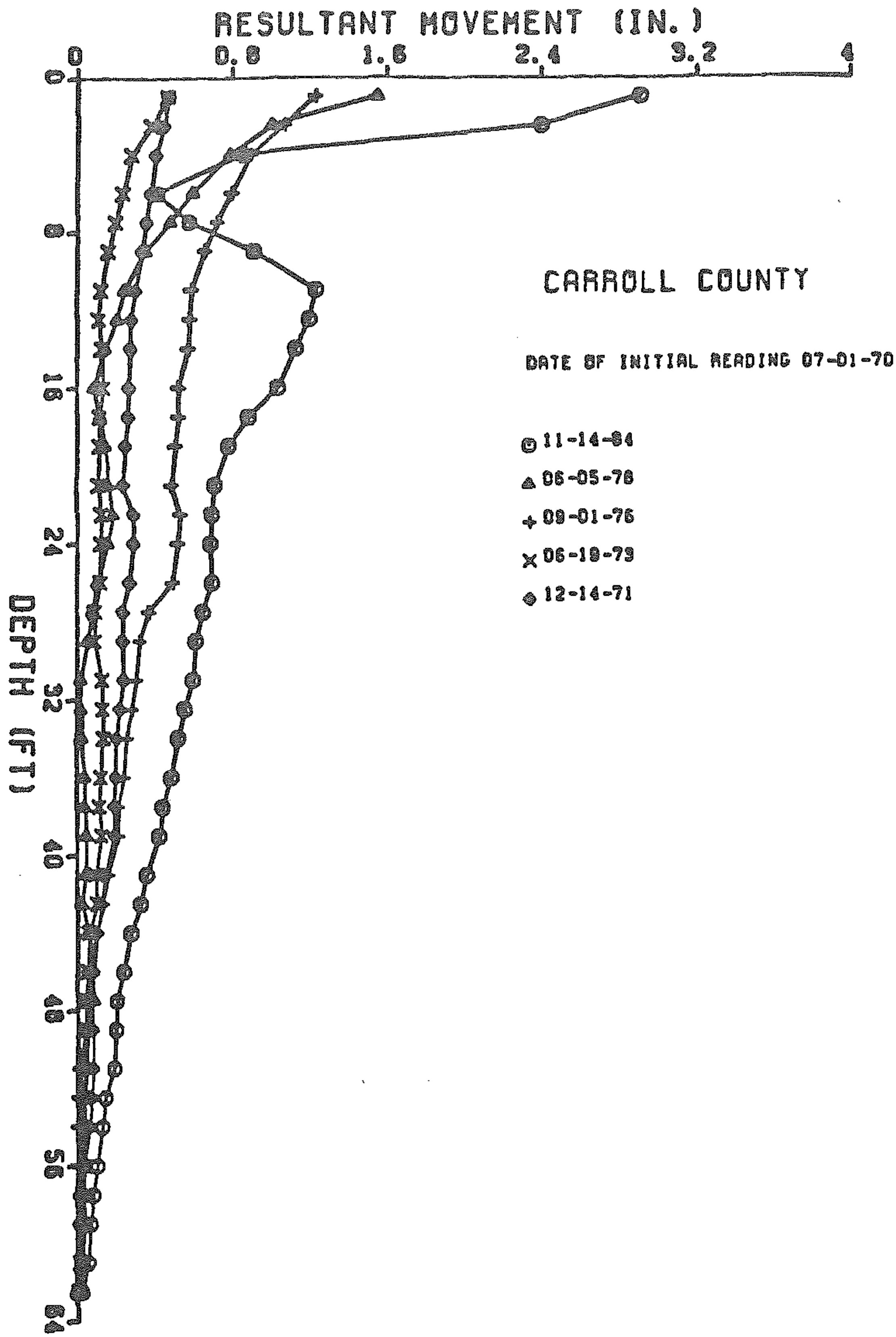


Figure 32. Slope Inclinometer 2, I 71 over the Kentucky River.

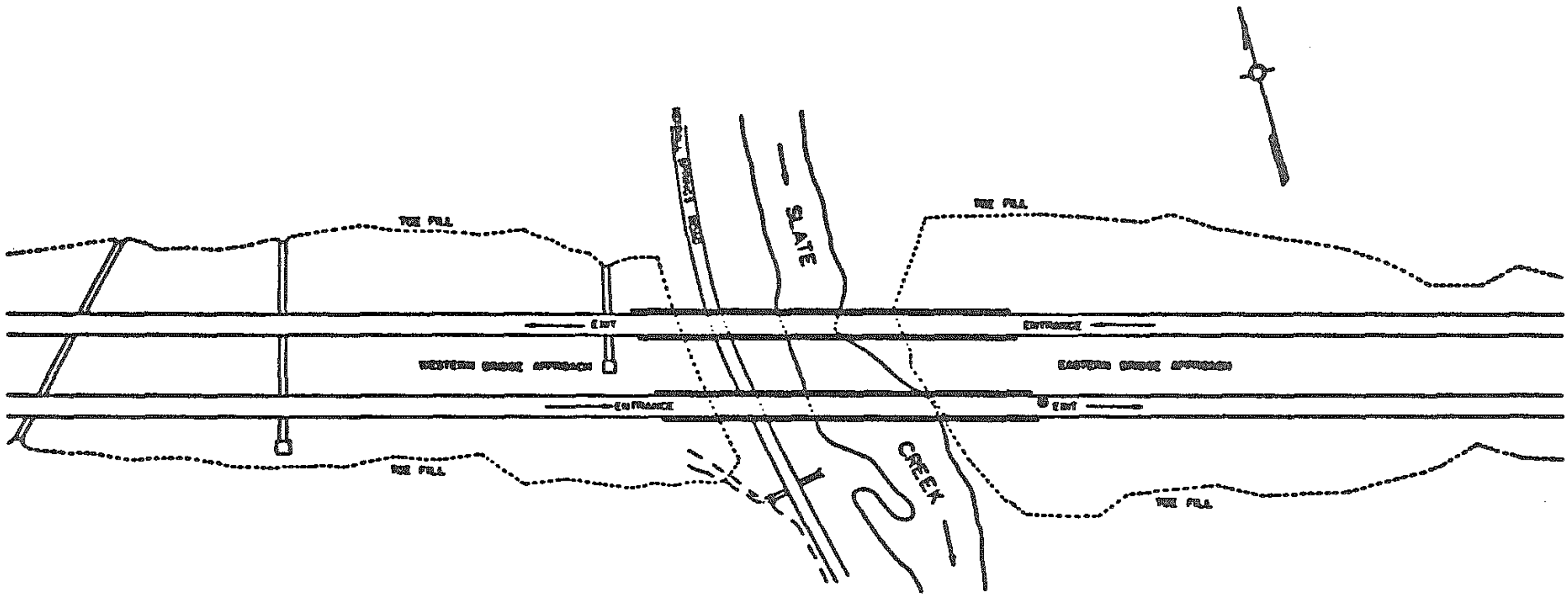


Figure 33. I 64 over Slate Creek.

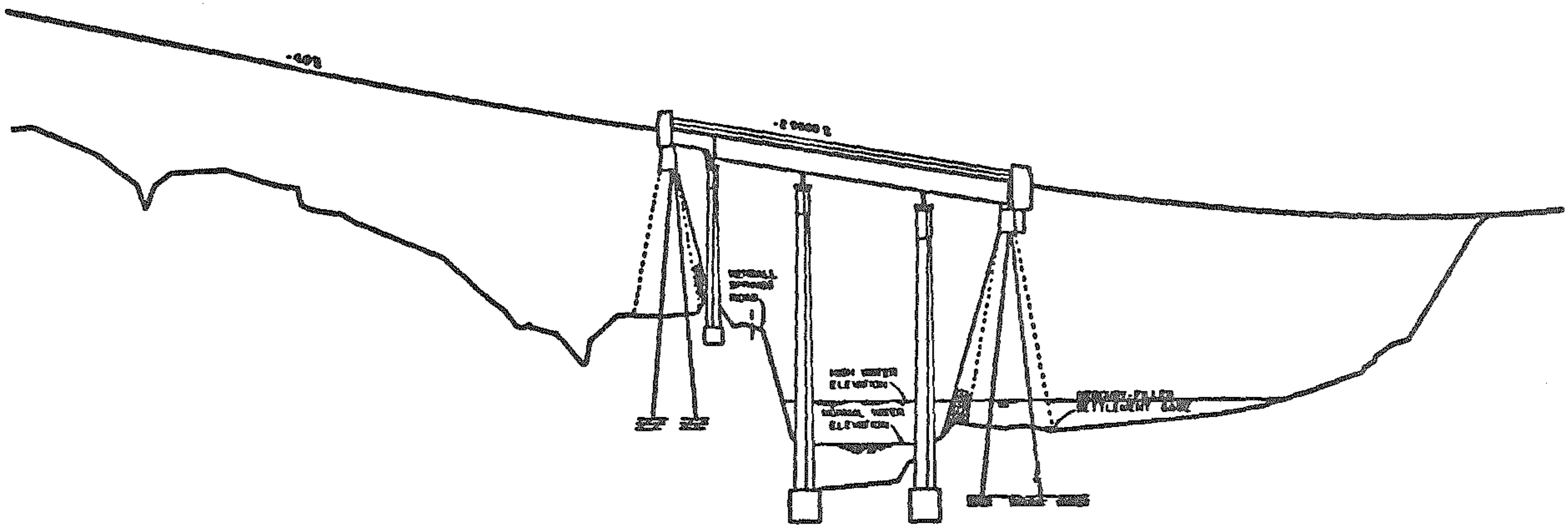


Figure 34. Centerline Section at I 64 over Slate Creek.

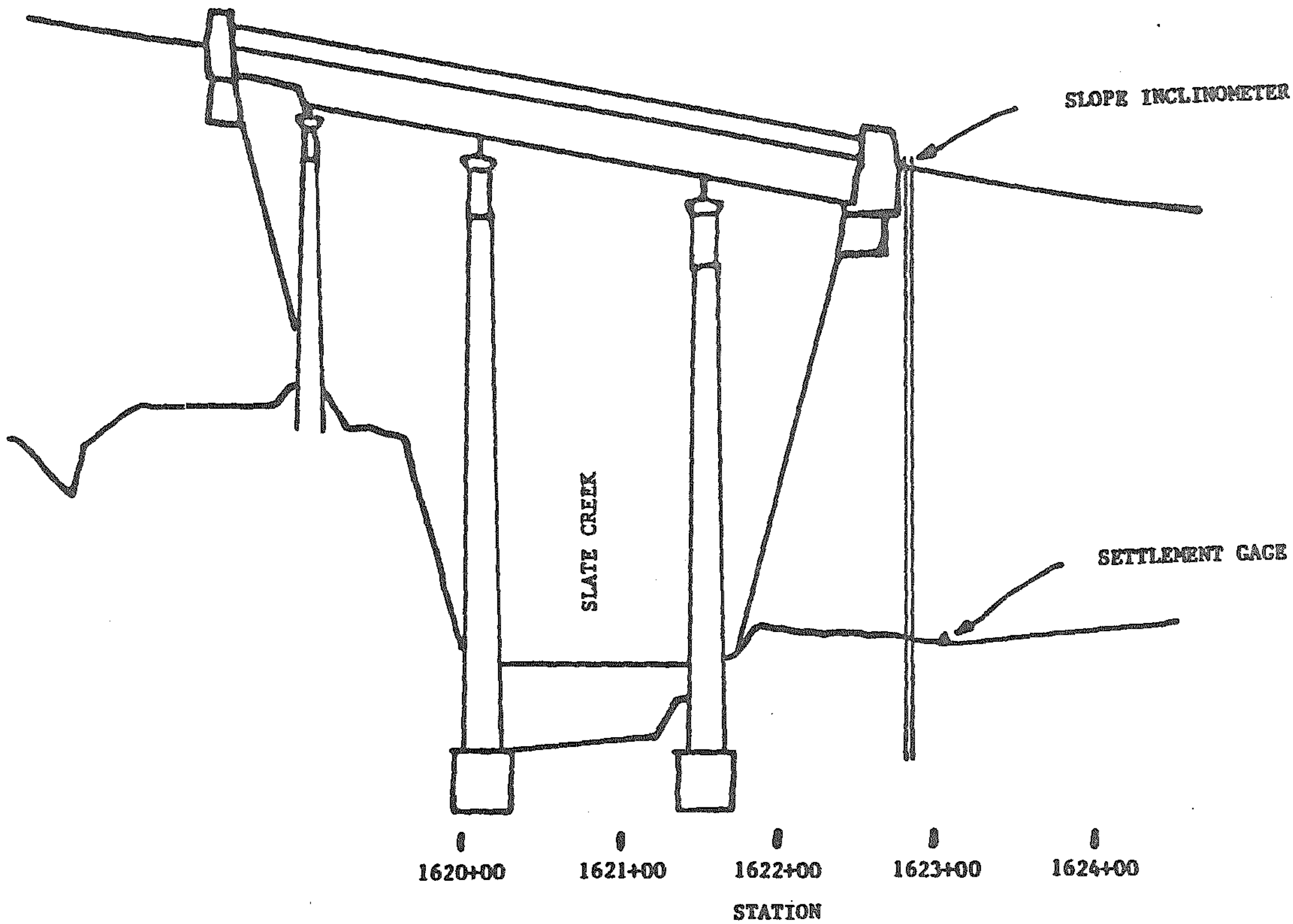


Figure 35. Instrumentation of I 64 over Slate Creek.

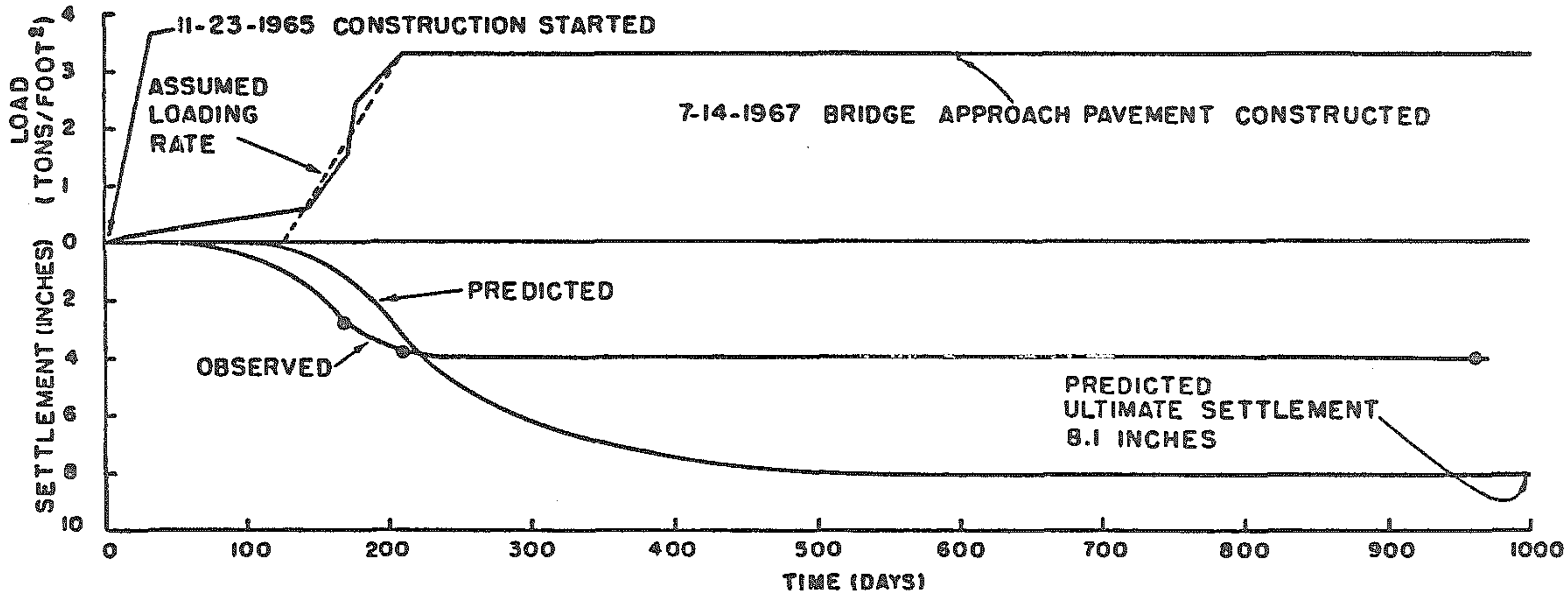


Figure 36. Observed and Predicted Time-Settlement Curves, 42 feet right of Centerline, Station 622, I 64 over Slate Creek, Bath County.

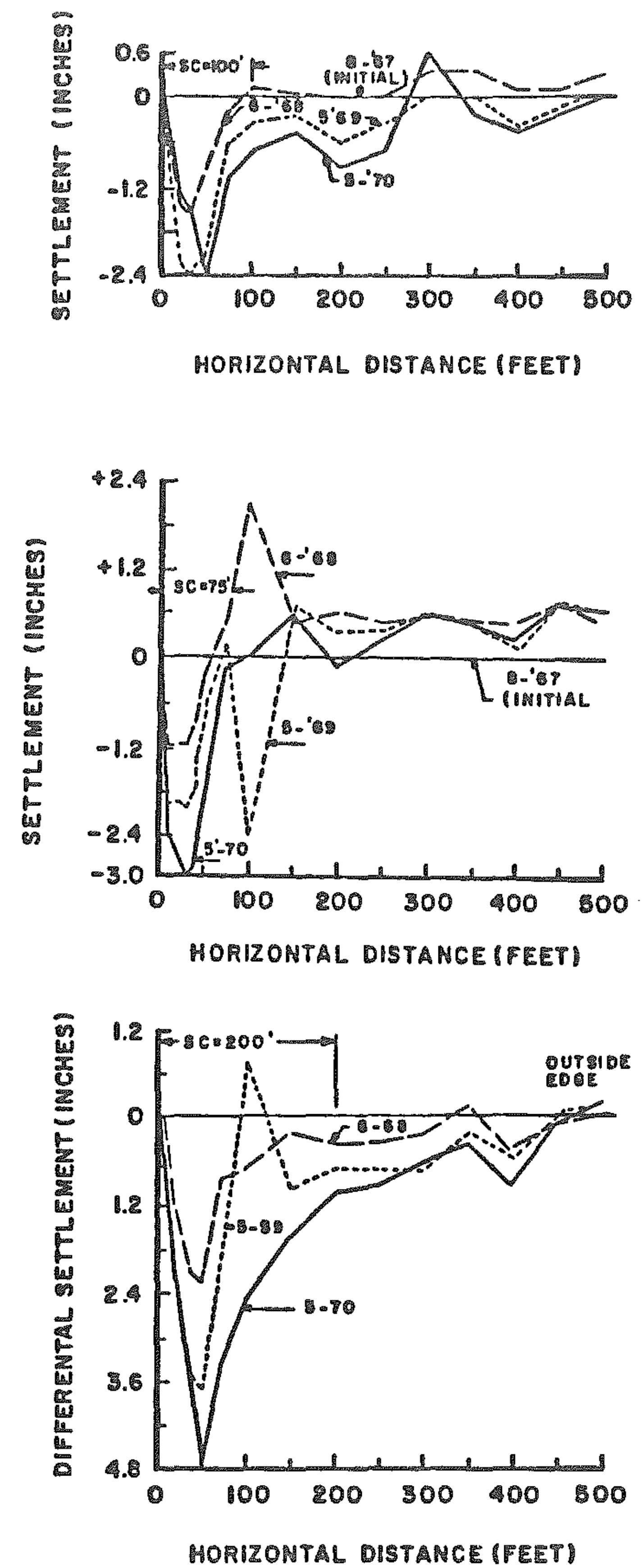


Figure 37. Settlement of Eastbound Approach Pavement, I 64 over Slate Creek.

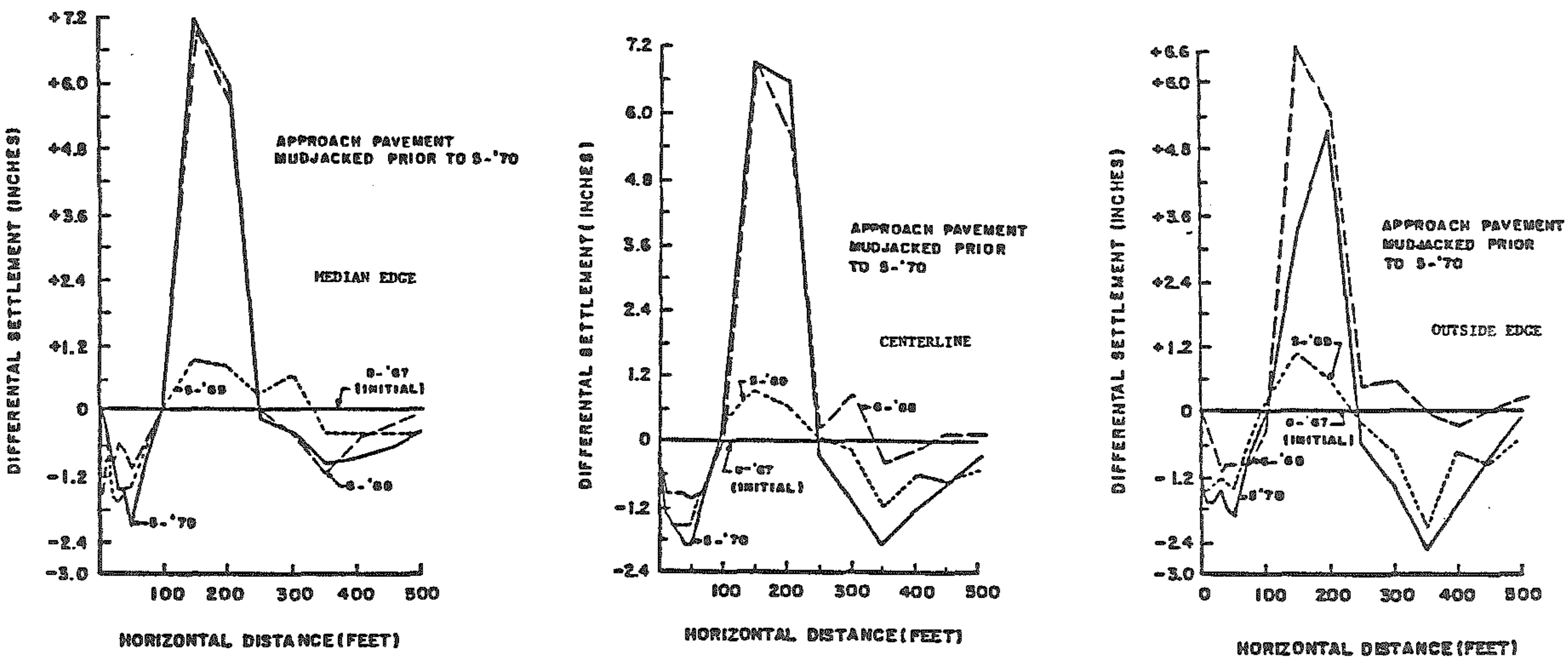


Figure 38. Settlement of Westbound Approach Pavement, I 64 over Slate Creek.

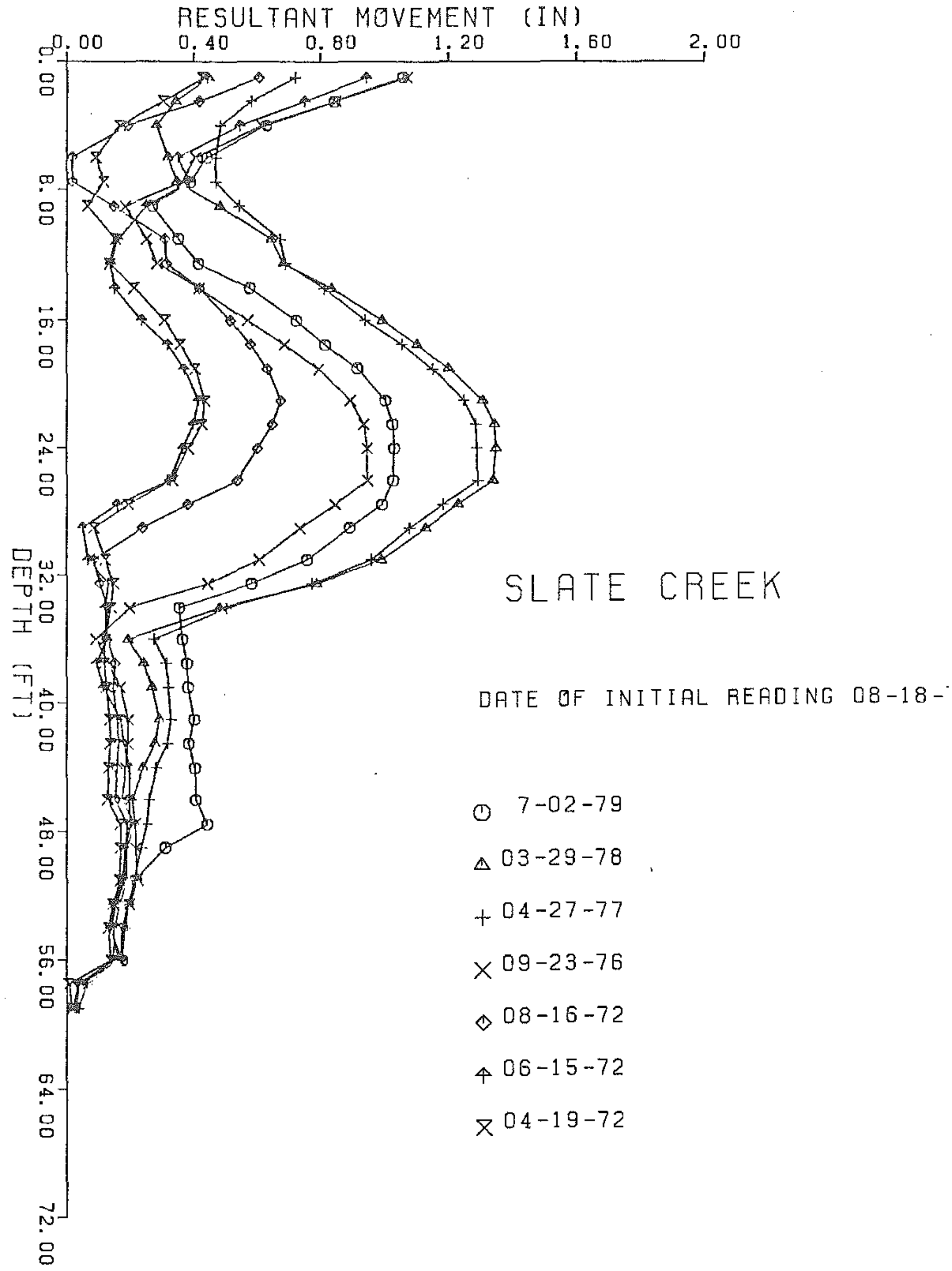


Figure 39. Slope Inclinometer at I 64 over Slate Creek.

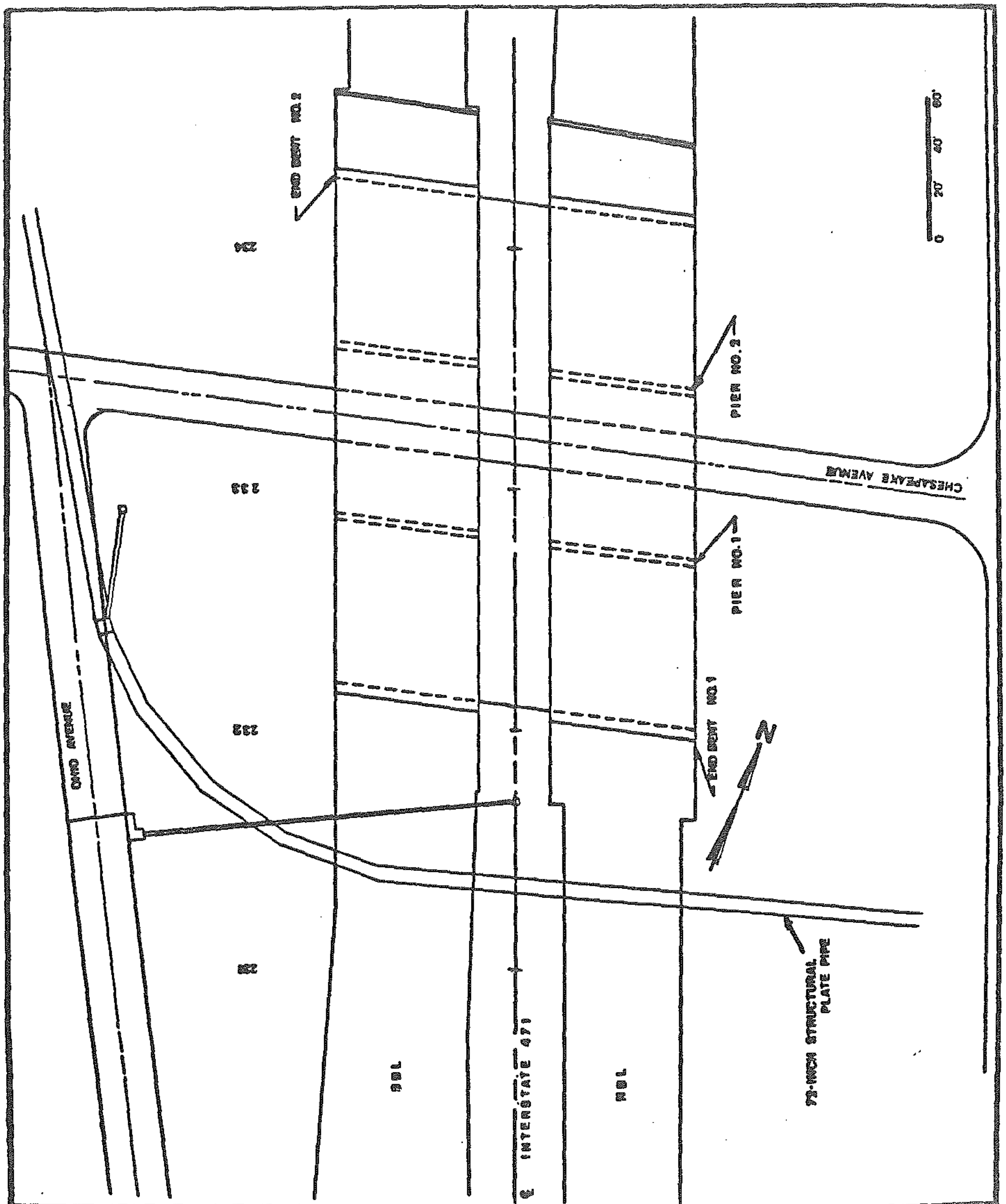


Figure 40. I 71 over Chesapeake Avenue.

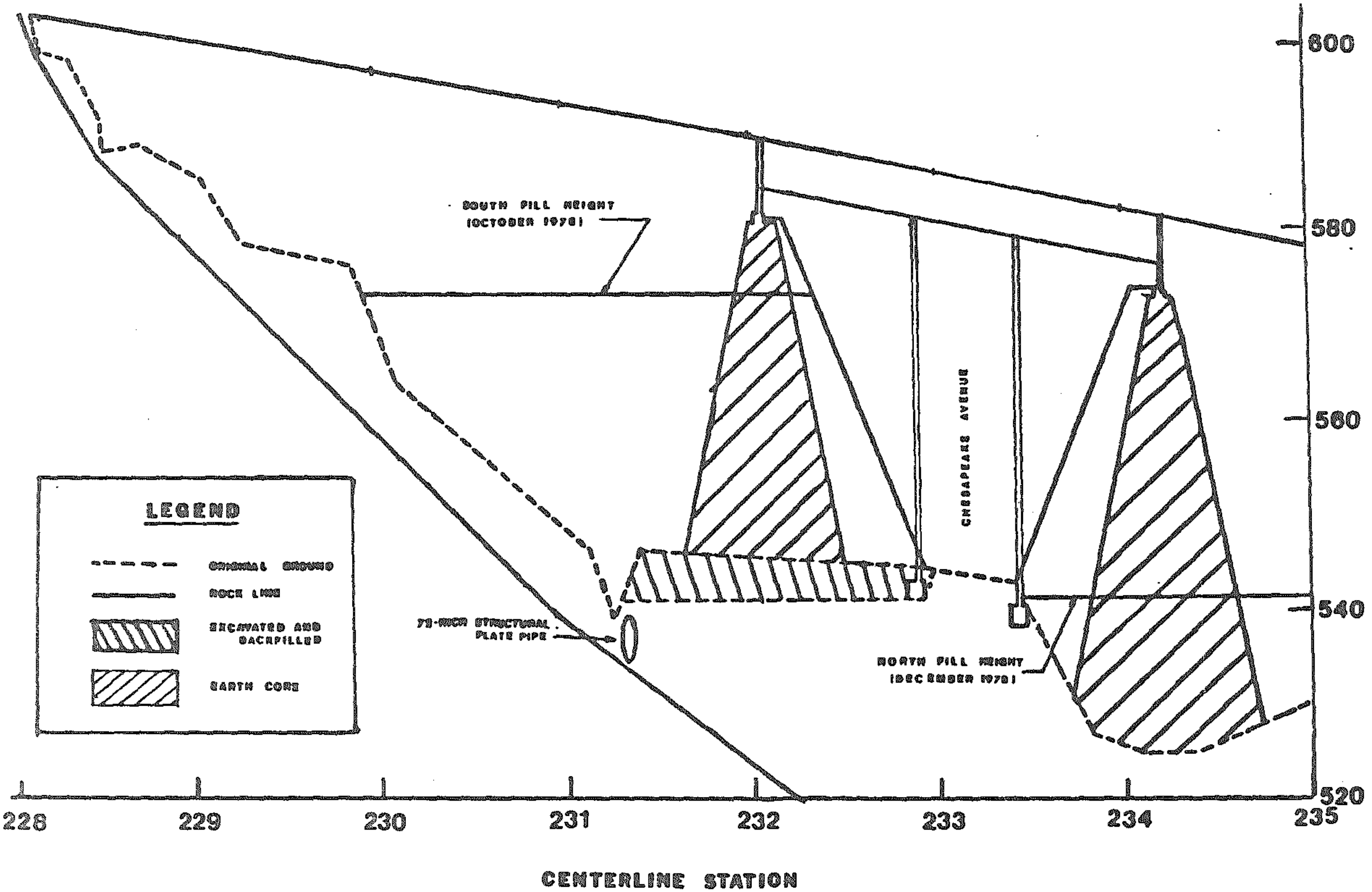


Figure 41. Centerline Section at I 471 over Chesapeake Avenue.

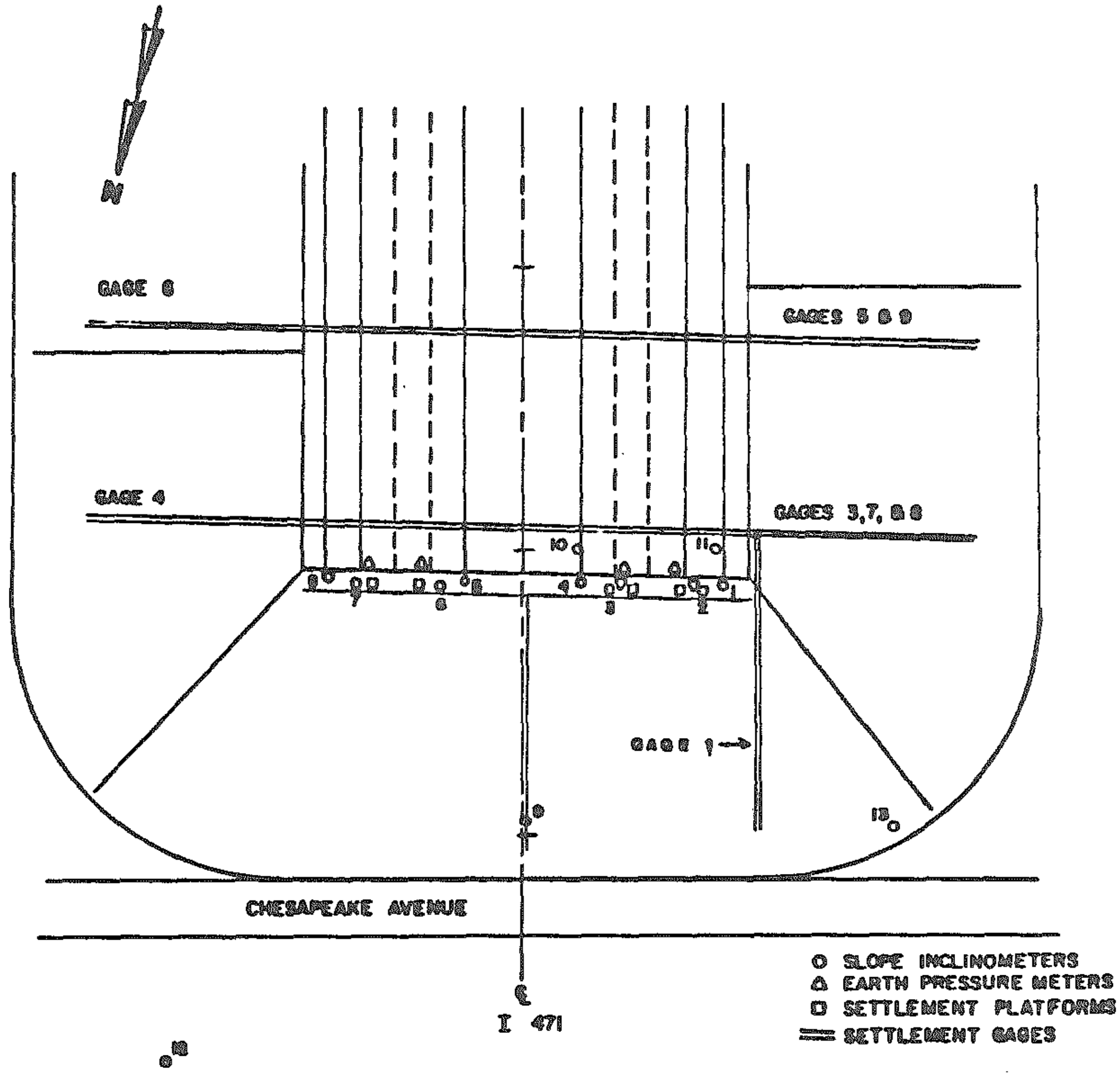


Figure 42. Location of Instrumentation at I 471 over Chesapeake Avenue.

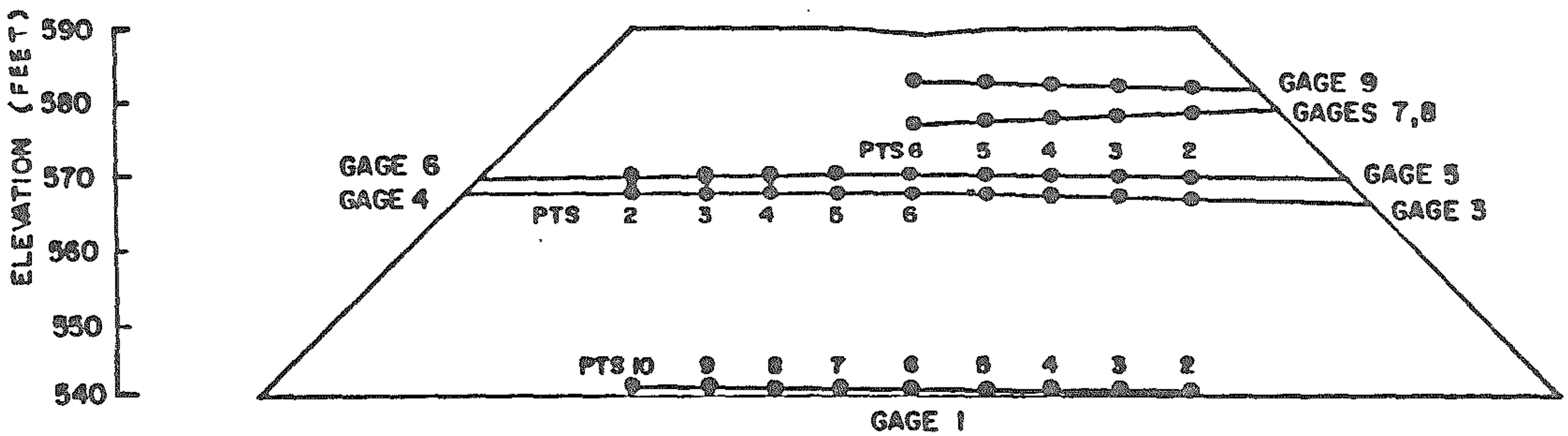


Figure 43. Cross Section of South Bridge Approach with Settlement Gage Location, I 471 over Chesapeake Avenue.

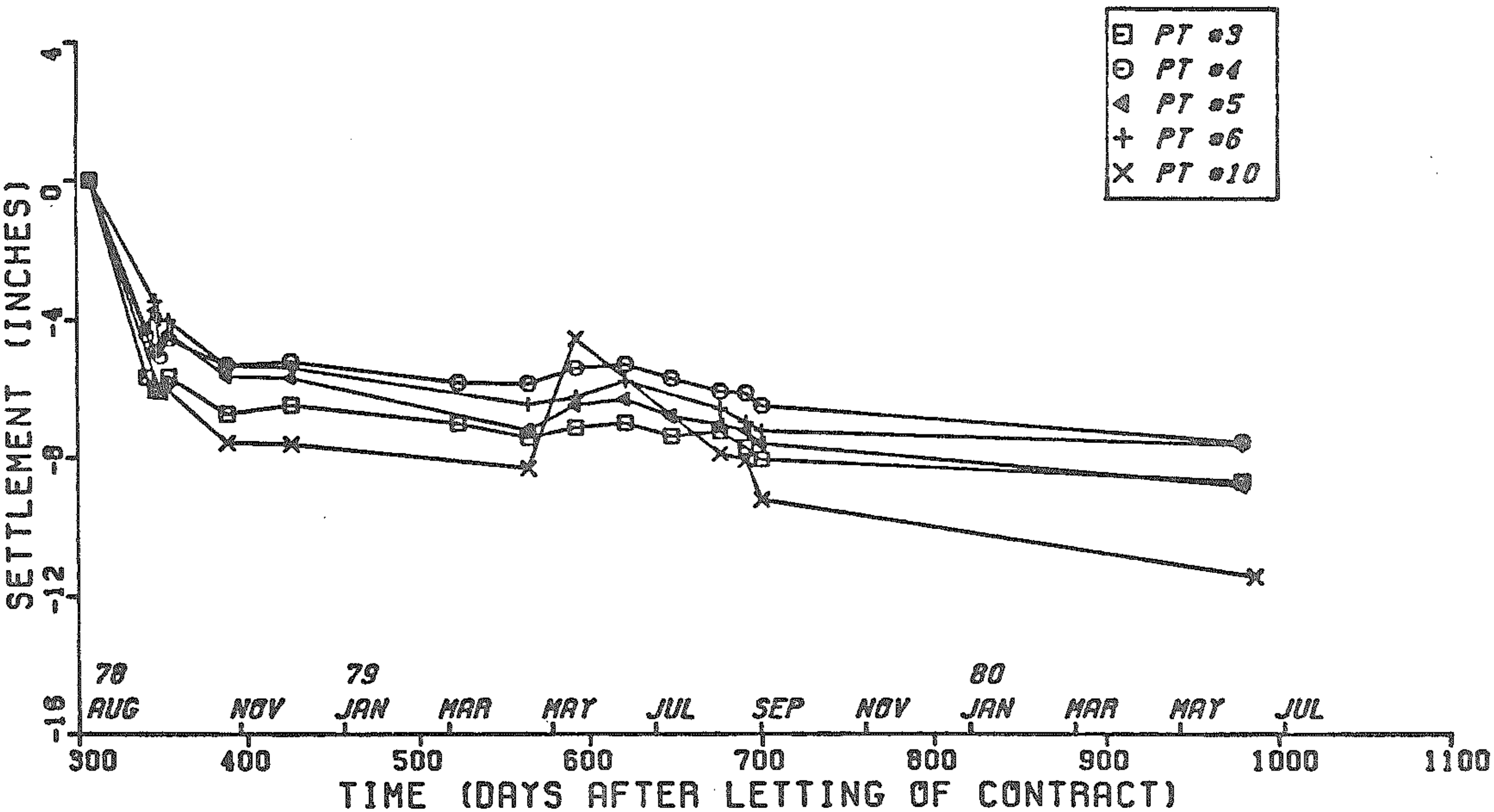


Figure 44. Settlement (Gage 1) Versus Time, I 471 over Chesapeake Avenue.

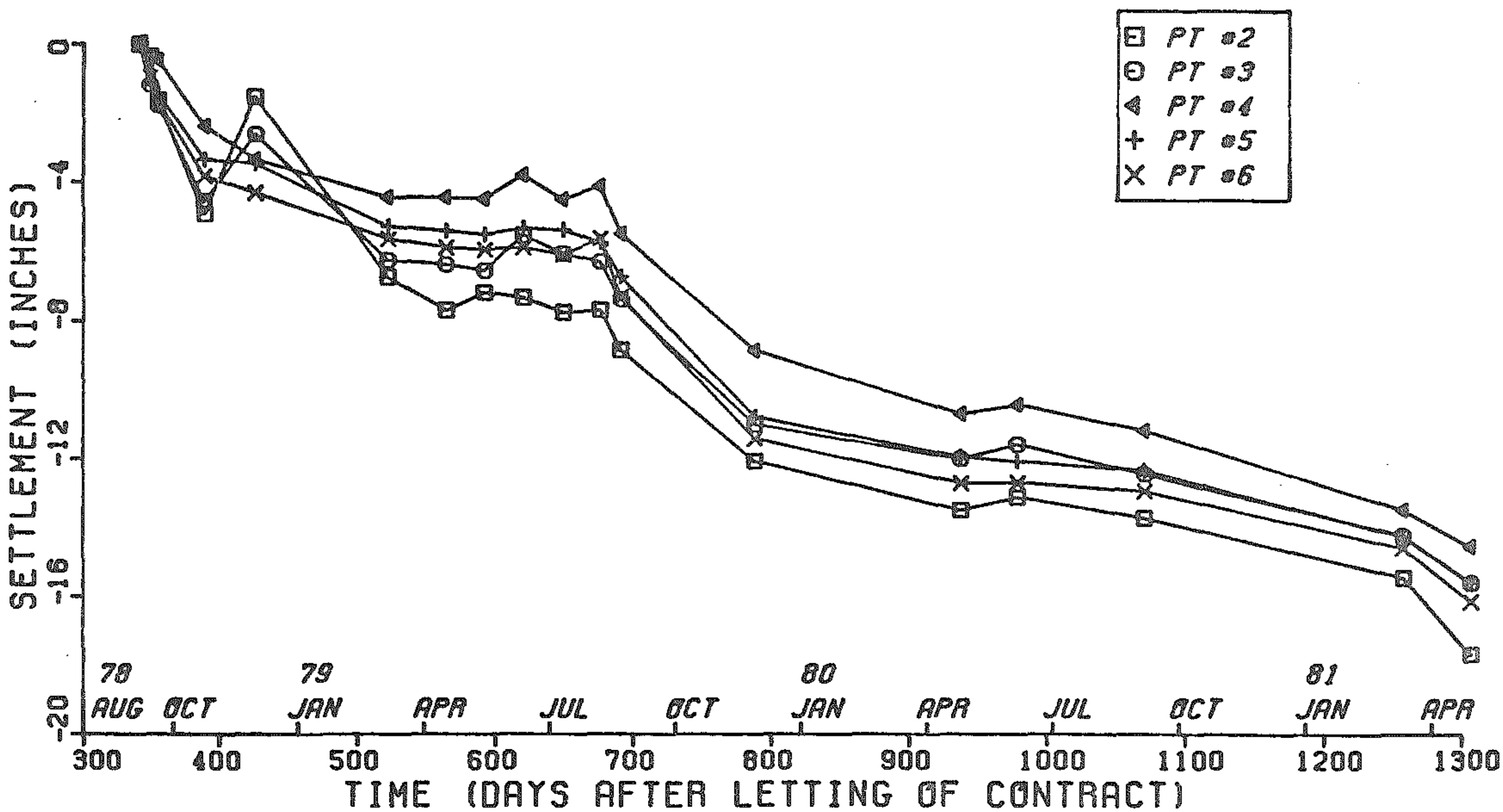


Figure 45. Settlement (Gage 3) Versus Time, I 471 over Chesapeake Avenue.

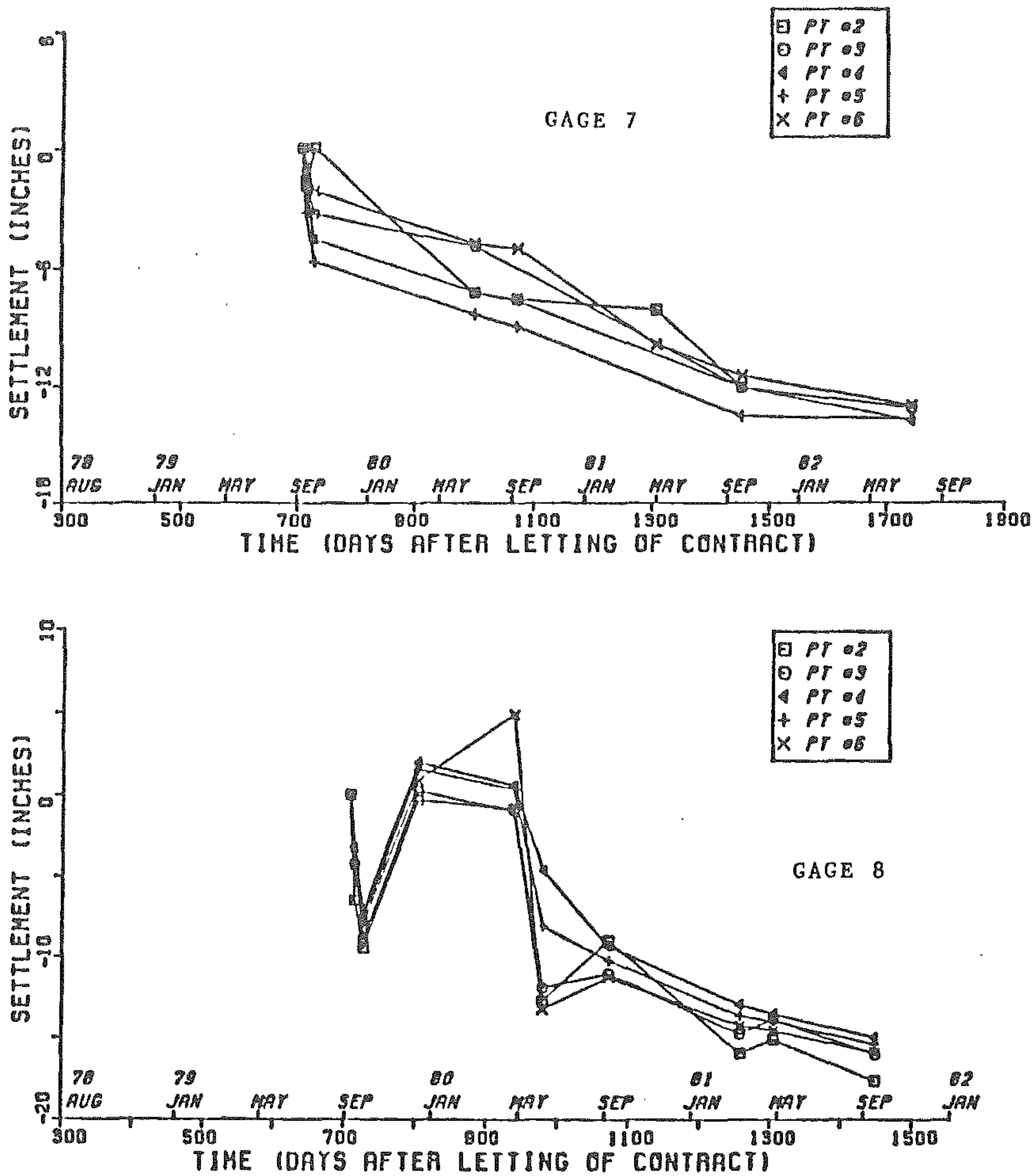


Figure 46. Settlement Versus Time, I 471 over Chesapeake Avenue.

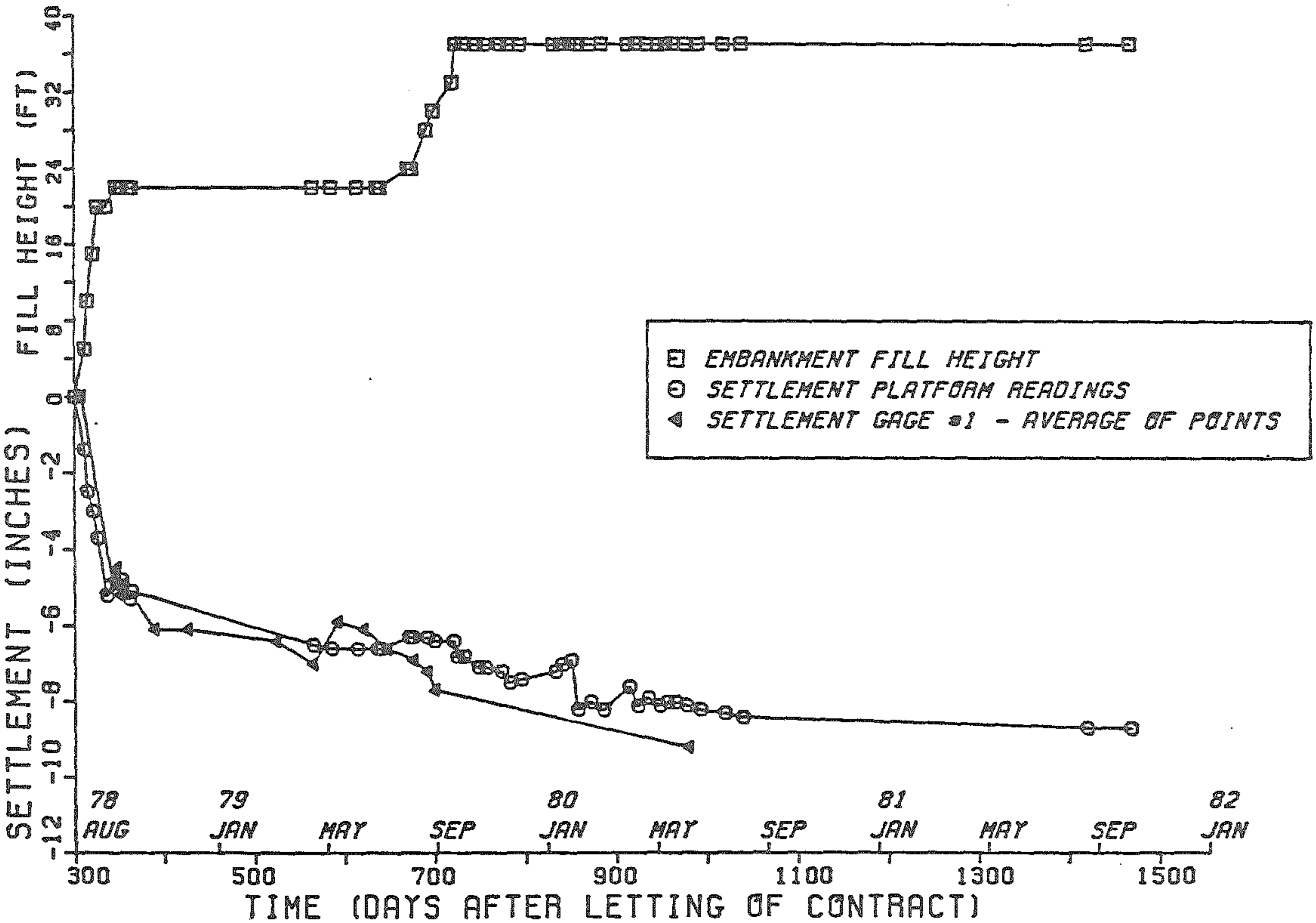


Figure 47. Fill Height and Settlement Versus Time, I 471 over Chesapeake Avenue.

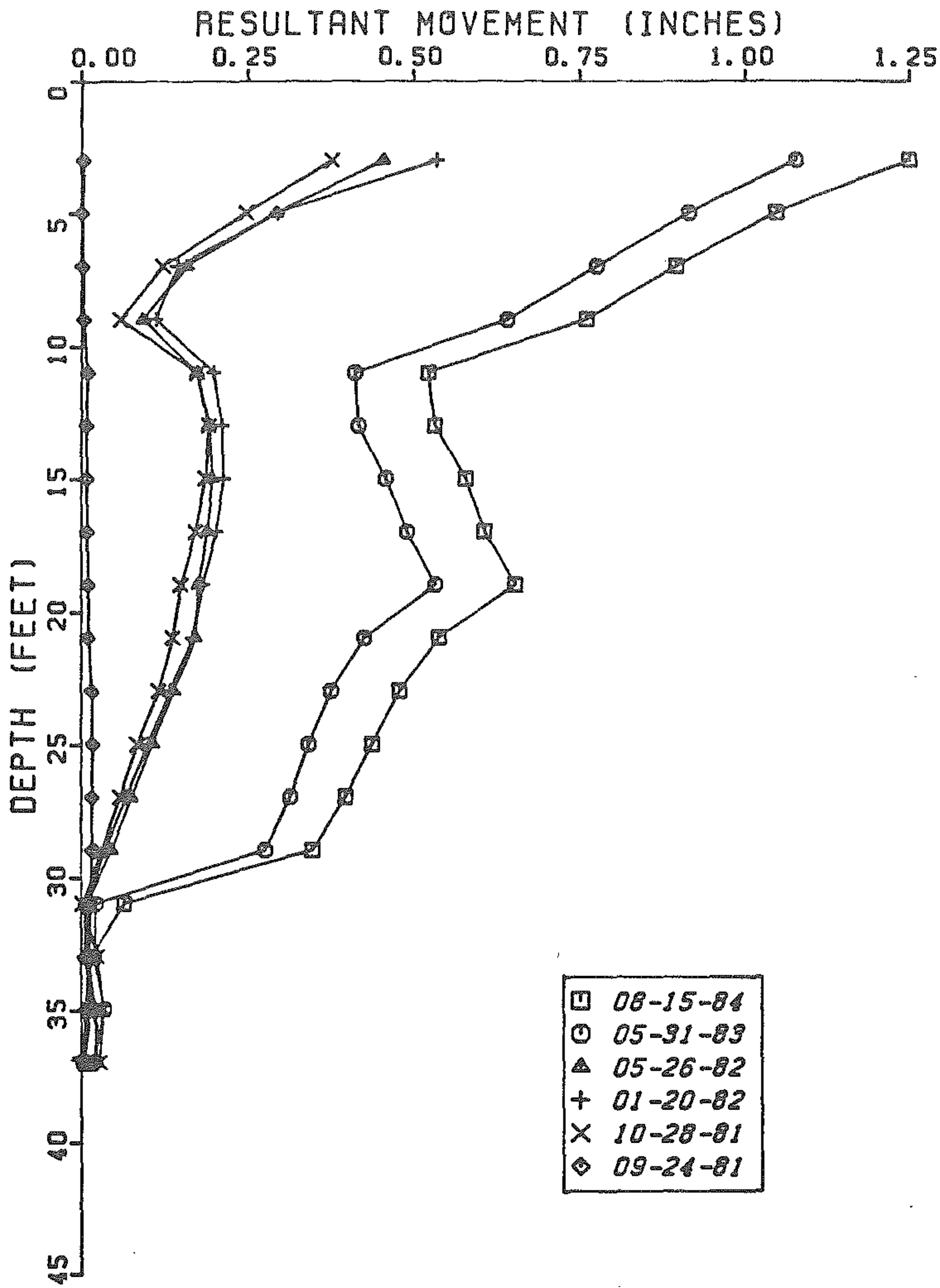


Figure 48. Movement Versus Depth (Slope Inclinometer 9), I 471 over Chesapeake Avenue.

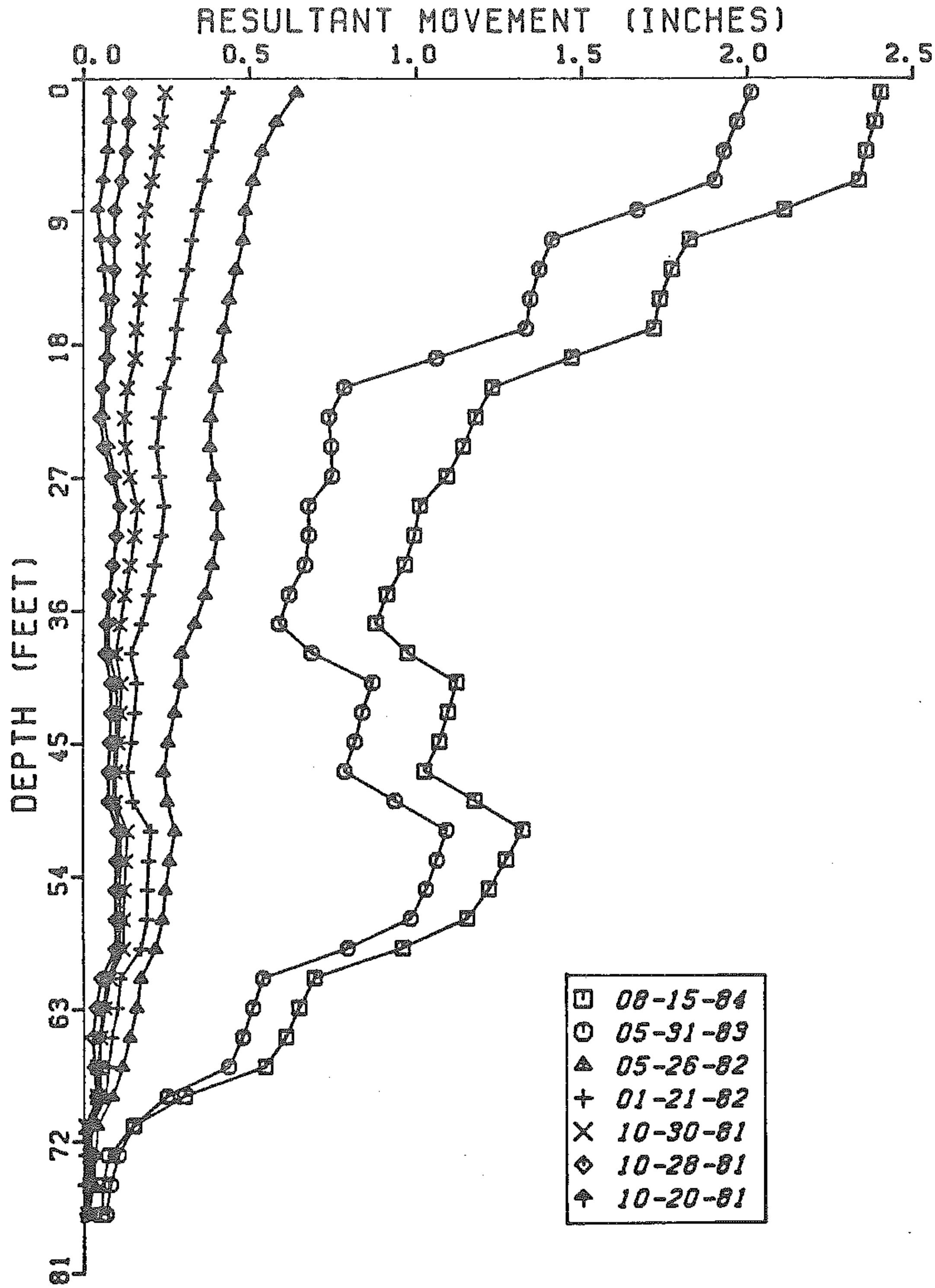


Figure 49. Movement Versus Depth (Slope Inclinometer 10), I 471 over Chesapeake Avenue.

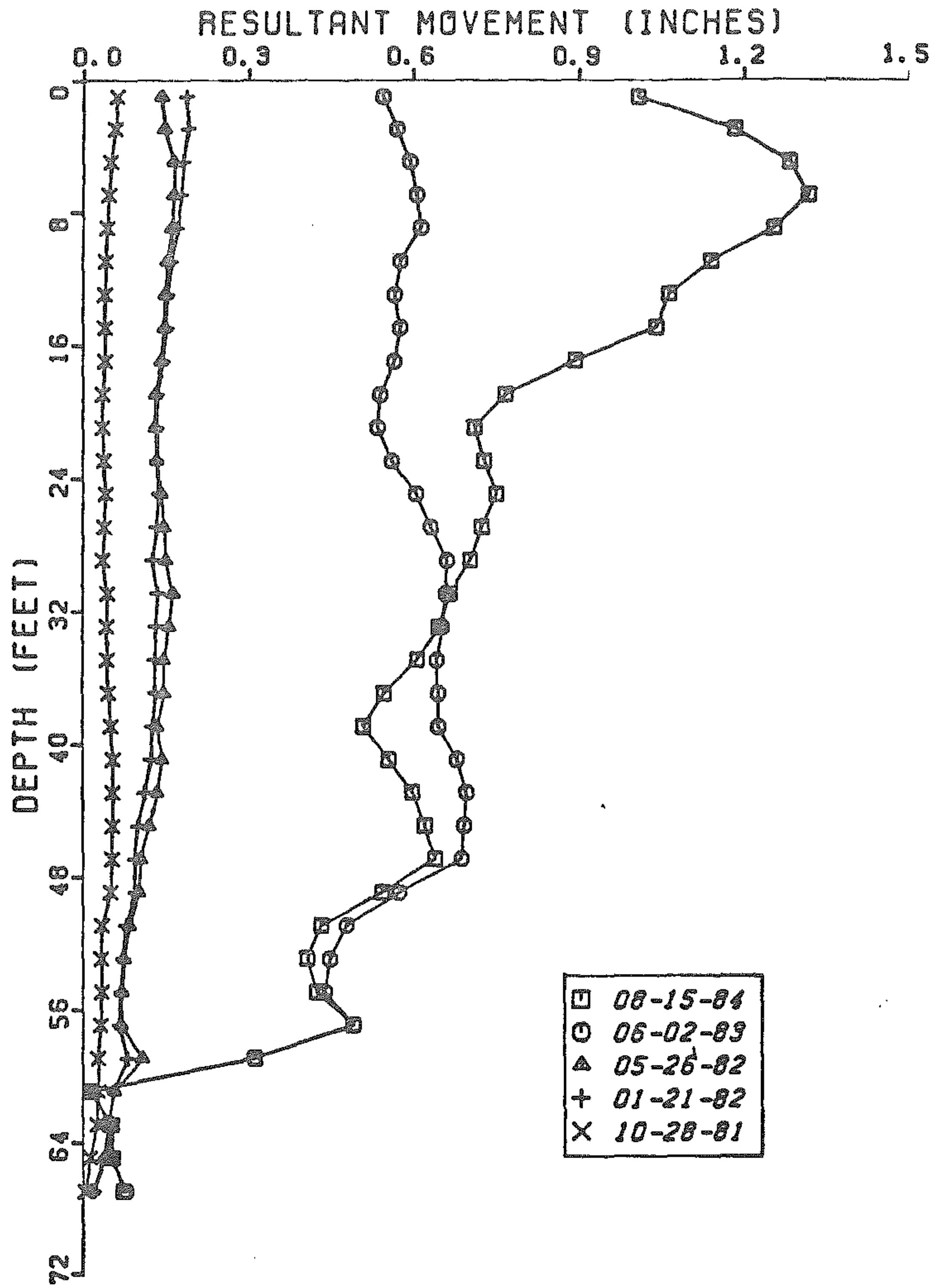


Figure 50. Movement Versus Depth (Slope Inclinometer 11), I 471 over Chesapeake Avenue.

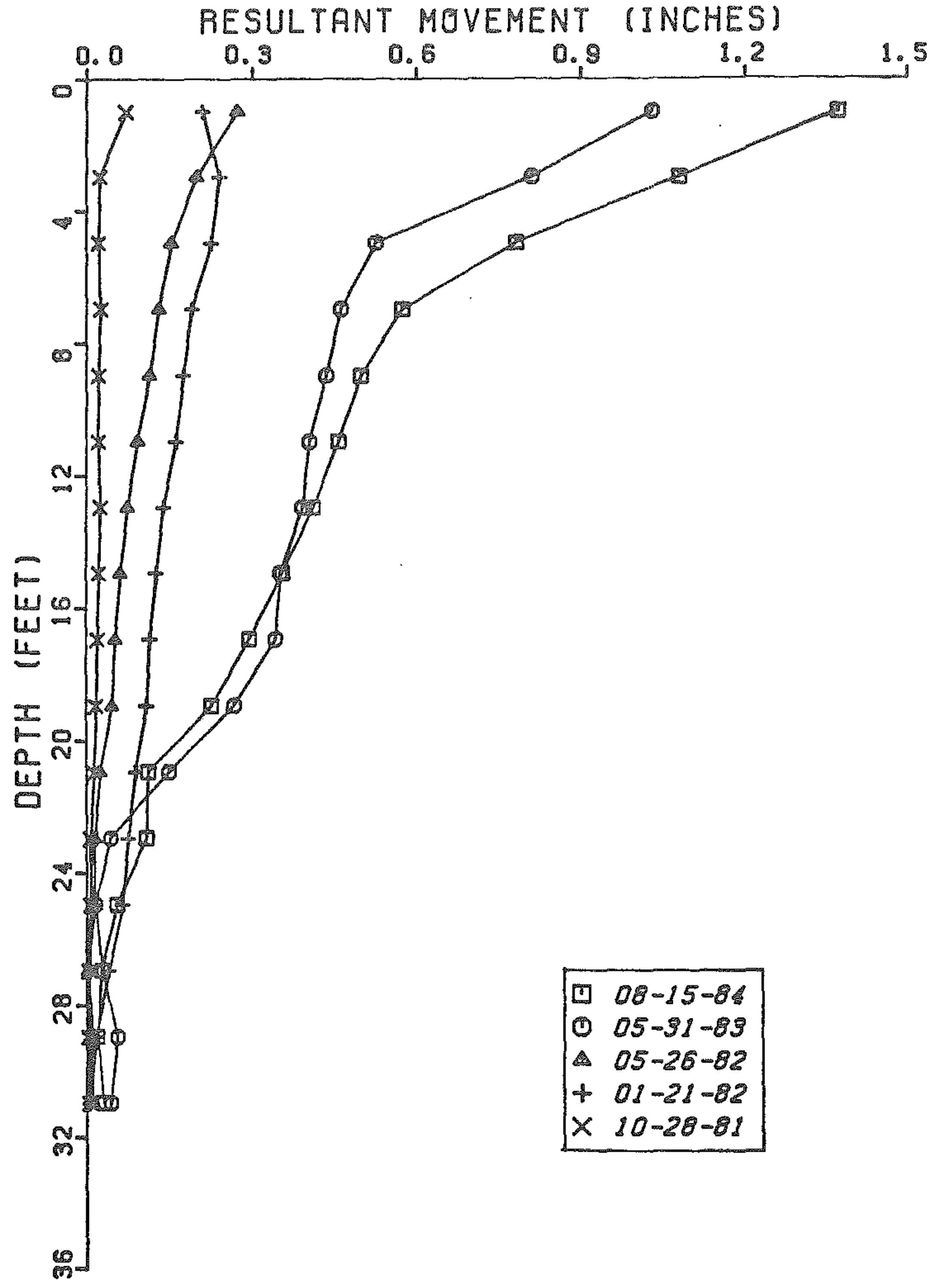


Figure 51. Movement Versus Depth (Slope Inclinometer 12), I 471 over Chesapeake Avenue.

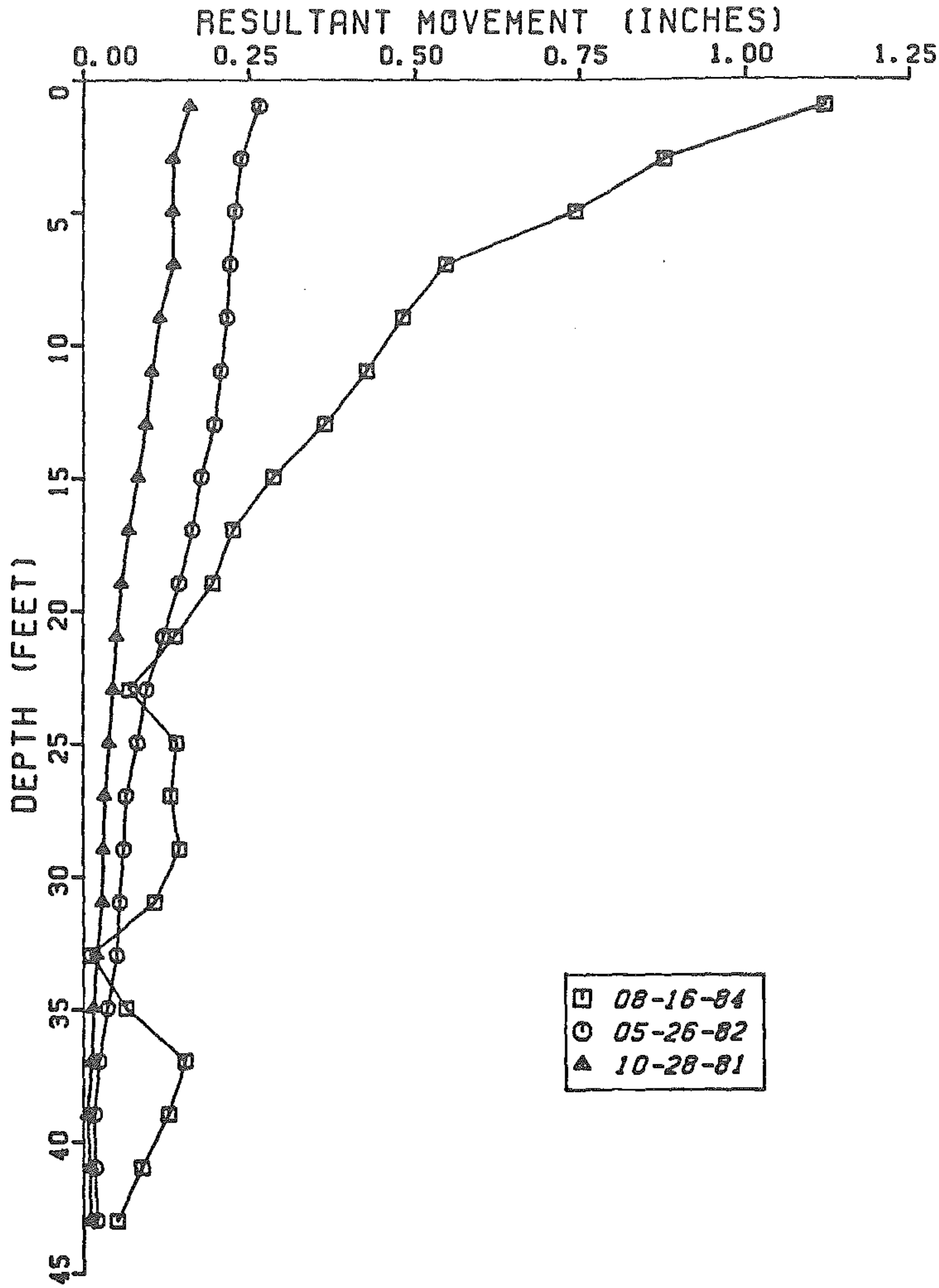


Figure 52. Movement Versus Depth (Slope Inclinometer 13), I 471 over Chesapeake Avenue.

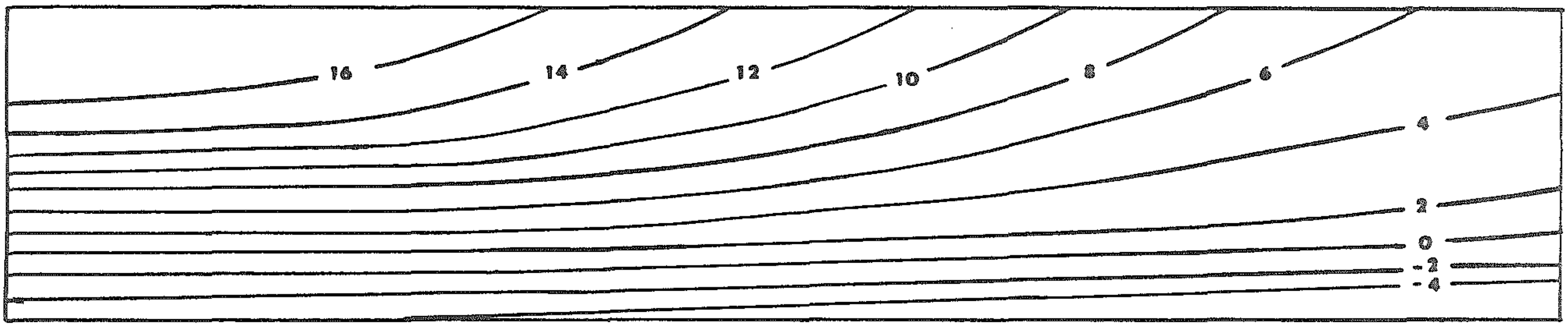


Figure 53. Earth Pressure (psi) Distribution on Vertical Face of South Approach End Bent of Southbound Lanes, I 471 over Chesapeake Avenue.

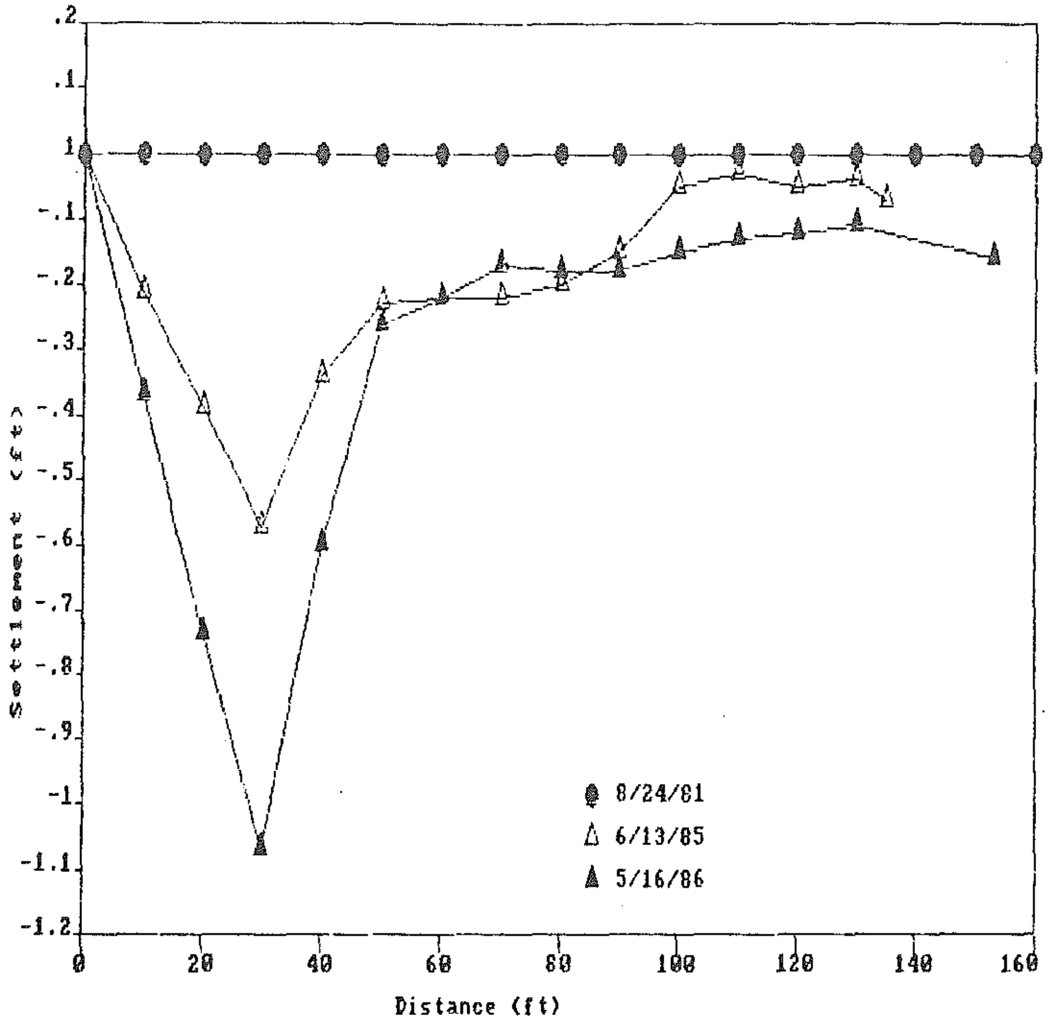


Figure 54. Pavement Settlement, Outside Edge, at South Approach at I 471 over Chesapeake Avenue.

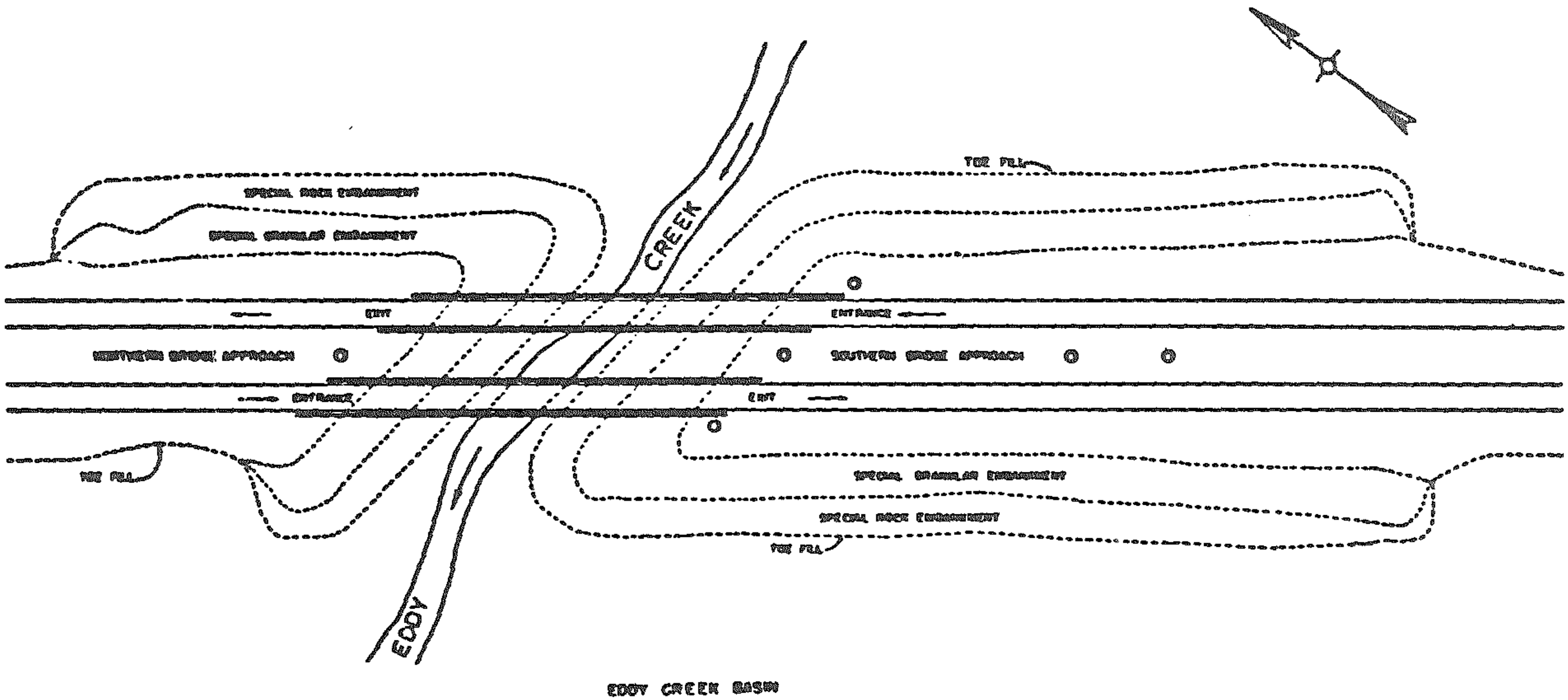


Figure 55. I 24 over Eddy Creek.

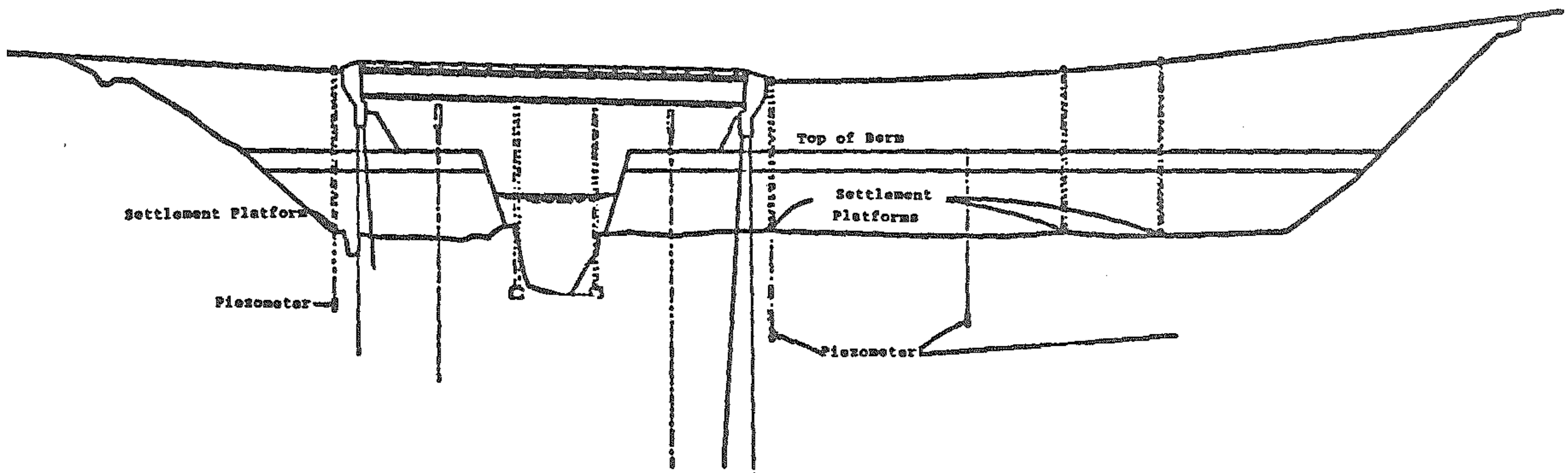


Figure 56. Centerline Section at I 24 over Eddy Creek.

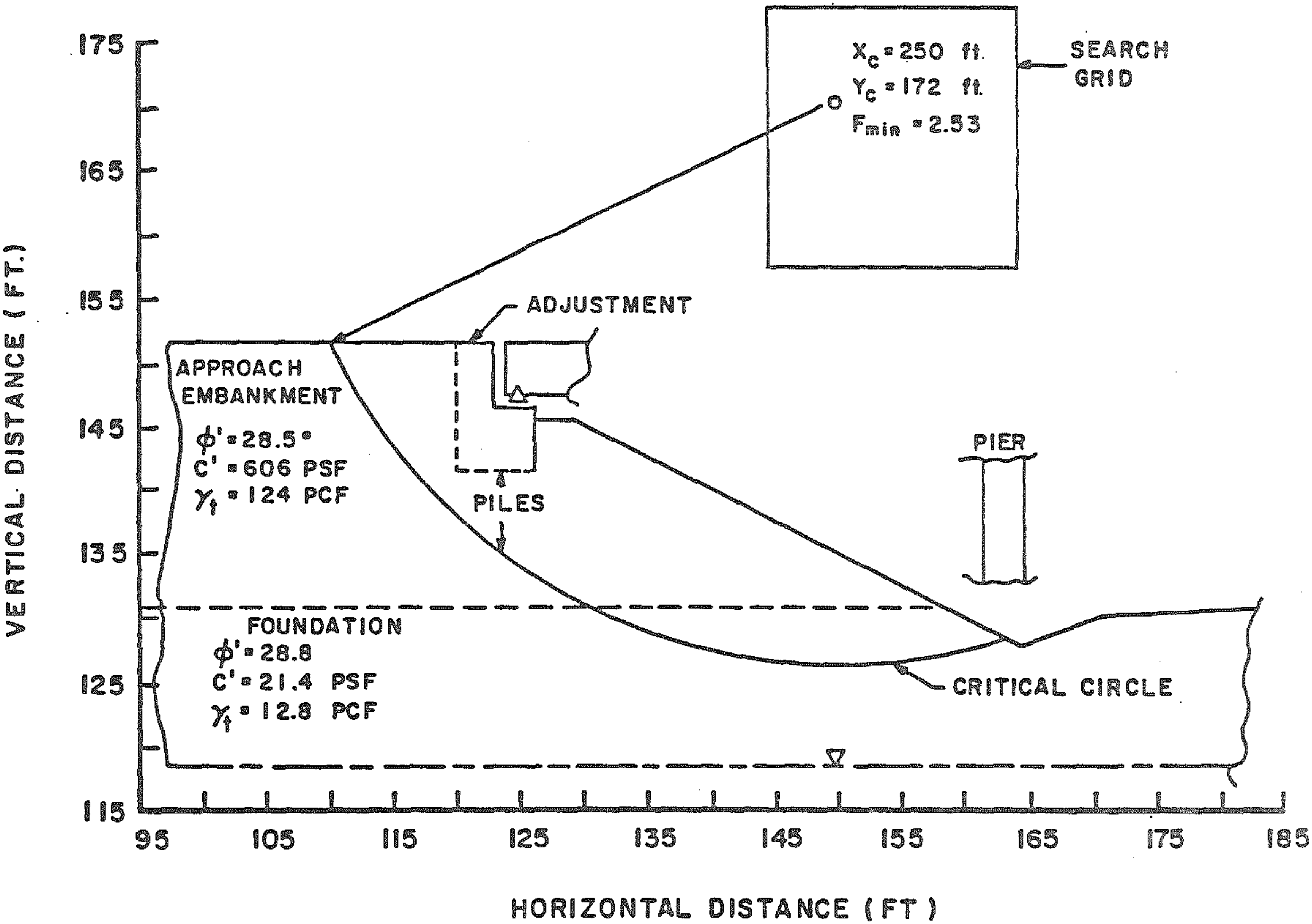


Figure 57. Stability Analysis Section, Parkersmill Road.

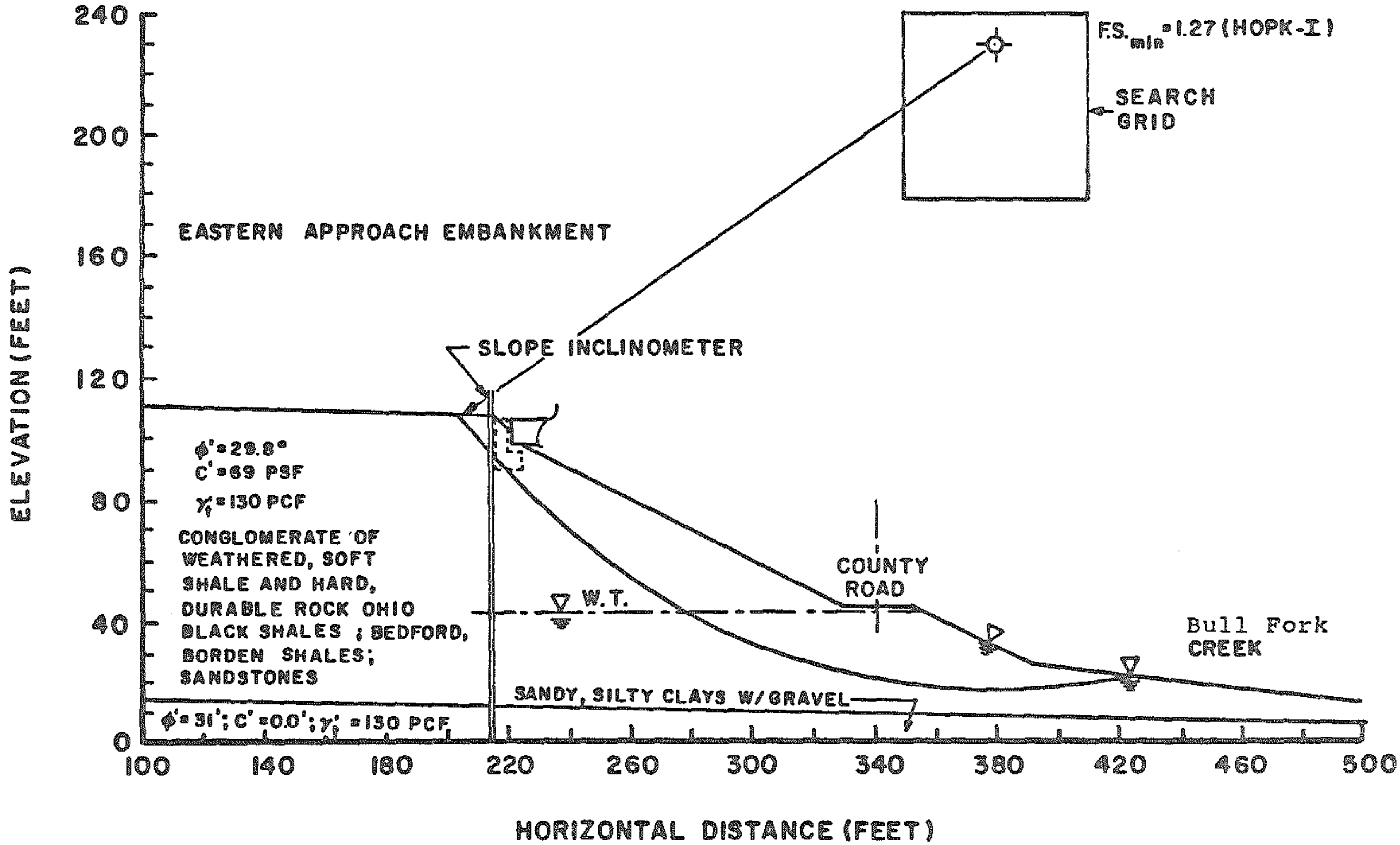


Figure 58. Stability Analysis Section of Bull Fork Creek Eastern Approach Embankment.

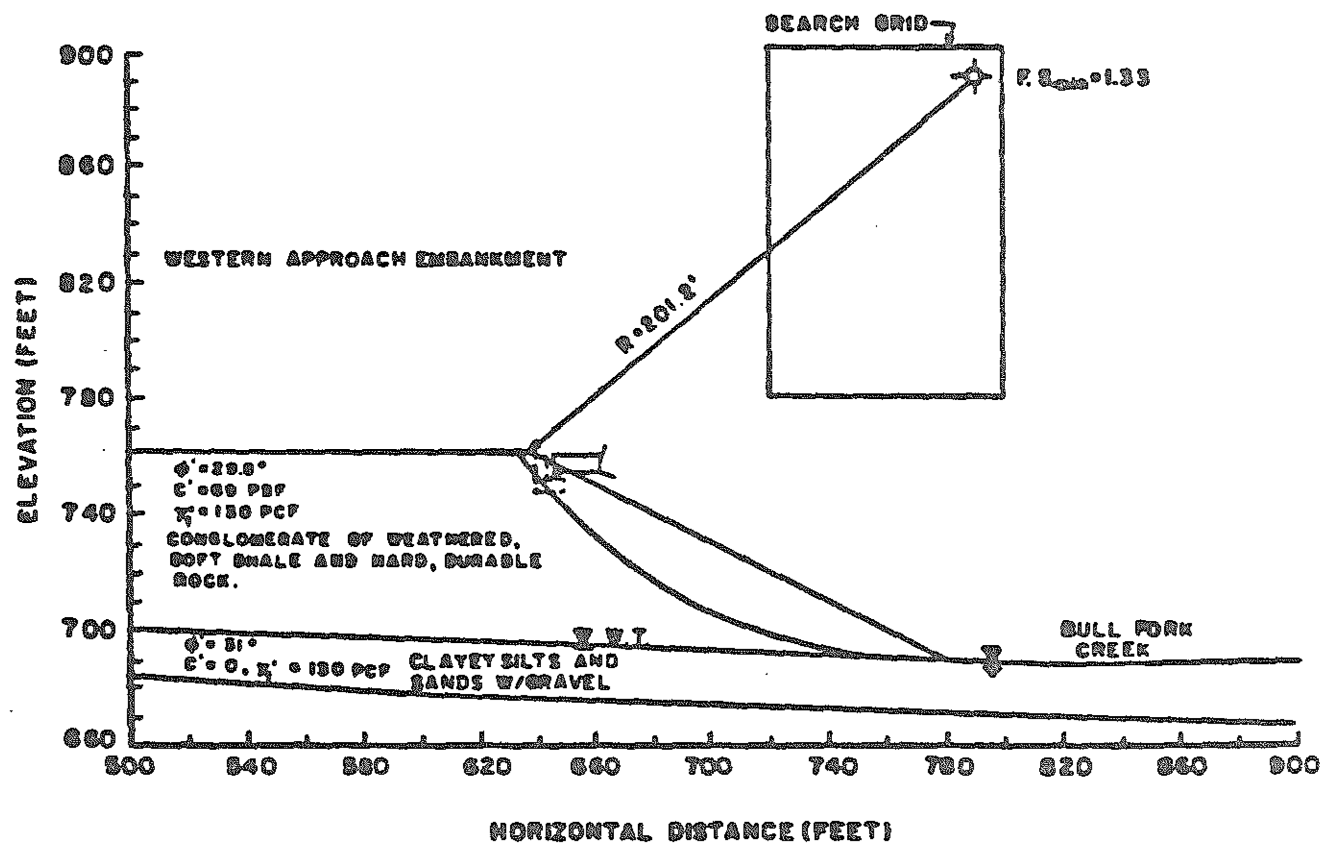


Figure 58A. Stability Analysis Section (along Centerline) of the Western Approach at Bull Fork Creek.

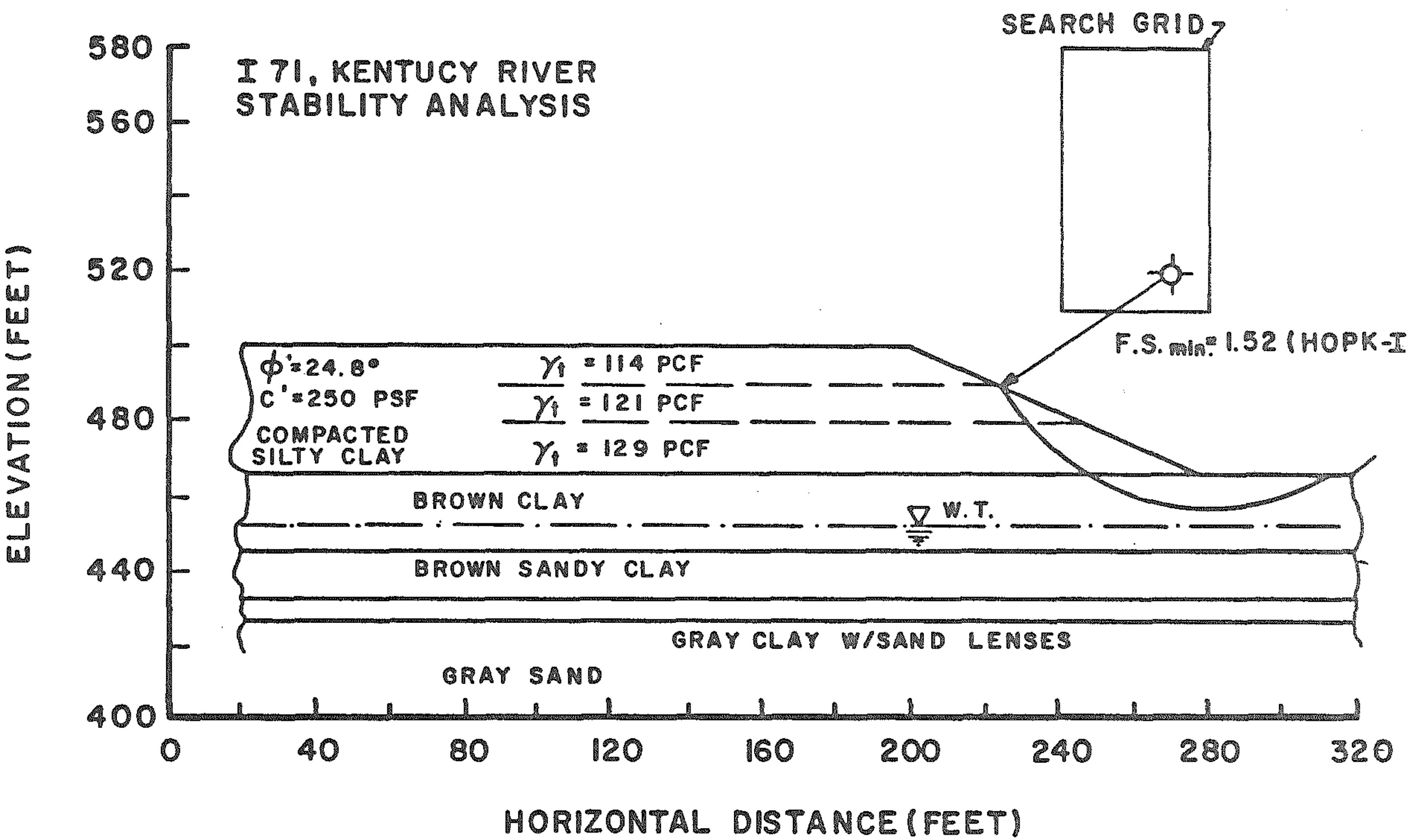


Figure 59. Stability Analysis Section along Centerline, I 71 over the Kentucky River.

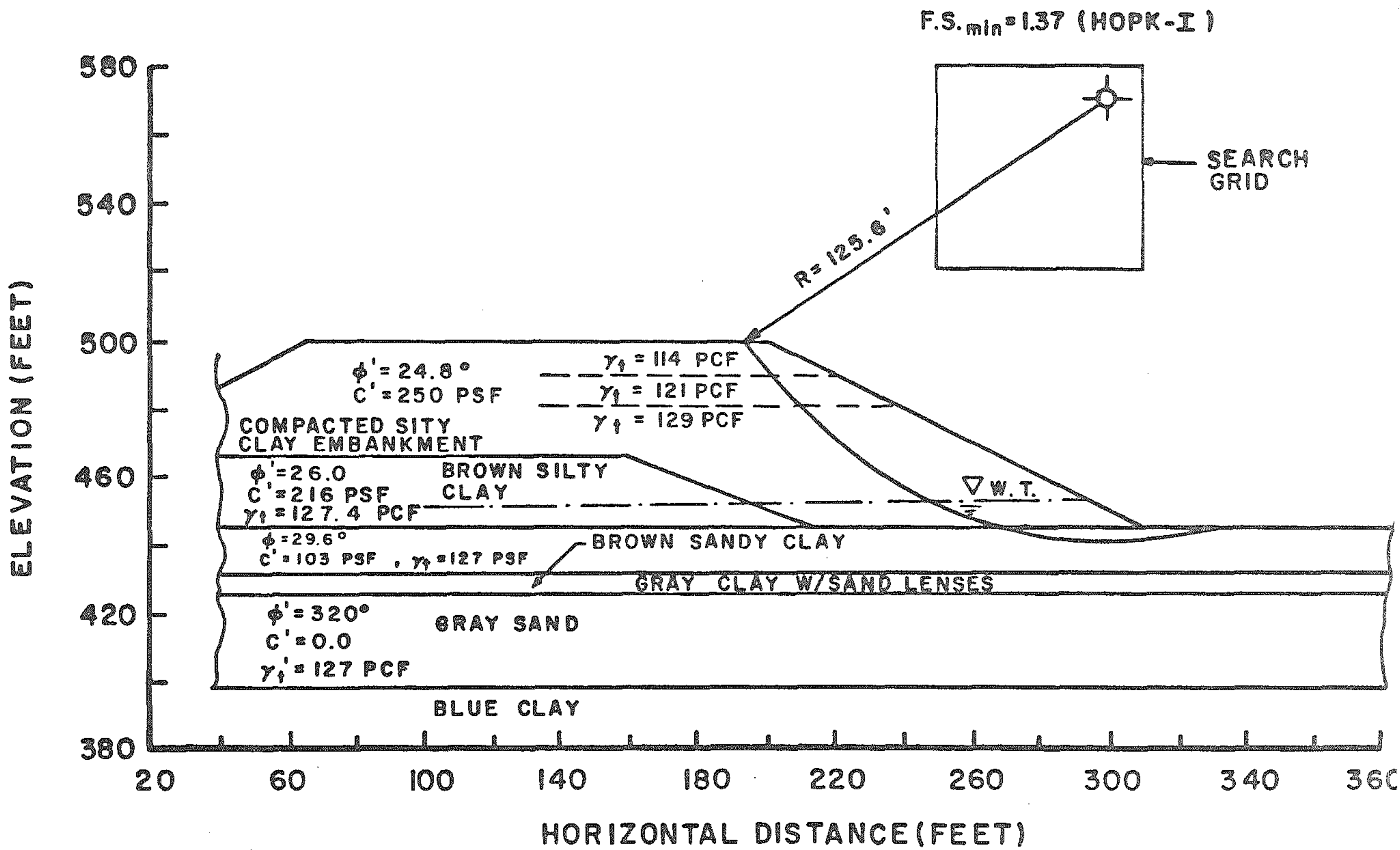


Figure 60. Stability Analysis Section, Cross Section, at I 71 over the Kentucky River.

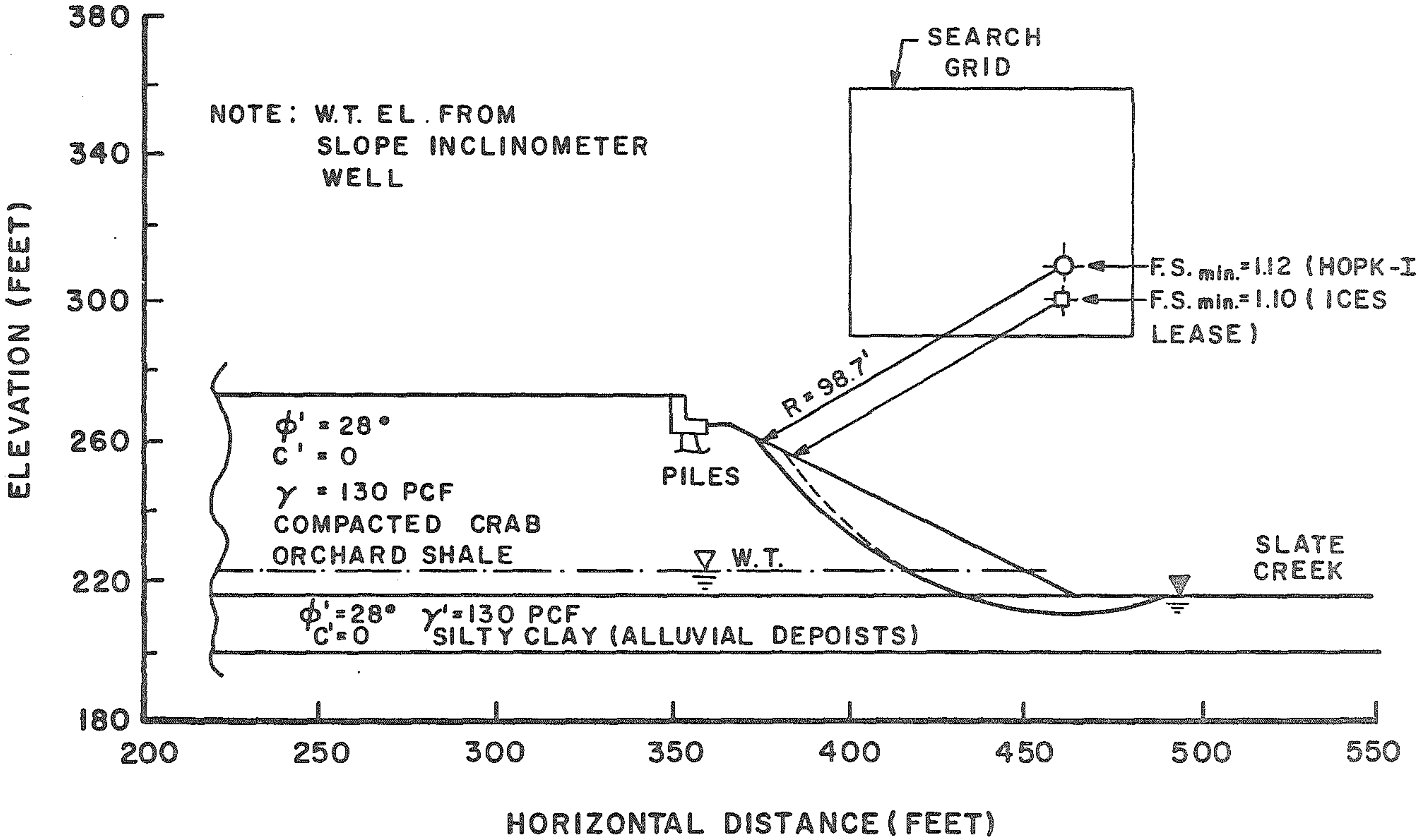


Figure 61. Stability Analysis Section of I 64 over Slate Creek.

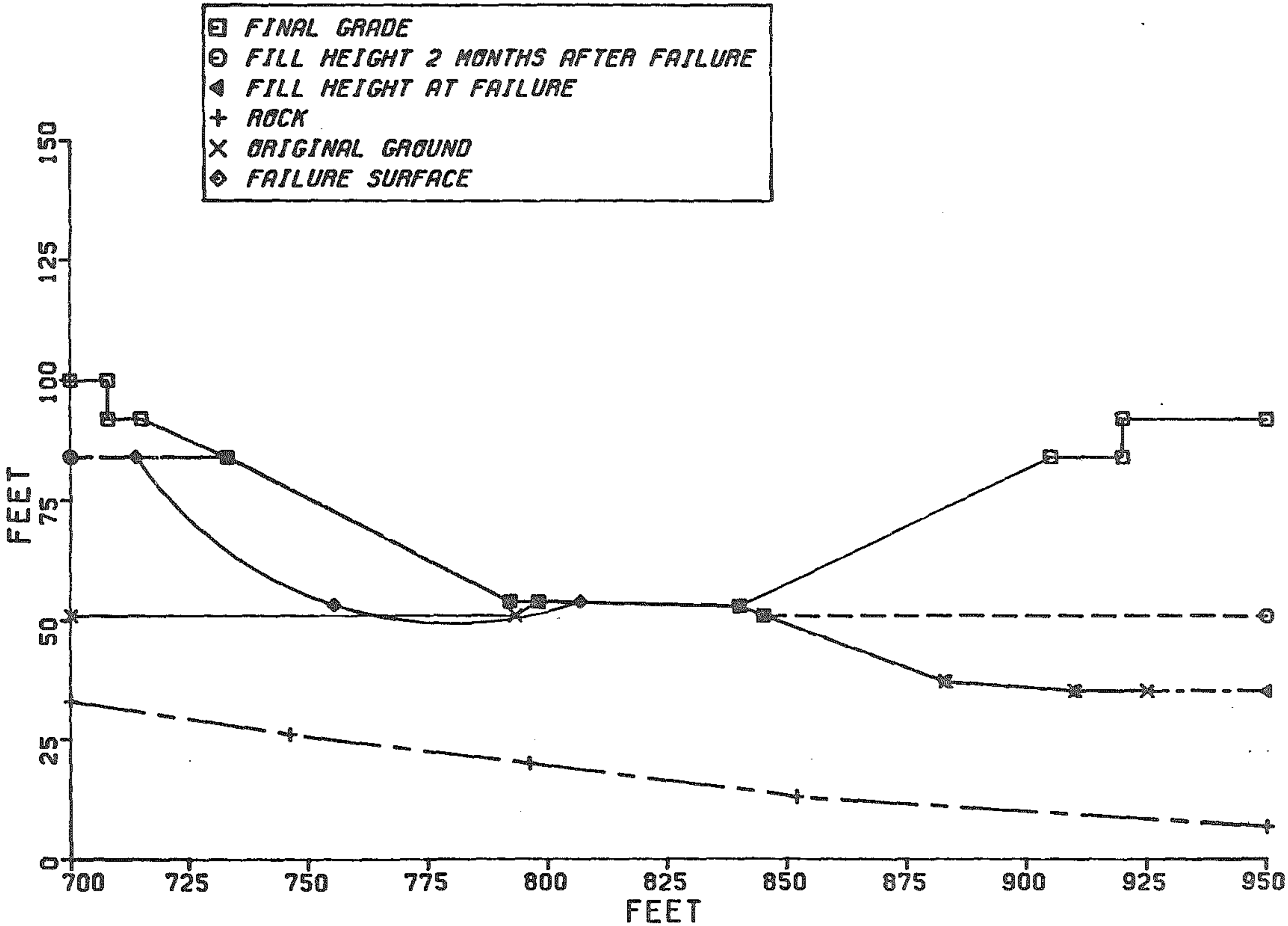


Figure 62. Stability Analysis Section at I 471 over Chesapeake Avenue.

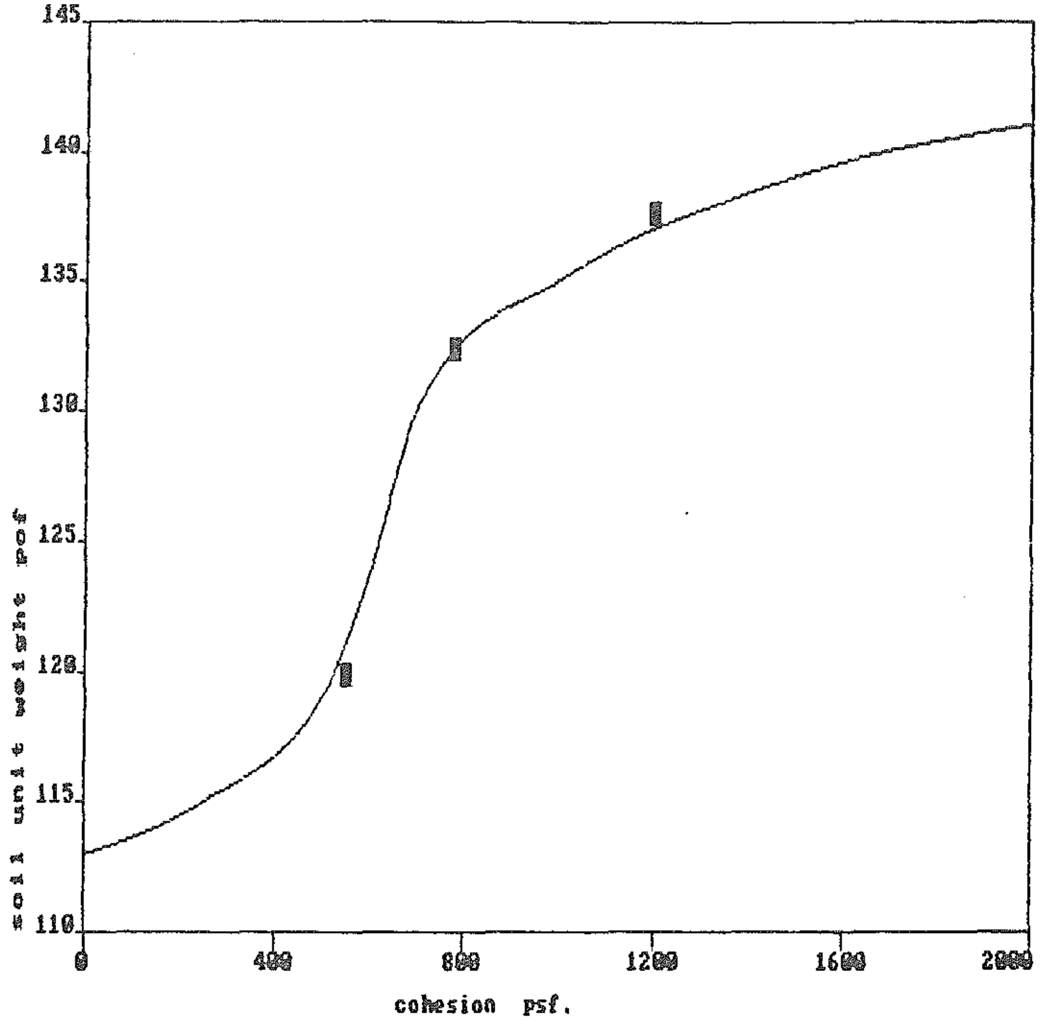


Figure 63. Cohesion Versus Wet Density of Crab Orchard Shale.

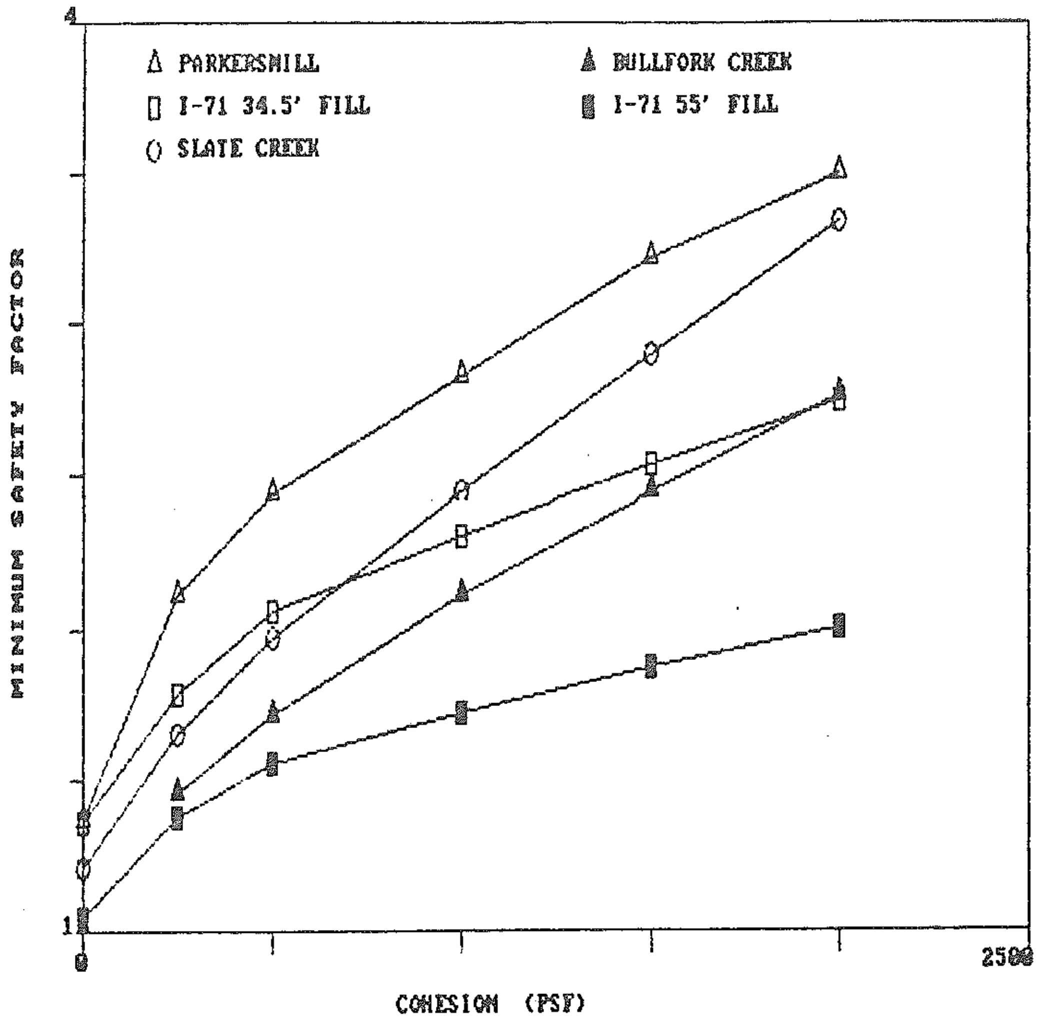


Figure 64. Safety Factor Versus Cohesion.

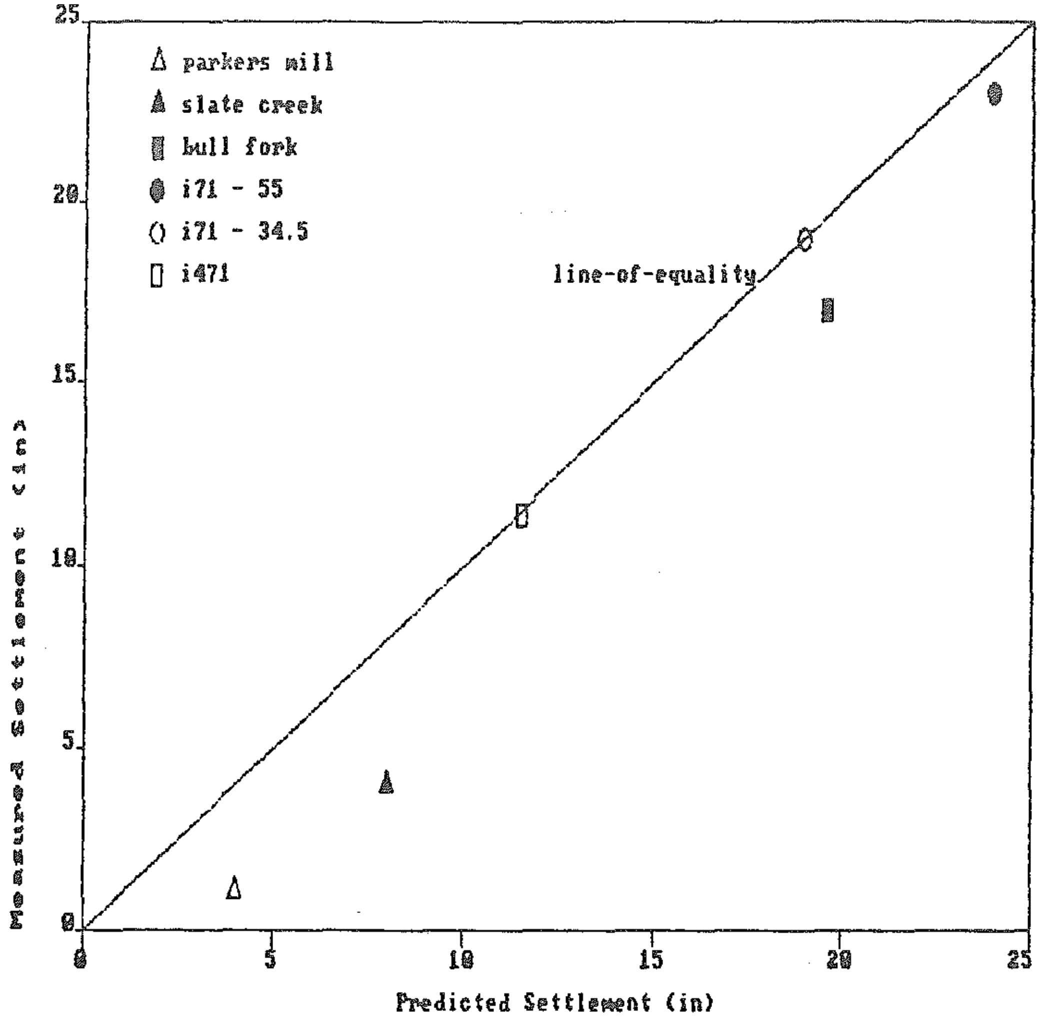


Figure 65. ISBILD Predicted Foundation Versus Measured Foundation Foundation Settlement.

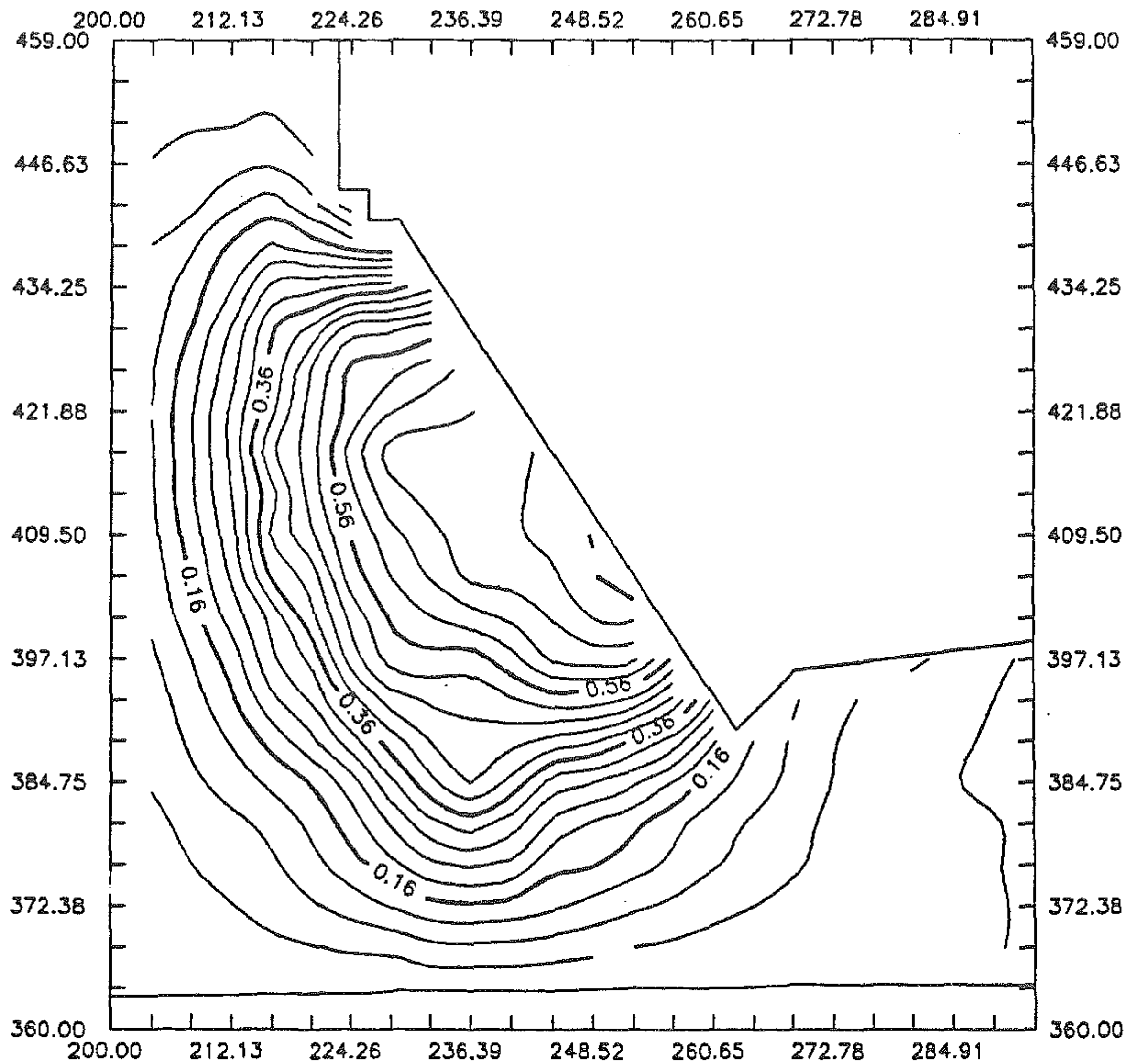


Figure 66. ISBILD Predicted Lateral Movement at Parkers Mill Site.

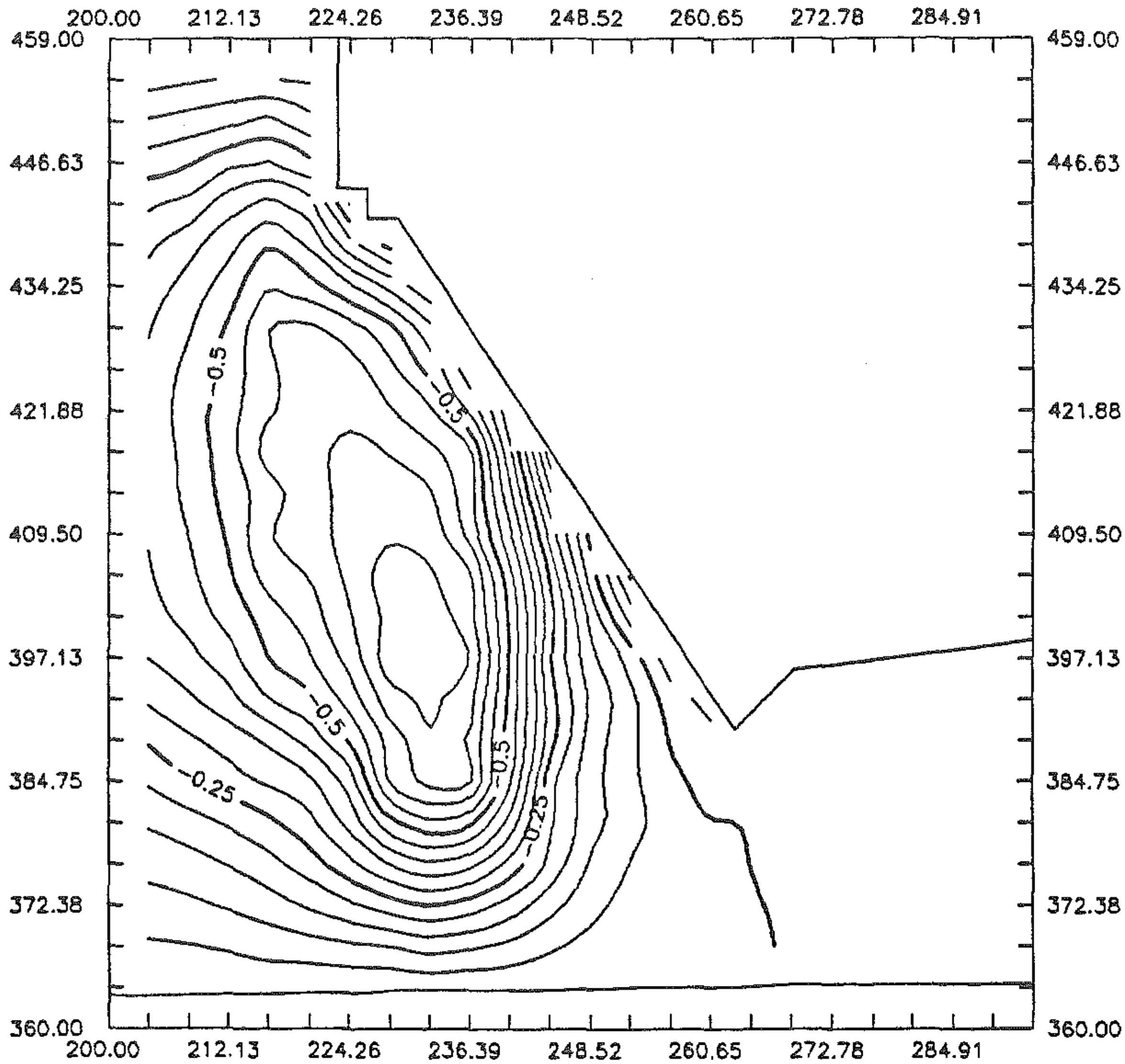


Figure 67. ISBILD Predicted Settlement at Parkers Mill Site.

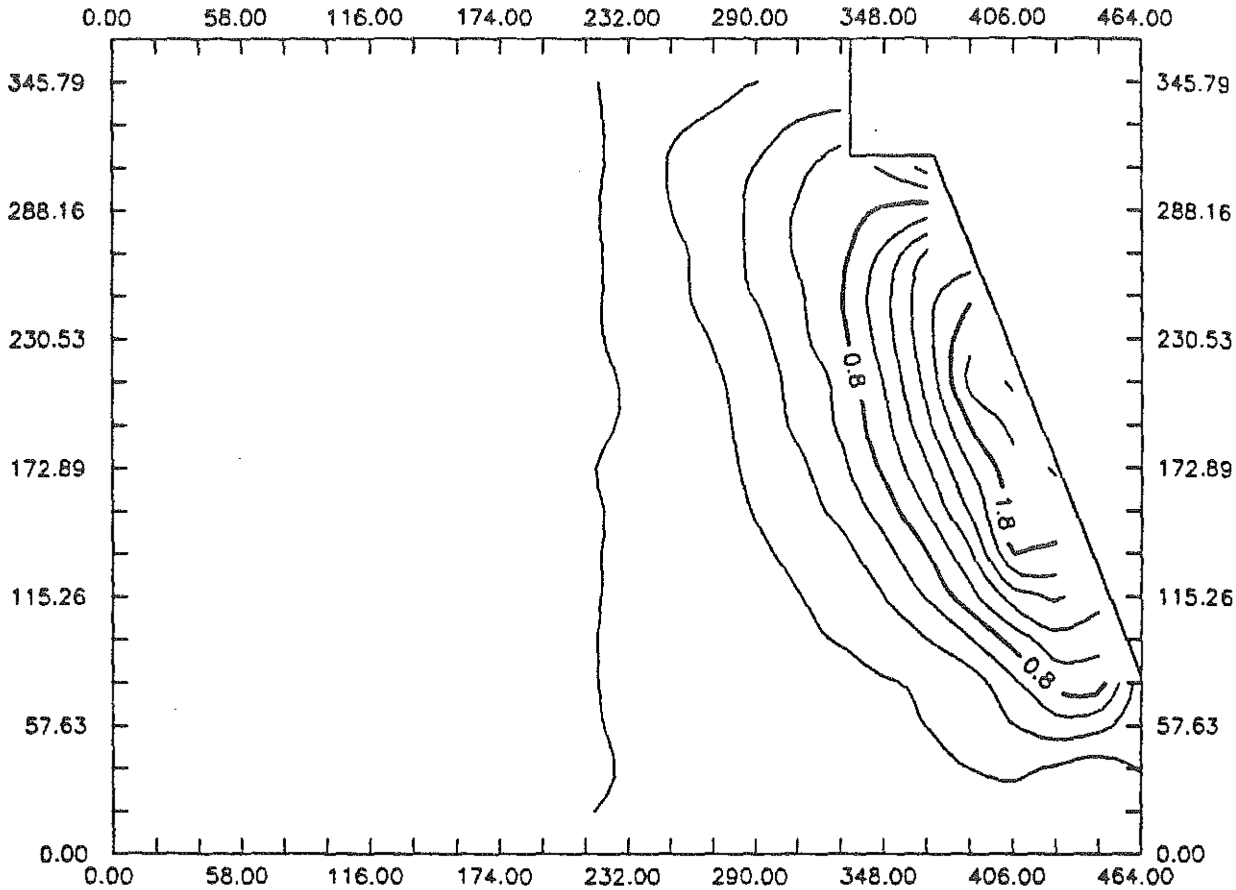


Figure 68. ISBILD Predicted Lateral Movement at Slate Creek Site.

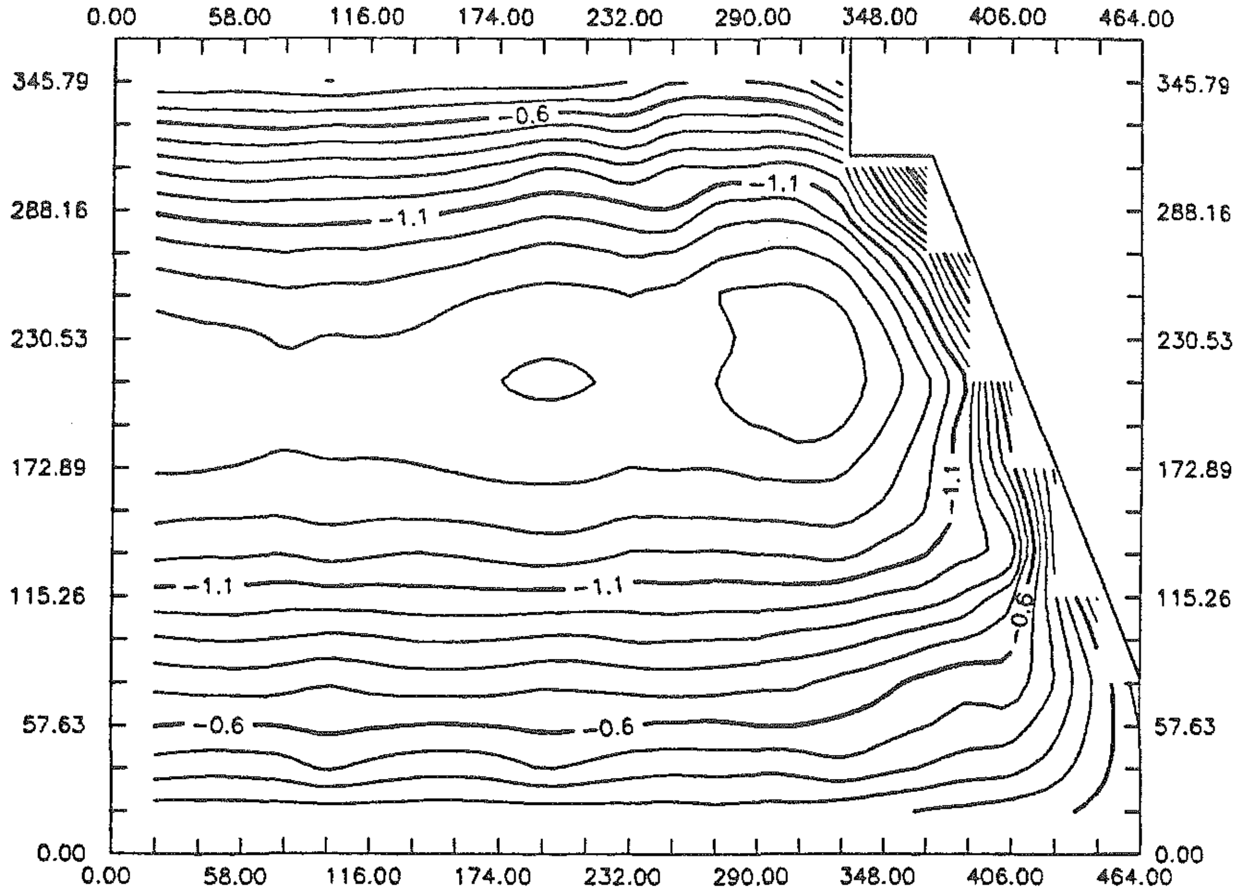


Figure 69. ISBILD Predicted Settlement at Slate Creek Site.

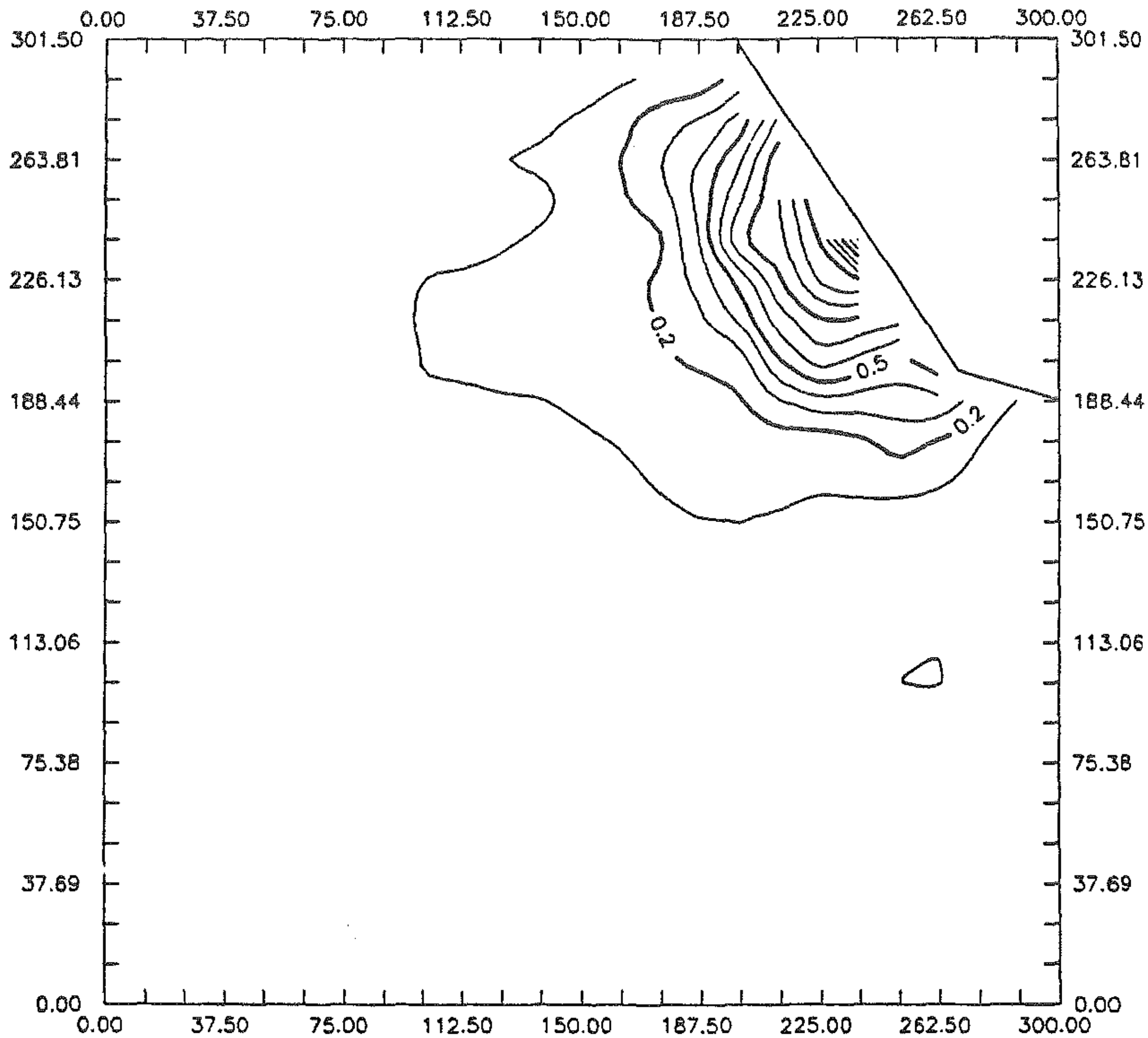


Figure 70. ISBILD Predicted Lateral Movement at I 471 (34.5 feet) over the Kentucky River.

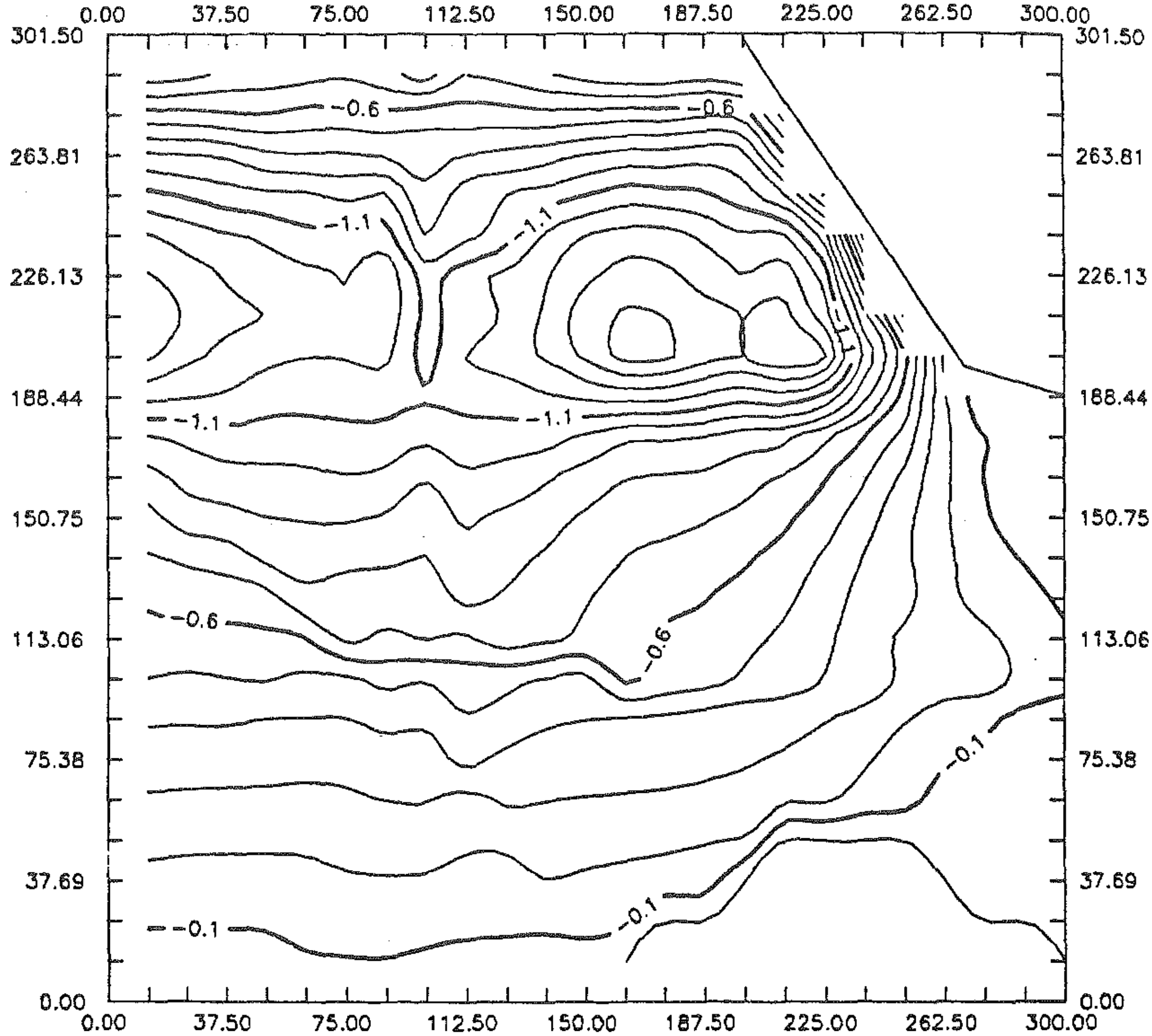


Figure 71. ISBILD Predicted Settlement at Slate Creek Site.

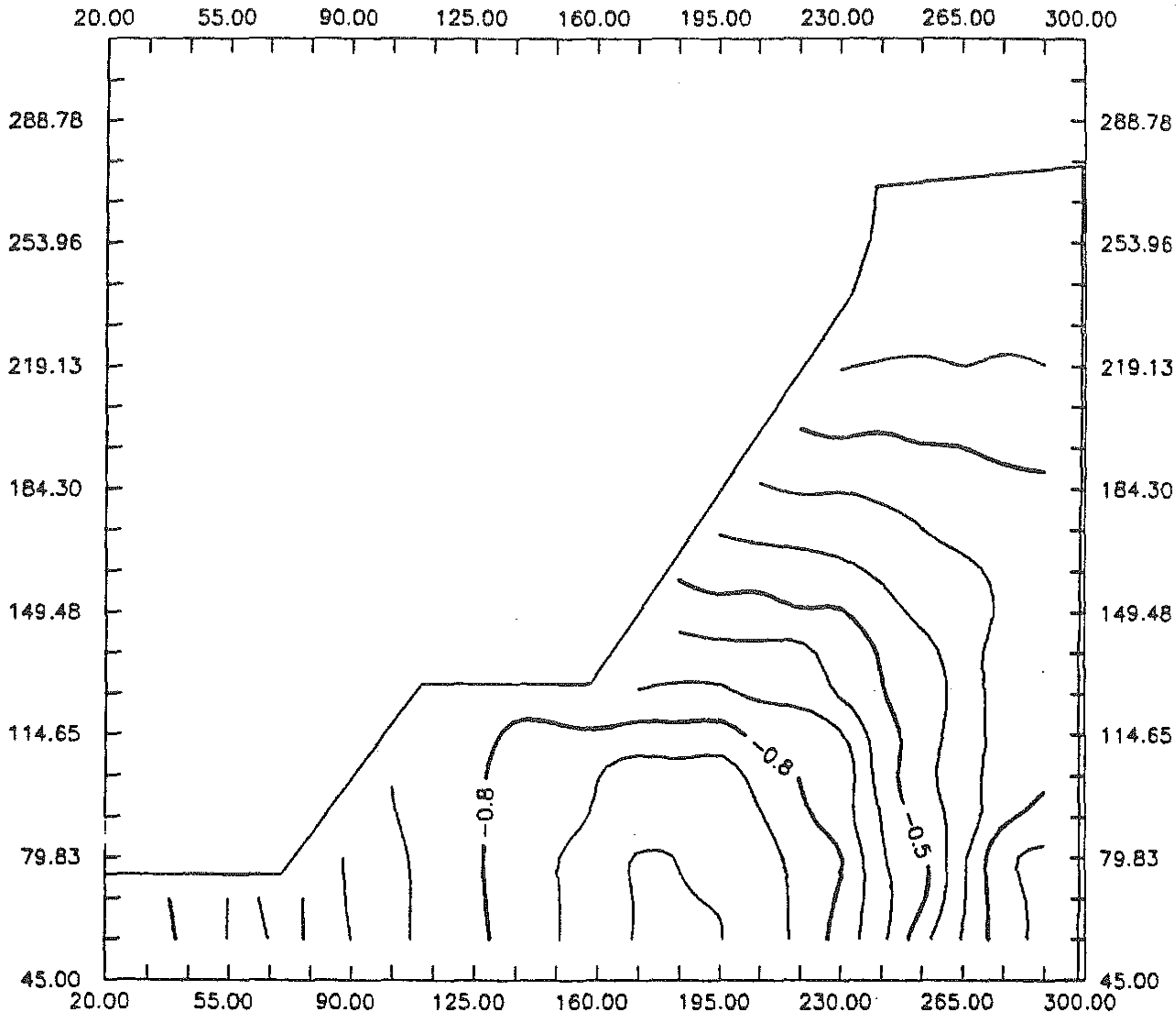


Figure 72. ISBILD Predicted Lateral Movement at I 471 Site.

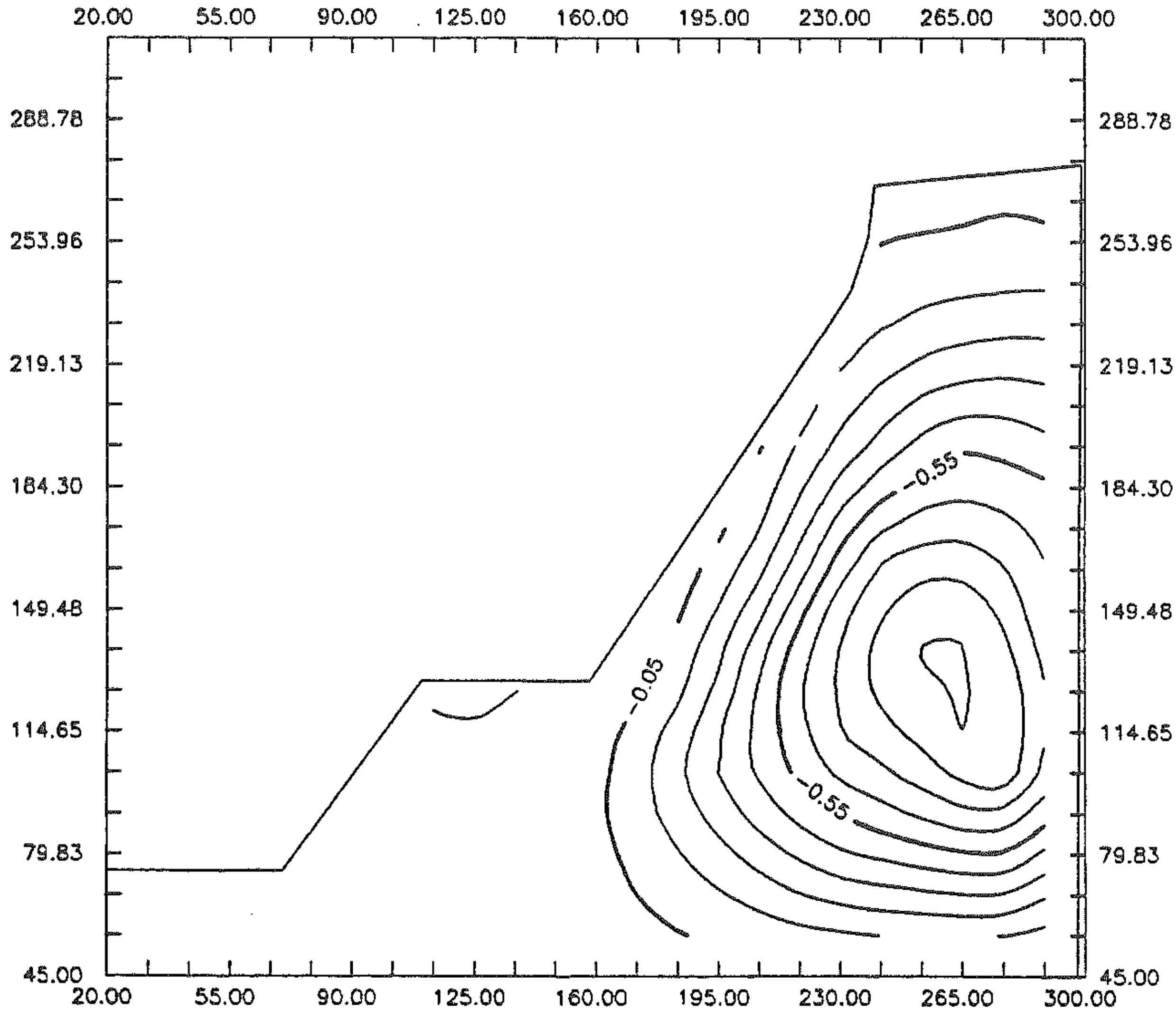


Figure 73. ISBILD Predicted Settlement at I 471.

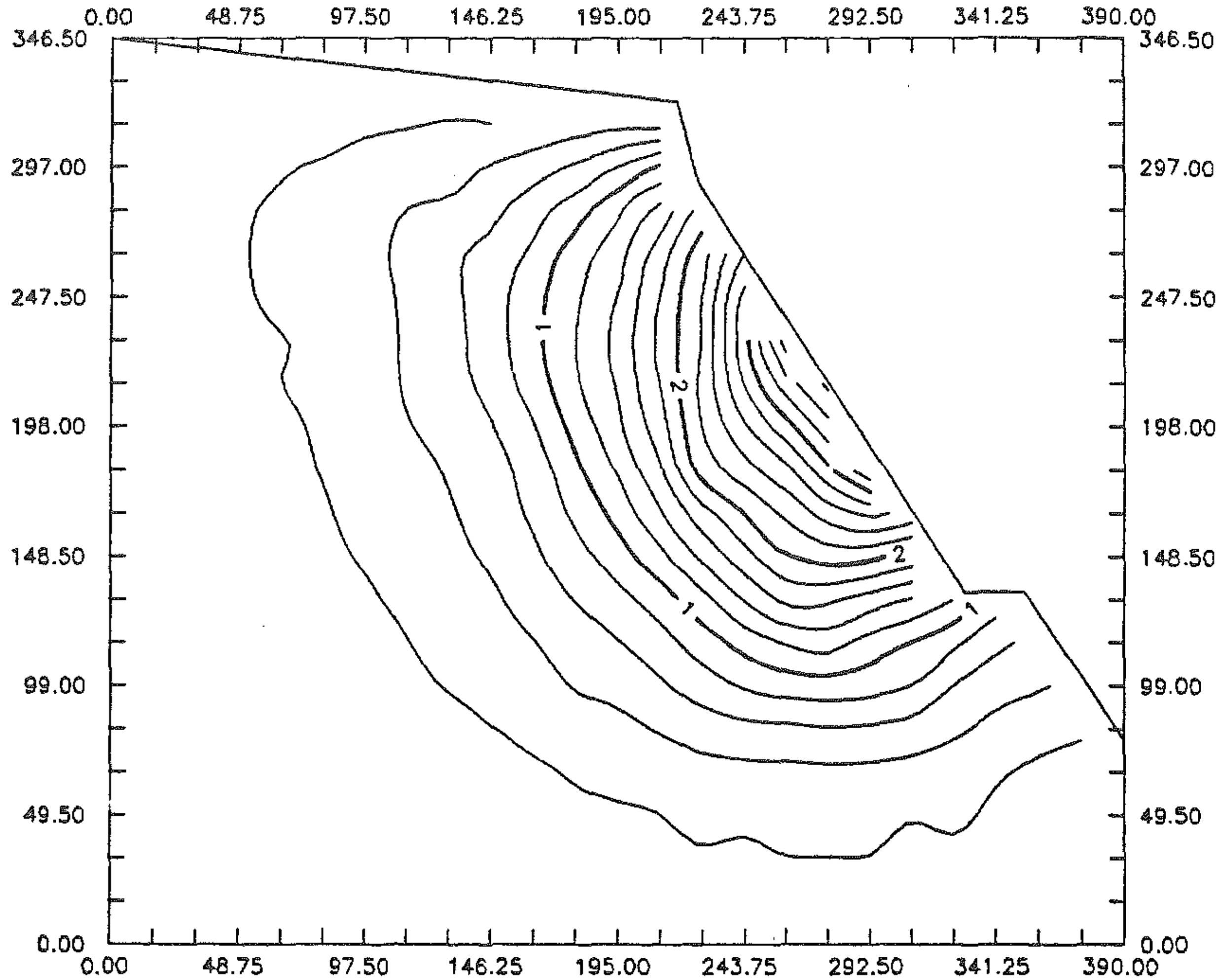


Figure 74. ISBILD Predicted Lateral Movement at Bull Fork Site.

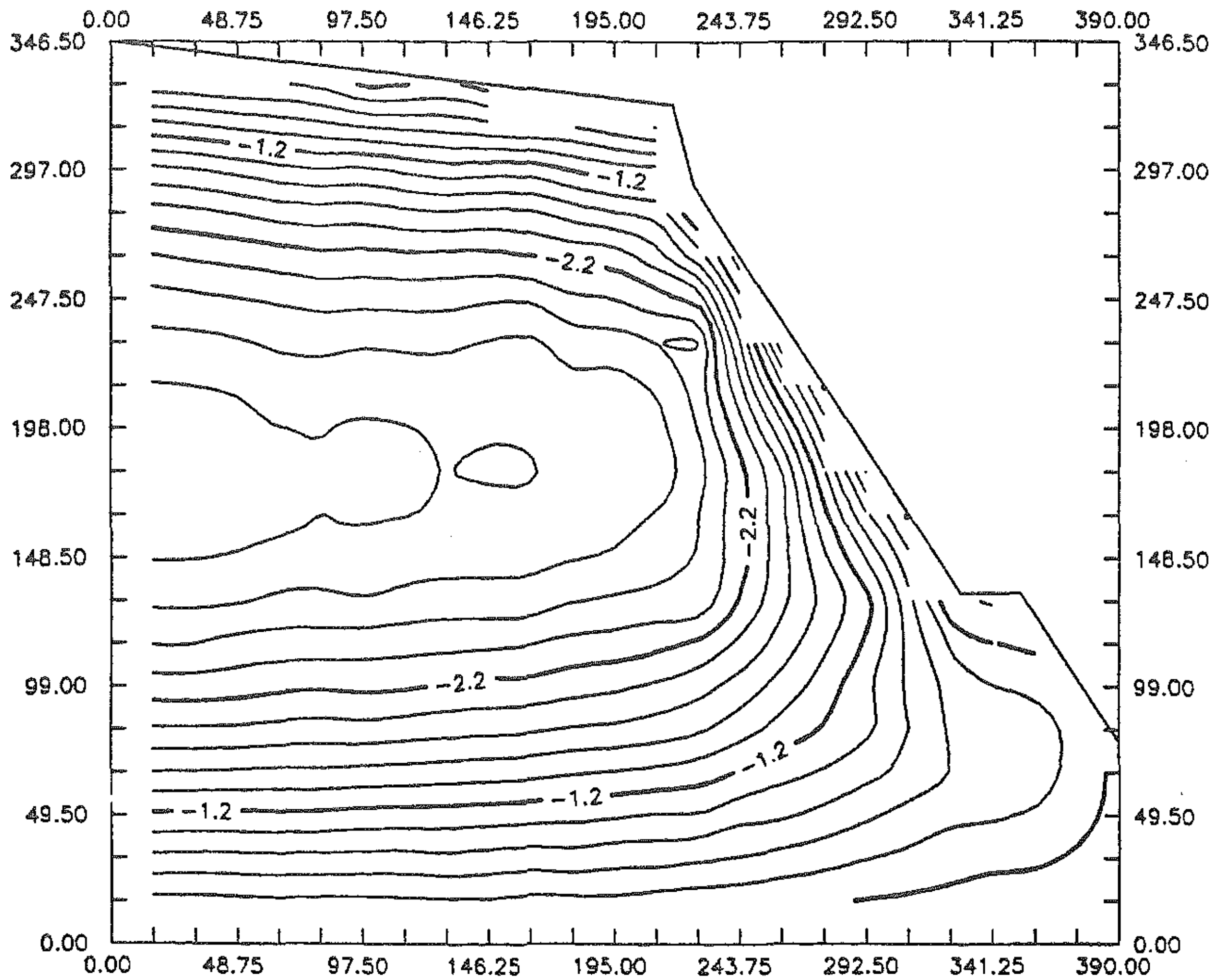


Figure 75. ISBILD Predicted Settlement at Bull Fork Site.

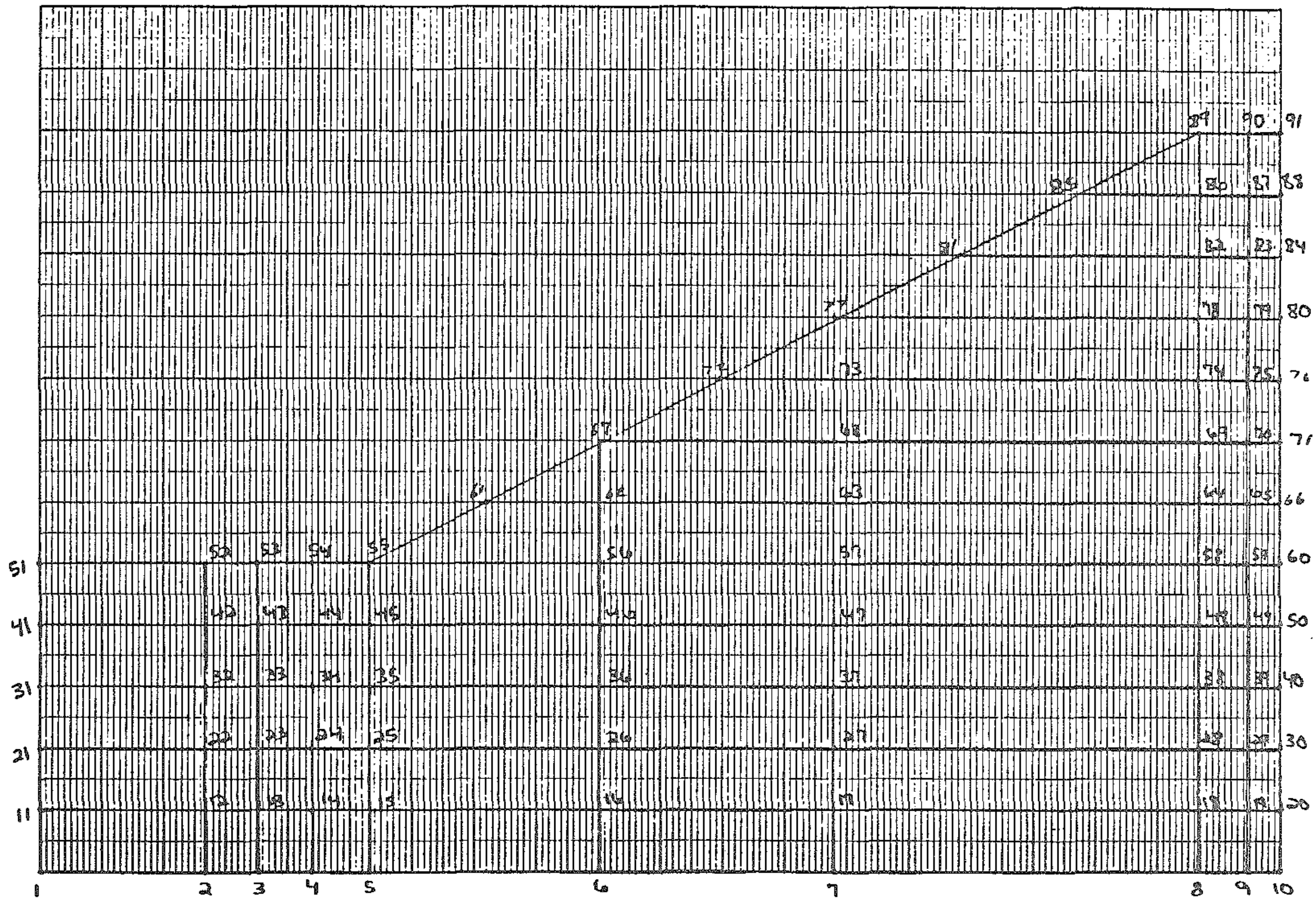


Figure 76. Finite Element Grid Used for Theoretical Models.

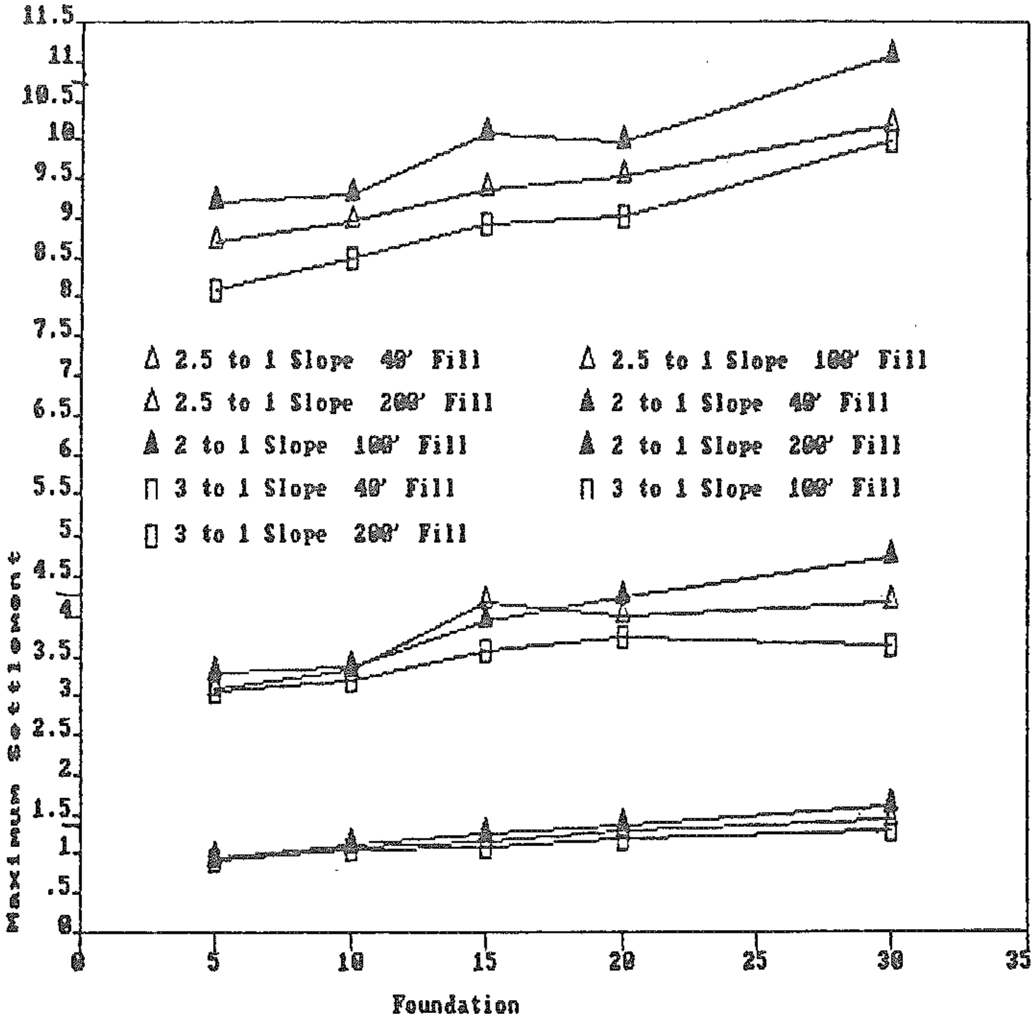


Figure 77. Embankment Settlement for Theoretical Models with Constant Soil Strength Parameters.

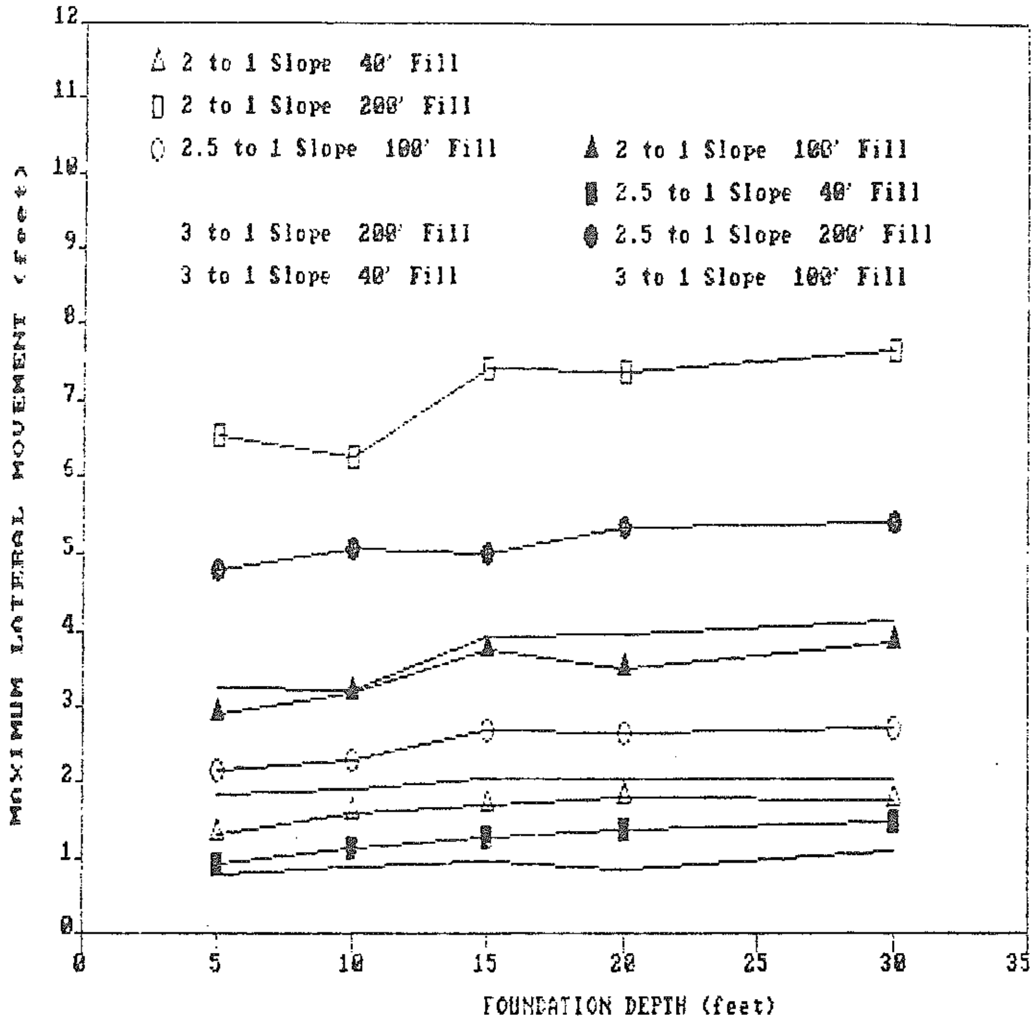


Figure 78. Embankment Lateral Movement for Theoretical Models with Constant Soil Strength Parameters.

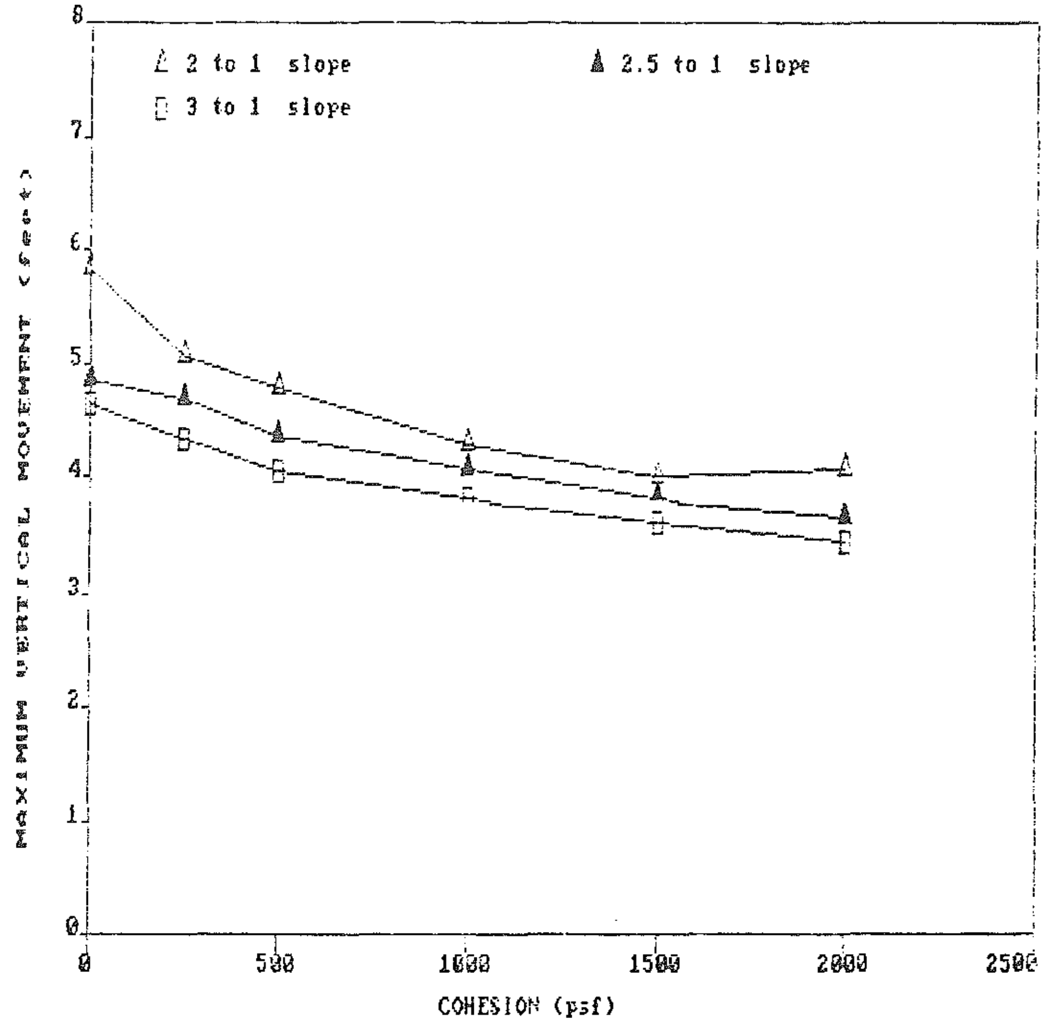


Figure 79. Embankment Settlement for Theoretical Models with Constant Fill and Foundation Depths.

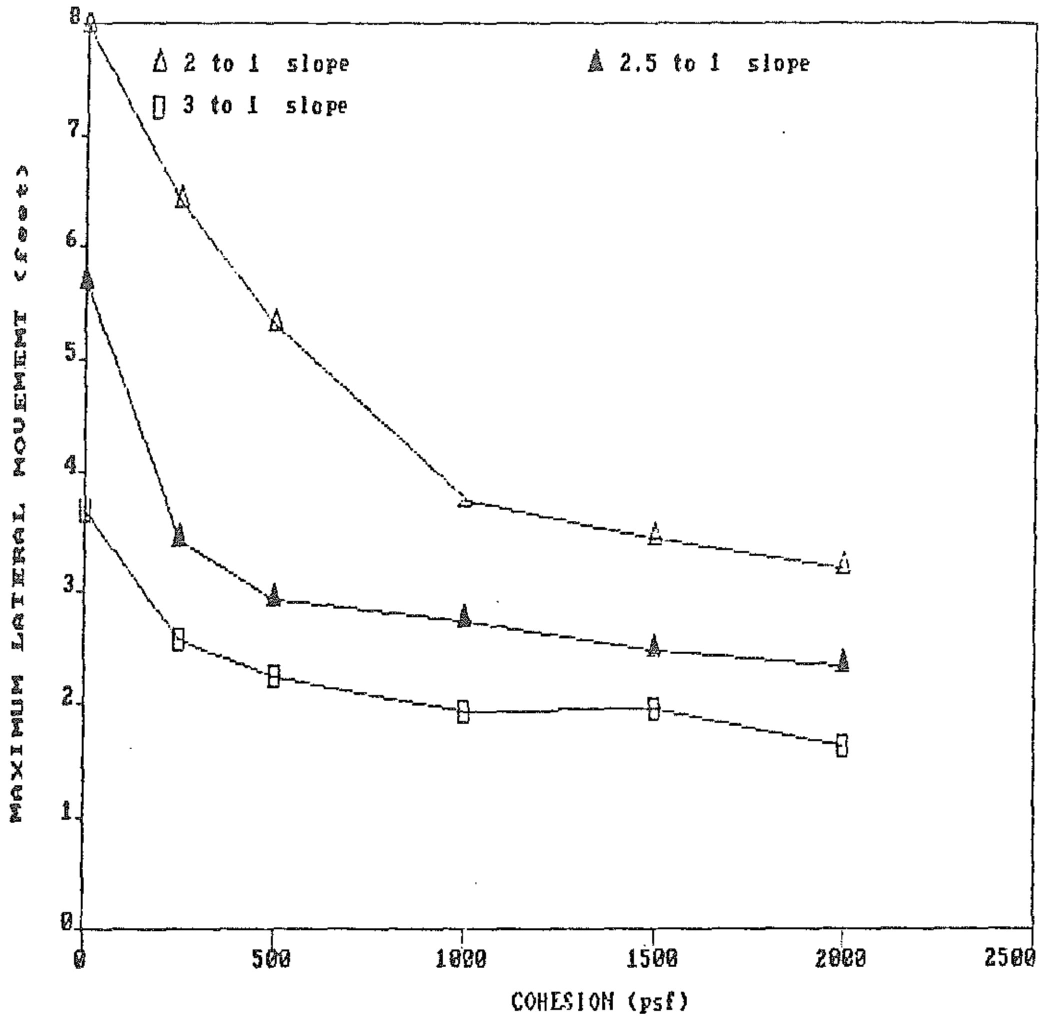


Figure 80. Embankment Lateral Movement for Theoretical Models with Constant Fill and Foundation Depths.

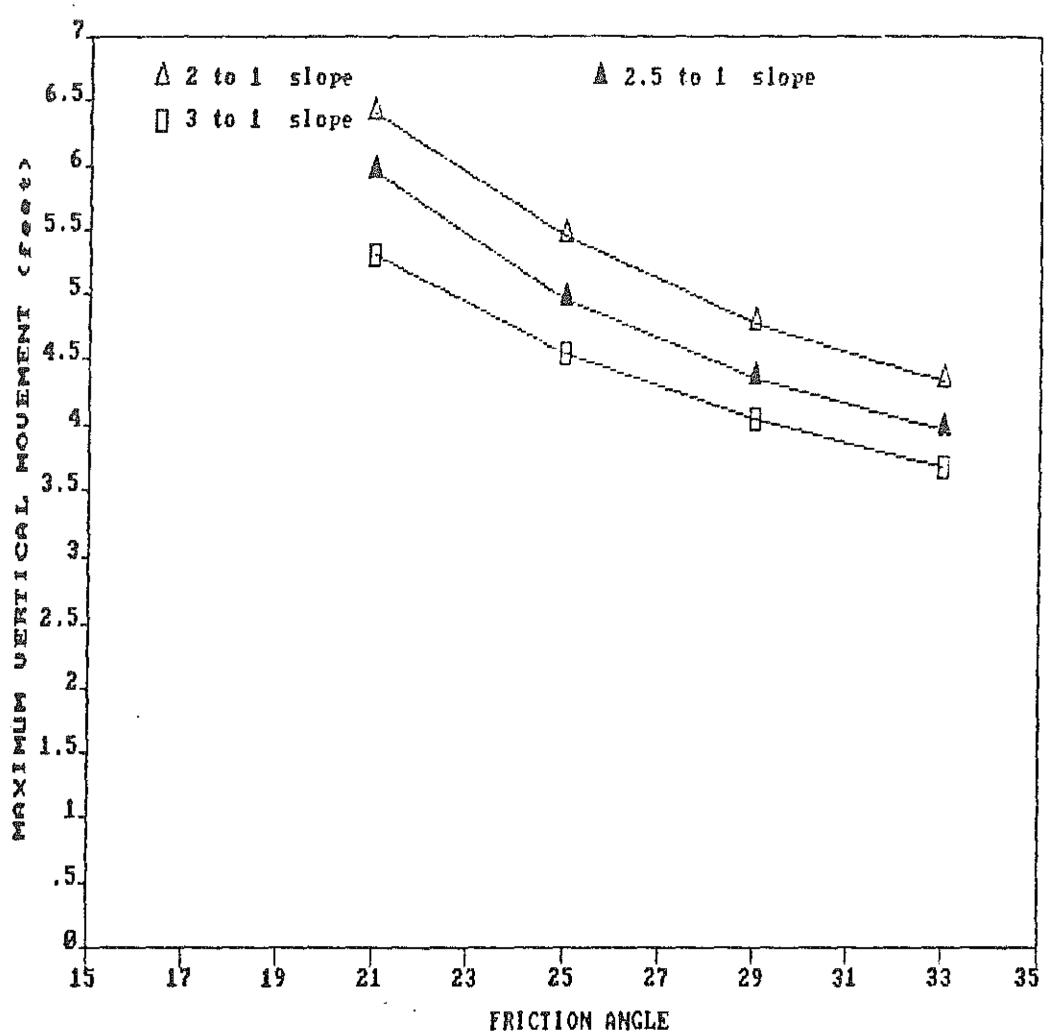


Figure 81. Embankment Settlement for Theoretical Models with Constant Fill and Foundation Depths and Cohesion.

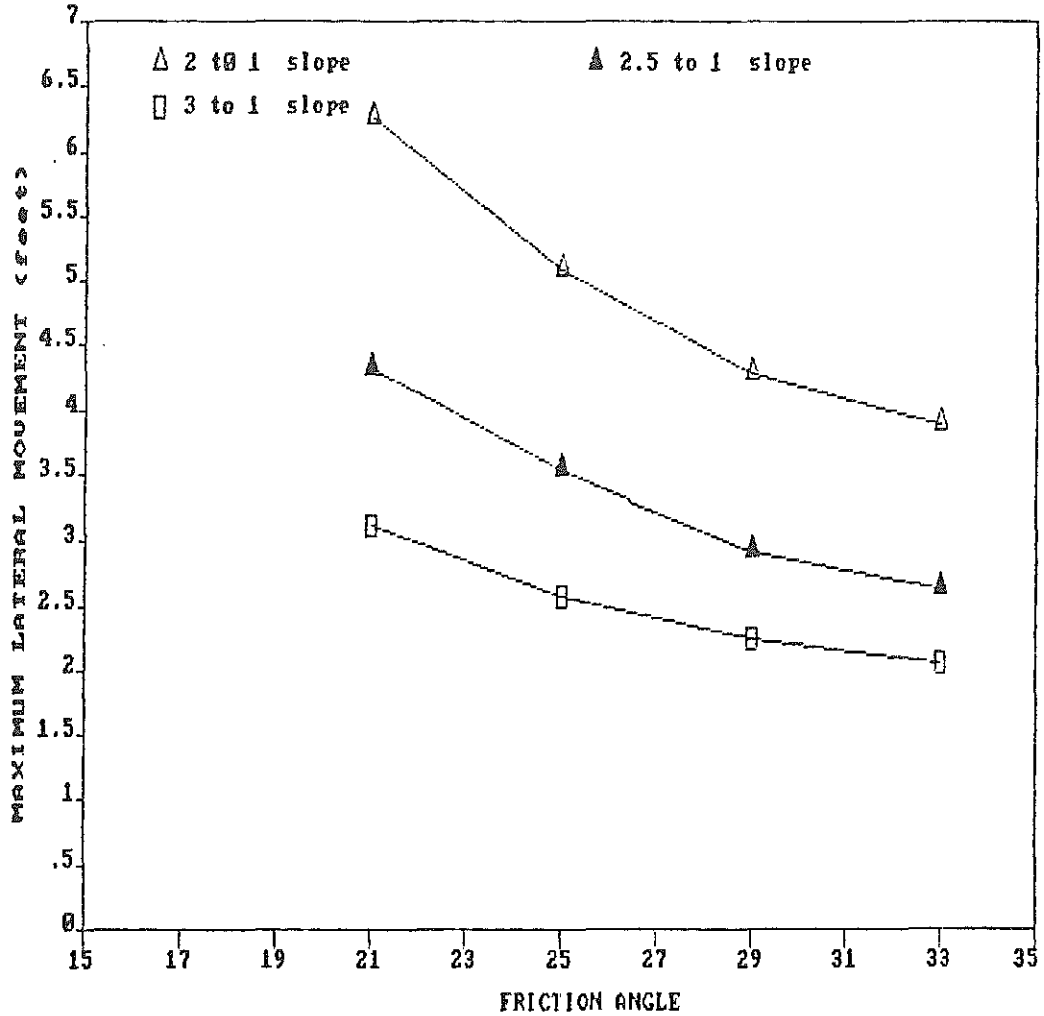


Figure 82. Embankment Lateral Movement for Theoretical Models with Constant Fill and Foundation Depths and Cohesion.

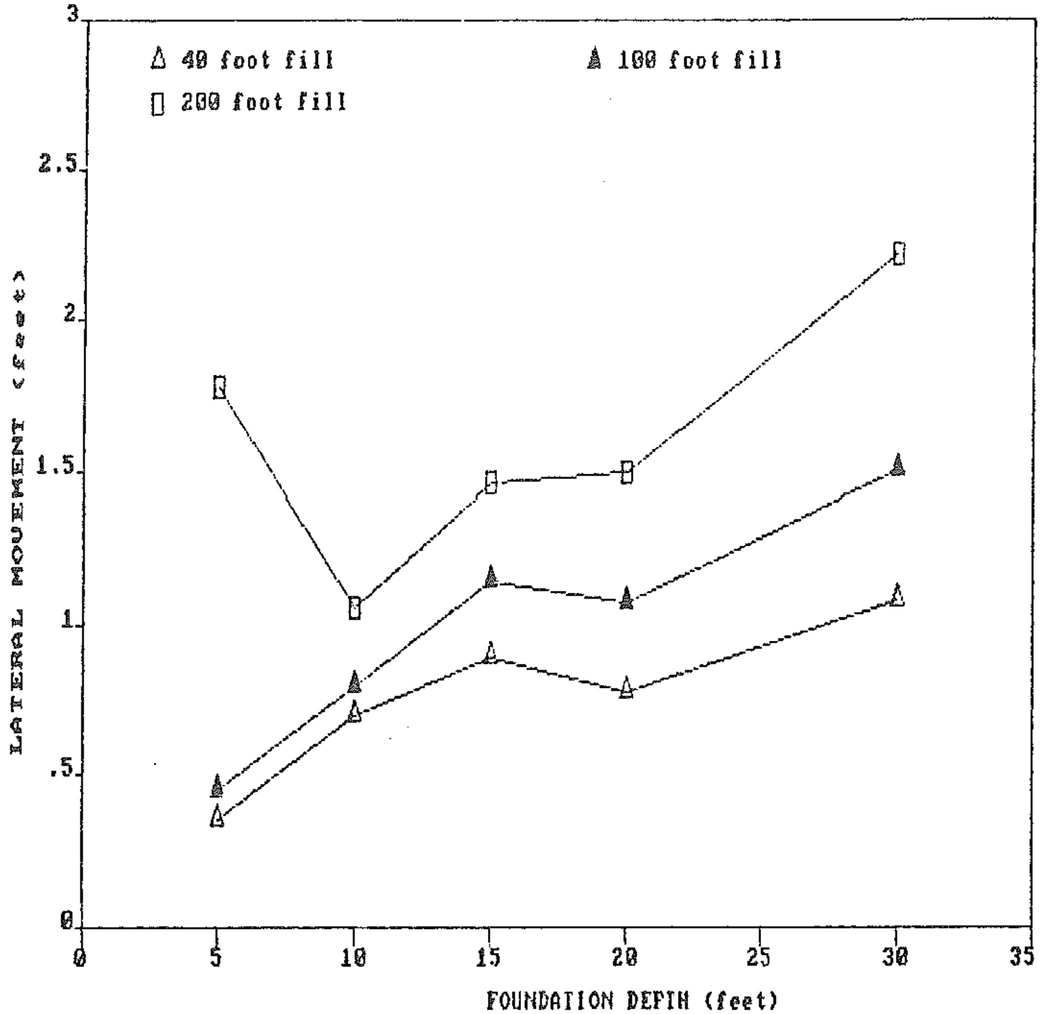


Figure 83. Foundation Lateral Movement for Theoretical Models with Constant Soil Strength Parameters at 3:1 Slope.

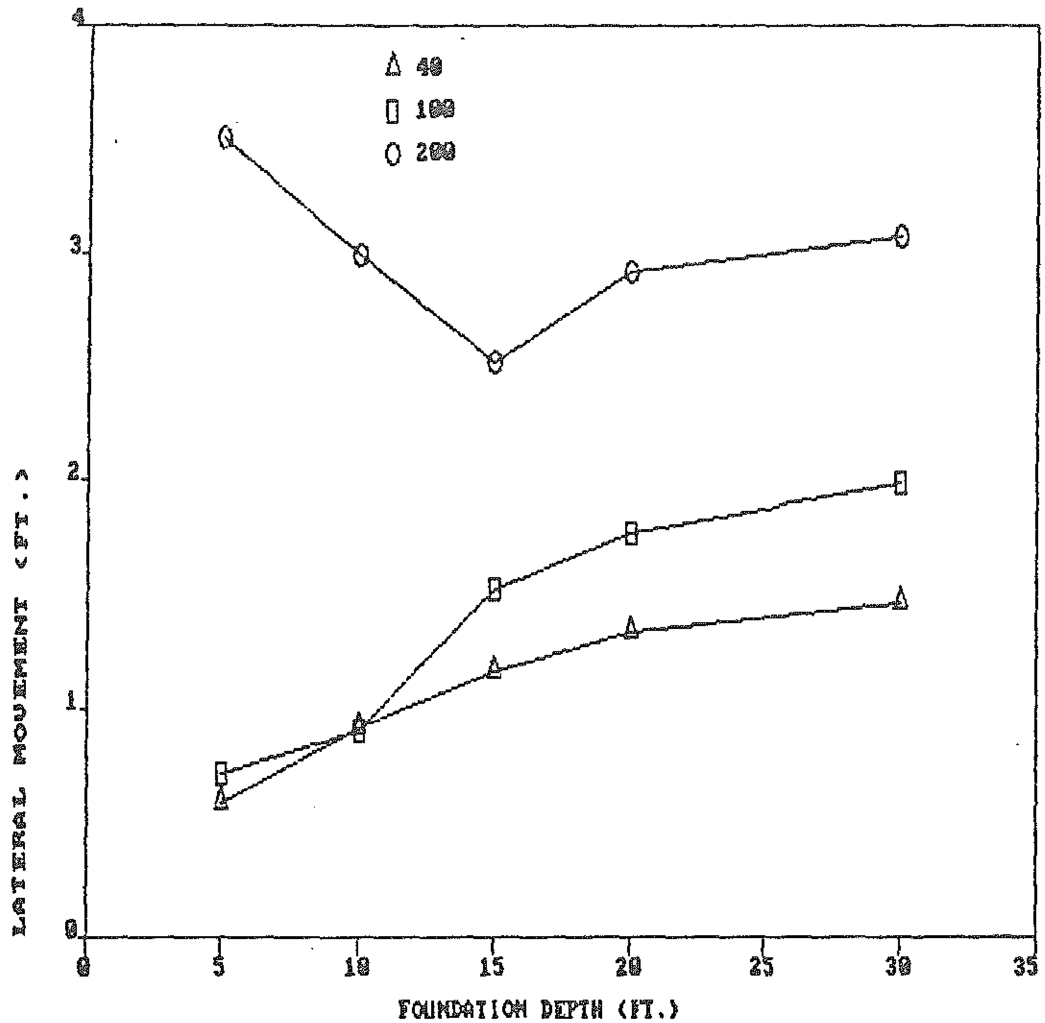


Figure 84. Foundation Lateral Movement for Theoretical Models with Constant Soil Strength Parameters at 2.5:1 Slope.

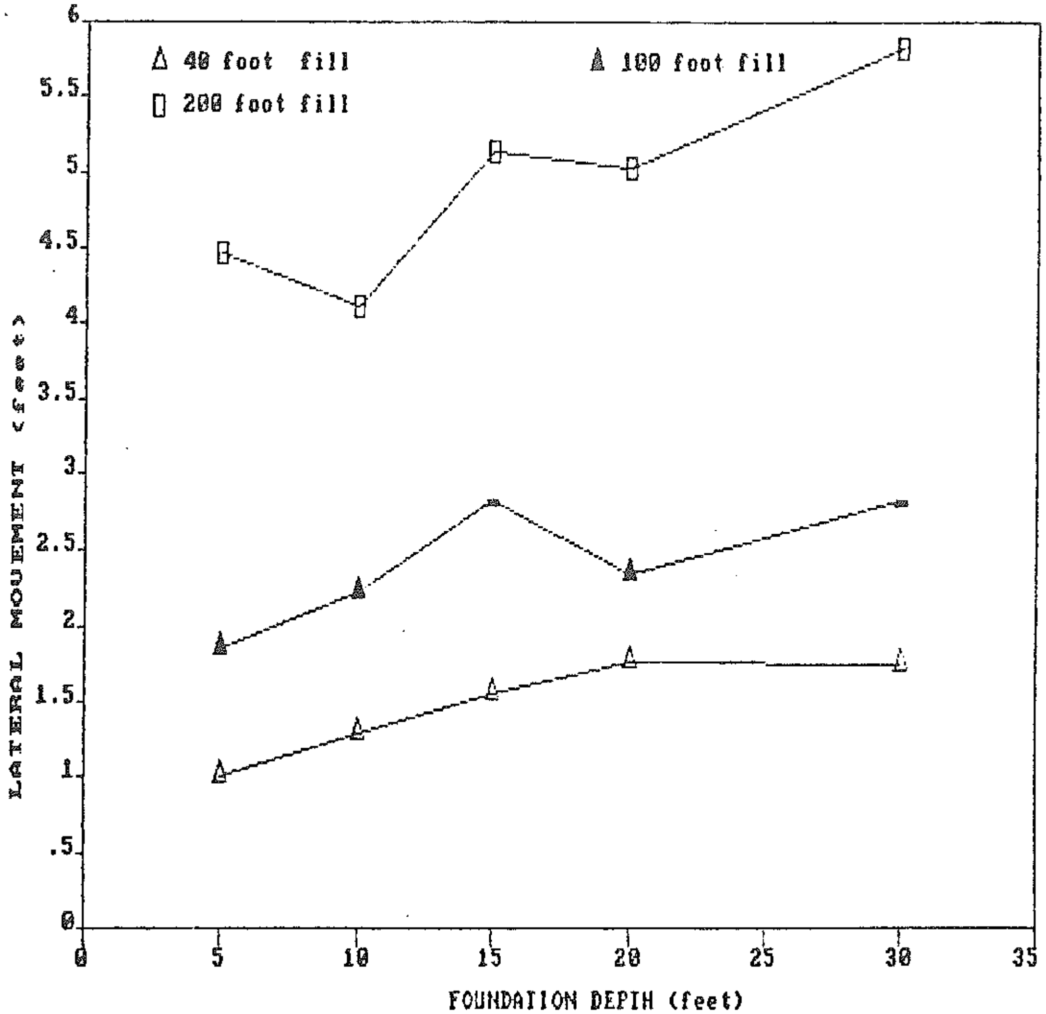


Figure 85. Foundation Lateral Movement for Theoretical Models with Constant Soil Strength Parameters at 2:1 Slope.

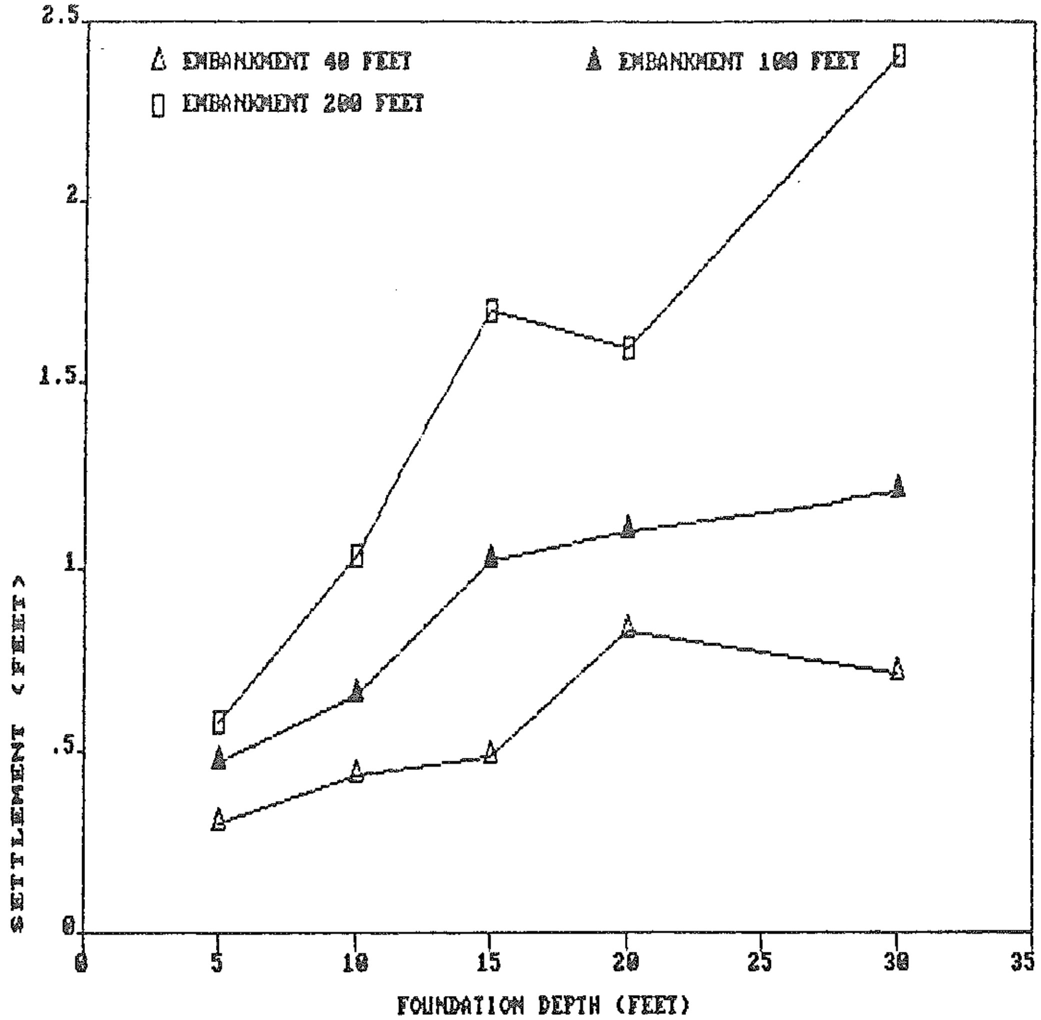


Figure 86. Foundation Settlement for Theoretical Models with Constant Soil Strength Parameters at 3:1 Slope.

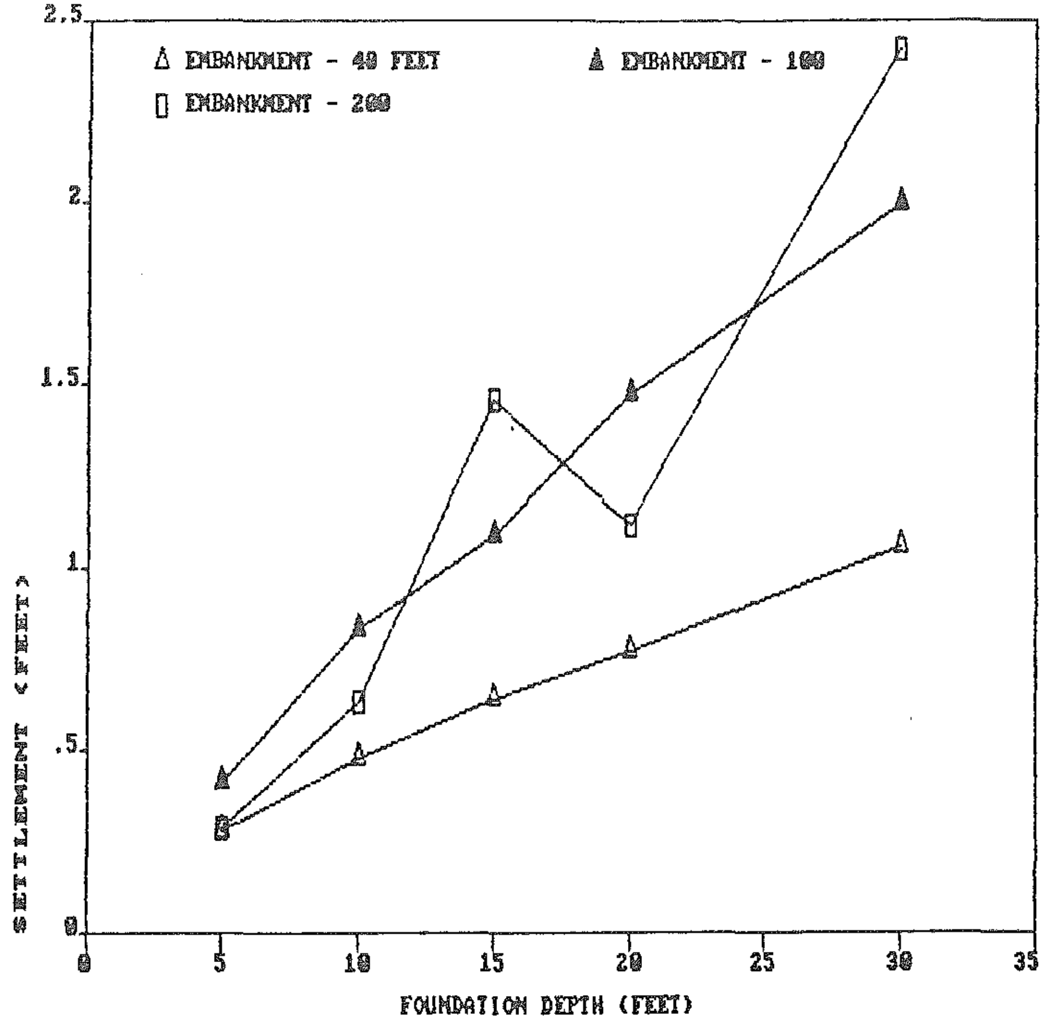


Figure 87. Foundation Settlement for Theoretical Models with Constant Soil Strength Parameters at 2:1 Slope.

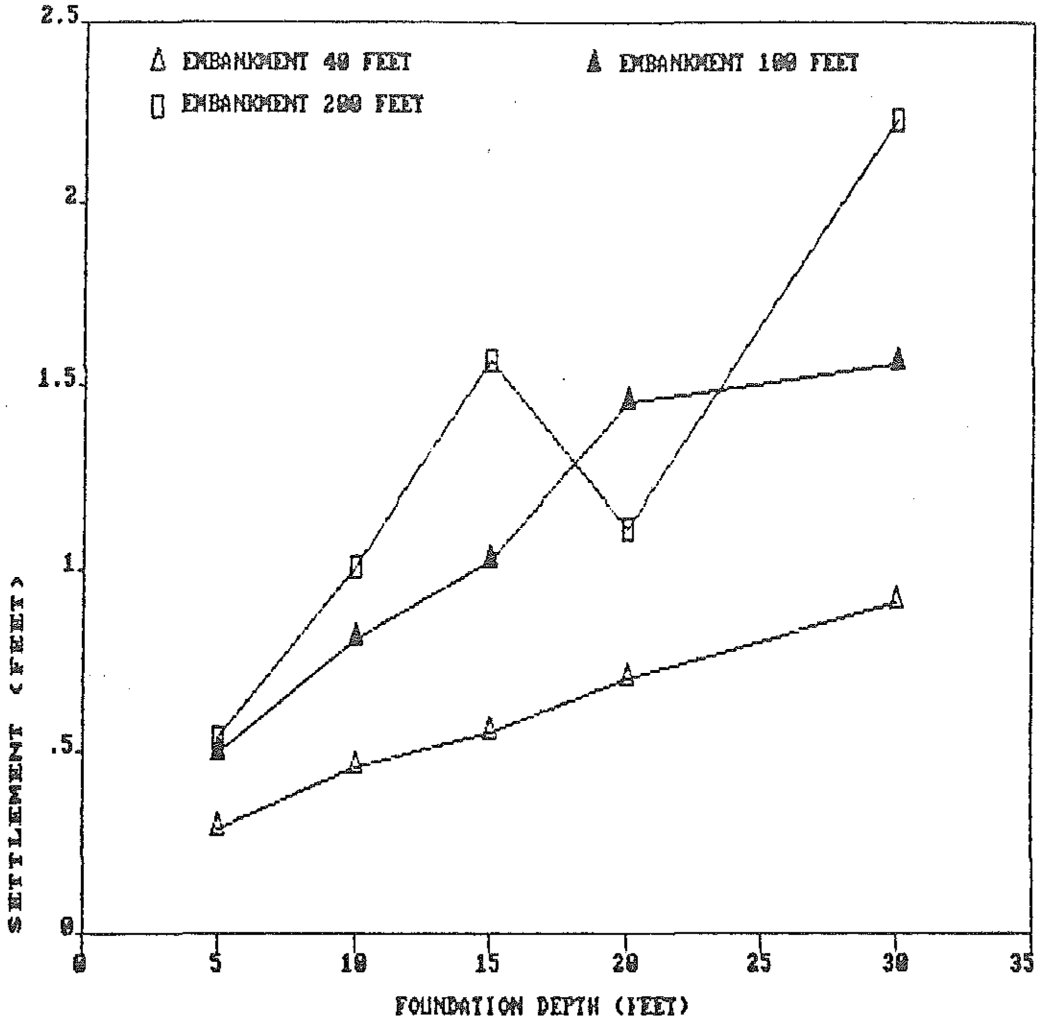


Figure 88. Foundation Settlement for Theoretical Models with Constant Soil Strength Parameters at 2.5:1 Slope.

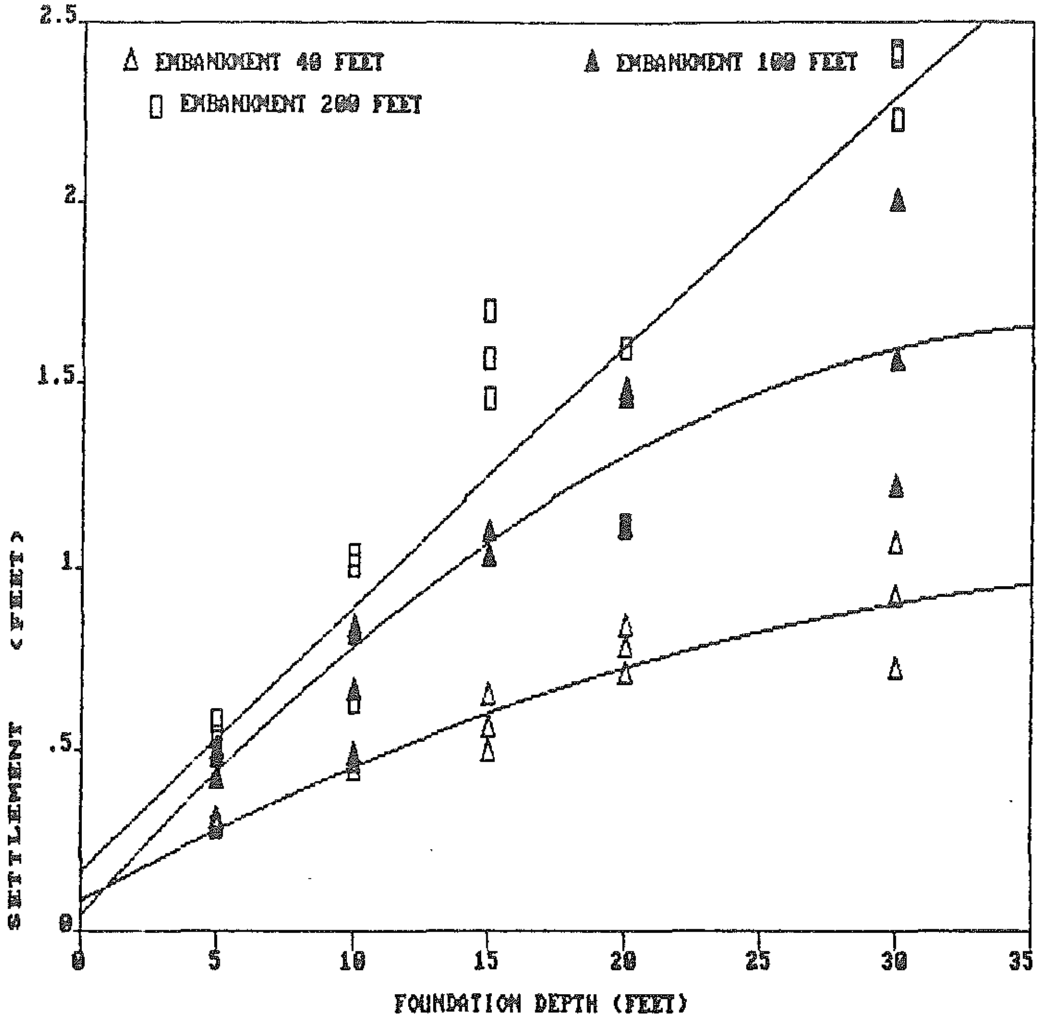


Figure 89. Foundation Settlement for Theoretical Sites.

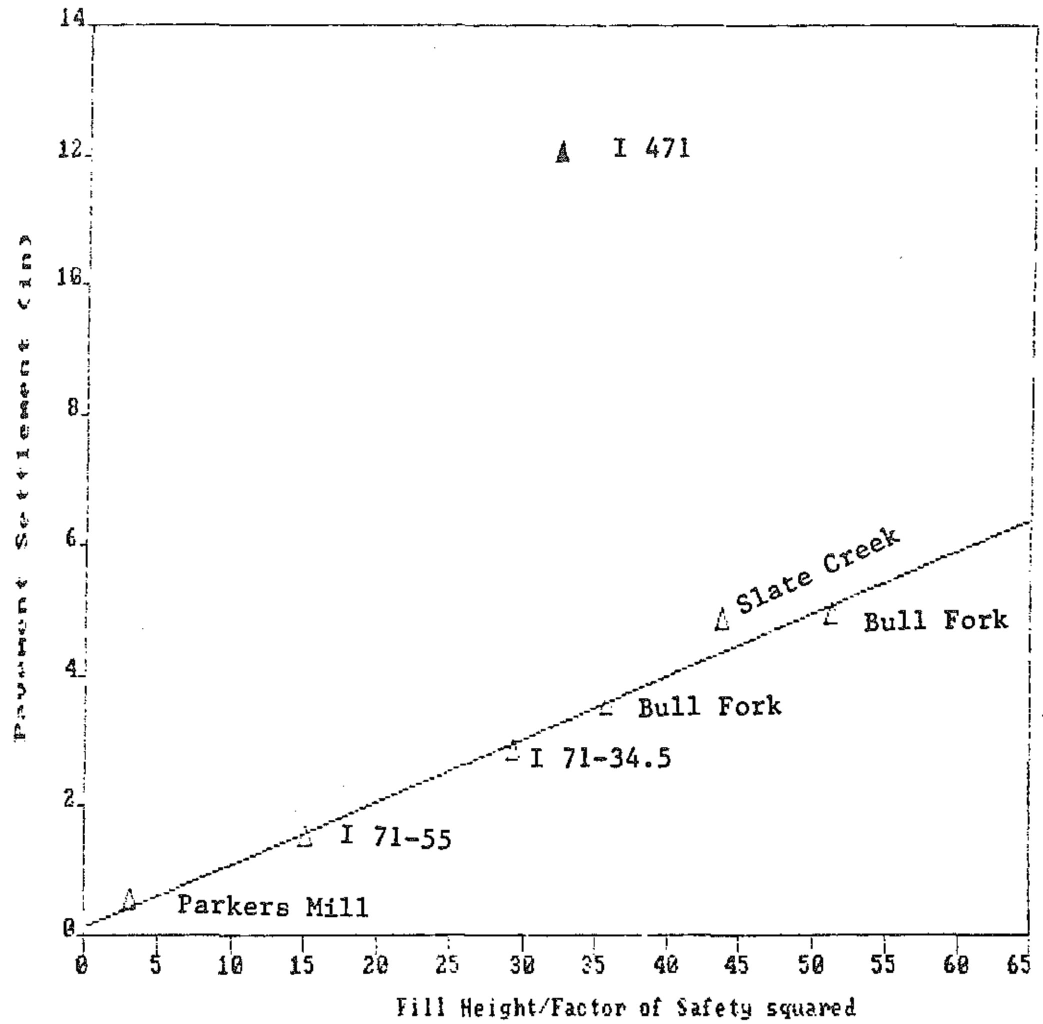


Figure 90. Measured Settlement Versus Fill Height Divided by the Square of Factor of Safety for Actual Sites.

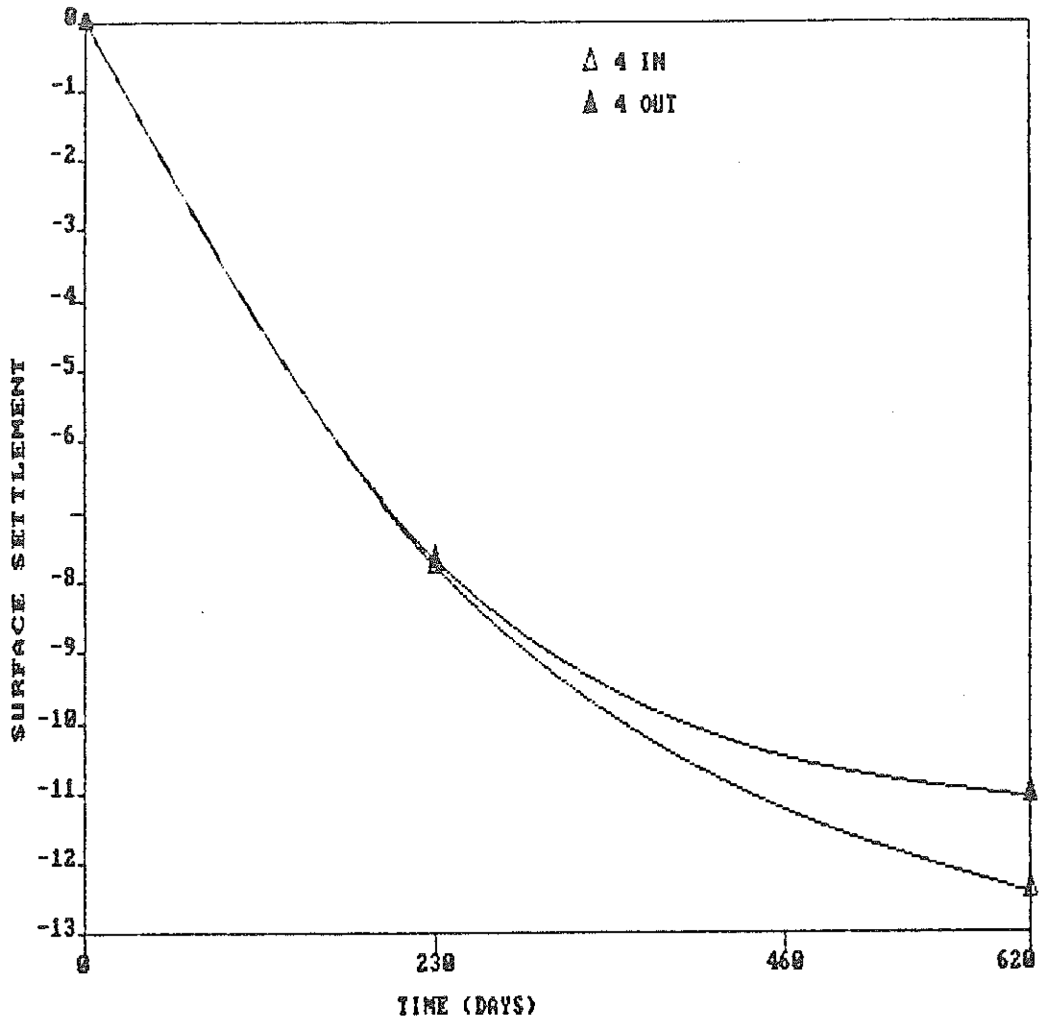


Figure 91. Near Surface Settlement (2-3 Feet) of Embankment (190 feet) in Laurel County, Kentucky.

TABLE 1. SITE CONDITIONS

SITE	FOUNDATION			FILL			FOUNDATION SETTLEMENT		SAFETY FACTOR
	COHESION PSF	FRICTION ANGLE DEGREE	UNIT WEIGHT PCF	COHESION PSF	FRICTION ANGLE DEGREE	UNIT WEIGHT PCF	FINITE ELEMENT INCHES	FOUNDATION SETTLEMENT INCHES	
BULL FORK EAST	0.0	31.0	130.0	69.0	29.8	130.0	20.9	17.0	1.21
BULL FORK WEST	0.0	31.0	130.0	69.0	29.8	130.0			1.35
PARKERS MILL	214.0	28.8	128.0	606.0	28.5	124.0	4.2	1.3	2.53
SLATE CREEK	0.0	28.0	130.0	0.0	28.0	130.0	8.6	4.0	1.12
I 24	120.0	31.0	120.0	120.0	31.0	120.0			2.8
I 471	0.0	27.4	131.0	0.0	25.9	131.0	11.5	11.4	* <1.18
I 71 @34.5	103.0	29.6	127.0	250.0	24.8	121.0	19.22	19.5	1.52
I 71 @55.0	103.0	9.6	127.0	250.0	224.8	121.0	24.5	23.0	1.37

* - FAILURE PLANE EXISTS - WATER TABLE RISING FACTOR OF SAFETY PROBABLY LESS THAN 1.18 NOW

TABLE 2. THEORETICAL SITE CONDITIONS

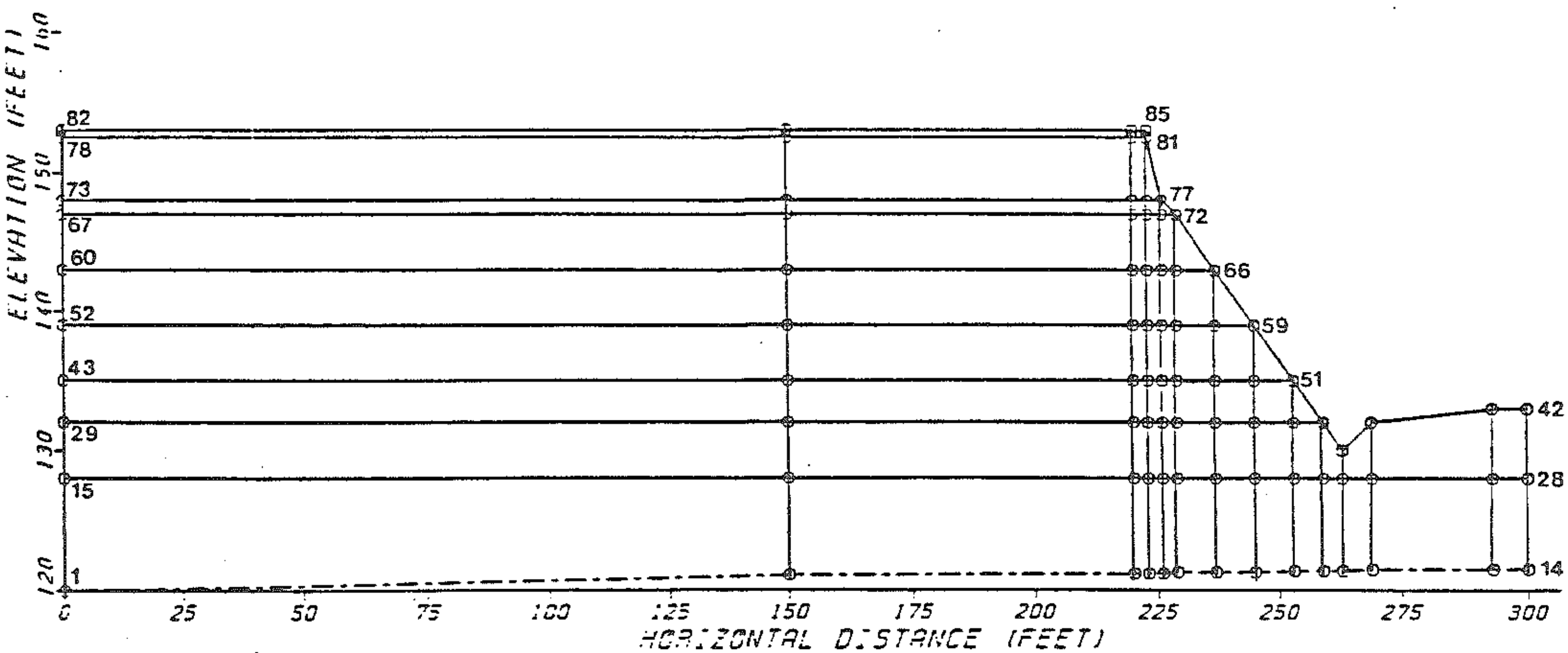
SERIES 1 - "+"
SERIES 2 - "X"
SERIES 3 - "O"

CONSTANTS: FOUNDATION COHESION 250(PSF)
FOUNDATION FRICTION ANGLE 24

			FILL COHESION (PSF)																											
			0.0				250				500				900				1000				1500				2000			
			FILL FRICTION ANGLE				FILL FRICTION ANGLE				FILL FRICTION ANGLE				FILL FRICTION ANGLE				FILL FRICTION ANGLE				FILL FRICTION ANGLE				FILL FRICTION ANGLE			
SIDE SLOPE	FILL HEIGHT (FEET)	FOUNDATION DEPTH (FEET)	21	25	29	33	21	25	29	30	21	25	29	33	21	25	29	33	21	25	29	33	21	25	29	30	21	25	29	3
		5											+																	
		10											+																	
	40	15											+																	
		20											+																	
		30											+																	
		5											+																	
		10											+																	
2:1	100	15											+																	
		20			X				X		O	O	+O	O			X				X				X				X	
		30											+																	
		5											+																	
		10											+																	
	200	15											+																	
		20											+																	
		30											+																	
		5											+																	
		10											+																	
	40	15											+																	
		20											+																	
		30											+																	
		5											+																	
		10											+																	
2.5:1	100	15											+																	
		20			X				X		O	O	+O	O			X				X				X				X	
		30											+																	
		5											+																	
		10											+																	
	200	15											+																	
		20											+																	
		30											+																	
		5											+																	
		10											+																	
	40	15											+																	
		20											+																	
		30											+																	
		5											+																	
		10											+																	
3:1	100	15											+																	
		20			X				X		O	O	+O	O			X				X				X				X	
		30											+																	
		5											+																	
		10											+																	
	200	15											+																	
		20											+																	
		30											+																	

APPENDIX

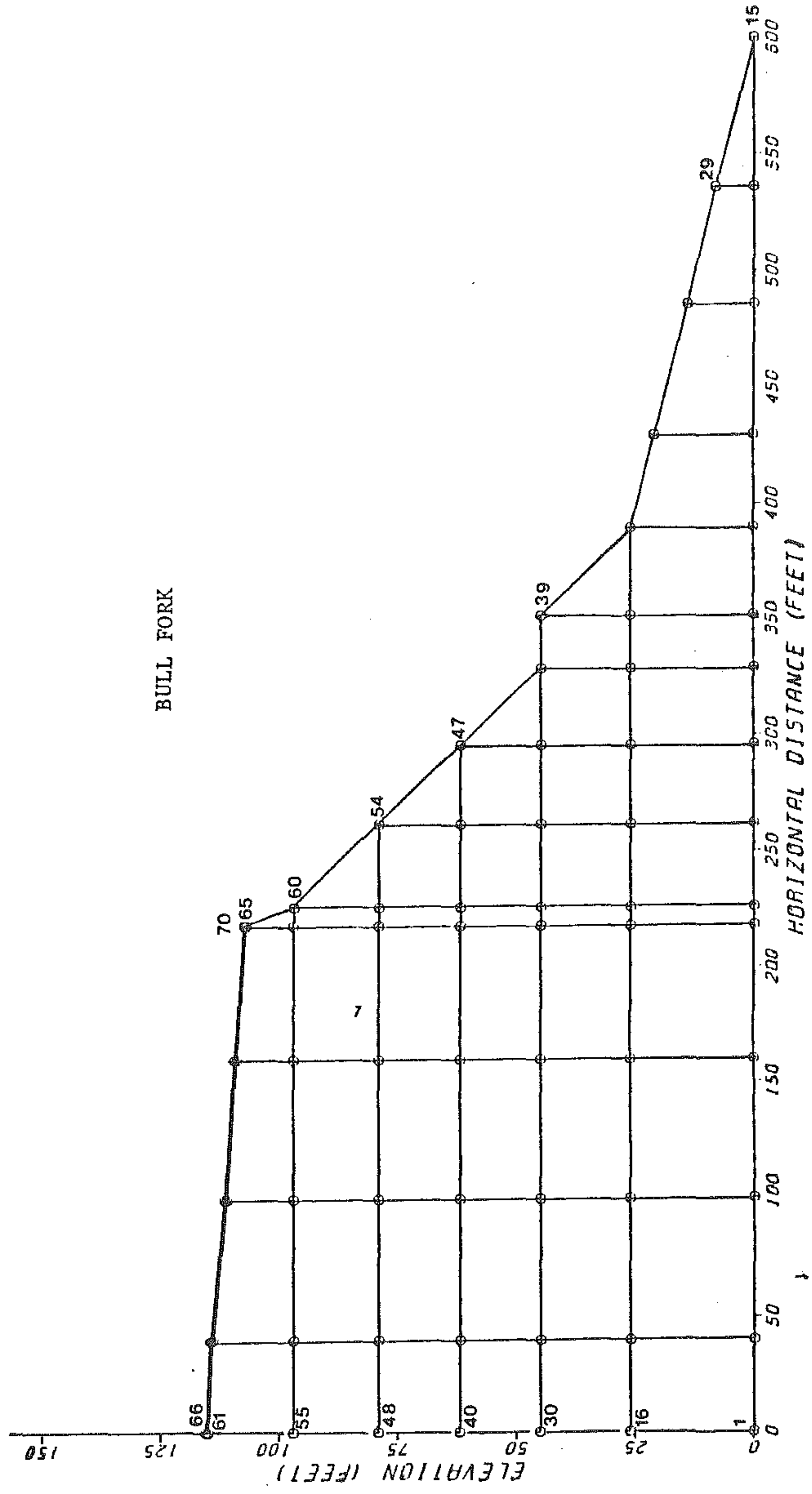
PARKERS MILL



PARKERS MILL

MAT	UNIT WT	K	MODULUS KUR	N	D	POISSON RATIO G	F	C	PHI	FAIL.RATIO	KO	CI
1	145.0000	51.0	153.0	1.2600	5.2056	0.3000	0.1400	*********	0.0	0.9200	0.0	38.0
2	124.0000	51.0	90.0	1.2600	5.2056	0.3000	0.1400	606.0000	28.5000	0.9200	0.0	38.0
3	128.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	214.0000	28.8000	0.9200	0.5000	38.0

NP	DELTA-X	DELTA-Y	X-DISP	Y-DISP	TOTAL	NP
1	0.0	0.0	0.0	0.0	0.0	1
2	0.0	0.0	0.0	0.0	0.0	2
3	0.0	0.0	0.0	0.0	0.0	3
4	0.0	0.0	0.0	0.0	0.0	4
5	0.0	0.0	0.0	0.0	0.0	5
6	0.0	0.0	0.0	0.0	0.0	6
7	0.0	0.0	0.0	0.0	0.0	7
8	0.0	0.0	0.0	0.0	0.0	8
9	0.0	0.0	0.0	0.0	0.0	9
10	0.0	0.0	0.0	0.0	0.0	10
11	0.0	0.0	0.0	0.0	0.0	11
12	0.0	0.0	0.0	0.0	0.0	12
13	0.0	0.0	0.0	0.0	0.0	13
14	0.0	0.0	0.0	0.0	0.0	14
15	0.0	-0.0003	0.0	-0.1721	0.1721	15
16	-0.0009	-0.0001	-0.0147	-0.1745	0.1751	16
17	0.0070	-0.0013	0.1225	-0.3012	0.3252	17
18	0.0076	-0.0036	0.1546	-0.3326	0.3668	18
19	0.0083	-0.0066	0.1966	-0.4420	0.4837	19
20	0.0100	-0.0074	0.2690	-0.5999	0.6575	20
21	0.0110	-0.0065	0.3874	-0.6221	0.7328	21
22	0.0038	0.0009	0.2475	-0.1783	0.3050	22
23	0.0021	0.0005	0.2112	-0.0622	0.2202	23
24	0.0012	0.0003	0.1371	0.0102	0.1374	24
25	0.0009	0.0002	0.1040	0.0054	0.1041	25
26	0.0005	0.0001	0.0629	0.0090	0.0636	26
27	-0.0000	0.0000	-0.0058	0.0014	0.0059	27
28	0.0	-0.0000	0.0	-0.0015	0.0015	28
29	0.0	-0.0029	0.0	-0.3251	0.3251	29
30	-0.0013	-0.0030	-0.0423	-0.3263	0.3290	30
31	0.0099	-0.0047	0.2540	-0.4710	0.5352	31
32	0.0126	-0.0052	0.3231	-0.5292	0.6200	32
33	0.0134	-0.0075	0.4114	-0.6262	0.7492	33
34	0.0129	-0.0080	0.4594	-0.7211	0.8550	34
35	0.0095	-0.0047	0.4572	-0.6804	0.8198	35
36	0.0086	0.0027	0.5854	-0.2294	0.6288	36
37	0.0049	0.0013	0.5987	-0.0543	0.6012	37
38	0.0028	0.0013	0.4389	0.1193	0.4548	38
39	0.0011	0.0002	0.1508	0.0293	0.1536	39
40	0.0007	0.0003	0.0823	0.0333	0.0888	40
41	-0.0000	0.0000	-0.0035	0.0031	0.0047	41
42	0.0	-0.0000	0.0	-0.0033	0.0033	42
43	0.0	-0.0047	0.0	-0.3421	0.3421	43
44	-0.0015	-0.0051	-0.0552	-0.3550	0.3592	44
45	0.0114	-0.0073	0.3301	-0.5167	0.6131	45
46	0.0132	-0.0074	0.4178	-0.5966	0.7283	46
47	0.0141	-0.0074	0.4894	-0.6566	0.8190	47
48	0.0134	-0.0073	0.5207	-0.7184	0.8873	48
49	0.0113	-0.0038	0.6297	-0.6310	0.8914	49
50	0.0087	0.0030	0.6583	-0.2348	0.6989	50
51	0.0077	0.0026	0.7578	0.1629	0.7751	51
52	0.0	-0.0076	0.0	-0.3518	0.3518	52
53	-0.0014	-0.0082	-0.0674	-0.3695	0.3756	53
54	0.0121	-0.0112	0.3942	-0.5362	0.6655	54
55	0.0147	-0.0099	0.4899	-0.6155	0.7867	55
56	0.0151	-0.0077	0.5649	-0.6498	0.8610	56
57	0.0150	-0.0063	0.6330	-0.6277	0.8915	57
58	0.0113	-0.0021	0.6328	-0.5590	0.8444	58
59	0.0090	0.0036	0.6818	0.0133	0.6819	59
60	0.0	-0.0110	0.0	-0.3396	0.3396	60
61	-0.0008	-0.0120	-0.0563	-0.3542	0.3587	61
62	0.0109	-0.0157	0.3480	-0.4988	0.6082	62
63	0.0131	-0.0135	0.4520	-0.5941	0.7465	63
64	0.0147	-0.0070	0.5221	-0.5050	0.7263	64
65	0.0130	-0.0057	0.4786	-0.5293	0.7136	65
66	0.0129	-0.0009	0.5903	-0.2822	0.6543	66
67	0.0	-0.0155	0.0	-0.2602	0.2602	67
68	0.0007	-0.0168	-0.0074	-0.2744	0.2745	68
69	0.0065	-0.0208	0.1266	-0.3444	0.3670	69
70	0.0052	-0.0159	0.0903	-0.2706	0.2853	70
71	0.0	-0.0048	0.0	-0.1250	0.1250	71
72	0.0017	-0.0058	0.0308	-0.1150	0.1191	72
73	0.0	-0.0169	0.0	-0.2290	0.2290	73
74	0.0012	-0.0181	-0.0028	-0.2423	0.2424	74
75	0.0050	-0.0220	0.0944	-0.2954	0.3101	75
76	0.0	-0.0163	0.0	-0.1895	0.1895	76
77	0.0	-0.0028	0.0	-0.0509	0.0509	77
78	0.0	-0.0276	0.0	-0.0276	0.0276	78
79	0.0033	-0.0292	0.0033	-0.0292	0.0294	79
80	-0.0007	-0.0325	-0.0007	-0.0325	0.0325	80
81	0.0	-0.0264	0.0	-0.0264	0.0264	81
82	0.0	0.0	0.0	0.0	0.0	82
83	0.0	0.0	0.0	0.0	0.0	83
84	0.0	0.0	0.0	0.0	0.0	84
85	0.0	0.0	0.0	0.0	0.0	85

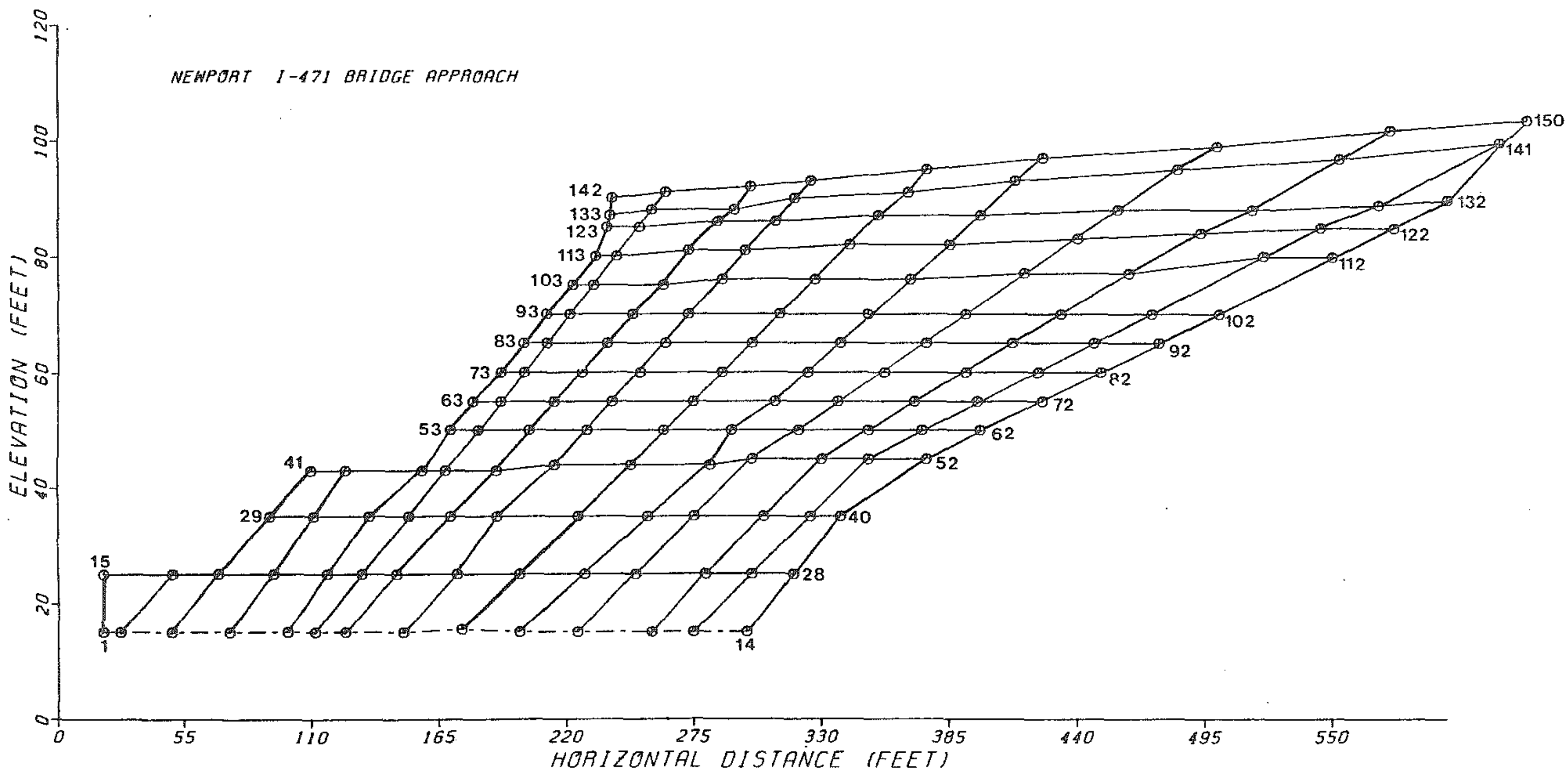


BULL FORK
HORIZONTAL DISTANCE (FEET)
ELEVATION (FEET)
0
50
100
150
200
250
300
350
400
450
500
550
600
25
75
125
66
61
55
48
40
30
16
1
70
65
60
54
47
39
29
15

BULL FORK

MAT	UNIT WT	K	MODULUS KUR	N	D	POISSON RATIO G	F	C	PHI	FAIL.RATIO	KO	CF
1	130.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	69.0000	29.8000	0.9200	0.0	38.0
2	130.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	0.0	31.0000	0.9200	0.5000	38.0

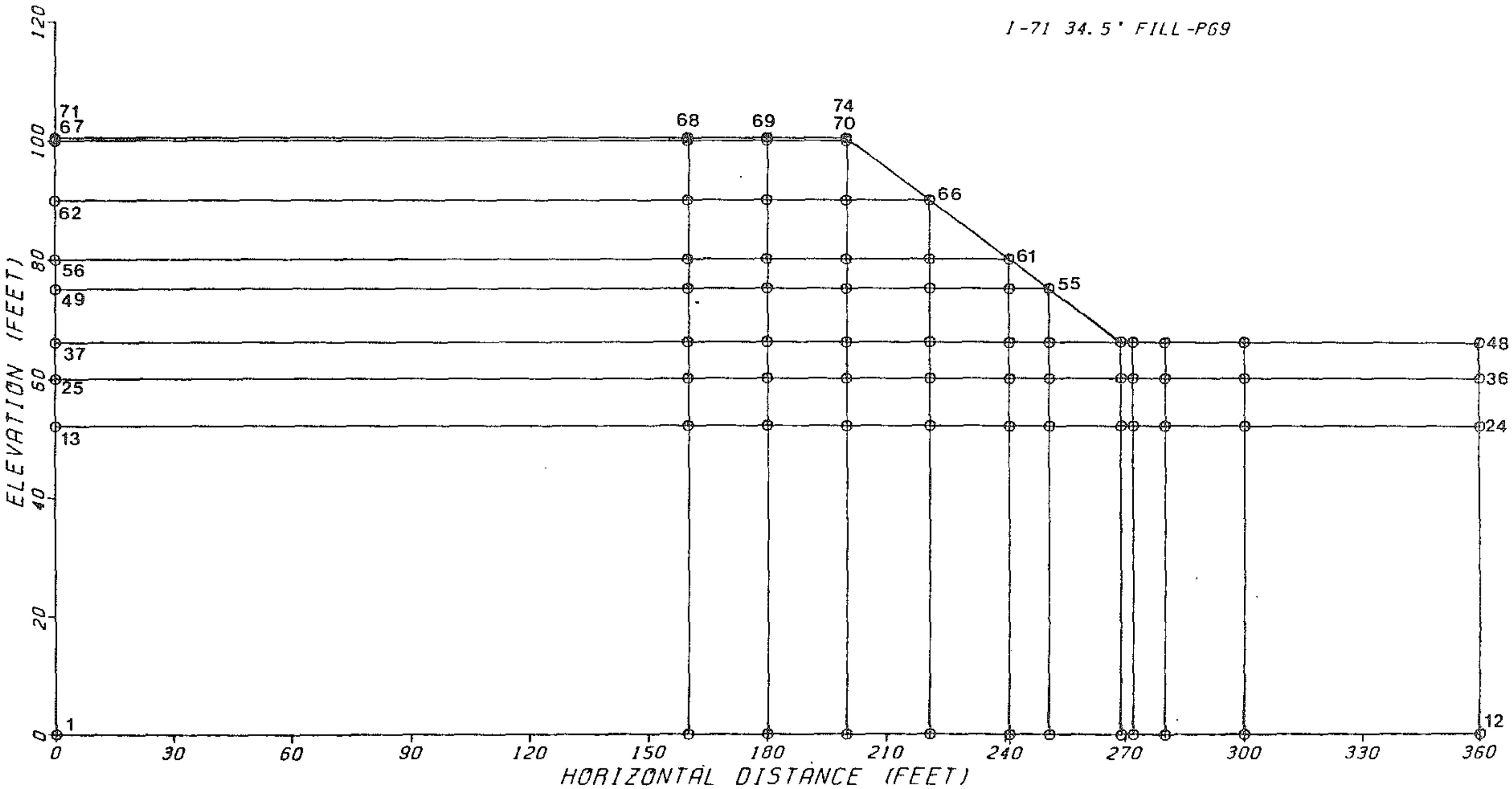
NP	DELTA-X	DELTA-Y	X-DISP	Y-DISP	TOTAL	NP
1	0.0	0.0	0.0	0.0	0.0	1
2	0.0	0.0	0.0	0.0	0.0	2
3	0.0	0.0	0.0	0.0	0.0	3
4	0.0	0.0	0.0	0.0	0.0	4
5	0.0	0.0	0.0	0.0	0.0	5
6	0.0	0.0	0.0	0.0	0.0	6
7	0.0	0.0	0.0	0.0	0.0	7
8	0.0	0.0	0.0	0.0	0.0	8
9	0.0	0.0	0.0	0.0	0.0	9
10	0.0	0.0	0.0	0.0	0.0	10
11	0.0	0.0	0.0	0.0	0.0	11
12	0.0	0.0	0.0	0.0	0.0	12
13	0.0	0.0	0.0	0.0	0.0	13
14	0.0	0.0	0.0	0.0	0.0	14
15	0.0	0.0	0.0	0.0	0.0	15
16	0.0	-0.0103	0.0	-1.8927	1.8927	16
17	0.0003	-0.0104	0.0350	-1.8832	1.8836	17
18	0.0016	-0.0106	0.1004	-1.8403	1.8430	18
19	0.0045	-0.0090	0.2087	-1.7431	1.7555	19
20	0.0054	-0.0059	0.3916	-1.5924	1.6399	20
21	0.0056	-0.0074	0.4232	-1.5674	1.6235	21
22	0.0040	-0.0017	0.4732	-1.2382	1.3256	22
23	0.0026	-0.0013	0.4848	-1.0139	1.1238	23
24	0.0011	-0.0003	0.3604	-0.5238	0.6358	24
25	0.0006	-0.0002	0.2343	-0.6665	0.7065	25
26	0.0001	-0.0000	0.1619	-0.1028	0.1918	26
27	0.0000	0.0000	0.0048	0.0147	0.0154	27
28	-0.0000	-0.0000	0.0005	-0.0018	0.0019	28
29	0.0000	0.0000	0.0019	0.0003	0.0019	29
30	0.0	-0.0176	0.0	-2.9090	2.9090	30
31	0.0009	-0.0177	0.0726	-2.8963	2.8972	31
32	0.0028	-0.0187	0.1867	-2.8366	2.8427	32
33	0.0102	-0.0185	0.4702	-2.7743	2.8139	33
34	0.0155	-0.0086	0.9941	-2.5405	2.7281	34
35	0.0163	-0.0133	1.1500	-2.6364	2.8763	35
36	0.0176	-0.0004	1.7737	-1.8130	2.5364	36
37	0.0099	-0.0001	1.5419	-1.2218	1.9673	37
38	0.0047	0.0016	1.2038	0.1417	1.2121	38
39	0.0025	-0.0000	0.7419	-0.0448	0.7433	39
40	0.0	-0.0253	0.0	-3.1419	3.1419	40
41	0.0017	-0.0255	0.0958	-3.1243	3.1258	41
42	0.0044	-0.0274	0.2819	-3.0673	3.0802	42
43	0.0142	-0.0303	0.6545	-3.0397	3.1094	43
44	0.0302	-0.0124	1.7615	-2.9267	3.4159	44
45	0.0309	-0.0189	2.1855	-2.7230	3.4916	45
46	0.0316	0.0033	2.6483	-1.7493	3.1739	46
47	0.0261	0.0021	3.6241	0.0939	3.6253	47
48	0.0	-0.0349	0.0	-2.8359	2.8359	48
49	0.0025	-0.0350	0.1515	-2.7861	2.7902	49
50	0.0070	-0.0374	0.2997	-2.7109	2.7274	50
51	0.0178	-0.0453	0.9076	-2.8154	2.9581	51
52	0.0461	-0.0208	2.0304	-2.4315	3.1678	52
53	0.0472	-0.0211	2.0492	-2.8336	3.4969	53
54	0.0422	0.0058	3.9374	-0.0876	3.9383	54
55	0.0	-0.0483	0.0	-1.9095	1.9095	55
56	0.0040	-0.0483	0.1522	-1.7959	1.8024	56
57	0.0104	-0.0520	0.3325	-1.6609	1.6938	57
58	0.0226	-0.0633	0.5831	-1.5832	1.6872	58
59	0.0546	-0.0422	1.4519	-1.2524	1.9174	59
60	0.0615	-0.0208	1.6808	-0.6310	1.7953	60
61	0.0	-0.0708	0.0	-0.0708	0.0708	61
62	0.0071	-0.0717	0.0071	-0.0717	0.0721	62
63	0.0155	-0.0709	0.0155	-0.0709	0.0725	63
64	0.0258	-0.0859	0.0258	-0.0859	0.0897	64
65	0.0584	-0.0586	0.0584	-0.0586	0.0827	65
66	0.0	0.0	0.0	0.0	0.0	66
67	0.0	0.0	0.0	0.0	0.0	67
68	0.0	0.0	0.0	0.0	0.0	68
69	0.0	0.0	0.0	0.0	0.0	69
70	0.0	0.0	0.0	0.0	0.0	70



NEWPORT I-471 BRIDGE APPROACH
ELEVATION (FEET)
HORIZONTAL DISTANCE (FEET)
0
20
40
60
80
100
120
0
55
110
165
220
275
330
385
440
495
550
1
14
15
28
29
40
41
52
53
62
63
72
73
82
83
92
93
102
103
112
113
122
123
132
133
141
142
150

MAT	UNIT WT	K	MODULUS KUR	N	D	POISSON RATIO G	F	C	PHI	FAIL.RATIO	KO	CF
1	120.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	288.0000	26.0000	0.9200	0.5000	38.0
2	115.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	0.0	30.5000	0.9200	0.0	38.0

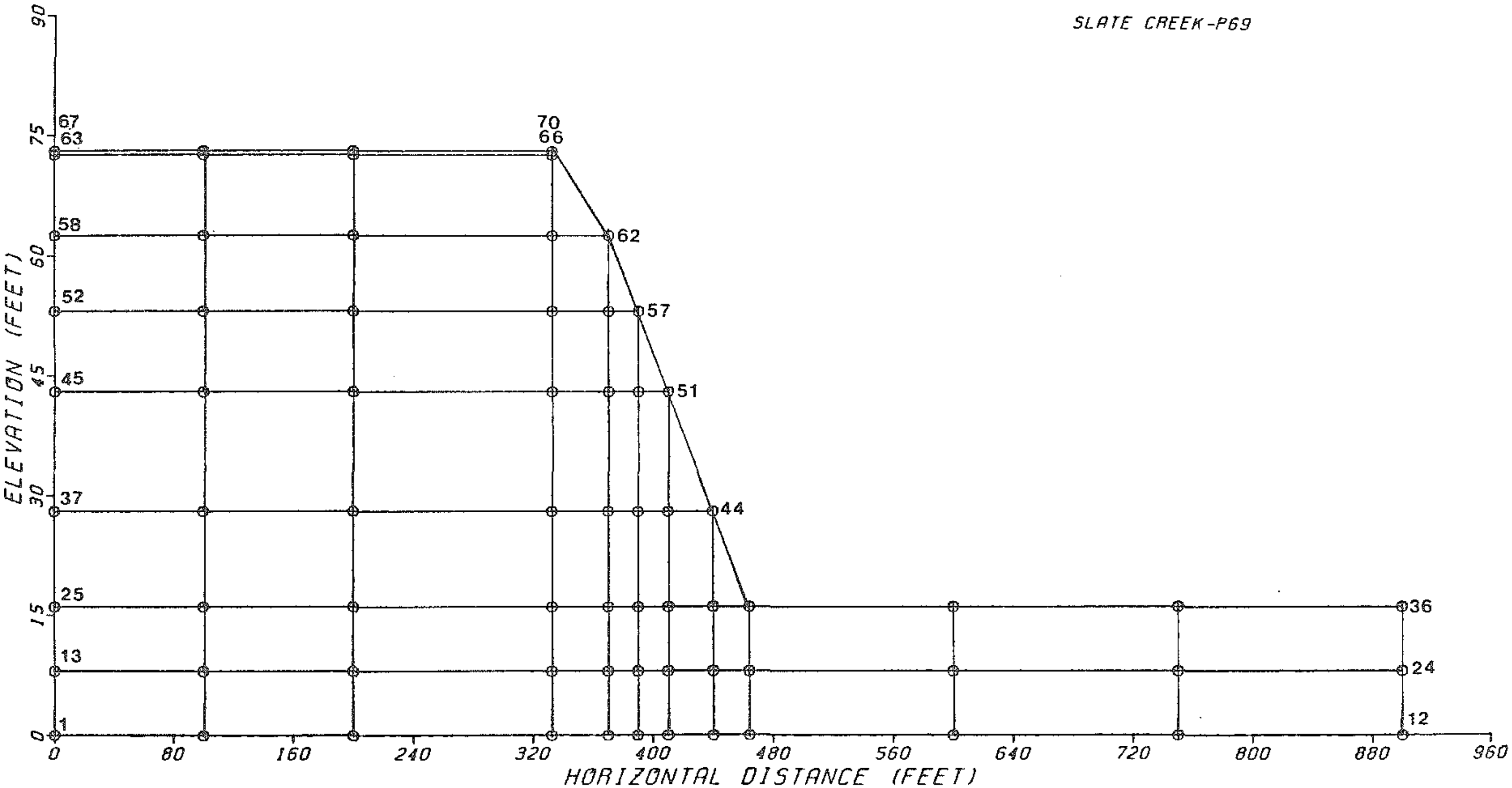
NP	DELTA-X	DELTA-Y	X-DISP	Y-DISP	TOTAL
1	0.0	0.0	0.0	0.0	0.0
2	-0.0023	0.0	-0.0562	0.0	0.0562
3	-0.0118	0.0	-0.2587	0.0	0.2587
4	-0.0226	0.0	-0.4845	0.0	0.4845
5	-0.0305	0.0	-0.6521	0.0	0.6521
6	-0.0338	0.0	-0.7208	0.0	0.7208
7	-0.0370	0.0	-0.7887	0.0	0.7887
8	-0.0425	0.0	-0.9286	0.0	0.9286
9	-0.0483	0.0	-1.0516	0.0	1.0516
10	-0.0534	0.0	-1.0368	0.0	1.0368
11	-0.0578	0.0	-0.8615	0.0	0.8615
12	-0.0534	0.0	-0.4656	0.0	0.4656
13	-0.0298	0.0	-0.0450	0.0	0.0450
14	-0.0064	0.0	0.1435	0.0	0.1435
15	0.0	0.0	0.0	0.0	0.0
16	-0.0116	0.0014	-0.2596	0.0311	0.2615
17	-0.0215	0.0009	-0.4613	0.0216	0.4618
18	-0.0292	0.0008	-0.6258	0.0195	0.6261
19	-0.0352	0.0009	-0.7469	0.0226	0.7472
20	-0.0387	0.0007	-0.8223	0.0193	0.8226
21	-0.0421	0.0006	-0.9004	0.0189	0.9006
22	-0.0477	0.0008	-1.0252	-0.0706	1.0276
23	-0.0528	-0.0004	-1.0115	-0.2115	1.0334
24	-0.0563	-0.0036	-0.8358	-0.3709	0.9144
25	-0.0567	-0.0144	-0.5570	-0.4239	0.7000
26	-0.0317	-0.0248	-0.0716	-0.4586	0.4642
27	-0.0110	-0.0175	0.0385	-0.2005	0.2042
28	0.0	0.0	0.0	0.0	0.0
29	-0.0308	0.0014	-0.6593	0.0342	0.6602
30	-0.0344	0.0018	-0.7262	0.0433	0.7275
31	-0.0396	0.0014	-0.8263	0.0355	0.8270
32	-0.0438	0.0012	-0.8966	0.0177	0.8968
33	-0.0472	0.0018	-0.9608	-0.0859	0.9646
34	-0.0502	0.0007	-0.9843	-0.3188	1.0347
35	-0.0547	-0.0062	-0.8140	-0.6691	1.0537
36	-0.0472	-0.0214	-0.4270	-0.8296	0.9330
37	-0.0437	-0.0489	-0.2428	-0.9785	1.0082
38	-0.0097	-0.0336	-0.0236	-0.2907	0.2916
39	-0.0032	-0.0188	-0.0597	-0.1352	0.1478
40	0.0	0.0	0.0	0.0	0.0
41	-0.0351	0.0022	-0.7426	0.0513	0.7443
42	-0.0374	0.0023	-0.7825	0.0585	0.7846
43	-0.0466	0.0017	-0.6953	0.0314	0.6960
44	-0.0475	0.0026	-0.6973	0.0183	0.6976
45	-0.0487	0.0010	-0.7156	-0.1840	0.7389
46	-0.0520	-0.0039	-0.6940	-0.6005	0.9177
47	-0.0500	-0.0310	-0.5005	-0.9593	1.0821
48	-0.0334	-0.0600	-0.2217	-0.8784	0.9060
49	-0.0216	-0.0621	-0.1843	-0.4986	0.5316
50	-0.0075	-0.0216	-0.0951	-0.2045	0.2255
51	-0.0040	-0.0020	-0.0569	-0.0897	0.1062
52	0.0	0.0	0.0	0.0	0.0
53	-0.0478	0.0028	-0.5784	0.0323	0.5793
54	-0.0477	0.0016	-0.5661	-0.0196	0.5665
55	-0.0487	0.0015	-0.5476	-0.2689	0.6100
56	-0.0492	-0.0173	-0.5099	-0.6972	0.8638
57	-0.0410	-0.0500	-0.3545	-0.9542	1.0180
58	-0.0267	-0.0704	-0.2436	-0.6498	0.6940
59	-0.0150	-0.0402	-0.1318	-0.3099	0.3368
60	-0.0062	-0.0075	-0.0823	-0.1084	0.1361
61	-0.0025	-0.0052	-0.0342	-0.0598	0.0689
62	0.0	0.0	0.0	0.0	0.0
63	-0.0479	0.0015	-0.4694	0.0230	0.4699
64	-0.0477	0.0040	-0.4496	-0.0256	0.4503
65	-0.0457	-0.0058	-0.4200	-0.3117	0.5230
66	-0.0443	-0.0290	-0.3801	-0.7224	0.8164
67	-0.0324	-0.0715	-0.2870	-0.7450	0.7984
68	-0.0207	-0.0578	-0.1814	-0.4478	0.4832
69	-0.0116	-0.0240	-0.1091	-0.1773	0.2082
70	-0.0049	-0.0103	-0.0639	-0.1169	0.1332
71	-0.0040	-0.0054	-0.0440	-0.0566	0.0717
72	0.0	0.0	0.0	0.0	0.0
73	-0.0462	0.0041	-0.3577	0.0127	0.3579
74	-0.0439	0.0013	-0.3353	-0.0501	0.3390
75	-0.0427	-0.0158	-0.3113	-0.3876	0.4971
76	-0.0376	-0.0536	-0.2936	-0.6284	0.6936
77	-0.0279	-0.0754	-0.2132	-0.6066	0.6430
78	-0.0178	-0.0453	-0.1569	-0.3037	0.3419
79	-0.0076	-0.0190	-0.0824	-0.1465	0.1681
80	-0.0089	-0.0119	-0.0799	-0.1060	0.1328
81	-0.0035	-0.0064	-0.0367	-0.0630	0.0729
82	0.0	0.0	0.0	0.0	0.0
83	-0.0406	0.0016	-0.2455	-0.0016	0.2455
84	-0.0409	-0.0029	-0.2317	-0.0944	0.2502
85	-0.0372	-0.0363	-0.2249	-0.3587	0.4234
86	-0.0315	-0.0704	-0.2021	-0.5474	0.5835
87	-0.0257	-0.0702	-0.1739	-0.4466	0.4792
88	-0.0136	-0.0398	-0.1253	-0.2328	0.2644
89	-0.0090	-0.0134	-0.0741	-0.0878	0.1149
90	-0.0095	-0.0160	-0.0845	-0.1300	0.1551
91	-0.0026	-0.0058	-0.0089	-0.0477	0.0485
92	0.0	0.0	0.0	0.0	0.0
93	-0.0363	-0.0029	-0.1597	-0.0510	0.1677
94	-0.0358	-0.0164	-0.1576	-0.1202	0.1984
95	-0.0323	-0.0577	-0.1501	-0.3331	0.3654
96	-0.0281	-0.0757	-0.1475	-0.4069	0.4328
97	-0.0223	-0.0705	-0.1402	-0.3292	0.3578
98	-0.0130	-0.0312	-0.0861	-0.1452	0.1688
99	-0.0100	-0.0160	-0.0738	-0.0889	0.1155
100	-0.0077	-0.0151	-0.0543	-0.0974	0.1116
101	-0.0011	-0.0089	0.0736	-0.0396	0.0836
102	0.0	0.0	0.0	0.0	0.0
103	-0.0277	-0.0181	-0.0764	-0.0689	0.1029
104	-0.0275	-0.0351	-0.0755	-0.1225	0.1439
105	-0.0263	-0.0700	-0.0789	-0.2377	0.2504
106	-0.0263	-0.0784	-0.0851	-0.2430	0.2575
107	-0.0236	-0.0605	-0.0826	-0.1771	0.1954
108	-0.0127	-0.0206	-0.0466	-0.0637	0.0790
109	-0.0101	-0.0223	-0.0424	-0.0953	0.1043
110	-0.0036	-0.0226	-0.0116	-0.0965	0.0972
111	-0.0049	-0.0104	-0.0038	-0.0483	0.0485
112	0.0	0.0	0.0	0.0	0.0
113	-0.0201	-0.0371	-0.0323	-0.0637	0.0714
114	-0.0210	-0.0512	-0.0344	-0.0871	0.0936
115	-0.0241	-0.0738	-0.0423	-0.1190	0.1263
116	-0.0246	-0.0833	-0.0459	-0.1203	0.1288
117	-0.0229	-0.0463	-0.0420	-0.0750	0.0860
118	-0.0141	-0.0241	-0.0249	-0.0445	0.0510
119	-0.0064	-0.0328	-0.0136	-0.0911	0.0921
120	-0.0052	-0.0132	-0.0155	-0.0428	0.0454
121	-0.0049	-0.0152	-0.0133	-0.0484	0.0502
122	0.0	0.0	0.0	0.0	0.0
123	-0.0154	-0.0467	-0.0154	-0.0467	0.0491
124	-0.0182	-0.0589	-0.0182	-0.0589	0.0617
125	-0.0230	-0.0821	-0.0230	-0.0821	0.0853
126	-0.0259	-0.0789	-0.0259	-0.0789	0.0830
127	-0.0244	-0.0360	-0.0244	-0.0360	0.0435
128	-0.0108	-0.0381	-0.0108	-0.0381	0.0396
129	-0.0075	-0.0261	-0.0075	-0.0261	0.0272
130	-0.0057	-0.0179	-0.0057	-0.0179	0.0188
131	-0.0042	-0.0192	-0.0042	-0.0192	0.0196
132	0.0	0.0	0.0	0.0	0.0
133	0.0	0.0	0.0	0.0	0.0
134	0.0	0.0	0.0	0.0	0.0
135	0.0	0.0	0.0	0.0	0.0
136	0.0	0.0	0.0	0.0	0.0
137	0.0	0.0	0.0	0.0	0.0
138	0.0	0.0	0.0	0.0	0.0
139	0.0	0.0	0.0	0.0	0.0
140	0.0	0.0	0.0	0.0	0.0
141	0.0	0.0	0.0	0.0	0.0
142	0.0	0.0	0.0	0.0	0.0
143	0.0	0.0	0.0	0.0	0.0
144	0.0	0.0	0.0	0.0	0.0
145	0.0	0.0	0.0	0.0	0.0
146	0.0	0.0	0.0	0.0	0.0
147	0.0	0.0	0.0	0.0	0.0
148	0.0	0.0	0.0	0.0	0.0
149	0.0	0.0	0.0	0.0	0.0
150	0.0	0.0	0.0	0.0	0.0


I-71 34.5' FILL-PG9
ELEVATION (FEET)
HORIZONTAL DISTANCE (FEET)
0
20
40
60
80
100
120
0
30
60
90
120
150
180
210
240
270
300
330
360
71
67
68
69
74
70
66
61
55
62
56
49
37
25
13
1
48
36
24
12

I-75 34.5" FILL

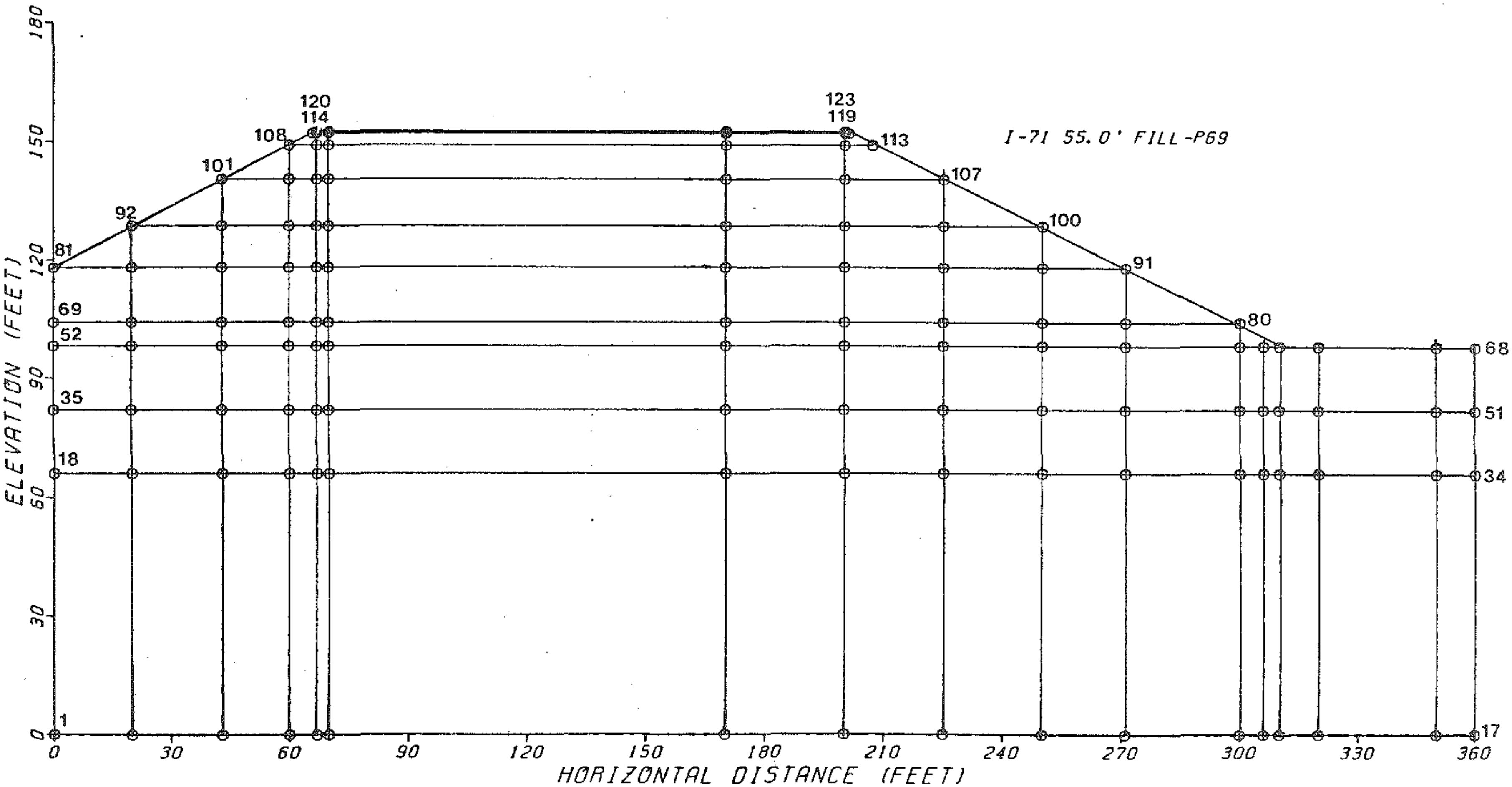
MAT	UNIT WT	K	MODULUS KUR	N	D	POISSON RATIO G	F	C	PHI	FAIL.RATIO	KO	CF
1	114.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	250.0000	24.8000	0.9200	0.0	38.0
2	121.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	250.0000	24.8000	0.9200	0.0	38.0
3	129.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	250.0000	24.8000	0.9200	0.0	38.0
4	127.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	216.0000	26.0000	0.9200	0.5000	38.0
5	127.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	103.0000	29.6000	0.9200	0.5000	38.0
6	127.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	0.0	32.0000	0.9200	0.5000	38.0

NP	DELTA-X	DELTA-Y	X-DISP	Y-DISP	TOTAL	NP
1	0.0	0.0	0.0	0.0	0.0	1
2	0.0	0.0	0.0	0.0	0.0	2
3	0.0	0.0	0.0	0.0	0.0	3
4	0.0	0.0	0.0	0.0	0.0	4
5	0.0	0.0	0.0	0.0	0.0	5
6	0.0	0.0	0.0	0.0	0.0	6
7	0.0	0.0	0.0	0.0	0.0	7
8	0.0	0.0	0.0	0.0	0.0	8
9	0.0	0.0	0.0	0.0	0.0	9
10	0.0	0.0	0.0	0.0	0.0	10
11	0.0	0.0	0.0	0.0	0.0	11
12	0.0	0.0	0.0	0.0	0.0	12
13	0.0	-0.0110	0.0	-0.7587	0.7587	13
14	0.0028	-0.0103	0.0875	-0.7629	0.7679	14
15	0.0032	-0.0087	0.1001	-0.7513	0.7580	15
16	0.0034	-0.0063	0.1064	-0.7118	0.7197	16
17	0.0031	-0.0040	0.1091	-0.6131	0.6227	17
18	0.0023	-0.0020	0.0860	-0.4316	0.4401	18
19	0.0019	-0.0015	0.0712	-0.3297	0.3373	19
20	0.0012	-0.0008	0.0550	-0.1577	0.1670	20
21	0.0011	-0.0007	0.0494	-0.1272	0.1364	21
22	0.0009	-0.0004	0.0482	-0.0668	0.0823	22
23	0.0006	0.0000	0.0379	-0.0035	0.0381	23
24	0.0	0.0000	0.0	0.0031	0.0031	24
25	0.0	-0.0113	0.0	-0.9550	0.9550	25
26	0.0033	-0.0106	0.1023	-0.9599	0.9653	26
27	0.0042	-0.0091	0.1300	-0.9541	0.9629	27
28	0.0046	-0.0063	0.1736	-0.9030	0.9196	28
29	0.0044	-0.0040	0.1401	-0.8047	0.8168	29
30	0.0038	-0.0016	0.2160	-0.6000	0.6377	30
31	0.0030	-0.0012	0.3117	-0.5030	0.5917	31
32	0.0020	-0.0004	0.1782	-0.0623	0.1887	32
33	0.0017	-0.0002	0.1527	-0.1230	0.1961	33
34	0.0011	-0.0001	0.0979	-0.0647	0.1173	34
35	0.0006	0.0000	0.0382	-0.0031	0.0383	35
36	0.0	0.0000	0.0	0.0035	0.0035	36
37	0.0	-0.0190	0.0	-1.5271	1.5271	37
38	0.0046	-0.0182	0.1260	-1.5306	1.5358	38
39	0.0064	-0.0156	0.2009	-1.5199	1.5331	39
40	0.0083	-0.0106	0.2000	-1.3628	1.3774	40
41	0.0070	-0.0047	0.6151	-1.5796	1.6951	41
42	0.0052	-0.0004	0.5012	-0.8898	1.0212	42
43	0.0040	-0.0012	0.3757	-0.6811	0.7778	43
44	0.0020	0.0005	0.6067	0.0354	0.6077	44
45	0.0017	0.0002	0.4821	0.0139	0.4823	45
46	0.0013	0.0002	0.2013	-0.0073	0.2015	46
47	0.0006	0.0000	0.0357	-0.0027	0.0358	47
48	0.0	0.0000	0.0	0.0038	0.0038	48
49	0.0	-0.0238	0.0	-1.4393	1.4393	49
50	0.0057	-0.0232	0.1720	-1.4495	1.4596	50
51	0.0092	-0.0215	0.2455	-1.4161	1.4372	51
52	0.0142	-0.0151	0.5151	-1.3014	1.3996	52
53	0.0138	-0.0039	0.9638	-1.3444	1.6542	53
54	0.0084	0.0017	1.0342	-0.4523	1.1288	54
55	0.0080	-0.0024	0.9966	-0.0685	0.9990	55
56	0.0	-0.0271	0.0	-1.3339	1.3339	56
57	0.0064	-0.0265	0.1729	-1.3240	1.3353	57
58	0.0104	-0.0260	0.2042	-1.3327	1.3482	58
59	0.0219	-0.0189	0.8088	-1.1948	1.4428	59
60	0.0225	-0.0006	0.9482	-1.1442	1.4860	60
61	0.0122	0.0019	1.8036	-0.0284	1.8038	61
62	0.0	-0.0352	0.0	-0.8225	0.8225	62
63	0.0078	-0.0354	0.1782	-0.8215	0.8406	63
64	0.0180	-0.0383	0.3329	-0.8101	0.8759	64
65	0.0304	-0.0293	0.4741	-0.8984	1.0158	65
66	0.0398	0.0042	1.0855	-0.1744	1.0994	66
67	0.0	-0.0510	0.0	-0.0510	0.0510	67
68	0.0097	-0.0512	0.0097	-0.0512	0.0522	68
69	0.0220	-0.0628	0.0220	-0.0628	0.0665	69
70	0.0339	-0.0361	0.0339	-0.0361	0.0496	70
71	0.0	0.0	0.0	0.0	0.0	71
72	0.0	0.0	0.0	0.0	0.0	72
73	0.0	0.0	0.0	0.0	0.0	73
74	0.0	0.0	0.0	0.0	0.0	74


SLATE CREEK-P69
ELEVATION (FEET)
HORIZONTAL DISTANCE (FEET)
0
15
30
45
60
75
90
0
80
160
240
320
400
480
560
640
720
800
880
960
67
63
70
66
58
62
52
57
45
51
37
44
25
36
13
24
1
12

MAT	UNIT WT	K	MODULUS KUR	N	D	POISSON RATIO G	F	C	PHI	FAIL.RATIO	KO	CF
1	130.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	0.0	28.0000	0.9200	0.0	38.0
2	130.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	0.0	28.0000	0.9200	0.5000	38.0

NP	DELTA-X	DELTA-Y	X-DISP	Y-DISP	TOTAL	NP
1	0.0	0.0	0.0	0.0	0.0	1
2	0.0	0.0	0.0	0.0	0.0	2
3	0.0	0.0	0.0	0.0	0.0	3
4	0.0	0.0	0.0	0.0	0.0	4
5	0.0	0.0	0.0	0.0	0.0	5
6	0.0	0.0	0.0	0.0	0.0	6
7	0.0	0.0	0.0	0.0	0.0	7
8	0.0	0.0	0.0	0.0	0.0	8
9	0.0	0.0	0.0	0.0	0.0	9
10	0.0	0.0	0.0	0.0	0.0	10
11	0.0	0.0	0.0	0.0	0.0	11
12	0.0	0.0	0.0	0.0	0.0	12
13	0.0	-0.0047	0.0	-0.5409	0.5409	13
14	-0.0003	-0.0049	-0.0023	-0.5462	0.5462	14
15	0.0002	-0.0047	-0.0146	-0.5422	0.5424	15
16	0.0031	-0.0033	0.1151	-0.5003	0.5133	16
17	0.0026	-0.0004	0.1741	-0.4032	0.4392	17
18	0.0018	-0.0005	0.2176	-0.3523	0.4141	18
19	0.0013	-0.0008	0.2635	-0.3448	0.4340	19
20	-0.0006	0.0001	0.0674	-0.1749	0.1875	20
21	0.0006	0.0000	0.2318	-0.0105	0.2321	21
22	-0.0005	0.0000	-0.1993	0.0093	0.1995	22
23	0.0000	-0.0000	0.0154	-0.0113	0.0191	23
24	0.0	0.0000	0.0	0.0092	0.0092	24
25	0.0	-0.0051	0.0	-0.7107	0.7107	25
26	-0.0004	-0.0052	-0.0028	-0.7175	0.7175	26
27	0.0002	-0.0050	-0.0216	-0.7171	0.7175	27
28	0.0037	-0.0035	0.1503	-0.6764	0.6929	28
29	0.0032	-0.0001	0.2423	-0.5815	0.6300	29
30	0.0028	-0.0003	0.3151	-0.5309	0.6174	30
31	0.0039	-0.0012	0.6591	-0.6013	0.8921	31
32	0.0047	0.0008	1.1391	-0.2373	1.1636	32
33	0.0005	0.0002	0.2406	0.0697	0.2505	33
34	-0.0004	-0.0000	-0.2013	-0.0115	0.2016	34
35	-0.0000	-0.0000	0.0103	-0.0103	0.0146	35
36	0.0	0.0000	0.0	0.0100	0.0100	36
37	0.0	-0.0111	0.0	-1.2912	1.2912	37
38	-0.0008	-0.0113	-0.0154	-1.2880	1.2881	38
39	-0.0002	-0.0115	-0.0515	-1.3039	1.3049	39
40	0.0115	-0.0095	0.3772	-1.2839	1.3382	40
41	0.0135	0.0013	0.8768	-1.1311	1.4311	41
42	0.0121	-0.0016	1.2626	-1.0100	1.6169	42
43	0.0104	-0.0000	2.0813	-1.0439	2.3284	43
44	0.0037	0.0018	1.5773	0.4794	1.6486	44
45	0.0	-0.0199	0.0	-1.4888	1.4888	45
46	-0.0015	-0.0198	-0.0178	-1.4564	1.4565	46
47	-0.0002	-0.0217	-0.1030	-1.5262	1.5297	47
48	0.0214	-0.0194	0.6876	-1.5496	1.6953	48
49	0.0206	0.0041	1.5408	-1.3041	2.0186	49
50	0.0183	-0.0025	2.1958	-1.0467	2.4325	50
51	0.0174	0.0025	2.2577	-0.1486	2.2626	51
52	0.0	-0.0273	0.0	-1.3130	1.3130	52
53	-0.0021	-0.0267	-0.0192	-1.2532	1.2533	53
54	-0.0001	-0.0305	-0.1053	-1.3737	1.3777	54
55	0.0278	-0.0281	0.7791	-1.3816	1.5861	55
56	0.0272	0.0086	1.5980	-0.8941	1.8311	56
57	0.0237	-0.0040	1.5370	-0.1593	1.5452	57
58	0.0	-0.0370	0.0	-0.8607	0.8607	58
59	-0.0010	-0.0354	-0.0129	-0.8073	0.8074	59
60	-0.0031	-0.0414	-0.0909	-0.9645	0.9688	60
61	0.0390	-0.0392	0.7325	-0.8621	1.1313	61
62	0.0075	0.0199	0.1468	0.4371	0.4611	62
63	0.0	-0.0554	0.0	-0.0554	0.0554	63
64	0.0046	-0.0519	0.0046	-0.0519	0.0521	64
65	-0.0152	-0.0597	-0.0152	-0.0597	0.0616	65
66	0.0608	-0.0618	0.0608	-0.0618	0.0867	66
67	0.0	0.0	0.0	0.0	0.0	67
68	0.0	0.0	0.0	0.0	0.0	68


I-71 55.0' FILL-P69
ELEVATION (FEET)
HORIZONTAL DISTANCE (FEET)
0
30
60
90
120
150
180
210
240
270
300
330
360
1
17
18
34
35
51
52
68
69
80
81
91
92
100
101
107
108
113
114
119
120
123

I-71 55.0" FILL

MAT	UNIT WT	K	MODULUS KUR	N	D	POISSON RATIO G	F	C	PHI	FAIL.RATIO	KO	CF
1	114.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	250.0000	24.8000	0.9200	0.0	38.0
2	121.0000	51.0	87.0	1.2600	5.2056	0.3000	0.1400	250.0000	24.8000	0.9200	0.0	38.0
3	129.0000	51.0	95.0	1.2600	5.2056	0.3000	0.1400	250.0000	24.8000	0.9200	0.0	38.0
4	127.4000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	216.0000	26.0000	0.9200	0.0	38.0
5	127.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	103.0000	29.6000	0.9200	0.5000	38.0
6	127.0000	51.0	76.0	1.2600	5.2056	0.3000	0.1400	0.0	32.0000	0.9200	0.5000	38.0

NP	DELTA-X	DELTA-Y	X-DISP	Y-DISP	TOTAL	NP
1	0.0	0.0	0.0	0.0	0.0	1
2	0.0	0.0	0.0	0.0	0.0	2
3	0.0	0.0	0.0	0.0	0.0	3
4	0.0	0.0	0.0	0.0	0.0	4
5	0.0	0.0	0.0	0.0	0.0	5
6	0.0	0.0	0.0	0.0	0.0	6
7	0.0	0.0	0.0	0.0	0.0	7
8	0.0	0.0	0.0	0.0	0.0	8
9	0.0	0.0	0.0	0.0	0.0	9
10	0.0	0.0	0.0	0.0	0.0	10
11	0.0	0.0	0.0	0.0	0.0	11
12	0.0	0.0	0.0	0.0	0.0	12
13	0.0	0.0	0.0	0.0	0.0	13
14	0.0	0.0	0.0	0.0	0.0	14
15	0.0	0.0	0.0	0.0	0.0	15
16	0.0	0.0	0.0	0.0	0.0	16
17	0.0	0.0	0.0	0.0	0.0	17
18	0.0	-0.0028	0.0	-0.7163	0.7163	18
19	-0.0003	-0.0030	-0.0084	-0.7335	0.7336	19
20	-0.0005	-0.0036	-0.0135	-0.7758	0.7739	20
21	-0.0005	-0.0042	-0.0026	-0.7961	0.7961	21
22	-0.0005	-0.0044	0.0070	-0.7981	0.7982	22
23	-0.0005	-0.0044	0.0124	-0.7959	0.7960	23
24	0.0007	-0.0053	0.0817	-0.9807	0.9841	24
25	0.0010	-0.0041	0.1477	-1.3353	1.3434	25
26	0.0011	-0.0027	0.2624	-1.3257	1.3519	26
27	0.0009	-0.0016	0.3526	-1.0658	1.1227	27
28	0.0008	-0.0010	0.3329	-0.6137	0.6982	28
29	0.0006	-0.0006	0.2670	-0.2352	0.3558	29
30	0.0006	-0.0005	0.2534	-0.1932	0.3186	30
31	0.0005	-0.0005	0.2454	-0.1631	0.2947	31
32	0.0005	-0.0004	0.2244	-0.1041	0.2474	32
33	0.0002	-0.0001	0.1516	-0.0063	0.1517	33
34	0.0	0.0000	0.0	0.0069	0.0069	34
35	0.0	-0.0032	0.0	-0.8726	0.8726	35
36	-0.0002	-0.0034	-0.0514	-0.8067	0.8882	36
37	-0.0006	-0.0042	-0.0715	-0.9397	0.9424	37
38	-0.0007	-0.0050	-0.0669	-0.9822	0.9845	38
39	-0.0007	-0.0054	-0.0569	-0.9946	0.9962	39
40	-0.0007	-0.0055	-0.0498	-0.9998	1.0010	40
41	0.0011	-0.0068	0.2708	-1.2467	1.2757	41
42	0.0011	-0.0058	0.3597	-1.5753	1.6159	42
43	0.0009	-0.0032	0.3351	-1.5516	1.5874	43
44	0.0015	-0.0022	0.3390	-1.2694	1.3139	44
45	0.0015	-0.0014	0.3845	-0.7874	0.8763	45
46	0.0010	-0.0008	0.4413	-0.2989	0.5330	46
47	0.0009	-0.0007	0.4101	-0.1931	0.4533	47
48	0.0008	-0.0006	0.3863	-0.1643	0.4198	48
49	0.0006	-0.0004	0.3119	-0.0649	0.3185	49
50	0.0003	-0.0001	0.1760	-0.0002	0.1760	50
51	0.0	0.0000	0.0	0.0080	0.0080	51
52	0.0	-0.0030	0.0	-1.0964	1.0964	52
53	-0.0003	-0.0036	-0.1067	-1.1450	1.1500	53
54	-0.0009	-0.0049	-0.1523	-1.2469	1.2562	54
55	-0.0010	-0.0061	-0.1355	-1.3089	1.3159	55
56	-0.0010	-0.0067	-0.1209	-1.3390	1.3445	56
57	-0.0010	-0.0070	-0.1135	-1.3561	1.3609	57
58	0.0015	-0.0087	0.4476	-1.6374	1.6975	58
59	0.0036	-0.0084	0.7197	-2.0398	2.1630	59
60	0.0084	-0.0066	0.9439	-2.2202	2.4125	60
61	0.0081	-0.0022	1.0612	-1.9278	2.2006	61
62	0.0042	-0.0011	0.9413	-1.2447	1.5605	62
63	0.0017	-0.0005	0.6377	-0.3532	0.7290	63
64	0.0013	-0.0005	0.5758	-0.3002	0.6493	64
65	0.0012	-0.0004	0.5282	-0.1335	0.5448	65
66	0.0008	-0.0002	0.4085	0.0009	0.4085	66
67	0.0003	-0.0001	0.1970	0.0077	0.1972	67
68	0.0	0.0000	0.0	0.0086	0.0086	68
69	0.0	-0.0032	0.0	-0.9510	0.9510	69
70	-0.0021	-0.0037	-0.2512	-1.0714	1.1005	70
71	-0.0051	-0.0056	-0.3600	-1.3266	1.3746	71
72	-0.0057	-0.0079	-0.3026	-1.4144	1.4464	72
73	-0.0056	-0.0085	-0.2628	-1.4844	1.5075	73
74	-0.0054	-0.0090	-0.2485	-1.4820	1.5027	74
75	0.0048	-0.0120	0.5279	-1.7471	1.8251	75
76	0.0108	-0.0112	0.9133	-2.0299	2.2259	76
77	0.0144	-0.0074	1.2521	-2.1383	2.4780	77
78	0.0116	-0.0016	1.5463	-1.6026	2.2270	78
79	0.0082	-0.0003	1.2103	-1.1371	1.6607	79
80	0.0015	0.0004	2.4032	0.0089	2.4032	80
81	0.0	-0.0019	0.0	-0.1152	0.1152	81
82	-0.0071	-0.0023	-0.5909	-0.6513	0.8794	82
83	-0.0145	-0.0058	-0.8556	-1.2122	1.4837	83
84	-0.0182	-0.0113	-0.6763	-1.5456	1.6871	84
85	-0.0174	-0.0150	-0.6116	-1.5376	1.6548	85
86	-0.0170	-0.0156	-0.5802	-1.5908	1.7008	86
87	0.0139	-0.0232	0.6670	-1.7536	1.8762	87
88	0.0285	-0.0187	1.3102	-1.9800	2.3743	88
89	0.0293	-0.0067	2.0886	-1.7540	2.7274	89
90	0.0242	0.0015	2.2421	-0.9874	2.4499	90
91	0.0170	0.0029	2.2375	0.1247	2.2410	91
92	-0.0183	0.0019	-1.6530	0.1259	1.6578	92
93	-0.0284	-0.0029	-1.0474	-1.1325	1.5426	93
94	-0.0316	-0.0139	-0.9911	-1.2448	1.5911	94
95	-0.0309	-0.0202	-0.8312	-1.5132	1.7265	95
96	-0.0296	-0.0233	-0.7730	-1.4717	1.6623	96
97	0.0237	-0.0346	0.7474	-1.6899	1.8478	97
98	0.0427	-0.0249	1.5776	-1.8577	2.4372	98
99	0.0458	-0.0031	1.9283	-1.4875	2.4354	99
100	0.0312	0.0053	2.5091	0.0333	2.5094	100
101	-0.0421	0.0023	-1.0651	-0.1249	1.0724	101
102	-0.0467	-0.0150	-0.7546	-0.9042	1.1777	102
103	-0.0459	-0.0281	-0.7243	-0.8945	1.1510	103
104	-0.0434	-0.0358	-0.7005	-1.0079	1.2274	104
105	0.0338	-0.0513	0.6140	-1.1784	1.3288	105
106	0.0567	-0.0335	1.1212	-1.2935	1.7118	106
107	0.0537	0.0040	1.2825	-0.1820	1.2953	107
108	-0.0539	-0.0130	-0.3367	-0.1352	0.3628	108
109	-0.0510	-0.0379	-0.2988	-0.3420	0.4541	109
110	-0.0478	-0.0502	-0.2740	-0.3711	0.4613	110
111	0.0360	-0.0671	0.2311	-0.4715	0.5251	111
112	0.0662	-0.0474	0.4248	-0.3977	0.5819	112
113	0.0633	-0.0182	0.4236	-0.1849	0.4622	113
114	-0.0671	-0.0415	-0.0671	-0.0415	0.0789	114
115	-0.0615	-0.0581	-0.0615	-0.0581	0.0846	115
116	-0.0425	-0.0596	-0.0425	-0.0596	0.0732	116
117	0.0290	-0.0769	0.0290	-0.0769	0.0821	117
118	0.0805	-0.0589	0.0805	-0.0589	0.0998	118
119	0.0821	-0.0476	0.0821	-0.0476	0.0949	119
120	0.0	0.0	0.0	0.0	0.0	120
121	0.0	0.0	0.0	0.0	0.0	121
122	0.0	0.0	0.0	0.0	0.0	122
123	0.0	0.0	0.0	0.0	0.0	123