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POSTMORTEM INVESTIGATION OF THE KANSAS TEST TRACK

NOVEMBER 1979

FINAL REPORT

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16. Abstract

The Kansas Test Track (KTT) was comprised of nine different track systems, which were built on a designed embankment. The KTT subgrade experienced premature failure in service, and KTT operations were terminated in June 1975 after six months of operation.

A postmortem investigation of KTT was carried out in 1975 and 1976. Volumes I and II document the structure, ballast, and embankment studies carried out by the U. S. Army Engineer Waterways Experiment Station in the postmortem investigation. Results presented in Volume I provide needed insight into track system behavior and are particularly descriptive of ballast and embankment performance. A rationale is presented to explain the mechanism of failure in the subgrade, and attention is drawn to the structure-ballast-subgrade interaction processes which govern track system performance. Inadequate drainage of the top of the KTT embankment led to moisture content increases in the top of subgrade from 1971 to 1974, when traffic commenced. The embankment clay was moderately to highly plastic, and moisture-induced loss of strength in the top few inches of subgrade was the principal cause of failure.

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PREFACE

This study was authorized by the Department of Transportation (DOT) Federal Railroad Administration (FRA) under Interagency Agreement AR 30025, dated 12 December 1972, and Amendments 3-8 thereto. The work was performed by personnel of the U. S. Army Engineer Waterways Experiment Station (WES), P. O. Box 631, Vicksburg, Mississippi 39180. Dr. R. M. McCafferty of the Office of Rail Safety Research, FRA, monitored the project.

Mr. A. J. Bush III, Pavement Evaluation Branch, Geotechnical Laboratory (GL), carried out the field and laboratory trenching study. Mr. H. C. Greer, Instrumentation Services Division, was responsible for the instrumentation investigation, and Mr. M. A. Vispi, Explorations Branch, GL, accomplished the penetrometer testing. Mr. M. M. Carlson was responsible for vibroseismic testing. Mr. S. S. Cooper, Geodynamics Branch (GDB), Earthquake Engineering and Vibrations Division (EE&VD), directed the investigation and prepared this report under the supervision of Mr. R. F. Ballard, Chief, GDB, and Messrs. P. F. Hadala and F. G. McLean, Chief and former Chief, respectively, of EE&VD. The work was performed under the general supervision of Mr. J. P. Sale, Chief, GL.

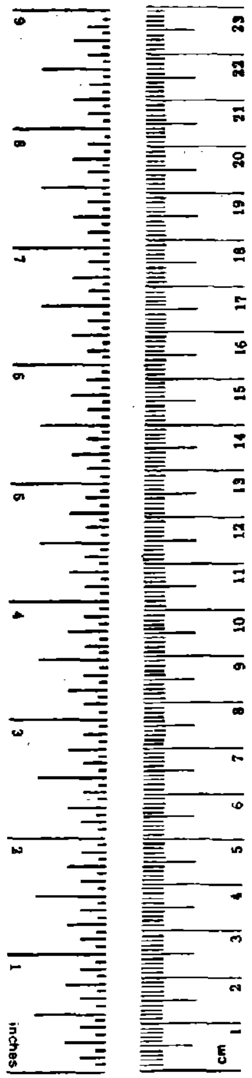
COL G. H. Hilt, CE, and COL J. L. Cannon, CE, were Directors of the WES during this study. Mr. F. R. Brown was Technical Director.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

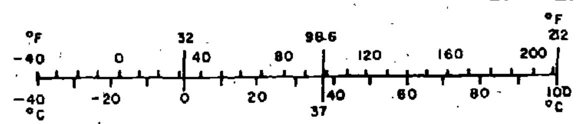
Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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POSTMORTEM INVESTIGATION OF THE KANSAS TEST TRACK, VOLUME I

INTRODUCTION

Background

The Department of Transportation (DOT) Federal Railroad Administration (FRA) is actively engaged in railway research programs whose primary objectives are (1) a reduction in frequency of track-caused derailments and (2) the development of improved track designs and maintenance procedures. As a part of this research effort, the 8000-ft-long Kansas Test Track (KTT) was constructed near Aikman, Kansas, under joint sponsorship of the FRA[1,2,3] and the Atchison, Topeka and Santa Fe (ATSF) railroad. The KTT was located adjacent to an existing ATSF main-line track, as shown in figure 1, so that the relatively heavy ATSF freight traffic could be used as the test loadings for KTT.

The KTT was conceived as a means of evaluating the effects of stiffness variations on track system stability, performance, and maintenance requirements under actual traffic conditions. It was built using nine track support systems, which were designed to provide various degrees of vertical stiffness.[4] The track systems were founded on a specially designed embankment[5] which, to conform with usual railroad practice, was built of locally available materials.[6] Expectations were that the embankment and structures would survive for at least the three-year period required for testing. The desired differences in track support system stiffness were achieved by using either conventional (tie) or nonconventional (beam or slab) construction and by varying other properties such as tie spacing and ballast thickness.[4] A summary of the test sections and their design variables is shown in Table 1. Since the clay embankment was not intended to be a test variable, it was constructed under rigid controls to achieve the maximum practical degree of subgrade uniformity.[6] Instrumentation arrays were built into each

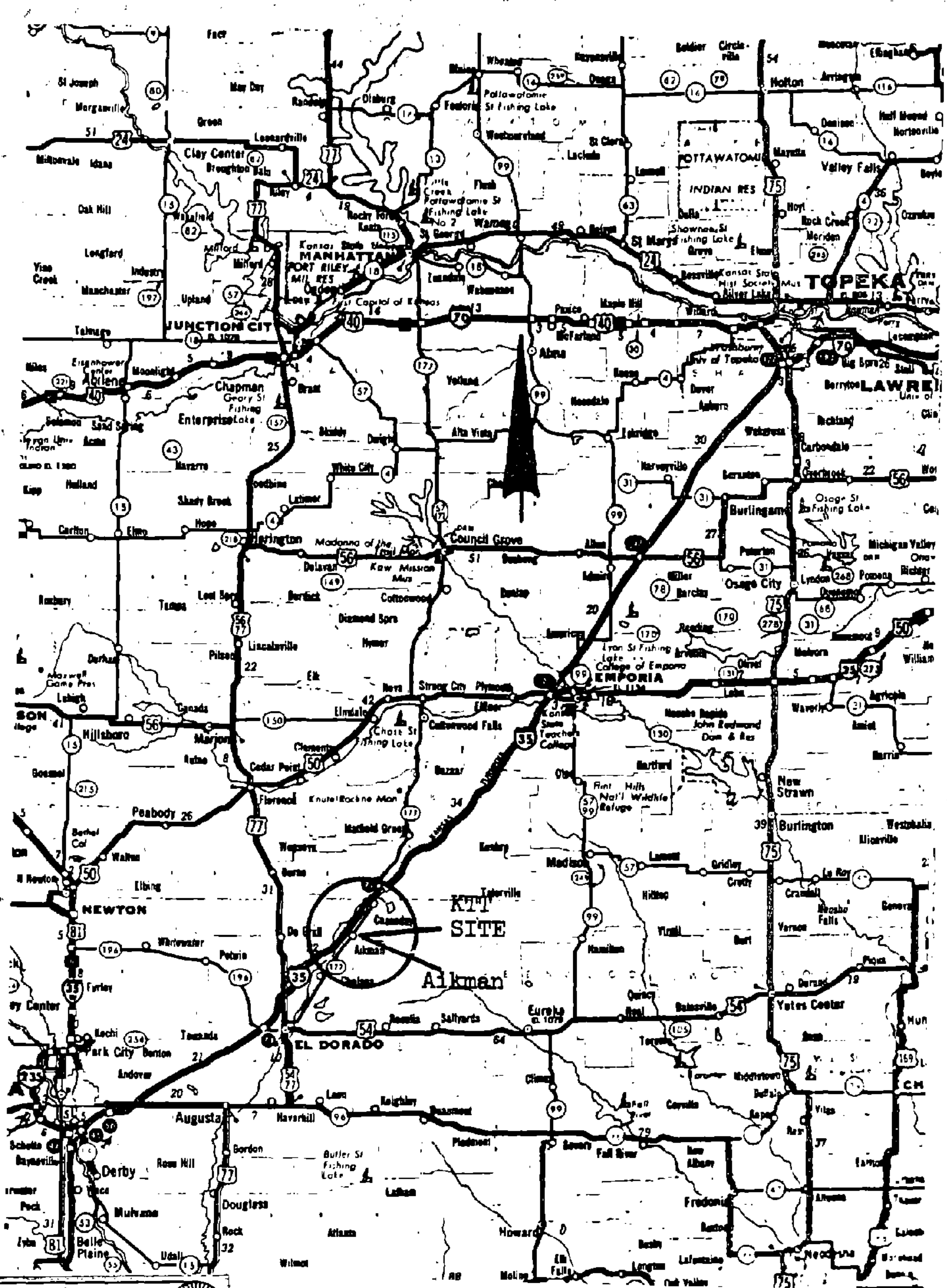


FIGURE 1. LOCATION OF KTT TEST SITE

TABLE 1. KTT TRACK STRUCTURE DESIGN VARIABLES

Section Number	System Type	Ballast Depth	Remarks
1	Concrete ties, 30-in. C-C	10 in.*	
2	Concrete ties, 27-in. C-C	10 in.	
3	Concrete ties, 24-in. C-C	10 in.	
4	Continuous concrete beams	6 in.	Cast-in-place structure
5	Continuous concrete slab	6 in.	Cast-in-place structure
6	Wood ties, 19.5-in. C-C	10 in.	6-in. stabilized ballast layer on subgrade
7	Continuous concrete beams	6 in.	Precast beams, installed and field joined
8	Concrete ties, 27-in. C-C	15 in.	
9	Wood ties, 19.5-in. C-C	10 in.	Control section (standard Santa Fe)

* A table of factors for converting units of measurement is presented on page ii.

section of the embankment to measure static and dynamic response of the structures and embankment during the life of the project. Upon completion of construction work on the KTT embankment in 1971, the U. S. Army Engineer Waterways Experiment Station (WES) conducted a vibroseismic study to determine in situ elastic properties at selected locations.[7]

By 1972 the track structures had also been installed, and the WES was asked to determine the dynamic properties of each KTT track system using mechanical impedance techniques.[8] This investigation was to be a feasibility study; however, if meaningful results were to be obtained in the initial study, then periodic (performance) retesting also needed to be done. Measurements of track system dynamic stiffness were the prime objective of the study, but other data were also collected.[8]

On 2 May 1973, slow-order freight traffic from the ATSF mainline was diverted onto the KTT. After a few hours of operation, the KTT had to be closed for repairs to the rail fasteners of the nonconventional track structures. The problem was caused by pullout of the rail fastener anchor bolts, which were then redesigned by the Portland Cement Association (PCA). The modified anchor bolts proved to have excellent pullout resistance; however, replacing all of the defective fastener bolts delayed reopening of the KTT until 31 October 1974. After 14 days of slow speed traffic, the KTT was closed as planned for final repairs and adjustments. On 10 December 1974, the KTT was finally open to unrestricted mainline ATSF freight traffic. Under the effects of high-volume traffic, KTT began to show signs of distress after only 2-1/2 months of operation.

Upper limit (FRA Class 5) profile and gage irregularities, as measured by a track survey device, occurring in the nonconventional structures (twin beams in Track Sections 4 and 7) were the first evidence of more serious problems to follow. Cracking of the beams at vertical construction joints and spalling at the joint cracks also occurred. While these conditions were not cause for immediate concern, the appearance of mud pumping through the ballast certainly was. Examples of spalling and pumping in the nonconventional structures are shown in Figure 2. During the spring thaw and rains of 1975, pumping

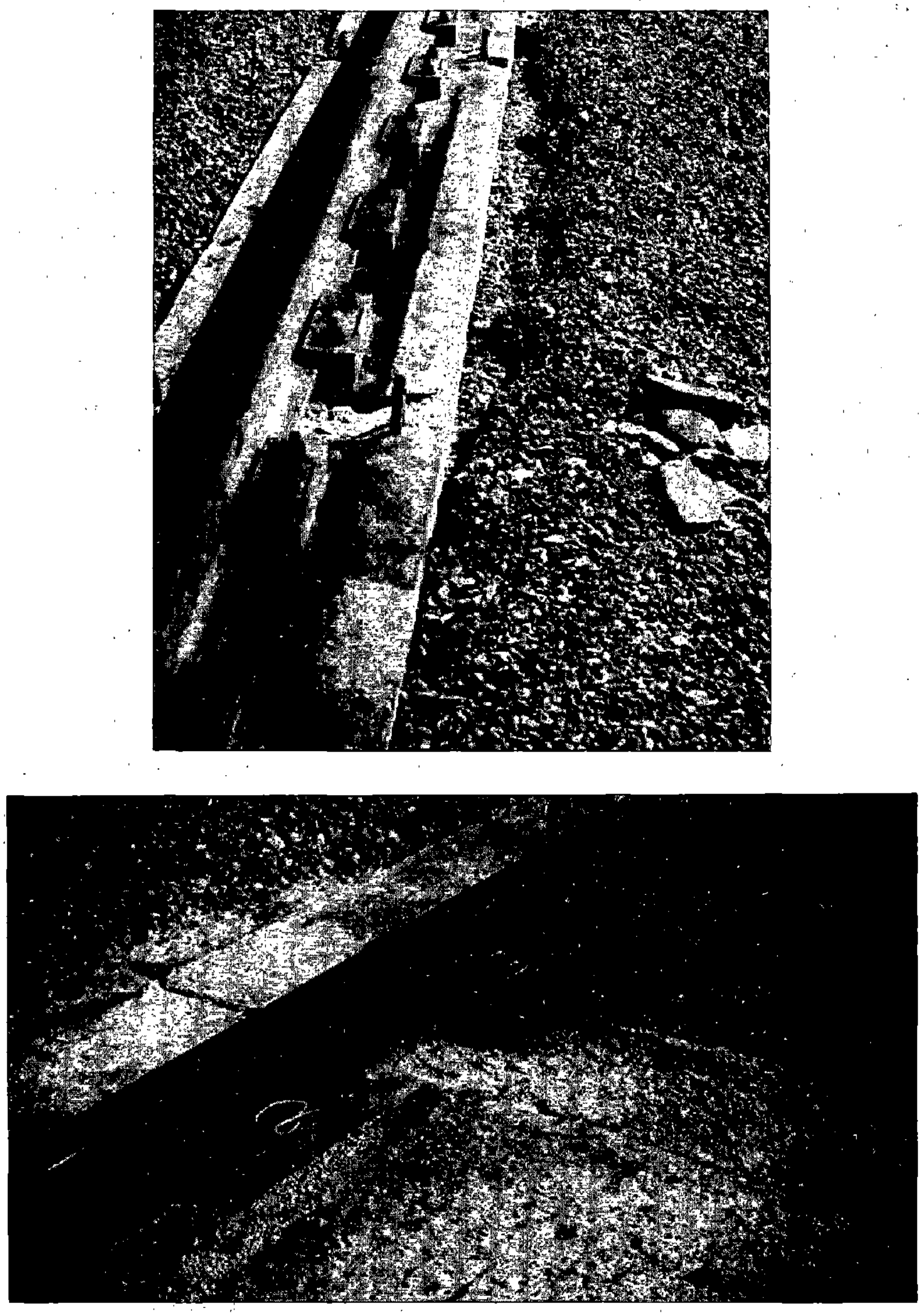


FIGURE 2. EXAMPLES OF PUMPING AND SPALLING AT CONTROL POINTS OF THE NONCONVENTIONAL TRACK STRUCTURES

conditions were in evidence at many locations, being particularly severe in Track Sections 1, 4, 6, and 7 after heavy downpours. Typical conditions in the conventional track structures are illustrated in Figure 3. Two kinks in the KTT track occurred on 6 June 1975; these kinks were located in Track Sections 1 and 2, respectively, and were attributed to loss of lateral tie support in the mud-fouled ballast. These and other operational problems prompted FRA to close the KTT permanently on 11 June 1975.

In subsequent meetings between DOT and WES representatives, the WES was asked to develop plans for a detailed postmortem investigation of the KTT embankment to include the acquisition of structural response data which would be used by other investigators. The postmortem test program proposed by WES was adopted by FRA and has since been carried out under authority provided in Interagency Agreement AR 30025 and Amendments 3-8 thereto. This report plus Reference 8 completes all the work scheduled under agreement.

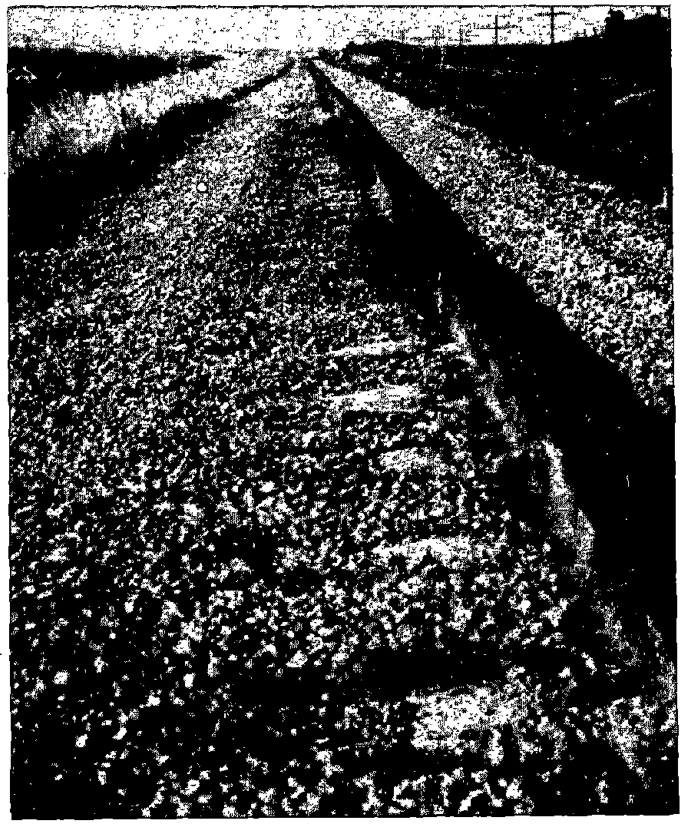
Purpose

Basic objectives of the KTT postmortem investigation carried out by the WES were as follows:

1. To determine the mechanism(s) of failure in the embankment.
2. To determine the condition of built-in instrumentation and, if possible, the validity of data obtained from same.
3. To acquire the static and dynamic load-deflection data needed to tune existing analytical models developed by others and to acquire posttraffic impedance test results for comparison purposes.
4. To recommend design improvements for future test facilities.

Scope

This report documents the field and laboratory investigations conducted by the WES during the FRA KTT postmortem program. Included are results of material property tests on the ballast, results of material property and strength tests on the embankment clays, results of in situ geophysical investigations of the embankment, and other pertinent data



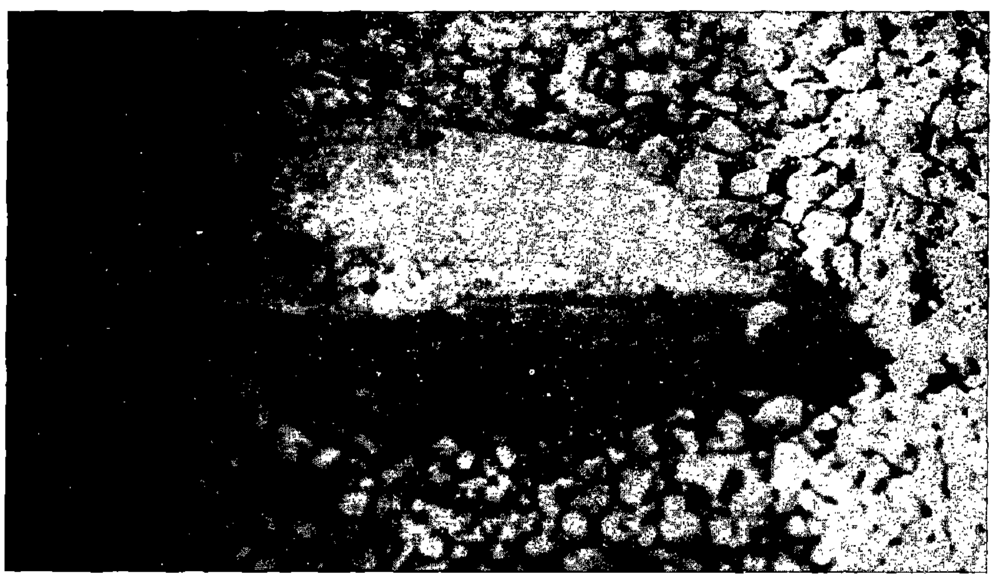


FIGURE 3. TYPICAL EXAMPLES OF PUMPING IN THE CONVENTIONAL TRACK STRUCTURES

derived from measurements and inspections of the site, structures, and test trenches. In addition, results of an investigation of the built-in embankment instrumentation are presented. Results of the various investigations are analyzed and correlated, and a rationale is developed which explains the behavior of the KTT embankment. Conclusions are drawn regarding the sources of KTT subgrade problems and recommended preventive and/or remedial treatments are advanced. Recommendations for future planning of similar test facilities are also offered.

Presentation

In consideration of the quantities of data and information generated in this study, results are presented in two volumes. Volume I summarizes the data obtained in the various KTT postmortem investigations and presents analyses, conclusions, and recommendations derived from same. Volume II describes the test procedures and equipment used in the investigation and also documents the original data. Hopefully, material contained in Volumes I and II will provide better insight into track system behavior under adverse conditions and also should promote a better understanding of the structure-ballast-interaction processes which govern performance.

SITE DESCRIPTION

The KTT paralleled an existing Santa Fe mainline track at a distance of 30 ft. Maximum relief in this area is approximately 30 ft, and the terrain is characterized by gently rolling, grassy fields with occasional limestone outcroppings. Drainage is to the southwest, and the KTT track runs northeast to southwest with a 0.4 percent slope towards the latter direction. The average site elevation is approximately 1400 ft above mean sea level. Subsurface materials in the site vicinity typically consist of 1 to 5 ft of residual clay overlying the parent limestone bedrock.

The KTT embankment consisted of approximately 6 ft of residual, reddish-brown, plastic clay (CH) founded on limestone that had previously been excavated to design depth. Instrumentation arrays were installed at preselected locations along the test track.[6] Locations of the various track support systems and built-in instrumentation arrays are shown in Figure 4. A typical cross section through the KTT embankment is shown in Figure 5. Construction details of the embankment and built-in instrumentation are given in Reference 6.

DOT-KANSAS TEST TRACK
(Paralleled an existing ATSF mainline track 30' away)

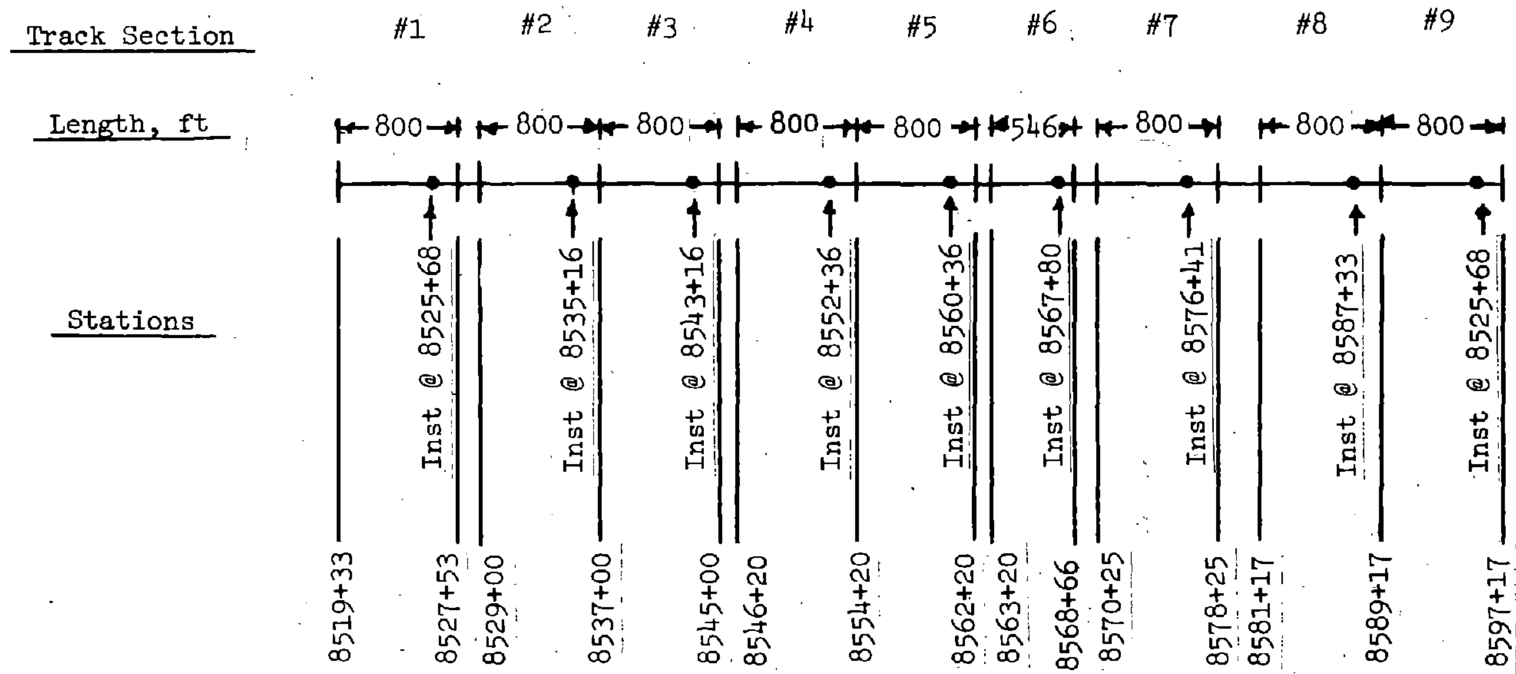


FIGURE 4. SCHEMATIC SHOWING THE KTT LAYOUT AND LOCATION OF MAIN INSTRUMENTATION ARRAYS

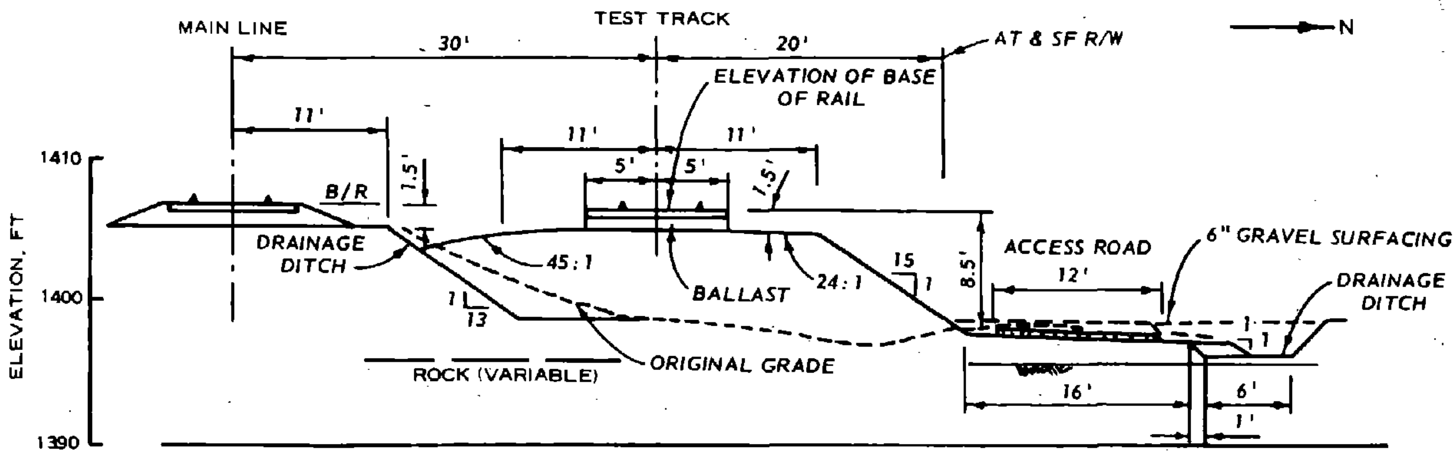


FIGURE 5. TYPICAL CROSS SECTION THROUGH KTT EMBANKMENT

DEVELOPMENT OF THE POSTMORTEM TEST PLAN

Planning for the WES participation in the KTT postmortem investigation began in August 1975, two months after final shutdown of the KTT. Early in the planning phase, input was solicited and received from many other investigators with an interest in the KTT program. Concurrently, the WES conducted a review of the KTT embankment construction history. With this background, final planning was accomplished by FRA and WES representatives. Representatives of MITRE Corporation, a DOT contractor organization, were also present, since the FRA wanted them to plan special structures tests that would be carried out by the WES during the postmortem investigation. Major objectives were established by the FRA; and by late August 1975, the WES technical approach was developed in detail. By this time ATSF maintenance records for the KTT had been reviewed in detail, and a graphical performance summary had been developed for Track Sections 1 through 5 and 7 through 9, as shown in Figures 6 through 13, respectively. For reasons given later in this report Track Section 6 was not tested. A WES reconnaissance trip to the KTT to liaise with ATSF representatives followed in mid-September 1975. In view of the extensive scope of work and the adverse effect of winter weather on field operations, WES and FRA representatives later agreed that the postmortem work could best be carried out in two phases. Phase I work, which would consist of load-deflection and impedance studies of track structure response, would be attempted in the period October and November 1975. Phase II work, consisting primarily of embankment investigations, could only be carried out after the track structures had been removed. Hence, the Phase II work was tentatively scheduled for April and May 1976, by which time the structures were to have been removed from the embankment. Other considerations which affected the development of a detailed test plan are discussed in the following sections.

Structures Test

Structures testing in Phase I of the postmortem investigation

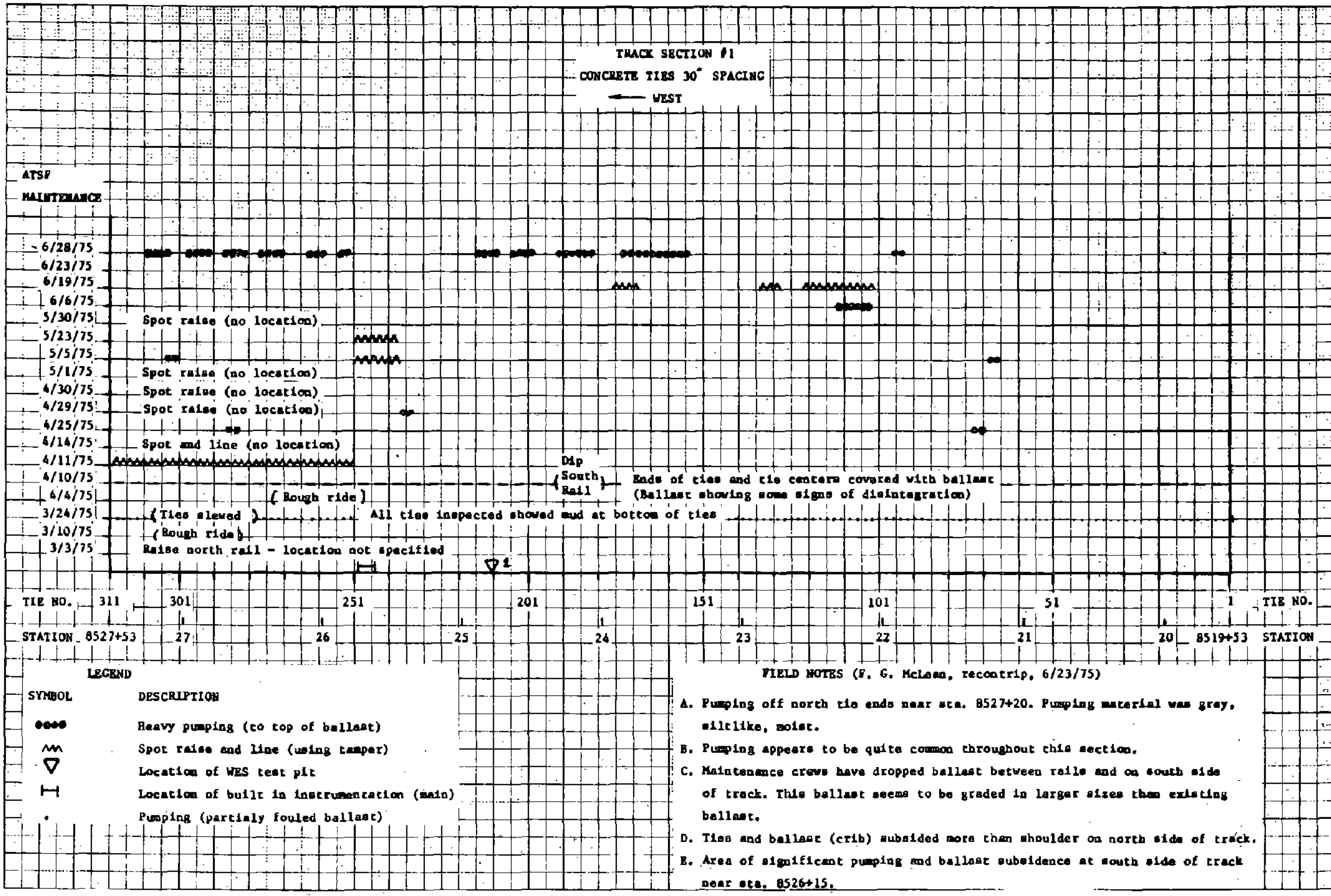


FIGURE 6. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 1

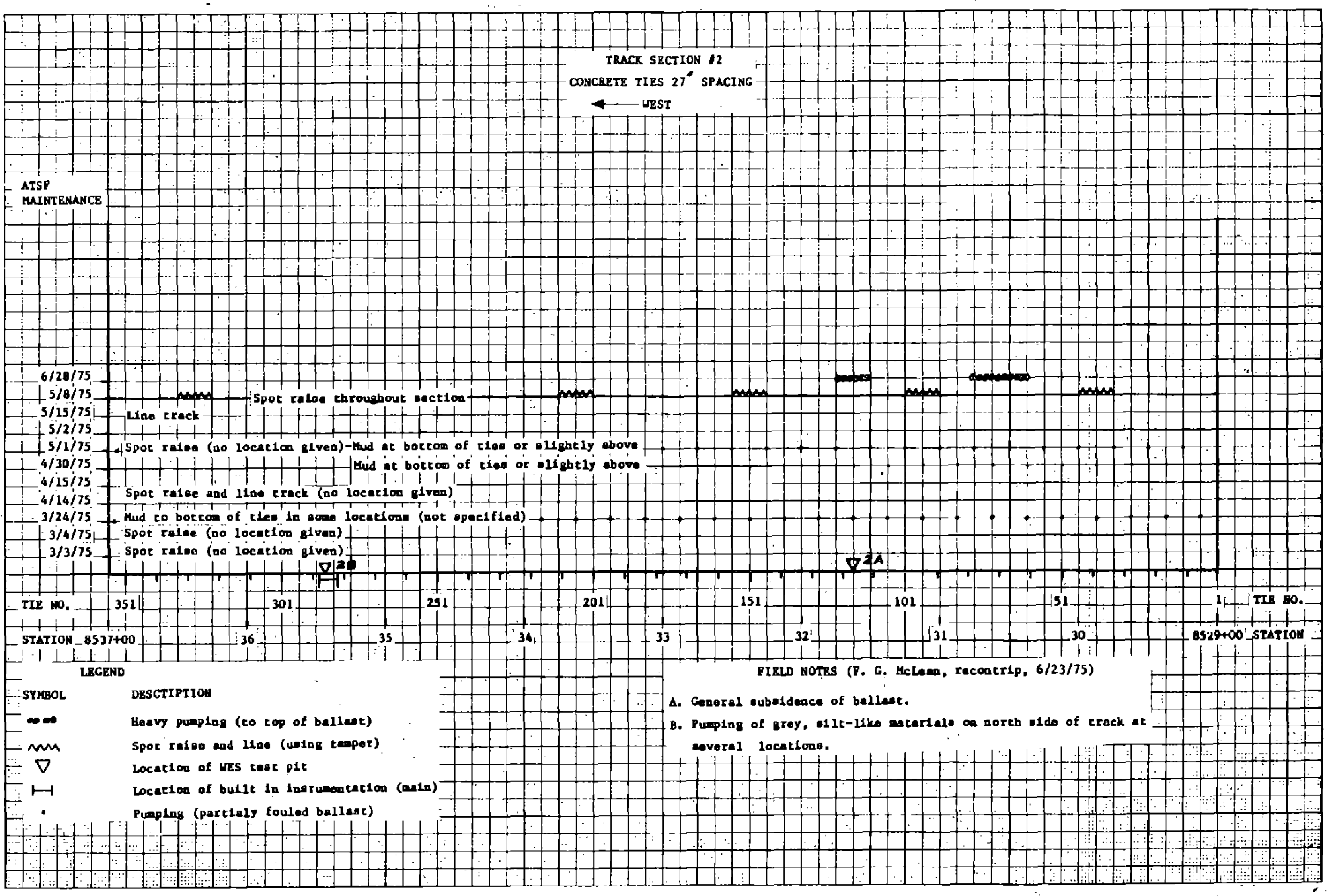


FIGURE 7. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 2

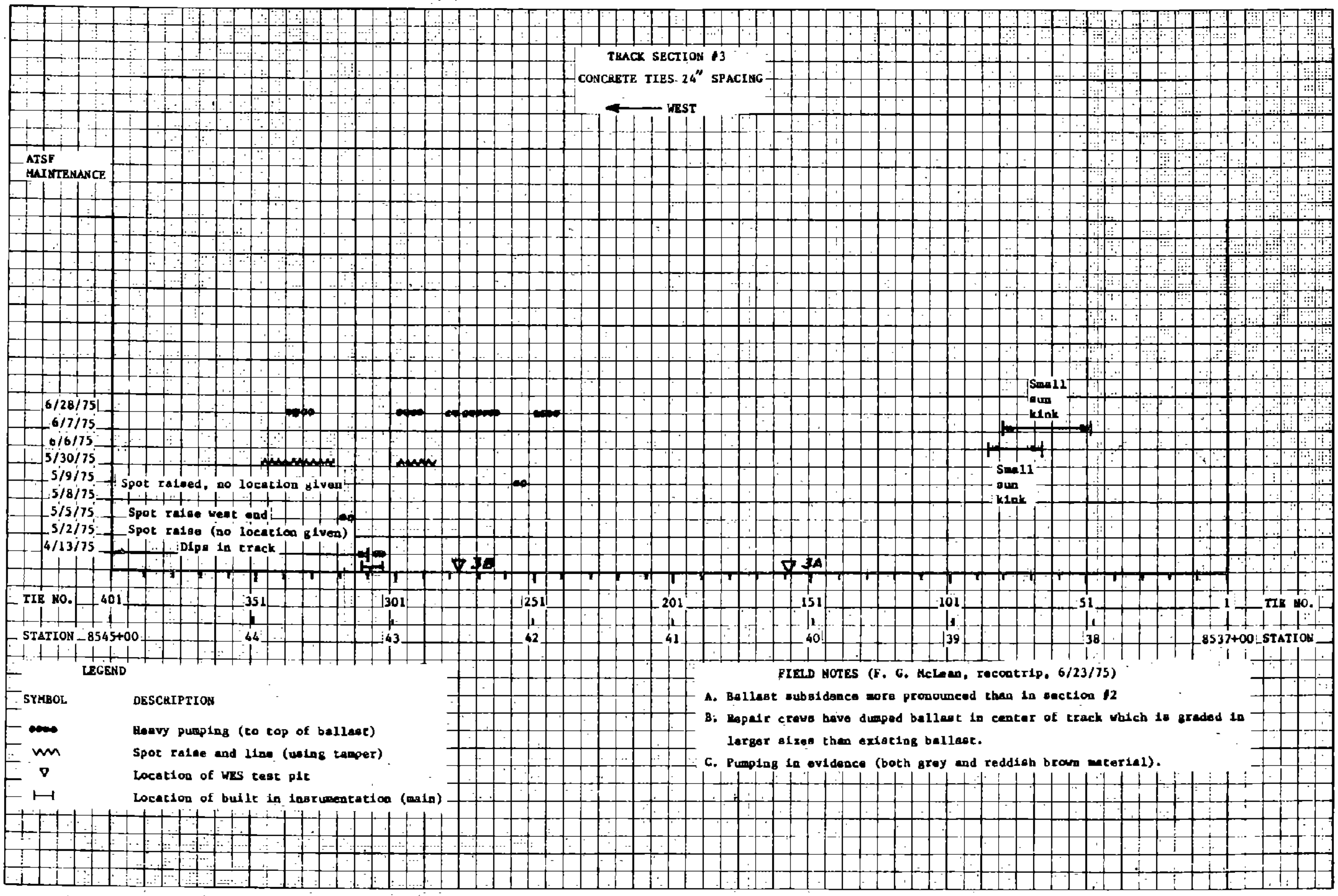


FIGURE 8. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 3

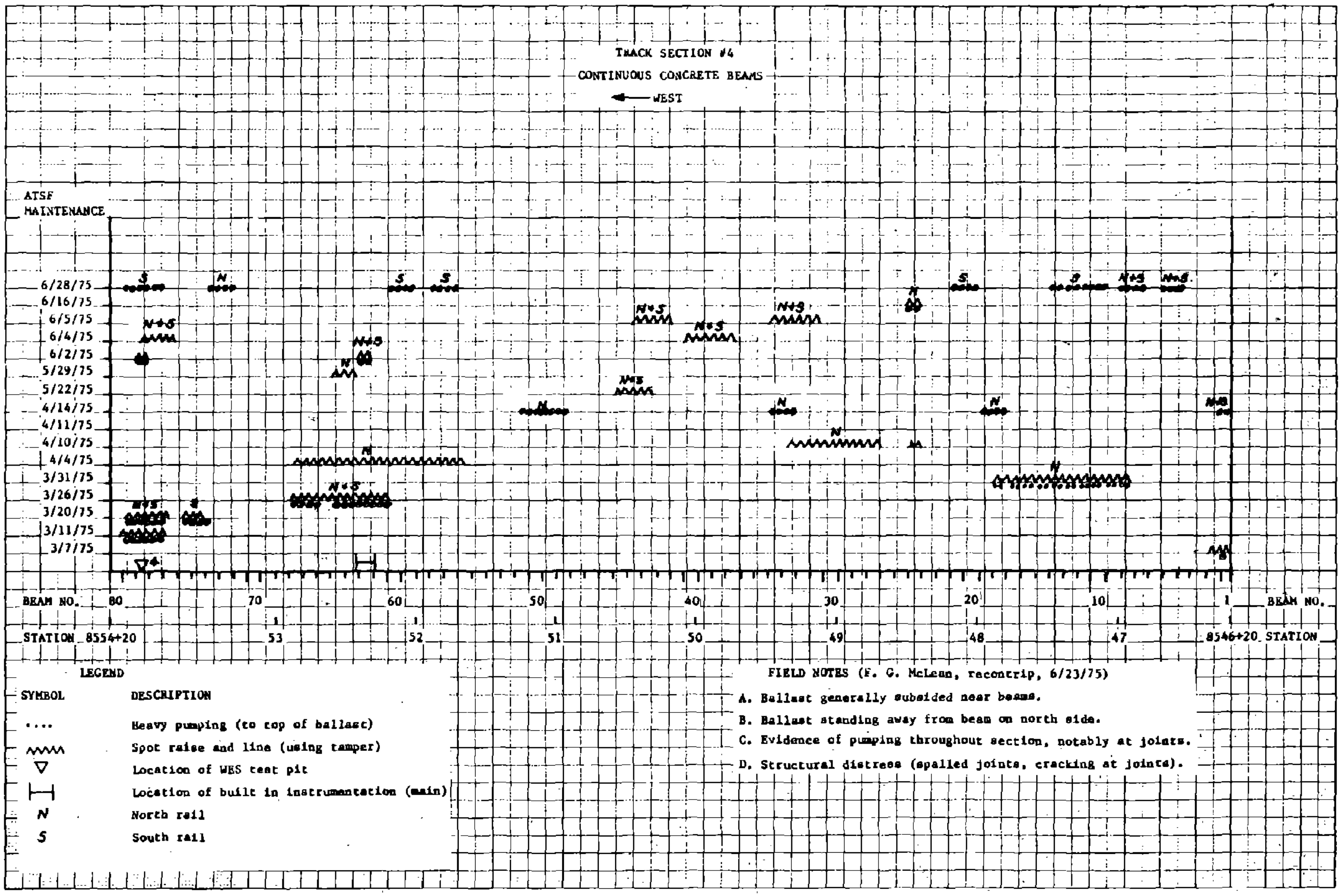


FIGURE 9. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 4

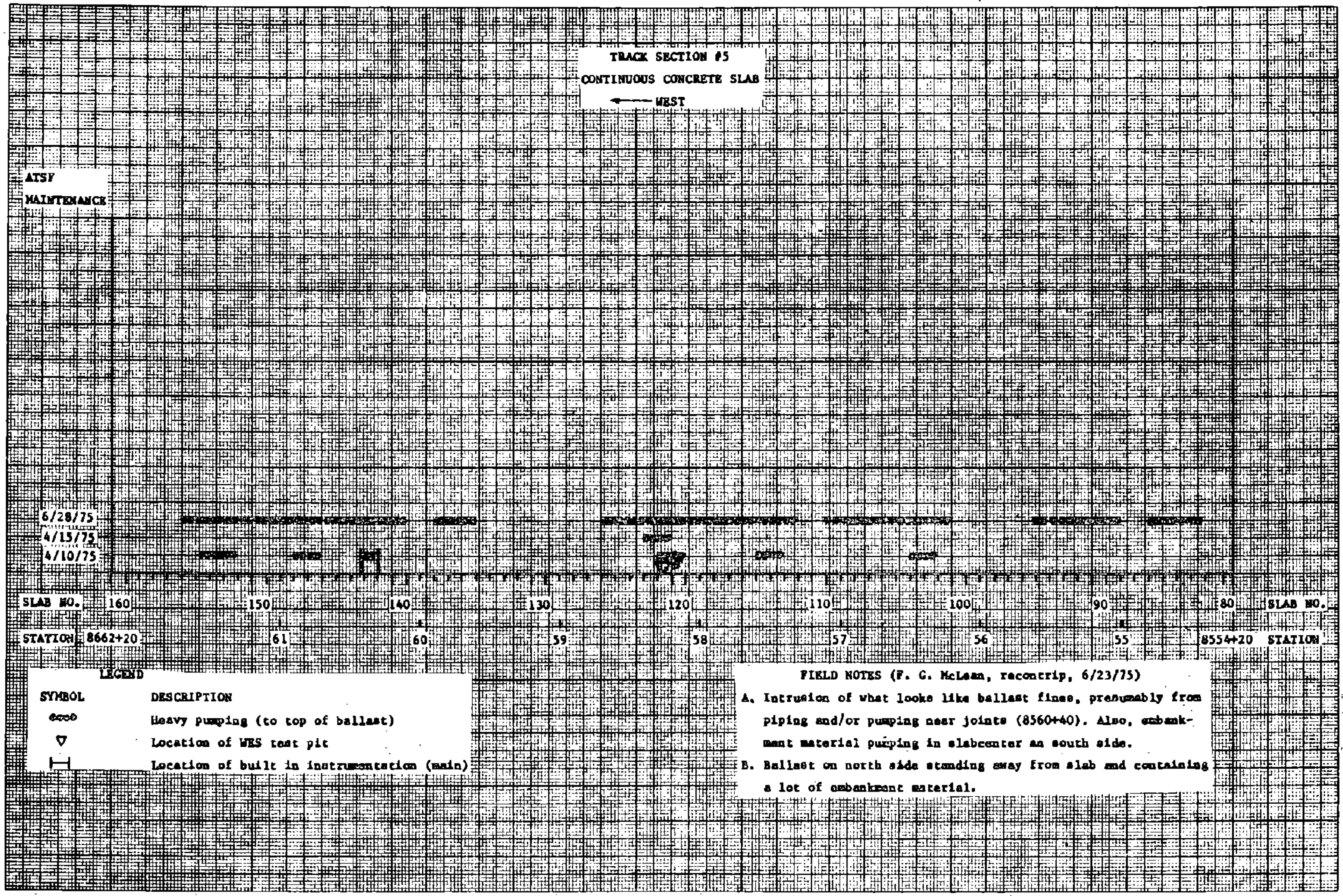


FIGURE 10. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 5

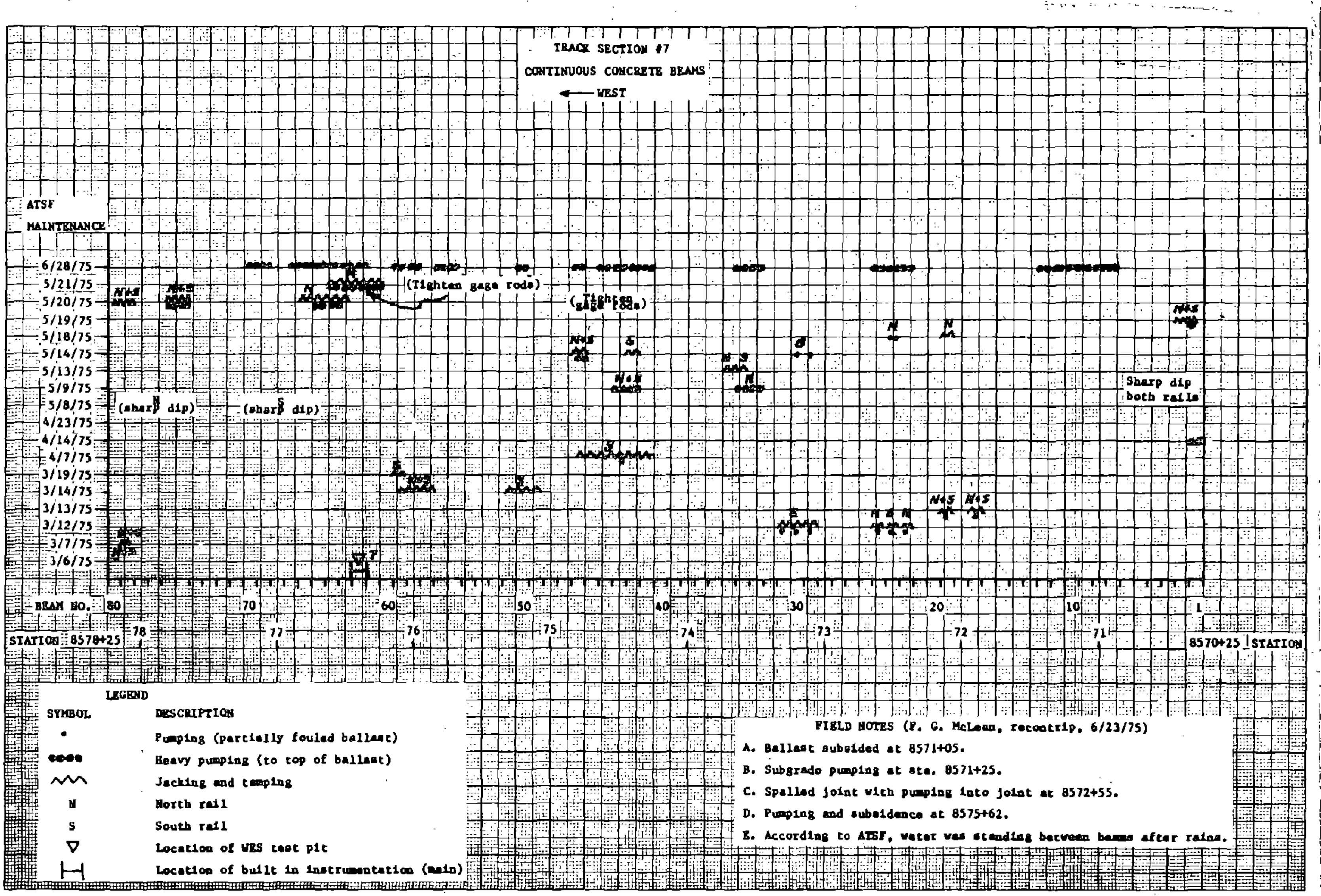


FIGURE 11. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 7

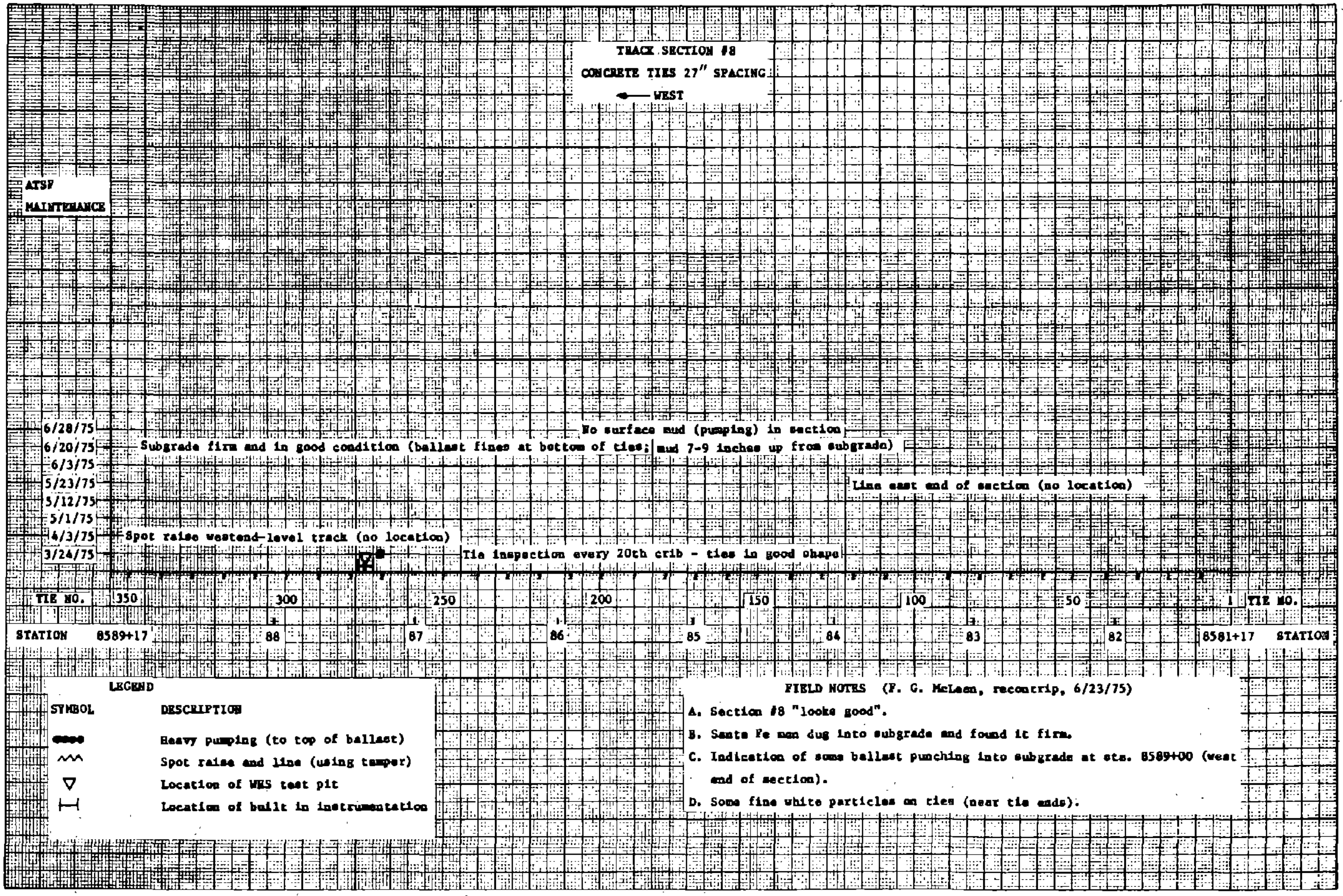


FIGURE 12. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 8

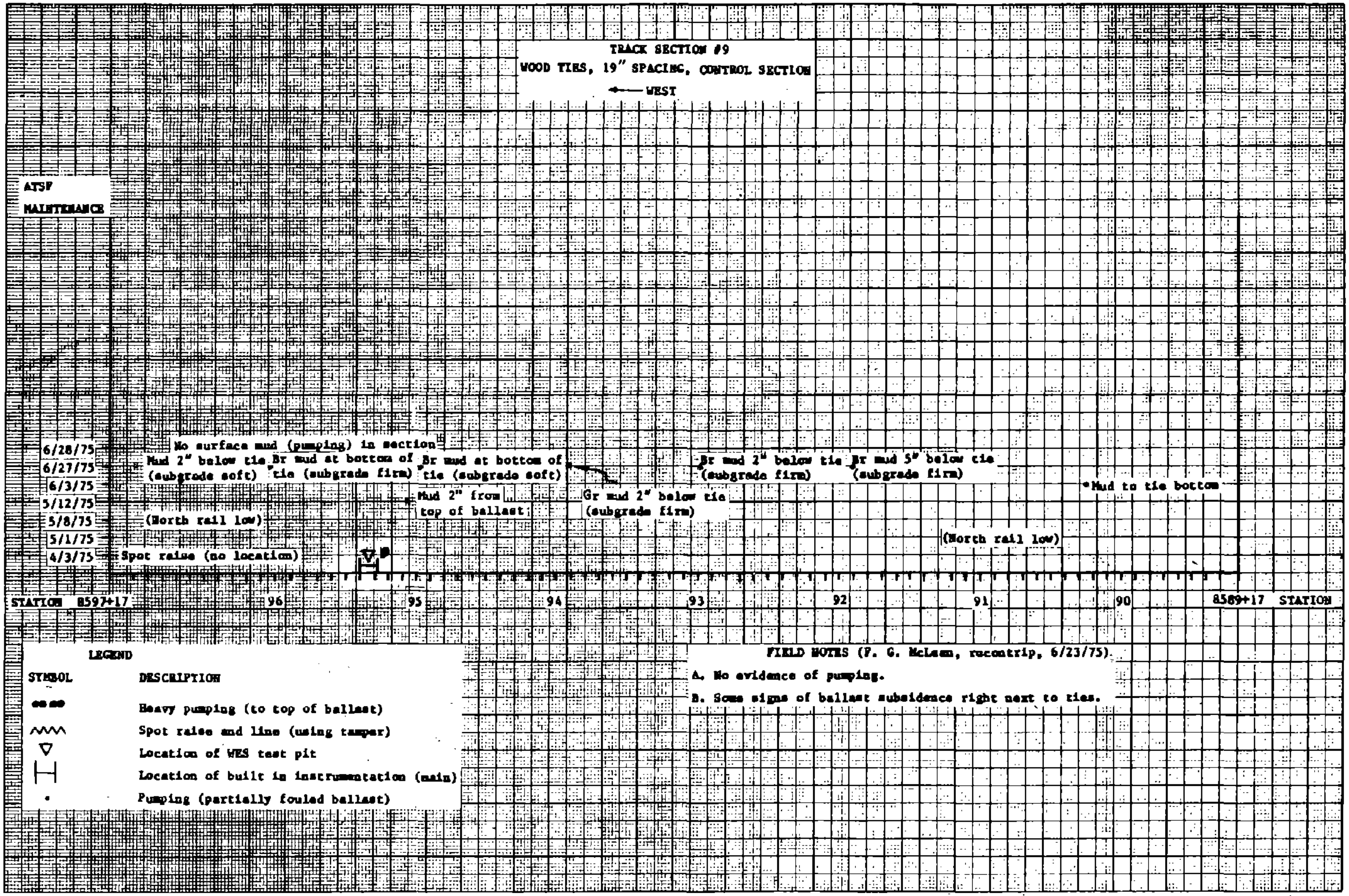


FIGURE 13. SUMMARY OF ATSF MAINTENANCE, TRACK SECTION 9

would take place in three steps, as follows:

1. Installation of vertical deflection sensors by the WES in Track Section 4 followed by dynamic testing under captive train loadings, per MITRE specifications.
2. Static load-deflection tests for MITRE at the same location.
3. Posttraffic impedance tests of selected track sections for comparison with earlier (pretraffic) WES impedance results.

Items 1 and 2 above were to provide data needed to refine a finite element analytical track model developed by the MITRE Corporation. Item 3, posttraffic impedance testing, was to provide a basis for assessing traffic-induced changes in track system stiffness through comparisons with pretraffic results. In accordance with FRA wishes, posttraffic impedance tests were performed only in Track Sections 1, 2, 3, 4, 5, 8, and 9. The inferior performance of Track Section 6 was attributed to the ballast stabilization (elastomeric polymer) treatment used. The FRA did not desire further study of this stabilization technique. Accordingly, Track Section 6 was not tested. Track Sections 4 and 7 were physically alike and had shown similar responses in pretraffic tests, so retesting of Track Section 4 was deemed to be sufficient for postmortem purposes.

The FRA also desired that posttraffic impedance tests be carried out under conditions similar to those existing at the time of track failure, i.e. subgrade pumping at the test locations.[8] In the interval (April-October 1975) between track closure and posttraffic impedance testing, the subgrade was expected to dry to some degree under the effects of the summer and fall weather. Consequently, pumping conditions had to be re-created by some expedient means. The method adopted included multiple applications of weather to the preselected test locations and repeated passes of a captive train to stimulate pumping of the subgrade. The captive train, consisting of an engine, two hopper cars, and a caboose, was provided by ATSF. This train was also used for the MITRE dynamic structural response tests conducted in Track Section 4.

Ballast and Embankment

The embankment and ballast studies were to be conducted during Phase II of the field work, which was tentatively scheduled for April-May 1976. The principal factors which influenced this timing have already been discussed; however, it was also reasoned that testing the embankment materials during the spring of 1976 (when surface water conditions should be similar to those existing at the time of track closure in April 1975) would constitute an added advantage. Implicit to this reasoning was the assumption that softening of the embankment surface, if any, was at least partly related to the presence and cumulative absorption of surface water. One such instance of surface softening from water absorption had occurred during construction of the embankment.[6] For this reason, an effort was made to protect the top surface of the completed embankment by adding a 6-in.-thick, 3 percent lime-stabilized surface soil layer and a thin (sprayed) asphaltic membrane.[6] These measures were taken in 1971 during final construction of the embankment.

Verbal reports and field observations had indicated that KTT subgrade problems were confined to the upper region of the embankment, hence this was a logical focal point for the planned embankment soils investigation. The WES and FRA agreed that the investigation should address the following major areas of concern:

1. Existing structure-ballast-subgrade conditions at locations of interest, with particular emphasis in the region of the ballast-subgrade interface.
2. Variations in embankment material properties and strength as a function of location and depth in the embankment.
3. Uniformity of embankment properties throughout its length.
4. Differential settlements and related phenomena resulting from differences in track structure geometry and load intensity.
5. Other pertinent matters, including primarily the site drainage conditions.

Items 1, 2, and 4 above could best be dealt with by staged trenching to a depth of 3 ft into the embankment, but time and money constraints would necessarily limit the physical size and number of test trenches. Other more cost-effective methods would be needed to provide an index of embankment uniformity (in terms of strength versus depth) in areas outside the test pits. The Dutch cone penetrometer and in situ vibroseismic methods were well suited to the purpose and were selected to satisfy item 3 above. Vibroseismic results would also be compared with earlier (pretraffic) results to determine traffic-related changes in embankment elastic moduli. Item 5 above would be accomplished using photographs and physical (i.e. survey) measurements as needed.

As finalized, the ballast investigation was to consist of material property tests on bag samples obtained beneath the structures and at the ballast shoulder in each test location. Gradation curves derived from these data would, it was hoped, indicate the severity of subgrade pumping from variations in the amount of fines present. Classification tests on the fines passing a No. 200 sieve would identify the source of the pumped materials. Gradation comparisons between shoulder and understructure ballast samples would also serve as indexes of any ballast deterioration under traffic loadings. Since part of the original structures design criteria were based on results of 30-in.-diam plate bearing tests on the embankment, plate bearing tests were also to be run as part of the ballast-embankment investigation.

Each embankment trench or test pit would typically consist of a 3- to 4-ft-wide staged excavation that would span the structure and one shoulder of the embankment. In an effort to identify zones of either "good" or "poor" performance, locations of the various test pits would be determined from Santa Fe track maintenance records (summarized in Figures 6-13). Zones that pumped and/or required relatively frequent maintenance were termed to have poor performance, the opposite was termed to be good performance. First, ballast samples would be taken; then the ballast would be carefully removed by hand to expose the ballast subgrade interface (the 0 level of the test pit). Depth increments for data acquisition in the test pits would be at 0, 6, 12, 24, and 36 in. below the ballast-subgrade interface. California bearing ratio (CBR) and conventional field density and moisture content tests would be performed at each stage of the excavation. Two undisturbed box samples would also be obtained from each pit for later laboratory testing. Locations for data acquisition in the pits were chosen to typify conditions at, for example, heavily stressed (beneath rail seat of tie), moderately stressed (center of tie), and relatively unstressed (center of crib) areas of the subgrade. Conditions in each test pit would be also documented with color photographs.

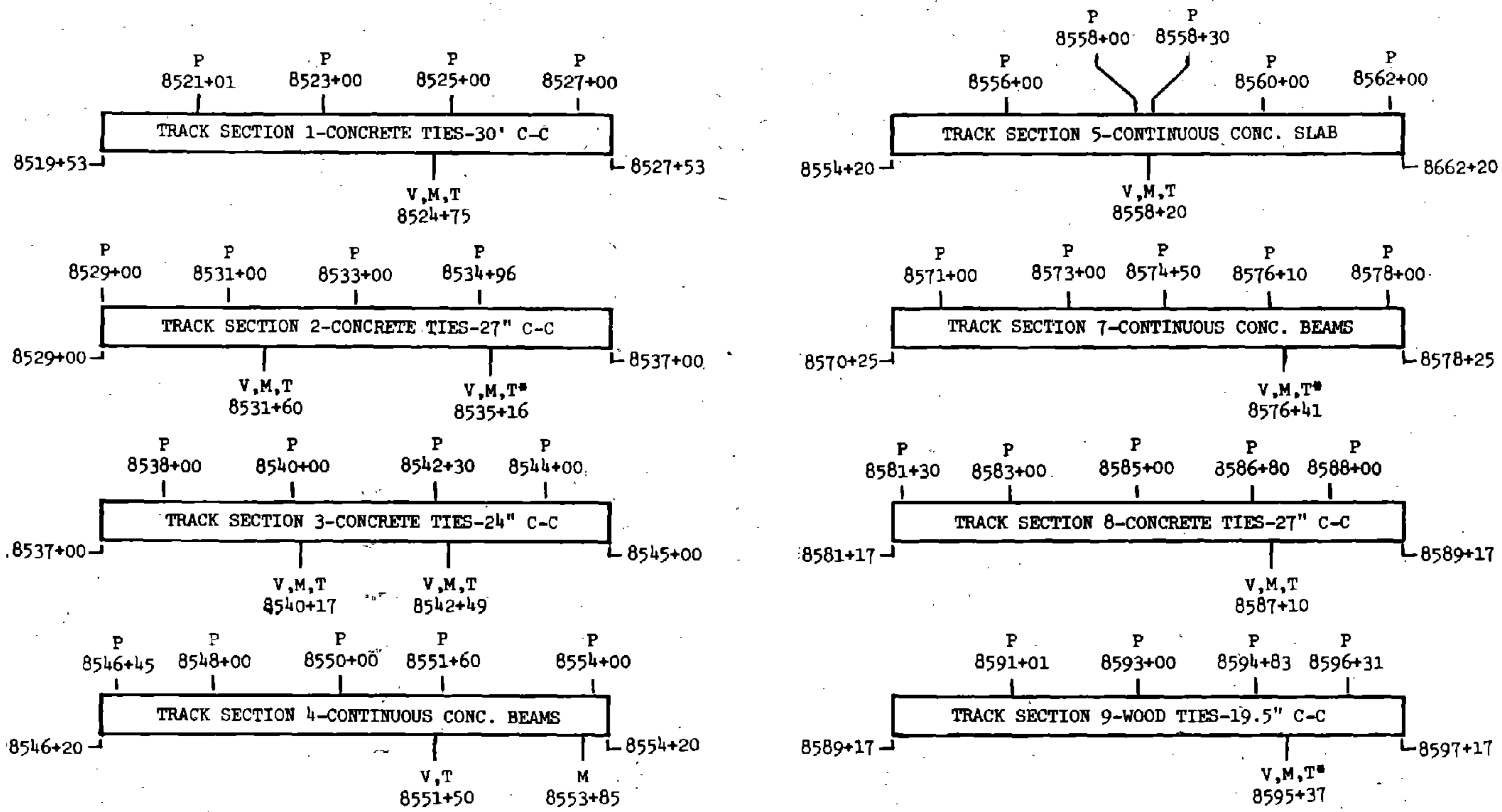
Built-In Instrumentation

The embankment was built by Shannon and Wilson[6] (S&W), who also installed the built-in embankment instrumentation. The PCA installed top of subgrade pressure cells, instrumented the track structures, and was responsible for all phases of data acquisition during testing (including the S&W instrumentation).

There had been three major data acquisition periods in the brief life of KTT; but part of these data, notably moisture measurements and some of the final readings of the vertical extensometers, were thought to be unreliable.[9] The FRA desired an evaluation of the embankment instrumentation to include its design, function, calibration, and condition. This work was to be carried out in Phase II of the field investigations. The WES would base its instrumentation evaluation on sample calibration readings from selected arrays, excavation of selected instruments, and a series of recalibrations performed on the excavated instruments. Pressure cells, vertical extensometers, and moisture and temperature cells from the main instrumentation arrays[6] would be studied in this way. In addition, most of the Carlson pressure cells, installed by PCA just beneath the ballast-subgrade interface, would be exposed for visual inspection. Finally, both data acquisition procedures and signal conditioning equipment would be reviewed. Results would be used to assess the validity of data obtained.

The Test Plan and Schedule

The KTT postmortem investigation by the WES was structured to include the preceding studies. Figure 14 is a graphic summary of the test plan that also shows the locations selected for testing. Figure 15 presents a chronological history of the KTT, including the timing of the postmortem investigations.



LEGEND

SYMBOL	DESCRIPTION
P	LOCATION OF DUTCH CONE PENETROMETER TEST
V	LOCATION OF VIBROSEISMIC TEST
M	LOCATION OF MECHANICAL IMPEDANCE TEST
T	LOCATION OF TEST TRENCH IN EMBANKMENT
*	LOCATION OF MAIN (BUILT-IN) INSTRUMENTATION ARRAY

FIGURE 14. GRAPHIC SUMMARY OF THE WES TEST PLAN

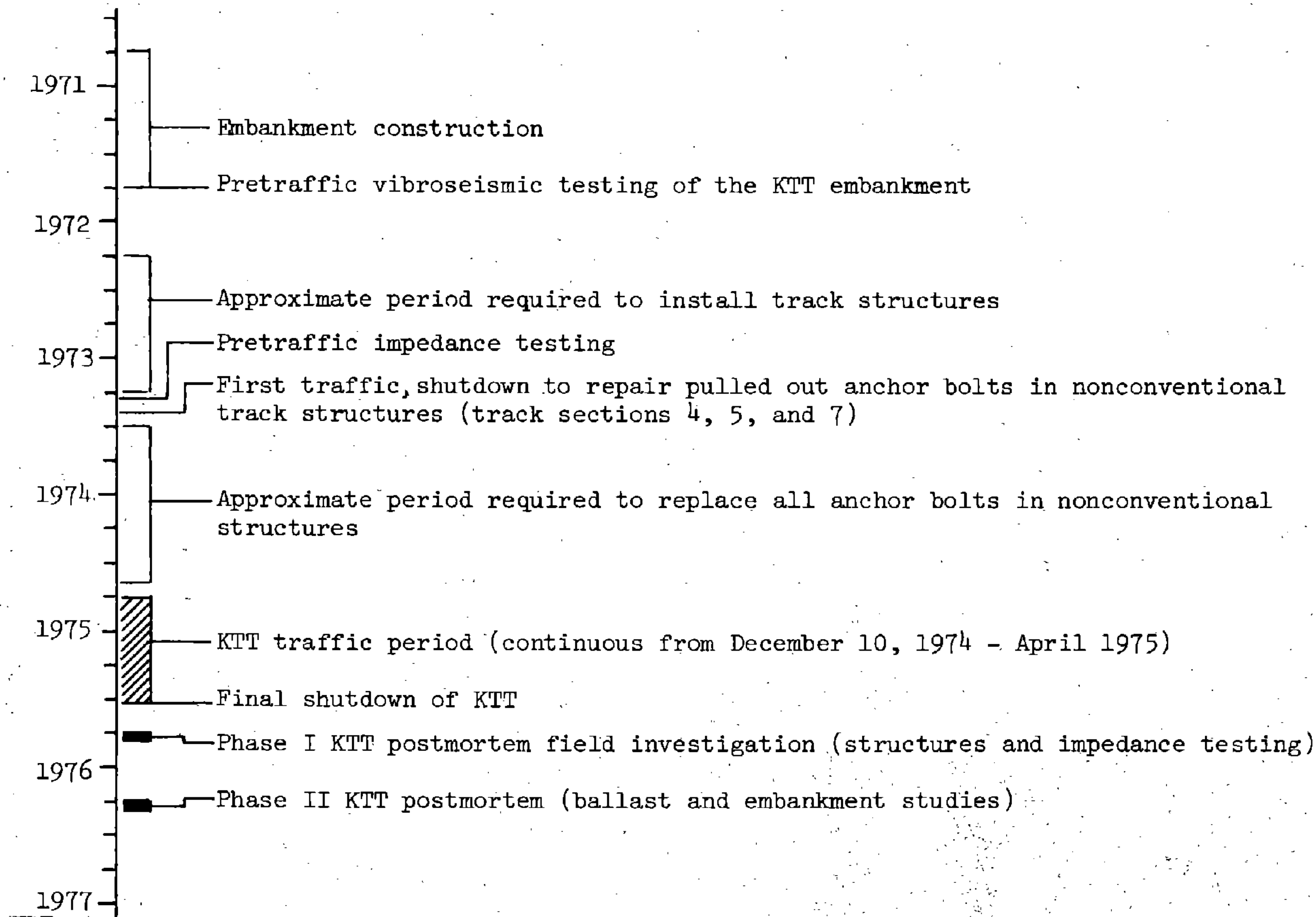


FIGURE 15. CHRONOLOGY OF KTT EVENTS, INCLUDING TIMING OF POSTMORTEM INVESTIGATIONS

FIELD INVESTIGATIONS

The KTT postmortem field investigation began with static and dynamic testing of the KTT track support systems (Phase I). This work was performed in the period September-October 1975 by a WES field party (a project engineer and two technicians). The MITRE representatives were present during part of this time to observe the static and dynamic tests conducted for their purposes. Upon completion of the Phase I work, the field party returned to the WES to document these results and to prepare for the Phase II field effort.

In February 1976 all parties scheduled to participate in Phase II work, consisting of studies of the ballast and embankment, had been contacted to coordinate WES planning. These participants included ATSF, MITRE, PCA, and S&W representatives. Phase II work was scheduled to commence on 21 April 1976, and final planning by the WES and the ATSF took place during a 24 March inspection trip to the KTT site. Findings during the inspection trip were as follows:

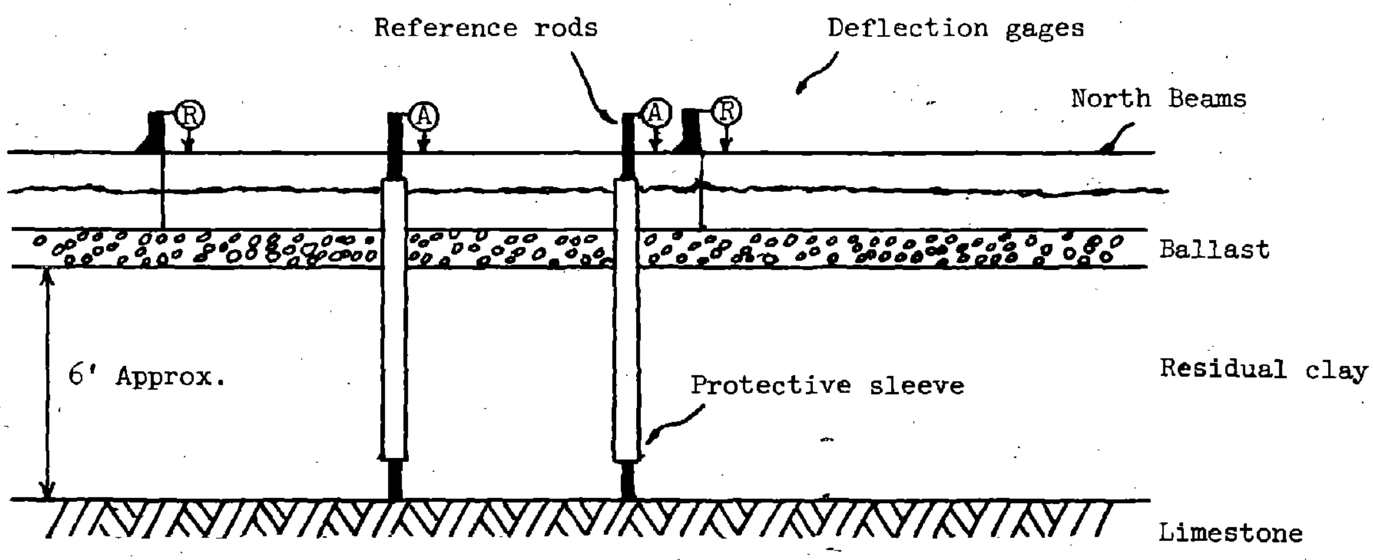
1. The test locations previously selected and marked by the WES were in excellent condition and showed no signs of disturbance.
2. The KTT concrete ties and rails had been removed as required for Phase II testing, but their removal had so disturbed the ballast between test locations that WES vehicles could not travel over it.
3. The ATSF did not feel that the nonconventional track structures could be removed prior to arrival of the WES field party, because contract negotiations for the work were then in progress.

The ATSF agreed to arrange for a motor grader to level the ballast; however, it was apparent that removal of the nonconventional track structures would have to be arranged to suit circumstances. In view of other WES commitments, which precluded rescheduling KTT field work, the only alternative was to remove the structures in stages as WES field work progressed. The ATSF also agreed to secure the necessary heavy equipment and to provide other assistance as needed so the Phase II investigation could proceed as scheduled. Although plans had been made for long-term protection of the foundation materials beneath the structures, removal of the structures just prior to testing minimized this requirement. Hence, only a weighted tarpaulin was needed to protect the test area from environmental changes in moisture content while testing was in progress. The nonconventional structures were continuous reinforced concrete, and jackhammers and cutting torches were used to sever them for removal. Each severed beam or slab section was vertically lifted with a 60-ton crane to minimize disturbance of the ballast and foundation materials. Testing was begun immediately thereafter.

The Phase II field work was performed in April and May 1976 by a WES field party (three engineers and nine technicians). The PCA, MITRE, and S&W representatives observed the work in progress during the week of 17 through 24 May. An ATSF representative was present at all times when work was under way at the site.

Structure Testing (Phase I)

Structure testing began 16 October 1975 with the instrumentation of north rail Beams 14, 15, 16, 17, and 18 in Track Section 4. Vertical extensometers were installed at the midpoint and at one end of each beam to measure absolute deflections of the beams with respect to the limestone bedrock. In the dynamic tests, absolute and relative motions were measured using film potentiometers. The dynamic and static test configurations specified by MITRE are shown in Figure 16; detailed descriptions of the installation and data acquisition procedures are given in Appendix A, Volume II, of this report. Data were acquired for 17 passes of the captive train at speeds ranging from 2 to 55 mph. For the dynamic test series, 13 data channels were recorded, including absolute and relative motion of the beams, rail accelerations, bending strain in the beams, and peak particle velocity measurements of the rail and beam motions. A second series of structural tests, consisting of three static load-deflection tests on Beams 14 through 18 in Track Section 4 was then conducted. Dial indicator measurements of relative and absolute deflections of these beams, as well as strain gage measurements of



Legend

(R) Relative deflection measured between beams

(A) Absolute deflection referenced to limestone bedrock

Note: Dial gages were used to make static test measurements. In the dynamic tests, deflection measurements were made using film potentiometers and analog signal conditioning and tape systems.

FIGURE 16. TYPICAL DEFLECTION MEASUREMENT CONFIGURATION FOR EACH BEAM IN THE MITRE STATIC AND DYNAMIC TESTS, BEAMS 14-18, TRACK SECTION 4

bending strains induced in Beam 16, were recorded for each load increment and decrement.

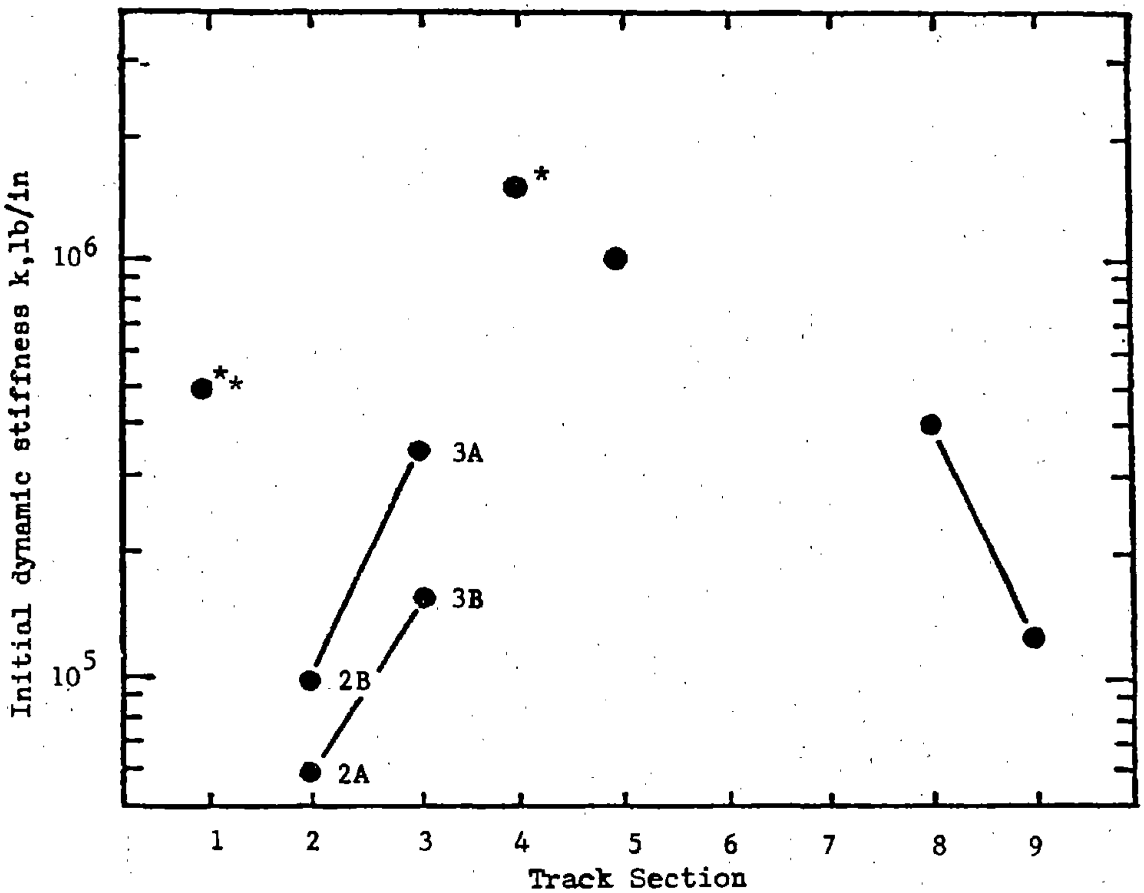
The above static and dynamic data were transmitted to MITRE in February 1976 and have since been used to refine an existing analytical model of nonconventional track structure response.[10]

The final postmortem structures testing consisted of WES posttraffic mechanical impedance tests on the track support systems in Track Sections 1, 2, 3, 4, 5, 8, and 9. As planned and prior to testing, an effort was made to re-create subgrade pumping conditions at each test location by the application of water and repeated passes of a captive train. Despite up to 600 train passes and the surface application of over 16,000 gal of water to some test locations, surface indications of pumping were achieved only in Track Sections 1, 2, and 3, although some traces of the onset of pumping were observed in Track Section 5.

Detailed descriptions of the pretraffic and posttraffic impedance investigations performed by the WES are available in a separate report[8] but will not be repeated here in the interest of brevity. However, a ranking of KTT track systems according to their initial dynamic stiffness, as determined under posttraffic (postmortem) test conditions, is significant to the postmortem investigation and is shown in Figure 17. Comparisons based on these and other pertinent data will be treated in the analysis section of this report.

Embankment Vibroseismic Study

The postmortem vibroseismic study was performed as part of the Phase II field work conducted in April and May 1976. Vibroseismic testing was performed at each test pit location (Figure 18), according to procedures that are documented in the literature[7] and described in detail in Appendix B, Volume II, of this report. Volume II also contains all of the original data acquired in the study. Postmortem vibroseismic results from each track section tested are summarized in Figures 19 through 28. Each figure contains a plot of Young's (E) and shear (G) moduli versus depth in the embankment. A summary plot of all vibroseismic data is shown in Figure 29. The E and G moduli curves in



*Not considered to be representative of performance under traffic; see text.

FIGURE 17. SUMMARY OF INITIAL DYNAMIC STIFFNESS RESULTS FROM POST-TRAFFIC IMPEDANCE TESTING

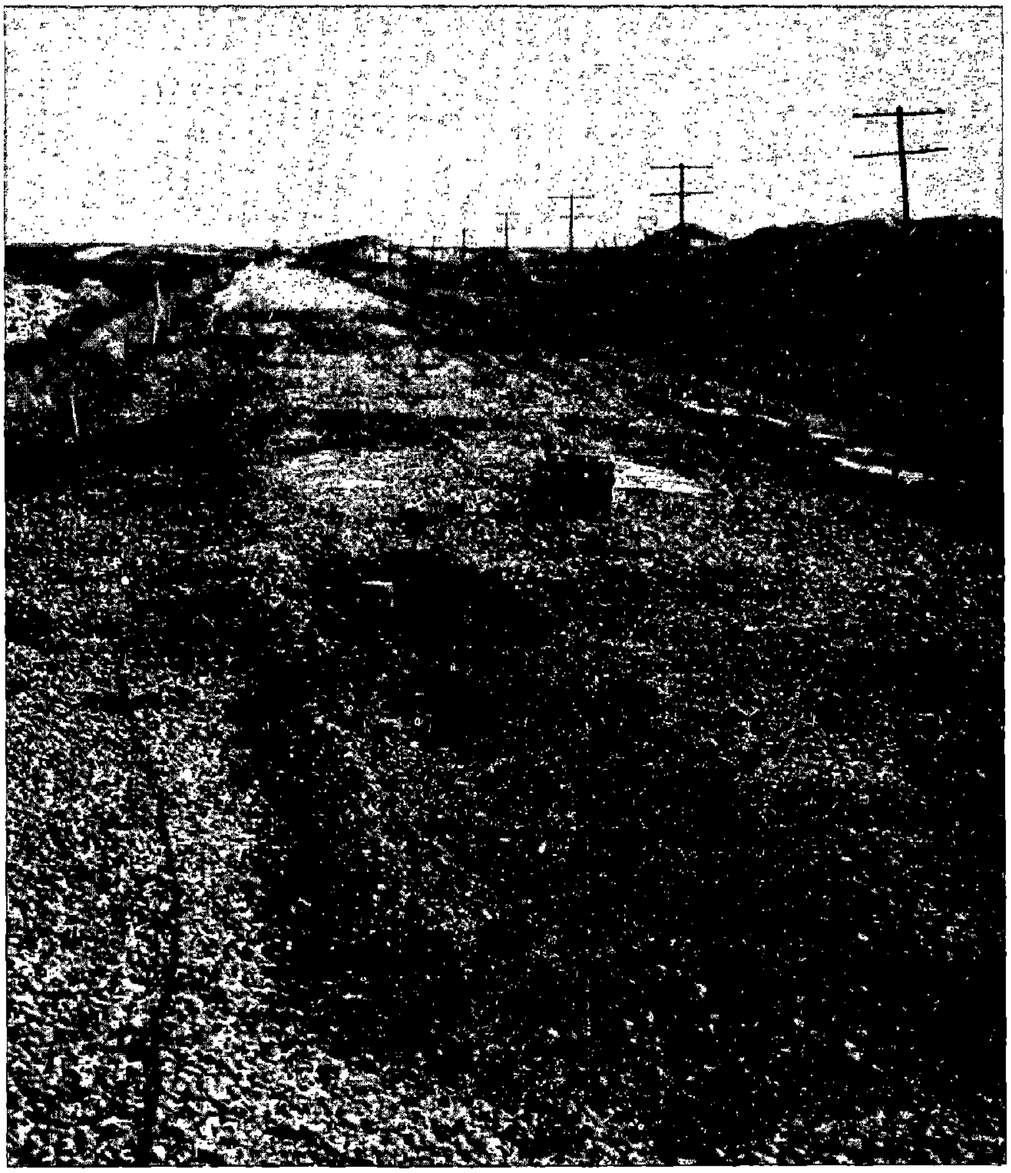


FIGURE 18. TYPICAL VIBROSEISMIC TEST CONFIGURATION

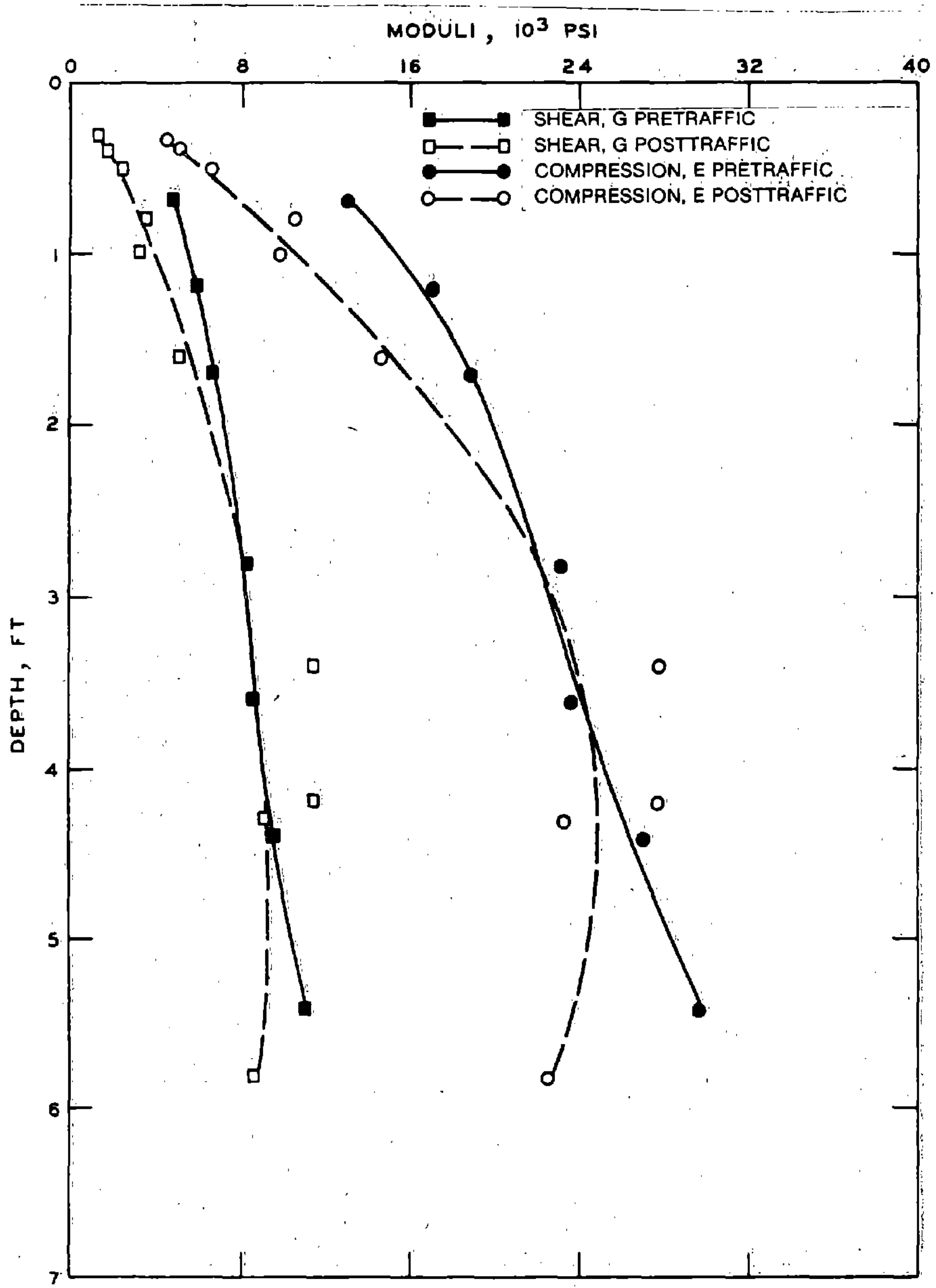


FIGURE 19. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 1 (KTT EMBANKMENT)

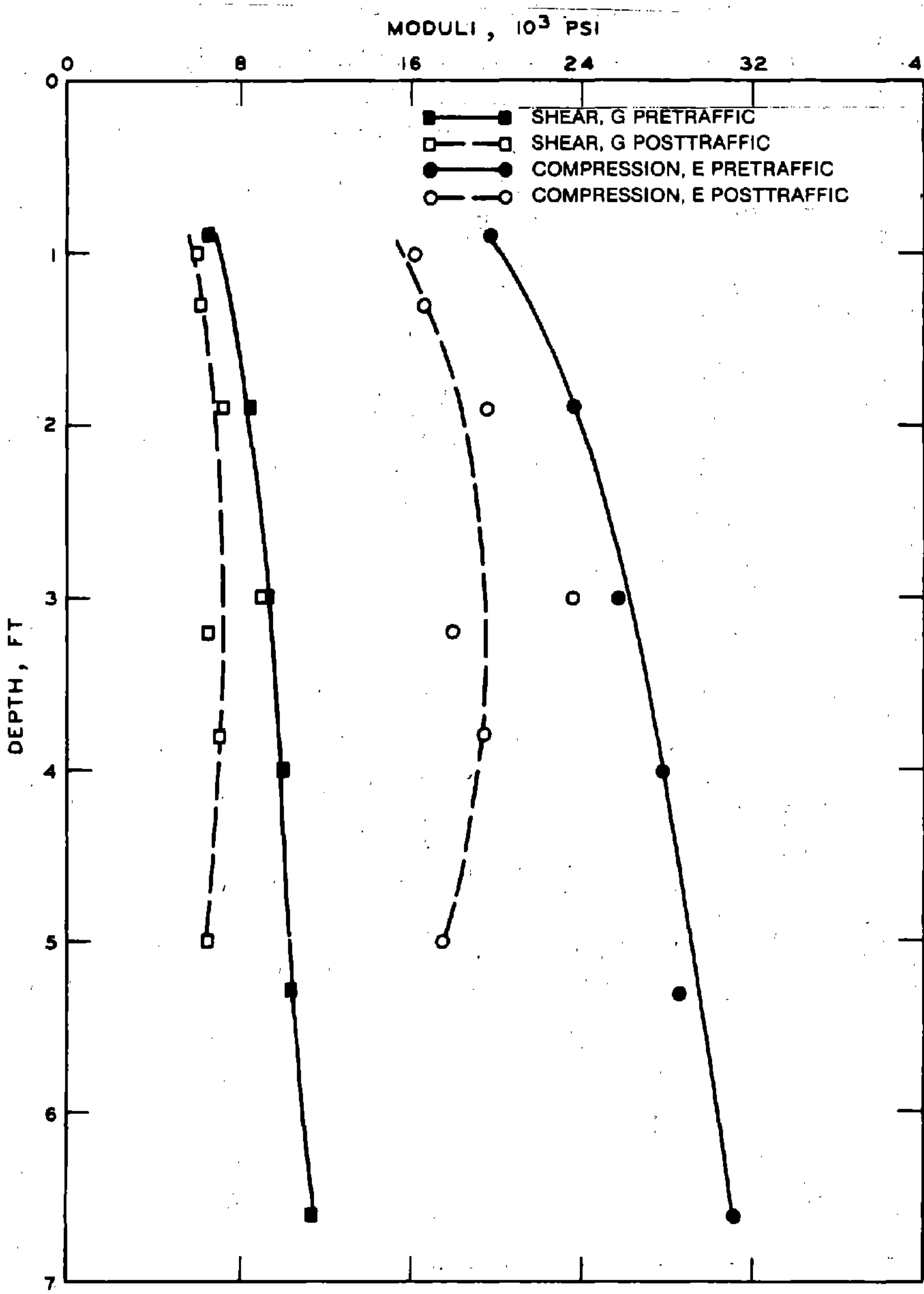


FIGURE 20. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 2 (LOCATION 2A, KTT EMBANKMENT)

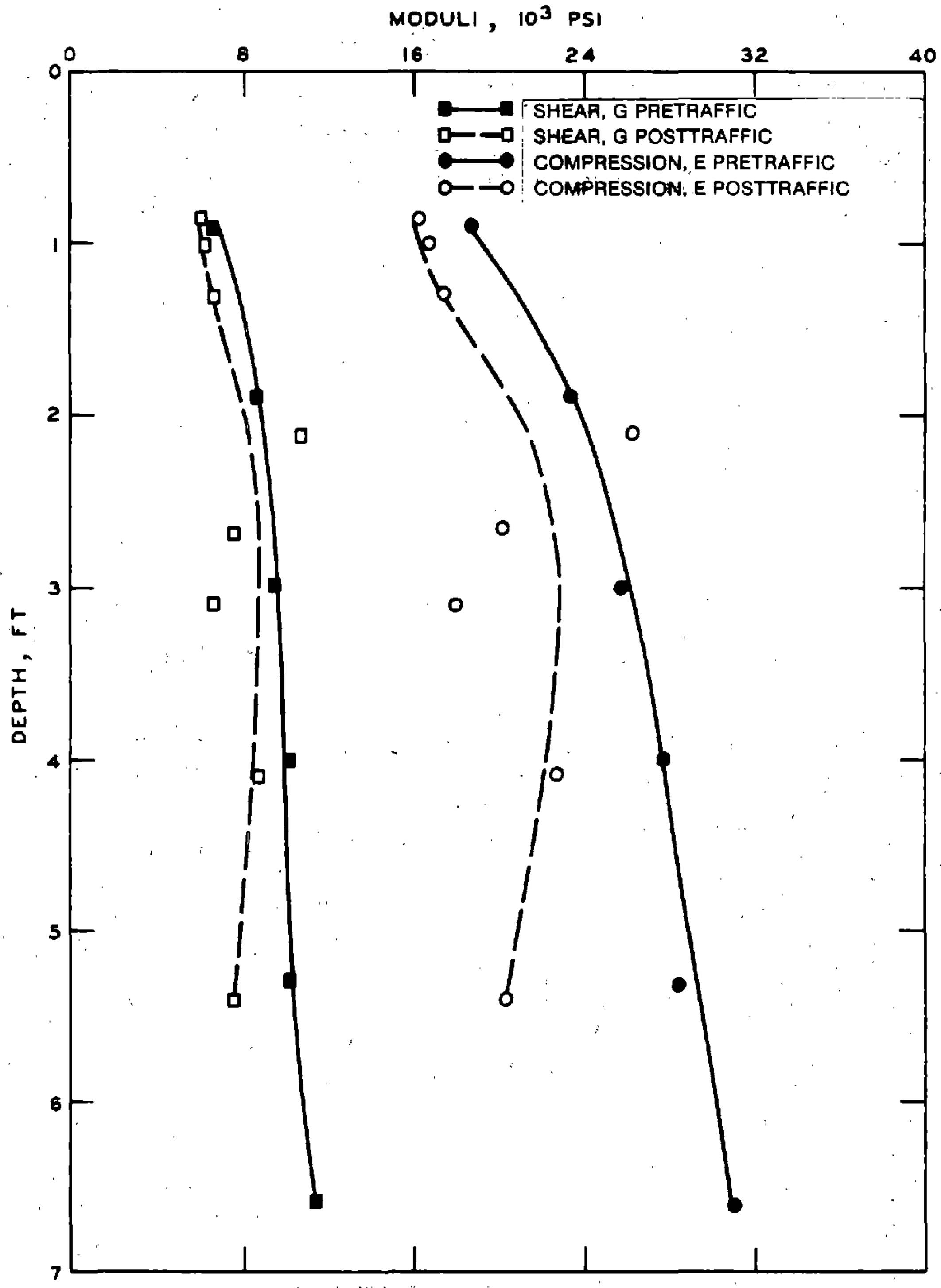


FIGURE 21. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 2 (LOCATION 2B, KTT EMBANKMENT)

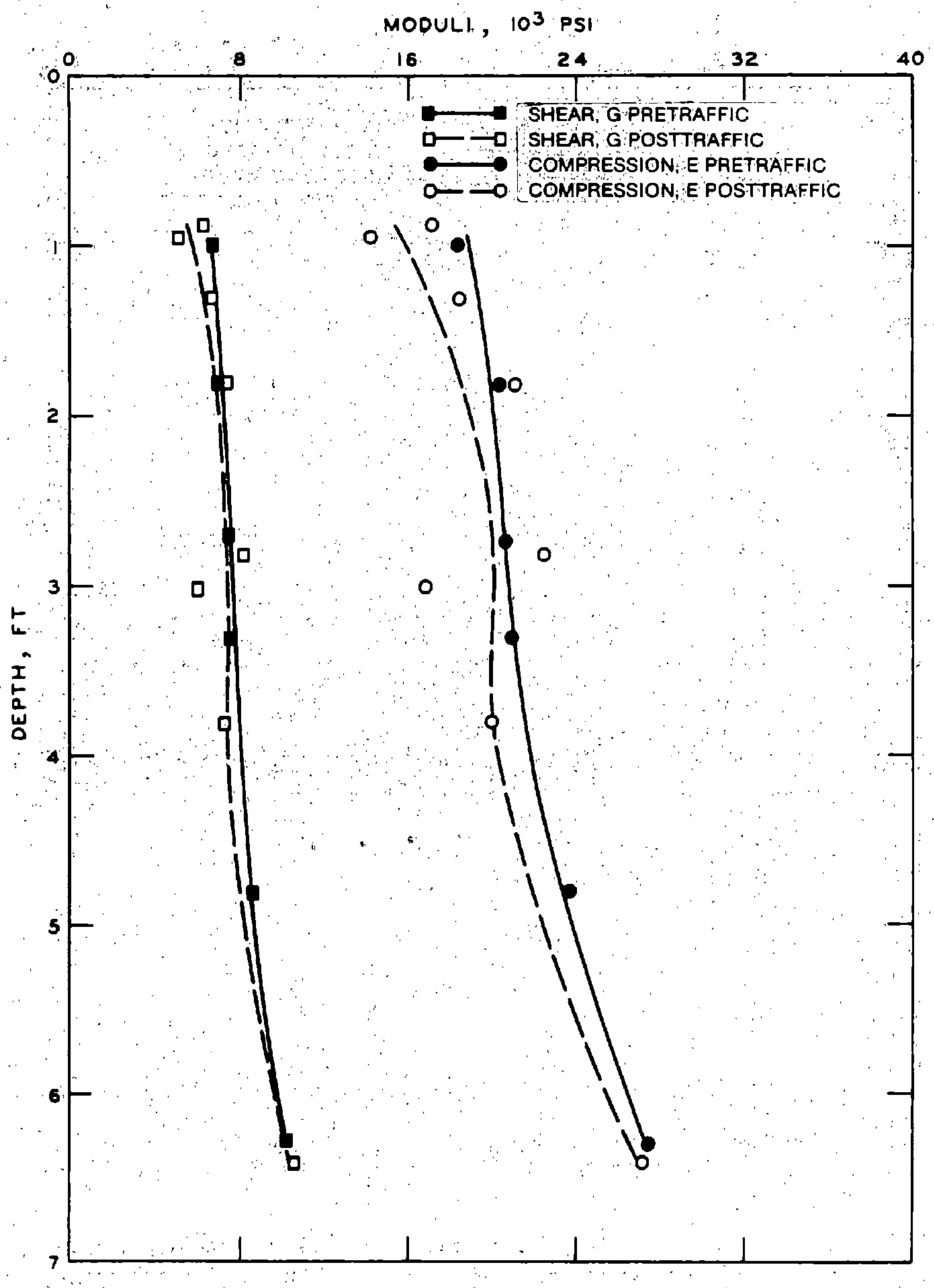


FIGURE 22. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 3 (LOCATION 3A, KTT EMBANKMENT)

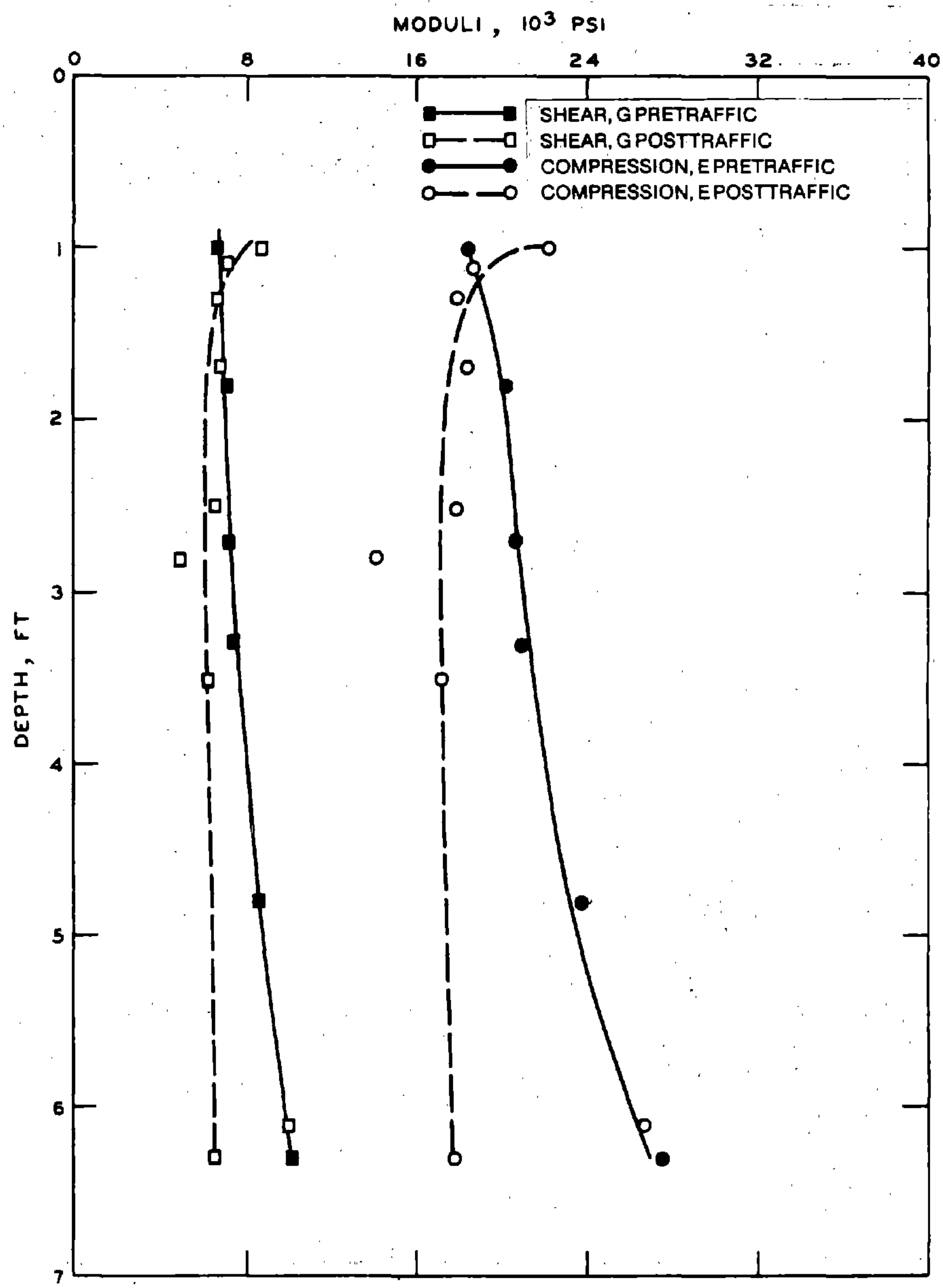


FIGURE 23. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 3 (LOCATION 3B, KTT EMBANKMENT)

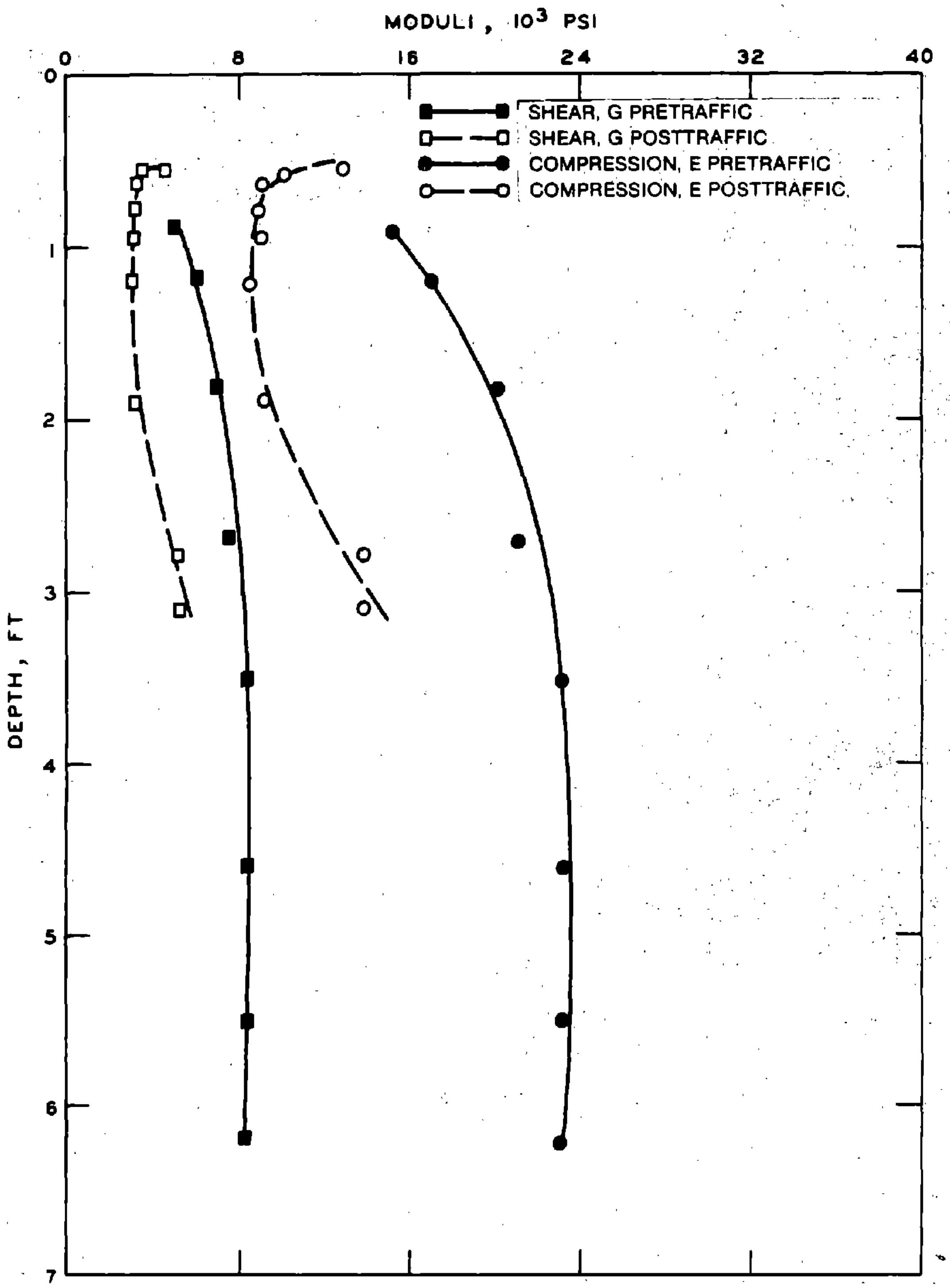


FIGURE 24. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 4 (KTT EMBANKMENT)

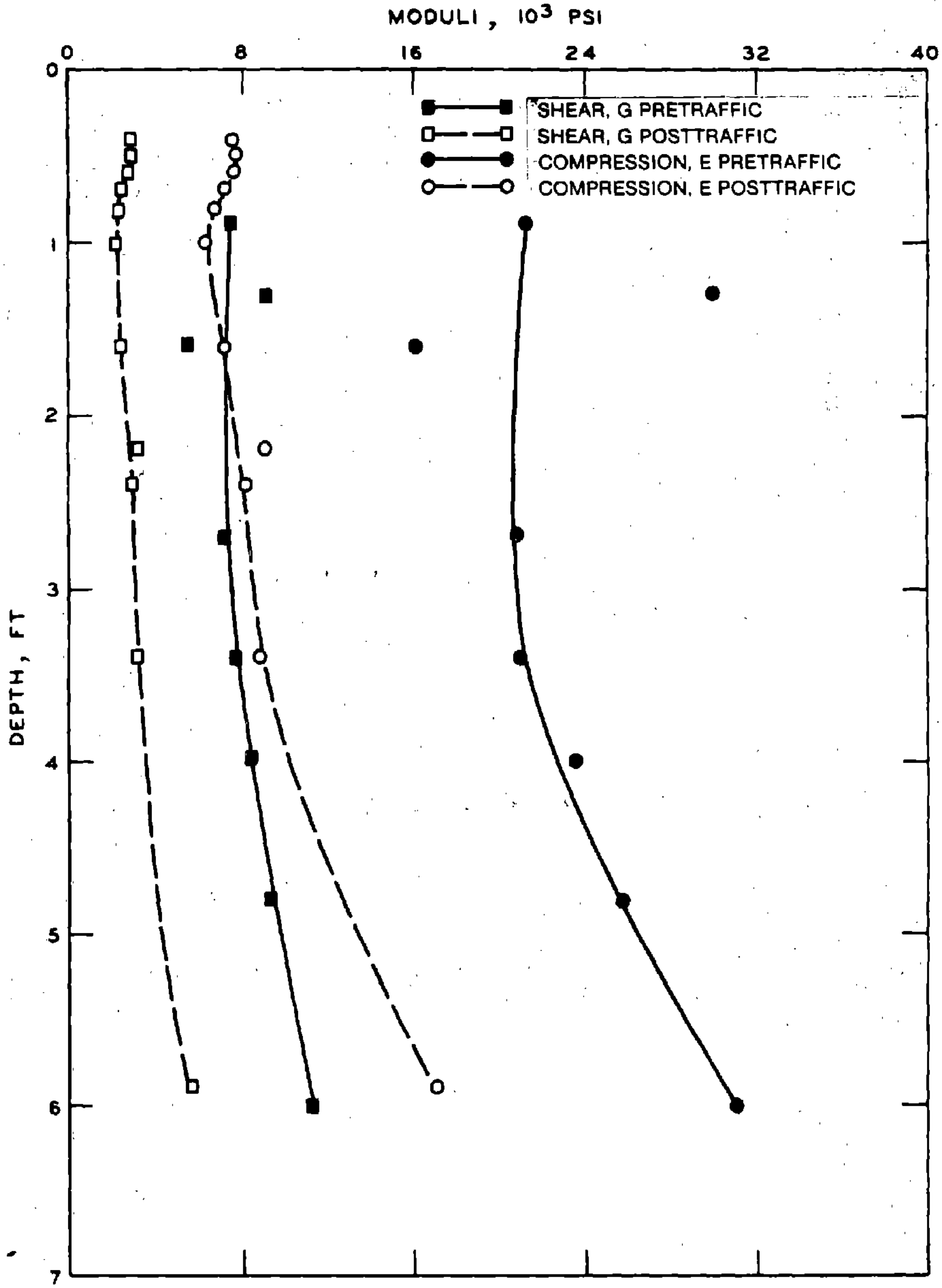


FIGURE 25. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 5 (KTT EMBANKMENT)

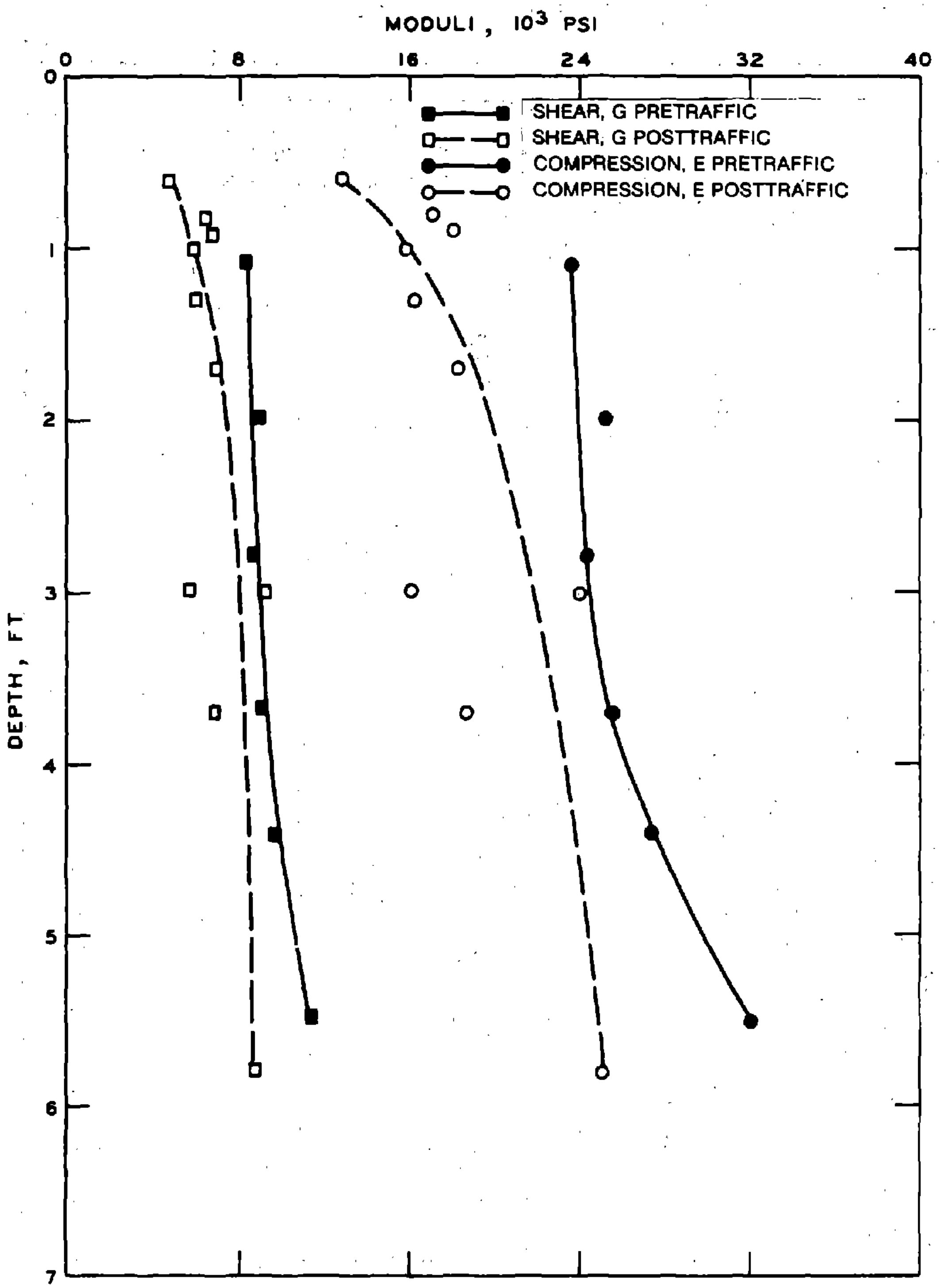


FIGURE 26. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 7 (KTT EMBANKMENT)

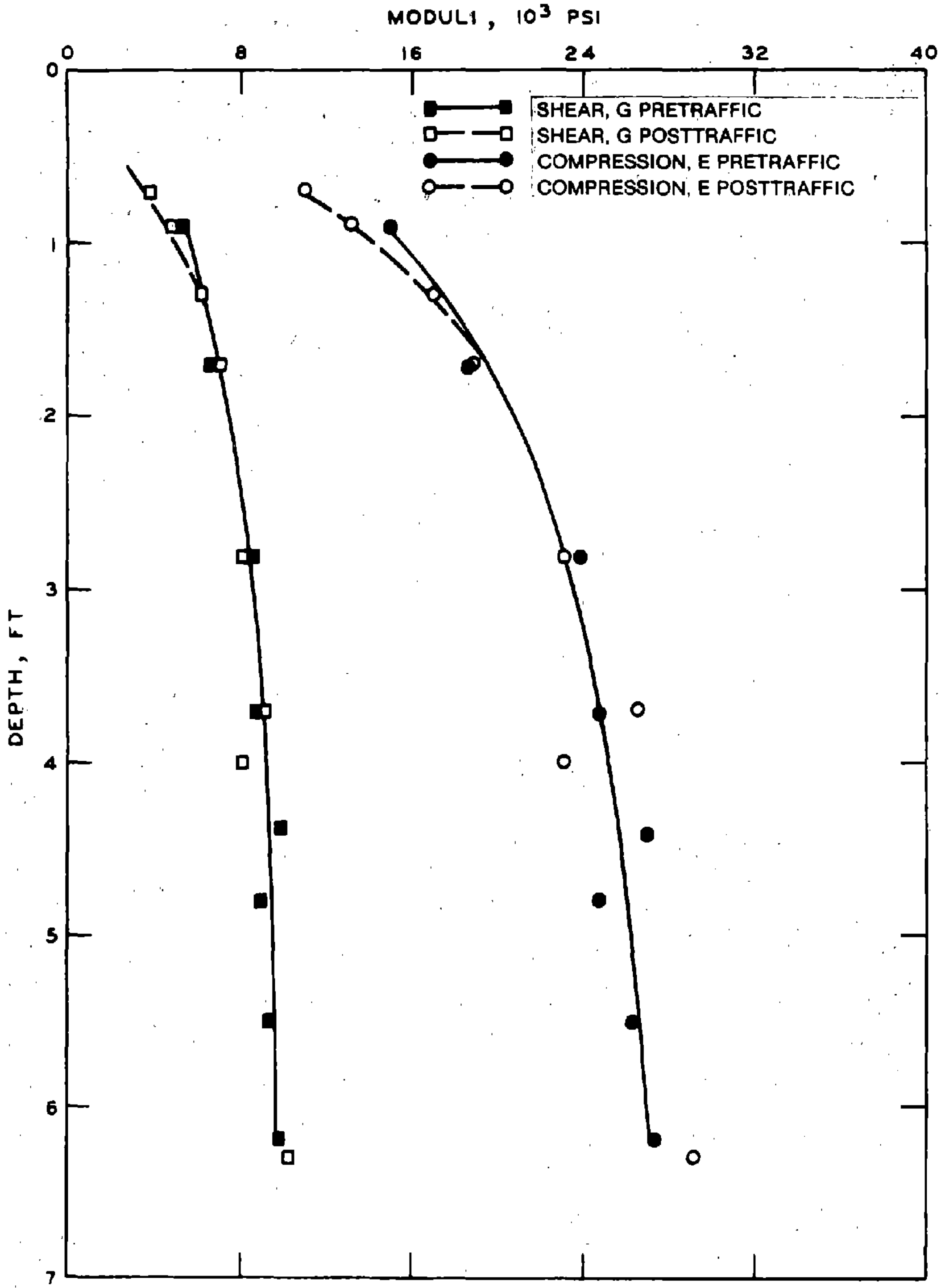


FIGURE 27. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 8 (KTT EMBANKMENT)

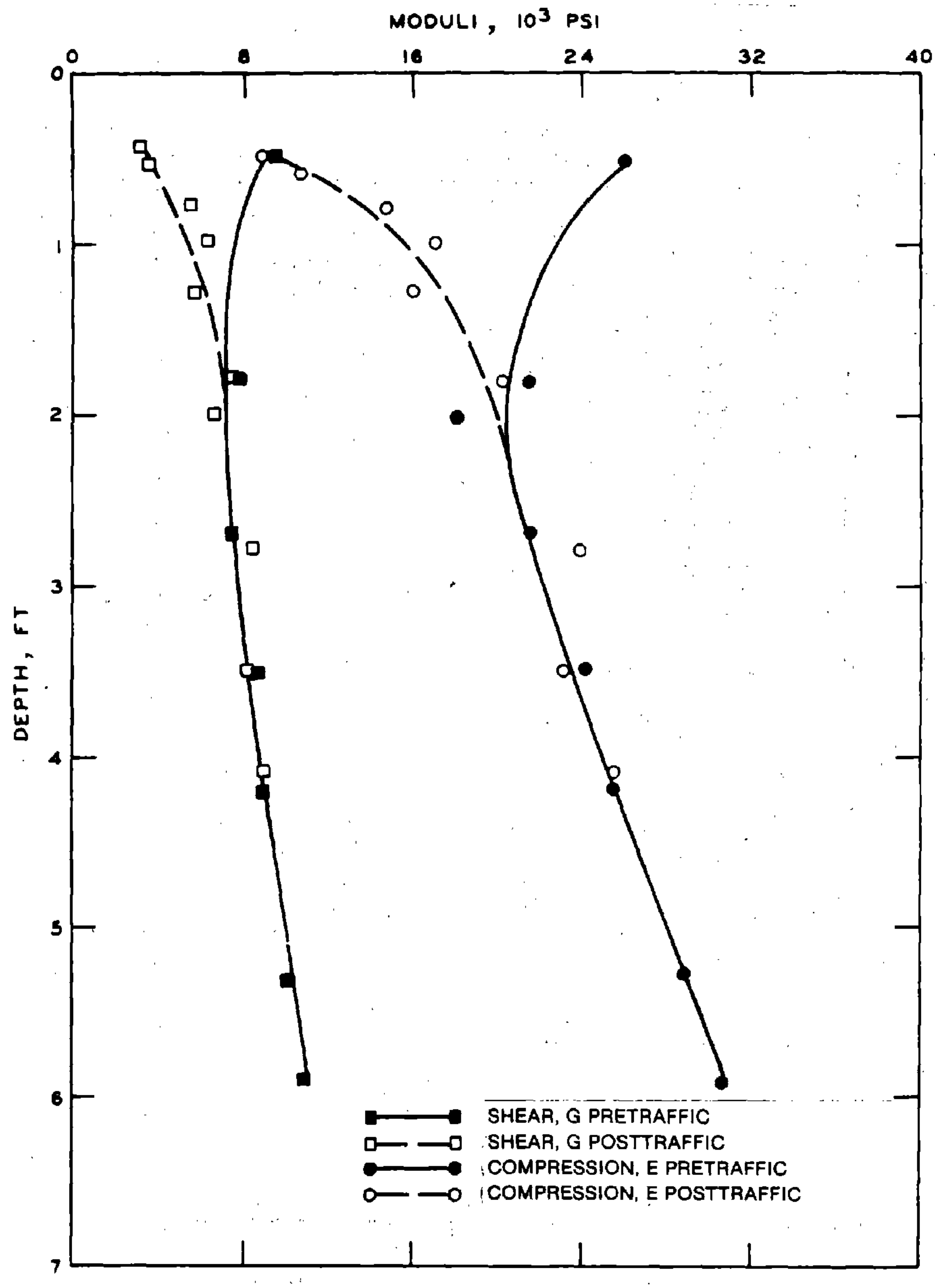


FIGURE 28. PRETRAFFIC AND POSTTRAFFIC PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTION 9 (KTT EMBANKMENT)

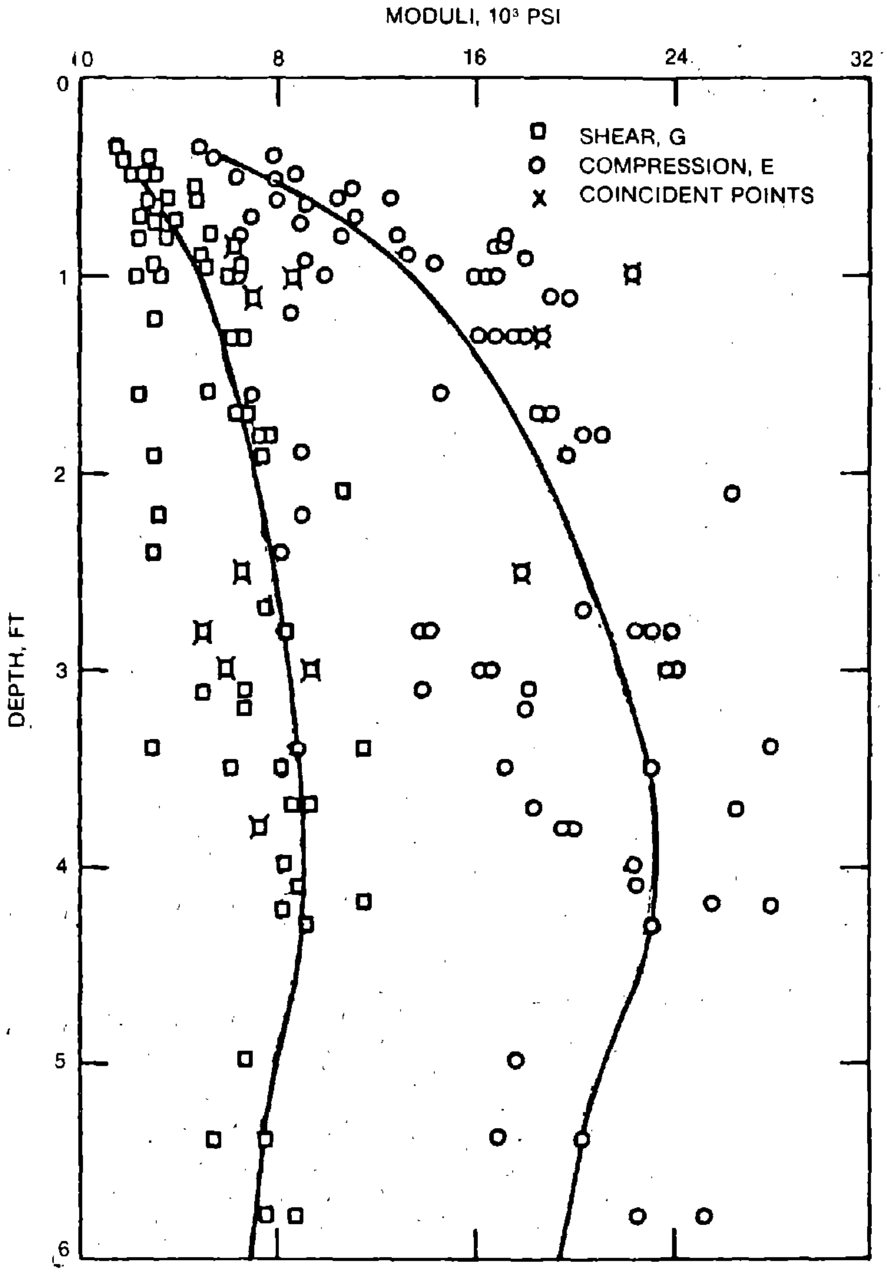


FIGURE 29. PRETRAFFIC AND POSTTRAFFIC SUMMARY PLOTS OF ELASTIC MODULI VERSUS DEPTH, TRACK SECTIONS 1 THROUGH 9 (KTT EMBANKMENT)

Figure 29 represent average values for the entire embankment. All of the above vibroseismic data were obtained at relatively low stress levels in comparison with rail traffic induced soil stresses, and the E and G moduli derived are considered to represent upper bound values of embankment strength.

Embankment Dutch Cone Penetrometer Study

Dutch cone penetrometer tests (CPT) were performed using a portable penetrometer. Penetrometer testing was originally envisioned as an expedient means for assessing embankment uniformity throughout its length, and penetrometer tests were conducted at four to five locations in each section, as shown in Figure 14. The portable Dutch cone penetrometer used by the WES was adapted to a forklift (Figure 30) so the apparatus could be easily transported to each test location. Two lead blocks, weighing 2000 lb each, were used to react against the vertical uplift force generated by the penetrometer. Two to three penetrations of the subgrade were made at each test location. These penetration positions were patterned to typify soil conditions at various points of interest beneath the structures. Readings of penetration resistance began at the ballast-subgrade interface and were obtained every 6 in. in depth to a total depth of 6 ft in the embankment. Figures 31 through 33 present averaged CPT results for each section tested, in numerical order. Figure 34 presents the averaged CPT results for each penetration position in the conventional structure (Track Sections 1, 2, 3, 8, and 9) test pattern. Figure 35 presents averaged CPT results for the penetration positions used in the nonconventional structure (Track Sections 4, 5, and 7) test pattern. Figure 36 shows the mean and standard deviation curves for all of the penetrometer data. In each figure, cone point penetration resistance in tons per square foot (tsf) versus depth in the embankment is plotted. The equipment and procedures used and the data acquired in the cone penetrometer study are documented in Appendix C, Volume II, of this report. Further details regarding cone penetration equipment and procedures are available from the literature.[11]

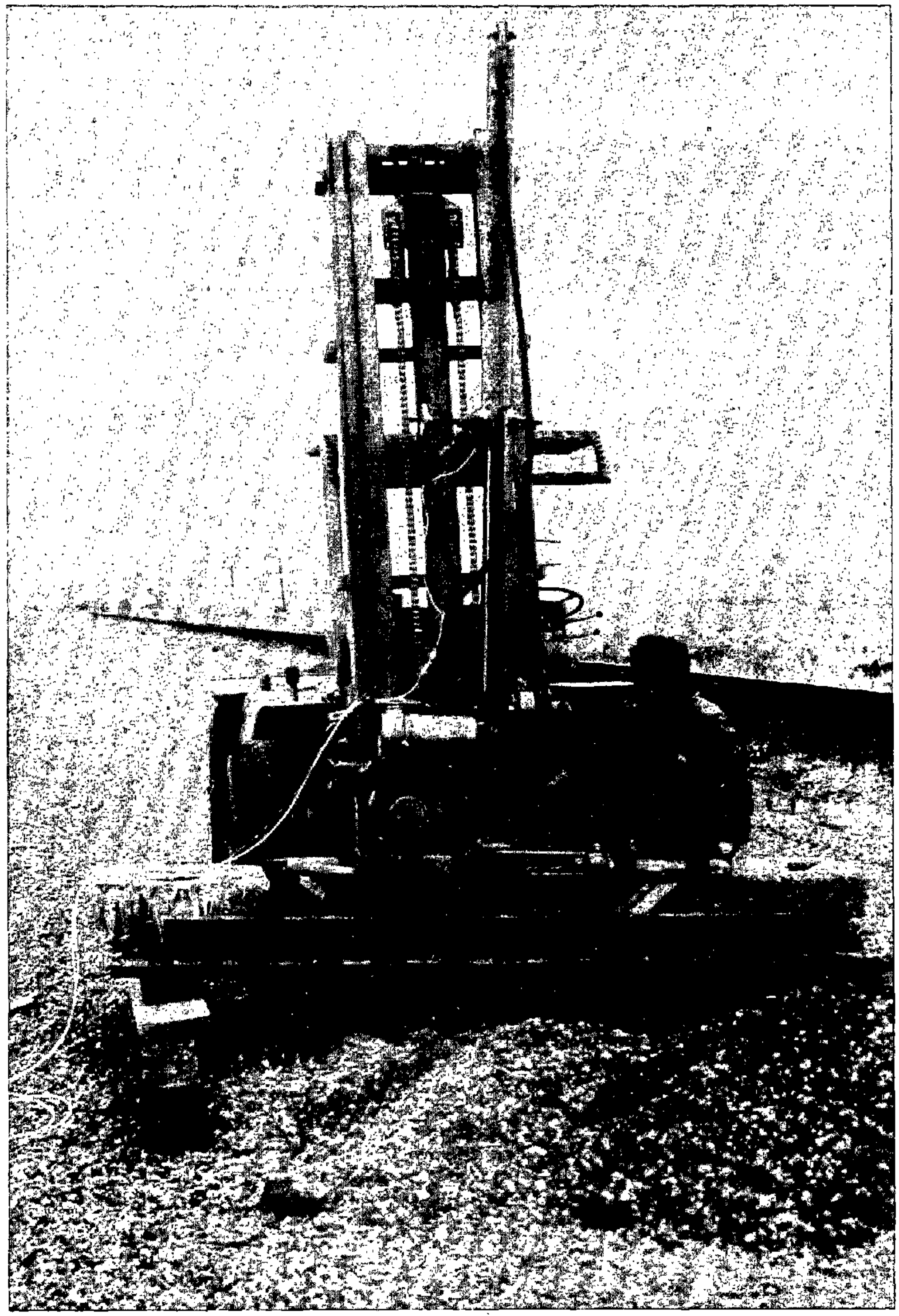


FIGURE 30. WES MODIFIED DUTCH CONE PENETROMETER AT TEST LOCATION IN TRACK SECTION 3

Embankment

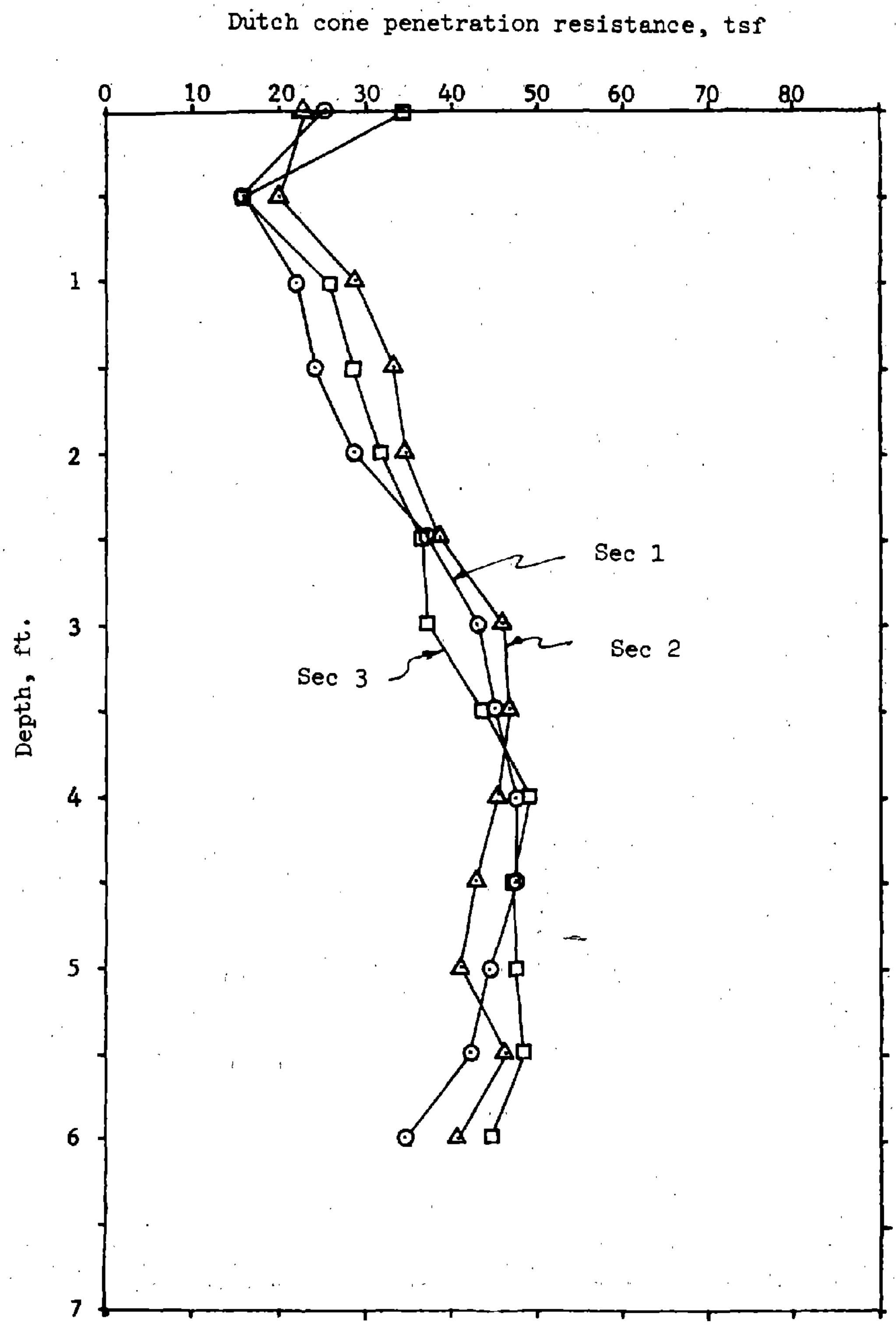


FIGURE 31. AVERAGE PENETRATION RESISTANCE IN TRACK SECTIONS 1, 2, AND 3

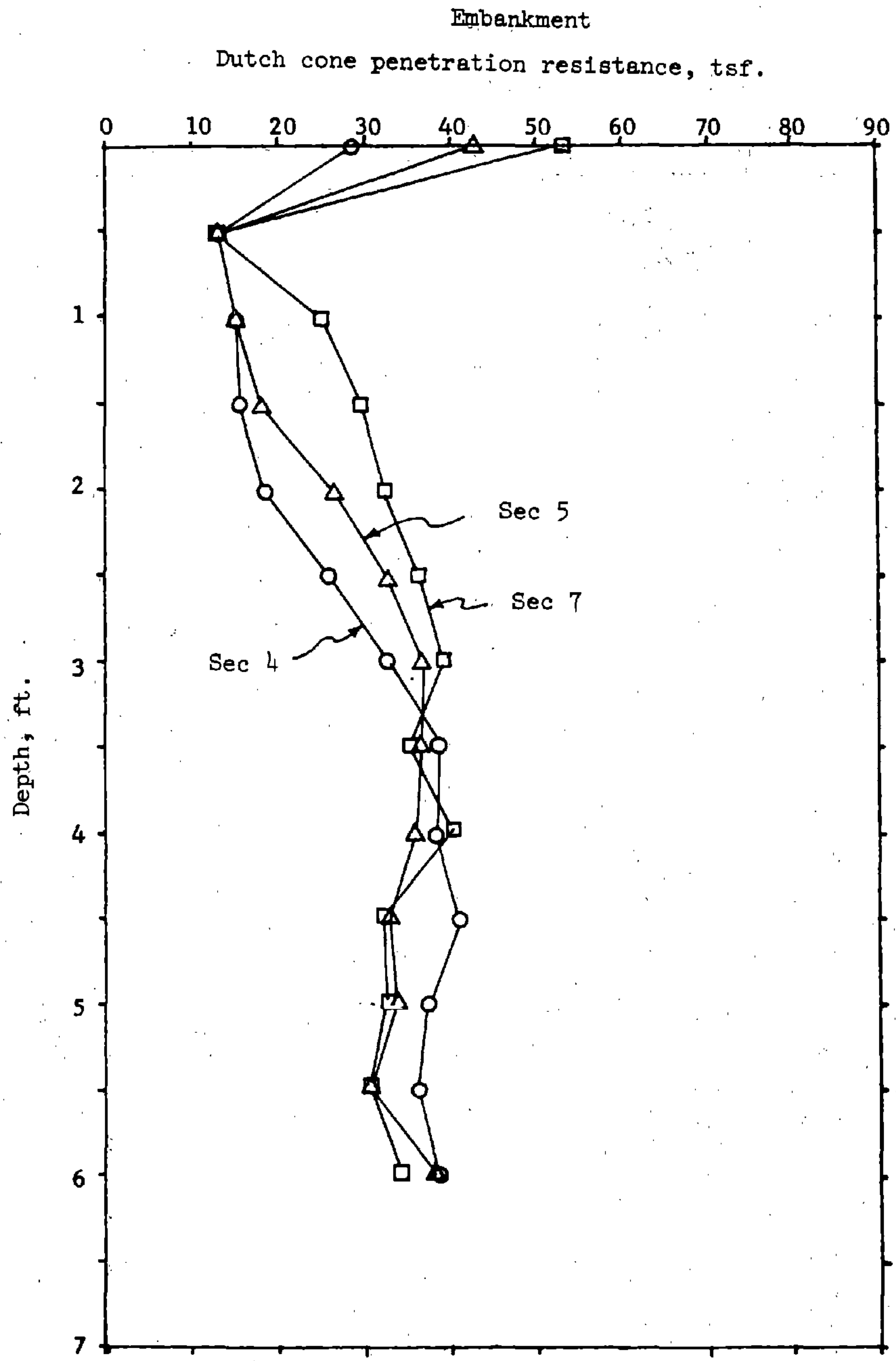


FIGURE 32. AVERAGE PENETRATION RESISTANCE IN TRACK SECTIONS 4, 5, AND 7

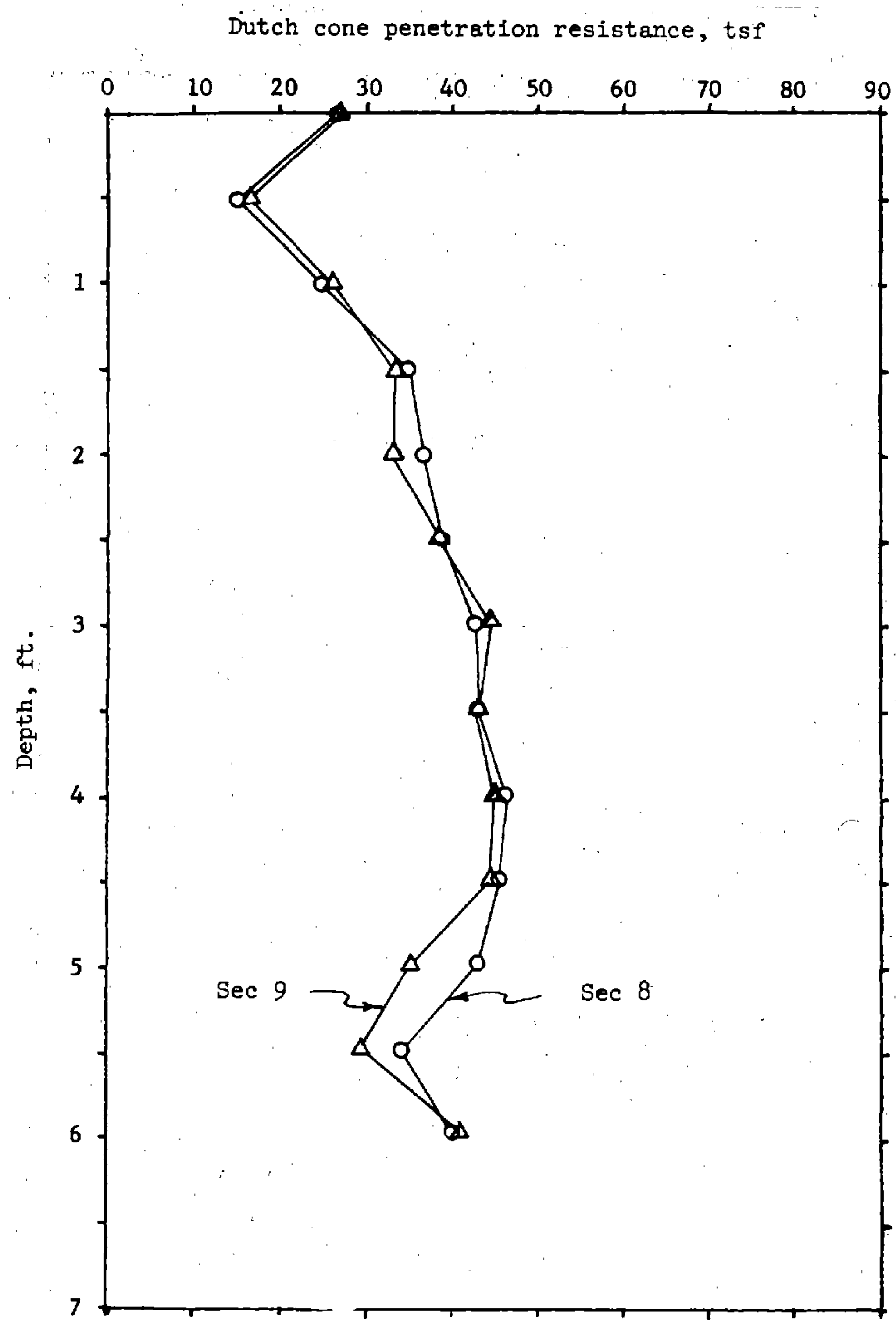


FIGURE 33. AVERAGE PENETRATION RESISTANCE IN TRACK SECTIONS 8 AND 9

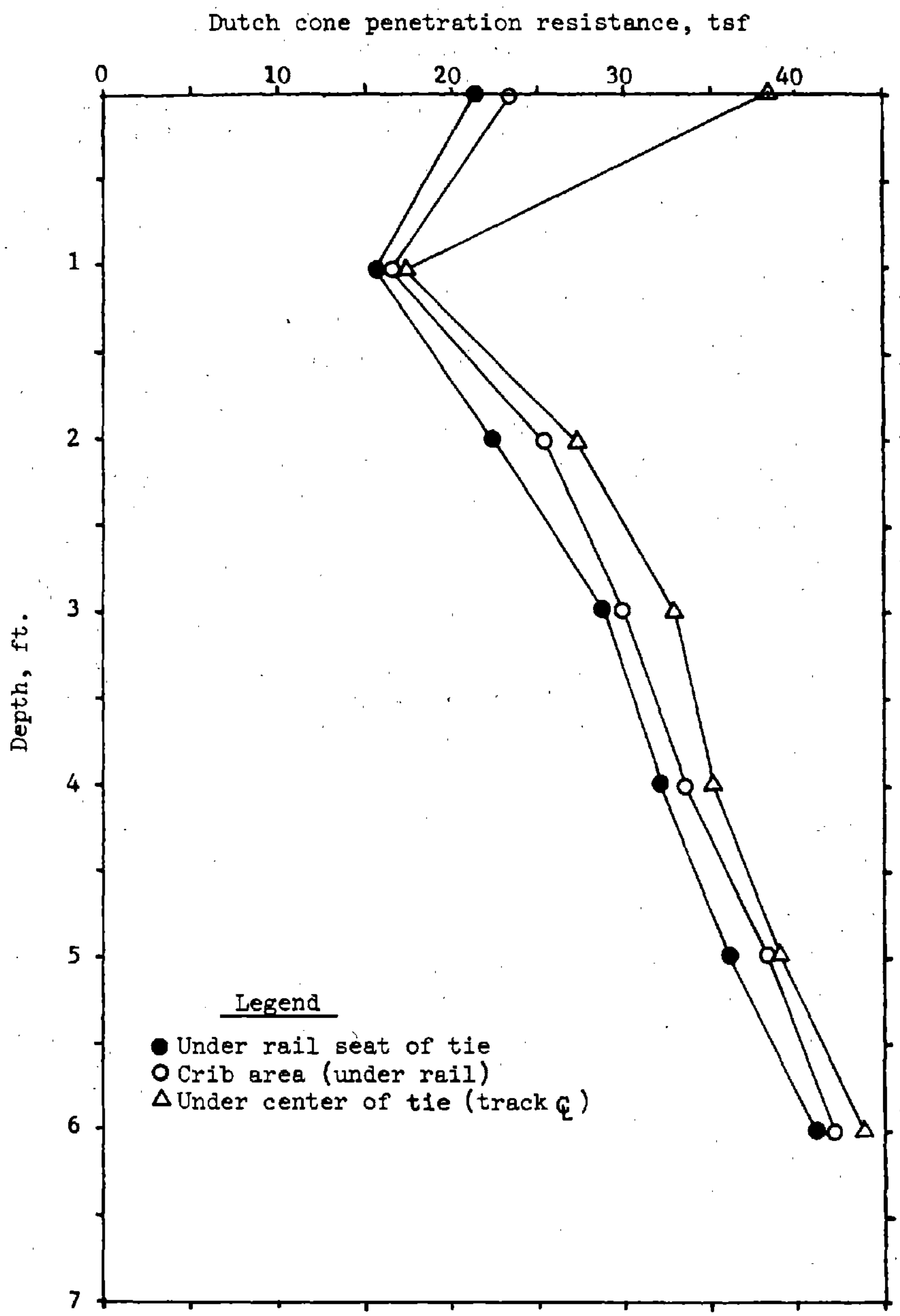


FIGURE 34. AVERAGE PENETRATION RESISTANCE AT SELECTED LOCATIONS UNDER THE CONVENTIONAL (TIE) TRACK STRUCTURES (TRACK SECTIONS 1, 2, 3, 8, AND 9)

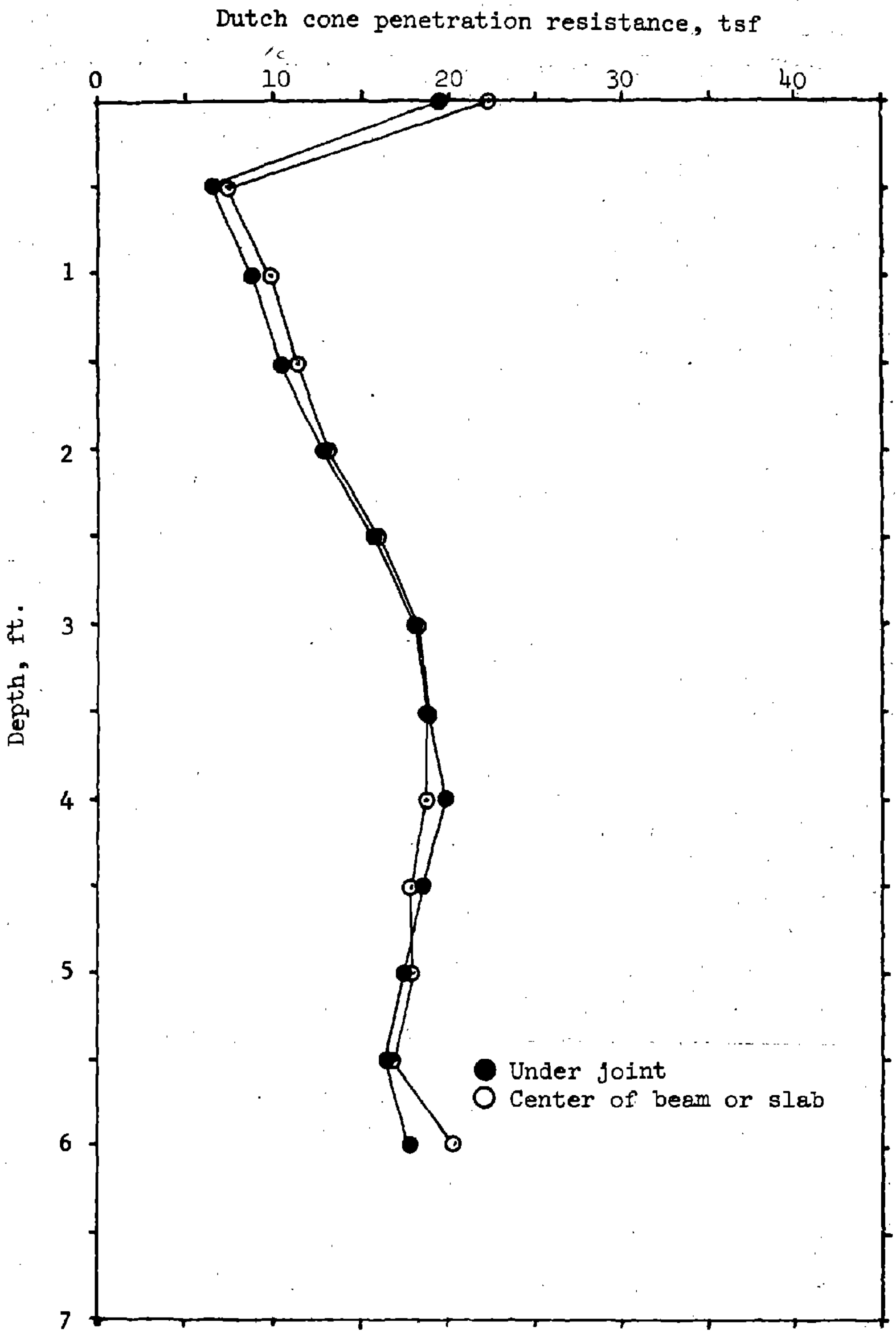


FIGURE 35. AVERAGE PENETRATION RESISTANCE AT SELECTED LOCATIONS UNDER THE NONCONVENTIONAL TRACK STRUCTURES (TRACK SECTIONS 4, 5, and 7)

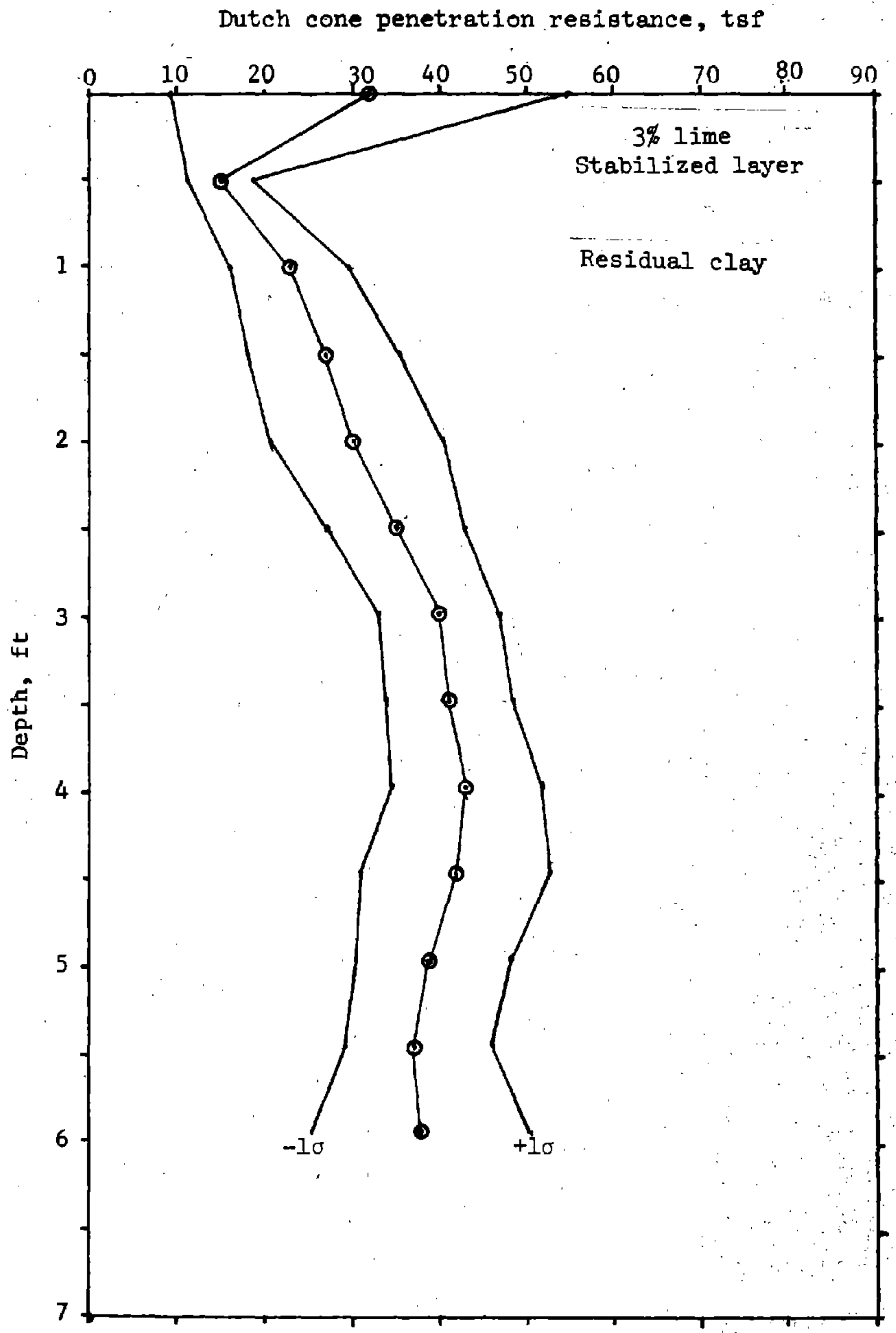


FIGURE 36. MEAN AND STANDARD DEVIATION CURVES OF PENETRATION RESISTANCE VERSUS DEPTH, BASED ON ALL PENETROMETER DATA (1235 DATA POINTS)

Test Trenching

Ten test pits were excavated through the ballast and 3 ft into the KTT embankment. Details of equipment and procedures used in the trenching operations, as well as all the original data, are presented in Appendix D, Volume II, of this report. Locations of the pit center lines are given in Figure 14. Each track section, except Section 6, had one test pit; Track Sections 2 and 3 had two pits each, primarily to assess pretest performance judgments. Track Section 6 was not tested for reasons given earlier in this report.

At each pit location the track structure was first removed by lifting vertically with either a backhoe (Figure 37) or a crane (Figure 38) to minimize disturbance of the ballast beneath. Next, 30-in.-diam plate bearing tests were run at the structure-ballast interface, as described in ASTM Method D1196-64.[12] Results of these tests are summarized in Table 2. A typical load deflection curve for the plate bearing test is shown in Figure 39. It can be seen from Figure 39 that a large part of the total deflection was not recoverable.

Bag samples were next obtained from the ballast just beneath the structure (0- to 6-in. depth) and from just above the ballast-subgrade interface (6- to 15-in. depth). Only one sample could be obtained beneath the nonconventional structures, because the ballast was no more than 2-3 in. thick in Track Sections 4, 5, and 7. A sample of the shoulder ballast was also obtained for purposes of comparison. Results of material property tests on the ballast samples are discussed in the next section.

After the ballast samples were obtained, a backhoe was used to remove the excess ballast to within 4 to 6 in. of the subgrade. The remaining ballast in the test pit area was removed by hand to preserve the ballast-subgrade interface. Cross sections and profiles of the exposed ballast-subgrade interface were taken as well as photographs. An example of the latter, Figure 40, shows the subgrade surface in Test Pit 1. This photograph reveals the area beneath the north rail; the pronounced depressions in the subgrade were located beneath the north

FIGURE 37. REMOVAL OF CONVENTIONAL TRACK STRUCTURES FROM TEST LOCATION 2A (TRACK SECTION 2)

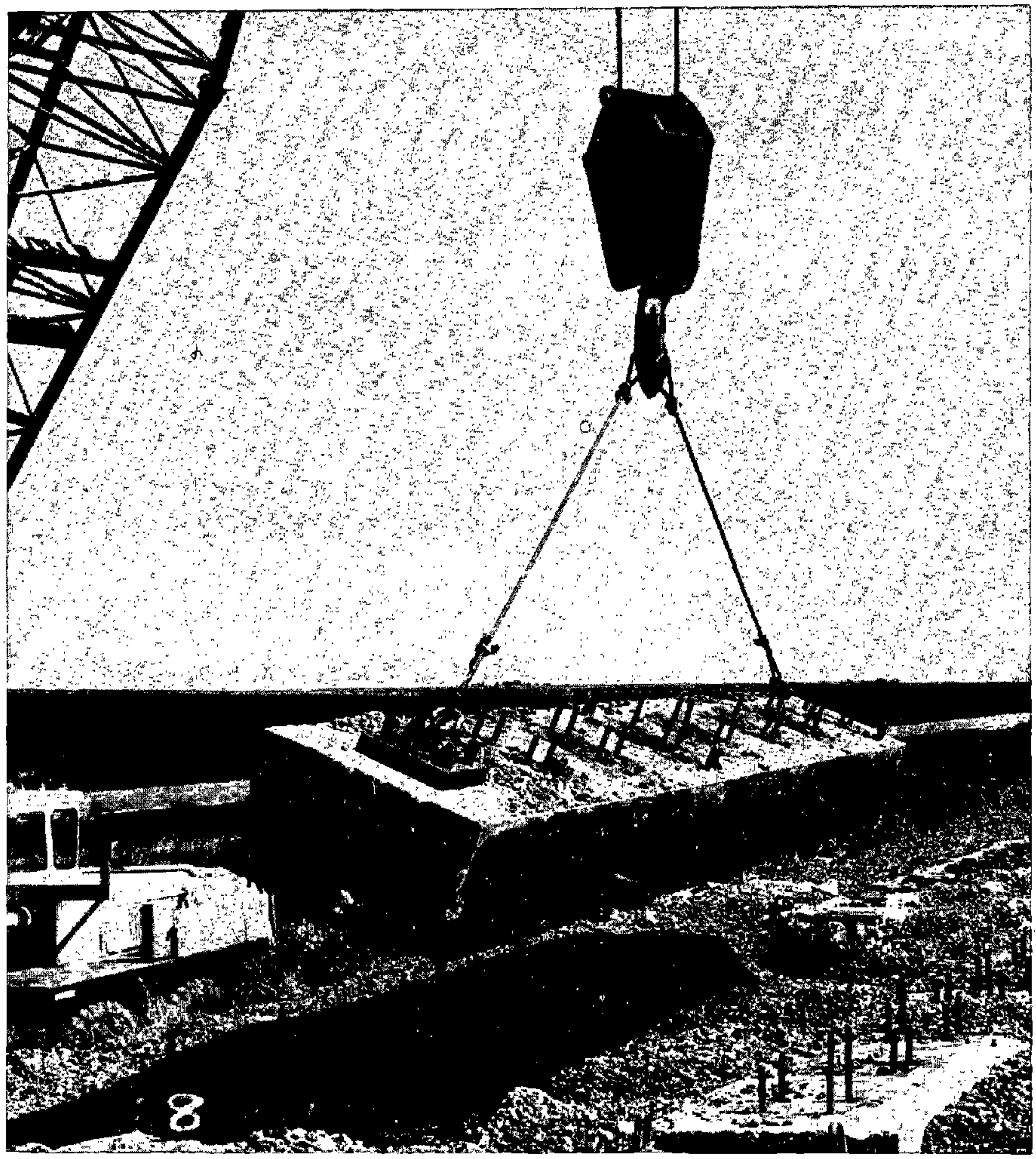


FIGURE 38. REMOVAL OF NONCONVENTIONAL TRACK STRUCTURES FROM TEST LOCATION (TRACK SECTION 7)

TABLE 2. PLATE BEARING TEST RESULTS

Track Section	Location	k pci	Inelastic Response % of Total Response
1	Sta 8524+75 (south rail)	380	62
2A	Sta 8531+62 (south rail)	303	58
2B	Sta 8535+16 (north rail)	317	60
3A	Sta 8542+49 (south rail)	229	60
3B	Sta 8540+20 (north rail)	294	44
4	Sta 8551+50 (north rail)	217	48
5	Sta 8558+20 (center)	88	45
5	Sta 8558+10 (center)	232	43
5	Sta 8562+05 (center)	317	47
7	Sta 8576+41 (south rail)	338	65
8	Sta 8587+08 (north rail)	261	52
9	Sta 8595+33	203	31
	Mean =	265	51

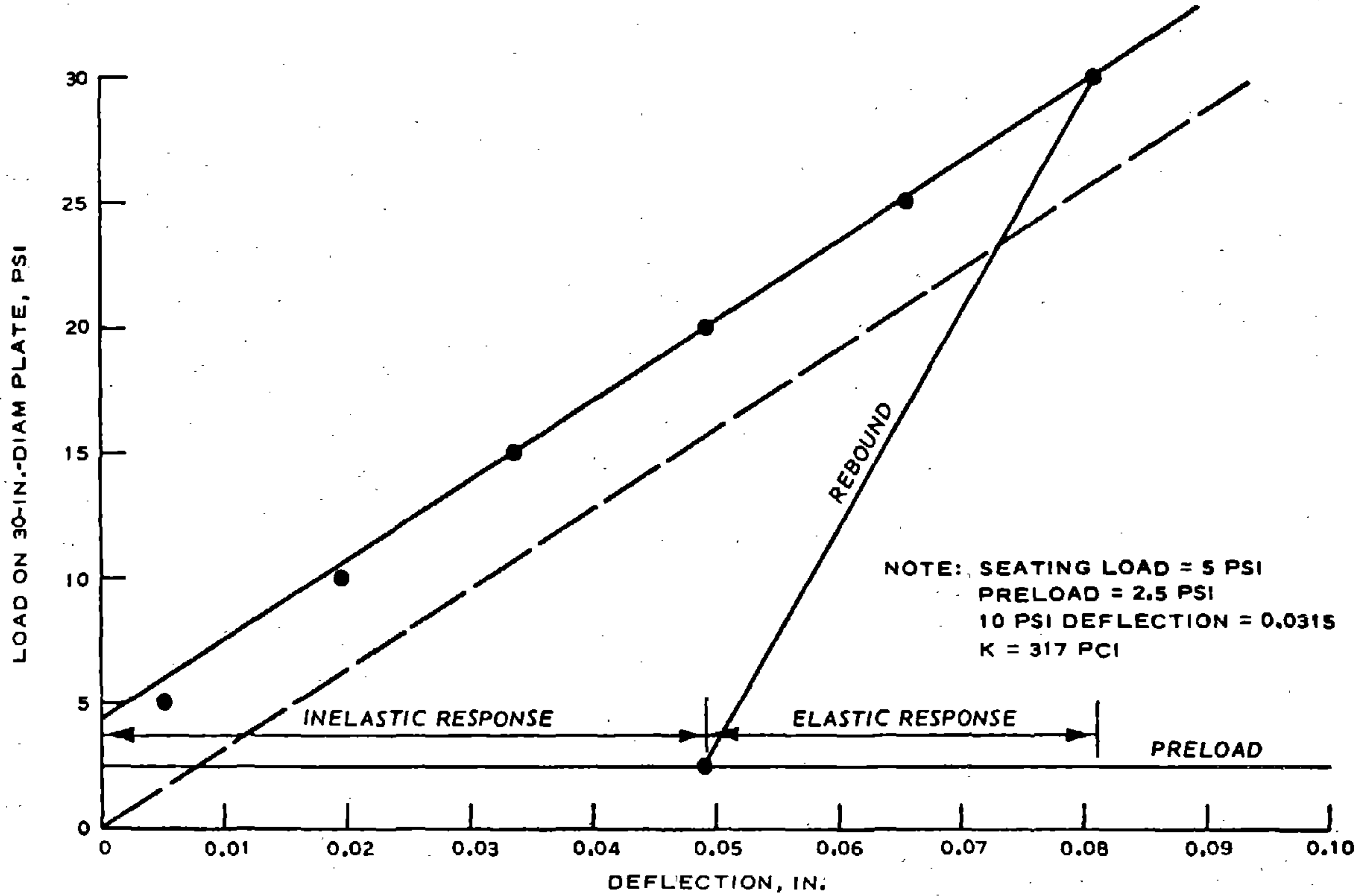


FIGURE 39. TYPICAL LOAD-DEFLECTION CURVE FROM 30-IN.-DIAM PLATE BEARING TESTS ON THE BALLAST BENEATH THE TRACK STRUCTURES, KANSAS TEST TRACK SECTION 2B, CROSSTIE NORTH RAIL

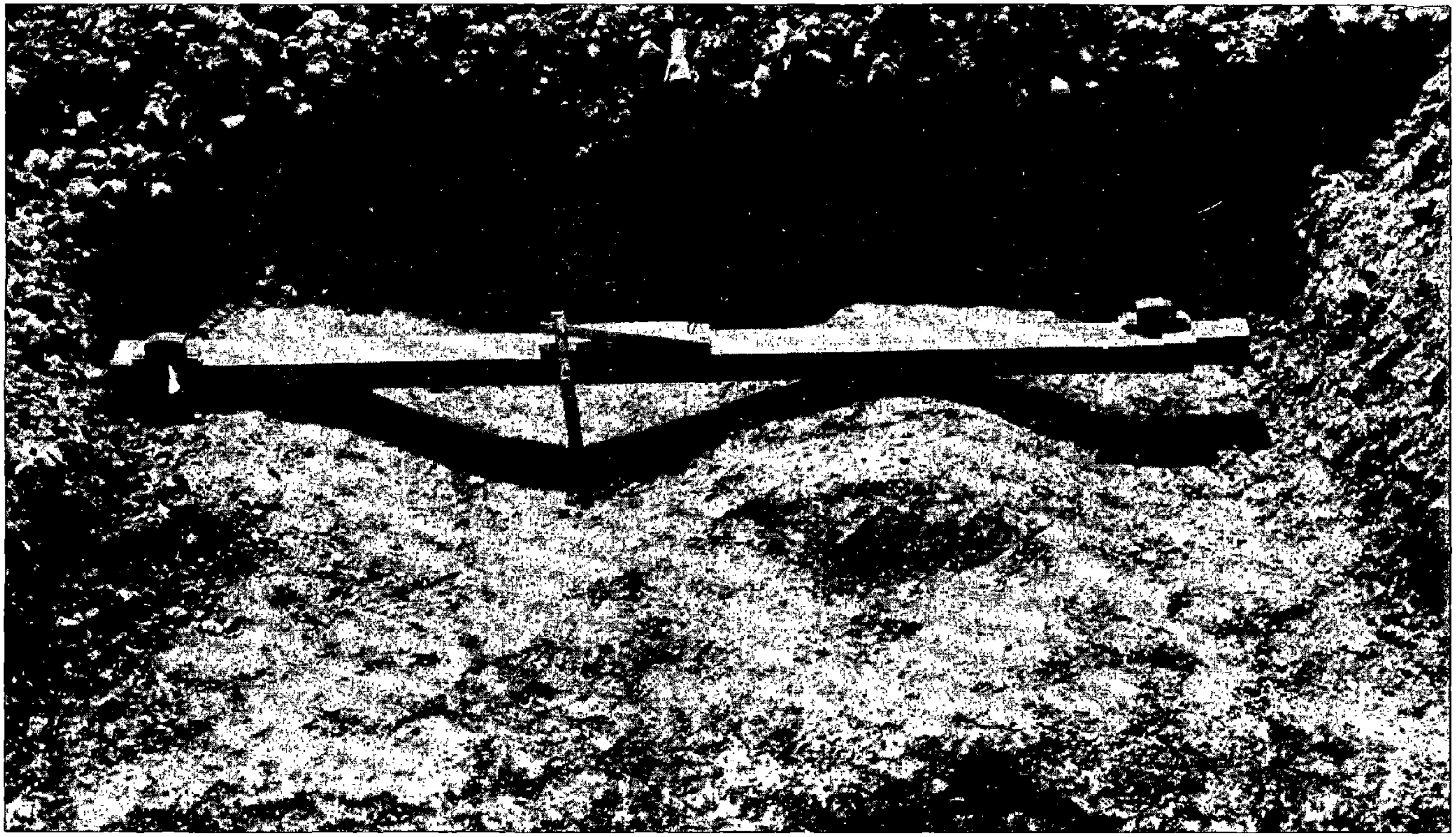


FIGURE 40. SUBGRADE IRREGULARITIES FOUND AT THE BALLAST-SUBGRADE INTERFACE, TEST PIT 1 (TRACK SECTION 1)

rail seats of the ties. An example of the profiles and cross-section data taken at the subgrade surface is shown in Figure 41; these data were obtained in Test Pit 2B, Track Section 2. Maximum absolute and relative deformations of the subgrade in each test pit, from construction and postmortem survey data, are summarized in Table 3. Both absolute (survey) and differential (pit cross section or profile maximum) deformations are shown in Table 3. However, the absolute deformations shown in Table 3 are not believed to be accurate because (1) finished subgrade elevations were established only at four points over the length of the KTT, none of which corresponded with any pit location,[9] and (2) the embankment heaved differentially between 1971 and 1974 and settled differentially under traffic in the period October 1974 through April 1975.[6] Consequently, the beginning subgrade elevation at each test pit is not well defined. The accuracy of vertical deformation measurements is questionable when main array deformations exceeded ±1 in. from mechanical zero, and only the postmortem survey measurements provide detailed profile and cross-section elevations within each test pit area. The postmortem data are presented in more detail in Volume II.

Water content, CBR, and field density tests were conducted at depths of 0, 6, 12, 24, and 36 in. as the test pits were advanced. These tests were patterned to typify conditions at various locations beneath the structures. In Test Pit 1, tests were conducted under two ties and between two ties at each of the five specified depths. Since there was no significant difference in results obtained at similar locations beneath or adjacent to the ties, only one tie and crib area was tested in the remaining test pits in conventional track sections. Summary plots of CBR, water content, and density results for Test Pits 1, 2A, 2B, 3A, 3B, 4, 5, 7, 8, and 9 are shown in Figures 42 through 51, respectively. Figure 52 shows the mean and standard deviation curves for all the CBR, water content, and density data obtained versus depth in the embankment.

Two undisturbed subgrade samples of dimensions 12 by 12 by 12 in. each were obtained from each test pit for later laboratory testing. One sample was taken from 0 to 12 in. below the ballast-subgrade interface;

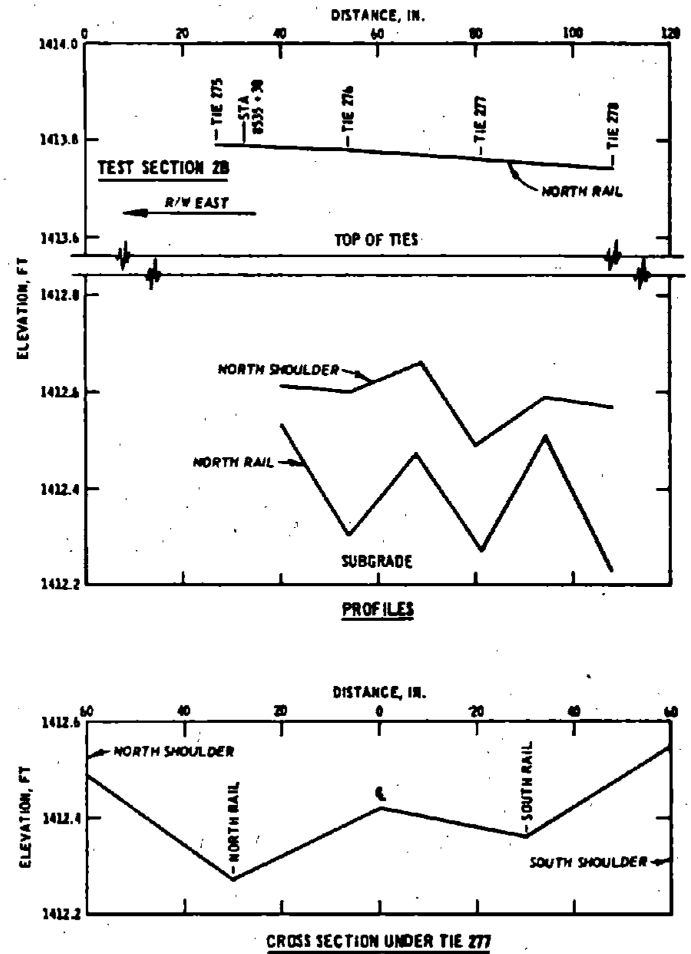


FIGURE 41. SURVEY CROSS SECTION AND PROFILES OBTAINED AT THE BALLAST-SUBGRADE INTERFACE, TEST PIT 2A (TRACK SECTION 2)

TABLE 3. MAXIMUM VERTICAL DEFORMATIONS UNDER THE TRACK STRUCTURES (MEASURED AT THE BALLAST-SUBGRADE INTERFACE)

Track Section	As-Built to Postmortem Deformation, ft*	Differential Deformation Within Pit Location, ft
1	0.10	0.26
2A	0.44	0.17
2B	0.50	0.34
3A	0.15	0.24
3B	0.16	0.22
4	0.08	0.17
5	+0.06	0.09
7	+0.03	0.24
8	0.21	0.24
9	0.04	0.30

* Absolute measurements are not believed to be accurate. See text.

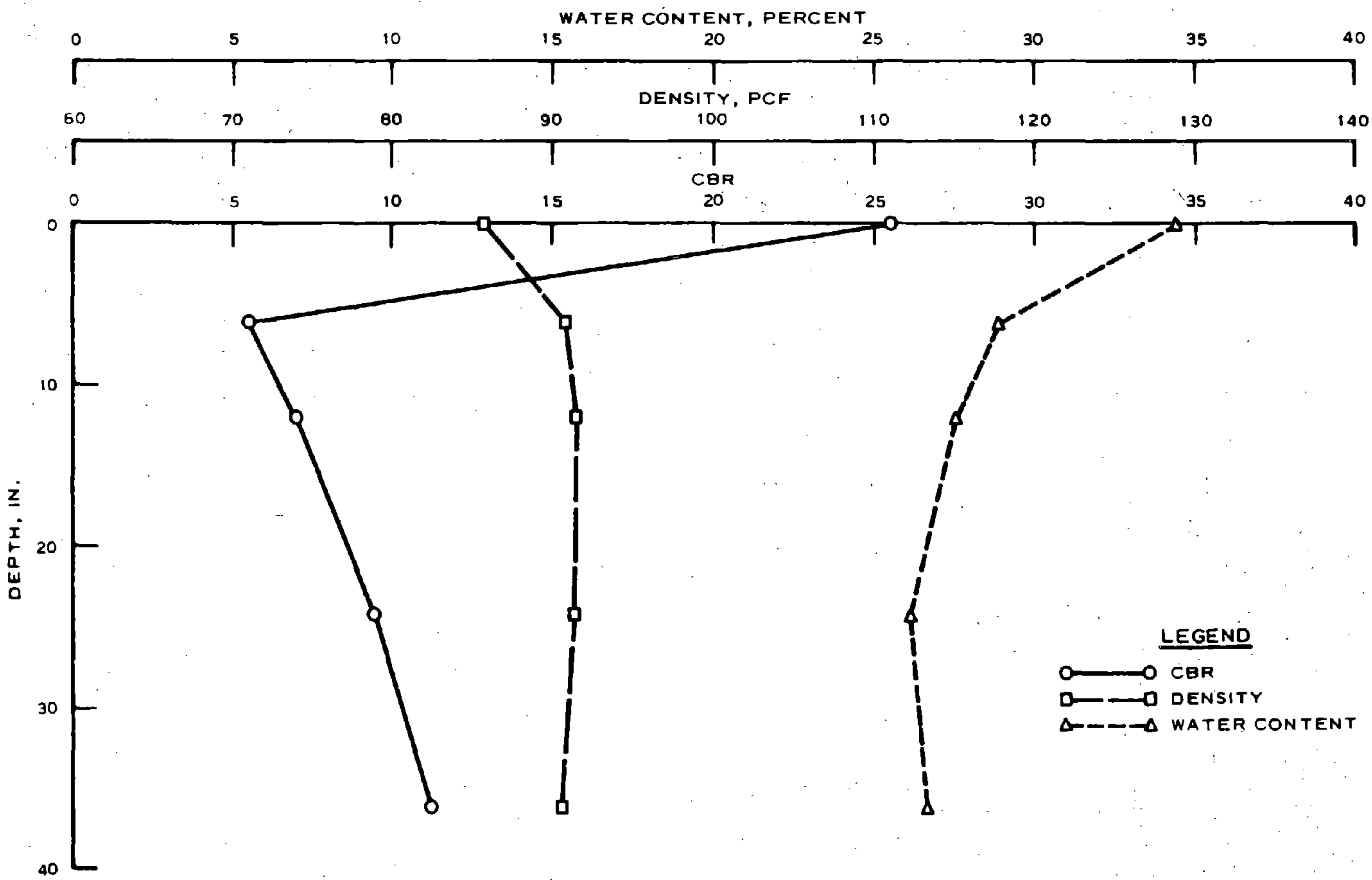


FIGURE 42. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 1 (TRACK SECTION 1)

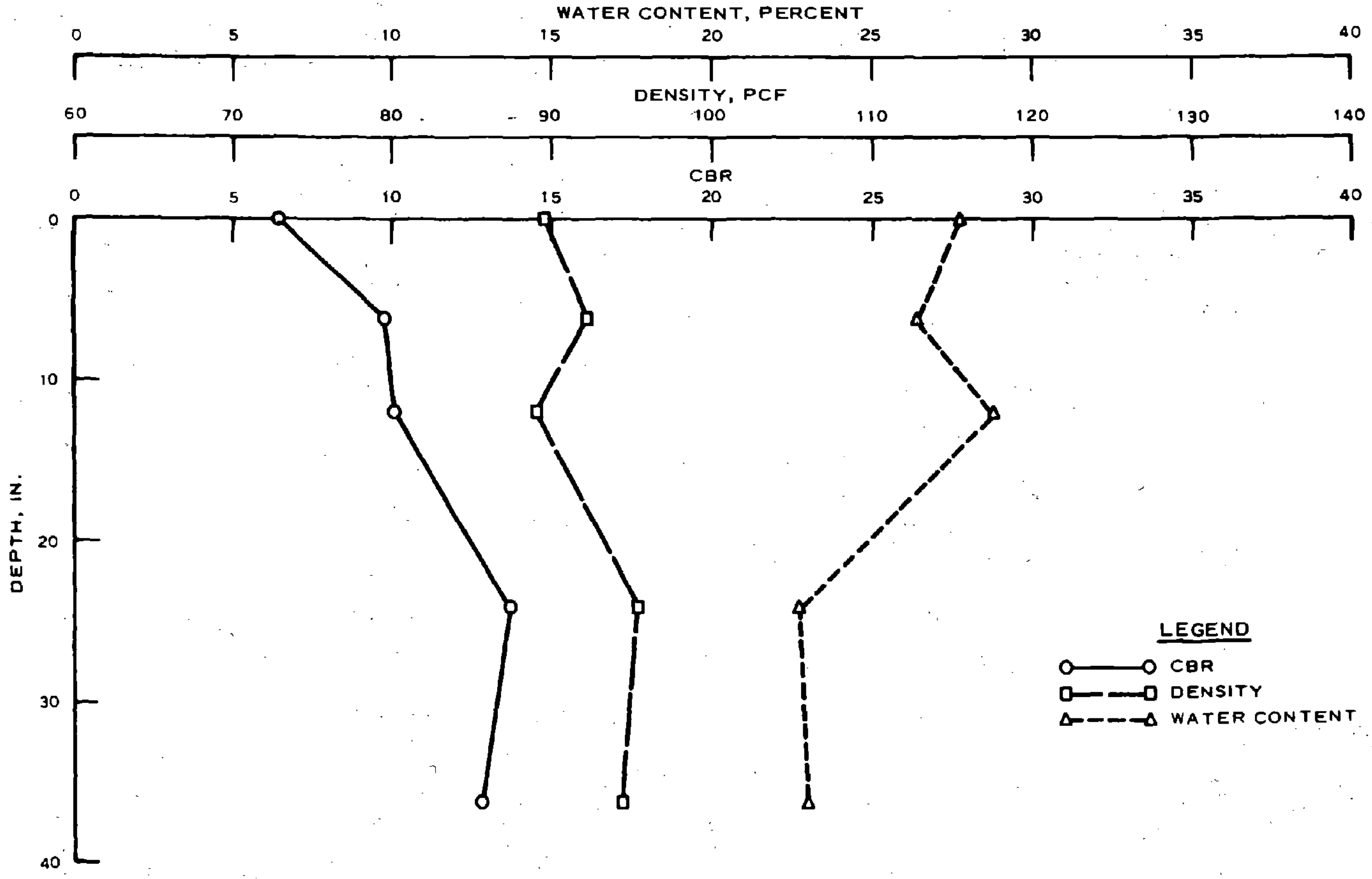


FIGURE 43. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 2A (TRACK SECTION 2)

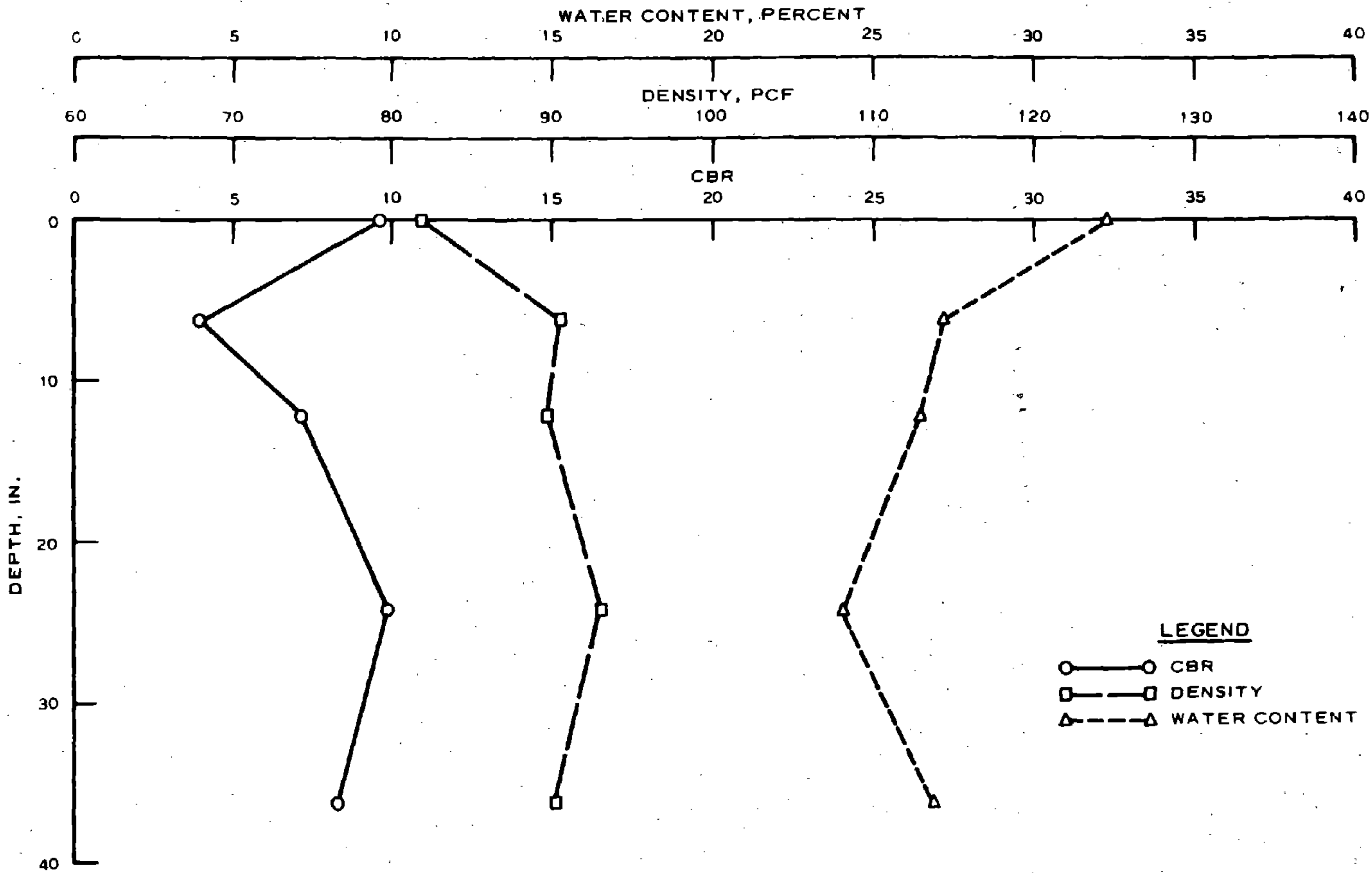


FIGURE 44. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 2B (TRACK SECTION 2)

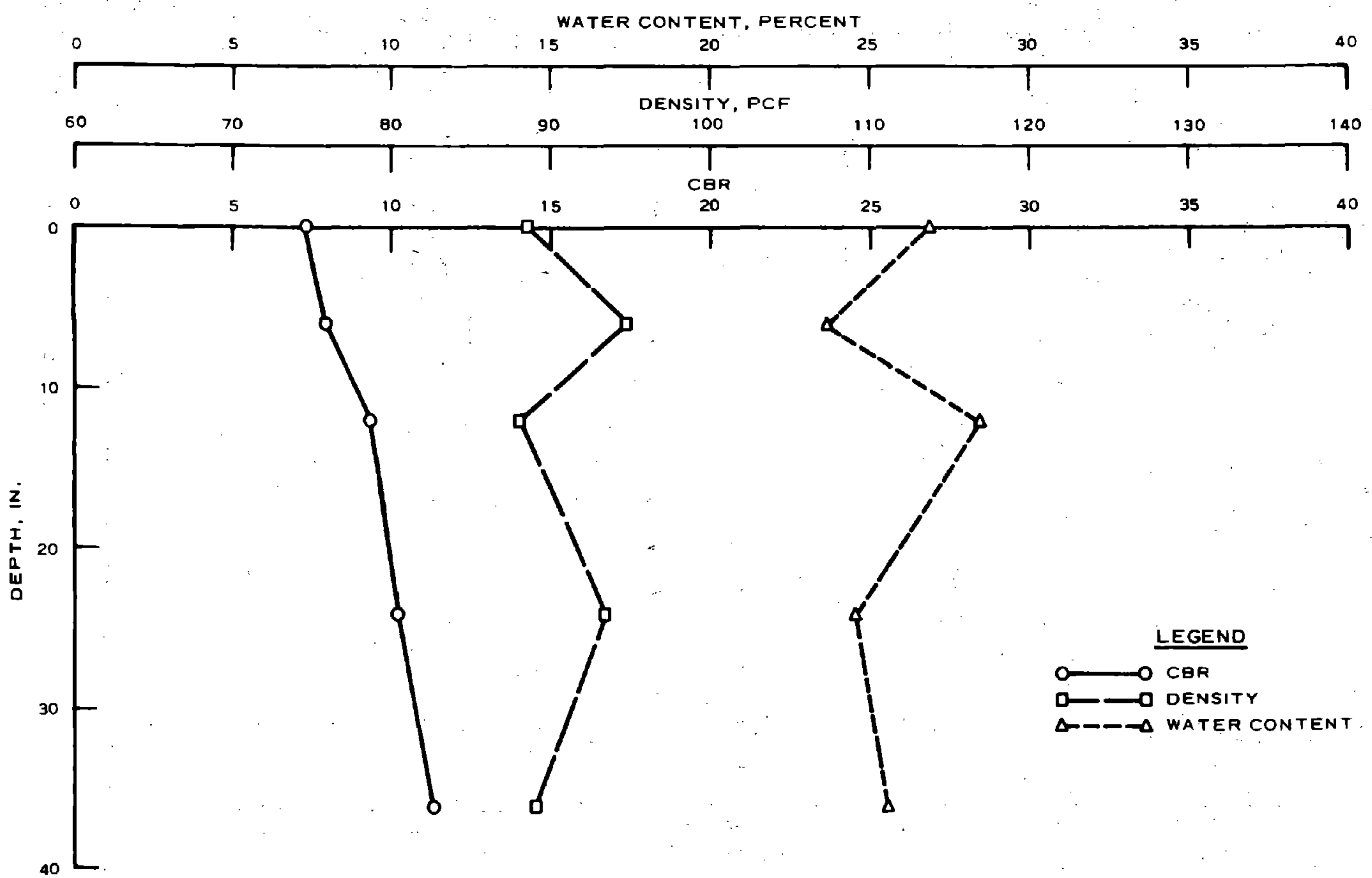


FIGURE 45. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 3A (TRACK SECTION 3)

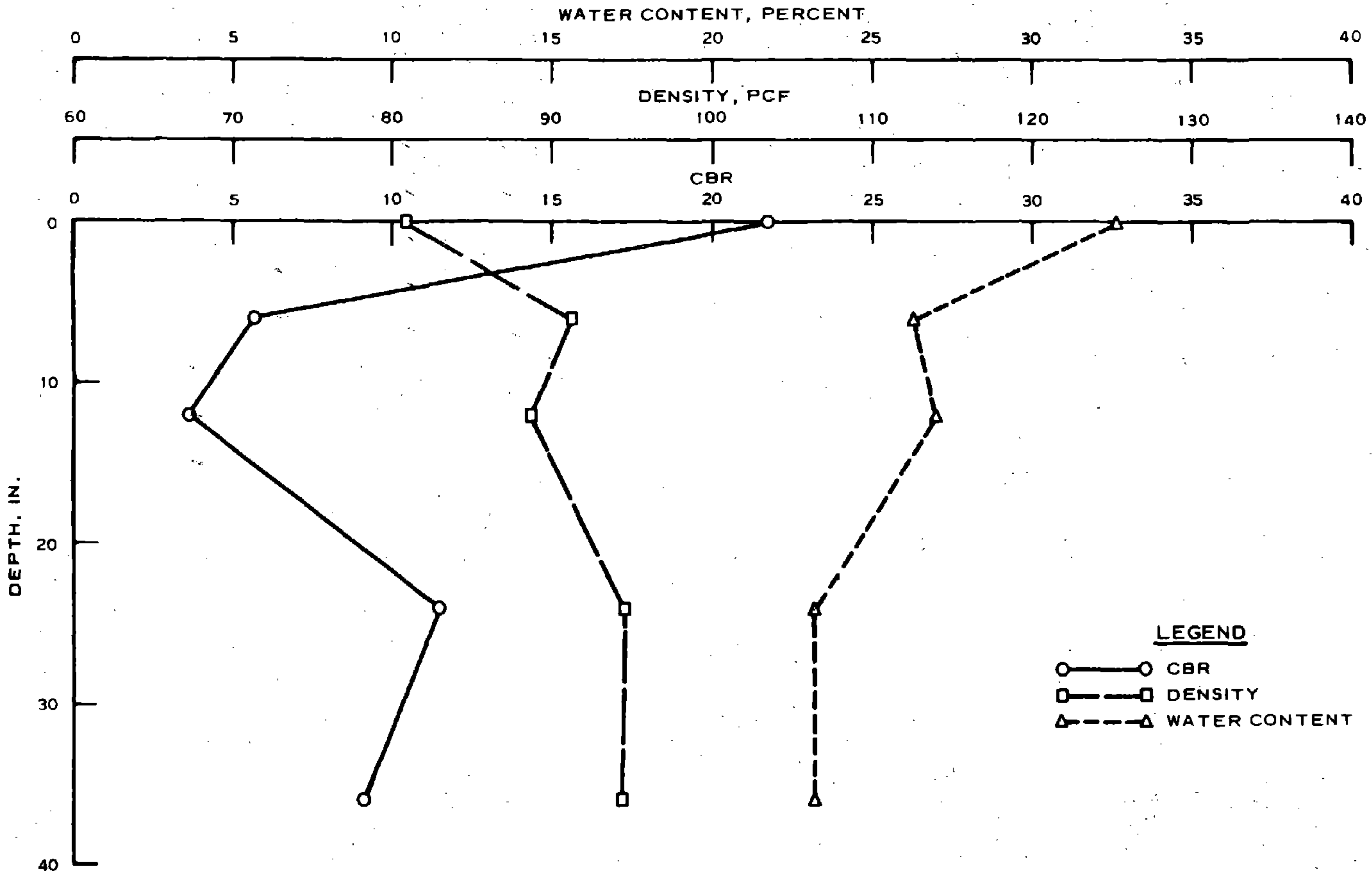


FIGURE 46. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 3B (TRACK SECTION 3)

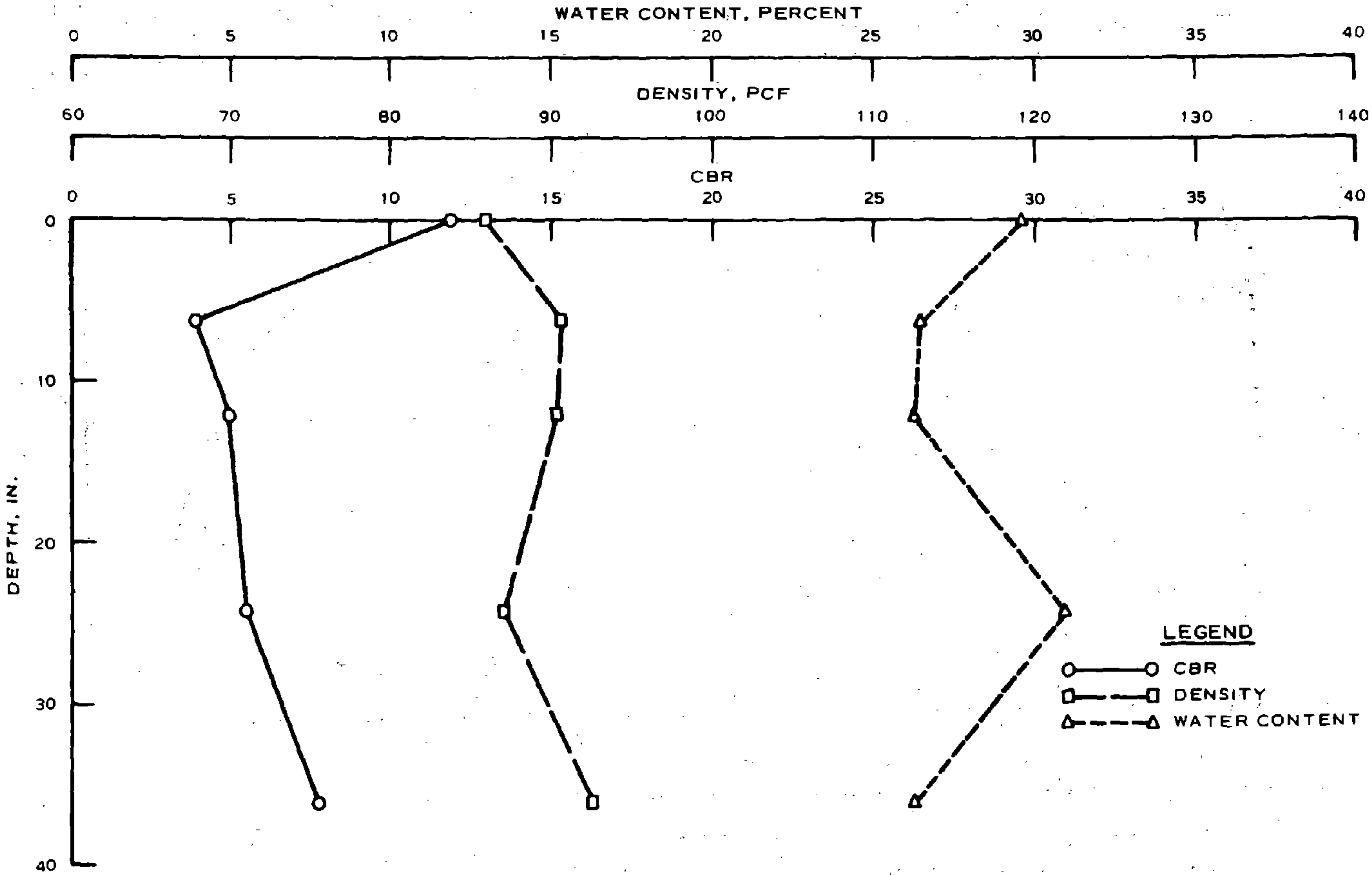


FIGURE 47. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 4 (TRACK SECTION 4)

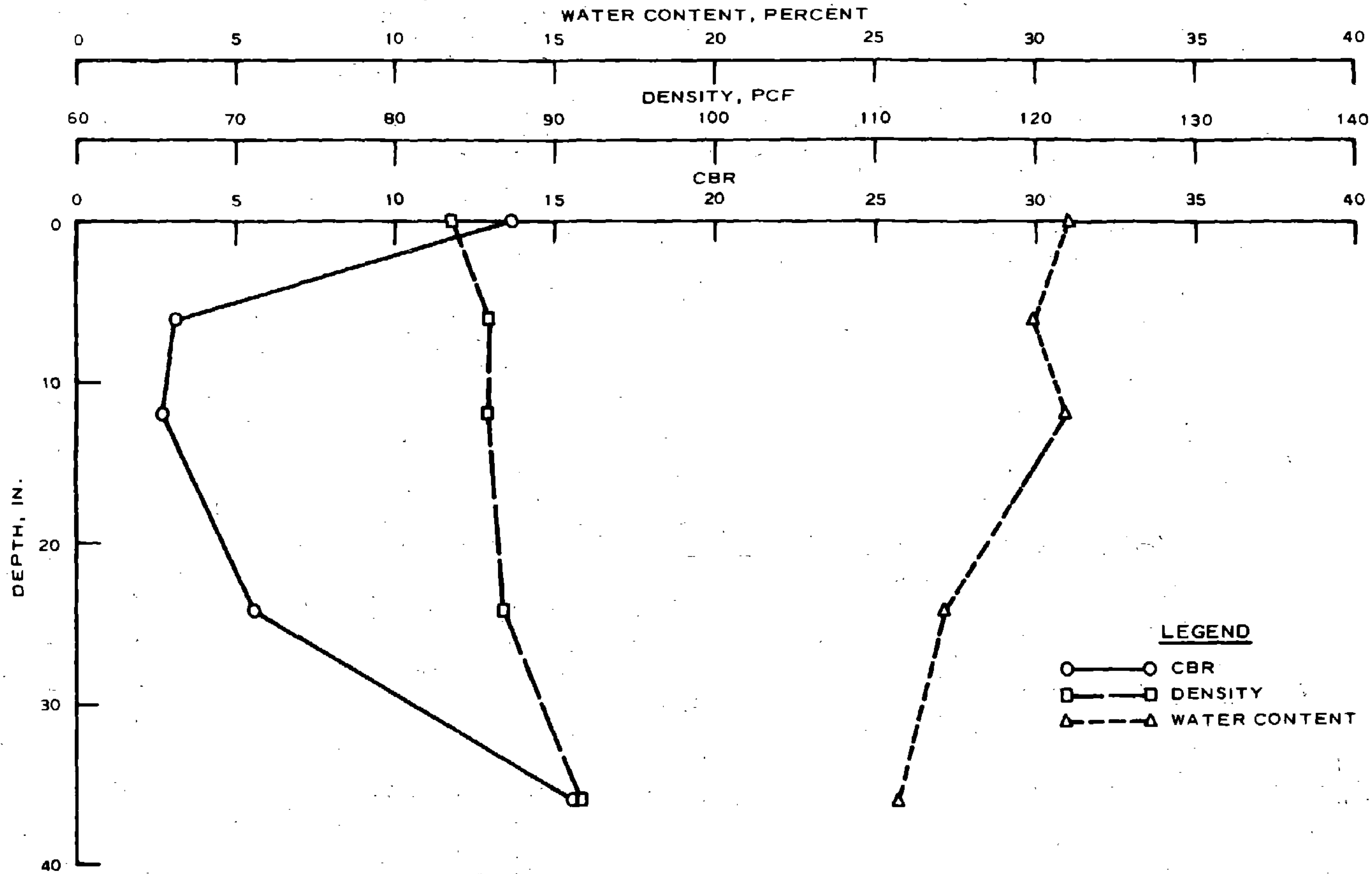


FIGURE 48. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 5 (TRACK SECTION 5)

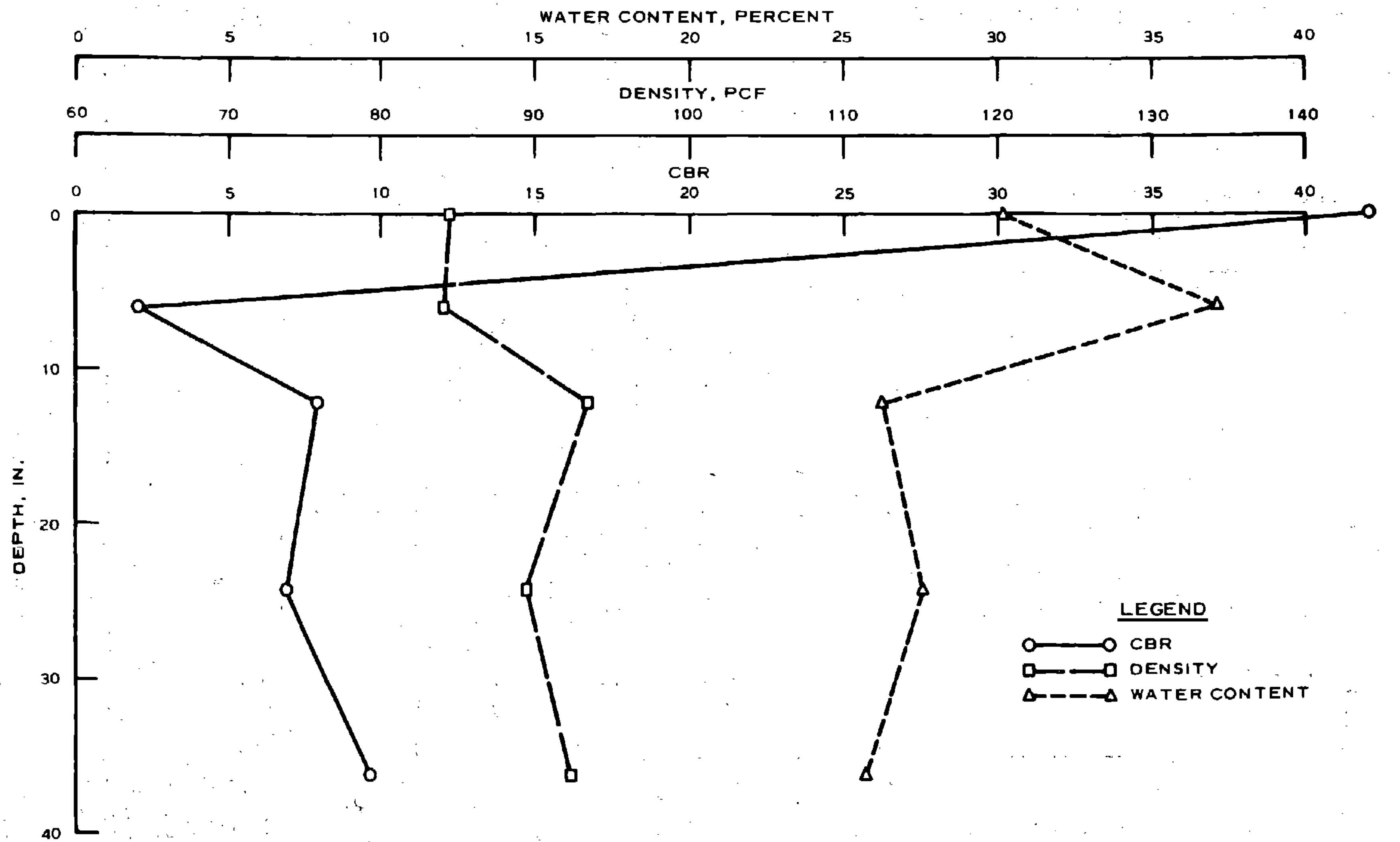


FIGURE 49. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 7 (TRACK SECTION 7)

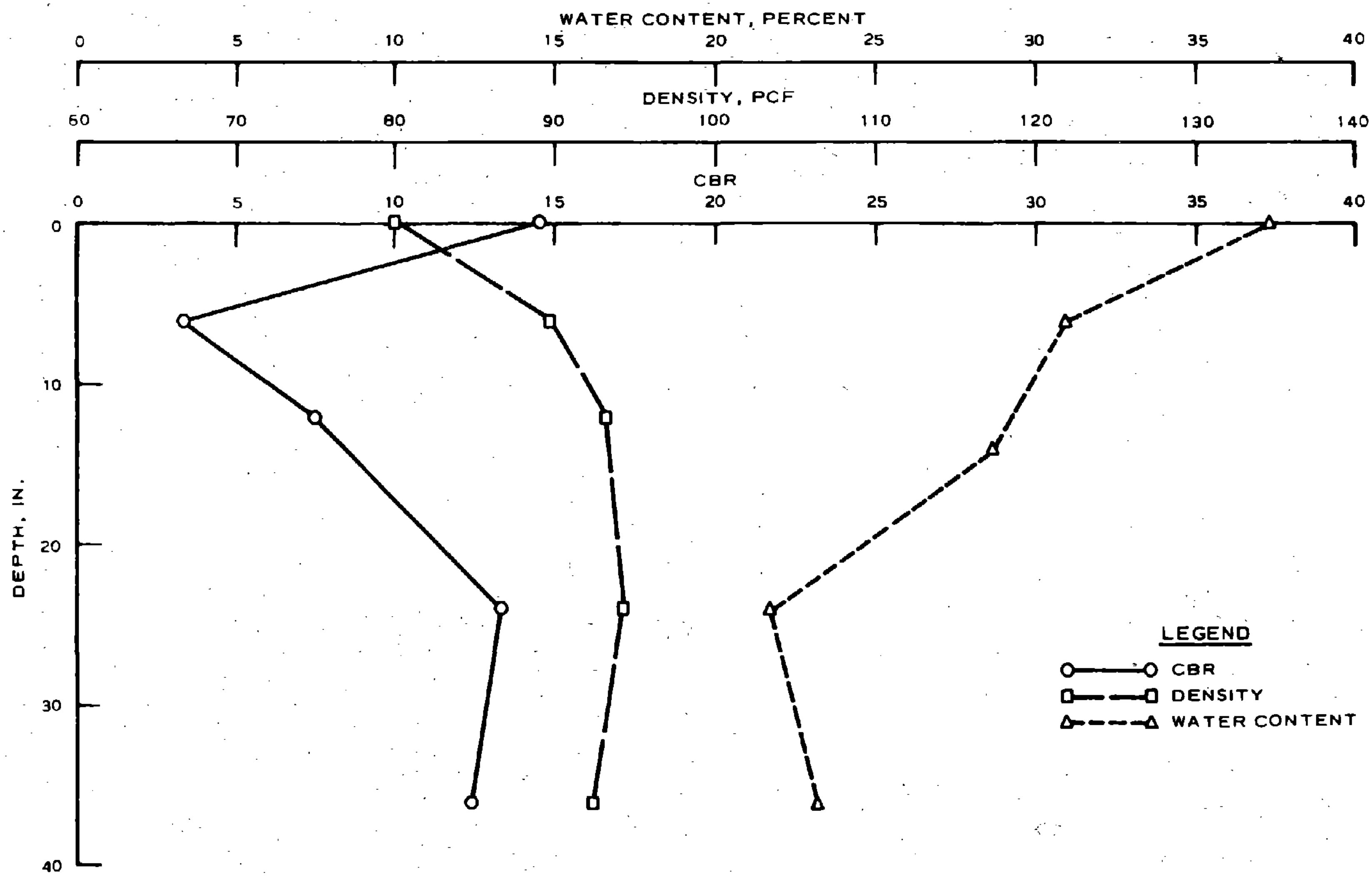


FIGURE 50. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 8 (TRACK SECTION 8)

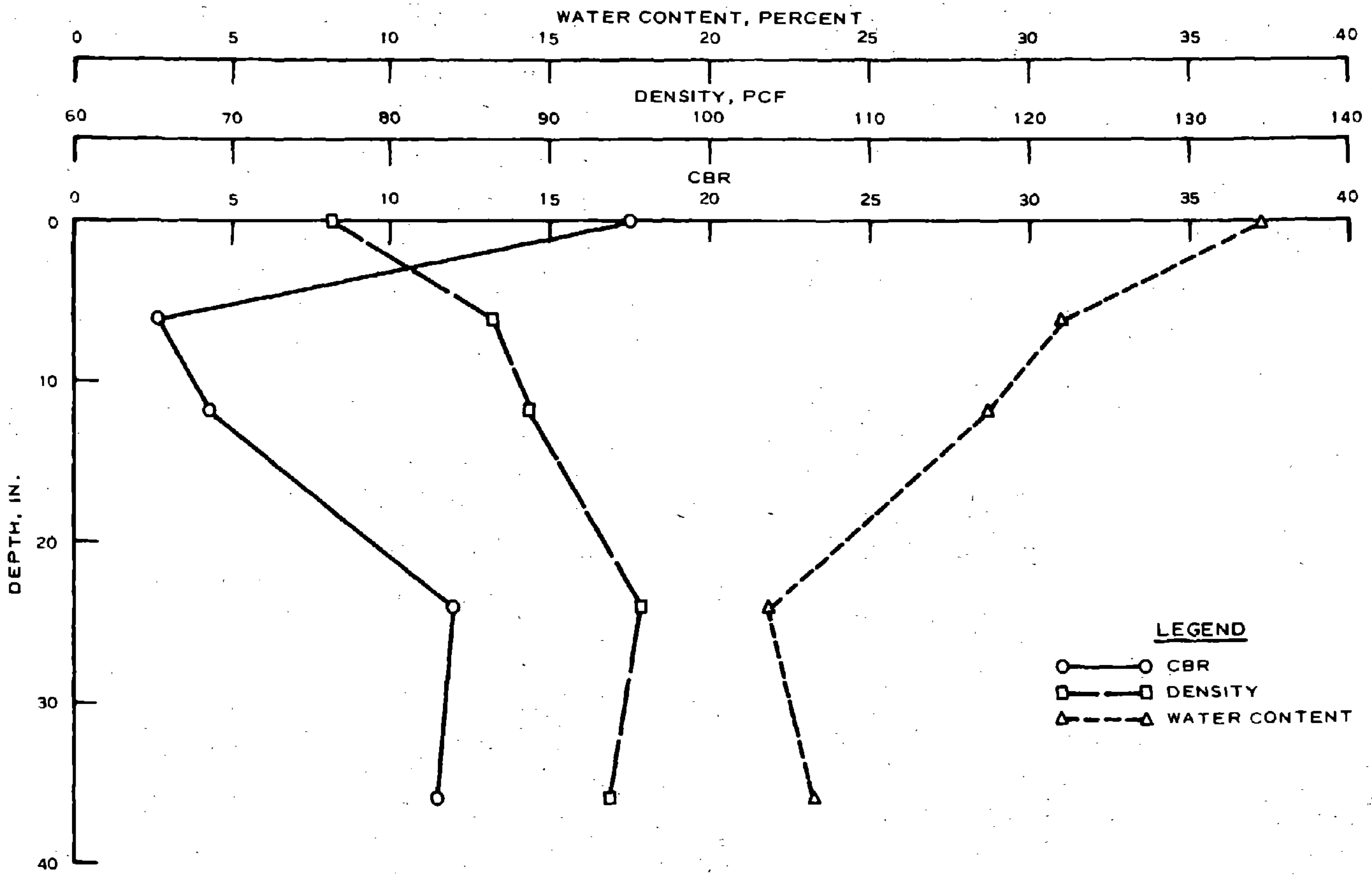


FIGURE 51. AVERAGE WATER CONTENT, DRY DENSITY, AND CBR TEST RESULTS VERSUS DEPTH IN THE EMBANKMENT, TEST PIT 9 (TRACK SECTION 9)

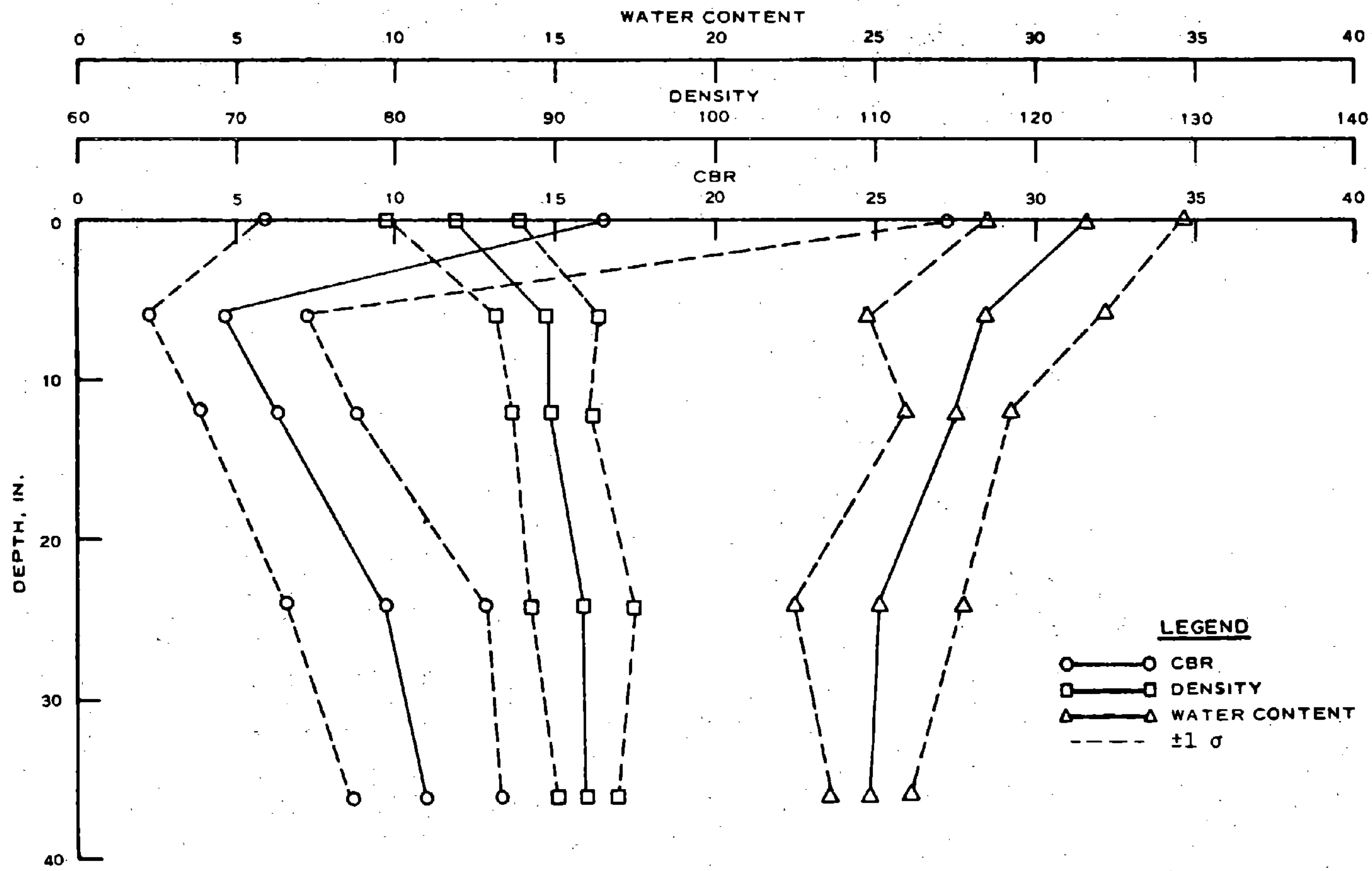


FIGURE 52. MEAN AND STANDARD DEVIATION CURVES FOR AVERAGE WATER CONTENT, DRY DENSITY, AND CBR DATA FROM ALL TEST PITS VERSUS DEPTH IN THE EMBANKMENT

and the other sample was taken from 12 to 24 in. in depth from that interface. These samples were sealed into wood boxes with wax. Disturbed (bag) samples were also obtained for laboratory material property tests. Laboratory results are described in a following section.

Embankment Instrumentation

The embankment instrumentation investigation was carried out by an electronics engineer. In the first phase of this work, a field check was made to determine insulation values for the pressure, moisture, temperature, and main array vertical extensometer circuitry. Figure 53 shows this work in progress. Typical results of the field insulation checks are summarized in Table 4. In general, 100 megohms or higher values of insulation resistance to ground are desired for relatively noise-free electrical measurements. Usable electronic signals can, of course, be obtained with insulation values less than optimum; but below 10 megohms insulation resistance, the signal to noise ratio decreases rapidly and noise levels tend to affect measurement precision. Approximately 50 percent of the insulation values recorded for the vertical extensometers and pressure cells in Track Section 2 were less than 10 megohms, as shown in Table 4. Circuits of the moisture-temperature cells connect through the sensor to ground, so relatively low values of insulation are to be expected when readings are made at the cable terminals. Final readings of the vertical extensometers were also made, and these readings are tabulated in Table 5.

The next phase in the field work involved excavation of the embankment instrumentation in Test Pits 2B, 7, and 9. Time and funding constraints limited this effort to three pits and to a depth of 3 ft into the embankment. However, these excavations exposed the bulk of the main array instrumentation, including the vertical extensometer heads, upper and lower soil pressure cells, moisture-temperature gages, and associated wiring, as shown in Figure 54. Conditions in each pit were similar and are summarized as follows:

FIGURE 53. FIELD CHECK OF MAIN ARRAY INSTRUMENTATION READOUTS

TABLE 4. TYPICAL RESULTS OF FIELD INSULATION MEASUREMENTS
INSULATION TO GROUND TEST SECTION 2, 20 MAY 1976

Gage	A	B	C	D	E	F	G	H	J	K	L	M	N	P	R	S	T	U	V	W	X	Y	Z	α	Connector
2401-1	>20M	>20M	>20M	>20M	>20M																				1
2401-2						1.5M	1.5M	700K	700K	700K															1
2401-4																100K	120K	120K	120K	285K					1
2402-1	20M	7M	20M	20M	1M																				2
2402-2						20M	720K	800K	800K	720K															2
2402-4																>20M	>20M	>20M	>20M	>20M					2
2403-1	20M	20M	20M	20M	20M																				3
2403-2						>20M	>20M	>20M	>20M	>20K															3
2403-4																1M	1M	500K	500K	230K					3
2201	7M	5M	3M	>20M	>20M	5M																			4
2202							7M	7M	350K	720K	20M	120K													4
2203													20M	20M	20M	20M	20M	20M							4
2301	10K	5K	8K																						5
2302				19K	15K	19K																			5
2303							20K	17K	19K																5
2304										17K	14K	14K													5
2305													18K	14K	15K										5
2306																70K	90K	40K							5
2307																			30K	26K	30K				5
2308																						30K	13K	17K	5
2309	15K	15K	12K																						6
2310				80K	100K	3M																			6
2311							40K	25K	40K																6
2312										40K	25K	40K													6
2313													15K	10K	15K										6

K = 1,000 ohms
M = 1,000,000 ohms

TABLE 5. FINAL EXTENSOMETER READINGS

Gage Location	TR100 Indication	Gage Sensitivity in $MV/V/IN^4$	Deflection in inches	Date
2401-1	1.25L*	382.3	1.15L	5/20/76
2401-2	1.10L	378.2	1.02L	5/20/76
2401-4	1.25L	372.3	1.18L	5/20/76
2402-1	1.45L	379.2	1.34L	5/20/76
2402-2	1.40L	379.0	1.30L	5/20/76
2402-4	1.30L	381.2	1.20L	5/20/76
2403-1	0.55R**	379.3	0.51R	5/20/76
2403-2	0.00	377.5	0.00	5/20/76
2403-4	0.03R	379.0	0.028R	5/20/76
7401-1	0.82L	377.7	0.76L	5/18/76
7401-2	0.90L	378.0	0.841L	5/18/76
7401-4	1.20L	375.8	1.12L	5/18/76
7402-1	0.50R	380.0	0.46L	5/18/76
7402-2	1.50L	374.2	1.41L	5/18/76
7402-4	0.045L	377.3	0.042L	5/18/76
7403-1	0.27L	379.8	0.25L	5/18/76
7403-2	0.55L	374.0	0.516L	5/18/76
7403-4	1.10L	378.7	1.02L	5/18/76
9401-1	1.05L	377.3	0.97L	5/19/76
9401-2	0.95L	380.2	0.88L	5/19/76
9401-3	0.55L	377.3	0.51L	5/19/76
9401-4	0.45L	376.5	0.42L	5/19/76
9402-1	1.35L	372.8	1.27L	5/19/76
9402-2	1.25L	376.3	1.17L	5/19/76
9402-3	1.30L	381.0	1.20L	5/19/76
9402-4	1.30L	374.0	1.22L	5/19/76
9403-1	0.30R	379.2	0.28R	5/19/76
9403-2	0.60R	375.8	0.56R	5/19/76
9403-3	0.90R	370.5	0.85R	5/19/76
9403-4	1.45R	378.0	1.35R	5/19/76

* L - down
** R - up

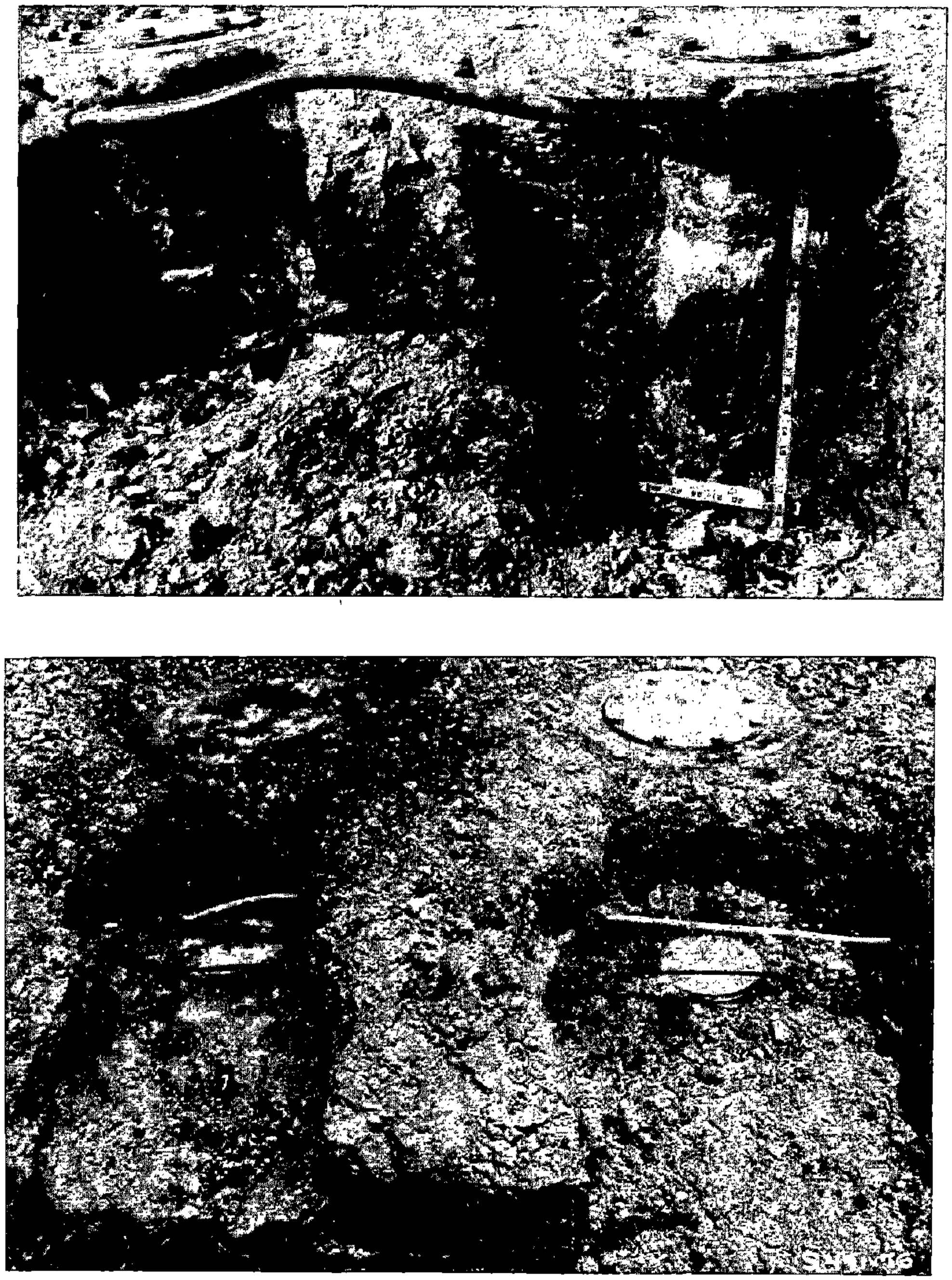


FIGURE 54. TYPICAL EXCAVATION OF MAIN ARRAY INSTRUMENTATION, TEST PIT 7 (TRACK SECTION 7)

1. Heads of the vertical deflectometers were found to be tilted, sometimes severely so.
2. The upper pair of deflectometer anchor prongs appeared to be well seated and in intimate contact with the surrounding soil.
3. In every case the surfaces of the main array soil pressure cells appeared to be in intimate contact with the soil.
4. The excavated moisture-temperature cells were plated to a dark color, as if they had been anodized.

Figure 55 illustrates the extensometer tilt observed in Test Pit 9, Track Section 9. In this example the extensometer was not centered beneath the crosstie, and tilting resulted. Tilting usually occurred when the large (16-in.-diam) extensometer heads followed sharp differential settlements in the KTT subgrade. The tilting phenomenon was also noticed in excavations to expose the Carlson pressure cells installed by PCA (just beneath the ballast-subgrade interface). A typical example of this condition is shown in Figure 56; however, studies of these instruments were not included in the current scope of work. Table 6 summarizes the internal mechanical condition of the vertical extensometers, and Figure 57 illustrates the internal damage which typically resulted from overranging as the subgrade settled. In Figure 57, note the broken fittings which had been attached to the LVDT rods. In some instances, mud and/or water was found inside the heads, presumably caused by infiltration.

Calibration valves for the LVDT's in the main array vertical extensometers of Track Sections 2, 7, and 9 were also verified in the field. Results of the field verification are shown in Table 7.

The calibration history of the soil pressure cells was also researched; results of this study are presented in Table 8. Detailed descriptions of the test procedures and equipment used onsite and in the limited series of verification tests carried out at the WES may be found in Appendix E, Volume II, of this report.

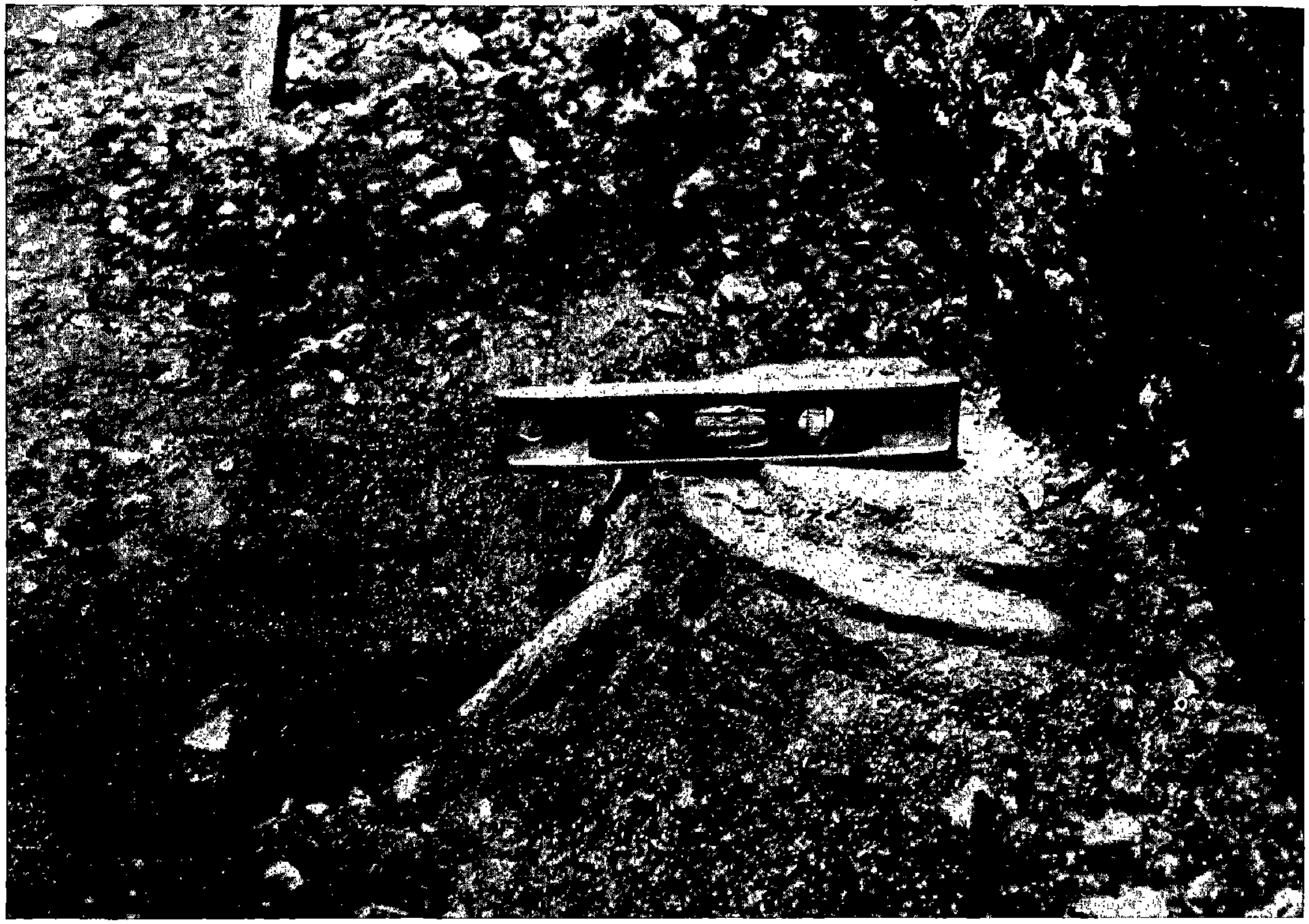


FIGURE 55. VERTICAL EXTENSOMETER TILT OBSERVED IN TEST PIT 9 (TRACK SECTION 9)

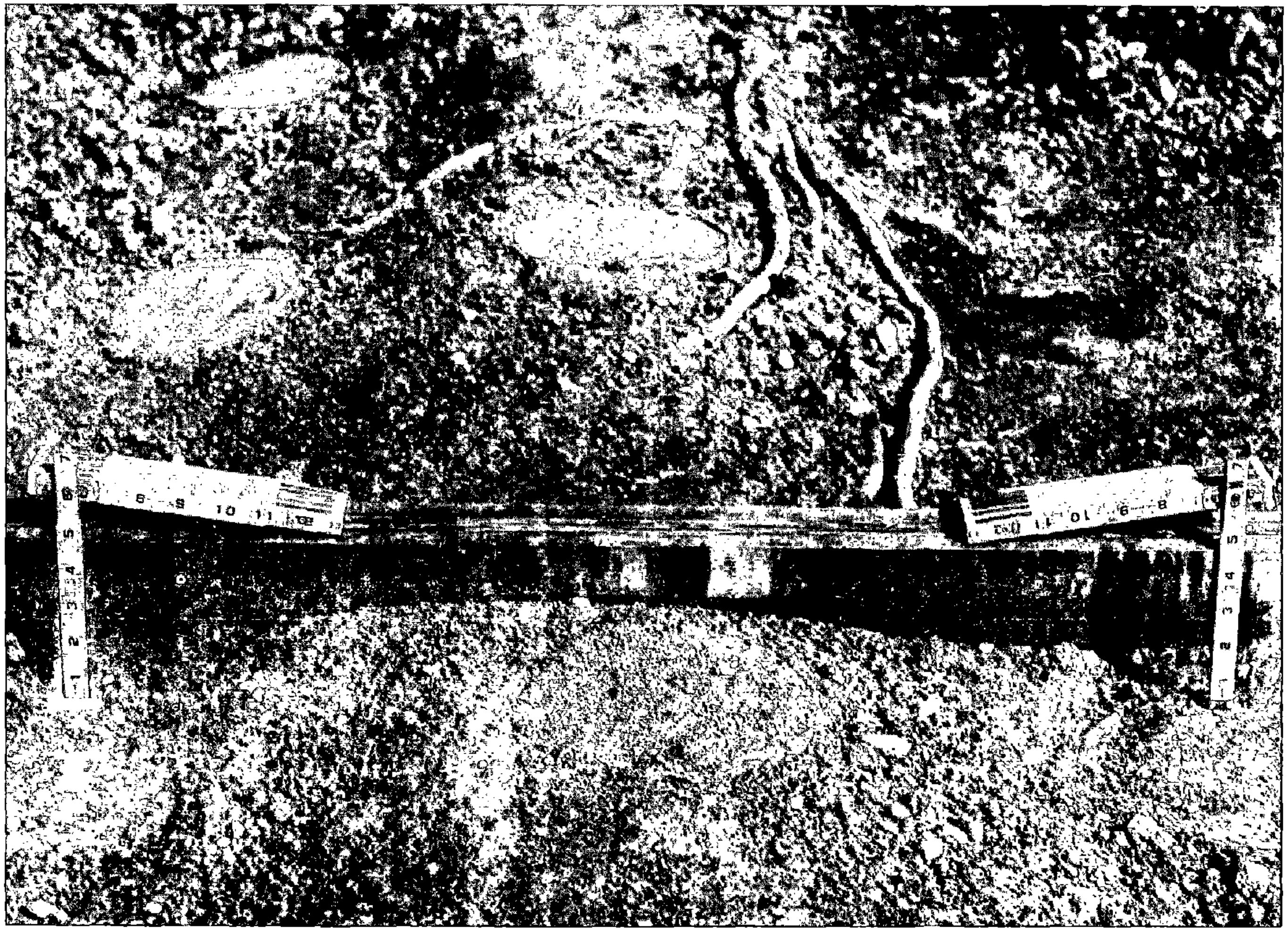


FIGURE 56. CONDITION OF PCA PRESSURE CELLS (LOCATION JUST BENEATH THE BALLAST SUBGRADE INTERFACE, TRACK SECTION 3)

TABLE 6. POSTMORTEM CONDITION OF EXTENSOMETER LVDT ASSEMBLY

Transducer Location	Anchor Support Broken	LVDT Core Extension Rod			Leaks/Pumping		Core Resting on Bottom
		Bent	Jammed	Broken	Water	Mud	
1401-1					X		
1401-2					X		
1401-4					X		
1402-1	X						
1402-2	X						
1402-4	X						
2401-1			X	X			X
2401-2	X						X
2401-4							X
2402-1			X	X	X	X	
2402-2	X	X	X		X	X	
2402-4	X	X			X	X	
3401-1					X		
3401-2	X				X		
3401-4					X		
3402-1	X				X		
3402-2	X				X		
3402-4	X				X		
3403-1							
3403-2							
3403-4							
6401-1							
6401-2							
6401-3							
6401-4							
6402-1	X				X		
6402-2					X		
6402-3	X				X		
6402-4	X				X		
7401-1							
7401-2							
7401-4							
7402-1					X	X	
7402-2					X	X	
7402-4	X		X		X	X	
8401-1							
8401-2							
8401-4							
8402-1					X		
8402-2					X		
8402-4					X		
9401-1							
9401-2			X				
9401-4							
9402-1					X		
9402-2	X				X		
9402-3					X		
9402-4					X		

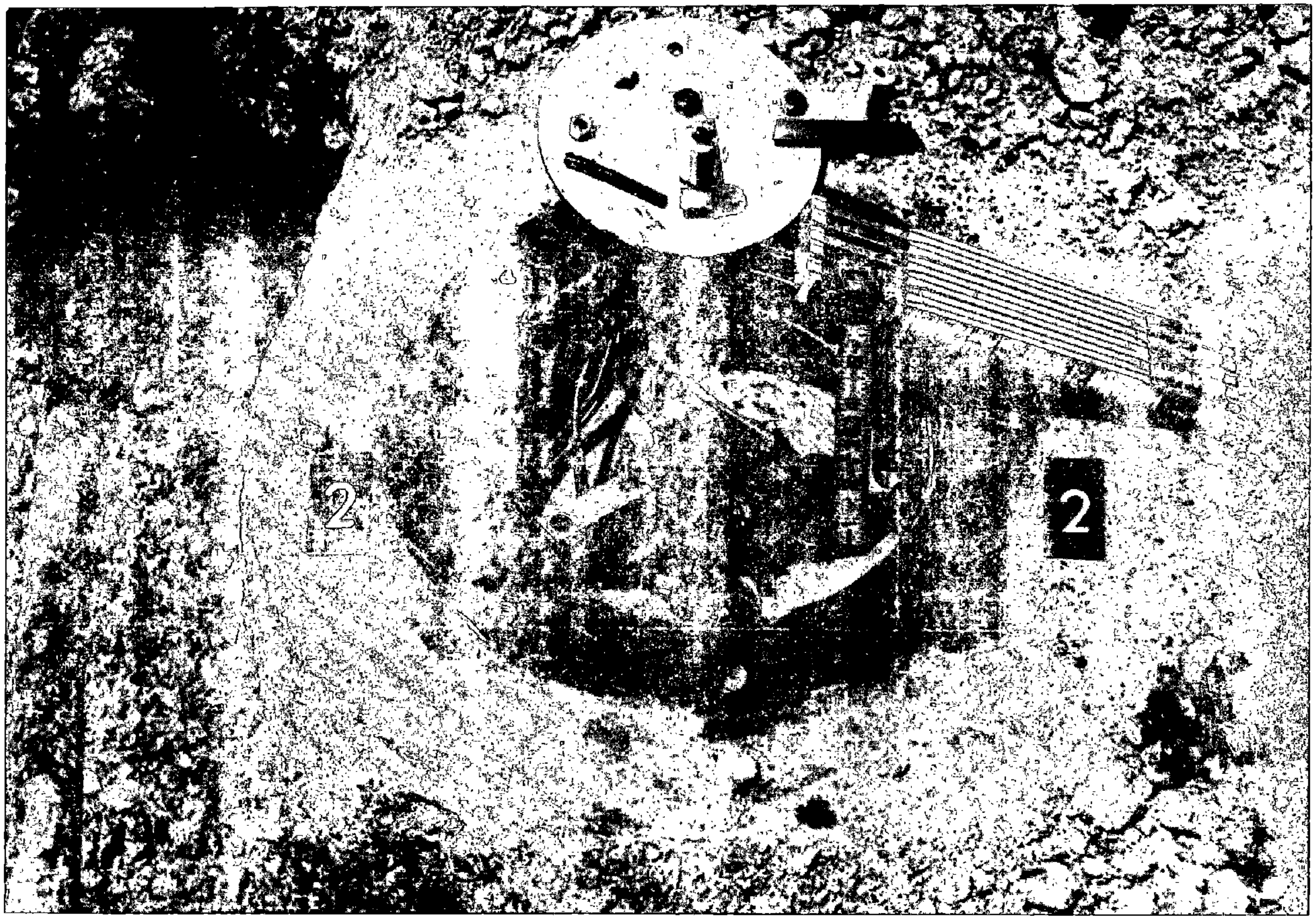


FIGURE 57. EXAMPLE OF INTERNAL DAMAGE TO MAIN ARRAY VERTICAL EXTENSOMETER (TRACK SECTION 2)

TABLE 7. EXTENSOMETER IN SITU CALIBRATION RESPONSE TO TO 1-IN. DISPLACEMENT, 20 MAY 1976

Gage	TR-100 Indication	LVDT Sensitivity* mv/v/in.	Displacement in inches
2402-1**			
2401-2	1.1R (up)	378.2	1.02R
2401-4	0.97R	372.3	0.91R
2402-1**			
2402-2**			
2402-4	0.95R	379.0	0.88R
7401-1†			
7401-2	1.05R	378.0	0.98R
7401-4†			
7402-1	0.97R	380.0	0.98R
7402-2	1.05R	374.0	0.98R
7402-4	1.05R	377.3	0.98R
9401-1	1.05R	377.3	0.98R
9401-2	1.05R	380.2	0.97R
9401-3	1.05R	377.3	0.98R
9401-4††	1.5R	376.5	1.4R
9402-1**			
9402-2	1.06R	376.3	0.99R
9402-3	1.05R	381.0	0.97R
9402-4	1.05R	374.0	0.99R

* See Reference 3.
** Rod jammed tube.
† Could not loosen LVDT core from reference assembly.
†† This calibration was done twice.

TABLE 8. CALIBRATION HISTORY OF MAIN INSTRUMENTATION ARRAY PRESSURE CELLS

Cell Number	Range in psia	Sensitivity in mv/v/psi		
		Slope Indicator Company Calibration*	WES Calibration	
			0 ft of cable	30 ft of cable
2201	25	8.667	**	**
2202	50	4.263	**	**
2203	100	2.225	2.211	2.288
7201	50	3.120	3.070	3.110
7202	50	4.303	4.254	4.400
7203	100	1.802	1.787	1.839
9201	25	8.693	8.137	8.680
9202	50	4.086	3.991	4.190
9203	100	2.000	†	†

* See Reference 3.

** Defective cell

† Leads and air tube cut too short for calibration vessel.

LABORATORY INVESTIGATION

Detailed descriptions of the sampling and laboratory test procedures and the original data plots are presented in Appendix D, Volume II, of this report.

Ballast Tests

Laboratory sieve analysis tests were performed on all the ballast samples. This sampling was intended to determine the amount and, if possible, the origin of fines present in the ballast.

For purposes of a preliminary evaluation, ballast samples from Track Sections 1, 4, and 9 were tested first. These preliminary tests consisted of sieve analyses and, if sufficient fines were collected, Atterberg limit tests. Samples from the other test pits were tested later. Summary plots showing the range in gradation of (under structure) "top" and "bottom" ballast samples are presented in Figures 58 and 59, respectively. The range in gradation of the shoulder ballast samples is shown in the summary plot in Figure 60.

Embankment Material Property Tests

The disturbed (bag) samples taken at 0 to 12 in. in depth and at 12 to 24 in. in depth in each test pit were subjected to sieve analyses and Atterberg limit tests. Results of these tests are summarized in Table 9. All but one of the samples were classified as highly plastic clay, CH, according to the Unified Soil Classification System.[13] The only exception was the lower sample from Pit 3B, Track Section 3, which classified as a lean clay (CL).

Laboratory compaction and CBR tests were conducted on specimens remolded from the bag samples. These specimens were prepared using the same modified American Association of State Highway Officials (AASHO) compactive effort specified for similar tests made while the embankment was under construction.[6] CBR tests were conducted on the as-molded

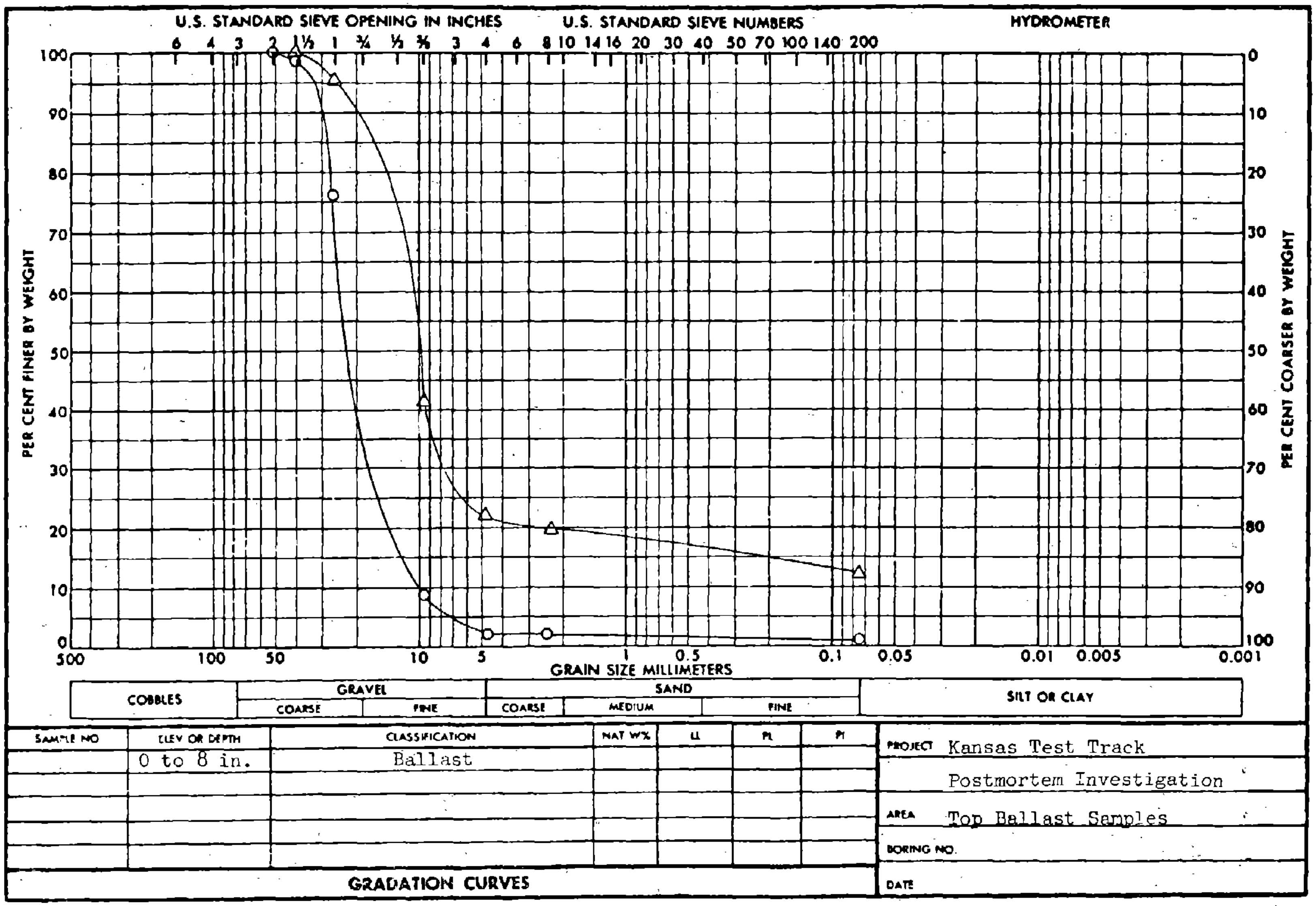
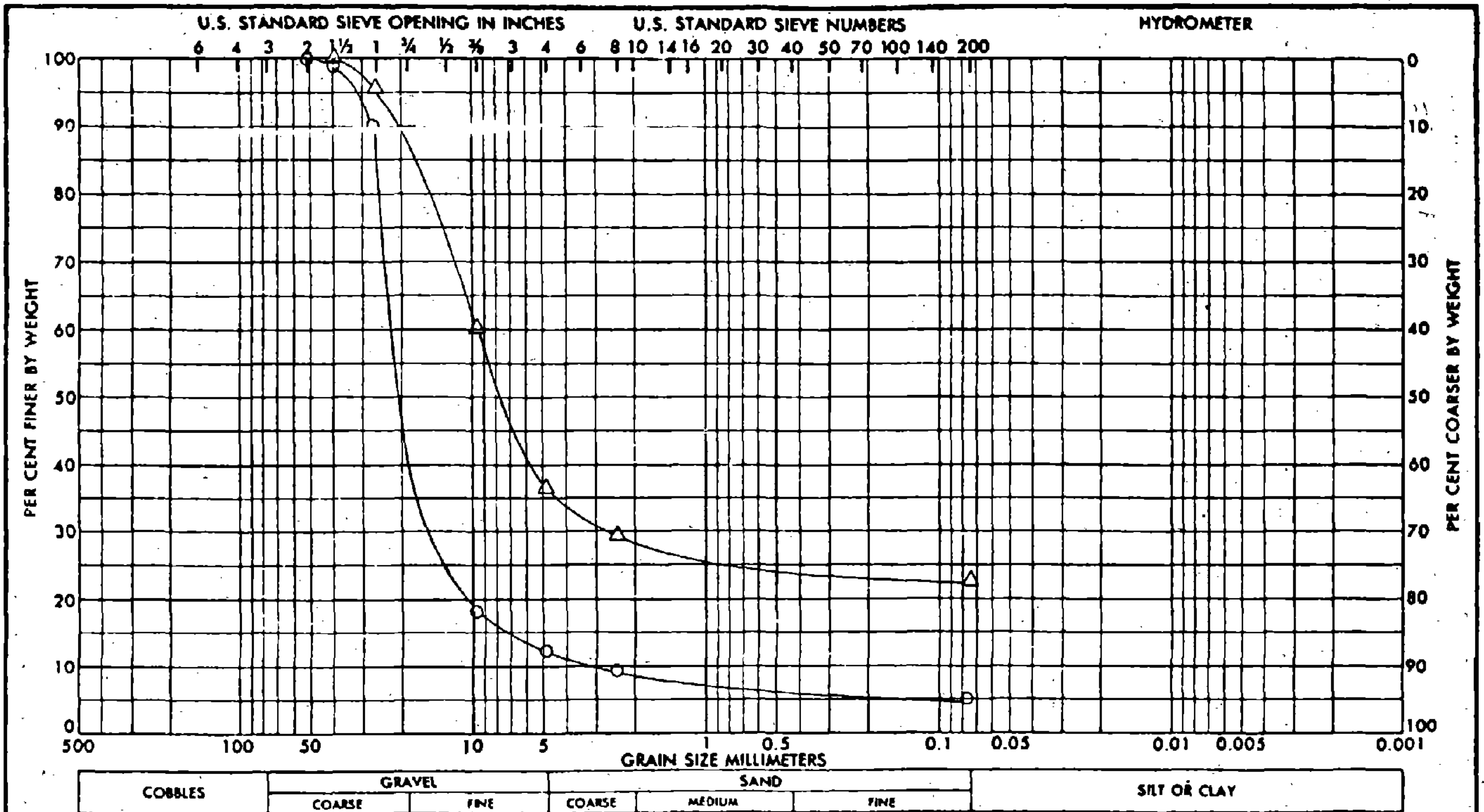


FIGURE 58. RANGE IN GRADATION OF "TOP" BALLAST SAMPLES

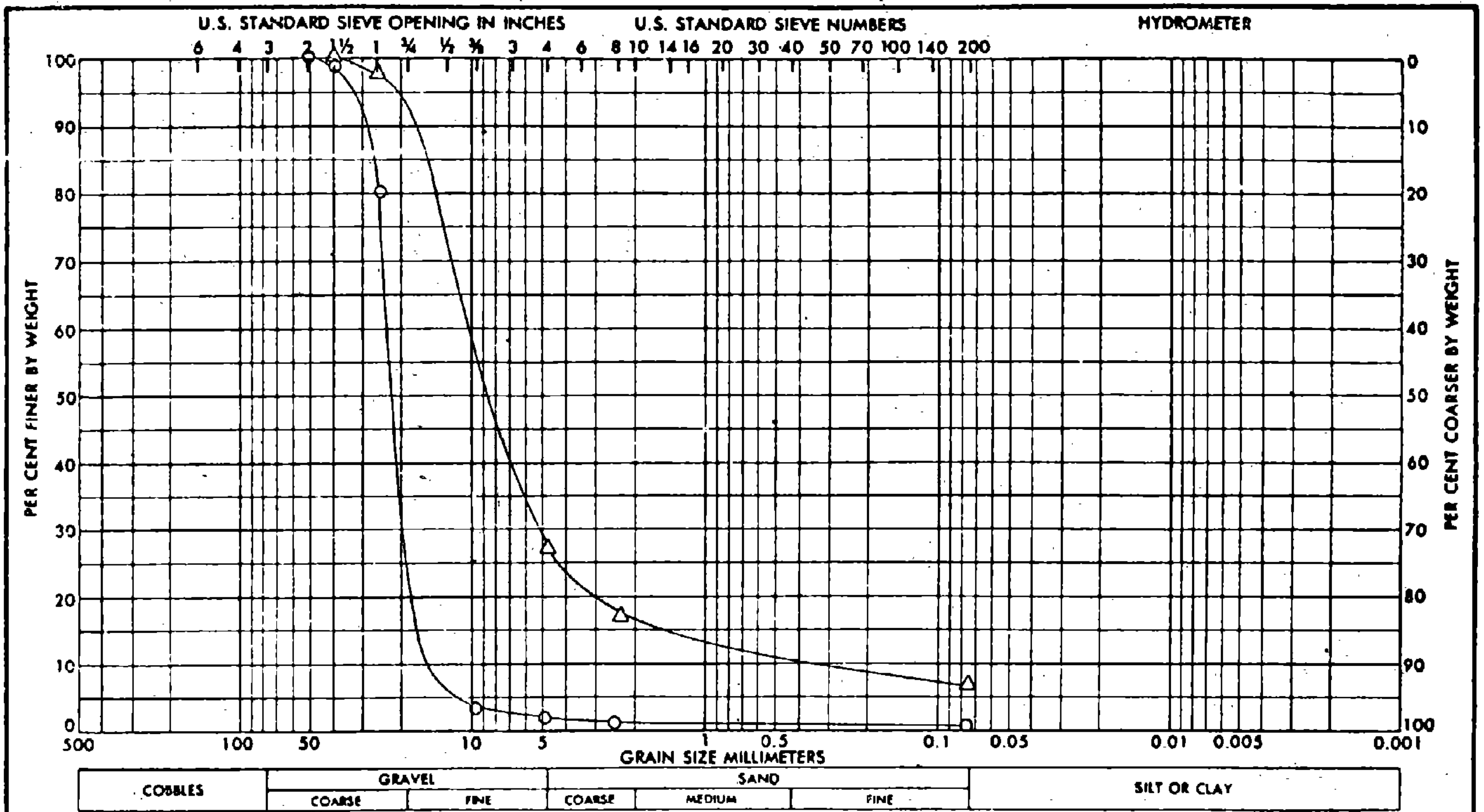


SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
	8 to 18 in.	Ballast				

PROJECT Kansas Test Track
Postmortem Investigation
AREA Bottom Ballast Samples
BORING NO
DATE

GRADATION CURVES

FIGURE 59. RANGE IN GRADATION OF "BOTTOM" BALLAST SAMPLES



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI	
	0 to 3 in.	Ballast					PROJECT Kansas Test Track Postmortem Investigation
							AREA Shoulder Ballast Samples
							BORING NO.
GRADATION CURVES							DATE

FIGURE 60. RANGE IN GRADATION OF "SHOULDER" BALLAST SAMPLES

TABLE 9. LABORATORY CLASSIFICATION TESTS ON SUBGRADE SAMPLES

Pit No.	Depth From Top of Subgrade	USGS Classification	LL	PL	PI	Percent Finer Than No. 200 Sieve	Color
1	0-12	CH	57	28	34	93	Brown
1	24-36	CH	62	23	39	91	
2A	0-12	CH	62	24	38	95	
2A	24-36	CH	61	24	37	96	
2B	0-12	CH	59	23	36	95	
2B	24-36	CH	63	23	40	95	
3A	0-12	CH	59	23	36	94	
3A	24-36	CH	60	22	38	94	
3B	0-12	CH	53	23	30	93	
3B	24-36	CL	46	20	26	69	
4	0-12	CH	61	24	37	94	
4	24-36	CH	66	24	42	95	
5	0-12	CH	63	20	43	89	
5	24-36	CH	59	25	34	92	
7	0-12	CH	58	24	34	91	
7	24-36	CH	60	19	41	91	
8	0-12	CH	56	23	33	93	
8	24-36	CH	58	19	39	90	
9	0-12	CH	62	24	38	97	Brown
9	24-36	CH	55	21	34	95	Reddish-Brown

specimens and on specimens which had soaked in water for 4 days. Typical CBR and compaction test results from Track Section 1 are shown in Figure 61. By averaging all the compaction test results, an optimum water content of 16.6 percent and a maximum dry density of 111.7 pcf were determined to be representative values for the embankment clay.

It should be noted for future reference (Figure 61) that the soaked specimens had little strength.

Embankment Strength Tests

Unconfined compression tests were conducted on 1.4-in.-diam by 3.0-in.-high specimens trimmed from the undisturbed samples obtained in each test pit. These tests were conducted for comparison with results of similar tests made during construction of the KTT embankment. Results of the unconfined compression tests, including moisture contents and densities, are summarized in Table 10. Unconsolidated, undrained triaxial tests were also made on samples from Test Pits 1, 4, and 9. Test data were obtained at confining pressures of 0.25, 0.5, and 0.75 tsf to simulate in situ pressures at depths of about 4, 8, and 12 ft in the embankment, respectively. At 0.75 tsf confining pressure, these tests showed only a slight average increase in strength above unconfined results. Hence, the triaxial results are presented only in Appendix D, Volume II, of this report.

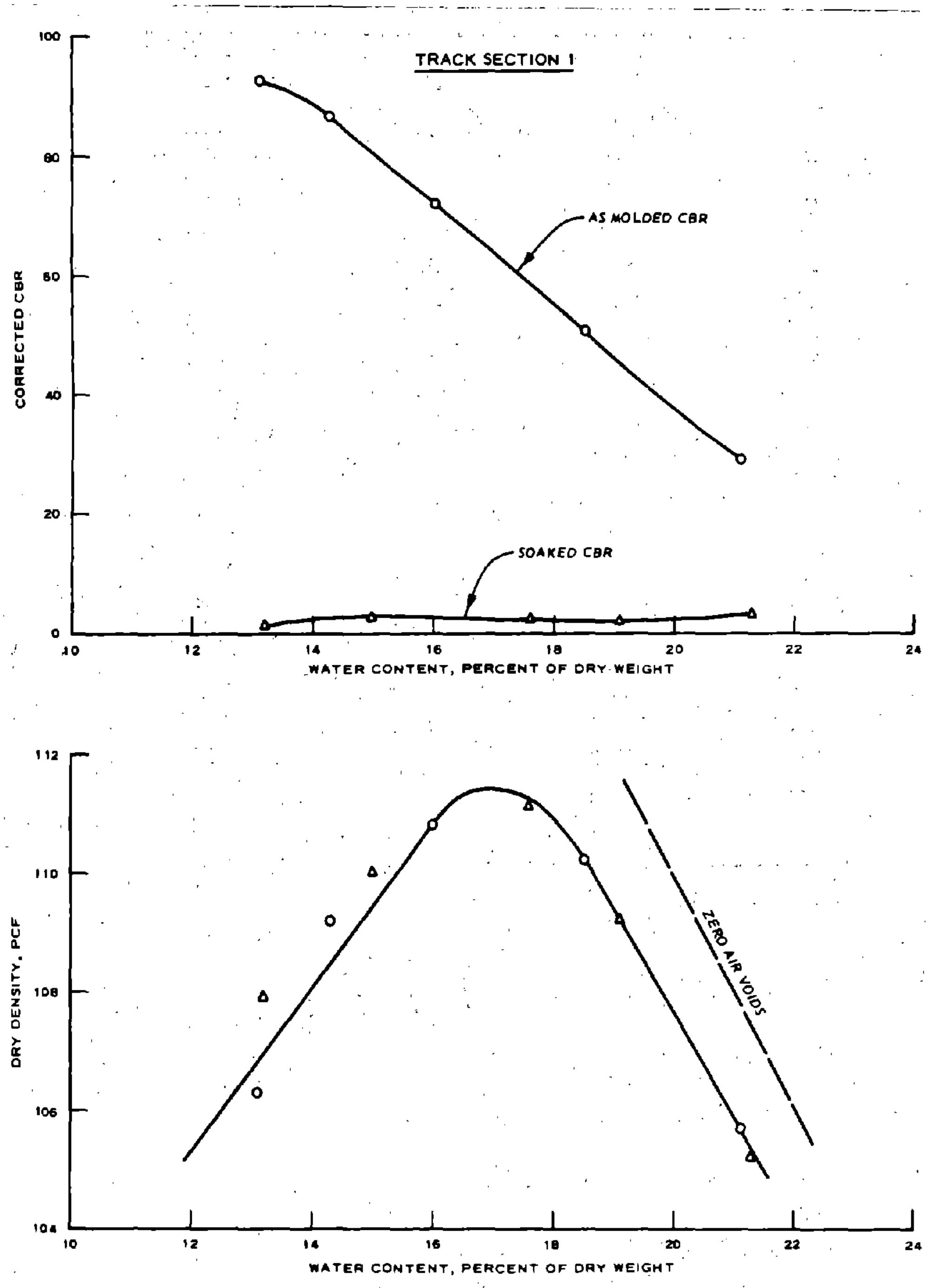


FIGURE 61. TYPICAL LABORATORY CBR AND COMPACTION TEST RESULTS (ON SPECIMENS REMOLDED FROM BAG SAMPLE OBTAINED IN TEST PIT 1, TRACK SECTION 1)

TABLE 10. UNCONFINED COMPRESSION TEST RESULTS ON SUBGRADE SAMPLES

Pit No.	Water Content %	Dry Density PCF	Unconfined Compression Strength TSF
1T	27.9	94.9	1.15
1B	27.2	95.7	2.25
2AT	28.3	90.7	2.33
2AB	28.0	93.4	3.03
2BT	27.7	95.7	2.06
2BB	25.8	95.5	2.42
3AT	30.1	89.9	1.18
3AB	31.7	88.8	1.26
3BT	27.4	93.0	5.21
3BB	30.1	91.0	1.50
4T	29.2	93.3	1.27
4B	26.8	93.8	1.30
5T	29.4	90.2	3.94
5B	28.2	94.2	1.20
7T	27.0	95.2	1.21
7B	25.0	99.2	2.34
8T	32.4	83.7	2.02
8B	25.0	98.1	2.05
9T	27.0	95.4	0.99
9B	25.8	96.4	0.96

ANALYSIS OF RESULTS

Structures

The static and dynamic load-deflection tests carried out in Track Section 4 were primarily intended to provide data for validation of an existing analytical model. Results of the study have already been reported by others,[10] but the test procedures, equipment, and static test data are documented in Appendix A, Volume II, of this report. Impedance test results have also been reported separately.[8] However, both the load-deflection and impedance studies contain information that is relevant to the postmortem investigation and will be briefly summarized.

Analyses of the load-deflection tests indicate that the twin beams in Track Section 4 behaved essentially as rigid plates and that appreciable relative motion occurred at the control joints between beams.[10] This motion caused the cracking and spalling illustrated in Figure 2. Plate bearing tests, conducted on the thin (typically 2-3 in. deep) ballast layer beneath north rail Beam 16, showed a variation in ballast-subgrade stiffness (k = 195 lb/in.^{3} in the middle of the beam and about 175 lb/in.^{3} at either end) over the length of the beam. The condition of reduced support stiffness beneath control joints and other highly stressed areas will be explored more fully in following discussions. The twin beams in Track Sections 4 and 7 also acted as water traps.[10] This and other aspects of KTT drainage will also be treated in detail later in this report.

Impedance test results provide a means for ranking KTT track sections according to their initial dynamic stiffness for both pretraffic and posttraffic conditions. Such a ranking is shown in Figure 62. A precondition for the posttraffic (October 1975) testing was the recreation of pumping conditions (December 1974 through April 1975) at each test location. To that end, up to 16,000 gal of water and 600 passes of a captive train were applied to some test locations. Surface indications of pumping were achieved only in Track Sections 1, 2, and 3. The impedance results in Figure 62 show that this approach resulted in a

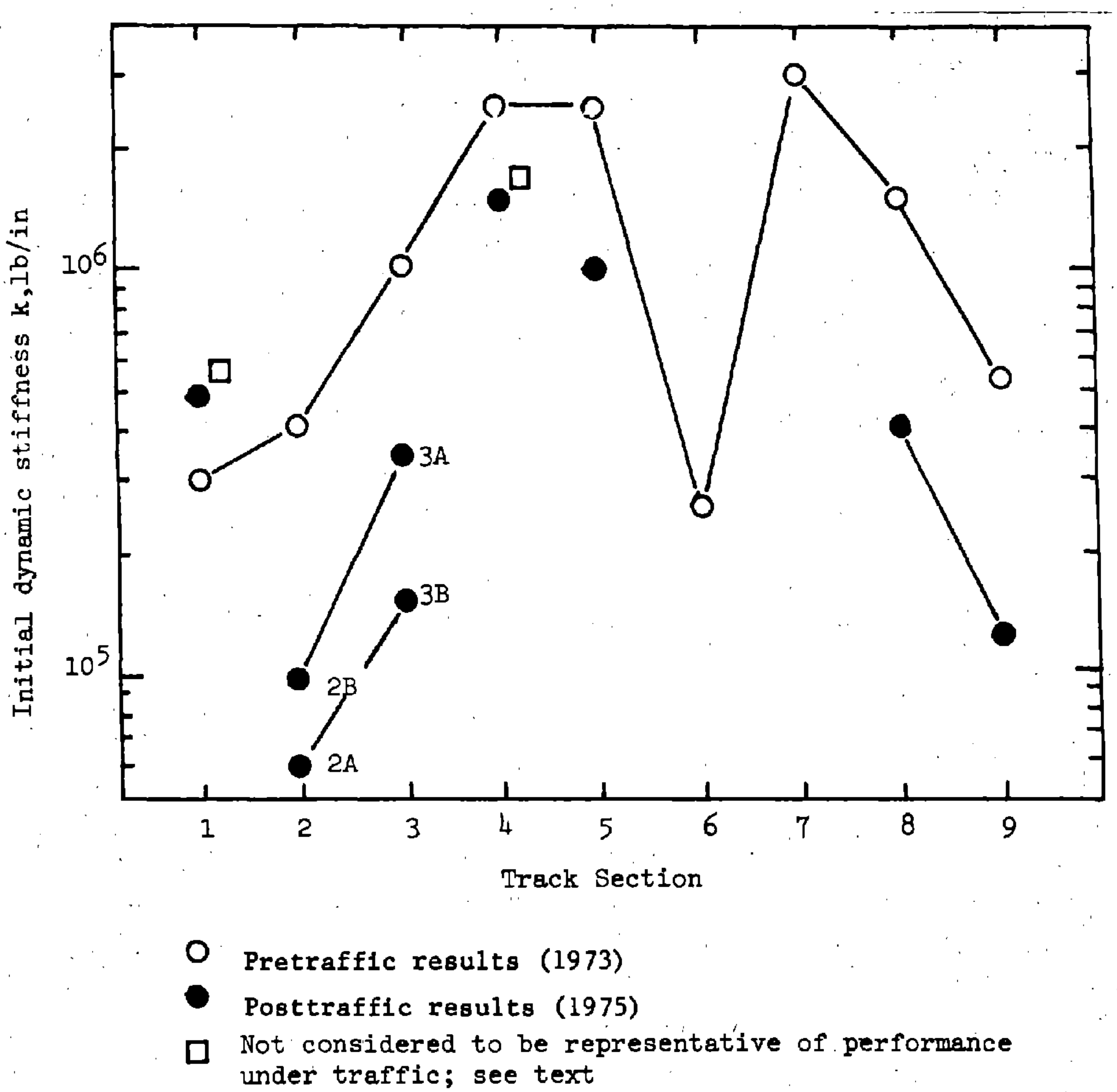


FIGURE 62. RANKING OF THE KTT TRACK SECTIONS ACCORDING TO THEIR INITIAL DYNAMIC STIFFNESS IN PRETRAFFIC AND POSTTRAFFIC IMPEDANCE TESTS

reduction of stiffness (posttraffic) of Track Sections 2, 3, 4, 5, 8, and 9. Track Sections 6 and 7 were not tested for reasons given earlier. However, the attempt to re-create pumping did not succeed in all instances, because Track Section 1 actually exhibited an increase in stiffness and Track Section 4 did not exhibit the degree of softening expected from pretest judgments of performance under traffic.[8] A relative stiffness ranking of KTT track sections was developed by other investigators[14] from tests conducted in December 1974 (when the KTT subgrade was in distress and pumping under traffic). Results of this testing are shown in Figure 63 and are believed to accurately reflect structure, ballast, and, most important, subgrade conditions existing in December 1974.

Among the more significant results of the structures testing are the following:

1. While the nonconventional track structures suffered some distress, such as cracking and spalling of the concrete, these and the conventional (tie) structures remained essentially intact throughout the brief life of the KTT.
2. The twin beams in Track Sections 4 and 7 were noted as being effective water traps, to the detriment of ballast/subgrade drainage.
3. Relative motion occurring at control joints between adjacent beams and slabs was linked to softening of subgrade support at these locales of relatively high stress.
4. The nonconventional structures were to have been founded on a 6-in.-thick ballast layer. In fact, ballast thickness was generally found to be 2 in. or less under the twin beams in Track Sections 4 and 7.
5. The beams in Track Section 4, north rail, were found to have rotated outward, away from the track center line.

The behavior of the track structures and the heavy pumping observed under traffic had focused attention on ballast/subgrade support conditions. These conditions will be discussed next.

Ballast

Results of the ballast tests indicated the presence of a large

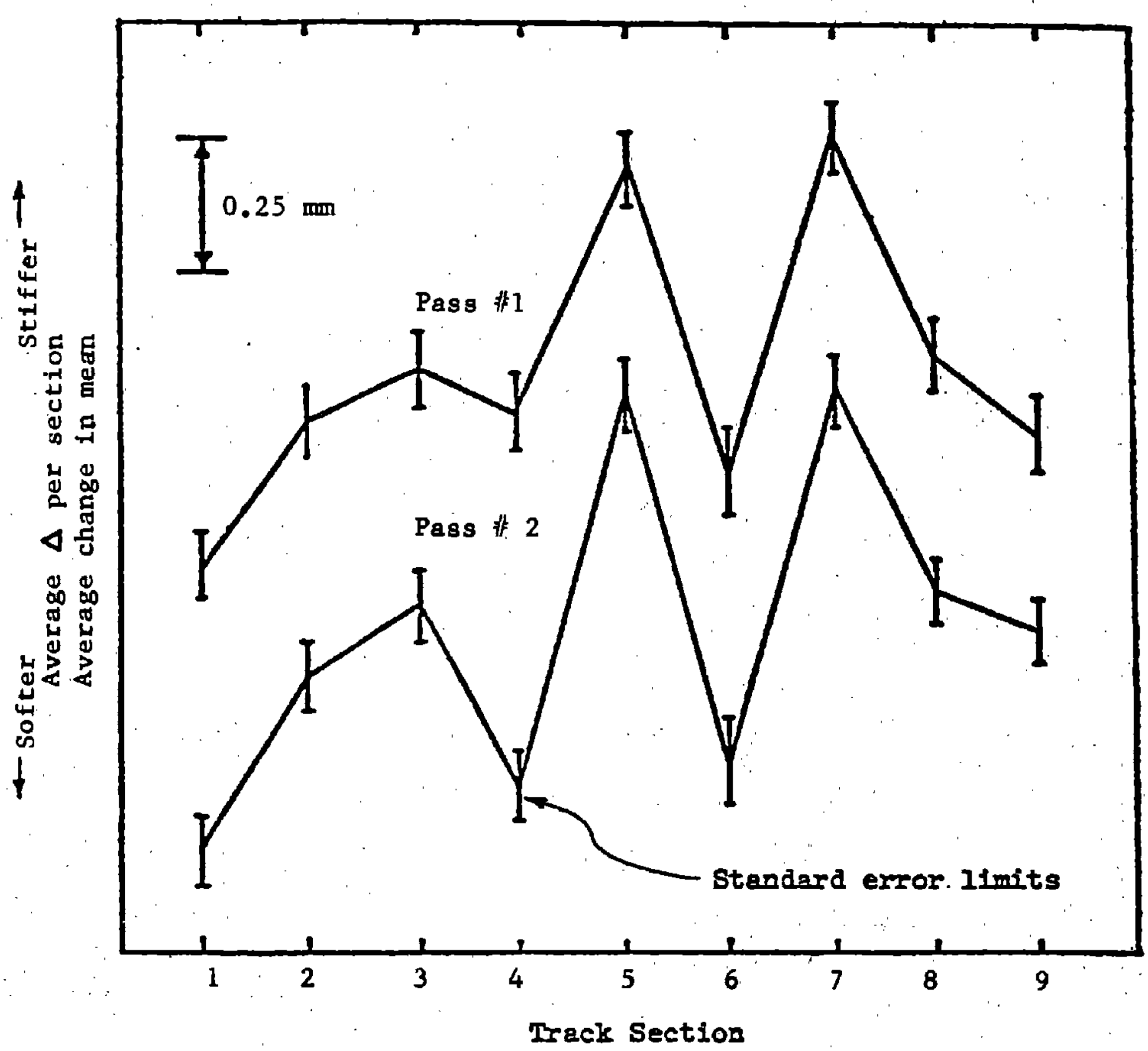


FIGURE 63. RANKING OF THE KTT TRACK SECTIONS ACCORDING TO THEIR RELATIVE STIFFNESS UNDER TRAFFIC (FROM TRACK GEOMETRY CAR MEASUREMENTS BY ENSCO, DECEMBER 1974)

amount of fines (15 or more percent by weight passing the 200 sieve) in the lower ballast level. The control (shoulder) sample gradation tests indicated less than 7 percent passing the 200 sieve. Atterberg limit tests were performed on the lower ballast fines; however, these test results were inconclusive because of small rock particles which were present in the material. These particles probably originated as (1) fines originally present in the ballast, (2) the products of ballast abrasion, (3) concrete particles resulting from ballast-tie abrasion, or, more likely, (4) a combination of all of these processes. The raw data and statistical analyses indicated that a slightly greater amount of fines was typically present under the north rail than under the south rail. While this difference was not statistically significant, the raw data do support the argument that pumping conditions were probably worse under the north rail.

The ballast fines (pumped material) were characterized from visual observations and photographs as predominantly brown or reddish-brown clay with occasional traces of gray material near the concrete structures. The black and white photograph shown in Figure 64 was reproduced from a color photograph taken in Test Pit 8, Track Section 8. Figure 64 clearly illustrates that the subgrade material pumped upward, filling the lower 11 in. of the 15-in.-thick slag ballast. Little or no traces of pumped material were present in the upper 4 in. of ballast. A fine white powder, which does not show clearly in Figure 64, coated the slag ballast that contacted the concrete crosstie. On its bottom surface and edges the crosstie itself was slightly pitted, a condition which is illustrated in Figure 64. The above conditions of tie pitting and white powder at the tie-ballast interface were characteristic of every location examined in Track Sections 1 through 3. In fact, the photograph in Figure 65 shows that when the ballast was graded off prior to testing, the seating area of the concrete ties in Track Sections 1 through 3 was clearly defined by white patches on the otherwise reddish-brown fouled ballast. The gray-colored material, which was noted as pumping at some locations under traffic, was undoubtedly a mixture of abrasion products (tie and ballast) and muddy water.

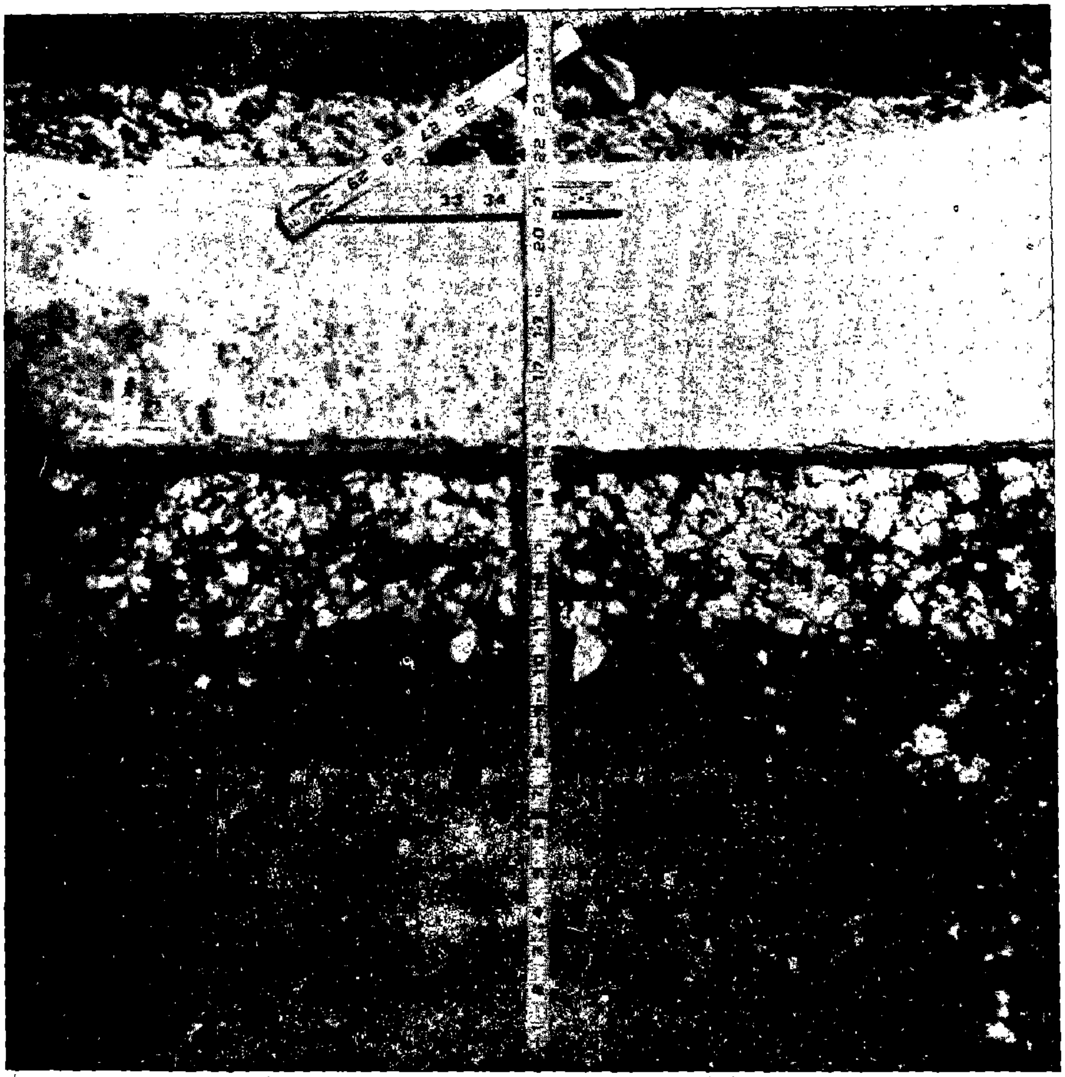


FIGURE 64. ILLUSTRATION OF SUBGRADE MATERIALS PUMPED 11 IN. UPWARD INTO THE 15-IN.-THICK BALLAST OF TRACK SECTION 8. NO TRACES OF SOIL (PUMPED) MATERIAL WERE DETECTED IN THE TOP 4 IN. OF BALLAST AT THIS LOCATION (TEST PIT 8). NOTE ALSO SLIGHT PITTING OF THE CROSSTIE AS A RESULT OF BALLAST ABRASION

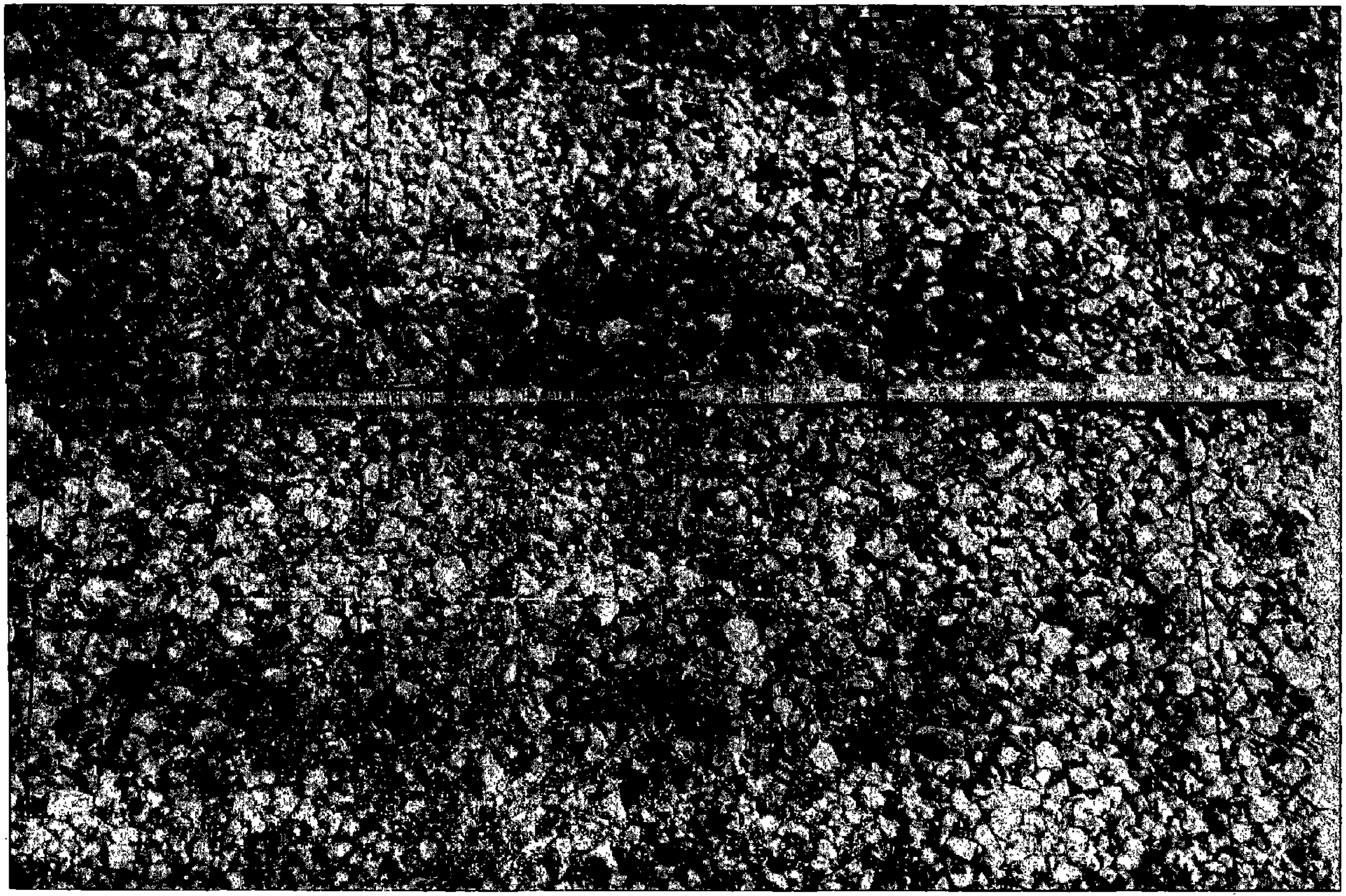


FIGURE 65. ABRASION PRODUCTS, IN THE FORM OF FINE WHITE POWDER ON THE BALLAST, WHICH TYPICALLY DEFINED THE SEATING AREA OF CONCRETE CROSSTIES. NOTE ALSO THE EVIDENCE OF (DRIED) PUMPED MATERIAL

There were other studies of interest, such as degree of ballast abrasion, and severity of pumping according to locale, which could not be pursued for the following reasons:

1. Any changes in ballast gradation under the structures as a result of abrasion were well within the range of gradation in shoulder ballast samples and so could not be detected.
2. Further attempts to assess ballast fines according to their origin would have required physical and chemical tests that would have far exceeded the scope of this study in number and cost. Facts already in evidence prove that the bulk of the fines present in the fouled ballast originated from the subgrade.

Embankment

Based on the postmortem cone penetrometer results summarized in Figure 36, the KTT embankment is characterized as having had reasonably uniform properties in terms of strength versus depth. In general, the remanent lime-stabilized surface layer (presumed to be 6 in. thick at the time of placement) exhibited a relatively high strength, but the soil from 0.5 to 1.0 ft in depth was much softer. From 0.5 to 3 ft in depth, the embankment strength increased; from 3 to about 4.5 ft in depth, the embankment had virtually the same strength; and below 4.5 ft in depth, soil strength decreased. Thus, for the postmortem (post-traffic) test condition, the embankment was softest just beneath the lime-stabilized surface layer, and some softening was indicated in the 1.5- to 2-ft-thick zone just above limestone bedrock.

The postmortem vibroseismic and penetrometer results show very similar trends, as illustrated by the summary plots of vibroseismic and penetrometer data presented in Figure 66. Figure 66 demonstrates a particularly good correlation between modulus (derived from vibroseismic tests) and cone point penetration resistance as a function of depth in the embankment. Since the cone penetrometer direct index of strength and the vibroseismic (indirect index of strength) results are in excellent agreement for the posttraffic test condition, it is also instructive to compare the pretraffic and posttraffic vibroseismic results to identify time-induced and/or traffic-induced changes in response. This

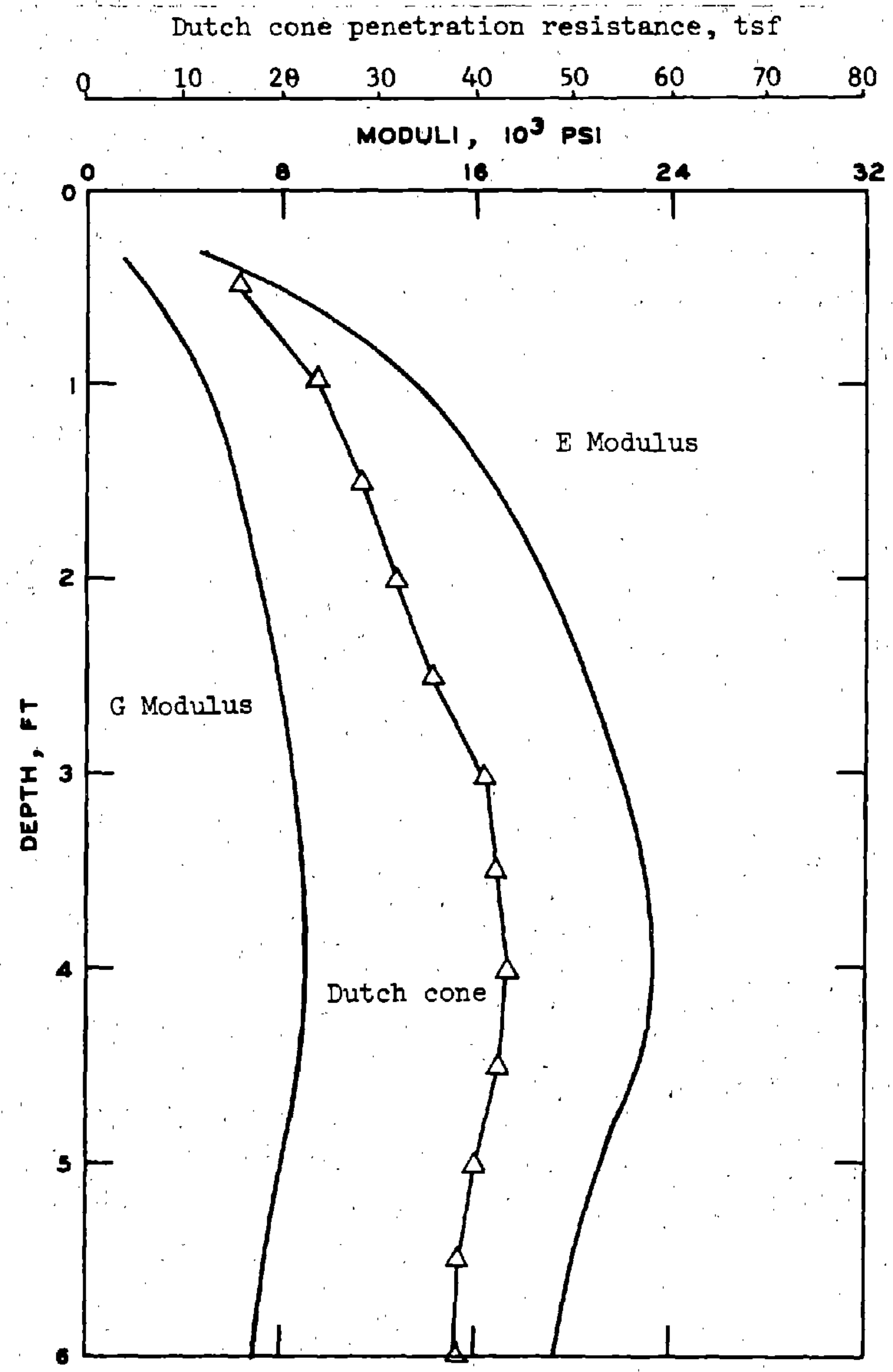


FIGURE 66. AVERAGE CURVES FOR ELASTIC MODULI AND PENETRATION RESISTANCE VERSUS DEPTH IN THE EMBANKMENT FROM ALL VIBROSEISMIC AND PENETROMETER DATA, RESPECTIVELY

comparison is presented in Figure 67. Both the E and G moduli plots in this figure indicate a softening in the upper and lower 1.5 to 2 ft of the embankment in the time interval (1972-1975) between pretraffic and posttraffic tests. These trends, i.e., softening in the upper and lower levels of the embankment, are also confirmed by the main array (built-in) instrumentation results which have been analyzed by other investigators.[9] Embankment vertical deformation as a function of time, from Reference 9, is summarized in Figure 68. Figure 68 shows that embankment heave occurred from 1971 until October 1974 when, under traffic, rapid settlement took place. The initial heave was attributed to an increase in moisture content above construction levels, and the heave occurred primarily in the upper 1.5 ft of the embankment.[6] The increase in moisture content (with attendant heave) is stressed as an important factor in assessing embankment behavior and performance.

Based on the average postmortem field moisture content, density, and CBR test results presented in Figure 52, the embankment is also characterized as having high-surface and near-surface moisture contents which tend to decrease with depth. The postmortem (1976) average moisture contents to a depth of 3 ft exceed the average (1971) embankment construction moisture content of 24.3 percent.[6] The CBR curves in Figures 52 and 61 illustrate that strength decreases in the embankment clay as water content increases. However, and as expected, the remanent lime-stabilized surface layer provided higher CBR values than the underlying clay. The CBR test locations, which were patterned to typify conditions at points of interest beneath the structures, were statistically analyzed with the results shown in Table 11. The "F" (variance) and Student "t" (mean) statistical tests were used to assess the CBR results. At the 90 percent confidence level, the means were not equal in only two statistical cases. There is then a 90 percent probability that:

1. At the surface, the subgrade under the north rail was softer than the subgrade under the south rail. At depth, the differences are much less significant.
2. The top of the subgrade under the rail seats was softer than the top of the subgrade beneath the middle of the crib.

With less statistical confidence, it can also be argued from the data

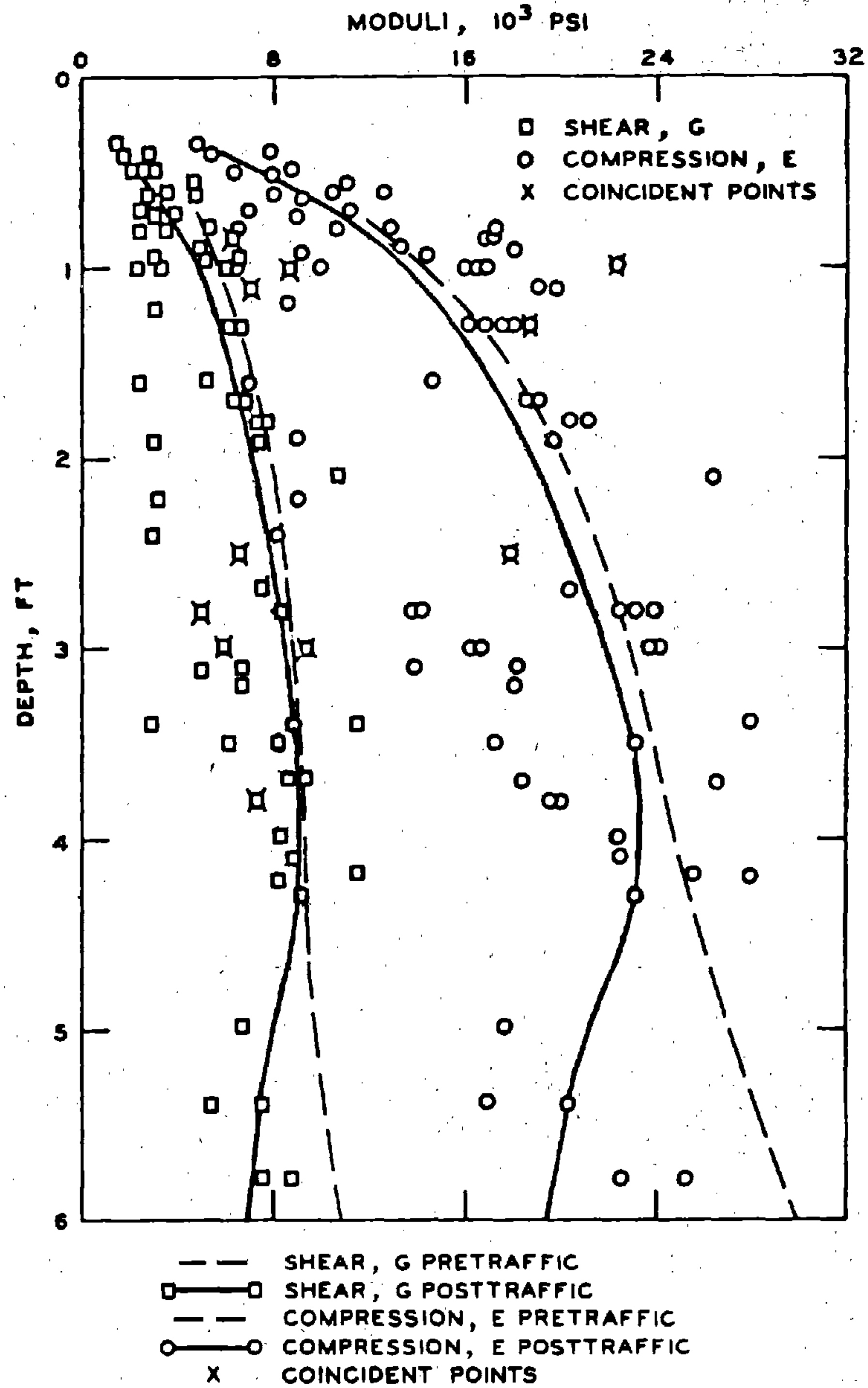


FIGURE 67. COMPARISON OF PRETRAFFIC AND POSTTRAFFIC VIBROSEISMIC DATA TO DETERMINE TIME AND TRAFFIC-RELATED CHANGES IN STRENGTH AS A FUNCTION OF DEPTH IN THE EMBANKMENT

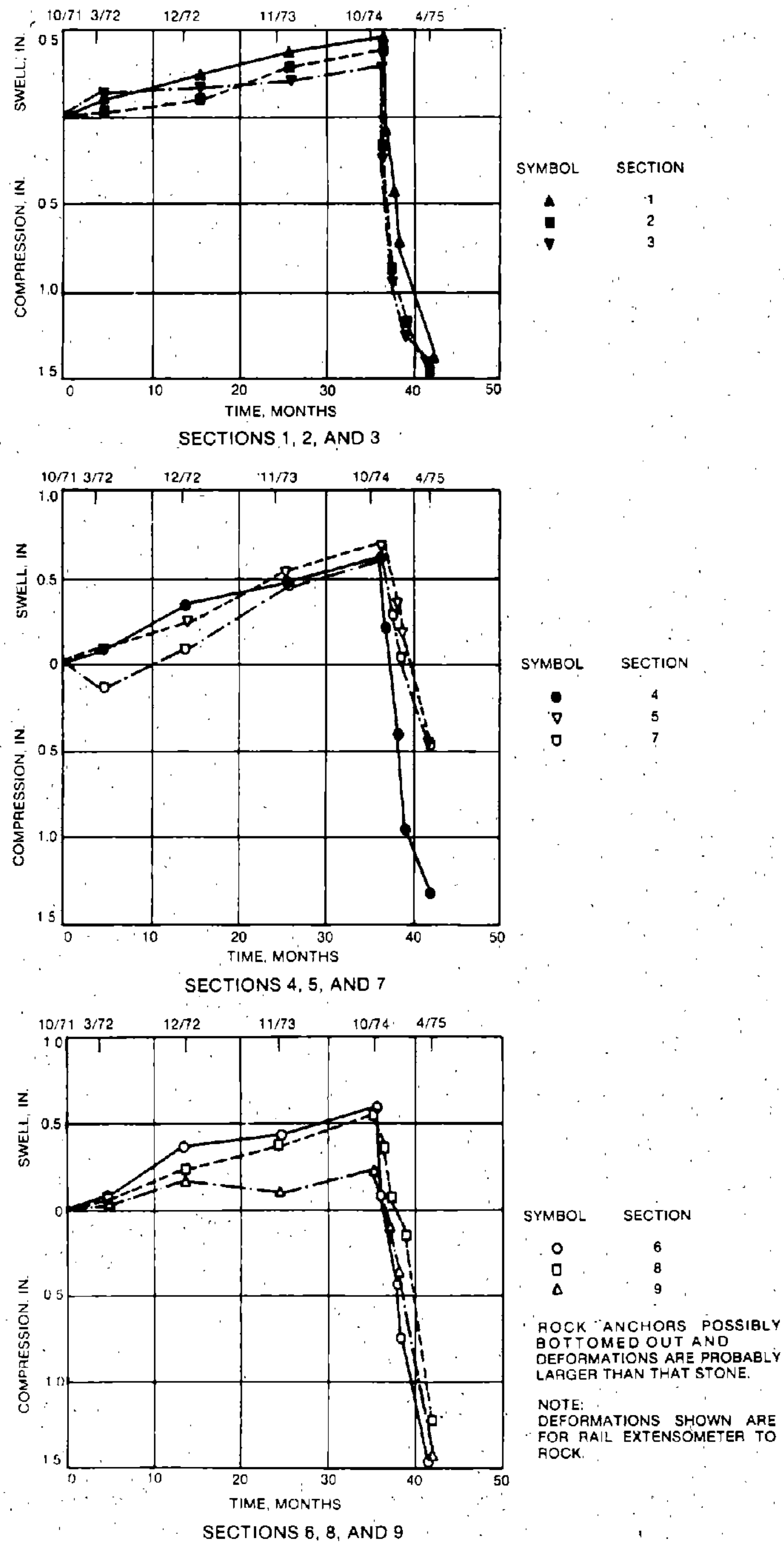


FIGURE 68. EMBANKMENT VERTICAL DEFORMATION AS A FUNCTION OF TIME (REPRINTED FROM FRA-ORD-76-258)

TABLE 11. RESULTS OF STATISTICAL F AND T TESTS ON MEAN CBR'S AT DIFFERENT LOCATIONS

Test	Location	90% confidence: ARE Variances Equal ?	90% confidence: ARE Means Equal ?	$\overline{X}$	$\overline{X}$
				north	south
North versus south rail	Surface	yes	no	12.25	16.86
	6 in.	yes	yes	4.97	4.59
	12 in.	yes	yes	6.20	6.73
	24 in.	yes	yes	10.53	10.10
	36 in.	yes	yes	10.27	11.16
	All combined	no	yes	9.20	10.57
				under	between
Under versus between ties	Surface	yes	yes	13.81	15.59
	6 in.	yes	yes	5.23	5.79
	12 in.	yes	yes	7.40	6.70
	24 in.	yes	yes	11.06	11.50
	36 in.	yes	yes	10.92	11.10
	All combined	no	yes	10.23	10.85
	Comb surface, 6 in. & 12 in.	no	yes	9.82	10.61
				joint	middle
Joints versus middle	Surface	yes	yes	25.27	28.48
	6 in.	yes	yes	3.05	2.73
	12 in.	yes	yes	6.20	6.68
	24 in.	yes	no	4.78	7.43
	36 in.	no	yes	9.40	7.95
	All combined	yes	yes	9.74	10.66
	All combined less surface	yes	yes	5.86	6.20
				comb. N&S	middle
Combined rails versus middle	Surface	yes	no	14.59	19.28
	6 in.	yes	yes	4.77	5.09
	12 in.	no	yes	6.47	7.09
	24 in.	yes	yes	10.31	9.22
	36 in.	yes	yes	10.73	11.11
	All combined	no	yes	9.90	11.27

that the subgrade was softer under the joints of the nonconventional structures than under the middle. While any one data set may be statistically inconclusive in this regard, the fact remains that the penetrometer (Figure 35), CBR (Table 11), plate bearing test results from Track Section 4, and analytical model results all support the argument, although the differences in strength are generally small.

Average CBR and plate bearing test results are shown in Figure 69. These results show generally similar trends; however, some variations are to be expected because of the relatively shallow depth of influence (approximately 3 to 4 in.) of the CBR tests and the appreciably greater depth of influence (approximately 45 to 60 in.) of the plate bearing tests. The high stiffness values recorded for Track Section 7 (Pit 7) are attributed primarily to the thickness and strength of the lime-stabilized layer in that section (the highest surface strength measured in any track section). A relatively stiff response was also recorded in Track Section 1. This response may partly explain the anomalously high stiffness recorded for that section in impedance testing.[8]

In order to assess embankment performance the following factors must be considered:

1. The relationship between density, moisture content, and strength in the embankment clay.
2. The effects of time-dependent fluctuations in moisture content.
3. The source of the moisture content increases noted in December 1974 and in the postmortem tests.
4. A rationale which correlates 1 through 3 above with track system performance and other data.

The embankment construction data[6] and postmortem test data were used to develop the desired relationship between moisture content, density, and strength of the embankment clay. The unconfined compression test results obtained during construction and in postmortem laboratory testing were converted to shear strength using the equation

$$S_u = \frac{Qu}{2} \qquad (1)$$

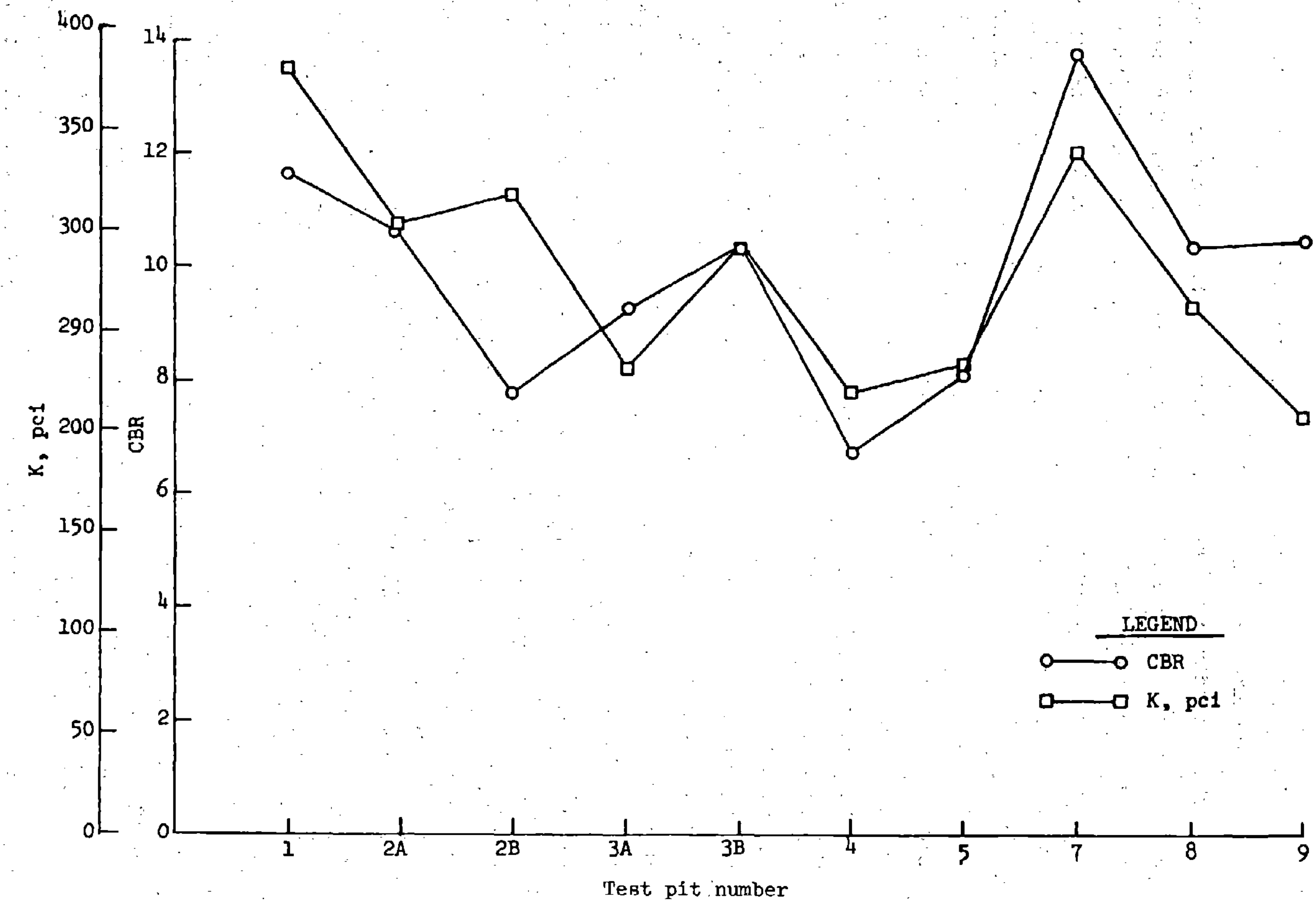


FIGURE 69. AVERAGE CBR AND PLATE BEARING TEST RESULTS BY TRACK SECTION

where

Qu = unconfined compressive strength, tsf

S_u = Undrained shear strength

All of the construction and postmortem unconfined compressive strength data were used to develop the density, moisture content, and shear strength plot shown in Figure 70. Postmortem test specimens that had an appreciable lime content are identified in Figure 70. They were not considered in formulating the various curves shown because the intent of the plot was to illustrate behavior of the subgrade clay rather than the lime-stabilized surface layer material. Accordingly, the upper and lower bound strength and density curves shown in Figure 70 were fit to the subgrade clay density and strength data. Also shown in Figure 70 are (1) the average embankment conditions during construction in 1971,[6] (2) average conditions in 1976 from the postmortem trenching data, and (3) conditions in December 1974 (under traffic) at shallow depths in the subgrade. The December 1974 envelope was developed from moisture contents measured by other investigators in Track Sections 4 and 9,[9] and relates to subgrade conditions just beneath the lime-stabilized surface layer. In the zone from 6 to 24 in. in depth, the postmortem average field dry density was 3.5 pcf less than the average density recorded for postmortem laboratory specimens (which were obtained from comparable depths). However, field and laboratory moisture contents were in good agreement. The average values for density and moisture content were:

	Averages	
	Dry Density γd, pcf	Water Content W, percent
Field	90.8	27.1
Laboratory	94.3	27.7

The density-moisture content-strength relationship shown in Figure 70 represents a conservative case, since field results indicate a lower average value of in situ density.

The effects of moisture content variations in the subgrade clay are clearly illustrated in Figure 70. From 1971 to October 1974, as moisture contents increased, the embankment heaved, primarily in the top

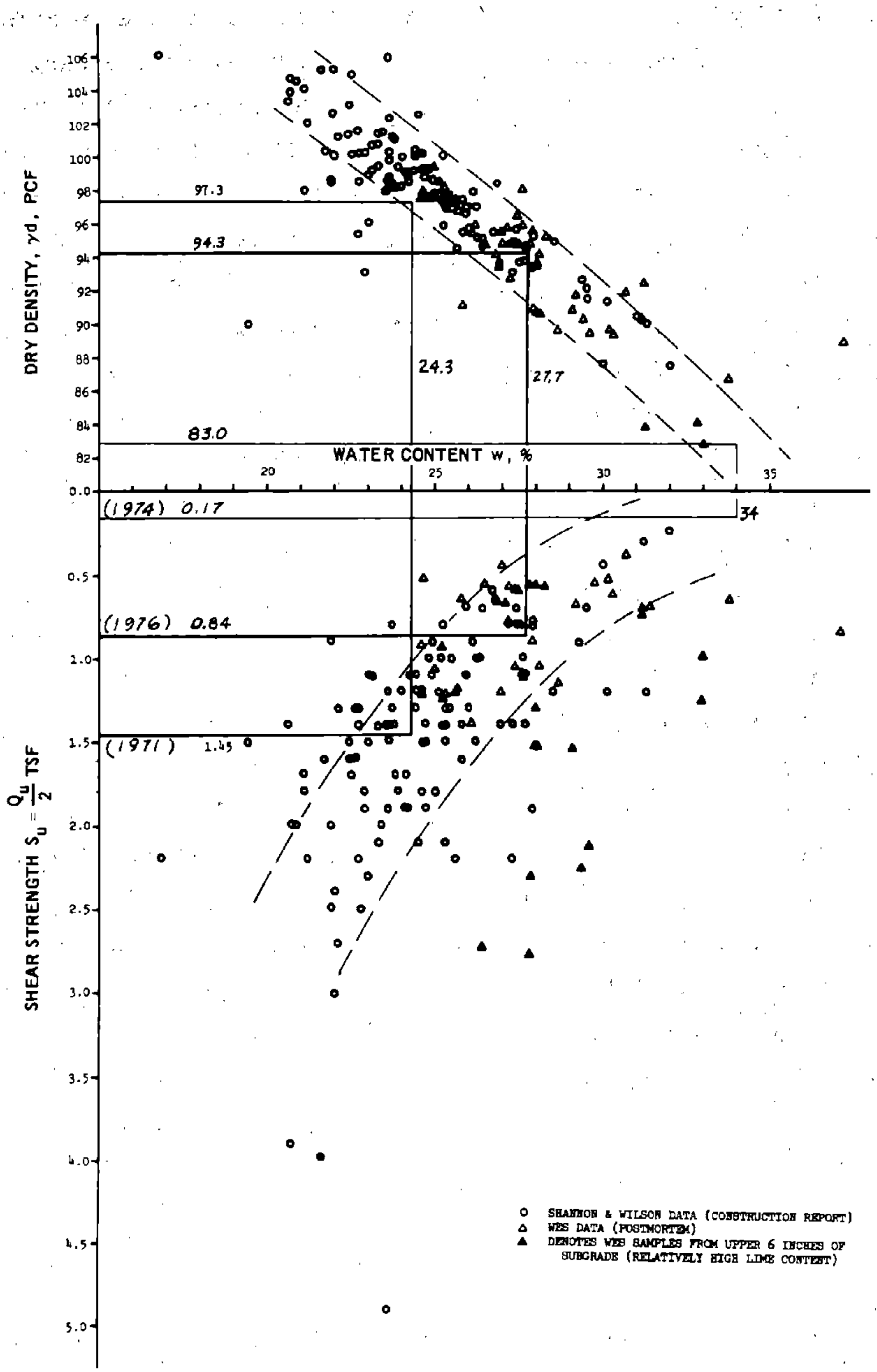


FIGURE 70. RELATIONSHIP BETWEEN DRY DENSITY, MOISTURE CONTENT, AND SHEAR STRENGTH DERIVED FROM CONSTRUCTION (1971) AND POST-MORTEM (1976) STRENGTH TEST RESULTS

1.5 ft of the subgrade. Figure 70 shows that this increase in moisture content was accompanied by a decrease in the strength of the subgrade clay. By December 1974 moisture contents in the subgrade (just beneath the lime-stabilized layer) had increased from 32 to 38 percent. In this range of moisture contents, the KTT embankment clay was nearly saturated; and strength in the upper layer of subgrade must have decreased drastically, as indicated by the strength curves in Figure 70. Postmortem embankment testing was performed in 1976, when the subgrade had partially recovered from traffic conditions (postmortem moisture contents 6 to 12 in. into the subgrade of Track Sections 4 and 9 had decreased from those measured in December 1974). Figure 71 shows rainfall amounts in the period 1971 to 1976 from the Cassoday, Kansas, gage. Although a weather station was located onsite, these records were incomplete; whereas the Cassoday gage, only 3 miles away, could provide continuous data. The rainfall records indicate that KTT received more rainfall (12.4 in.) in the traffic period of October 1974 to April 1975 than the rainfall (10.6 in.) recorded for the period from October 1975 to April 1976, which preceded postmortem embankment testing.

Based on data presented in Table 9, the KTT embankment soil has been characterized as a highly plastic clay (CH). Average Atterberg limit values for this clay (from Table 9) are LL = 60 and PI = 37. These values may be used to estimate the potential swell (heave, expansion) characteristics of the KTT embankment clay based on recent WES studies of expansive clay from which the following criteria were developed:[15]

LL, Percent	PI, Percent	Potential Swell, Percent	Potential Swell Classification
>60	>35	>1.5	High
50-60	25-35	0.5-1.5	Marginal
<50	<25	<0.5	Low

The KTT clay was obviously susceptible to swelling in the presence of water, and the degree of swell may have ranged from moderate to high.

In 1971, the KTT embankment clay was placed at water contents wet of optimum;[6] however, Table 12 shows that all the track sections still

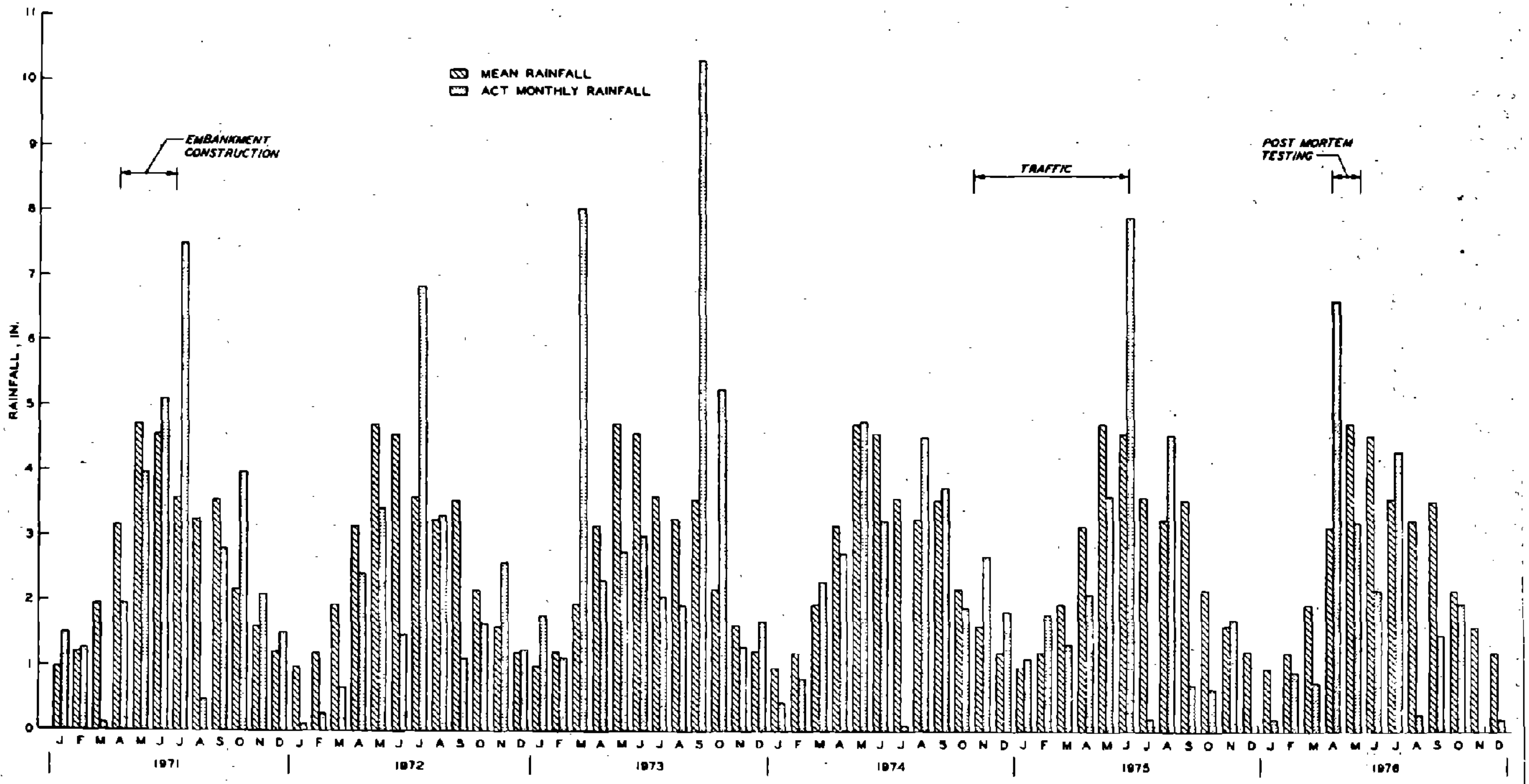


FIGURE 71. AVERAGE RAINFALL AT THE CASSODAY, KANSAS, GAGE LOCATION (3 MILES EAST OF THE KTT) IN THE PERIOD 1971-1976

TABLE 12. CUMULATIVE DEFORMATION, UPPER 1.5 FT OF KTT EMBANKMENT, 1971-OCTOBER 1974 (FROM FRA-ORD-76-258)

Track Section	Embankment Deformation, Inches: ① North Shoulder	② Centerline	③ North Rail	Average Deformations (① + ② + ③)/3 Inches
1	0.30*	0.18*	-0.28*	0.07*
2	0.25	0.13	0.13	0.17
3	0.43	0.32	-0.01	0.25
4	0.44	0.46	0.15	0.35
5	0.36	0.35	0.35	0.35
6	0.70	0.06	0.20	0.33
7	0.10	0.37	0.25	0.24
8	0.28	0.13	0.08	0.16
9	0.19	0.12	-0.12	0.06

* Positive values indicate swell (heave); negative values indicate settlement.

experienced some swelling in the period from 1971 to October 1974. An interesting fact is that Track Sections 4, 5, and 6 experienced the highest average swell in this period. Track Sections 4 and 6 later pumped heavily under traffic and were also deemed to have poor performance. Track Section 5 also pumped, but its physical size (full width slab) also resulted in lower top of subgrade stresses. Equally interesting is the relatively high swell recorded for the center line in Track Sections 4 and 7, where these twin beam structures had been noted as trapping water.[10] The uniform swelling recorded for Track Section 5 suggests that this full-width track structure tended to retain moisture beneath it once moisture contents increased. Track Section 6 had very high swell at the embankment north shoulder but much less swell under the structure. Track Section 7, which had the highest strength lime-stabilized surface layer tested, also swelled appreciably and later pumped under traffic. It will be noted from Table 13 that Track Sections 8 and 9 had less average swell than Track Sections 2 through 7, which lay upslope. This pattern is significant and will be discussed next.

A survey of drainage conditions was also made as part of the KTT trenching program, since pretest analyses had suggested possible problems in this area. Two drainage ditches had been provided during embankment construction[6] (Figure 5). The largest of these, located one access-road-width north of the KTT embankment toe, afforded adequate parallel drainage on the north side of the track. The remaining ditch was centered between the KTT embankment and the Santa Fe mainline track. This ditch did not serve its purpose of draining surface water from the top of the KTT and ATSF embankments. Poor drainage of the embankment top is illustrated by photographs (Figures 72, 73, and 74) that were taken during the 1976 (Phase II) postmortem investigations. Figure 72 is a view looking eastward toward Track Section 7 and was taken from the KTT grade crossing in Track Section 8. Figures 73 and 74 show details of the drainage culverts located under and near the grade crossing, respectively. Figure 75 shows the top of subgrade and ditch bottom profiles along the length of KTT. At its east end the ditch bottom was

TABLE 13. SUMMARY OF INSTRUMENTATION DEFECTS BY TRACK SECTION AND AND DATA ACQUISITION PERIOD*

a. Faulty Extensometer Channels

Test Section	Nov 74 Main Array	Nov 74 Total	Dec 74/Jan 74 Main Array	Dec 74/Jan 74 Total	Apr 75 Main Array	Apr 75 Total	May 76 Main Array
1	3	3	5	5	6	9	6
2	3	3	3	4	6	8	5
3	3	3	1	2	6	7	4
4	1	3	1	2	6	8	
5	1	1	1	1	2	3	
6	2	2	2	3	7	9	3
7	0	1	2	2	3	4	1
8	2	2	1	2	3	4	
9	3	4	2	3	5	6	2

b. Faulty Main Array Pressure Cell Channels

Test Section	Nov 74	Dec 74/Jan 75	Apr 75	May 76
1	3	1	0	
2	3	2	2	2
3	2	0	0	
4	1	0	0	
5	2	0	0	
6	2	2	2	
7	1	0	0	0
8	2	1	1	
9	1	0	1	0

* The number of faulty channels from one period to the next does not necessarily include the same channels.

FIGURE 72. PONDING OF SURFACE WATER IN THE DRAINAGE DITCH BETWEEN THE KTT AND ATSF MAINLINE TRACK (VIEW LOOKING EASTWARD TOWARD TRACK SECTION 7)

FIGURE 73. LATERAL DRAINAGE CULVERT (TRACK SECTION 8, ADJACENT TO GRADE CROSSING). NOTE LOCATION OF INLET WITH RESPECT TO DITCH

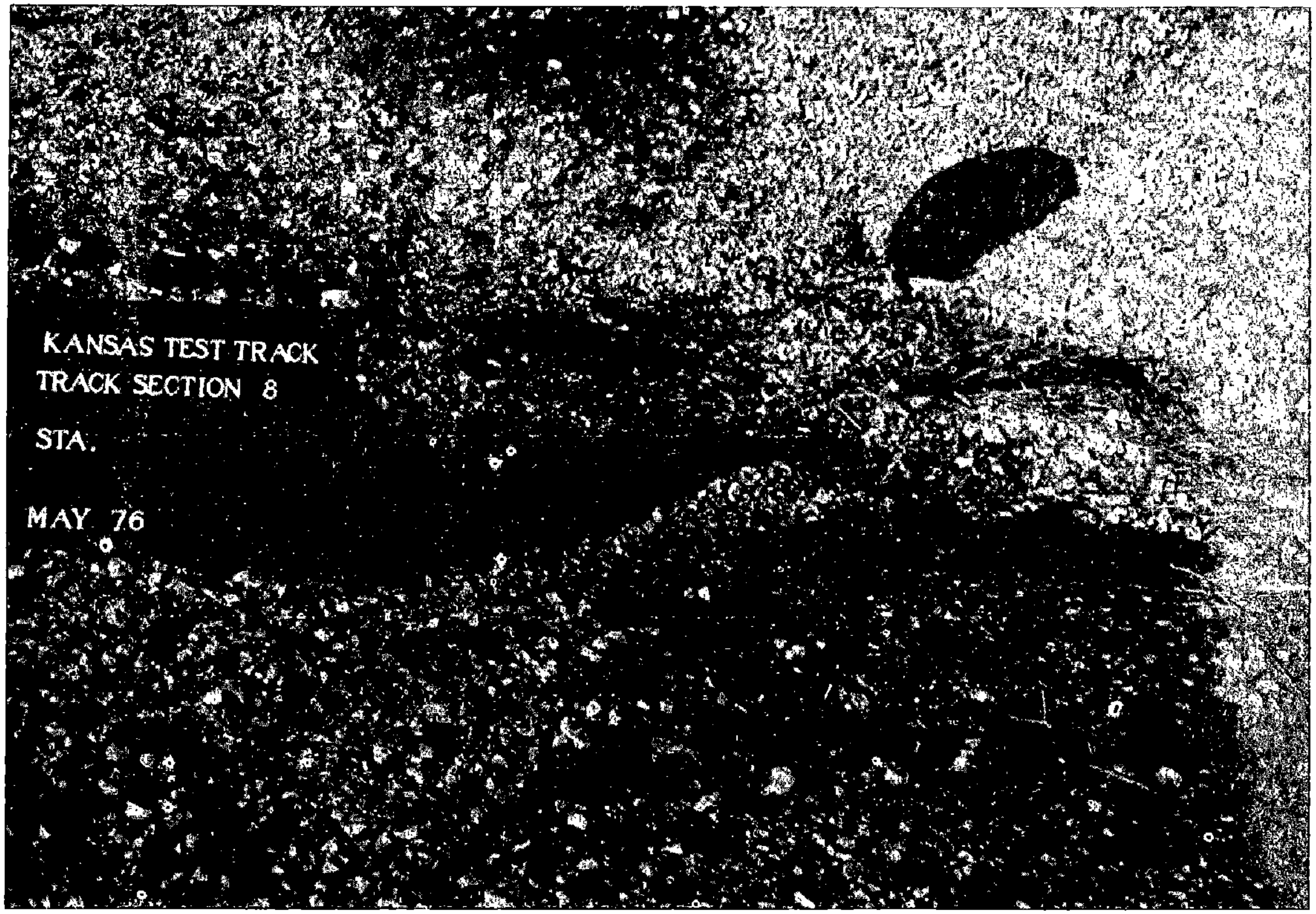


FIGURE 74. DRAINAGE CULVERT UNDER TRACK SECTION 8 (NOTE PARTIAL CLOSURE THROUGH SILTING)

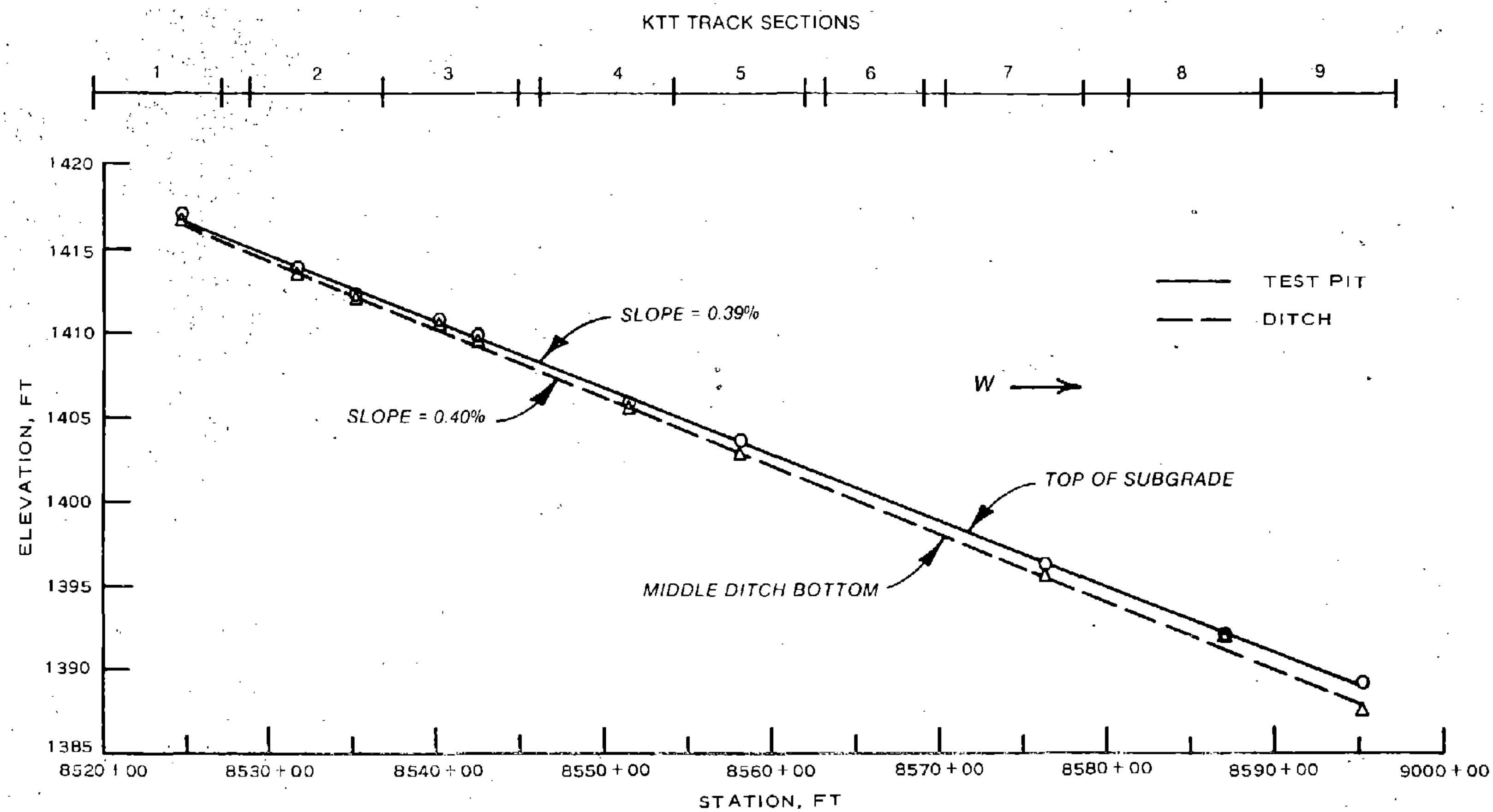


FIGURE 75. TOP OF SUBGRADE AND DITCH BOTTOM (BETWEEN TRACKS) PROFILES ALONG THE LENGTH OF THE KTT

about 6 in. below the top of the KTT embankment and dipped toward the west with a 0.4 percent slope. It should be stressed that any water present in the ditch was thus in direct contact with the top of the KTT subgrade along its southern flank, and that overflow conditions (easily achieved in the upper reaches of the ditches) would flood water over the top of the KTT embankment. Either condition was most undesirable and would have been aggravated by the ponding (channel storage) conditions which are obvious in Figures 72 through 74. It is also likely that the preventive measures taken during construction, i.e., the thin (sprayed) asphalt membrane and the (nominally) 6-in.-thick lime-stabilized layer, did not provide an effective barrier to water migration. Lime-stabilized soil typically develops tensile cracks upon curing, and the KTT asphaltic membrane appeared to be no more than 1/16 in. thick from field observations. Also, the lime-stabilized material would have a higher permeability than the intact clay.

Other drainage conditions, such as the tendency of the twin beams in Track Sections 4 and 7 to trap water, must have contributed to the subgrade problem. Once traffic-induced pockets had formed beneath the rail seats of the crossties, these depressions must also have served to trap water and to accelerate the process of subgrade deterioration. In Track Section 6 the ballast stabilization treatment left puddles of treatment material at the ballast-subgrade interface, which formed very effective water traps (from postmortem field observations during removal of the PCA subgrade pressure cells). Track Sections 8 and 9 also pumped under traffic but not with the severity of other track sections. It is instructive to note that Track Section 8 had the thickest (15 in.) ballast of any conventional track section and that Track Sections 8 and 9 were located where the middle drainage ditch was deepest and where at least some of the runoff water was diverted by an upstream lateral drain. These favorable conditions must have benefited performance under traffic.

Instrumentation

The built-in (main array) instrumentation will be discussed in the following order:

1. Data acquisition procedures
2. Vertical extensometers (deflectometers)
3. Soil pressure cells
4. Moisture-temperature cells

Detailed descriptions of the test procedures, equipment, and original data appear in Appendix E, Volume II, of this report.

The main array instrumentation was installed during construction of the embankment.[6] Data acquisition tasks were performed by the PCA.[9] Discussions between WES and PCA representatives indicate that a creditable effort was made to record these and other data.[16] The infiltration of pumped material into most sensor locations must certainly be classed as adverse operating conditions. Mud infiltration must have caused at least some of the low insulation values recorded in Table 4. Low values of insulation frequently result in low signal to noise ratios (noisy data) and ground loops. Problems of this sort were noted by PCA representatives.[16]

The vertical extensometers (deflectometers) experienced some mechanical failures as a result of the unexpectedly large embankment displacements. Both embankment heave and settlement were measured, and mechanical failures typically occurred when displacements exceeded the ±1 in. travel limit of the extensometers. These conditions are summarized in Table 6. Extensometer heads were usually found to be tilted to conform with the undulating surface at the ballast-subgrade interface. There were also instances of mud infiltration into the extensometer head, presumably through construction joints, since the head seals were intact in all instruments opened for inspection. However, the primary problem with the extensometer installation was that unexpectedly large displacements caused internal mechanical damage. If overranged, the extensometer became either inoperative or erratic in behavior (phase reversals, etc.). A summary of defective extensometers, according to track section and data acquisition periods,[9] is shown in Table 13. Soil pressure cells are included in Table 13.

The soil pressure cells (upper and lower) installed in the main array fared better than the other transdu ers (Table 13). The six pressure cells calibrated at the WES exhibited good linearity, and the differences in calibration values shown in Table 8 could be a result of the type and length of cable used during calibration. All six of the excavated pressure cells were well seated in the subgrade, but cables and conduits installed in their vicinity (Figure 55) may have influenced stress distribution in the soil, hence measurement accuracy.

Temperature and moisture sensors were located in the same cell. The temperature sensors appeared to function properly; however, the moisture meters were found to be undependable and inaccurate.[9] The moisture sensor used is basically a device to measure conductivity, which usually increases with moisture content in the surrounding soil; however, chemical impurities can also influence the sensor output. Conversations between the WES and the sensor manufacturer revealed that the recommended measurement procedure is to match the required number of sensors according to their internal resistance (to eliminate individual calibrations). Calibrations should be then performed in the same soil in which measurements are to be made. Unfortunately, chemical impurities, such as organic matter and dissolved salts, could still affect the accuracy of the readings. Since organic material was found in the sensing mesh of KTT moisture sensors, the data derived from these instruments are certainly questionable, if not worthless. It is probable that the cumulative effect of the organic matter on moisture sensors buried since 1971 would have rendered them inaccurate before KTT testing began in October 1974.

CONCLUSIONS

No single event prompted closure of the KTT. Progressive deterioration of the subgrade, excessive maintenance, the prohibitive cost of rehabilitation, and potential safety hazards were all considered in reaching the decision. The KTT "failure" must be viewed in this context.

Mechanism of Failure

The KTT operational problems stemmed primarily from a loss of strength in the upper 12 to 18 in. (immediately below the lime-stabilized surface layer) of the highly plastic clay embankment as a result of moisture content increases in the subgrade. Inadequate surface drainage of the top of subgrade was the principal cause of the moisture content increases. There were other contributing factors whose influence will be assessed in the sections to follow.

Structures

Analyses of structural behavior are beyond the scope of this report, and, in the case of the nonconventional track structures, have already been accomplished by other investigators. However, some generalizations are in order, because structural behavior has an important influence on the structure-ballast-subgrade interaction processes that govern track system performance. Among the more important conclusions regarding KTT track structures are the following:

1. Although KTT track structures suffered some distress, both the conventional and nonconventional track structures were essentially intact when traffic was terminated.
2. The twin beam structures in Track Sections 4 and 7 behaved as efficient traps for surface water, to the detriment of the subgrade. The north rail beams in Track Section 4 were observed to tilt toward the north side of the track as the subgrade softened.
3. Track Sections 5 and 7 exhibited the highest stiffness

recorded in the midchord offset traffic tests of December 1974. Track Section 4, although virtually identical to Track Section 7 in physical characteristics, had a much lower stiffness in the same tests.

4. All of the track structures settled appreciably under traffic, and all showed evidence of pumping (Track Sections 8 and 9 pumped but not to the top of ballast as in the other track sections).

Ballast

Ballast test results led to the following conclusions:

1. Ballast thickness under the nonconventional structures was typically 1 to 2 in., which was inadequate in terms of either drainage or load distribution to the subgrade.
2. Ballast gradation tests showed that more fines were present under the north rail, which indicates that pumping was more severe under this rail.
3. Data scatter in the gradation test results prevented an assessment of ballast deterioration under traffic, but some deterioration undoubtedly occurred.
4. The ballast in Track Section 8 was 15 in. thick, and subgrade pumping filled only the bottom 11 in. of this material. Increases in ballast thickness obviously serve to reduce stresses on the subgrade, increase stiffness, and improve performance. However, it is also likely that practical increases in ballast thickness (up to say 36 in. under the crossties) would only postpone complete fouling (in the case of KTT or similar subgrade conditions).
5. The ballast treatment (elastometric polymer) used in Track Section 6 acted as a most effective water trap, and this section's performance under traffic was very poor.

Embankment

The KTT embankment was built of a locally available residual highly plastic clay which was moderately to highly susceptible to swelling in the presence of water. Swelling in this material results from an increase in moisture content and is accompanied by a marked reduction in strength.

As a result of poor site drainage conditions, the KTT embankment swelled up to 1 in. in the period from 1971 to October 1974. The most detrimental swelling occurred in the upper 1.5 ft of the embankment; however, swelling was also recorded in the lower level of the embankment (just above the embankment-limestone interface).

All KTT track structures settled rapidly and pumped under traffic; however, Track Sections 8 and 9 performed somewhat better than other KTT track systems. This result is attributed to:

1. Better drainage condition in Track Sections 8 and 9.
2. The thickness (15 in.) of ballast in Track Section 8 and the tie spacing (19.5 in. C-C) in Track Section 9, which resulted in relatively low top of subgrade stress levels.

The KTT was conceived as a means to evaluate track system performance under actual revenue traffic in which various track structure design variables would be utilized to provide a basis for comparisons. In fact, KTT subgrade support conditions proved to be the prime test variable, and track system behavior had to be rationalized accordingly. In recognition of this fact, it is concluded that:

1. Subgrade support conditions were the primary factor influencing KTT track system performance.
2. Subgrade support conditions varied with time, according to moisture content fluctuations in the subgrade.
3. Distress of KTT structures (other than the pulled-out studs), maintenance problems, and operational hazards were effects which were caused primarily by subgrade problems.
4. Preventive measures, such as the KTT lime-stabilized surface layer and the sprayed asphaltic membrane, were beneficial but could not compensate for the embankment softening caused by inadequate drainage.
5. Increased thickness of ballast and smaller tie spacings tend to reduce top of subgrade stresses and thus serve to enhance track system performance and durability, as might be expected. In the case of a very soft subgrade, such as in the KTT, however, these features may only serve to postpone the inevitable problem.
6. Track structural features, such as the "water-trapping" twin beams in Track Sections 4 and 7, can cause subgrade problems.
7. Finally, and most important, the KTT embankment study illustrates that any railroad track is a complex system whose

performance is governed by structure-ballast-subgrade interaction processes. If a balanced design is not produced, for whatever cause, component incompatibility will result in premature failure. It is through studies such as the KTT postmortem investigation and ongoing accelerated service tests that the necessary insight and understanding of track behavior will be secured.

Instrumentation

In this study only the instrumentation installed below the top of subgrade was evaluated, and findings were as follows:

1. Apparently, contractual arrangements did not provide for troubleshooting defective sensors of any type. Hence, data were obtained only from those instruments which were functioning properly during each recording session, and no repairs could be made between sessions.
2. The KTT moisture sensors proved to be very unreliable;[6] however, the temperature sensors (located in the same cell) apparently gave reliable results. The organic matter present in KTT embankment clay is believed to have adversely affected moisture measurements.
3. Many of the vertical extensometers failed when embankment deformations exceeded the nominal ±1 in. range of the LVDT sensors. The KTT multiple vertical extensometers were relatively bulky; and the large soil mass displaced by the vertical and horizontal extensometers in the main array may have, to some degree, influenced behavior of the surrounding soil. The combination of relatively large-size vertical and horizontal measurement devices was, in this sense, undesirable.
4. While the soil pressure cells used in the main array generally behaved well, the WES-developed soil pressure cell is preferred because it was designed and developed specifically for use in soil masses.[17]
5. The data acquisition system used was adequate for recording dynamic data, but it did not provide enough capability to record all channels simultaneously in a format suited to automatic data processing. Hence, data had to be recorded from different train passes, complicating the analyses and limiting comparability.

RECOMMENDATIONS

The KTT was unique in terms of its configuration and approach to track structure performance comparisons; however, contrary to the original design concept, the KTT subgrade turned out to be the primary test variable. The highway and railroad literature are replete with examples of subgrade pumping similar to that observed at the KTT. The following ballast and embankment recommendations should be generally applicable to other track systems founded on clay material.

Ballast

Railroad ballast primarily serves to reduce top of subgrade stresses by distributing structural loads and also acts to provide free drainage of surface water. In keeping with these functions it is suggested that:

1. Ballast materials which satisfy railroad criteria for strength, abrasion resistance, etc., should be placed in thicknesses compatible with lower bound subgrade strengths (seasonal variations in strength may be expected with lowest strengths generally occurring in the wet spring months). In general, experience with highway and railroad foundations proves that long-term maintenance costs arising from inadequate ballast thickness will greatly exceed any initial savings during construction.
2. Ballast layers, graded in particle size from top (coarse) to bottom (fine), are recommended as a measure to protect against subgrade pumping. The bottom subballast layer should be sufficiently thick (minimum 6 to 8 in.) to cushion the subgrade against coarse ballast intrusion and should have particle sizes on the order of concrete sand to act as a filter to prevent pumping of the subgrade fines.
3. Other filter materials, such as cloth membranes, should be investigated for possible usage to prevent pumping.

Embankment

The KTT embankment was built of locally available clay that was susceptible to swelling in the presence of water. Inadequate drainage was responsible for most of the KTT subgrade problems and was a direct cause of the subgrade pumping which occurred. Accordingly, it is recommended that:

1. Particular attention be given to basic drainage design for railroad tracks founded on clay embankments, and provisions be made to maintain such drainage systems in good working order. Inadequate drainage will result in increased ballast and subgrade maintenance or possibly failure, in extreme cases.

2. Measures such as the KTT lime-stabilized surface layer can provide beneficial increases in foundation strength and stability under seasonal fluctuations in moisture content. Lime stabilization is, therefore, a recommended treatment when the foundation clay is susceptible to swelling. However, for railroad applications, strength and stability are primary concerns; and a thicker (say 12 to 18 in.) stabilized layer would better serve the purpose. Lime stabilization should not be viewed as a means of providing a moisture barrier, since the lime-stabilized layer will generally have greater permeability than the intact clay.

3. Recent moisture barrier technology, including membranes, rubberized asphalt, and the like, should be investigated to determine their applicability to railroad foundation problems.

Instrumentation

The following recommendations should be considered in future designs of railroad instrumentation arrays:

1. The attempt to measure moisture contents in the KTT embankment was laudable; however, the moisture sensors used were ineffective because of contamination by organic matter. Research to develop improved moisture sensors is now under way, and this approach should not be abandoned in future planning.[18] Piezometers could also be used to determine pore water pressures. Nuclear moisture-density measurements in the subgrade (through preinstalled tubes) might also prove to be practical.

2. Every effort should be made to reduce the physical size of railway embankment instrumentation. Large inclusions in any soil mass tend to influence its behavior, and relatively large devices such as the KTT vertical extensometers should be avoided for this reason. Also, large reference plates (such as the 16-in.-diam heads of the KTT extensometers) cannot easily conform to sharp differential settlements such as occurred at KTT.

3. Given the accuracy possible with current LVDT's and the foundation settlements experienced at KTT, nominal ranges for vertical deformation in this instance should have been at least ±3 in. rather than the ±1 in. range specified, depending upon the subgrade. Circumstances will dictate future choices; however, it would be well to emphasize adequate range and serviceability as primary design criteria.

4. If at all possible, simultaneous recording and a format suitable for automatic data processing should be incorporated in future data acquisition procedures.

5. Future instrumentation responsibility should be vested in one organization with one project officer in control, and contractual arrangements should provide for onsite and laboratory maintenance of installed instrumentation as needed.

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