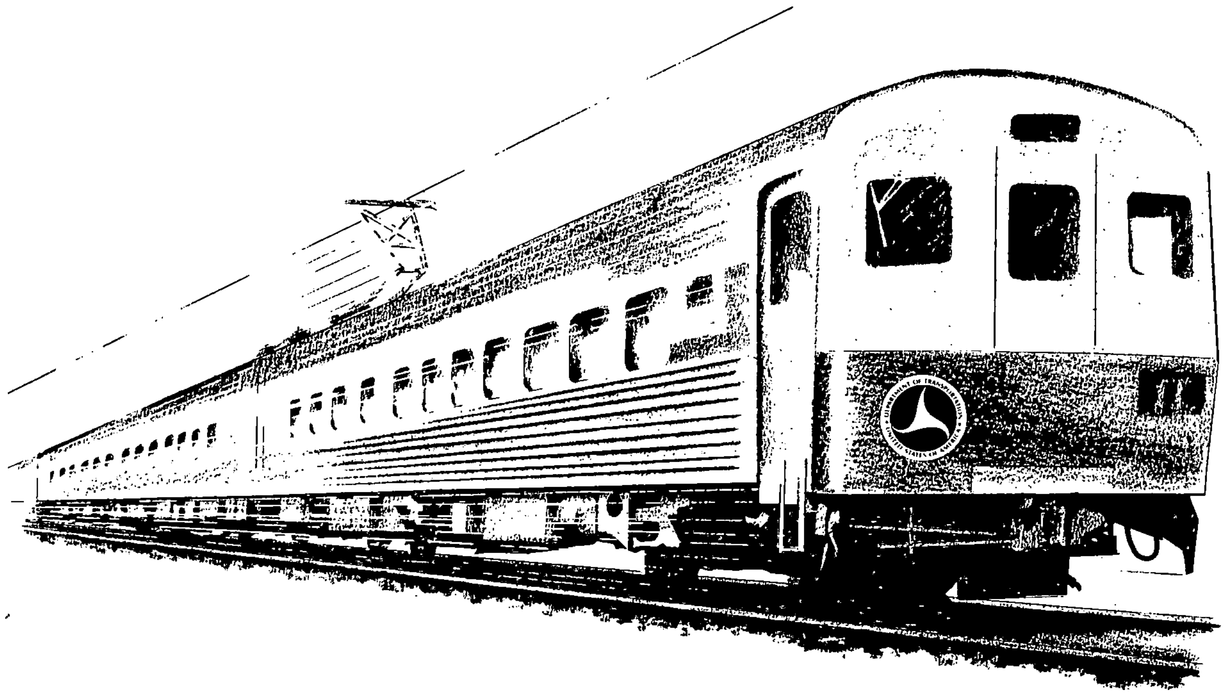
PER

EMBANKMENT SUPPORT FOR A RAILROAD TEST TRACK

DESIGN STUDIES



FINAL REPORT

AUGUST 1971

DEPARTMENT OF TRANSPORTATION
UNITED STATES OF AMERICA

FEDERAL RAILROAD ADMINISTRATION

OFFICE OF HIGH-SPEED GROUND TRANSPORTATION

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FRA-RT-72-07		

4. Title and Subtitle	5. Report Date
Embankment Support For A Railroad Test Track	August 1971
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
R.J. Dietrich and J.R. Salley	
9. Performing Organization Name and Address	10. Work Unit No.
Shannon and Wilson, Inc. San Francisco, California	11. Contract or Grant No.
	13. Type of Report and Period Covered
12. Sponsoring Agency Name and Address	
Rail Technology Division, Federal Railroad Administration and Atchison Topeka and Santa Fe Railway Co.	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

The events and considerations leading up to the production of an embankment design for the support of the Kansas test track are described. Included are discussion of:

Site description, field investigations, Laboratory investigations, sub-surface conditions, embankment design, and instrumentation.

17. Key Words	18. Distribution Statement
Geology, topography, vibration measurement, clay, bedrock, embankment stability, soil strain measurements.	Availability is unlimited. Document may be purchased from the National Technical Information Service, Springfield, Va. 22151, for $3.00 a copy.

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

Form DOT F 1700.7 (8-69)

TABLE OF CONTENTS

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

DESIGN STUDIES
HIGH SPEED TEST EMBANKMENT
AIKMAN-CHELSEA, KANSAS

INTRODUCTION

Background

The United States Department of Transportation (DOT) and The Atchison, Topeka and Santa Fe Railway Company (ATSF) are jointly sponsoring an investigation into methods of providing more stable railroad track structures for present and future operating conditions with high speed trains and heavily loaded cars. ATSF will provide support services for the investigation and will establish a test embankment on a main line of its system. Earlier studies had located the test embankment between Aikman and Chelsea, Kansas, on the ATSF main line. The site selection was influenced by the presence of abundant rail traffic, a long straight tangent section, uniform and relatively flat grades, a good performance record under main line traffic conditions and other factors. The test segment was considered to be reasonably typical of much of the country's railways, and the uniform soil conditions and gentle terrain were suitable for construction of a uniform test embankment at reasonable cost.

The project consists of an embankment and track structures constructed adjacent to and offset 30 feet from the existing ATSF main line. The test embankment will be nearly two miles long on a slight grade, and transition sections at each end will divert main line traffic onto it and then back to the main line. The embankment is to have nine test sections and each section will have a unique track support system. The support systems include concrete ties at four different spacings, a concrete slab, a continuous concrete beam, a precast beam, stabilized ballast, and a control section typical of conventional ATSF construction. Each test section will have identical embankment instrumentation. An access road adjacent to the test embankment will provide easy access for periodically reading the instrumentation and observing the test track.

Scope

Shannon & Wilson, Inc. was engaged to design the embankment for the test structures, observe construction, and instrument the embankment to provide soils data for evaluation of the various test track structures. Specifically the work includes 1) design of the embankment; 2) preparation of construction plans and specifications; 3) observation of construction; and 4) design, fabrication and/or procurement, calibration, and installation of embankment instrumentation. Track support structures and reading of the instrumentation are not within the scope of this contract.

This report concerns only the embankment design portion of the project. It contains the "pilot" geophysical measurements, field investigations, laboratory test results, and embankment and instrumentation design considerations that led to the preparation of construction plans and specifications. The plans and special provisions of the specifications are included as Appendix F.

Authorization

Shannon & Wilson, Inc. was selected by DOT and ATSF through acceptance of its technical proposal, and contractual arrangements were made by ATSF. Shannon & Wilson, Inc. was authorized to begin work on July 24, 1970, upon approval of Contract No. 34753 with ATSF.

SITE DESCRIPTION

Location and Topography

The test track is located between railroad mile posts 161 and 163 approximately 11.3 miles northeast of El Dorado, Kansas on State Route 177 or 7.2 miles southwest of Cassoday, Kansas. The railway-designated locations of Aikman and Chelsea are to the southwest and northeast, respectively of the site. The existing ATSF track lies parallel to State Route 177 and offset 134 feet south in the vicinity of the test section. The test track will lie between the highway and existing track at a constant distance of 30 feet from the track. A vicinity map is included as Fig. 1 and a plan and profile of the test embankment are included as Fig. 2.

The terrain of the site area is gently rolling with drainage to the south. The existing track grade drops about 35 feet along the section from northeast to southwest. The average site elevation is about 1,400 feet MSL. The track passes through minor cut and fill zones, and spoil piles are adjacent to the track in cut sections. The surrounding terrain is cultivated or in pasture and the site is covered with a dense growth of prairie grass.

Geology

Bedrock at the site is close to the surface and consists of the Fort Riley limestone formation of Permian Age. This limestone is of medium hardness although it is interbedded with thin clay-shale to shale layers. In some instances the shale occurs as laminae. The limestone is buff to gray in color. Fractures and jointing are quite common in this formation according to the Kansas State Geological Survey. The limestone is somewhat eroded on its upper surface and vuggy openings due to solution are common. The limestone bedding dips somewhat to the southwest, but less than one degree.

FIELD INVESTIGATIONS

General

During the site selection stage of the project before Shannon & Wilson, Inc. became involved, preliminary field investigations were performed by the Hemphill Corporation, Tulsa, Oklahoma, in April 1969. Results of the 16 borings by Hemphill were utilized in establishing and interpreting follow-on soil investigations. Under the present contract, field investigations to supplement the Hemphill investigation consisted of test pits and test borings along the proposed embankment centerline and in potential borrow areas. In addition, vibration measurements were made on the existing embankment.

Vibration Measurements

Vibration measurements were made simultaneously at each of four points at six selected locations on the existing embankment. The purpose of the measurements was to determine the general embankment response characteristics to rail loading to assist in establishing the required minimum thickness of the embankment and to determine the range of dynamic response for design of project instrumentation. The measurements were also to determine if varying embankment thicknesses responded differently and if any particular embankment thickness presented response problems.

The vibration measurements were made between July 7 and 10, 1970, using four triaxial transducers buried at 1) the centerline of the tracks at the base of the ballast; 2) the edge of the ballast; 3) midway down the embankment slope; and 4) at the toe of the embankment slope. The work was performed by Geo-Recon, Inc. under the direction of Shannon & Wilson, Inc. Measurements were made at cut sections and at fill sections of differing fill and overburden thicknesses. The transducers sensed particle velocities of the embankment vibrations induced by passing trains and the results were recorded on a light beam oscillograph chart recorder. Typical results are presented in Fig. 3 on sections through the test locations. Displacements computed from the measured frequencies and velocities are plotted in the figure. The complete report on the vibration measurements is included as Appendix A.

Field Explorations

Borings. Eleven borings (B-21 through B-30) were advanced at the site to depths ranging between 2.5 and 34.5 feet below existing ground surface at the locations shown on Fig. 2. Overburden materials were drilled and sampled to refusal on rock. No borings extended into rock. Detailed logs of the borings are presented in Fig. B1 of Appendix B. Boring B-22 encountered a boulder at a depth of 6.1 feet; consequently the hole was abandoned and Boring B-22a was drilled five feet north of Boring B-22.

Drilling was started on July 13, 1970, and completed on July 17, 1970. The drilling was accomplished under sub-contract to the Wichita Testing Laboratories of Wichita, Kansas. Holes were advanced with a truck-mounted B-40 Mobil auger using a four-inch diameter flight auger. All drilling and sampling was performed under the direction of a geologist from Shannon & Wilson, Inc.

Relatively undisturbed three-inch thin-wall tube samples of overburden soils were generally taken continuously in the borings. The sampling tubes were pushed into the soil with the hydraulic ram of the drill rig. The ends of each tube were classified in the field, sealed with wax, and the tubes were delivered to our laboratory for extrusion and testing.

Several samples were obtained by driving a two-inch O.D. split-spoon sampler into undisturbed material with a 140-lb. hammer falling freely a distance of 30-inches. The blows per foot (Standard Penetration Resistance) were recorded and are noted on the boring logs. Samples were classified in the field, placed in air-tight jars, and shipped to our laboratory.

<u>Test Pits</u>

Twenty-seven test pits were excavated in potential borrow areas and beneath the proposed embankment to depths ranging between two and 11 feet below the existing ground surface. Twenty-four pits were dug on July 14 and 15, 1970, at the locations shown on Fig. 2. Detailed logs of the test pits are shown on Fig. B2 through B4. Three additional test pits were dug near the spoil piles in the vicinity of mile post 161 on September 9, 1970. Logs of these pits are shown in Fig. B5.

Digging of the test pits was accomplished by a sub-contractor provided by Lewis & West, Inc., El Dorado, Kansas. The test pits were dug with a Davis D130 backhoe mounted on a Massey-Ferguson 2200 tractor. A 24-inch bucket was used.

The work was directed and the test pits were logged by an engineer and geologist from Shannon & Wilson, Inc.

Representative soil samples were taken from the test pits. Samples were classified in the field, placed in air-tight plastic bags, and shipped to our laboratory for further classification and testing. In addition a hand-operated torsional vane shear device was used to evaluate the shear strength along the sides of the test pits. Shear strengths determined in this manner are plotted on the test pit logs.

LABORATORY INVESTIGATION

General

Laboratory tests were performed to establish the engineering properties of in-situ and compacted soils in sufficient detail to support embankment design studies. The scope of the testing was not adequate to establish the full range of static and dynamic soil properties that may be required in the future analysis of track structure and embankment performance. It is anticipated that further laboratory testing may be required in the future, depending upon the requirements for dynamic response studies.

Scope of Testing

All samples received in the laboratory were visually classified and index property tests were performed on representative materials. Strength tests were performed on selected undisturbed samples. Suitable embankment materials from potential borrow areas were subjected to compaction tests, and strength, swelling, and shrinkage tests were performed on the compacted specimens. These tests on compacted samples provided basic design data for the embankment materials. A series of tests was also performed to determine response of the natural and compacted soils to static and dynamic loading. Standard laboratory consolidation tests, vibration tests, and pulsating load tests disclosed the response of the materials to various rates and levels of loading. This information will be of use primarily for interpretation of embankment response.

Finally, basic chemical and mineralogical tests were performed to determine if embankment materials would provide a favorable environment to instrumentation.

Descriptions of laboratory test procedures and test results are contained in Appendix C. Sample descriptions, test assignments, and test results are tabulated in Table 1.

Tests on Compacted Samples

Engineering properties of compacted specimens from one medium and two highly plastic clay samples were evaluated. Modified AASHO compaction tests were performed on the three samples, and selected compaction test specimens were tested in unconfined compression, swell, and shrinkage. The purpose of the tests was to determine how these parameters for potential embankment materials varied with moisture content and density. The test results are plotted in Fig. 4 versus moisture content relative to optimum moisture content for each sample.

The shrinkage test was performed by permitting a cylindrical specimen to air dry and measuring the volume change. Swell test specimens were placed in consolidometer rings, given a small surcharge of 0.063 Kg/cm^2, submerged in water, and allowed to swell until volume change was essentially complete. Percentage swell at 3,000 minutes test duration are plotted in Fig. 4. Strength and swelling increase and shrinkage decreases as moisture contents decrease. At optimum moisture content, strengths averaged 11 Kg/cm^2 and percent volume change due to swelling at 3,000 minutes averaged +15 percent. Based on very limited data, percent volume change due to shrinkage at optimum moisture content is expected to be about -15 percent.

Initial tangent moduli were determined for the unconfined compressive tests summarized in Fig. 4. The moduli were also plotted against difference in sample and optimum moisture contents and the results are contained in Fig. 5. The moduli for the three series of tests on compacted specimen fall in a band having decreasing moduli with increasing moisture content. At optimum moisture content, the band of initial tangent moduli from the unconfined compressive test specimens ranged from 240 to 880 Kg/cm^2.

Additional descriptions of the test procedures and test results are contained in Appendix C.

Chemical and Mineralogical Analyses

Sample 4 from Test Pit 1, a typical reddish brown highly plastic clay, was subjected to chemical and mineralogical analyses to see if there were any unusual characteristics of the material. Mr. James K. Mitchell, Consulting Geotechnical Engineer, supervised the testing. His report is contained as Appendix D. He determined that the sample contains 40 to 50 percent clay minerals of which montmorillonite is predominant followed by hydrous mica, and then kaolinite. The sample contained no carbonate so it is suspected that the soil was formed by weathering of a shale deposit rather than limestone. Mr. Mitchell concluded that there was nothing unusual about this highly plastic clay and that it appeared to contain no chemicals deleterious to the instrumentation that would be placed in the embankment composed of such clays.

SUBSURFACE CONDITIONS

General

The soil conditions along the test track alignment consist of a shallow relatively uniform soil stratum overlying rock. The inplace overburden generally varies in thickness from two to six feet and consists predominantly of clay of high plasticity. The surface of the overburden generally is of medium plasticity to a depth of one or two feet and the material immediately overlying rock is generally clayey silt.

The natural conditions along the alignment have been altered in certain areas by excavation to provide fill for the existing embankment and by spoil piles of excess excavation from shallow cuts for the existing grade.

Based upon the results of borings by Hemphill Corp. and borings and test pits by Shannon & Wilson, Inc., the soil conditions shown in the soil profile on Fig. 2 have been identified. Interpretation of the laboratory tests for the materials involved are discussed below. The soil properties are summarized in Table 2; soil test results are contained in Appendix C.

Medium Plastic Clay (CL)

Generally the upper one to two feet of soil at the site consists of stiff gray to brown silty clay (CL). As shown in Fig. Cl, the liquid limits of this material range from 30 to 50. This material was formed by weathering from the underlying highly plastic clay, and its color has been altered by oxidation and by organic materials from the vegetation it supports. The lower plasticities of this material were probably caused by leaching of clay minerals by percolating groundwater. Previous construction for the railroad and highway have caused this layer to be covered or removed over portions of the site. Water contents varied from about 15 percent to 28 percent. The water contents were generally the lowest at the ground surface, because of the dry season (July 1970), and increased with depth. The average water content of the samples was 23.3 percent. The shear strength of this material was directly influenced by its water content; stiffer soils had lower water contents. The one compaction test performed developed a maximum density of 111.5 pcf at an optimum water content of 15.6 percent.

Highly Plastic Clay (CH)

The predominant material at the site is a very stiff reddish brown clay (CH) of high plasticity which overlies rock and is capped by the medium plastic clay. This material has liquid limits between 50 and 90 and the Atterberg limits plot above the A-line in the plasticity chart, Fig. Cl. As determined by minerology analysis, the predominant clay mineral in this material is montmorillonite which accounts for its high plasticity. Water contents of the material in-situ range from 17.3 to 32.2 and average 24.5 percent. Other measured properties of the material are summarized in Table 2. This stratum varies from being absent in some areas to 34.5 feet thick at Boring B-25. Four compaction tests had maximum dry densities ranging from 97.0 to 107.3 pcf and averaging 103.4 pcf. Optimum water contents ranged from 19.3 to 24.3 percent and averaged 21.0 percent.

As this material occurs naturally at an average water content of 24.5 percent and its optimum water content is 21.0, some drying of the material will probably be required for its satisfactory placement as embankment.

Tan Clayey Silt (ML-CL)

A one to four-foot layer of tan clayey silt (ML-CL) lies between the reddish brown clay and bedrock. This material was probably developed by weathering of a siltstone. In some locations the material has softened due to the presence of perched water. In other locations the material is very hard and friable. The average in-situ water content is 28.0 percent although the range is 13.7 to 46.6 percent.

Limestone Bedrock

During its investigations, Hemphill Corporation obtained several cores of the limestone. They reported the limestone was dense, buff to gray, and of medium hardness. It is often interbedded with thin clay shale to shale layers. Vuggy openings due to solution activity were observed in the upper rock portions of Borings 3, 7, 12, and 16. Vertical fractures were found in the cores from Borings 2, 13, and 14. The rock is generally weathered in the upper 6 to 12 inches and becomes corable just below the weathered zone.

The limestone bedrock is essentially flat laying with a southwest dip of 0.3 degree. It occasionally outcrops. A discontinuity is suspected near Station 8566 since rock (bedrock) not confirmed) was encountered in Boring B-25 at a depth of 34.5 feet. Based on nearby Borings B-9 and B-19, one would antipate rock to be eight feet deep at Boring B-25.

Groundwater

No definite groundwater table was discovered in the investigations. There were instances where depressions in the bedrock surface trapped percolated surface water and caused a softening of the tan clayey silt. Hemphill interviewed a water well drilling firm active in the area which reported the water table at a depth of approximately 180 feet.

Spoil Piles

Construction of the existing mainline track resulted in shallow cuts in the vicinity of Stations 8525 and 8585. The excavated material was piled on both sides of the track as it was excavated. Consequently, clay forms the base of the piles, tan silt follows and limestone fragments are generally found at the tops of the piles. The limestone and silt on top of the piles may also result from cleaning of drainage ditches along the track bed. The clay in these piles is generally a mixture of the gray to brown silty clay (CL) and reddish brown clay (CH). Properties of the component materials are summarized in Table 2.

EMBANKMENT DESIGN

General

The principal design objective was to produce uniform support of good quality for the test track structures. It was required that this objective be reasonably attainable using present construction methods, equipment, and locally available materials. It was also considered important to reduce to the absolute minimum the number of variables which might complicate the interpretation of track structure data. These requirements are necessary in order to facilitate a comparative analysis between the different test track structures and to make the findings relevant for application to existing road beds or to road beds that may be constructed in the foreseeable future.

The standard ATSF embankment section was selected for the test track. This section has proven to be satisfactory based on extensive experience, and the relative advantages of slight modifications in embankment geometery are not readily discernable. The use of the standard section also affords the opportunity to compare the test section with the long-term performance of existing road beds.

The principal design decision concerning embankment geometry was to establish a minimum thickness of embankment within reasonable cost which would provide essentially uniform support and dynamic response to the track loading. This was required regardless whether the embankment was founded on rock or overburden. A profile of the test track embankment is shown in Fig. 6 with alternate embankment thicknesses of four, six, and eight feet indicated.

The studies leading to the embankment design included "pilot" vibration measurements on the existing main line, theoretical analysis of static stresses, stability analyses and earthwork quantity studies for different depths of embankment.

Embankment Vibration Measurements

Typical results of the "pilot" vibration measurements are noted on the appropriate cross section in Fig. 3. The measured displacement at the track centerline is plotted versus embankment thickness in Fig. 7. The vertical displacements appear to peak at an embankment thickness of three feet and then decrease to a negligible amount at a thickness of eight feet. However, the horizontal motions tend to increase with increasing thickness of embankment. The maximum absolute horizontal displacement for an eight-foot embankment is still quite small compared to maximum vertical motions. Also plotted in Fig. 7 for general information is the relative amplitude versus embankment thickness at a 16-foot distance from the centerline. Relative amplitude is defined as the amplitude at any position on the embankment compared to the amplitude at the track centerline.

At a thickness of three feet the displacements are at a maximum. Although the full significance of this observation is not immediately discernible due to the limited data available, it appears desirable to avoid this condition and select an embankment thickness well over three feet to minimize the vertical response of the subgrade. Embankment thicknesses less than four feet were generally not considered due to the wide variation in thickness of fill and natural overburden that would result over the test track using this thickness (see Fig. 6), and the greater possibility of undesirable reflections or concentrations of stress due to the relatively shallow rigid rock boundary. The use of an embankment four feet in thickness or less would also place severe restraints on the amount and type of instrumentation that could be provided within the embankment.

An embankment thickness of five or six feet would appear to give a reasonable range of vertical centerline deformations and an acceptable range of horizontal motion 16 feet from the track centerline. It should be noted that combined thicknesses of embankment plus natural overburden will exceed six feet in some areas of low bedrock, and this is unavoidable unless the entire embankment is extended uniformly to rock.

Embankment Stress

Theoretical embankment stresses corresponding to static conditions have been calculated using several elastic stress solutions. In Fig. 8a vertical stresses beneath the center of the track are plotted versus depth below the tie and in Fig. 8b the vertical stresses beneath the rail are plotted. Several approximations were used to represent train loads exerted on different track structure configurations in order to bracket the intensity of stress. The ties were alternately considered to be line and strip loads and the wheels were also considered to be point loads. Stresses were calculated by Boussinesq, Westergaard, and Burmister techniques for a locomotive, passenger car, and an empty hopper. Burmister's techniques were chosen to simulate the effect of a rigid base at a depth of five and seven feet below the tie. The stresses are significantly increased by the rigid base with the effect being only slightly more pronounced for the shallow embankment. The Boussinesq procedure is for a homogeneous isotropic linearly elastic material. The Westergaard solution is similar except it is assumed that the horizontal strains are equal to zero.

The maximum calculated stress at the top of the embankment occurs beneath the rail for locomotive loading, and it is nine tons per square foot. The stresses dissipate rapidly so that at three feet below the tie (two feet below top of subgrade) the maximum stress is about one ton per square foot. The stresses due to a moving train are expected to be less than the corresponding static stresses.

Earthwork Quantity Studies

The earthwork quantities for four and six-foot minimum thickness embankments were computed in order to study the relation between embankment thickness and construction cost. The quantity studies were based on soil and rock data from the test pits and borings shown earlier in this report. Forty-four cross sections at spacings varying from 65 feet to a maximum of 500 feet were used in computing the quantities by the prismoid formula which is as follows:

$$V = L/3 \; (A_1 + \sqrt{A_1 \;\; A_2} + A_2)$$

where V = Volume of material between stations 1 and 2

L = Distance between stations 1 and 2

A_1 = Area of cross section at station 1

A_2 = Area of cross section at station 2

A computer program was written for this formula. Surveyed coordinates of breaks in the terrain were used as input and the program calculated end areas and volume.

The computed quantities were divided into the categories defined below:

Common excavation is the total of reusable soil, stripping, and spoil which must be removed for the test embankment, transition sections, and borrow areas.

Rock excavation is the total rock excavation for the test embankment and transition sections.

Embankment fill is the total volume of embankment to be placed.

It was assumed that six inches of top soil would be stripped from the surface of all clay excavation to be reused as fill. It was also assumed that the tan silt layer which existed immediately above the limestone bedrock would not be suitable for use as fill and would be wasted. No surface stripping was assumed in areas where the unusable silt appeared at the ground surface.

The quantities and corresponding estimated earthwork construction costs are summarized on Fig. 9 for the embankment thicknesses shown in profile on Fig. 6. The quantities for four and six-foot embankments were computed and the quantities for an eight-foot embankment were approximated by rough calculation and extrapolation. The total earthwork cost is roughly proportional to the thickness of embankment.

Embankment Stability

The maximum loading at the top of the embankment will fall between the theoretical embankment stresses beneath a rail for a 206-ton locomotive on six axles as shown in Fig. 8. The range of stresses for line and point loading assumptions are 2.4 to 9.0 tsf, respectively, but a value of 4.0 tsf (or Kg/cm^2) appears reasonable as a maximum design stress. Assuming a safety factor of 2.5 against a bearing capacity failure, an unconfined compressive strength of 3.9 Kg/cm^2, (say 4 Kg/cm^2), is required in the embankment material. This minimum strength requirement has been plotted in Fig. 4 on the correlation plot of strength versus different material and optimum water contents. It is apparent from the tests on modified AASHO compaction specimens that the moisture content must be less than about four percentage points above optimum moisture content to insure adequate strength. However, the test fill placed in Fall 1970 revealed that field compaction produced densities slightly less, for a given water content, than modified AASHO compaction. Laboratory tests on samples from the test fill and an additional laboratory study conducted to determine strengths under simulated field compaction energy are summarized in the field curve of Fig. 4. The curve shows that to develop the minimum required strength in the field the soil must be compacted at a water content less than optimum plus 2.0 percent. A relative compaction greater than 92 percent is also a minimum criterion resulting from these studies. This expected strength versus moisture content relationship for the field will be verified by testing during placement of the embankment. Since the embankment should be as uniform as possible, the requirements also apply for the complete embankment.

The existing foundation soils and rock are adequate to support the embankment without significant long-term settlements or lateral deformation.

Final Embankment Design

Thickness. A six-foot minimum embankment thickness was adopted. This decision was reached primarily on the basis of the "pilot" vibration measurements which suggested a need for an embankment well in excess of three feet to minimize vertical embankment response and on the basis of engineering judgement. The judgement factor considers the results of the static stress calculations which indicated an appreciable increase in embankment stress due to a shallow rock boundary and suggested that the embankment should be as deep and as uniform in thickness as possible. The limiting constraint on thickness of embankment was cost. By inspection of Fig. 6 it can be seen that a four-foot minimum thickness would result in considerable variation in thickness over the test track segment. A six-foot minimum thickness provides a reasonably uniform test section and very little additional benefit in uniformity is achieved by increasing the depth to eight feet. The additional cost ($100,000) of an eight-foot thickness compared to a six-foot thickness does not appear warranted.

Materials. Suitable embankment materials include the gray to brown silty clay (CL) and the reddish brown clay (CH). The tan clayey silt (ML-CL) is available in relatively small quantities and is so different in properties from the clays that it should not be used. However, small quantities mixed with the clays by normal construction operations will be acceptable.

Placement water content should be on the wet side of optimum to facilitate placement and control of uniformity, but the strength requirements limit the placement to about two percentage points above optimum. The material occurs naturally wet of optimum, so it would be desirable to minimize drying to facilitate production. Assuming that the eight compaction tests performed are representative of the site soils, the average optimum water content is 19.7 percent and the average in-situ moisture content is 25.8 percent, for a difference of 6.1 percent. Therefore, this material will have to be dried about four percent before it can be suitably placed. However, the natural water content may increase slightly by heavy rains, and additional drying could be required.

A test fill was placed at the start of construction to determine the most suitable field procedure for achieving a uniform, competent embankment under the existing climatic conditions. Laboratory testing and field operations in the fall of 1970, have established a minimum requirement of 92 percent relative compaction (modified AASHO) at a moisture content not exceeding two percent above the optimum water content. At this placement water content, the material will approach saturation at 92 percent relative compaction, and further compactive effort beyond this point in the field will produce little increase in dry density unless the compaction operations produce drying of the soil and/or the weight of the roller is increased. When construction is resumed in Spring 1971, we will review the compaction criteria taking climatic conditions into consideration. Six-inch maximum compacted lift thicknesses will be used.

Broken limestone is present on top of the existing spoil piles and it is not acceptable for use in the test embankment. Placement of instrumentation during and after embankment construction would be hindered by the presence of gravel-sized particles or larger. Performance of the instrumentation could also be adversely influenced by the presence of large granular particles. A limited amount of granular material is permissible in the transition sections at each end of the test track because these sections contain no instrumentation. Medium to high plasticity clays without gravels are designated Material A for use in the test sections of the embankment. These highly plastic clays with scattered cobbles to four inches in diameter and/or with quantities of tan clayey silt are acceptable for use in the transition zones and shall be designated Material B. Acceptance of Material B will reduce required borrow quantities and reduce waste quantities.

Field Observations

Close construction control and extensive testing will be provided during construction to assist in achieving a uniform, competent embankment. For planning purposes the following table lists the type and anticipated frequency of embankment testing:

Test	Frequency of Testing(1), cy	Total Tests
Moisture content	100	620
In-situ field density	200	310
Laboratory compaction test	1,000	62
Field plate load test	600	100

Typical excavation and embankment sections with a six-foot embankment are shown in Fig. 10. The road was included to provide access for service vehicles and for reading embankment and track structure instrumentation. The access road will be founded on the existing bedrock surface in cut areas to reduce the required rock excavation. The cross section dimensions are from ATSF Standard Specifications for Earthwork and Structures. A set of Special Provisions to the Standard Specifications for the project are included as Appendix F along with a complete set of plans.

Anticipated Embankment Properties and Performance

Properties. The embankment properties have been predicted based on laboratory testing and the proposed placement criteria. At 92 percent relative compaction and at two percent above optimum moisture content, the properties tabulated in Table 4 are anticipated. Properties of the existing overburden, which will be present beneath the embankment in fill areas, are summarized in Table 2. Comparative values of elastic moduli from studies on undisturbed overburden materials and a compacted specimen are summarized in Table 1C. These studies include

(1) Frequency of testing is indicated as one test performed for the indicated number of cubic yards of embankment placed.

vibration tests and pulsating load tests for determining dynamic moduli and unconfined compressive tests for static moduli.

Performance. The test embankment is expected to perform favorably with no more maintenance required than is normal for this section of track. The embankment material is potentially highly expansive, but standard drainage has been provided to preclude ponding of water on and against the embankment. Swelling is expected to be minimal and non-damaging. If the embankment were to remain exposed over the winter of 1970-71, it would have been over-built by 0.2 foot of elevation to reduce the possibility of softening the designed top of subgrade. The excess material would have been removed prior to installation of vertical instrumentation.

Shrinkage of the embankment is also a potential problem. However, based on the test pits dug in the summer of 1970 after an extended dry priod, drying of the soil is not significant below a depth of about six inches. Likewise, no major adverse effects due to shrinkage are expected.

Spreading of the embankment due to static and dynamic loads is also considered unlikely. At the top of the embankment there will be a safety factor of about 2.5 against a bearing capacity failure for the maximum train loads based on the anticipated strength of the embankment. Below a depth of 1.0 foot in the embankment, the dynamic stress level will be less than 2.0 tsf. The maximum static stress due to the weight of the embankment will not exceed 0.5 tsf. Compared to the anticipated average unconfined strength of 4.0 tsf (or Kg/cm^2), the embankment stresses will be quite low. The significant stress for analysis of horizontal spreading is the static stress of the embankment. It is so low compared to the embankment strength, that spreading will be nominal and is not a design consideration.

INSTRUMENTATION

General

The response and performance of the track structures will be influenced by subgrade performance. Therefore, it is necessary to observe the embankment behavior in order to evaluate track structure performance. A system of embankment instrumentation has been developed for this purpose.

Embankment stress and strain under dynamic loading are of primary interest. However, it is very difficult to obtain reliable stress measurements so emphasis is placed on obtaining reliable measurements of embankment strain. Long-term deformations and volume changes of the embankment are also of interest in this highly plastic clay embankment.

The required operational life of the instrumentation has not been definitely established. However, designs are based on an effective life of at least five years under the site climatic conditions. Where physically possible, the instruments are designed to permit maintenance or component replacement.

Nine individual test sections will be provided in the test track. Each test section will have one principal instrument array. The main arrays will be supplemented by additional instruments spaced throughout the test section to verify that the performance of the embankment at the main array is typical of that particular test section.

The main array has been positioned near the west (downgrade) end of each test section on the premise that measurements will be made principally under west-bound rail traffic. This provides additional length of track for damping of non-uniform response which may develop between different test track structures before the main array is reached by west-bound traffic.

The embankment main array instrumentation includes vertical extensometers, portable horizontal extensometers which will be inserted in horizontal tubing embedded in the embankment, pressure cells,and moisture-temperature cells. With the exception of the moisture-temperature cells, the instrumentation has been designed specifically for this project. Wherever possible existing equipment has been adapted or stock components utilized. The instruments are currently in various stages of testing and calibration,and it is possible that details may change before delivery. No change in concepts are anticipated.

The instrumentation is illustrated in Fig. 10 and summarized in Table 5.

Measurement of Embankment Strains

General. The determination of embankment performance requires a knowledge of both permanent and transient deformations including the time-history of these motions. In the case of permanent deformations the deformation must be measured directly. In the case of transient motions it is possible to measure acceleration, particle velocity or deformation as a function of time to establish embankment response. The use of accelerometers or velocity meters was considered, but this type of instrumentation did not appear attractive because of the high capital cost of recording equipment. Very sophisticated recording equipment is required in order to provide the fidelity necessary to compute embankment deformations. This type of instrumentation would appear to be feasible only if the recording equipment were available from other sources at a reduced cost, or if the recording equipment were being utilized for the track structure instrumentation. Since it was not possible to determine the availability of this equipment, it was decided to adopt a strain measuring system utilizing LVDT transducers for sensing permanent and transient deformations. This equipment will measure displacements directly and recording equipment may consist of digital readout or pen or oscillograph type chart recorders. This equipment is available at a comparatively low cost. If desired a more sophisicated recording system may also be utilized with the LVDT sensors.

Vertical extensometers. Vertical embankment strains relative to the surface of the subgrade will be measured with vertical extensometers anchored in rock and at intermediate points within the embankment. All permanent strains will be referenced to the extensometers anchored in rock. Vertical holes will be drilled through the embankment and into rock following embankment construction. Anchors will be inserted into the hole and fixed at three or four levels by either grouting in rock or hydraulically expanding prong anchors into the soil. An LVDT transducer will be positioned immediately above the anchor point and a steel rod riser will extend to a fixed-point in a terminal box near the surface of the subgrade. Three anchors will be placed within the embankment as shown in Fig. 10 and, if the rock is present within 12 inches of the embankment base, the lowest embankment anchor shall be eliminated.

The pilot vibration measurements suggested that the maximum amplitudes at the surface of the subgrade may be on the order of 0.030 inch at a frequency of three cycles per second (cps). Measured frequencies ranged from three to 100 cps. It appears that motions of interest will range from 0.0008 to 0.04 inch amplitude and zero to 100 cps. The sensitivity and frequency of the measurements will be controlled by the recording equipment. For pen-type recorders the sensitivity will be in the order of 0.0003 inch at a frequency of zero to 100 cps. For frequencies above 100 cps an oscillograph recorder would probably be required.

A more complete summary of vertical extensometer design and performance criteria is presented in Appendix E.

Three multi-position extensometers will be placed in each main instrument array. One will underlie the center of track, the second will underlie a rail, and a third will be placed at the side of the embankment, four feet from the rail. These extensometers will be placed in the same cross-section of the embankment. In addition four single-position extensometers will be installed in each test section, all beneath the track centerline. Three will be spaced at 100-foot intervals uptrack (east) of the main array and one will be 100 feet downtrack (west) of the main array. Stationing of the instruments within the test section was determined by placing the furthest west vertical extensometer in each section a distance of 84 feet from the section end. This criterion was established by DOT so that if a different track response developed beyond the end of the test section, the longest rail car, 84 feet, would not transmit vibrations back through the car to the extensometer.

Horizontal extensometers. At each main instrument array, horizontal 4-inch diameter corrugated polyethelene tubing will be placed in the embankment at four levels during construction as shown in Fig. 10. The tubing will have PVC couplings at 2.5 and 5-foot spacings which are anchored in the embankment as also shown in Fig. 10. Strain rods with a hooking device to engage the anchored couplings will be used to measure the static horizontal deformation of the embankment. One set of strain rods will be used for measurement of all tubings. No absolute reference is provided, and all measurements will be relative to the end of the tubing.

Dynamic horizontal deformations in the tubings will be measured with portable extensometers with gage lengths of 2.5, 5.0, and 10 feet. Expandable anchor shoes at the ends of the extensometers will extend and lock into the tubing couplings. An LVDT at one end of the extensometer will record dynamic deformations caused by passing rail traffic.

The discussion of sensitivity of the LVDT transducers for vertical extensometers also applies to horizontal extensometers. A summary of design and performance criteria is presented in Appendix E.

The horizontal tubing also provides openings in the embankment which would be available for insertion of other types of instrumentation if desired at some future date. For instance, it may be advantageous to insert portable accelerometers or other types of transducers to obtain data for direct correlation with the track structure instruments.

Pressure Cells

Three pressure cells will be placed in the upper portions of the embankment in each main array to measure stresses. The cells will be placed as shown in Fig. 10 during embankment construction. The range of expected stresses are shown in Fig. 8. The cells will be fluid-filled stainless-steel, flat cells approximately six inches in diameter and 0.5 inch thick.

Both upper and lower surfaces of the cells will be relatively flexible. An LVDT pressure transducer will be attached near to the cell and electrical leads will extend to an external terminal box for readout purposes.

The sensitivity and range of the pressure cells will be controlled by the LVDT pressure transducers and recording system. Pressure transducers of zero to 25, 50, and 100 psi have been provided depending on the position of the cell with respect to the rail and depth below subgrade. A sensitivity of about 0.1 psi will be available, and for a pen type recorder the frequency response will be zero to approximately 100 cps.

Moisture-Temperature Sensors

Thirteen moisture-temperature sensors will be placed in each main array at the locations shown in Fig. 10. The cells will provide qualitative and possibly quantitative information on the variation of water content and temperature in the embankment with time. This information will be used to correlate with measured strains in the embankment and other aspects of embankment and track structure performance. A SoilTest MC-300A moisture meter will be used with SoilTest MC-310 moisture-temperature cells. A laboratory study is being conducted on this equipment to determine its range and sensitivity.

RUDY J. DIETRICH
LICENSED
6112
KANSAS
PROFESSIONAL ENGINEER

SHANNON & WILSON, INC.

Rudy J. Dietrich

Rudy J. Dietrich

J. Ronald Salley

J. Ronald Salley

REGISTERED PROFESSIONAL ENGINEER
James Ronald Salley
No. 19,222
CIVIL
STATE OF CALIFORNIA

TABLE 1

SUMMARY OF TEST DATA

LEGEND:

CH, CL, ML	Soil classification symbols for the Unified Soil Classification System.
SP. Gr.	Specific gravity.
SL	Shrinkage limit, percent.
Compaction	Modified AASHO compaction test, pcf.
Compact. Studies	Compaction test plus swelling, shrinkage and Q_u tests on selected compaction samples.
Chem. & Min.	Chemical and mineralogical analyses.
Consol.	Consolidation test.
Grain Size	Grain size tests performed with a hydrometer.
Vib.	Vibratory triaxial tests.
Puls.	Pulsating triaxial tests.
Q_u	Unconfined compressive strength tests, Kg/cm^2 (roughly equals twice the shear strength of saturated clays).
Q	Undrained triaxial compressive strength test, Kg/cm^2.
$\bar{Q}$	Q test with pore pressure measurements, Kg/cm^2.
$\bar{R}$	Consolidated, undrained triaxial compressive strength test with pore pressure measurements.
TV	Shear strength as determined by a small hand-operated Torsion shear device, tsf.
PP	Pocket penetrometer results, approximately equals Q_u, tsf.

Note: Kg/cm^2 are considered equivalent to $tons/_{foot^2}$ (tsf).

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterberg Limits LL	Atterberg Limits PL	Atterberg Limits PI	Insitu γ Dry PCF	Other Test	Classification
TP1	8585+00 (61'W)	1406.3	1	See Test Pit Logs	29.8 26.2	27	21	6			Med. tan silty CLAY sm ls. fgmts. (CL-ML)
			2		24.9 23.0	69	18	51		Sp.Gr=2.65 Grain Size Compact Studies	Vy. stiff red-brn CLAY tr. org. clay (CH)
			3		29.3 28.0	33	18	15			Med. blk-brn Silty CLAY (CL)
			4		27.2 26.1	72	16	56		Compaction 106.7@19.6% Chem+Min.	Med. red-brn CLAY sm. blk. org. clay, tr. roots (CH)
			5		19.8 18.0	62	19	43			brn. CLAY tr.Ls.fgmts. & roots (CH)
			6		25.5 26.4						red-brn. CLAY sm. tan silt, tr.wthrd Ls fgmts. (CH)
TP2	8582+00 (46'W)	1404.7	1		23.9 24.5	74	16	58		Compact Studies	Vy.stiff red-brn CLAY tr. blk.org.clay (CH)
			2		27.9 27.1	48	16	32		Compaction 111.5@15.6% Compact Studies	Med.blk-brn silty CLAY, tr. org. mtl. (CL-CH)
			3		23.2 24.1 19.6				Compacted Specimens	(Vib. (Puls. (Q_u=5.4 ((Q_u=4.2	Stiff red-brn CLAY tr. roots, org. clay & limestone fgmts.(CH)

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterberg Limits LL	PL	PI	Insitu γ Dry PCF	Other Test	Classification
TP2	8582+00 (46'W)	1404.7	4	See Test Pit Logs							Hard dense gry. to buff, slightly wthrd. SILICIFIED LIMESTONE
TP3	8582+00 (87'W)	1401.8	1		30.9 40.1						Med. tan & gry. clayey silt & silty CLAY (CL-MH)
			2		26.0 26.8	78	16	62		Compaction 107.3@19.3%	Vy. stiff red-brn CLAY tr. roots & blk. org. clay (CH)
TP4	8572+00 (70'W)	1398.0	1		15.1 15.2						brn. CLAY tr. roots (CH)
			2		24.5 25.2						Stiff red-brn. CLAY, tr. roots & blk. org. clay (CH)
			3		33.4 30.3						Vy. stiff tan & brn. clayey silt & silty CLAY sm. wthrd Ls. tr. roots & f-m sand (CH to ML)
TP5	8566+00	1399.5	1		30.0 25.6						Stiff brn & blk CLAY (CH)
			2		20.7 21.1	50	18	32			Vy. stiff red-brn. CLAY, tr. Ls. fgmts. (CH)

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterberg Limits LL	PL	PI	Insitu γ Dry PCF	Other Test	Classification
TP6	8555+00 (30'W)	1398.9	1	See Test Pit Logs	22.8 26.9						Stiff brn. CLAY tr. org.mtl.,Ls fgmts. & roots
TP7	8538+00 (78'W)	1412.4	1		17.1						Vy.stiff brn. CLAY tr. roots (CH)
			2		14.2						Vy. stiff gry-brn. CLAY slight tr. f sand (CH)
			3		21.0 21.9						Vy.stiff gry-brn CLAY tr. roots (CH)
TP8	8518+00 (72'W)	1417.0	1		23.1 22.8	86	25	61		Compaction 97.0@24.3%	Stiff gry-brn. CLAY tr. roots
TP9	8520+00 (75'W)	1419.7	1		24.9 23.2	84	21	63		Grain size Sp.gr=2.54	Vy. stiff gry-brn. CLAY tr. roots (CH)
TP 10	8535+00 (50'W)	1412.6	1		30.4 32.6						Stiff brn. CLAY (CH)
			2		13.3 15.5						gry-tan SILTSTONE sm. clayey silt (GM)
			3		29.0						Stiff gry-brn. CLAY (CH)
TP 11	8530+00	1416.8	1		32.2	82	21	61		Compact Studies	Stiff red-brn CLAY (CH)
			2		21.0						Stiff blk-brn CLAY tr. roots (CH)

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterberg Limits LL	PL	PI	Insitu γ Dry PCF	Other Test	Classification
TP 12	8525+00 (84'W)	1418.4	1	See Test Pit Logs	27.1 25.5	80	18	62		Compaction 102.8@21.0%	Vy. stiff brn. CLAY tr. roots & m sand (CH)
TP 13	8525+00 (62'W)	1420.8	1		23.3 24.0						Stiff gry-brn CLAY, tr. Ls. fgmts & roots (CH)
			2		26.7 26.3	82	19	63		Compact studies	Vy. stiff brn CLAY (CH)
TP 14	8525+00 (30'W)	1417.3	1		24.2 28.6						Stiff tan clayey SILT & silty CLAY tr-sm Ls fgmts, tr.brn clay & roots (CL-ML)
TP 15	8520+00 (30'W)	1417.6	1		23.9 23.4	74	20	54			brn CLAY tr. roots (CH)
TP 16	8517+00 (30'W)	1416.8	1		14.8 13.7						Hard tan SILT sm. siltstone fgmts. (ML)
TP 17	8572+00 (30'W)	1395.6	1		22.2 22.6						red-brn CLAY tr org. clay, Ls fgmts & roots (CH)
			2								Hard dense gray to buff, slightly weathered SILICIFIED LIMESTONE
TP 18	8544+50 (30'W)	1406.1	1								Hard dense gray to buff slightly weathered SILICIFIED LIMESTONE

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterberg Limits LL	PL	PI	Insitu Dry PCF	Other Test	Classification
TP 19	8538+00 (30'W)	1415.4	1	See Test Pit Logs	24.0 26.9						gry-brn CLAY, tr. roots (CH)
TP 20	8530+00 (57'E)	1430.1	1								brn CLAY tr.wthrd Ls (CH)
TP 21	8535+00 (56'E)	1423.3	1		25.0 25.1						Vy. stiff red-brn CLAY (CH)
TP 22	8582+00 (50'E)	1402.1	1		22.4 17.9						Stiff gry-brn silty CLAY (CL)
			2		24.7 22.7						Vy. stiff red-brn CLAY (CH)
TP 23	8585+00 (50'E)	1401.3	1		19.7	72	18	54		Compaction 107.6@19.5%	Vy stiff brn CLAY (CH)
TP 24	8585+00 (133'E)	1399.2	1		25.4 24.9						Vy stiff red-brn CLAY tr blk f. sand (CH)

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterberg Limits LL	PL	PI	Insitu γ Dry PCF	Other Test	Classification
B21	8995+02.5 (35'W)	1384.4	1	05-2.0	28.0						Med. brn. CLAY, trace ls. fgmts. & organic material (CH)
B22	8579+00 (37'W)	1398.3	1	08-2.3	18.2						Stiff brown CLAY trace roots (CH)
			2	3.0-4.5	17.4						Stiff mottled brown tan and black CLAY (CH)
			3	5.5-6.1	36.3						Top: Stiff mottled brown tan and black CLAY (CH)
					46.6						Bot: Stiff tan & light grey silty CLAY & CLAY tr. gravel (Weath. ls) (CL-CH)
B22a	8578+95 (37'W)	1398.3	1	0.5-2.3	17.3 19.2				100	Q_u=6.7	Hard brown CLAY, trace roots (CH)
			2	2.3-4.5	18.2 19.2	67	15	52		Sp.Gr=2.56 Consol.	Stiff red-brown CLAY (CH)
			3	4.5-6.0	22.2				101	Q_u=1.5	Stiff mottled grey tan brown & black silty CLAY (CL)
			4	6.0-7.0	24.5 25.4				98	Q_u=1.0 SL=12.1	Stiff tan-brown clayey SILT, trace organic (MH-CL)
			5	8.0-8.7	35.2				86	Q_u=0.6	Med. mottled grey tan brown & black silty CLAY trace-sm silt pockets (CL)

TABLE 1

SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterburg Limits			Insitu γ Dry PCF	Other Test	Classification
						LL	PL	PI			
B23	8584+94 (32.5'W)	1399.0	1	0.5-2.5	22.4 25.2				96	Q_u=1.3	Stiff red-brown CLAY trace roots (CH)
			2	2.8-4.5	24.3 25.9 25.1	74	17	57	92 97	Q_u=1.4 $\bar{Q}$	Stiff red-brown CLAY (CH)
			3	4.5-6.5	31.3 14.4					SL=26.8	Tan clayey SILT some siltstone (MH)
			4	7.0-7.2	25.0						Tan SILT some red-brown clay & gravel (rock fragments)(ML-GM)
B24	8575+42.5 (27'W)	1398.0	1	0.5-2.5	20.5 21.7 18.2				101	Q =4.0	Hard red-brown to gry-brown CLAY, trace gravel & roots (CH)
			2	2.5-4.5	24.9 25.2				92	Q_u=1.2	Stiff gry-brown silty CLAY (CL-CH)
			3	4.5-6.0	29.6 29.6 28.6				92 93	Q_u=1.8 Q_u=1.4	Stiff dark red-brown CLAY (CH)
			4	6.0-7.5	30.7 21.3						Top: Brn CLAY(CH) Bott: Tan clayey SILT, trace to some gravel (MH)(weathered limestone)
B25	8566+00 (52.5'W)	1399.4	1	0.5-2.7	20.6	54	21	33	101	Vib. Puls.	Hard red-brown CLAY (CH)
			2	2.7-4.6	21.6 20.9					SL=11.4 Q_u=1.0	Stiff brown CLAY trace roots (CH)
			3	4.6-6.8	21.5 21.5 22.0 22.2	34	16	18	104	Q_u=2.6 Q_u=2.3 Sp.Gr.=2.63 Consol.	Very stiff brown silty CLAY (CL)

TABLE 1
SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterburg Limits LL	PL	PI	Insitu γ Dry PCF	Other Test	Classification
B25	8566+00 (52.5'W)	1399.4	4	6.8-8.6	20.6 21.0	40	15	25			Stiff red-brown silty CLAY (CL)
			5	8.6-10.5	21.7 21.8 22.1				102	Q_u=2.5	Very stiff red-brown CLAY (CH)
			6	10.5-12.3	23.4 21.5				102	Q_u=2.3	Very stiff red-brown mottled with tan silty CLAY (CL)
B26	8523+00 (85'W)	1423.8	1	1.0-3.7	22.7 24.8 28.5				87	Q_u=1.35	Top: Stiff brown silty CLAY, trace roots Bott: Tan clayey SILT trace organic (MH)
B27	8525+00 (94'W)	1424.0	1	0.5-2.8	25.1 28.5 29.4					PP=3.7	Stiff brown CLAY, trace gravel & roots
			2	2.8-3.8	No recovery						
			3	6	32.2						Tan silty CLAY, trace to some c-f sand (CL) (Sample consists of cuttings)
B28	8533+50 (95'W)	1417.5	1	0.5-3.0	23.8 26.1 28.9 29.9				91	Q_u=1.4	Stiff dark brown to red-brown CLAY, trace roots & gravel (CH)
B29	8530=00 (51'W)	1429.2	1	5	13.0						Tan silty f-c sand, some gravel (SM-ML) (Sample consisted of cuttings)
			2	6.0-9.5	No recovery						
			3	9.5-11.5	29.3 Top 26.2 Bot.	89	27	62	88	Q_u=1.0 Vib. Puls. SL=10.5	Top: Brown silty CLAY, trace organic (CH) Bott: Tan clayey SILT w/pkts brown silty CLAY (MH-CL)

TABLE 1
SUMMARY OF TEST DATA

Boring No.	Station (ML Offset)	Elev.	Sample No.	Depth Feet	Water Content %	Atterburg Limits LL	PL	PI	Insitu γ Dry PCF	Other Test	Classification
B30	8582+50 (87'W)	1401.8	1	0.5-2.8	21.3 27.9					TV=0.71 TV=0.77 Remolded =0.68 PP=1.3 to 1.8	Stiff brown to red-brown CLAY (CH)
			2	3.0-5.0	26.6 28.4 27.5	62	17	45	95 95	Q_u=1.1 $\bar{R}$	Stiff red-brown CLAY (CH)
			3	7	23.4						Tan silty CLAY and c-f sand (CL) (limestone fragments)

TABLE 2

SUMMARY OF SOIL PROPERTIES

Material	Soil Property	No. of Tests	Range	Average
Gray to brown, silty CLAY (CL)	Water content, %	29	15.1 to 29.4	23.3
	Liquid limit	4	33 to 48	39
	Plastic limit	4	15 to 18	16
	Plasticity index	4	15 to 32	23
	Shrinkage limit	1	12.1	12.1
	Specific gravity	1	2.63	2.63
	In-situ dry density, pcf	3	86 to 104	95
	Unconfined strength, Kg/cm^2	4	0.6 to 2.6	1.7
	Modified AASHO compaction			
	Max. dry density, pcf	1	111.5	111.5
	Optimum water content, %	1	15.6	15.6
Reddish brown to brown CLAY (CH)	Water content, %	69	17.3 to 32.2	24.5
	Liquid limit	17	50 to 89	74
	Plastic limit	17	15 to 25	19
	Plasticity index	17	32 to 63	55
	Shrinkage limit	2	10.5 to 11.4	11.0
	In-situ dry density, pcf	16	88 to 102	96
	Unconfined strength, Kg/cm^2	14	1.0 to 6.7	2.0
	Effective stress angle of internal friction, degrees	1	19	19
	Specific gravity	3	2.54 to 2.65	2.58
	Modified AASHO compaction			
	Max. dry density, pcf	4	97 to 107.3	103.4
	Optimum water content, %	4	19.3 to 24.3	21
Tan, clayey SILT (ML-CL)	Water content, %	12	13.7 to 46.6	28.0
	Liquid limit	1	27	27
	Plastic limit	1	21	21
	Plasticity index	1	6	6
	Shrinkage limit	1	26.8	26.8

TABLE 3

EARTHWORK QUANTITY SUMMARY

Item	Quantities, yd^3	
	Four Foot Embankment	Six Foot Embankment
Rock excavation	26,350	36,400
Embankment fill	46,200	61,600
Common excavation	78,500	113,000
Reuseable soil for embankment	46,000	52,000
Required borrow (Incl. 6% shrinkage)	3,100	13,500

AVAILABLE BORROW

Borrow Area A	1,750 yd^3
Borrow Area B	380
Borrow Area C	270
Borrow Area D	1,880
Borrow Area E	8,600
Borrow Area F	1,950
Total Available Borrow	14,830 yd^3

Note:

1. See Fig. 2 for location of borrow areas.
2. Additional suitable borrow is available from the ATSF right-of-way east of the test embankment site.

TABLE 4

ANTICIPATED EMBANKMENT PROPERTIES
[At 92% Relative Compaction Modified AASHO and w=Optimum +2]

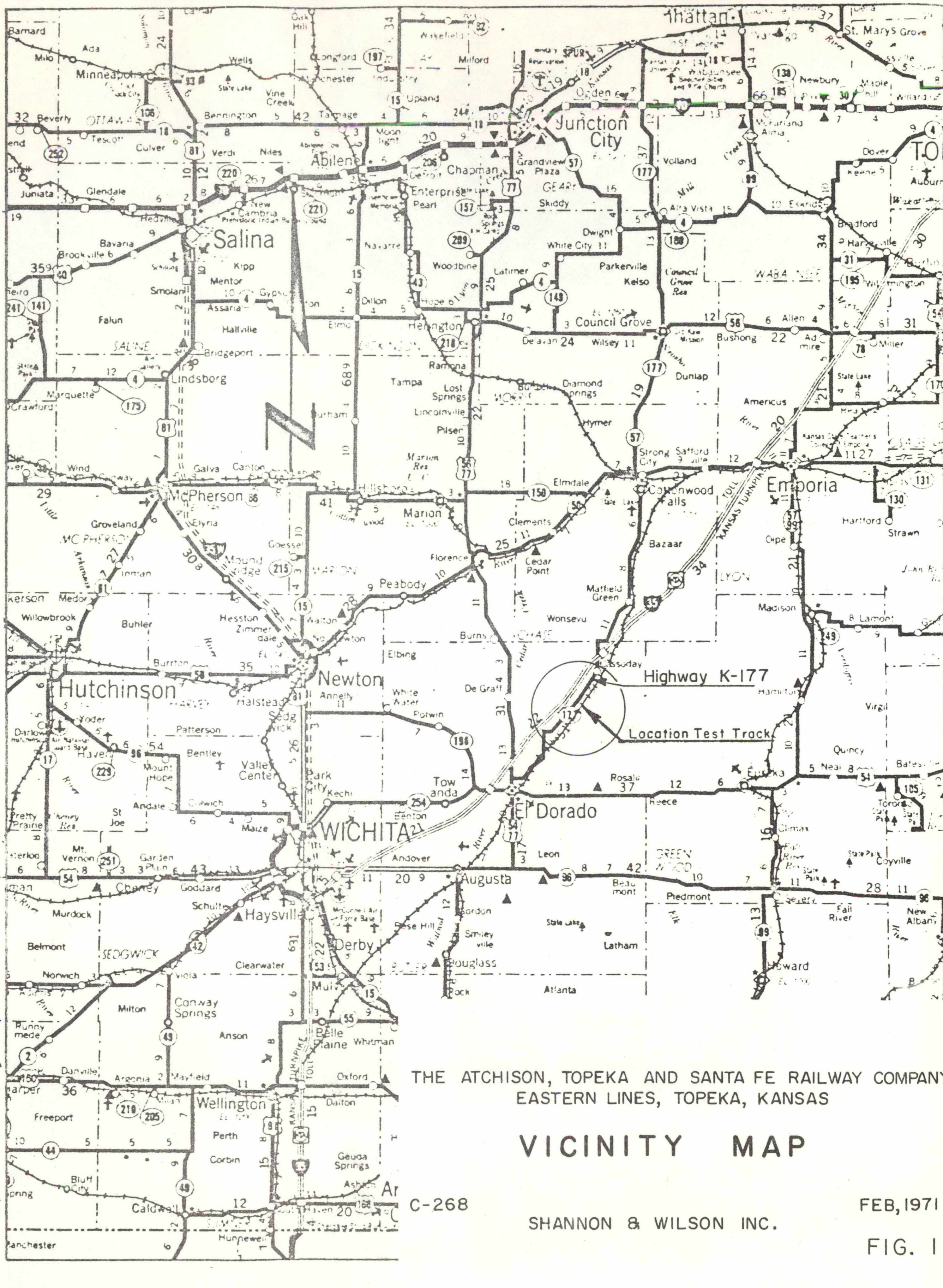
Soil Property	Range	Average
Dry density, pcf	92 to 106	100
Water content, %	18 to 24	22
Liquid limit	33 to 89	67
Plastic limit	15 to 25	18
Plasticity index	15 to 63	49
Shrinkage limit	10.5 to 12.1	11.4
Specific gravity	2.54 to 2.65	2.59
Unconfined compressive strength, Kg/cm^2	3.5 - 5.0	4.0
Initial tangent modulus from unconfined tests, Kg/cm^2	120 to 400	250
Maximum potential volume changes, %		
Shrinkage	10 to 17	13
Swelling*	5 to 15	11

*Taken from laboratory specimen swell @ 3,000 min.

TABLE 5

SUMMARY OF INSTRUMENTATION

Instruments per Test Section		Number of Test Sections	Total Number of Instruments
Type	Number		
Main Array:			
Horizontal instrument tubing	4	9	36
Pressure cells	3	9	27
Moisture temperature cells	13	9	117
Multi-position vertical extensometers	3	9	27
Single Position Vertical Extensometers	4	9	36



THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

VICINITY MAP

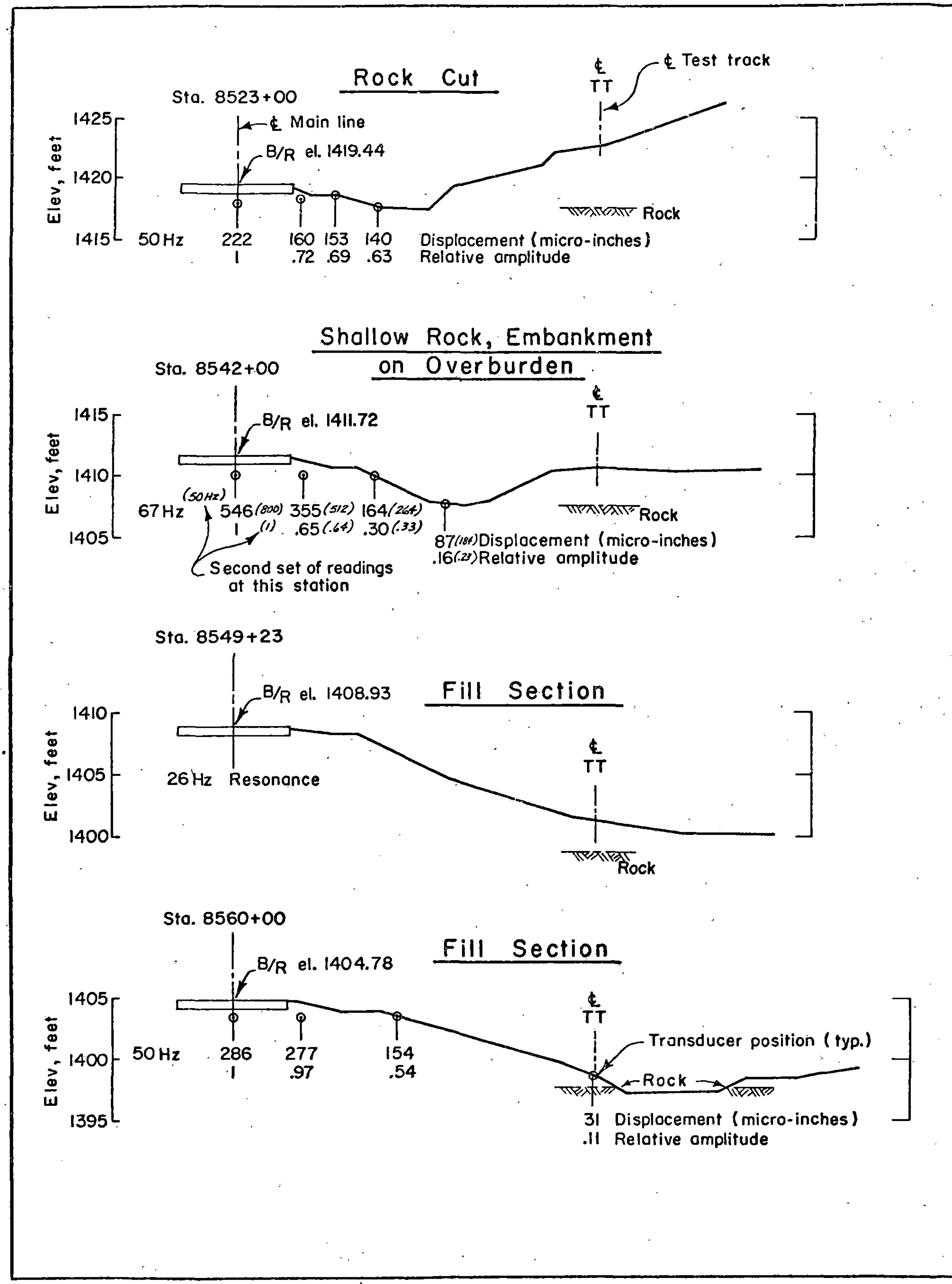
C-268

SHANNON & WILSON INC.

FEB, 1971

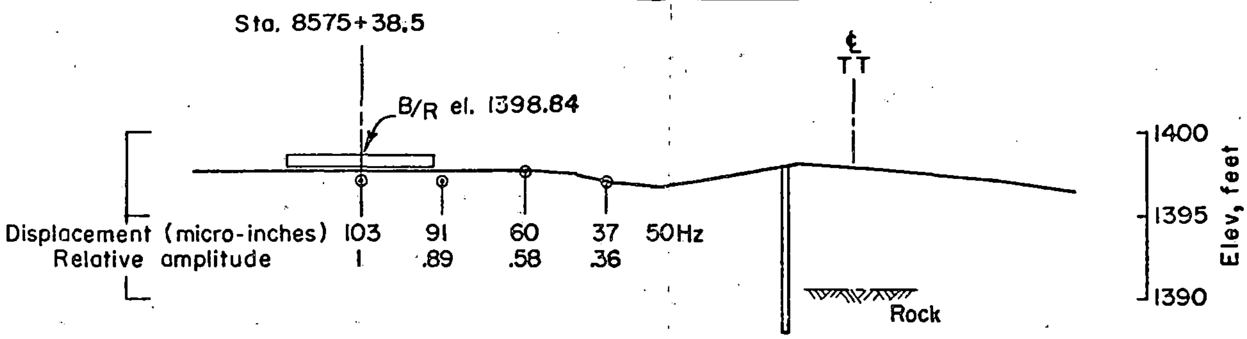
FIG. 1


LEGEND:
Main instrumentation array.
Single-position extensometer
Boring {B-1 - B-16 Hemphill Corp, April 1969; B-21 - B-30 Shannon & Wilson Inc, July 1970
Test pit {TP-1 - TP-24, July 1970; TP-25 - TP-27, Sept 1970
Note: 1. Stationing for Main Line and Test Track are identical in tangent section only.
2. Subsurface conditions between indicated borings or test pits may vary from that shown.
3. All elevations are based on AT & SF datum.
RAILROAD WEST
RAILROAD EAST
Mile Post 163
Mile Post 162
Mile Post 161
Section 9 - Control Section 800'
Section 8 - Concrete Ties 27" OC. 15" Ballast
Section 7 - Precast Beam 800'
Section 6 Stabilized Ballast 546'
Section 5 - Concrete Slab 800'
Section 4 - Continuous Concrete Beam 800'
Section 3 - Concrete Ties 24" OC. 10" Ballast 800'
Section 2 - Concrete Ties 27" OC. 10" Ballast 800'
Section 1 - Concrete Ties 30" OC. 10" Ballast 800'
Borrow Area D Sta. 8579+40 to 8591+40
Borrow Areas E, F Sta. 8581 to 8590
Borrow Area C Sta. 8572 to 8575+40
Borrow Area B Sta. 8563 to 8568
Borrow Area A Sta. 8520 to Sta. 8538
Kansas Highway 177
Existing Main Track
AT & SF R/W
Scale - feet
ELEVATION IN FEET
Existing ground line at ℄ of test track
Designed base of rail
Subgrade
Bottom of 6' embankment
Bottom of embankment
Red-brown CLAY (CH)
Pole yellow, clayey SILT (ML-CL)
Rock
7.5' min.
R.R. Grade change 8598+00
R.R. Grade change 8583+00
8536+00 R.R. Grade change
8520+00 R.R. Grade change
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS
DATED Sept. 17, '70
APPROVED Chief Engineer
U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT
PLAN AND PROFILE OF
TEST TRACK
EMBANKMENT
SHANNON & WILSON INC.
Rev.
2
12
FIG. 2

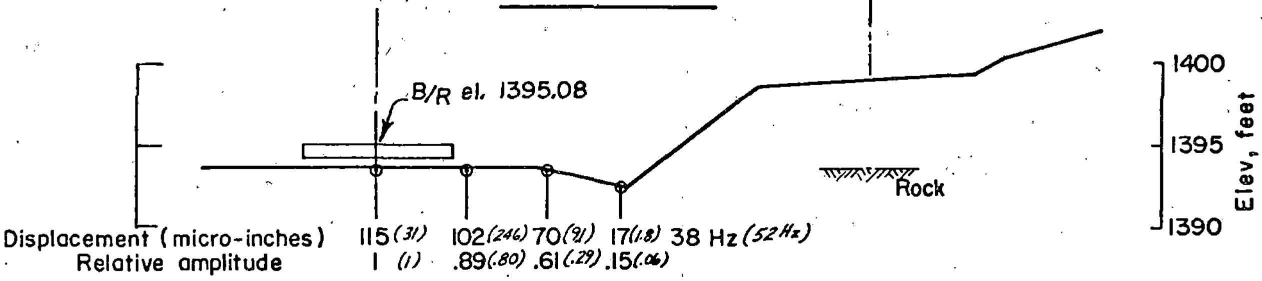


Rock Cut
Sta. 8523+00
℄ Test track
℄ TT
℄ Main line
B/R el. 1419.44
Elev, feet
1425
1420
1415
50 Hz
222
1
160 153 140
.72 .69 .63
Displacement (micro-inches)
Relative amplitude
Rock
Shallow Rock, Embankment on Overburden
Sta. 8542+00
B/R el. 1411.72
℄ TT
1415
1410
1405
67 Hz
(50 Hz)
546 (800) 355 (512) 164 (264)
1 (1) .65 (.64) .30 (.33)
87 (184) Displacement (micro-inches)
.16 (.23) Relative amplitude
Rock
Second set of readings at this station
Sta. 8549+23
B/R el. 1408.93
Fill Section
1410
1405
1400
26 Hz Resonance
℄ TT
Rock
Sta. 8560+00
B/R el. 1404.78
Fill Section
1405
1400
1395
50 Hz
286 277 154
1 .97 .54
℄ TT
Transducer position (typ.)
Rock
31 Displacement (micro-inches)
.11 Relative amplitude

Shallow Rock, Embankment on Overburden



Rock Cut



Station	Distance between top of Subgrade & Rock, feet
8585+00	0
8560+00	5.5
8549+23	8.6
8542+00	3.0
8575+38.5	6.8
8523+00	0.7

Notes:

1. Top of rock elevation scaled from profile along test track, Fig. 2.
2. Cross-sections were furnished by AT & SF.
3. Micro-inches = 10^{-6} inches.
4. Hz = Cycles per second.
5. Vibration measurements were taken from Geo-Recon Inc. report, Aug. 11, 1970, contained in Appendix A.

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY CO.
EASTERN LINES, TOPEKA, KANSAS

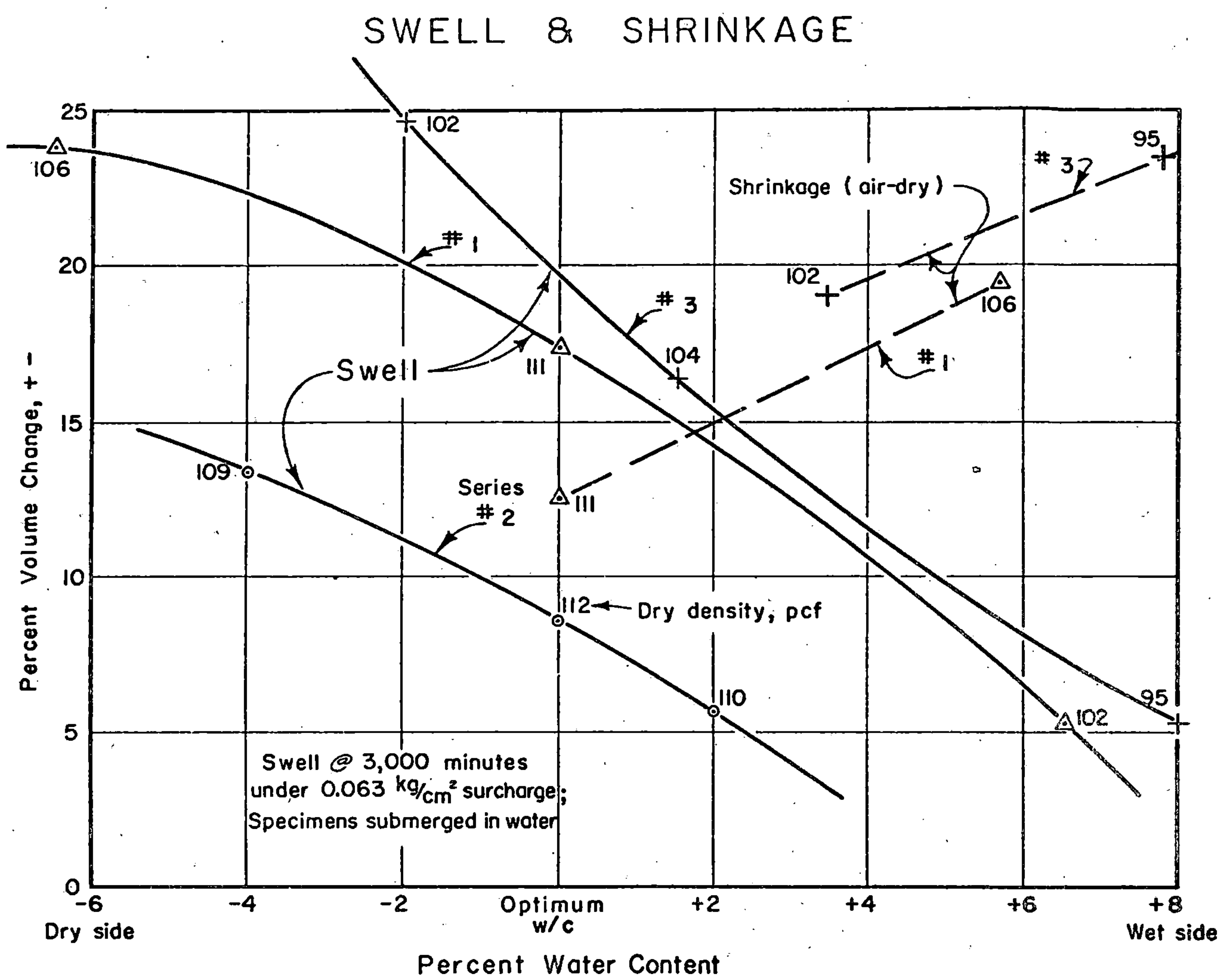
EMBANKMENT VIBRATION MEASURMENTS

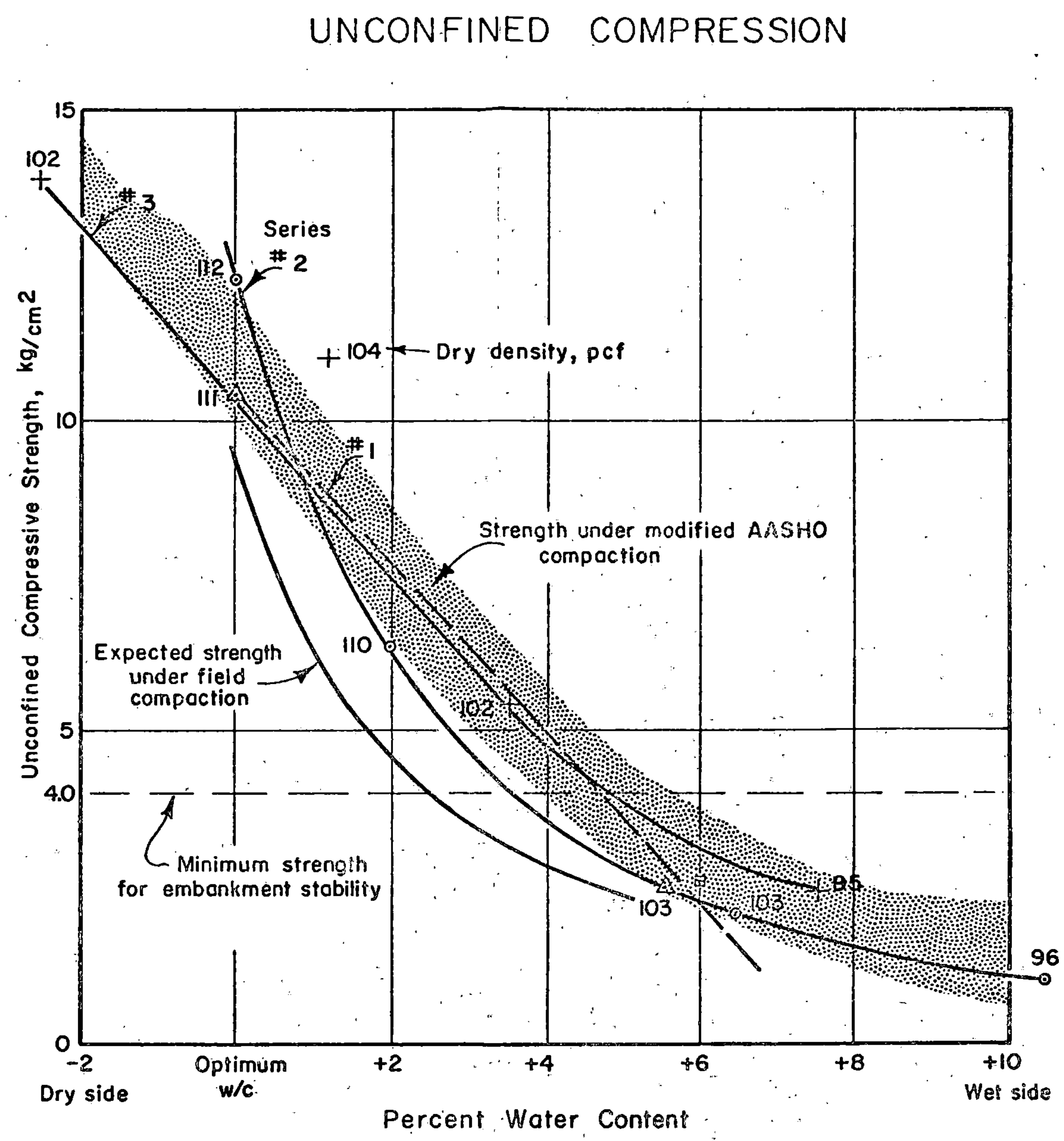
Scale: 1" = 10'

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 3





Notes:

1. Optimum moisture contents (Modified AASHO) were determined on the material for each series.
2. See Figs C7, C8, and C9 for compaction curves and other test data.
3. All test specimens were carved from Modified AASHO compaction test specimens, thus the test data are functions of density and moisture content.
4. Shrinkage conditions were air-drying until there was no further volume change.

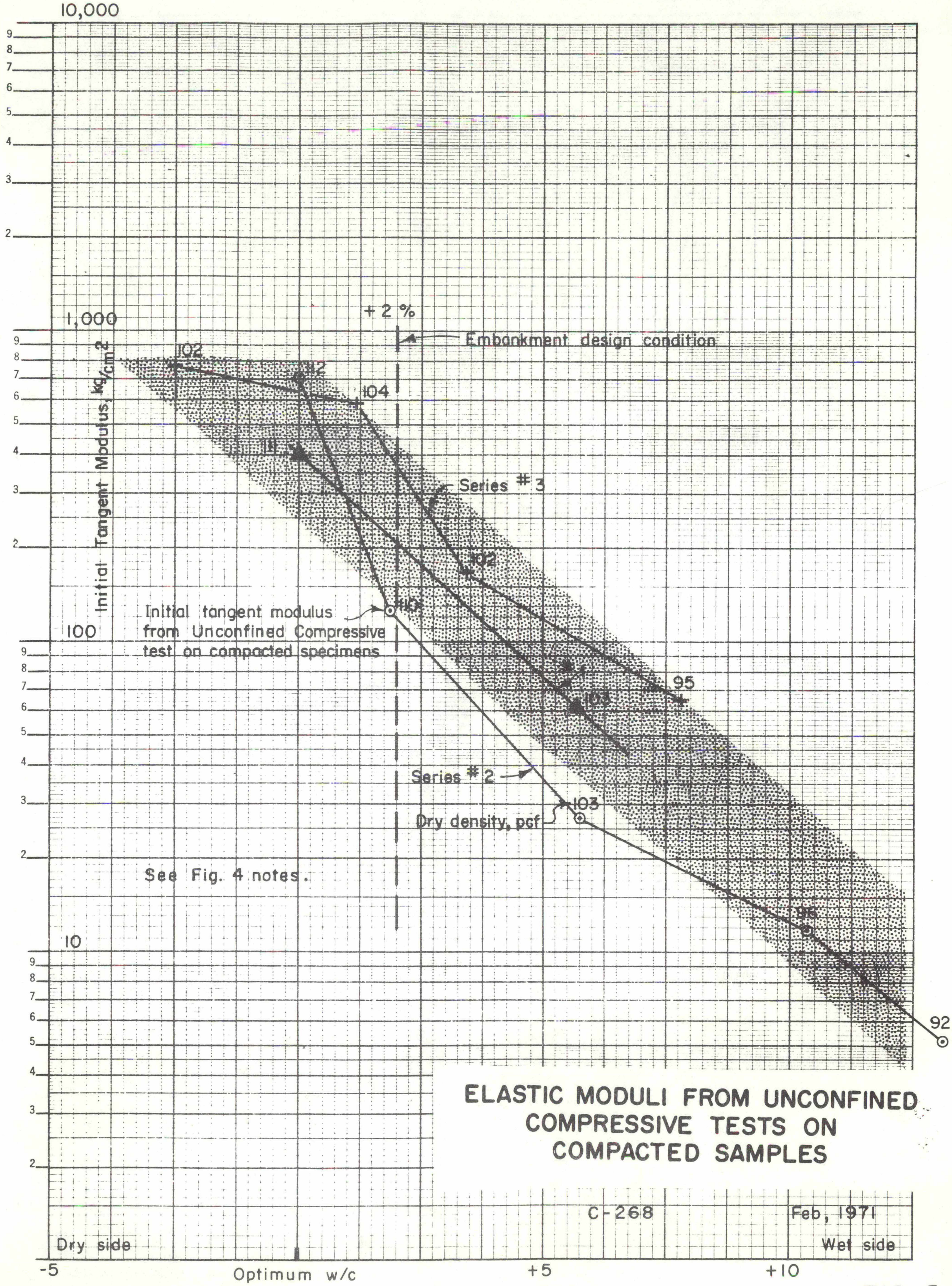
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY CO.
EASTERN LINES, TOPEKA KANSAS

STRENGTH, SWELL, & SHRINKAGE OF COMPACTED SOILS

C-268 FEB, 1971

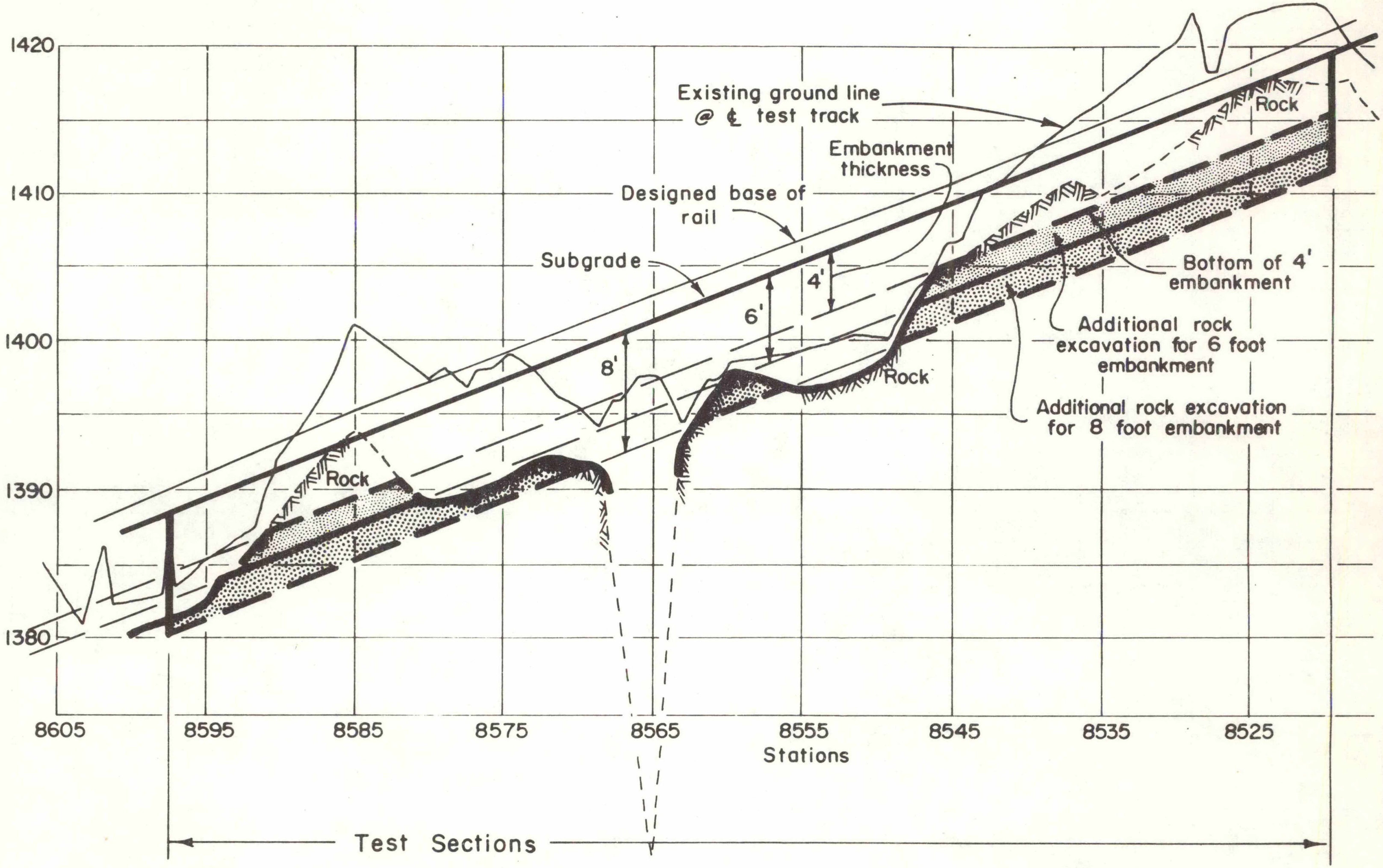
SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 4



10,000
1,000
100
10
Initial Tangent Modulus, kg/cm²
+ 2 %
Embankment design condition
102
112
104
111
Series # 3
102
110
Initial tangent modulus
from Unconfined Compressive
test on compacted specimens
95
103
Series # 2
103
Dry density, pcf
See Fig. 4 notes.
96
92
ELASTIC MODULI FROM UNCONFINED
COMPRESSIVE TESTS ON
COMPACTED SAMPLES
C-268
Feb, 1971
Dry side
Wet side
-5
Optimum w/c
+5
+10
Percent Water Content

FIG. 5



PROFILE OF TEST TRACK EMBANKMENT

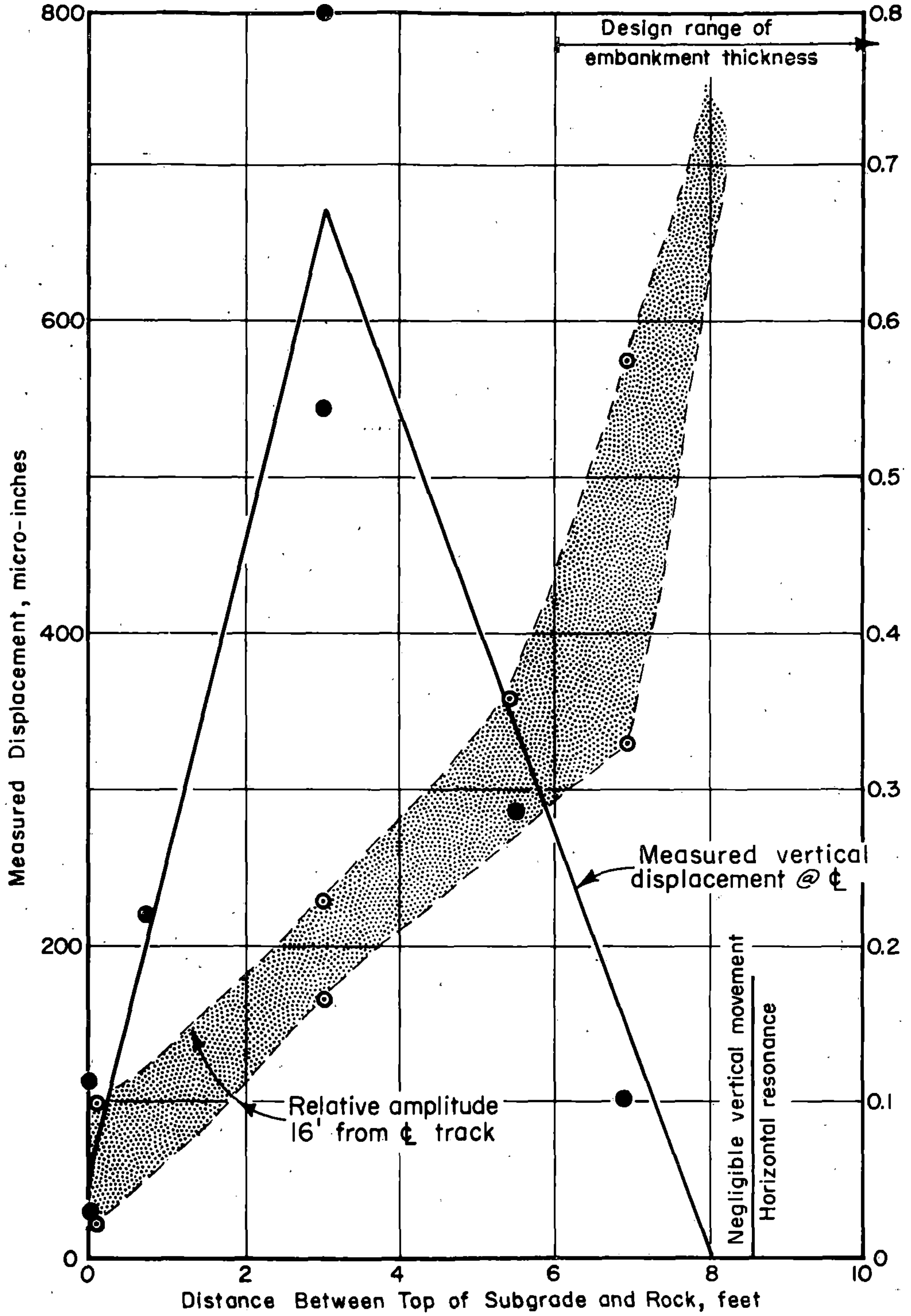
C-268

FEB, 1971

SHANNON & WILSON

SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 6



Note: microinches = 10^{-6} inches

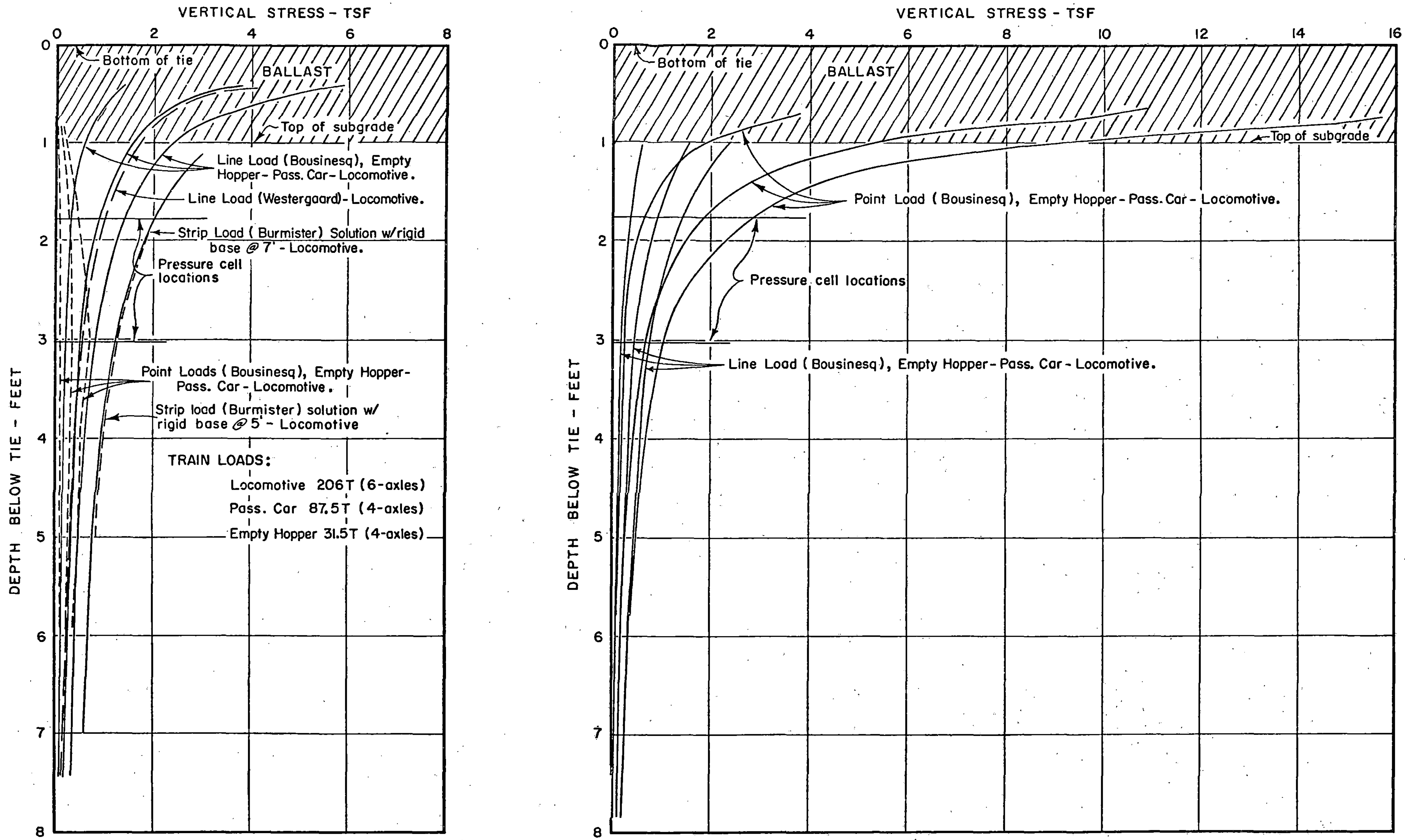
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY CO.
EASTERN LINES, TOPEKA, KANSAS

SUMMARY OF EMBANKMENT VIBRATION MEASUREMENTS

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 7



(a) VERTICAL STRESSES BENEATH CENTER OF TRACK

(b) VERTICAL STRESSES BENEATH RAIL

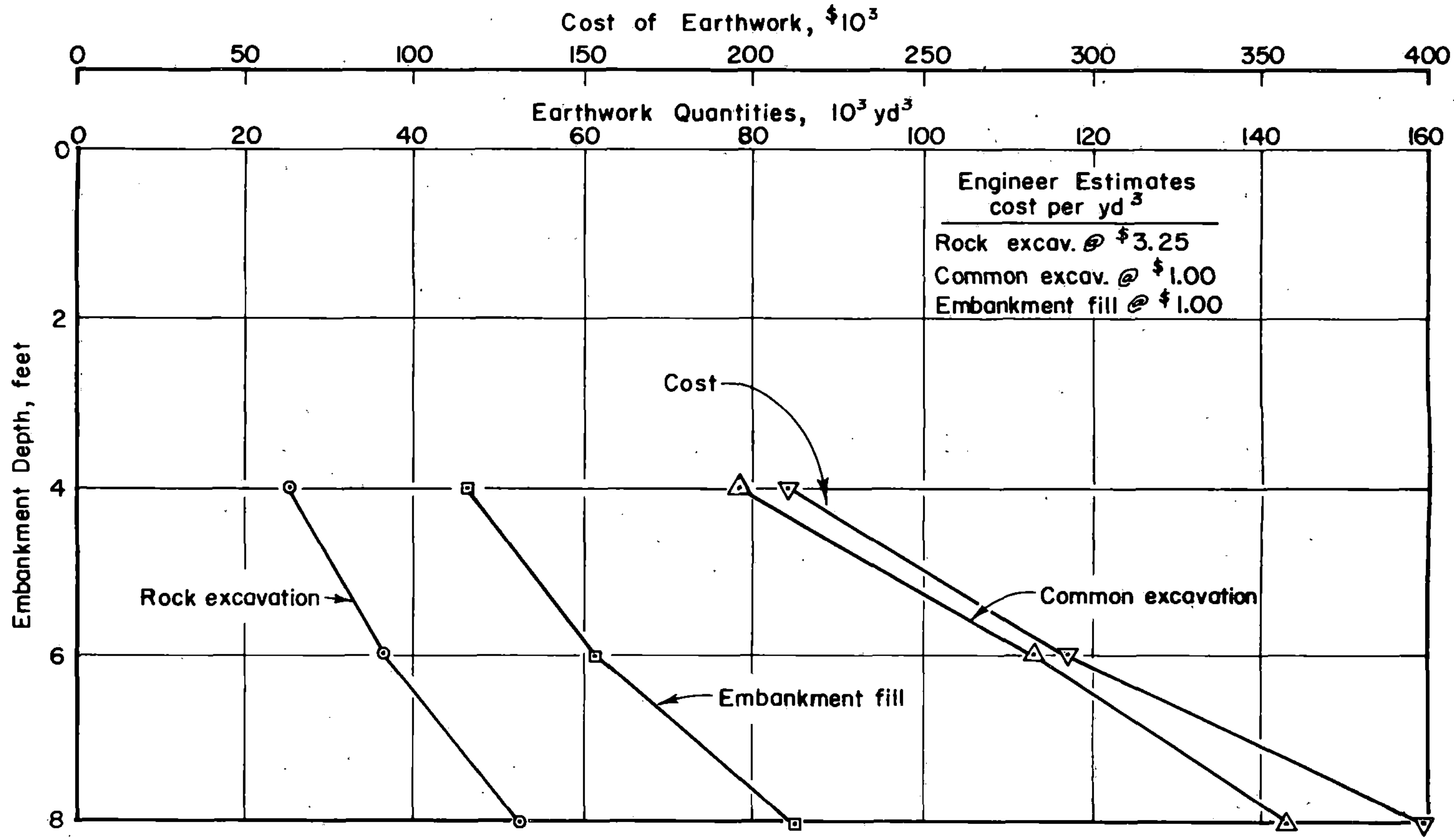
ATCHISON, TOPEKA & SANTA FE RAILWAY CO.
TOPEKA, KANSAS

THEORETICAL EMBANKMENT STRESSES

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 8



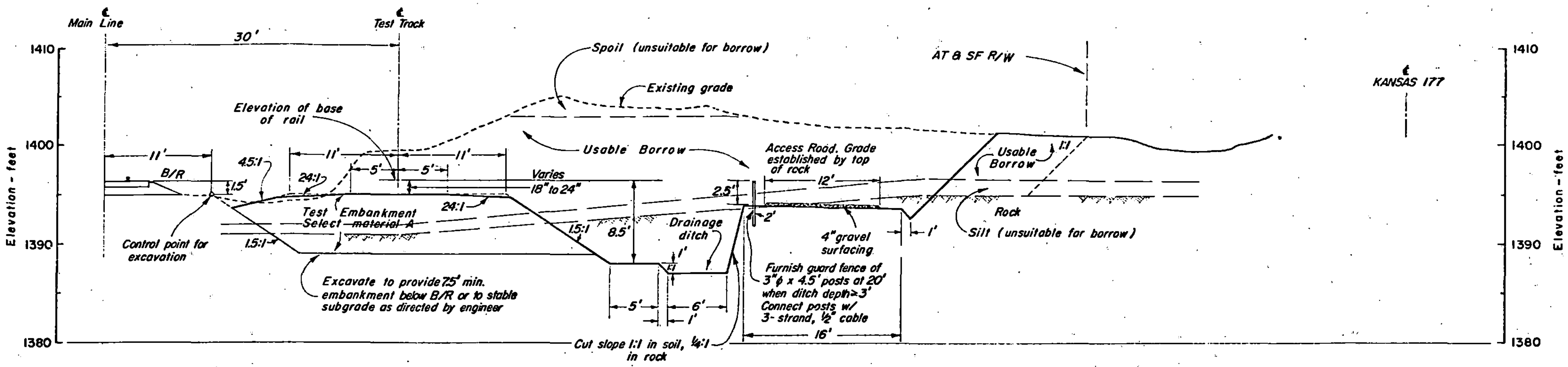
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY
EASTERN LINES, TOPEKA, KANSAS

ESTIMATED EARTHWORK QUANTITIES and COST

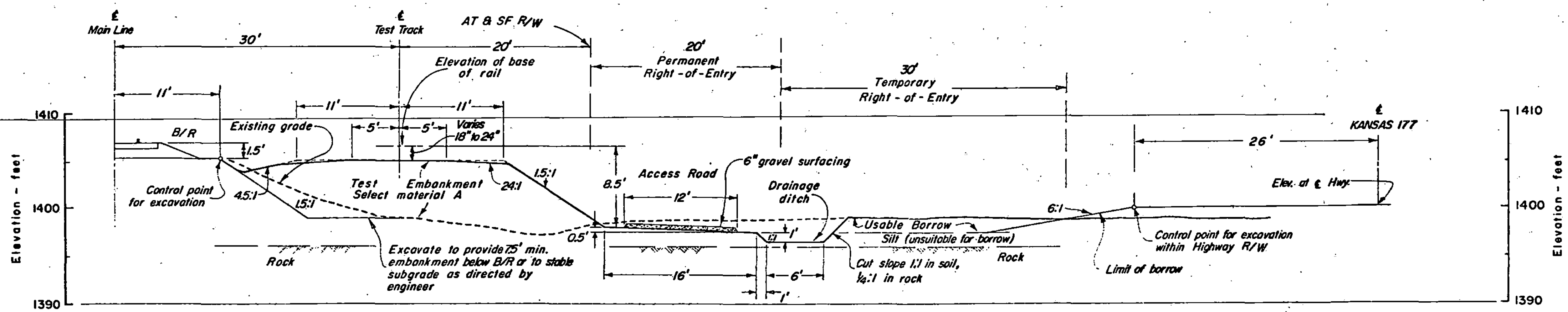
C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. 9



TYPICAL EXCAVATION SECTION
(STA. 8582+00)



TYPICAL EMBANKMENT SECTION
(STA. 8555+00)

0 5 10 15 20
Horiz. Scale - feet

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS
DATED
APPROVED Chief Engineer

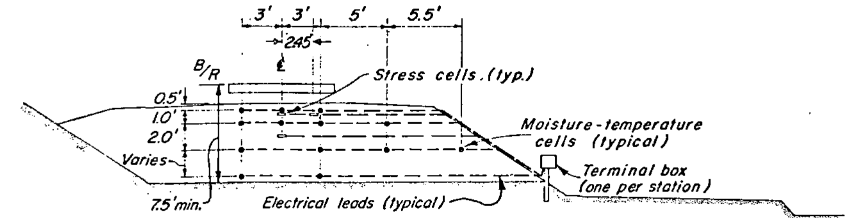
U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

TYPICAL EXCAVATION & EMBANKMENT SECTIONS

Shannon and Wilson
Soil mechanics and foundation engineers

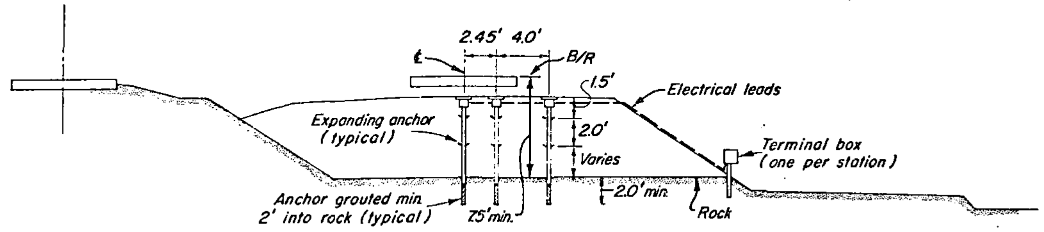
Rev. 7 / 12

FIG. 10



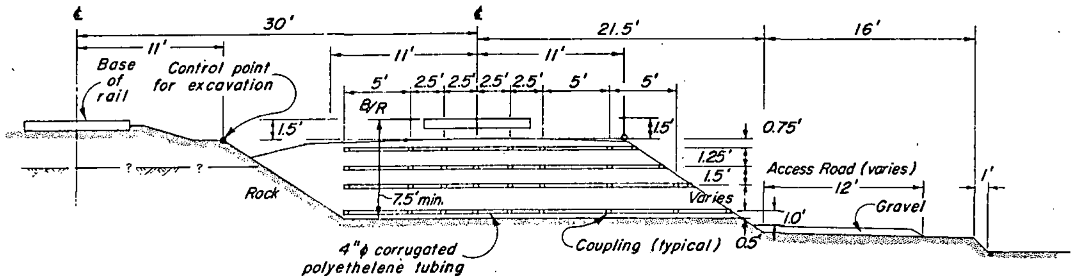
MOISTURE - TEMPERATURE CELLS AND STRESS CELLS — MAIN ARRAY

(To be installed as embankment is constructed)



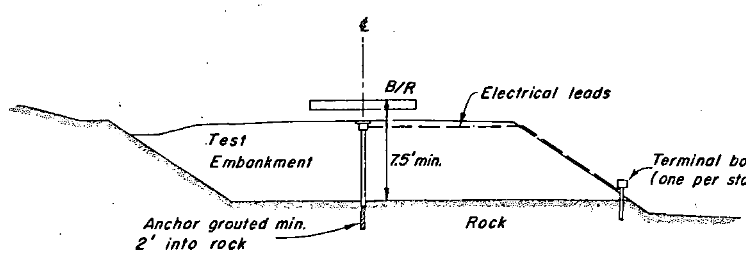
VERTICAL EXTENSOMETERS - MAIN ARRAY

(To be installed after completion of embankment)



HORIZONTAL TUBING - MAIN ARRAY

(To be installed as embankment is constructed)

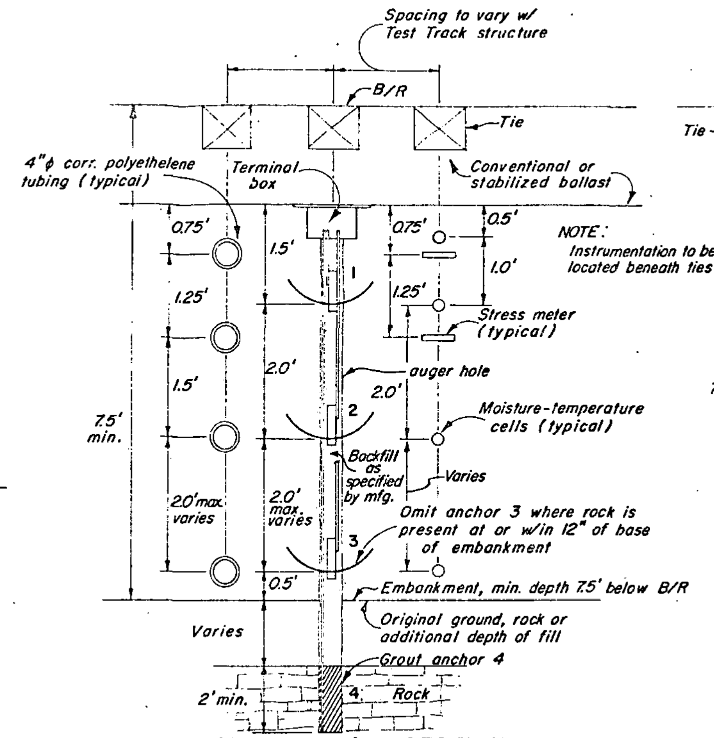


SINGLE - POSITION EXTENSOMETER

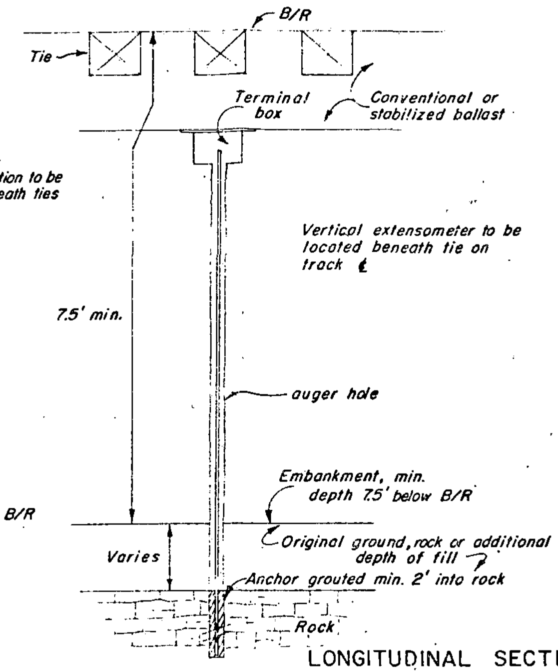
(To be installed after completion of embankment)

0 5 10 15

Scale - feet

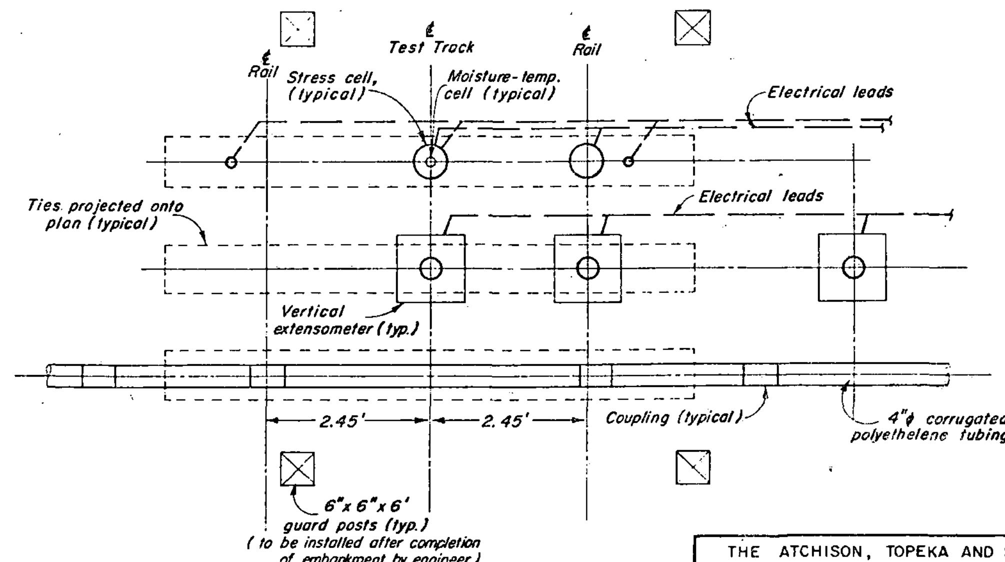


LONGITUDINAL SECTION
INSTRUMENTATION - MAIN ARRAY



LONGITUDINAL SECTION
SINGLE - POSITION EXTENSOMETER

(To be installed after completion of embankment)



PLAN
INSTRUMENTATION - MAIN ARRAY

0 10 20 30 40

Scale - feet

NOTES:

1. All instrumentation to be furnished and installed by the engineer.
2. Moisture - temperature cells, stress cells and horizontal tubing to be installed as the embankment is placed. Extensometers (vertical) to be installed after embankment is completed.

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY EASTERN LINES, TOPEKA, KANSAS DATED ___ APPROVED ___ Chief Engineer	
U. S. DEPARTMENT OF TRANSPORTATION EXPERIMENTAL TEST TRACK EMBANKMENT	
TYPICAL INSTRUMENTATION DETAILS	
Shannon and Wilson **Soil mechanics and foundation engineers**	Rev. 8 12

FIG. 11

APPENDIX A

VIBRATION MEASUREMENTS

GEO RECON INC.

GEOPHYSICAL EXPLORATIONS

1105 NORTH 38th STREET • SEATTLE, WASHINGTON 98103 • MElrose 2-8022

G-0450-00

August 11, 1970

SHANNON & WILSON, INC.
1550 Rollins Road
Suite F
Burlingame, California 94010

Re: Proposed Experimental
High Speed Test Track
Cassoday, Kansas

Gentlemen:

This letter presents the results of the vibration measurements of the existing embankment along the proposed Atchison, Topeka and Santa Fe Railway Co. high speed test track near Cassoday, Kansas (MP 161 to MP 162.8). The purpose of obtaining this information is to assist in establishing the required minimum thickness of the embankment and the range of dynamic response to be measured with project instrumentation.

The vibration measurements were made between July 7 and 10, 1970 under the direction of Mr. Rudy J. Dietrich; Shannon and Wilson, Inc. Six locations along the track section were picked to represent the various cut and fill sections for the proposed embankment. Stations 8560 and 8549+23 represent maximum embankment thickness; 8542 and 8575+38.5 represent shallow rock with the embankment on overburden; 8523 represents a rock-cut section; and 8585 represents a cut with the subgrade and ditch very close to rock.

At each location, four points were simultaneously measured at: (1) the centerline of the tracks on the top of the existing embankment, at the base of the ballast; (2) the top edge of the embankment and the base of the ballast; (3) a point midway between the top edge of the embankment and the toe of the slope; and (4) at the toe of the slope of the embankment. A triaxial transducer was buried at each of these points and the particle velocities of the vibrations induced by the passing trains were recorded on a light-beam oscillograph. This data was then either integrated or compared to obtain actual particle displacements in inches.

Typical results of the various sections selected for measurement are tabulated below. In the analysis, the point measured on centerline between the tracks has been taken as unity. The measurements from the other points were then compared to the unity measurements in order to arrive at a wave attenuation rate from this centerline point. The distances given are horizontal distances from centerline.

MEASURED DISPLACEMENT MICRO-INCHES	RELATIVE AMPLITUDE	HORIZONTAL DISTANCE	FREQUENCY
Station 8585	(Shallow overburden and rock section.)		
31	1	0	52 Hz
24.6	0.795	5.5	
9.1	0.294	10.4	
1.8	0.059	15.0	
115	1	0	38 Hz
102	0.89	5.5	
70	0.61	10.4	
17	0.15	15.0	

MEASURED DISPLACEMENT MICRO-INCHES	RELATIVE AMPLITUDE	HORIZONTAL DISTANCE	FREQUENCY
Station 8560	(Maximum fill section.)		
286	1	0	50 Hz
277	0.97	5.5	
154	0.54	13.5	
31	0.11	29.5	

Station 8549+23 (Fill section.)

Fill section, at this point, resonated at 26 Hz. No attenuation measurements computed. See explanation following.

Station 8542	(Shallow rock w/embankment on overburden.)		
546	1	0	67 Hz
355	0.65	5.5	
164	0.30	11.5	
87	0.16	17.3	
800	1	0	50 Hz
512	0.64	5.5	
264	0.33	11.5	
184	0.23	17.3	
Station 8575+38.5	(Shallow rock w/embankment on overburden.)		
103	1	0	50 Hz
91	0.89	5.0	
60	0.58	10.0	
37	0.36	15.0	

MEASURED DISPLACEMENT MICRO-INCHES	RELATIVE AMPLITUDE	HORIZONTAL DISTANCE	FREQUENCY
Station 8523	(Rock-cut section.)		
222	1	0	50 Hz
160	0.72	5.1	
153	0.69	8.0	
140	0.63	11.6	

In general, the random and transient motions recorded on the top of the embankment attenuated rapidly with distance, giving rise to periodic motions within the embankment. The predominant frequencies measured in the embankment were 26, 50 and 100 Hz. A low frequency motion of 3 to 6 Hz attenuated rapidly and, in most cases, was only recorded on the top of the embankment.

At Station 8549+23, the frequency recorded within the embankment was 26 Hz. The motion recorded here was typical for an underdamped vibrating system vibrating in one of its resonant modes. The vertical motion within the embankment at this station was negligible; most of the motion was in a horizontal plane. The transient and random motion produced at the centerline set the embankment into a periodic motion, which was reinforced by refractions and reflections from the underlying materials. For this reason, periodic motion induced at the top of the embankment could not be followed down the embankment.

The range of recorded motions at each of the six locations is listed below. These values represent the maximum particle displacements measured.

STATION	FREQUENCY IN HERTZ	PARTICLE AMPLITUDE IN INCHES
8585	4 Hz	0.032
	6 Hz	0.0033
	50 Hz	0.0023
	100 Hz	0.00085
8575+38.5	4 Hz	0.0086
	5.5 Hz	0.0039
	16.5 Hz	0.0019
	45 Hz	0.0029
8560	3.5 Hz	0.016
	4.5 Hz	0.0055
	45 Hz	0.0015
	100 Hz	0.002
8549+23	3 Hz	0.03
	4.5 Hz	0.0055
	5.5 Hz	0.0059
	100 Hz	0.0024
8542	4.5 Hz	0.007
	100 Hz	0.002
8523	4 Hz	0.015
	100 Hz	0.007

A wide range of frequencies is generated by the passing trains. The frequency range measured by the transducers ranged from 3 to 100 Hz. Maximum particle displacements occur at the lower frequencies, although most of the motion is above 25 Hz. The lower frequency motion, 3 to 6 Hz, is generated by the motion of the wheels passing over the detector point. The particle displacements at these frequencies were dependent on the loading weight of the cars. Excitation of the embankment is due to the frequencies in the range of 16 to 70 Hz. The motion from 70 to 100 Hz dissipates rapidly.

Please advise us if you have any questions regarding this report or if we may be of further assistance.

Very truly yours,

GEO-RECON, INC.

By John M Musser
John M. Musser

By Boyd O. Bush
Boyd O. Bush

JMM/BOB:cgg

APPENDIX B

BORING AND TEST PIT LOGS

BORING B-21

Sta. 8595+02.5 (35' W. of ℄ M.L.)

El. 1384.4

Depth (feet)	Description	Sample	Data
0 – 2.5	Medium, brown CLAY with trace limestone frag. & organic material (CH)	1	N=6, W=28
2.5	LIMESTONE		

BORING B-22

Sta. 8579+00 (37' W. of ℄ M.L.)

El. 1398.3

Depth (feet)	Description	Sample	Data
0 – 4.2	Stiff, brown CLAY, trace roots (CH)	1	N=11, W=18
		2	N=14, W=17
4.2 – 5.0	Stiff, brown mottled with tan & black CLAY (CH)		
5.0 – 6.1	Stiff, tan & lt. gray, si. CLAY, trace limestone fragment (CL)	3	N= 16/7", W=36, 47
6.1	BOULDER		

BORING B-25

Sta. 8566+00 (52.5' W. of ℄ M.L.)

El. 1399.4

Depth (feet)	Description	Sample	Data
0 – 6.0	Hard to very stiff, red-brown CLAY & silty CLAY (CH) (Slightly friable and slickensided)	1	W=21
		2	W=22, W=21
6.0 – 12.0	Very stiff, red-brown CLAY & silty CLAY (CH - CL)	3	W=22, W=22, W=22
		4	W=21, W=21
12.0 – 24.0	Stiff, lt. red-brown, silty CLAY & clayey SILT (CL - MH)	5	W=22, W=22
		6	W=23, W=22
24.0 – 34.5	Stiff, lt. red-brown, clayey SILT, trace large rock fragments (CL - MH)		
26.0	< Boulder		
34.5	ROCK		

BORING B-26

Sta. 8523+00 (85' W. of ℄ M.L.)

El. 1423.8

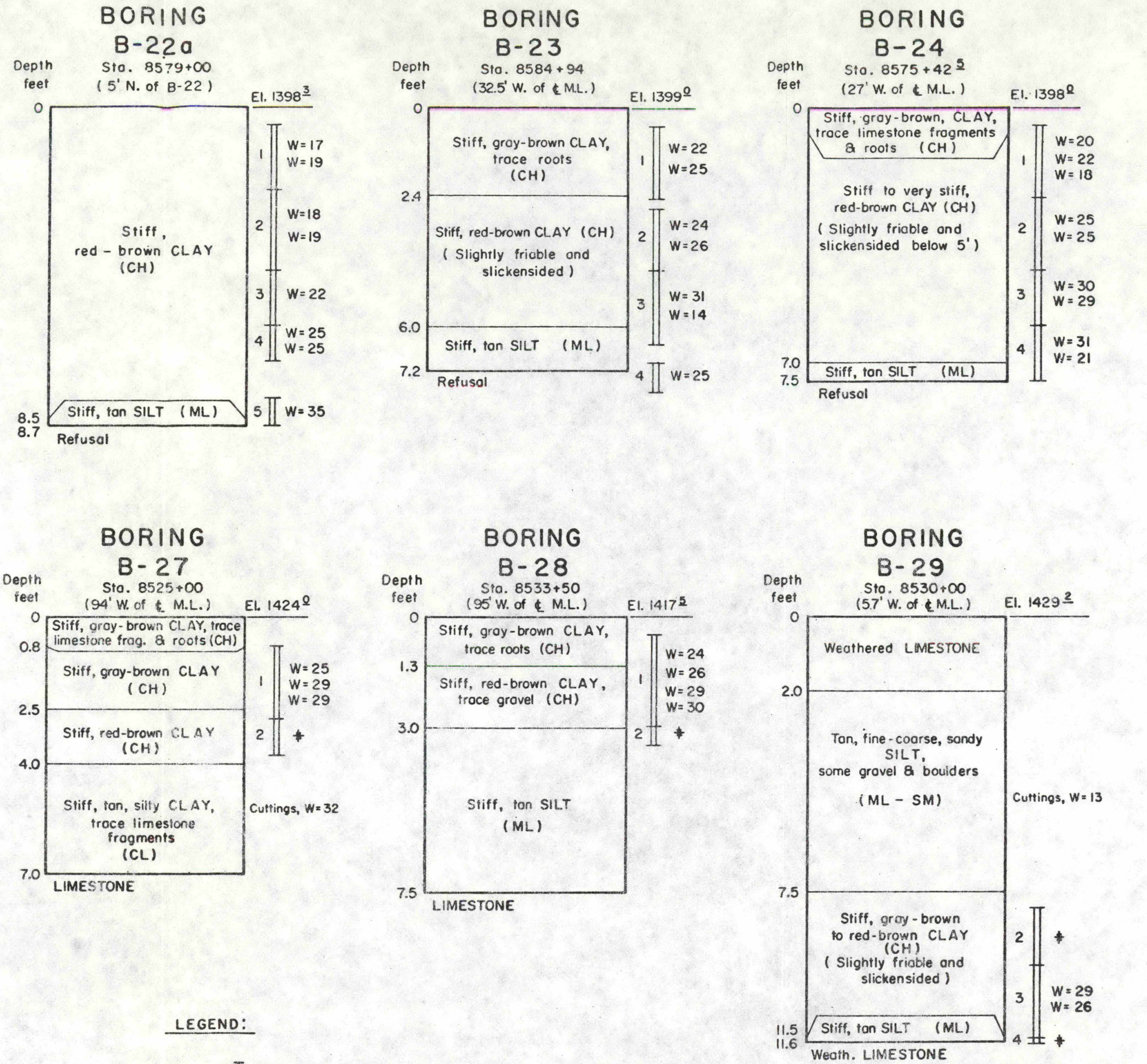
Depth (feet)	Description	Sample	Data
0 – 3.0	Stiff, gray-brown, CLAY, trace roots (CH)	1	W=23, W=25, W=29
3.0 – 5.3	Stiff, tan SILT (ML)	2	*
5.3	LIMESTONE		

BORING B-30

Sta. 8582+50 (87' W. of ℄ M.L.)

El. 1401.8

Depth (feet)	Description	Sample	Data
0 – 1.5	Stiff, gray-brown CLAY, (CH)	1	W=21, W=28
1.5 – 6.5	Stiff, red-brown CLAY, trace roots (friable) (CH)	2	W=27, W=28, W=28
6.5 – 8.5	Stiff, tan, silty CLAY and some limestone fragments (CL - SW)		Cuttings, W=23
8.5	Weath. LIMESTONE		



LEGEND:

- 2" O.D. split spoon sample
- 3" O.D. shelby sample
- N Number of blows with 140 lb. hammer falling 30" to drive a 2" O.D. split spoon sampler 1 foot.
- W Water content, %
- ⌖ No recovery

Note: Boring locations were interpolated from AT & SF cross-sections.
See Table I for sample data.

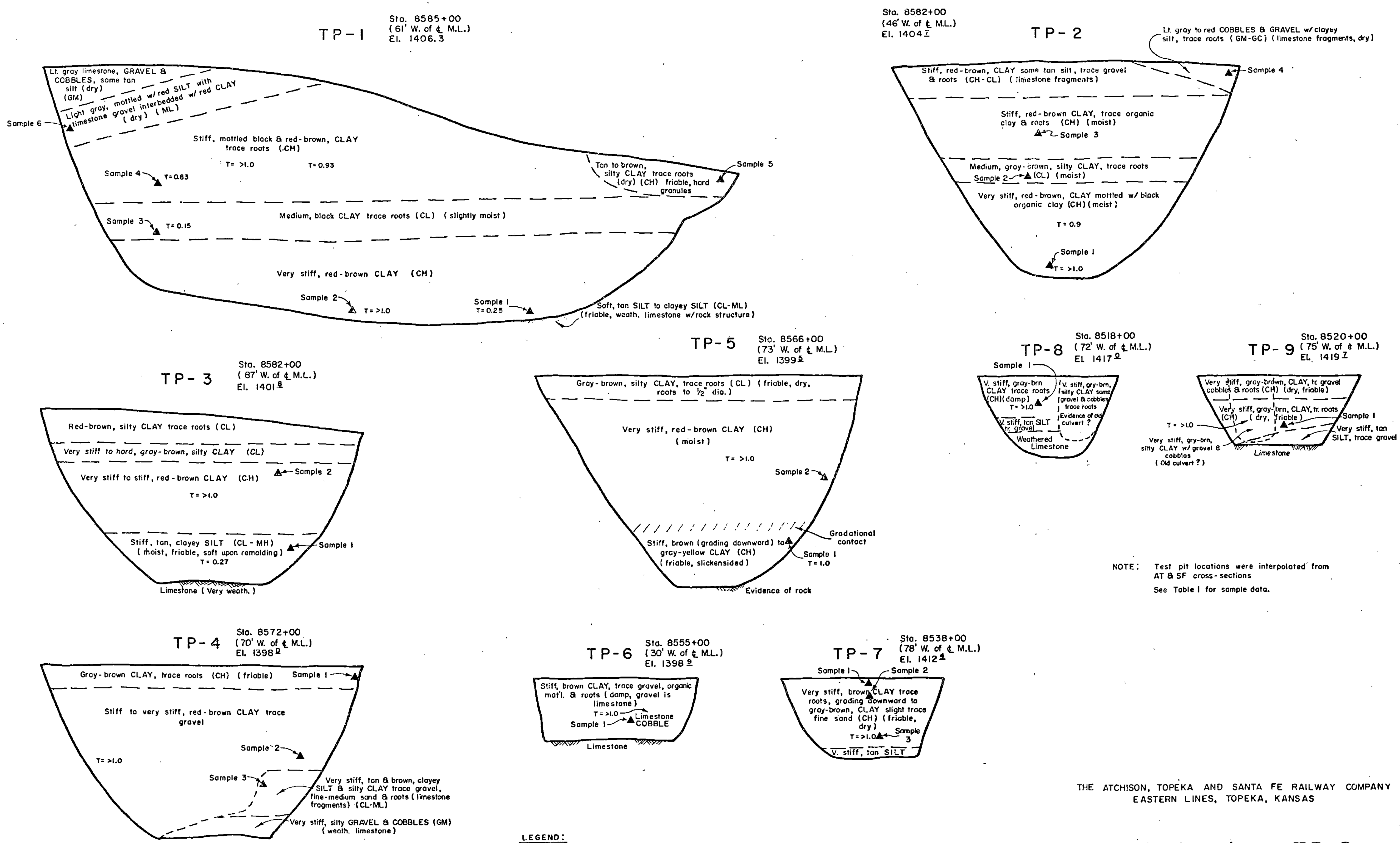
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

BORING LOGS
B-21 thru B-30

C-268 FEB. 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. B1



NOTE: Test pit locations were interpolated from AT & SF cross-sections
See Table 1 for sample data.

LEGEND:

T = 0.27 indicates shear strength of soil in tons per square foot as determined by hand held "Torvane" vane shear device.

0 1 2 3 4 5
Scale — feet

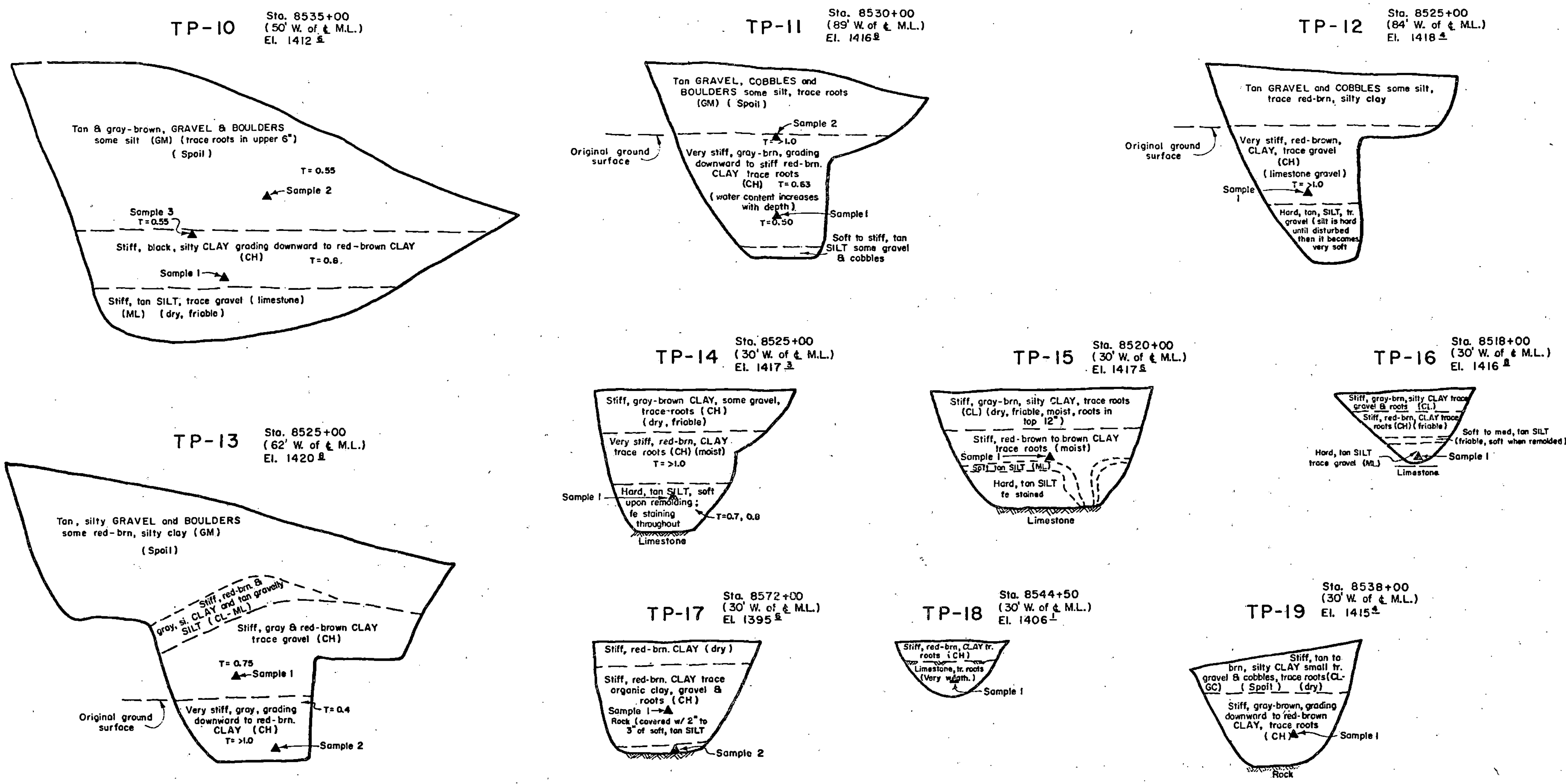
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

TP-1 thru TP-9

C-268 FEB. 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. B2



Note: Test pit locations were interpolated from AT & SF sections.
See Table I for sample data.

LEGEND:

T = 0.27 indicates shear strength of soil in tons per square foot as determined by hand held "Torvane" vane shear device.

0 1 2 3 4 5
Scale – feet

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

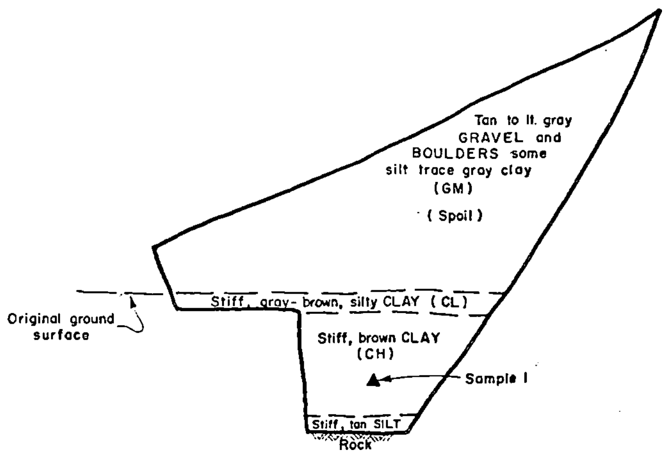
TP-10 thru TP-19

C-268 FEB, 1971

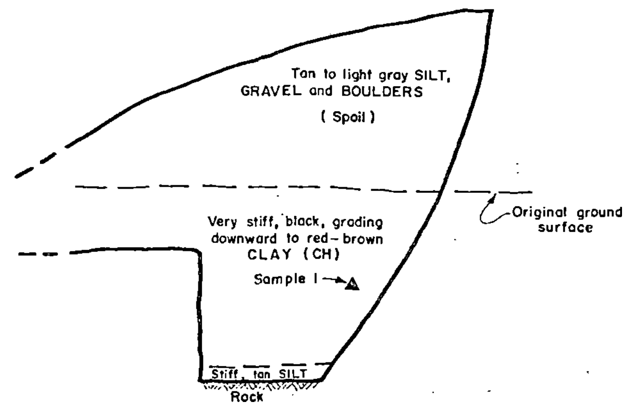
SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. B3

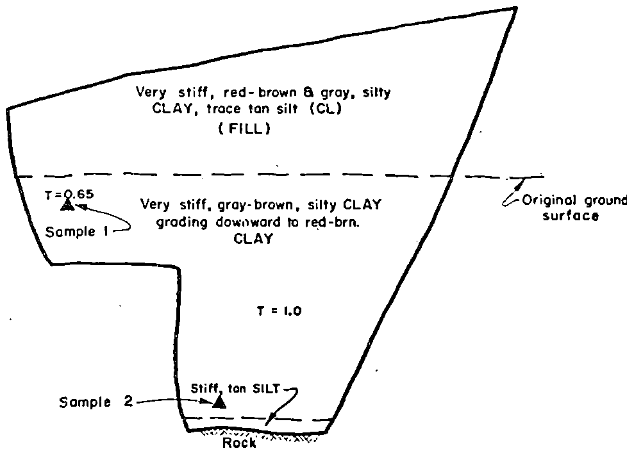
TP-20 Sta. 8530+00 (57' E. of ℄ M.L.) El. 1430±



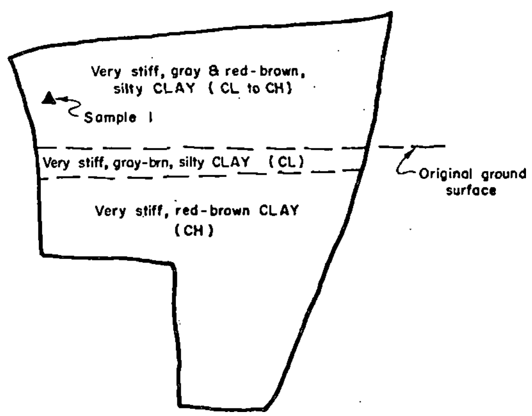
TP-21 Sta. 8535+00 (56' E. of ℄ M.L.) El. 1423±



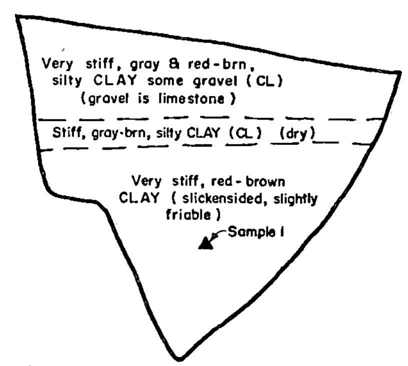
TP-22 Sta. 8582+00 (50' E. of ℄ M.L.) El. 1402±



TP-23 Sta. 8585+00 (50' E. of ℄ M.L.) El. 1401±



TP-24 Sta. 8585+00 (133' E. of ℄ M.L.) El. 1399±



Note: Test pit locations were interpolated from AT & SF sections.
See Table I for sample data.

LEGEND:

T = 0.27 indicates shear strength of soil in tons per square foot as determined by hand held "Torvane" vane shear device.

0 1 2 3 4 5
Scale – feet

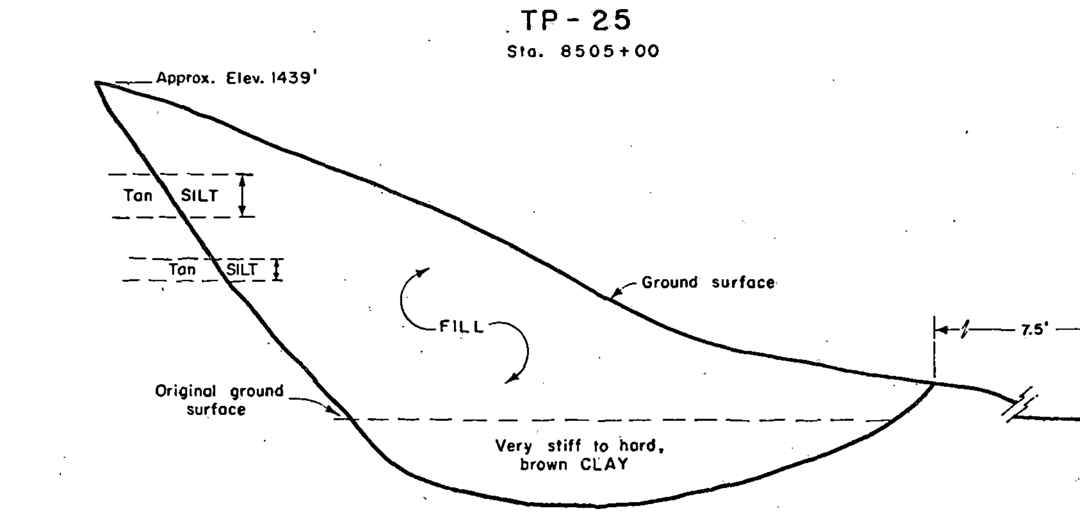
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

TP-20 thru TP-24

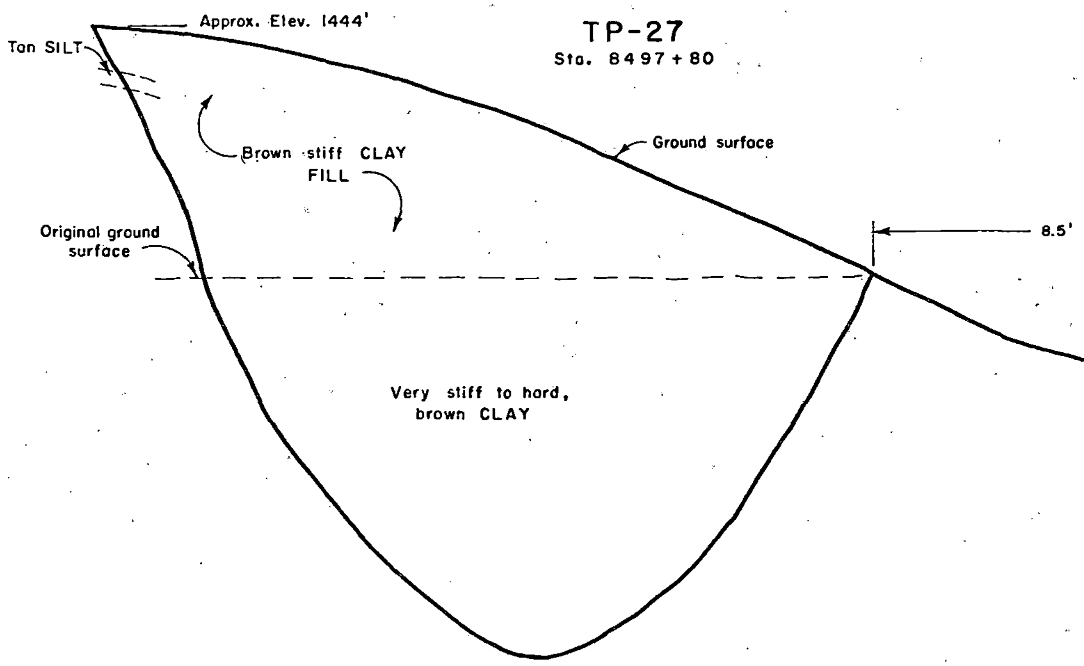
C-268 FEB. 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

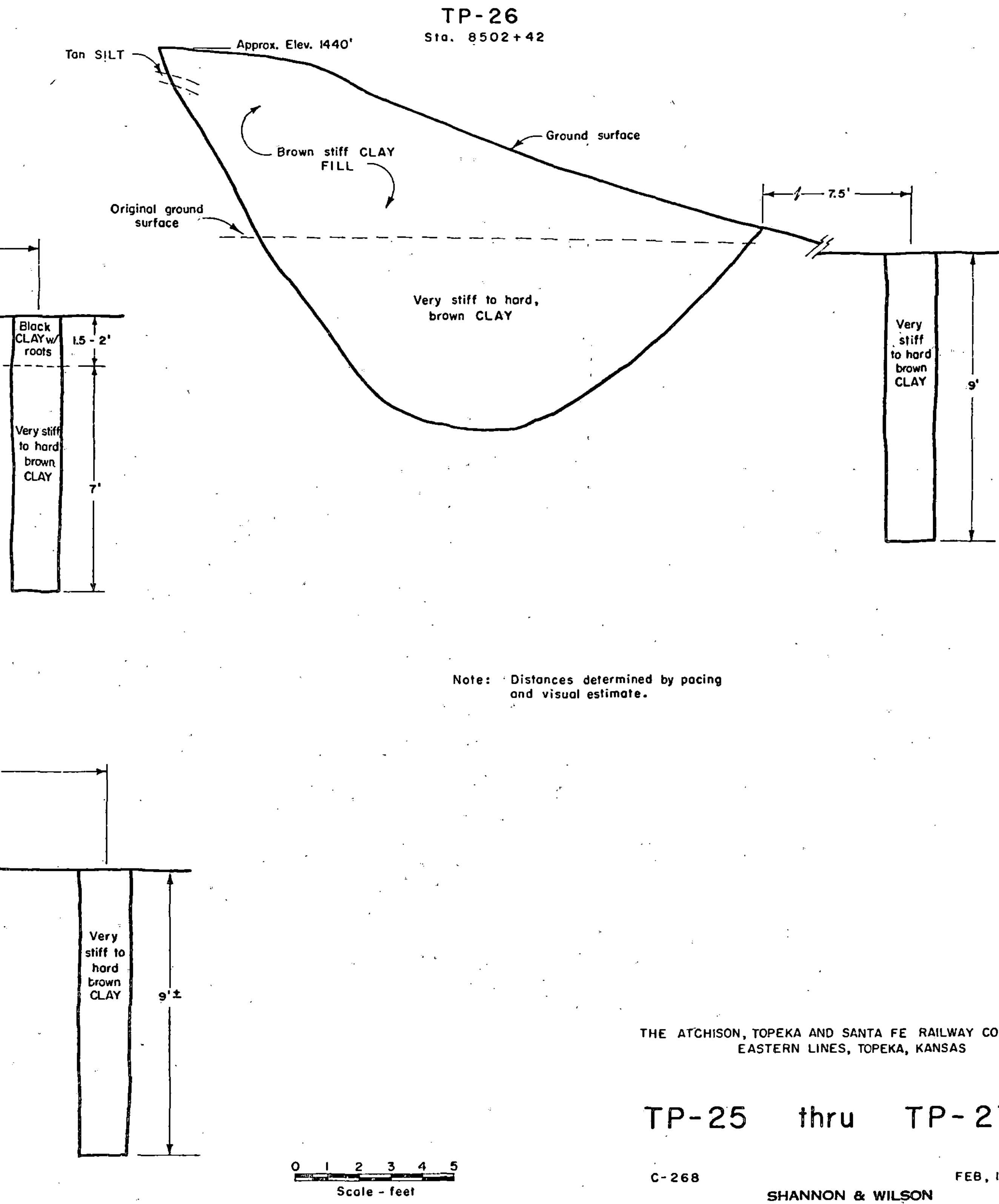
FIG. B4



TP - 25
Sta. 8505+00
Approx. Elev. 1439'
Tan SILT
Tan SILT
Ground surface
FILL
7.5'
Original ground surface
Very stiff to hard, brown CLAY



Approx. Elev. 1444'
TP-27
Sta. 8497+80
Tan SILT
Brown stiff CLAY FILL
Ground surface
Original ground surface
8.5'
Very stiff to hard, brown CLAY



Note: Distances determined by pacing and visual estimate.

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

TP-25 thru TP-27

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. B-5

APPENDIX C

LABORATORY INVESTIGATIONS

APPENDIX C

LABORATORY INVESTIGATIONS

Classification Tests

Visual classification. The following number and type of samples were received at our laboratory:

Type Sample	No.
3" O.D. Shelby Tube (Relatively undisturbed)	25
Jars (disturbed)	11
Bag (disturbed)	41

All samples were visually classified and assigned a Unified Classification Group Index. This information plus test assignments and results are summarized in Table 1.

Water contents and Atterberg limits. Water contents of nearly all samples are included in Table 1. Atterberg limits (ASTM Methods D-423 and D-424) were determined on 23 samples. Results of these tests are presented in Fig. Cl, Plasticity Chart, as well as in Table 1. The soils fall into three distinct groups on the Plasticity Chart. The red-brown clay, the predominant material on-site, generally has Atterberg limits plotting above the A-line with liquid limits exceeding 50; this material is a highly plastic clay. Gray-brown silty clay, comprising approximately the upper one foot of undisturbed soil, has Atterberg limits also falling above the A-line with liquid limits between 30 and 40; this is a medium plasticity clay. Tan clayey silt; which lies immediately above bedrock, has limits plotting near the A-line with liquid limits less than 30; this material is a slightly clayey silt of low plasticity.

Shrinkage limits. Four shrinkage limit tests (ASTM D427) were performed and the results are tabulated in Table 1. The shrinkage limit is the water content below which further reduction of the water content by drying is not accompanied by a decrease in volume. By comparing in-situ water content to shrinkage limit, one gets an indication of the material shrinkage potential. Shrinkage limits from three tests on red-brown clay ranged from 10.5 to 12.1 per cent. One test on tan silt produced a shrinkage limit of 26.8 percent.

Grain size and specific gravity. Two hydrometer tests (ASTM D422) were performed to determine the grain size distribution of the highly plastic clays. The results are included as Fig. C2. Ninety-eight percent of each sample passed the #200 sieve and from 47 to 57 percent was finer than two microns (0.002mm). Four specific gravity determinations (ASTM D854) were made on medium and highly plastic clays; the specific gravities ranged from 2.54 to 2.65 and averaged 2.60. They are also tabulated in Table 1.

Strength Tests

Unconfined compressive tests. A total of 18 unconfined compressive strength tests were performed on samples trimmed from three-inch diameter Shelby tubes. The stress-strain curves for these samples are contained in Fig. C3. Samples were either tested with the same diameter as they were extruded from the tubes, 2.82 inches, or they were trimmed to 1.4 inches diameter. Samples lengths were generally 2.0 to 2.5 times the sample diameters. Test procedures followed ASTM D2166-66. Tests were conducted on medium to highly plastic clays and the strengths ranged from 0.6 tsf to 6.7 tsf for samples with water contents of 35 percent and 17 percent, respectively.

Triaxial tests. Triaxial tests were conducted on three highly plastic clay samples. Sample 2 from Boring B-24 was tested under unconsolidated undrained conditions (Q test) with 0.703 Kg/cm^2 confining pressure. It failed at a deviator stress of 1.16 Kg/cm^2, or a shear strength of 0.58 Kg/cm^2. Results of this test are presented in Fig. C3.

Two specimens from Sample 2 of Boring B-23 were tested in Q tests with pore pressure measurements and with confining pressures of 1.05 Kg/cm^2. Both samples failed at shear strengths of 0.70 Kg/cm^2, see Fig. C4. The Q test samples were 2.85 inches diameter and 6.0 inches in length.

Three specimens from Sample 2 of Boring B-30 were tested under consolidated undrained conditions with pore pressure friction (ϕ') for the highly plastic clay. The specimens were 1.4 inches in diameter, 3.5 inches long, and were tested under effective cell pressures, after deducting the back pressure, of 1.09, 2.81, and 4.25 Kg/cm^2. The test produced a (ϕ') of 19°. This value is not as conclusive as desired because of difficulties in completely saturating these highly plastic clay specimens prior to shearing them. The test results are shown on Fig. C5.

Tests on Compacted Samples

Modified AASHO compaction tests. Eight modified AASHO compaction tests (ASTM D1557-58T) were conducted on materials from potential borrow areas. Seven materials were highly plastic clays and one material was of medium plasticity. Results from five compaction tests are shown in Fig. C6 along with the Atterberg limits for the samples. The remaining three compaction tests are shown in Fig. C7 through C9. The ranges and averages are tabulated below for maximum dry density, optimum moisture content, in-situ moisture content, and difference between in-situ and optimum moisture contents.

Compaction Test Results (8 tests)

	Range	Average
Maximum dry density, pcf	97.0 to 111.5	106.1
Optimum moisture content, %	15.6 to 24.3	19.7
In-situ moisture content, %	22.1 to 28.9	25.8
In-situ minus optimum moisture content, %	13.1 to - 1.4	6.1

Strength, swell, and shrinkage tests. Unconfined compressive strengths, swell tests, and shrinkage tests were performed on several compaction test specimens in the three compaction tests shown in Fig. C7, C8, and C9. The purpose of these tests was to determine the variation of strength, modulus, swell, and shrinkage with moisture content for materials to be used in embankment construction. The unconfined strength tests were performed on 1.4-inch diameter specimens 3.5 inches long which were carved from the compaction sample. Shrinkage test specimens were carved to 1.4 inch diameter and 2.0 inches in length. They were allowed to air dry in the laboratory until no further weight decrease occurred. The drying period was from five to eight days. The percentage reduction in volume is noted adjacent to the compaction points in Fig. C7 and C9. The swell specimens were also carved from the compaction sample into consolidometer rings 2.50 inches in diameter and 0.5 inches in height. The specimen was trimmed to a thickness of 0.3 to 0.4 inches to allow room for swell in the ring. The ring and sample were placed in a consolidometer with a 0.063 Kg/cm^2 surcharge, submerged in water, and allowed to swell until volume change was essentially complete. Maximum percentages of swell, shrinkage and unconfined compressive strength in Kg/cm^2 are shown in Fig. C7, C8, and C9 for the compaction test specimens. The swell, shrinkage, and strength test results are plotted versus difference in water content between the compaction specimen and optimum moisture content in Fig. 4.

Material Response Studies

Consolidation test. Consolidation tests were conducted on medium and highly plastic clay samples. Both samples were trimmed into 2.5 inch diameter consolidometer rings 0.744 inches in height. The samples were loaded to 32 tsf with one rebound to about 0.1 tsf occurring midway in the loading program. The tests were performed to determine compressibility of the natural soils which will underlie the embankment. Test results are shown in Fig. C10.

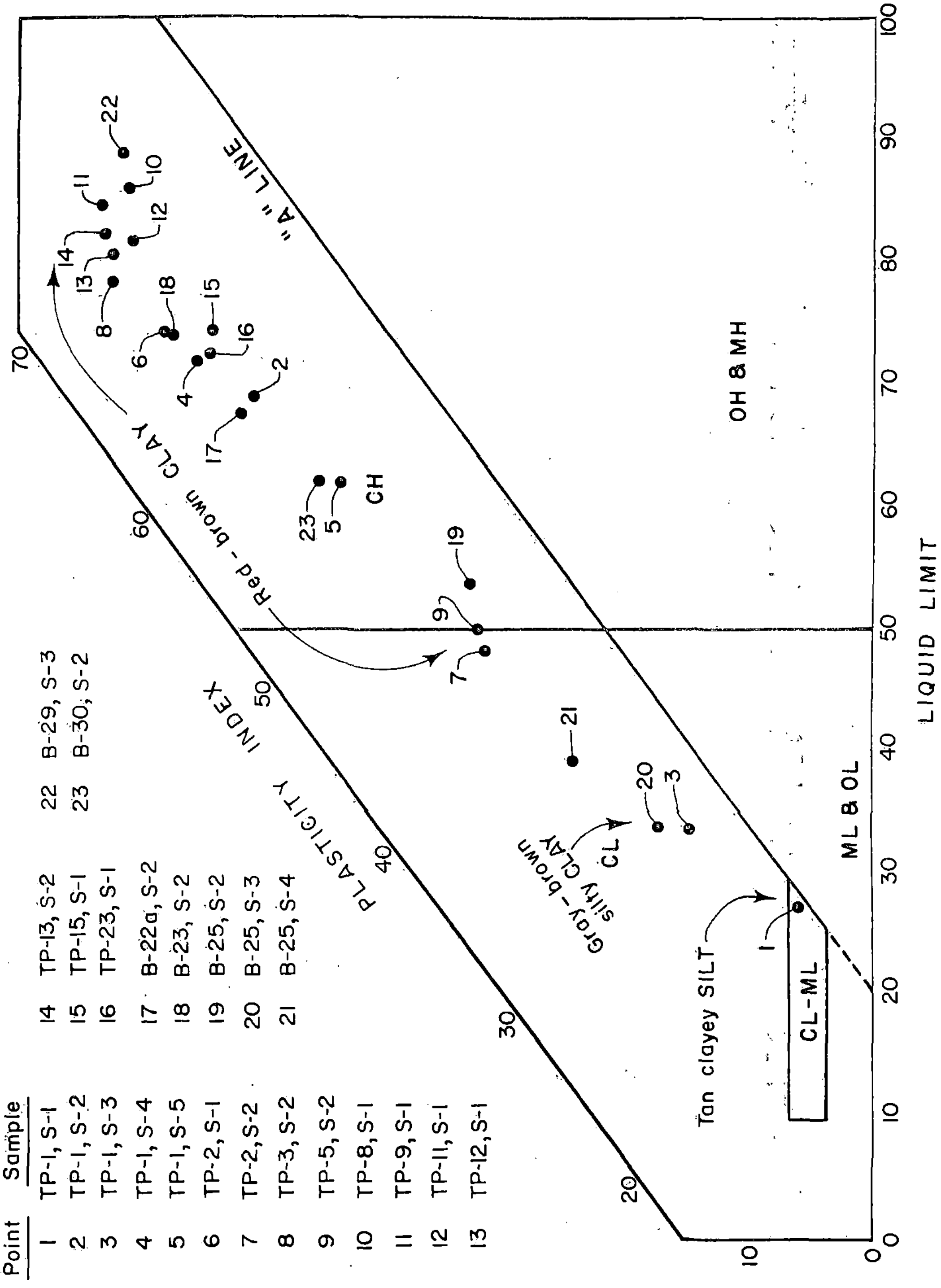
Pulsating load test Pulsating load tests were performed on three samples to determine their elastic moduli under repeated loads at several stress levels. Materials tested included undisturbed Samples 2 and 3, respectively, from Borings B-25 and B-29 plus a compacted specimen from Sample 3 of Test Pit 2. All three materials were highly plastic clays. Sufficient compaction energy was given Sample 3 from Test Pit 2 to achieve a specimen at the approximate optimum moisture content and maximum dry density (Modified AASHO).

Samples two inches in diameter and five inches in length were placed in a triaxial cell and consolidated under a cell pressure of 44 psi with a back pressure of 30 psi. Thus, the effective confining pressure was one ton per square foot. After consolidation the sample was tested without drainage by applying about 20 cycles of loading in successive stress levels of 0.5, 1.0, and 2.0 tsf. The loading rate was one cycle per second with each loading pulse lasting 0.4 second. The load was measured by a load cell fixed to the top of the loading piston. The load was applied by means of compressed air with a double-acting bellofram air cylinder through a Modernair single solenoid valve. Strain was measured with a linear variable differential transformer (LVDT). The load cell, LVDT, and pore water and cell pressure transducers were connected to Sanborn 321 dual channel carrier amplifier recorders. Stress versus strain for generally cycles 1, 10, and 20 were taken from the data records and plotted in Fig. C11, C12, and C13 for each of the three samples. Cell and pore pressure are not plotted since there was no change through the testing program, except for erratic pore pressure readings when two samples failed. The deflection transducer was reset to zero after the loading at each intensity. Specimens from Samples 2 and 3 of Borings B-25 and B-29, respectively, both failed at the first cycle of 2.0 tsf loading and results could not be plotted.

Vibration tests. Specimens from the same samples as the pulsating load tests were also subjected to vibration tests. Samples were also two inches in diameter and five inches long. The samples were confined in a triaxial cell under successive pressures of 10, 20, 50, and 100 psi and the natural vibration frequencies (resonance) in the longitudinal and torsional modes were determined. From the longitudinal and torsional natural frequencies it is possible to calculate the modulus of elasticity in compression, E, and the modulus of elasticity in shear, G, for each confining pressure. The test results are plotted in Fig. C14, C15, and C16 and include plots of torsional and longitudinal natural frequencies as well as E and G versus confining pressure. The samples were failed in unconfined compression following the vibration tests, and the results are noted in Fig. C14 and C16. Moduli resulting from the vibration tests, pulsating load tests, and unconfined compressive tests on the vibration test specimens are summarized in Table C1. The initial tangent modulus is listed for the unconfined compressive test. For the pulsating load test, the modulus was determined on the 20th loading cycle of 1.0 Kg/cm^2 for both the vibration and pulsating load tests.

TABLE C1 - SUMMARY OF STATIC AND DYNAMIC MODULUS STUDIES

Material	δ_d, pcf	w, %	Vibration Test σ_3=1.0 Kg/cm^2 G, Kg/cm^2	E, Kg/cm^2	Pulsating Load Test σ_3=1.0 Kg/cm^2 $\sigma_1-\sigma_3$=1.0 Kg/cm^2 E, Kg/cm^2 @ 20th Cycle	Unconfined Comp. Test Initial Tangent Modulus E, Kg/cm^2
B25,S2 (Undisturbed)	100.6	22.2	503	842		97
	95.7	26.5			127	
B29,S3 (Undisturbed)	88.1	32.4	417	877		37
	82.2	35.1			124	
TP2,S3 (Compacted)	106.1	19.6	453	763		286
	106.0	21.2			498	



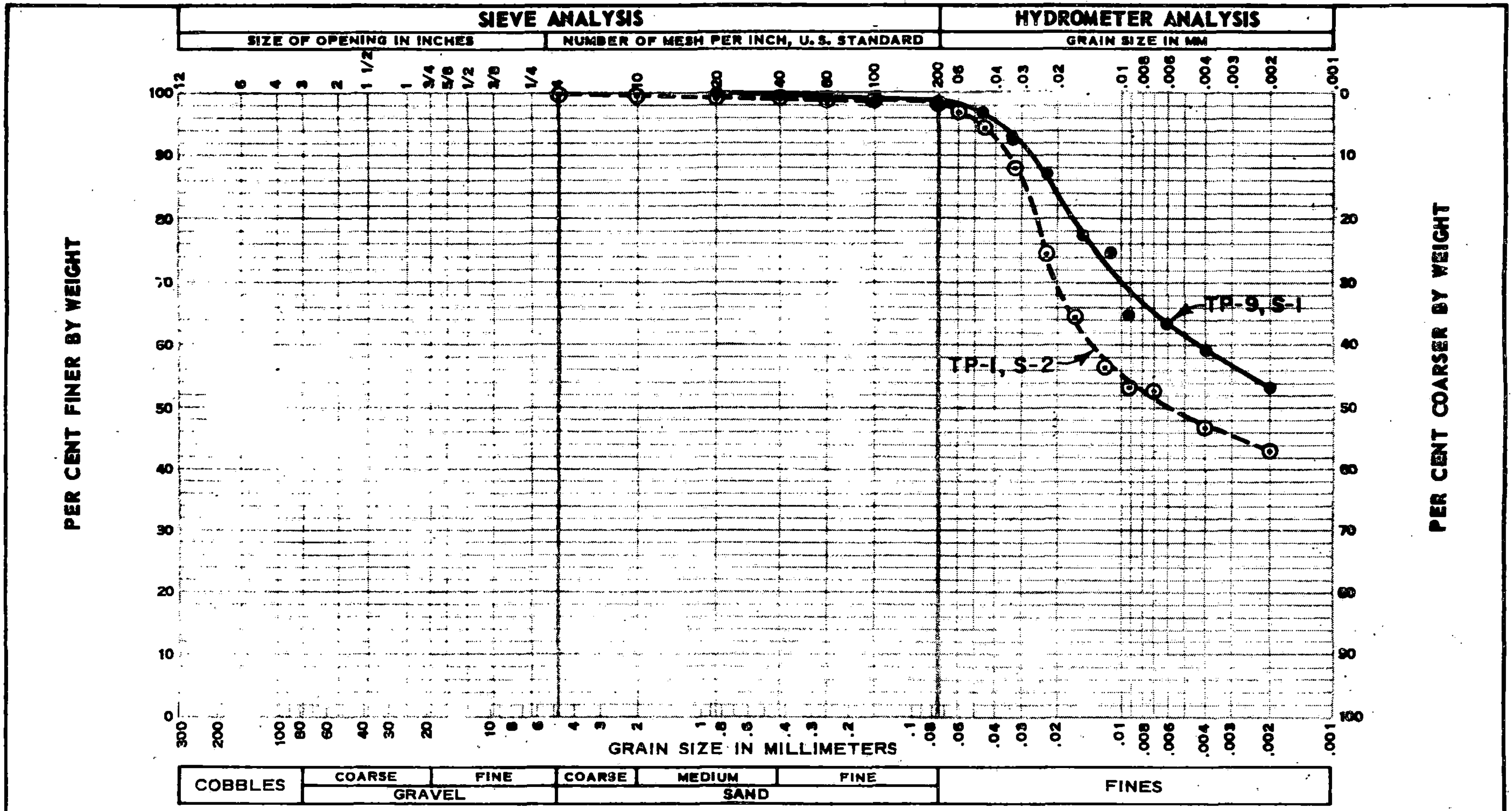
Point	Sample
1	TP-1, S-1
2	TP-1, S-2
3	TP-1, S-3
4	TP-1, S-4
5	TP-1, S-5
6	TP-2, S-1
7	TP-2, S-2
8	TP-3, S-2
9	TP-5, S-2
10	TP-8, S-1
11	TP-9, S-1
12	TP-11, S-1
13	TP-12, S-1
14	TP-13, S-2
15	TP-15, S-1
16	TP-23, S-1
17	B-22a, S-2
18	B-23, S-2
19	B-25, S-2
20	B-25, S-3
21	B-25, S-4
22	B-29, S-3
23	B-30, S-2

PLASTICITY CHART

C-268 DEC, 1970

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C1



SAMPLE NO.	DEPTH -FT.	U.S.C.	CLASSIFICATION	NAT. W.C. %	LL	PL	PI
TP-I, S-2		CH	Very stiff, red-brown CLAY Specific gravity 2.65	24.9 23.0	69	18	51
TP-9, S-I		CH	Very stiff, gray-brown CLAY Specific gravity 2.54	24.9 23.2	84	21	63

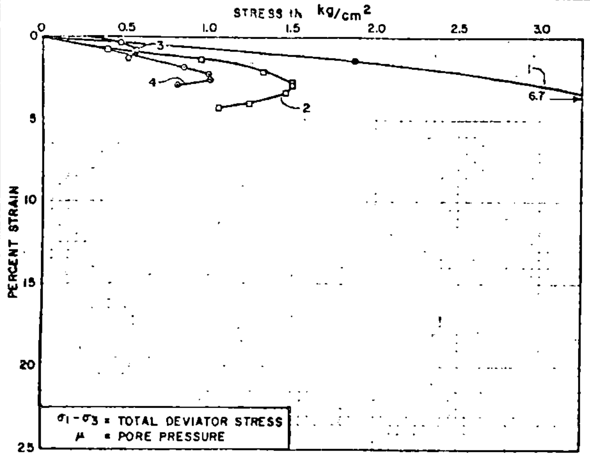
GRAIN SIZE CLASSIFICATION

TEST PIT I, SAMPLE 2
TEST PIT 9, SAMPLE I

C-268 DEC, 1970

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C2



TYPE OF TEST
■Q_u □R □S □PRESATURATED □BACKPRESSURE ____psi

REMARKS: 1. Slickensided 2. ——
3. Slickensided

σ_3 = CHAMBER PRESSURE = θ tons/sq. ft.

SPECIMEN

CLASSIFICATION 1. Brown CLAY (CH) 2. Mottled gray-brown, tan & black, silty CLAY (CL) 3. Mottled gray, brn, tan & blk., silty CLAY (CL)

□COMPACTED ■UNDISTURBED

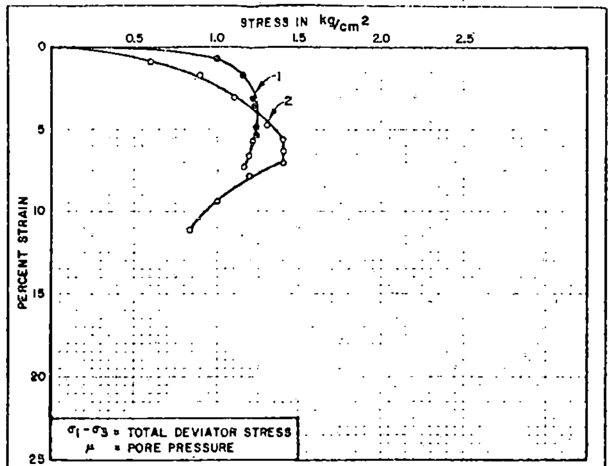
COMPACTIVE EFFORT:

Test No.	1	2	3	4
Sample No.	1	3	5	4
Height (in.)	3.5	6.0	6.0	4.5
Diam. (in.)	1.4	2.85	2.83	2.05
Unit Wt. (pcf)	99.6	100.8	86.0	98.2
Water Content	19.2	22.2	35.2	26.0

UNCONFINED COMPRESSION TEST
STRESS vs STRAIN

BORING B-22a
SAMPLES 1, 3, 4, & 5

C-268 FEB, 1971



TYPE OF TEST
■Q_u □R □S □PRESATURATED □BACKPRESSURE ____psi

REMARKS:

σ_3 = CHAMBER PRESSURE = θ tons/sq. ft.

SPECIMEN

CLASSIFICATION 1. Red-brown CLAY (CH)
2. Red-brown CLAY (CH)

□COMPACTED ■UNDISTURBED

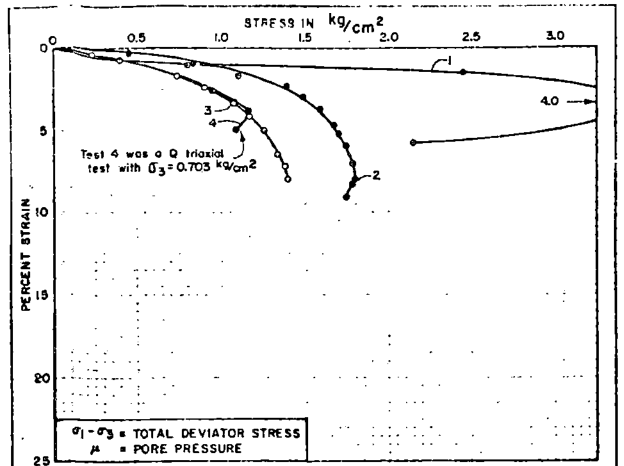
COMPACTIVE EFFORT:

Test No.	1	2
Sample No.	1	2
Height (in.)	6.00	3.15
Diam. (in.)	2.86	1.46
Unit Wt. (pcf)	95.5	91.7
Water Content	25.2	25.3

UNCONFINED COMPRESSION TEST
STRESS vs STRAIN

BORING B-23
SAMPLES 1 and 2

C-268 FEB, 1971



TYPE OF TEST
■Q_u □R □S □PRESATURATED □BACKPRESSURE ____psi

REMARKS:

σ_3 = CHAMBER PRESSURE = θ tons/sq. ft.

SPECIMEN

CLASSIFICATION 1. Red-brn. to gray-brn. CLAY (CH) 2. Gray-brn. CLAY (CH) 3. Dk. red-brn. CLAY (CH)

□COMPACTED ■UNDISTURBED

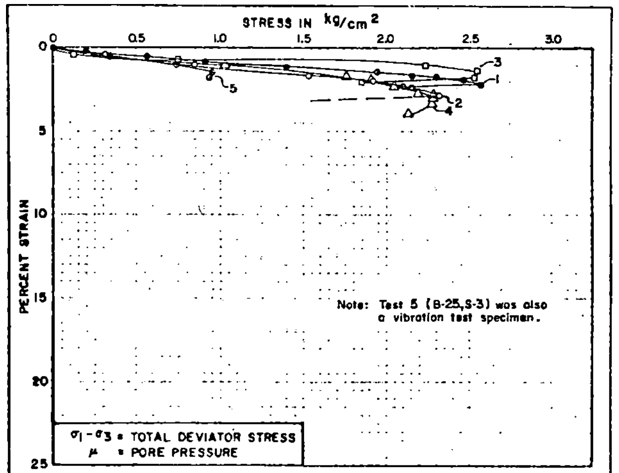
COMPACTIVE EFFORT:

Test No.	1	2	3	4
Sample No.	1	3 top	3 bott.	2
Height (in.)	2.8	6.0	6.0	6.0
Diam. (in.)	1.4	2.83	2.82	2.85
Unit Wt. (pcf)	100.5	91.6	92.6	92.0
Water Content	18.2	29.6	29.6	25.2

UNCONFINED COMPRESSION TEST
STRESS vs STRAIN

BORING B-24
SAMPLES 1, 2, and 3

C-268 FEB, 1971



TYPE OF TEST
■Q_u □R □S □PRESATURATED □BACKPRESSURE ____psi

REMARKS:

σ_3 = CHAMBER PRESSURE = θ tons/sq. ft.

SPECIMEN

CLASSIFICATION 1. Brn. silty CLAY (CL) 2. Brn. silty CLAY (CL) 3. Red-brown CLAY (CH) 4. Red-brn. CLAY (CH)

□COMPACTED ■UNDISTURBED

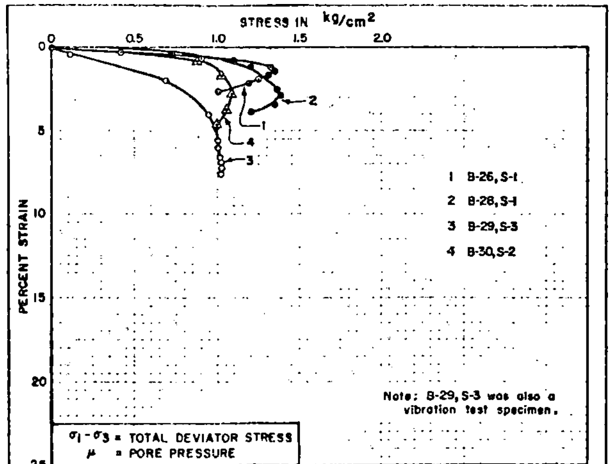
COMPACTIVE EFFORT:

Test No.	1	2	3	4	5
Sample No.	3 top	3 bott.	5	6	2
Height (in.)	6.0	6.0	6.0	6.0	4.88
Diam. (in.)	2.87	2.81	2.86	2.86	2.05
Unit Wt. (pcf)	104.0	107.9	102.1	102.1	100.6
Water Content	21.5	22.2	21.8	23.4	22.2

UNCONFINED COMPRESSION TEST
STRESS vs STRAIN

BORING B-25,
SAMPLES 2, 3, 5, & 6

C-268 FEB, 1971



TYPE OF TEST
■Q_u □R □S □PRESATURATED □BACKPRESSURE ____psi

REMARKS:

σ_3 = CHAMBER PRESSURE = θ tons/sq. ft.

SPECIMEN

CLASSIFICATION 1. Brown CLAY (CH); 2. Dark red-brown CLAY (CH) 3. Brown CLAY (CH); 4. Red-brown CLAY (CH)

□COMPACTED ■UNDISTURBED

COMPACTIVE EFFORT:

Test No.	1	2	3	4
Height (in.)	6.0	6.0	4.96	6.0
Diam. (in.)	2.86	2.88	2.05	2.85
Unit Wt. (pcf)	86.8	90.5	88.1	95±
Water Content	28.5	28.9	32.4	26.6

UNCONFINED COMPRESSION TEST
STRESS vs. STRAIN

B-26, S-1; B-28, S-1;
B-29, S-3; B-30, S-2

C-268 FEB, 1971

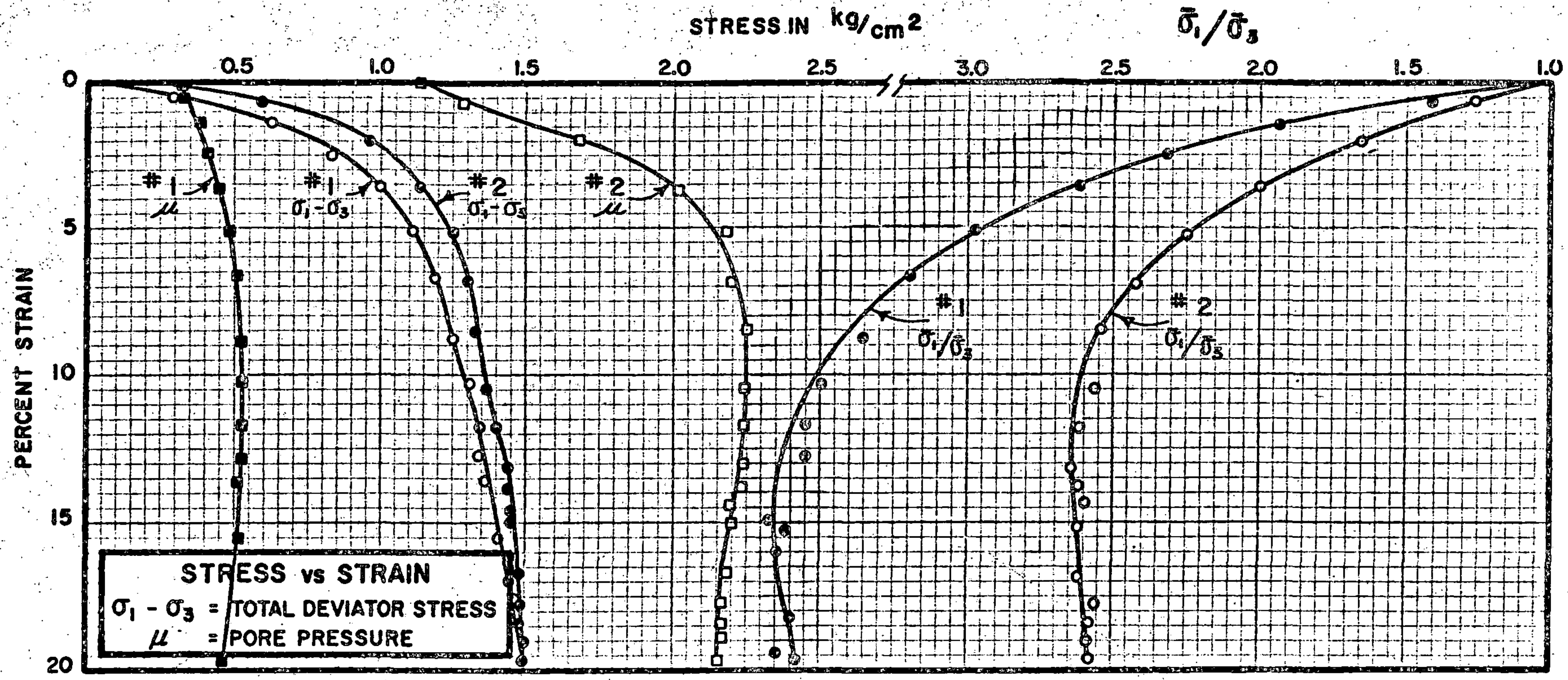
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

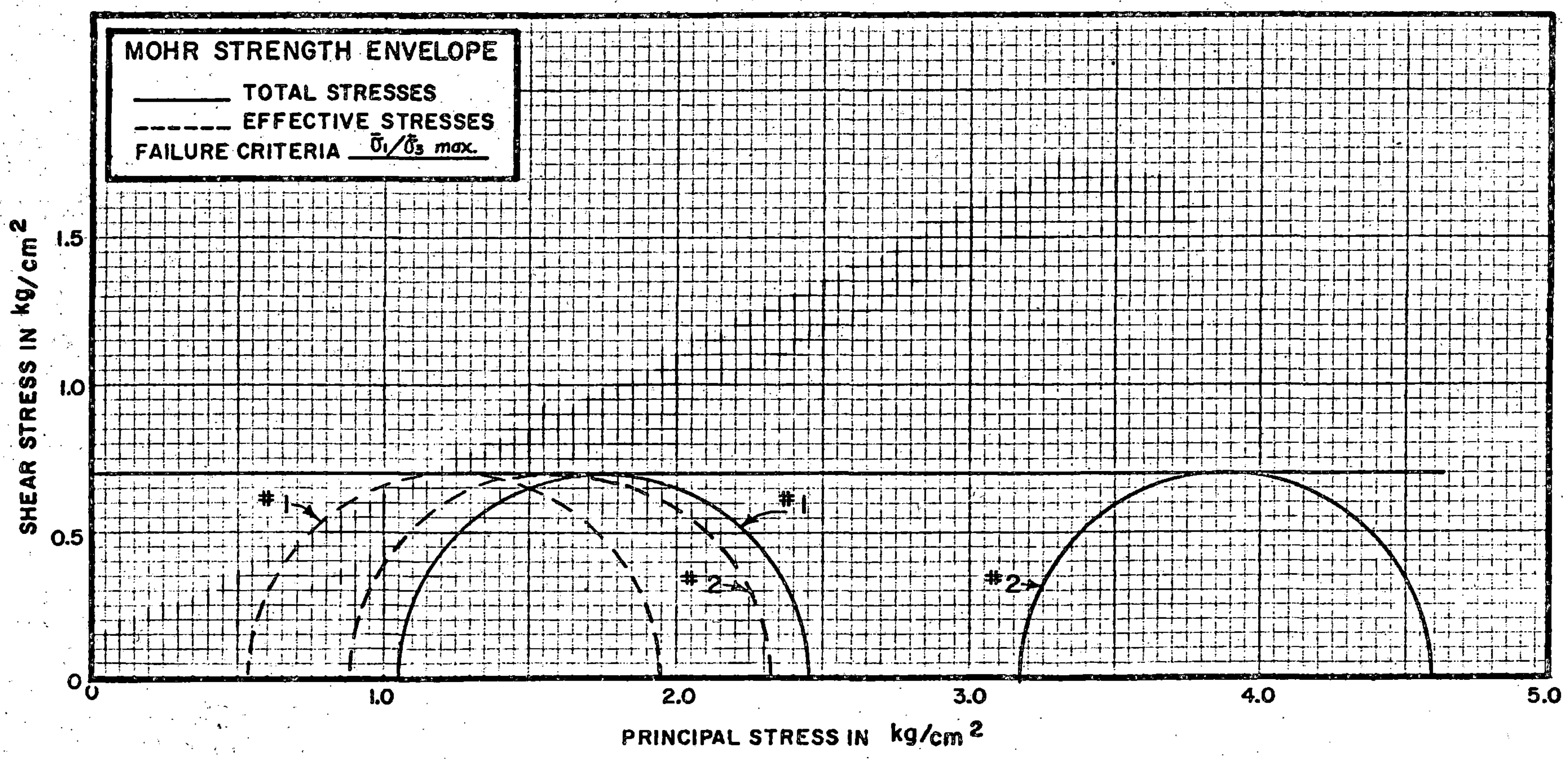
COMPRESSION TESTS ON UNDISTURBED SAMPLES

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C3





TYPE OF TEST

■ Q ☐ R ☐ S ■ UNDISTURBED ☐ COMPACTED
☐ PRESATURATED ☐ BACK PRESSURE

TEST NO.	1	2		
BORING NO.	23	23		
SAMPLE NO.	2	2		
CHAMBER PRESSURE, kg/cm^2	1.05	3.16		
HEIGHT, inches	6.0	6.0		
DIAMETER, inches	2.854	2.856		
UNIT WT., pcf. Wet	121.9	121.6		
W.C. %, BEFORE TEST	25.1	25.1		
W.C. %, AFTER TEST				

COMPACTIVE EFFORT

_____ lb. hammer _____ inch drop
_____ layers _____ blows/layer
_____ % compaction @ _____

REMARKS: Sample description: Stiff, red-brown CLAY (CH)

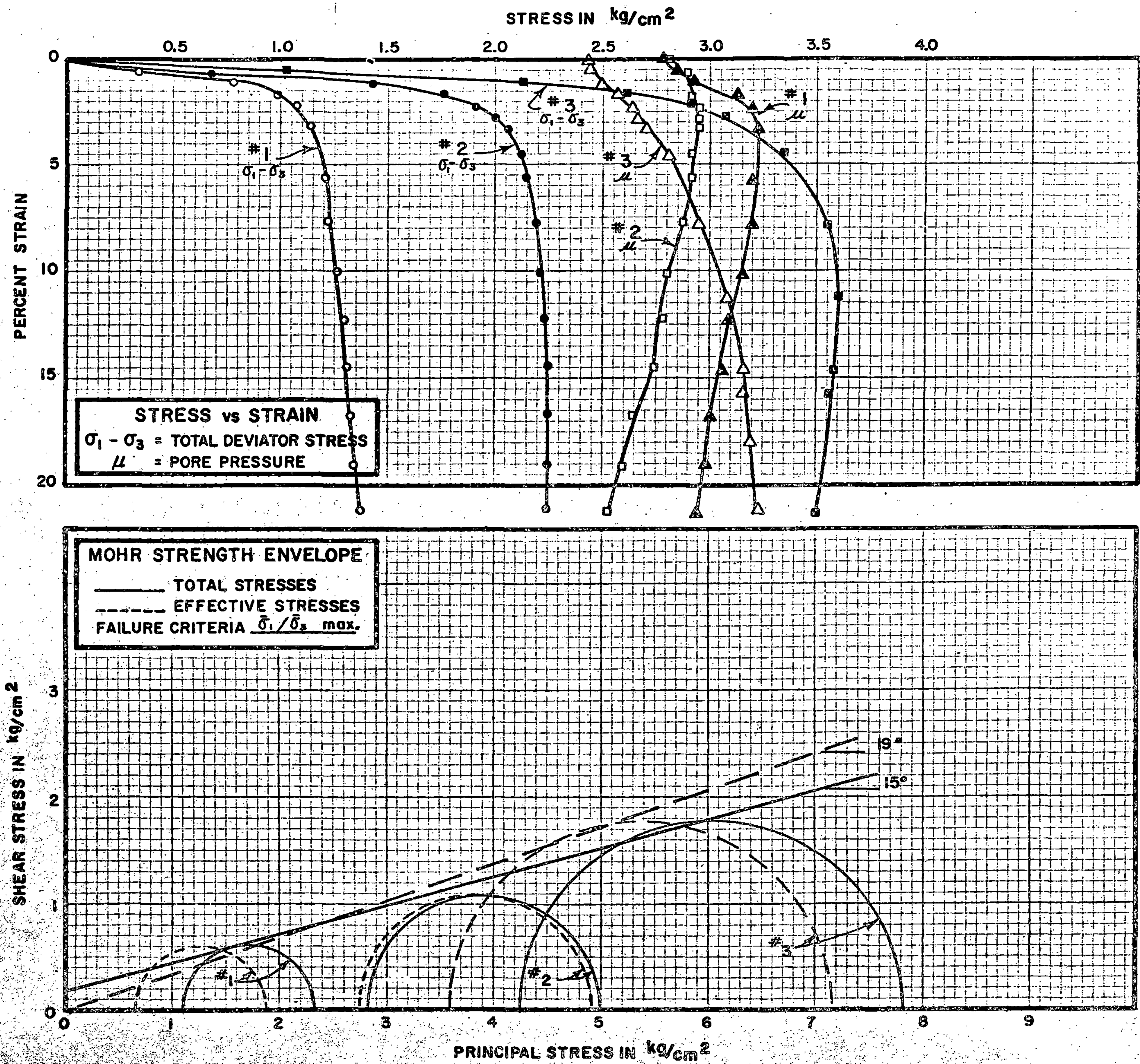
TRIAXIAL COMPRESSION TEST

BORING B-23, SAMPLE 2

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C4



TYPE OF TEST

☐ Q ■ R̄ ☐ S ■ UNDISTURBED ☐ COMPACTED

☐ PRESATURATED ■ BACK PRESSURE

TEST NO.	1	2	3	
BORING NO.	30	30	30	
SAMPLE NO.	2	2	2	
CHAMBER PRESSURE, kg/cm²	3.87	5.62	6.68	
HEIGHT, inches	3.49	3.50	3.50	
DIAMETER, inches	1.38	1.40	1.40	
UNIT WT., pcf. Wet	121.1	119.8	122.0	
W.C. %, BEFORE TEST	27.5 ±	27.5 ±	27.5 ±	
W.C. %, AFTER TEST	27.0	28.5	26.6	
Back pressure kg/cm²	2.78	2.81	2.43	
Effective σ_3 kg/cm²	1.09	2.81	4.25	

COMPACTIVE EFFORT

_______ lb. hammer _______ inch drop

_______ layers _______ blows/layer

_______ % compaction @ _______

REMARKS: Specimens were not completely saturated before testing.

Sample description: Stiff, red-brown CLAY (CH)

TRIAXIAL COMPRESSION TEST

BORING B-30, SAMPLE 2

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C5

COMPACTION TEST

Test Pit Samples
SHANNON & WILSON

☐ STANDARD AASHO (AASHO T 99-57, ASTM D698-58T)
■ MODIFIED AASHO (AASHO T180-57, ASTM D1557-58T)
☐ OTHER

PROJECT: Test Embankment
JOB NO.: C-268 DATE: Sept, 1970
SAMPLE NO.: 1-4 3-2 8-1 12-1 23-1
DEPTH:

	1-4	3-2	8-1	12-1	23-1
MAX. DRY DENSITY LBS./CU. FT.	106.7	107.3	97.0	102.8	107.6
OPTIMUM W.C., %	19.6	19.3	24.3	21.0	19.5
NATURAL W.C., %	26.7	26.4	22.9 to 24.3	26.2	22.1

HAMMER WT., LBS. 10
DROP, IN. 18
NO. LAYERS 5
NO. BLOWS/LAYER 25
DIA. MOLD., IN. 4.4
HEIGHT MOLD., IN. 4.65
VOL. MOLD., CU. FT. 0.03241
COMPACTIVE EFFORT-FT LBS/CU.FT. 56, 250
SAMPLE CLASSIFICATION

	1-4	3-2	8-1	12-1	23-1
LL	71.7	78.3	86.1	80.4	72.0
PL	16.5	16.3	25.2	18.6	17.8
PI	55.2	62.0	60.9	61.8	54.2

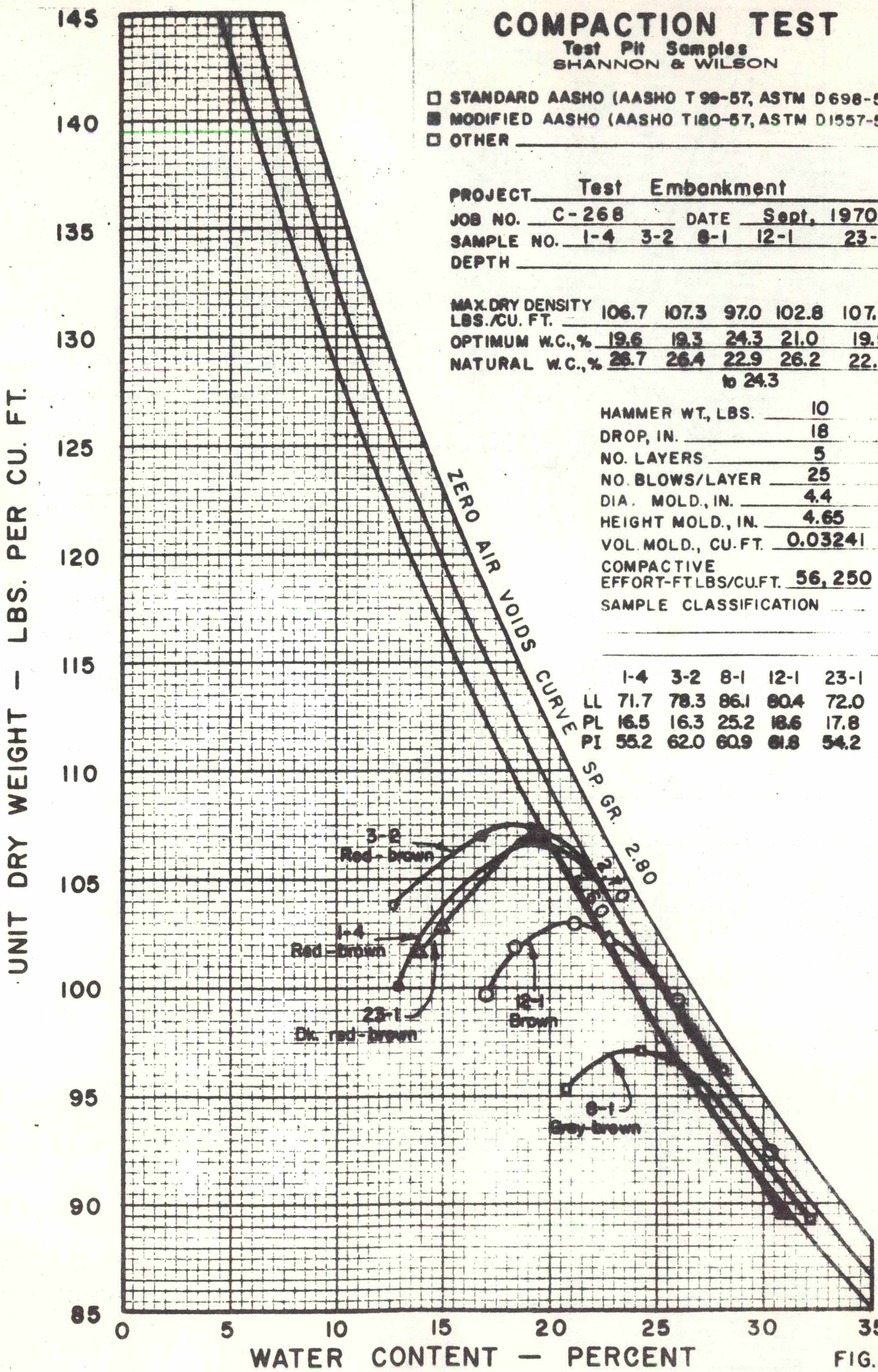
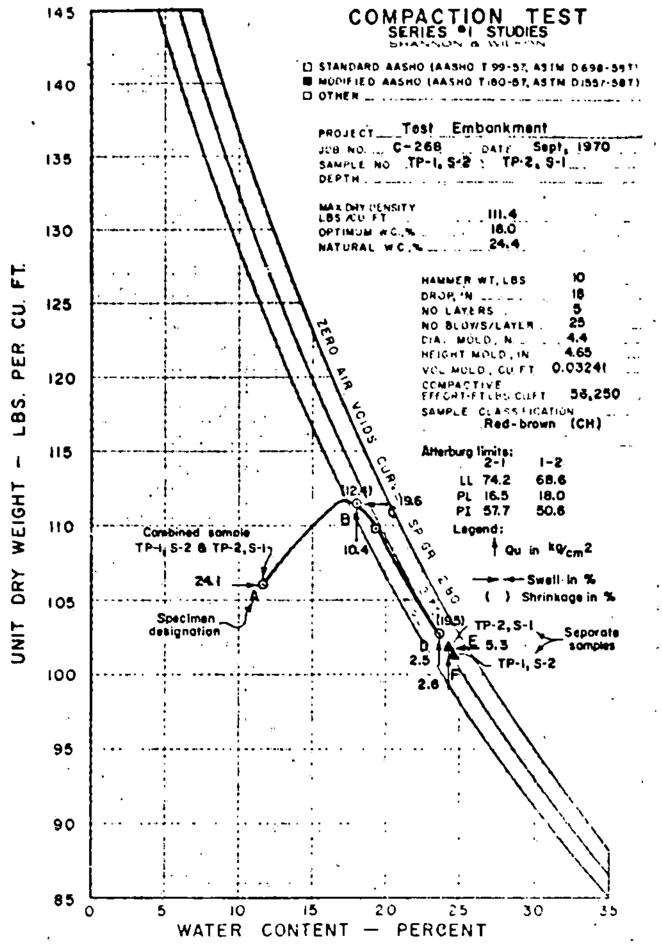
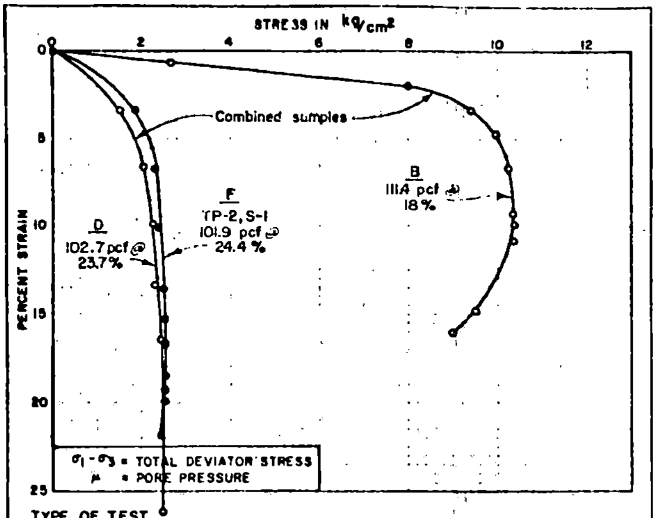


FIG. C6





TYPE OF TEST
■Qu ☐R ☐S ☐PRESATURATED ☐BACKPRESSURE ____ psi

REMARKS:

σ_3 = CHAMBER PRESSURE = 0 tons/sq. ft.

SPECIMEN CLASSIFICATION Red-brown, CLAY (CH)

■COMPACTED ☐UNDISTURBED

COMPACTIVE EFFORT: Mod. AASHO

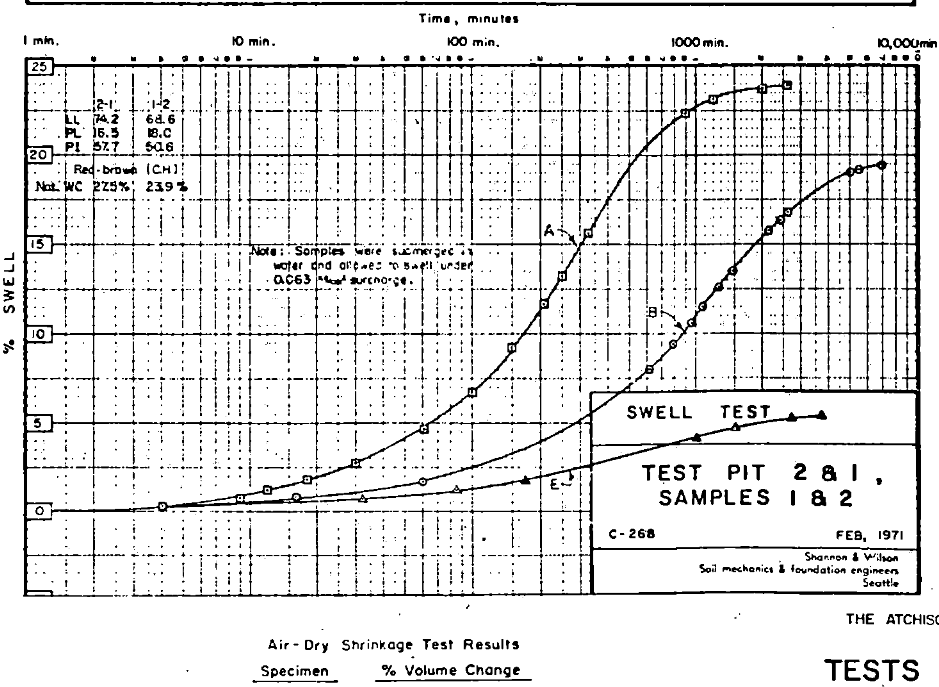
HEIGHT 3.5 Inches
DIAMETER 1.4 Inches
UNIT WEIGHT ____ pcf
WATER CONTENT:
BEFORE TEST ____ %
AFTER TEST ____ %

UNCONFINED COMPRESSION TEST
STRESS vs STRAIN
TEST PIT 2, SAMPLE 1
TEST PIT 1, SAMPLE 2
COMPACTED SERIES

C-268 FEB. 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

TEST PIT	SAMPLE	SPECIMEN	W.C. BEFORE	W.C. AFTER	HEIGHT	DIAMETER		CLASSIFICATION	
2	1	E	24.0	27.5	0.492	2.50	△ Wet side	101.9 @ 24.4	Qu = 2.6 kg/cm²
2 & 1	1 & 2	B	17.8	28.6	0.524	2.50	○ Optimum	111.4 @ 18.0	
2 & 1	1 & 2	A	11.7	25.4	0.511	2.50	☐ Dry side	106.0 @ 11.6	



SWELL TEST

TEST PIT 2 & 1, SAMPLES 1 & 2

C-268 FEB. 1971

Shannon & Wilson
Soil mechanics & foundation engineers
Seattle

Air-Dry Shrinkage Test Results

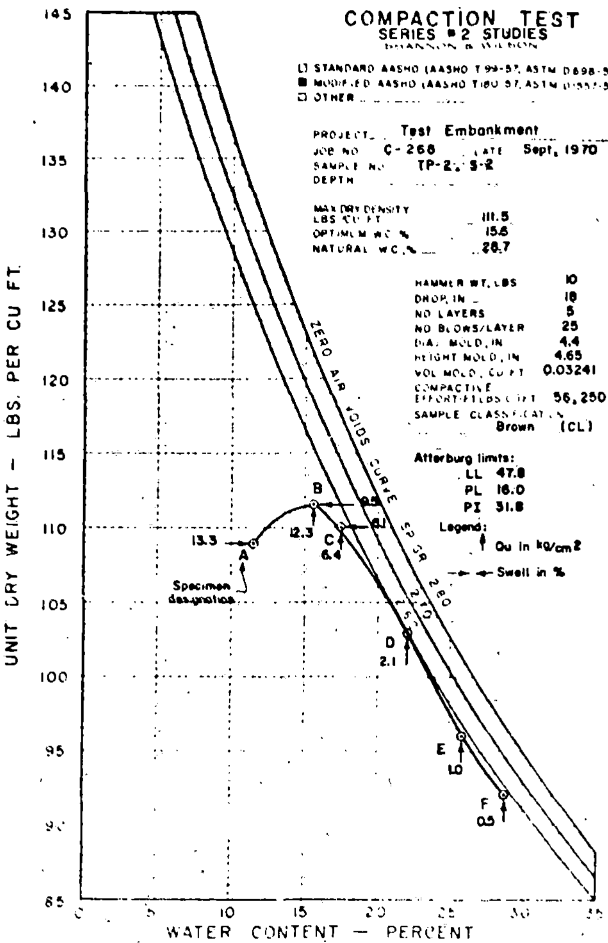
Specimen	% Volume Change
B	12.4
D	19.5

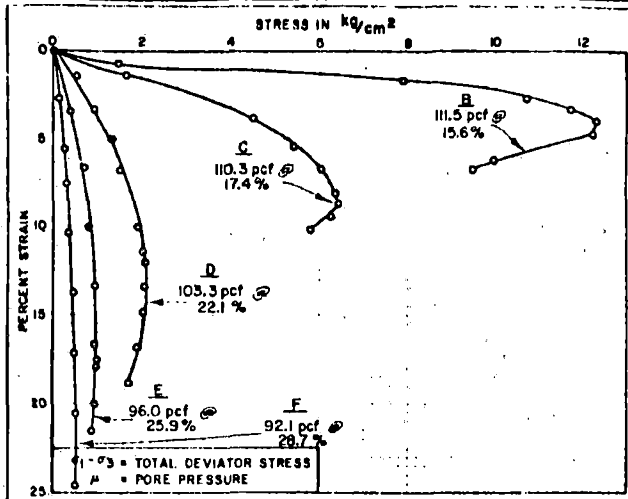
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

TESTS ON COMPACTED SAMPLES
SERIES # 1

C-268 FEB. 1971

FIG. C7





TYPE OF TEST
■Qu ☐R ☐S ☐PRESATURATED ☐BACKPRESSURE ____ psi

REMARKS: ____

σ_3 = CHAMBER PRESSURE = __0__ tons/sq. ft.

SPECIMEN

CLASSIFICATION Brown, silty CLAY (CL)

■COMPACTED ☐UNDISTURBED

COMPACTIVE EFFORT: Mod. AASHO

HEIGHT 3.5 inches

DIAMETER 1.4 inches

UNIT WEIGHT ____ pcf

WATER CONTENT:

BEFORE TEST ____ %

AFTER TEST ____ %

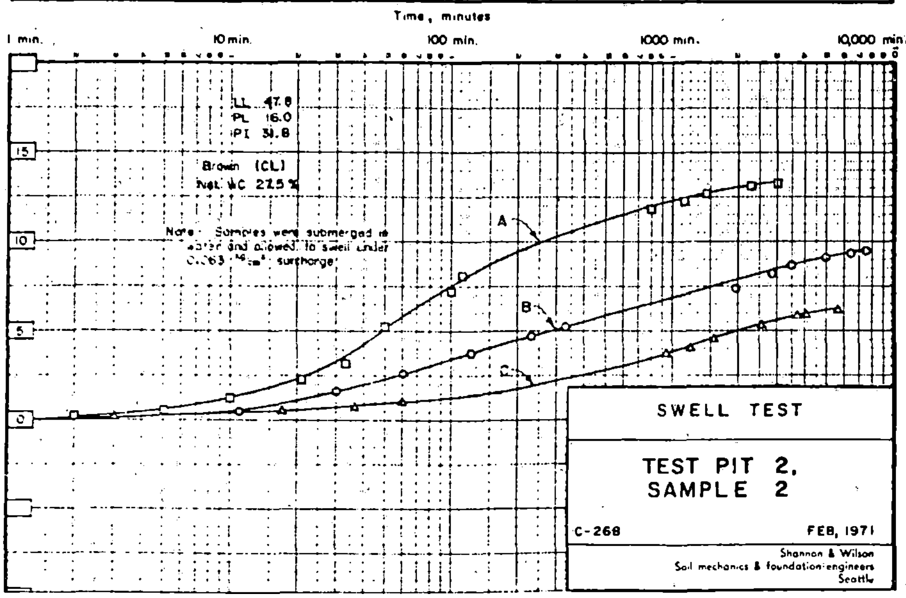
UNCONFINED COMPRESSION TEST
STRESS vs STRAIN

TEST PIT 2, SAMPLE 2
COMPACTED SERIES

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

TEST PIT	SAMPLE	SPECIMEN	W.C. BEFORE	W.C. AFTER	HEIGHT	DIAMETER		CLASSIFICATION	
2	2	C	17.6	21.0	0.496	2.50	△ Wet side	110.3 @ 17.4	Qu = 6.4 kg/cm²
2	2	B	15.4	21.3	0.607	2.50	○ Optimum	111.5 @ 15.6	Qu = 12.3 kg/cm²
2	2	A	11.8	19.2	0.435	2.50	☐ Dry side	108.8 @ 11.5	



SWELL TEST

TEST PIT 2, SAMPLE 2

C-268 FEB, 1971

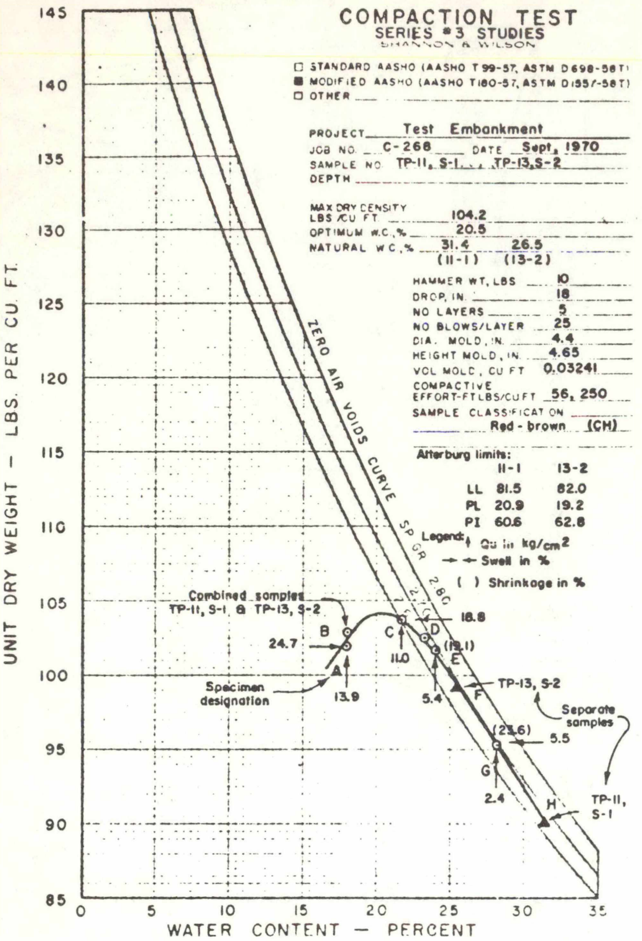
Shannon & Wilson
Soil mechanics & foundation engineers
Seattle

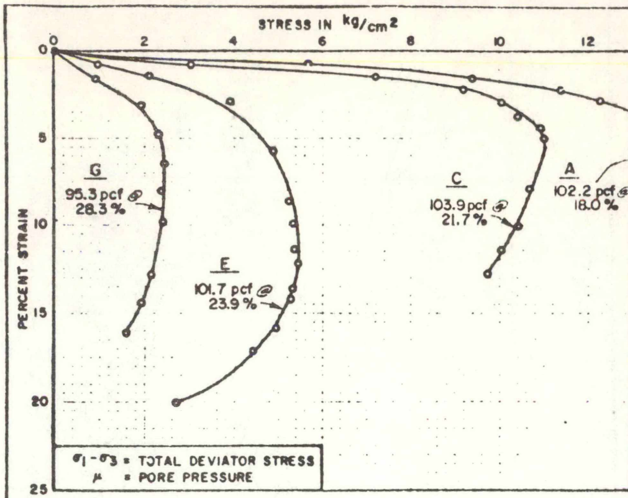
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

TESTS ON COMPACTED SAMPLES
SERIES # 2

C-268 FEB, 1971

FIG. C8





TYPE OF TEST
■ Q_u ☐R ☐S ☐PRESATURATED ☐BACKPRESSURE ____ psi

REMARKS:

σ_3 = CHAMBER PRESSURE = 0 tons/sq. ft.

SPECIMEN

CLASSIFICATION Dark red-brown CLAY (CH)

■COMPACTED ☐UNDISTURBED

COMPACTIVE EFFORT: Mod. AASHO

HEIGHT 3.5 inches

DIAMETER 1.4 inches

UNIT WEIGHT ____ pcf

WATER CONTENT:

BEFORE TEST ____ %

AFTER TEST ____ %

UNCONFINED COMPRESSION TEST
STRESS vs STRAIN

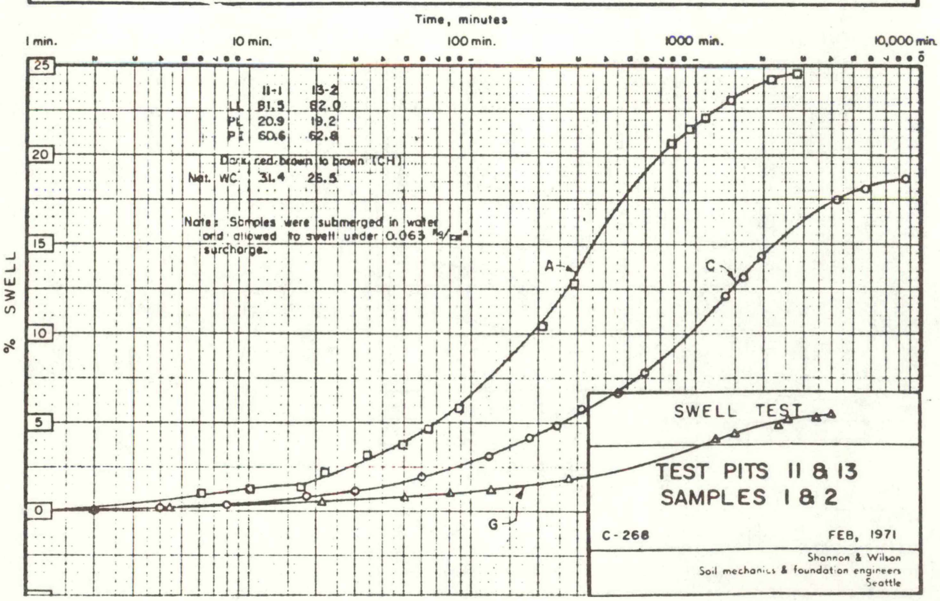
TEST PIT 11, SAMPLE 1
TEST PIT 13, SAMPLE 2

COMPACTED SERIES

C-268 FEB, 1971

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

TEST PIT	SAMPLE	SPECIMEN	W.C. BEFORE	W.C. AFTER	HEIGHT	DIAMETER		CLASSIFICATION	
11 & 13	1 & 2	G	28.4	31.9	0.369	2.50	△ Wet side	95.3 @ 28.3	Qu = 2.38 k/c²
11 & 13	1 & 2	C	21.3	32.5	0.489	2.50	○ Optimum	103.9 @ 21.7	Qu = 11.00 k/c²
11 & 13	1 & 2	A	18.0	32.9	0.461	2.50	☐ Dry side	102.2 @ 18.0	Qu = 13.87 k/c²



Air - Dry Shrinkage Test Results

Specimen	% Volume Change
E	19.1
G	23.6

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

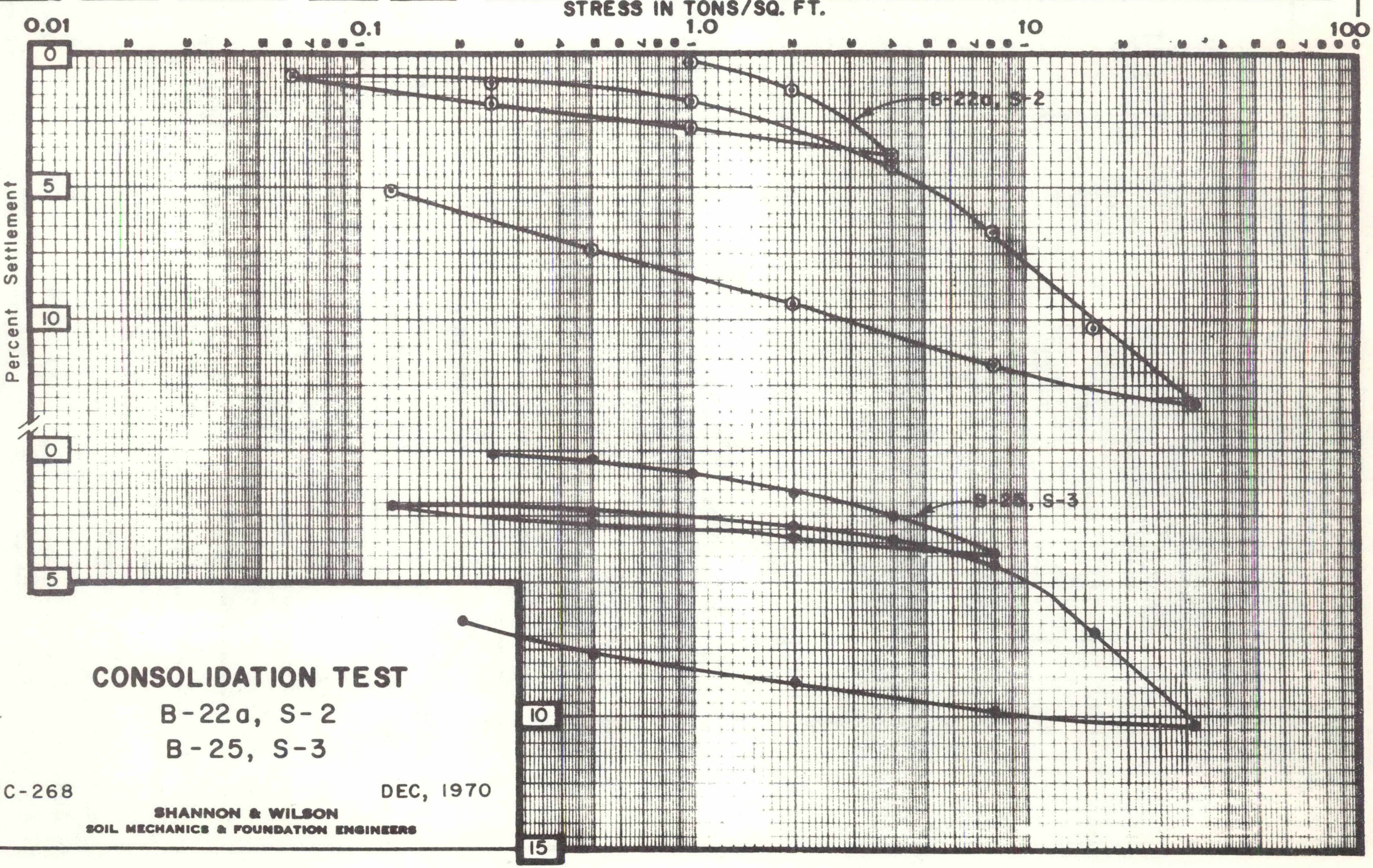
TESTS ON COMPACTED SAMPLE

SERIES # 3

C-268 FEB, 1971

FIG. C 9

BORING	SAMPLE	DEPTH feet	WATER CONTENT, % Before	After	HEIGHT inches	DIAMETER inches	CLASSIFICATION
22a	2	3.8	18.2	14.9	0.744	2.5	Stiff, red-brown CLAY (CH)
25	3	6.1	22.0	18.0	0.744	2.5	Very stiff, brown, silty CLAY (CL)



CONSOLIDATION TEST

B-22a, S-2

B-25, S-3

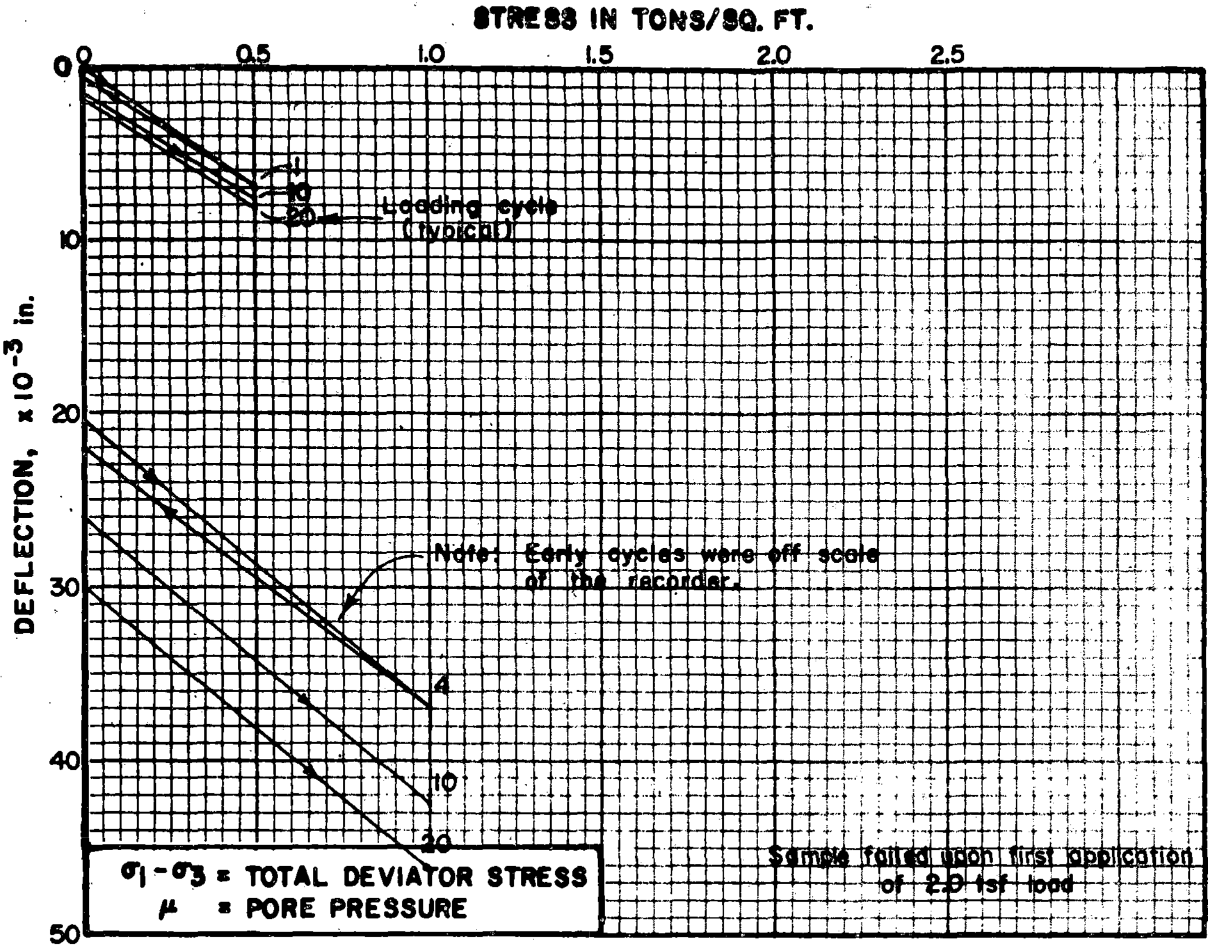
C-268

DEC, 1970

SHANNON & WILSON

SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C 10



TYPE OF TEST

☐Q ■$\bar{R}$ ☐S ■PRESATURATED ■BACKPRESSURE 30 psi

REMARKS: Cell pressure = 44 psi

σ_3 = 1.0 tons/sq. ft.

SPECIMEN

CLASSIFICATION Very stiff, brown CLAY (CH)

FAILURE SKETCH

☐COMPACTED ■UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT 2.05 inches

DIAMETER 4.88 inches

UNIT WEIGHT 95.7 pcf

WATER CONTENT:

BEFORE TEST ______ %

AFTER TEST 26.5 %

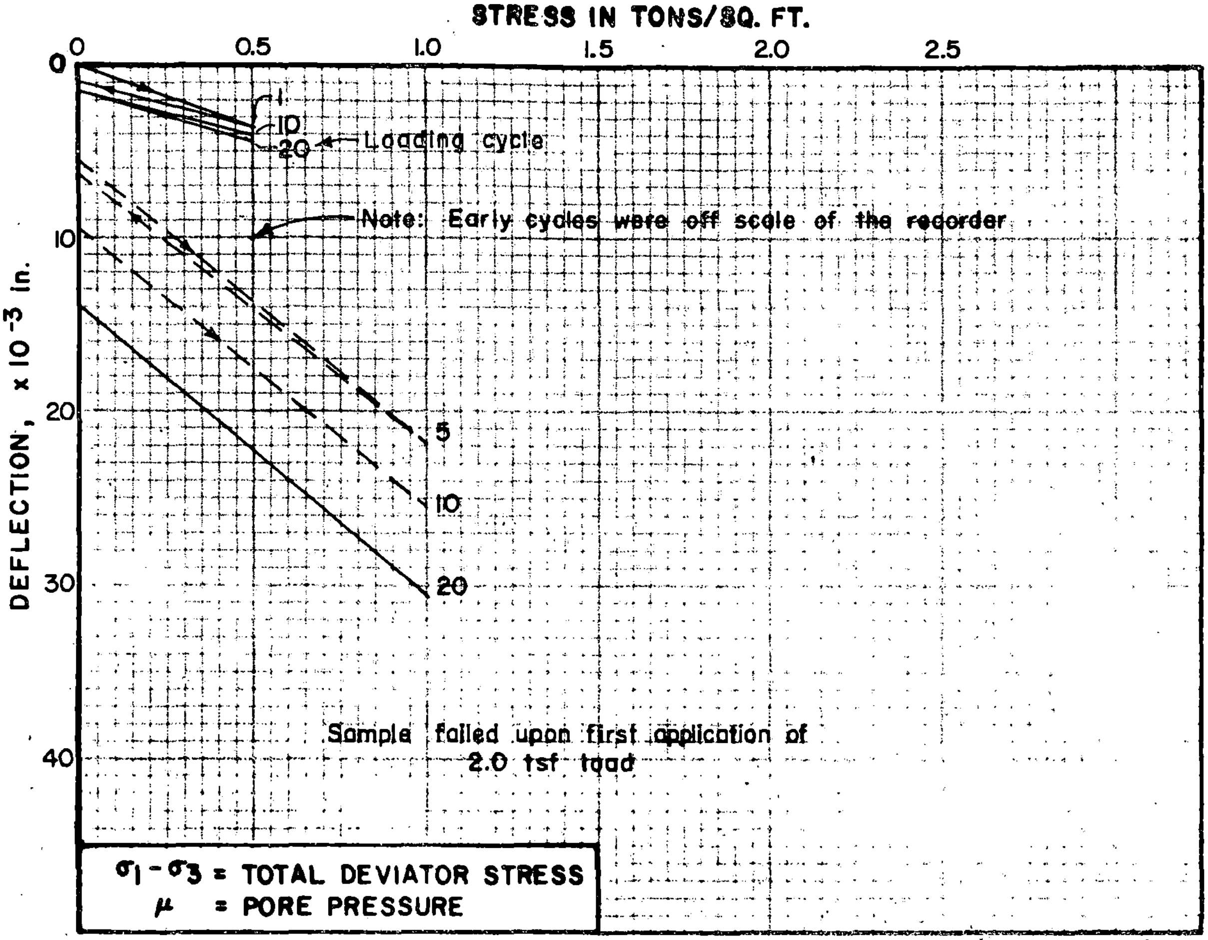
PULSATING LOAD TEST

B-25, S-2

C-268 DEC, 1970

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C11



TYPE OF TEST

☐ Q ■ R̄ ☐ S ■ PRESATURATED ■ BACKPRESSURE 30 psi

REMARKS: Cell pressure = 44 psi

σ_3 = 1.0 tons/sq. ft.

SPECIMEN

CLASSIFICATION Stiff, brown CLAY (CH).

FAILURE SKETCH

☐ COMPACTED ■ UNDISTURBED

COMPACTIVE EFFORT:

HEIGHT 2.05 inches

DIAMETER 4.98 inches

UNIT WEIGHT 82.2 pcf

WATER CONTENT:

BEFORE TEST ____ %

AFTER TEST 35.1 %

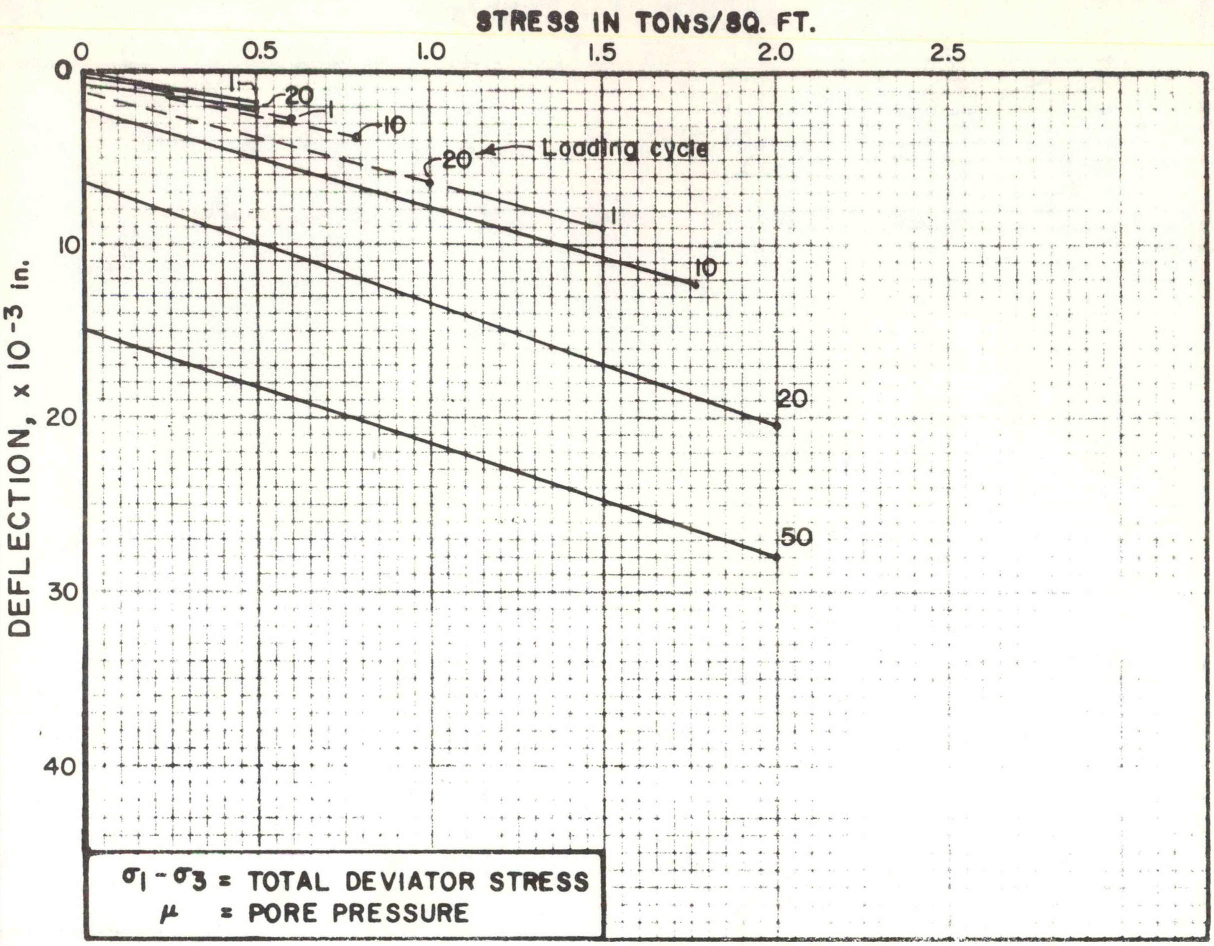
PULSATING LOAD TEST

B-29, S-3

C-268 DEC, 1970

SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C12



TYPE OF TEST

☐Q ■R̄ ☐S ☐PRESATURATED ■BACKPRESSURE 30 psi

REMARKS: Cell pressure = 44 psi

σ_3 = 1.0 tons/sq. ft.

FAILURE SKETCH

SPECIMEN

CLASSIFICATION Stiff, red-brown CLAY (CH)

■COMPACTED ☐UNDISTURBED

COMPACTIVE EFFORT:
6,600 ft-lb/ft³

HEIGHT 2.09 inches

DIAMETER 4.83 inches

UNIT WEIGHT 106.0 pcf

WATER CONTENT:

BEFORE TEST ______ %

AFTER TEST 21.2 %

PULSATING LOAD TEST

TP-2, S-3

C-268 DEC, 1970

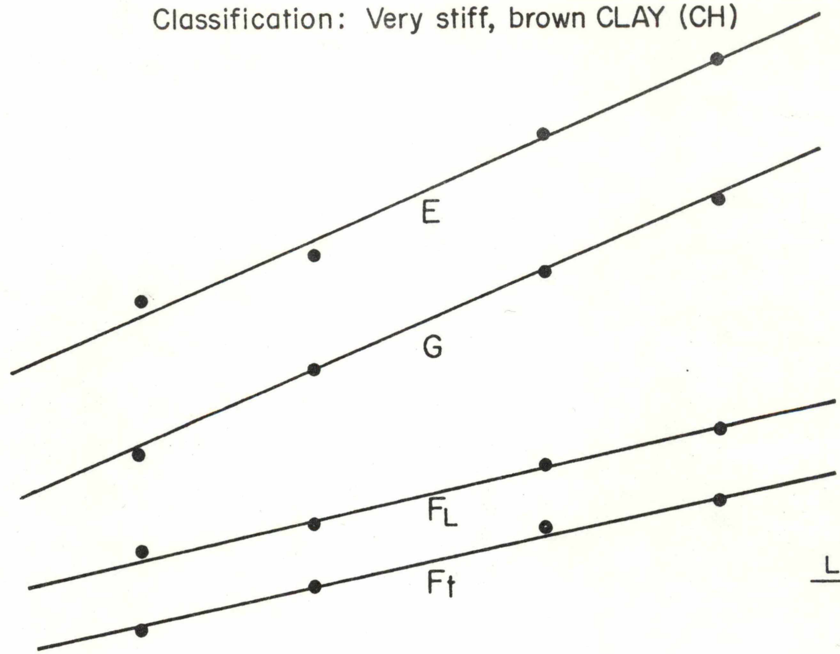
SHANNON & WILSON
SOIL MECHANICS & FOUNDATION ENGINEERS

FIG. C13

10 Confining Pressure, psi 100 1000

Sample data:

Dia.	2.05 in.
Length	4.88 in.
γ_d	100.6 pcf
w	22.2 %
Classification:	Very stiff, brown CLAY (CH)



LEGEND

E = Modulus of elasticity in compression.

G = Modulus of elasticity in shear.

F_L = Longitudinal resonant frequency.

F_t = Torsional resonant frequency.

Q_u test performed after vibration test.
$Q_u = 0.97\ kg/cm^2$

Modulus, psi: 100,000 / 10,000 / 1000

Frequency, cps: 1000 / 100

VIBRATION TEST

B-25, S-2

C-268 DEC, 1970

FIG. C14

10 Confining Pressure, psi 100 1000

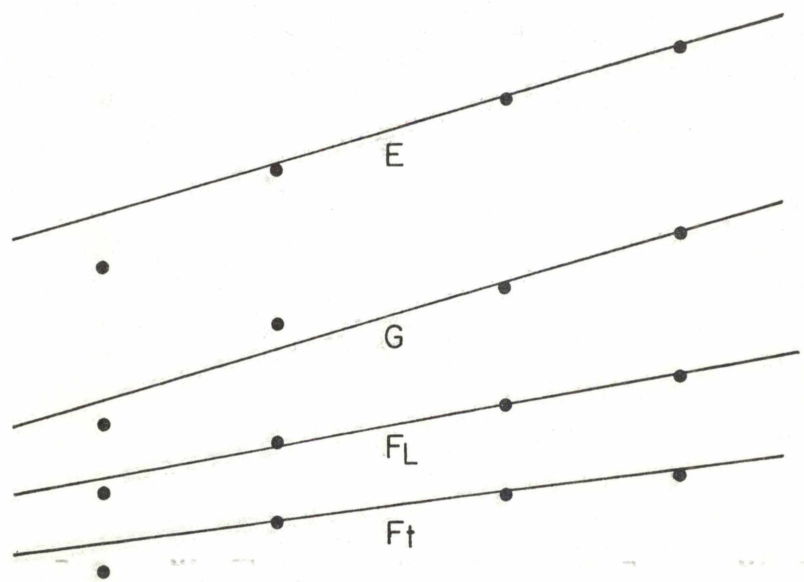
Sample data:

Dia.	2.05 in.
Length	4.96 in.
γ_d	88.1 pcf
w	32.4 %

Classification: Stiff, brown CLAY (CH)

Modulus, psi: 100,000; 10,000; 1000

Frequency, cps: 1000; 100



See Fig. C14 for Legend.

Q_u test performed after vibration test
$Q_u = 1.04$ kg/cm^2

VIBRATION TEST

B-29, S-3

C-268

DEC, 1970

FIG. C15

72R2133

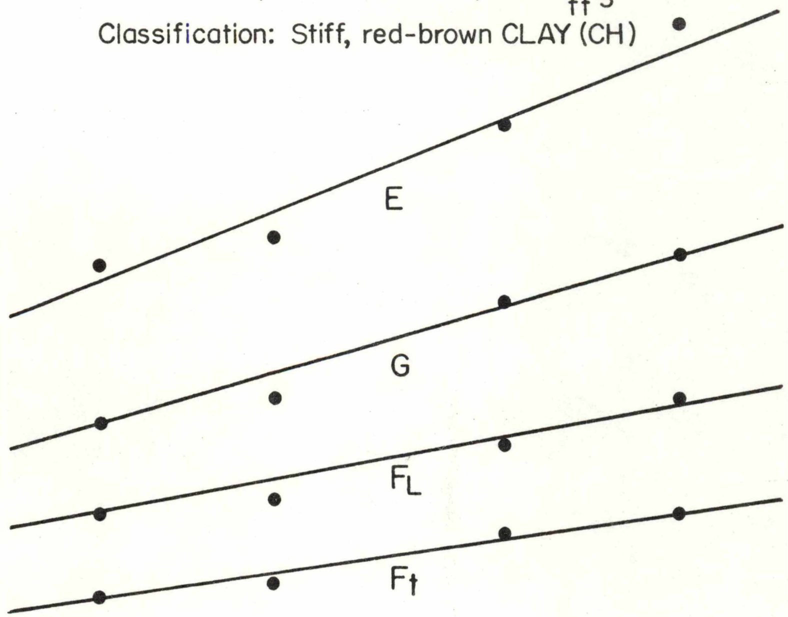
10 Confining Pressure, psi 100 1000

Sample data:

Dia.	2.07 in.
Length	4.84 in.
γ_d	106.1 pcf
w	19.6 %

Compaction effort 4,950 $\frac{\text{ft-lb}}{\text{ft}^3}$

Classification: Stiff, red-brown CLAY (CH)



100,000 Modulus, psi 10,000

1000 Frequency, cps 100 1000

See Fig. C14 for Legend.

Q_u test performed after vibration test

Q_u = 5.38 kg/cm^2

VIBRATION TEST

TP-2, S-3

C-268

DEC,1970

FIG. C16

APPENDIX D

CHEMICAL AND MINERALOGICAL ANALYSES

JAMES K. MITCHELL
CONSULTING GEOTECHNICAL ENGINEER
119 SELBORNE WAY
MORAGA, CALIFORNIA 94556
(415) 376-5618

October 2, 1970

Shannon & Wilson, Inc.
1550 Rollins Road
Suite "F"
Burlingame, California 94010

ATTN: James R. Salley

As requested by your office chemical and mineralogical analyses have been made of Sample 4 from Test Pit 1 from the D.O.T. Test Track, Kansas. This material as received was reddish brown, remolded plastic clay at a water content of 36.3%. Data furnished with the sample indicated a natural water content of 27%, a liquid limit of 71.7%, and a plasticity index of 55.2%. Although grain size data were not available for this sample, it was reported by your office that Sample 2 from the same test pit, which was slightly less plastic (LL = 68%, PI = 51%), contained 43% by weight finer than 2 microns.

The results of our analyses are presented below.

pH

The pH was determined using a mixture of 2 parts water to 1 part soil and found to be 7.7; i.e., slightly basic.

Organic Matter

Small organic particles can be observed in the soil and a positive reaction for organic matter was obtained using 15% hydrogen peroxide solution.

Carbonates

No reaction was observed upon treatment with 1N hydrochloric acid. This, as well as the absence of any X-ray reflections attributable to crystalline carbonate minerals, indicates the virtual absence of carbonates from this soil.

Pore Fluid Salt Content

The salt concentration in the pore water was determined as an equivalent NaCl content using a conductivity bridge. A mixture of 4 parts distilled water to one part dry soil was used. It was found that the equivalent NaCl content was 1.71 gm/liter (1710 ppm) (0.0296 N) for the reported natural water content of 27%. This corresponds to 0.046 gm. salt per 100 gm. dry soil. No chloride was detectable by silver nitrate titration.

Mineralogy

A series of X-ray diffraction patterns were obtained for determination of the crystalline mineral constituents of this soil. The results are presented in Table 1.

The only non-clay minerals present in this soil in quantities detectable by X-ray are quartz and feldspar. Quartz is the only non-clay mineral detectable in the clay-size (-2μ) fraction. The series of diffraction patterns obtained for the fraction finer than 2 microns indicates clearly the presence of montmorillonite, hydrous mica (illite), and kaolinite. Montmorillonite is confirmed by the expansion from 13.8Å basal spacing to 17.6Å as a result of glycol treatment, followed by collapse to 10Å after heat treatment. The presence of the 10Å reflection in the glycolated sample is attributable to hydrous mica, and the disappearance of the 7.2Å reflection after heating at 600°C indicates the removal of kaolinite.

Based on the grain size data obtained for Sample 2 (43% finer than 2μ) it is estimated that Sample 4 contains about 40 to 50% clay mineral. In this fraction montmorillonite is most abundant, hydrous mica next, and kaolinite least abundant.

Discussion

Nothing was detected in this sample to suggest that it will behave in an unusual manner as compared with other highly plastic clays of similar classification characteristics. Neither the salt concentration or pH are of a level to suggest particular difficulty in connection with the deterioration of instrumentation; although these factors alone cannot guarantee the absence of any effects.

TABLE 1

X-RAY DIFFRACTION DATA[1]

SAMPLE 4, TEST PIT 1, D.O.T. TEST TRACK, KANSAS

<200 MESH AIR-DRIED, POWDER		<2μ, AIR-DRIED ORIENTED[4]		<2μ, GLYCOLATED ORIENTED[5]		<2μ, 500°C HEAT TREATMENT[6]		<2μ, 600°C HEAT TREATMENT[6]		MINERAL RESPONSIBLE FOR REFLECTION[7]
d(Å)[2]	I[3]	d(Å)	I	D(Å)	I	d(Å)	I	d(Å)	I	
				17.6	10.0					M
		13.8	10.0							M
				10.2	3.6	10.0	10.0	9.71	10.0	HM, M
		7.14	1.9	7.19	1.4	7.20	0.7			K
		4.98	1.0	5.01	0.9	5.01	1.8	4.93	2.7	HM, M
4.46	0.5									K
4.25	2.2									Q
4.04	0.5									F
		3.56	1.8	3.57	1.7	3.58	0.7			K
3.35	10.0	3.33	3.3	3.34	4.0	3.34	5.3	3.30	8.0	Q, HM
3.24	0.5									?
3.20	0.5									HM
2.46	1.0									Q
2.29	0.7									K
2.24	0.4									M, HM
2.13	0.7									Q, M
1.98	0.4	1.99	0.6	2.00	0.5	1.99	0.6			M, HM, K
1.82	1.3									Q
1.68	0.6									Q
1.54	1.0									Q

SEE NOTES, NEXT PAGE

NOTES FOR TABLE 1

1. All diffraction patterns obtained using copper K_{α} radiation.

   Tube voltage = 40 kv

   Tube current = 20 ma

   Incident beam slit settings @ 1° beam, 4° scatter

   Detecter slit setting @ 1°

2. d(Å) = spacing of reflecting plane in Angstroms.

3. I = reflection intensity based on I = 10.0 for strongest line in pattern.

4. Sample sedimented onto glass slide.

5. Sample equilibrated with ethylene glycol vapor @ 60°C.

6. Oriented samples heat treated for at least 4 hours.

7. Mineral abbreviations:

   M = montmorillonite

   HM = hydrous mica (illite)

   K = kaolinite

   Q = quartz

   F = feldspar

It was reported that the sample tested was from the B-horizon of a profile overlying a limestone. The complete absence of carbonate as well as the significant content of both montmorillonite and kaolinite do not favor the accumulation of this soil as a residue of limestone weathering. It seems more likely that this soil originated from a glacial till or decomposition of a shale.

If there are any questions concerning these analyses or this report, or if I may be of further assistance on this project, please feel free to contact me.

Sincerely yours,

James K. Mitchell

James K. Mitchell

JKM/nh

APPENDIX E

INSTRUMENTATION - PERFORMANCE CRITERIA

APPENDIX E

INSTRUMENTATION - PERFORMANCE CRITERIA

VERTICAL EXTENSOMETERS

Vertical orientation.

To be installed in a vertical, drilled hole (approximate diameter 5 inches).

Multi-position (one, three and four positions).

Expanding hydraulic prong anchors of the "Geonor" type.
Bottom anchor to be grouted into rock.

Riser 0.25-inch stainless steel in 0.75-inch PVC tubing.

Hole to be backfilled. Material not determined. Possible material is poly-urethane plastic foam.

Transducer, LVDT mounted above the anchor, within the subgrade.

Terminal box to be accessible for maintenance when permitted by track structures.

Range (LVDT) ± 1.0 inch
Sensitivity: theoretically controlled by recorder.

Probably in the order of 0.003 inch for dynamic-using pen type recorder. (Anticipated measurable range 0.04 inch - 0.0008 inch).

Frequency: Dependent on recorder.

Pen type chart recorder 0 - 100 Hz and above.

For > 100 Hz an oscillograph type recorder is required.

(Anticipated meas. range 3 Hz to 100 Hz).

LVDT to be resetable.

Terminal - As small as possible, Durable, Shock proof.
Accessible for maintenance when permitted by track structures.

Water tight cover.

HORIZONTAL, PORTABLE EXTENSOMETERS

Horizontal orientation. To be inserted into horizontal tubing emplaced in the embankment.

Gage length - 2.5, 5 or 10 feet.

Gage length to be adjustable to accommodate permanent movements of the couplings.

Single-position.

Expandable anchor shoes that will engage and lock into tubing couplings which are spaced at 2.5 and 5.0 feet in the embankment.

Transducer, LVDT
Range ± 0.5 inch
Sensitivity - theoretically controlled by recorder.

In the order of 0.0002 inch for dynamic measurements, using a pen type recorder.

Frequency: Dependent on recorder.
0 - 100 Hz and above.

For > 100 Hz an oscillograph recorder is required.

HORIZONTAL TUBING

4" Ø (ID) corrugated polyethelene tubing.

Anchored couplings at 2.5 and 5.0 feet.

Vandal-proof exterior cover.

HORIZONTAL STATIC STRAIN DEVICE

Strain rods with hooking device to engage anchored couplings in the horizontal embankment tubing.

Precision 0.005 feet (0.06 inch).

STRESS CELLS

Oil filled cell with LVDT pressure transducer.

Dimensions - approximately 6-inch diameter
approximately 0.5-inch thick

Pressure range - 0 - 25, 0 - 50, 0 - 100 psi

Sensitivity - 0.1 psi

Frequency - 0 - 100 Hz

MOISTURE - TEMPERATURE SENSORS

SoilTest	MC-300 A	moisture meter
	MC-310	moisture - temperature cell

Range and sensitivity to be determined in the laboratory.

APPENDIX F

SPECIAL PROVISIONS
TO THE
ATSF STANDARD SPECIFICATIONS
FOR
EARTHWORK AND STRUCTURES

SPECIAL PROVISIONS

The work embraced herein shall be performed in accordance with ATSF Standard Specifications for Earthwork and Structures, CES 50153-58. All Sections of the Specifications which are cited herein are applicable except as they may be modified, amended or added to by these Special Provisions. Unless otherwise noted, references herein refer to the Standard Specifications.

SECTION SP-1 - GENERAL

Modifications, amplifications and additions to Section 1 - General Provisions of the Standard Specifications.

SP-1.1 General. The provisions of Standard Specifications, Section 1 shall apply except as modified and supplemented herein.

SP-1.2 Description of Entire Project. The proposed construction under this contract, as generally described in the Invitation for Bids, is a part of a proposed single line test track to be constructed by the Company in cooperation with the Office of High Speed Ground Transportation of the U. S. Department of Transportation. The entire project further includes the placement of embankment instrumentation, various test track structures including ballast, ties, slabs, beams and rail and test track structure instrumentation.

It is the purpose of this contract to provide a test embankment with properties as uniform as practically possible using standard construction practices and reasonable care upon which to place the experimental test track structures (ballast, ties, rails, etc.). The test embankment will be instrumented at designated locations during construction and following completion of the embankment. The test track structures will be instrumented.

The embankment and test track instrumentation will be monitored over an extended period of several years under main line traffic operations for the purpose of evaluating the performance of the various test track structures. It is essential for the success of the entire project to produce a test embankment of uniform properties which corresponds to a typical railroad embankment of high relatively high quality. Field observations and testing will be accomplished by the Engineer to insure uniformity and to document the in-place properties of the embankment.

SP-1.3 Time of Completion. The Contractor shall start construction within five (5) days of notice to proceed and complete the work within forty (40) working days unless time for completion is extended as provided elsewhere in the contract. It is expected that two eight (8)-hour shifts will be required for a portion of the allowed time in order to complete the project on schedule.

SP-1.4 Determination of Working Days. The Engineer will issue to the Contractor a Notice to Proceed stating the date upon which work may proceed. Five (5) calendar days (including the effective date of the Notice to Proceed) shall be allowed the Contractor in which to begin work before the Engineer starts counting working days.

Work, which will require inspection by the Engineer, will not be permitted on Sundays or legal holidays except that which may be necessary to preserve and protect the work. Legal holidays are defined as New Year's Day, Memorial Day, Independence Day, Labor Day, Thanksgiving Day, and Christmas Day. If the Company proclaims other days as holidays, work shall be at the Contractor's option. On these proclaimed holidays, a working day will be charged only if the Contractor elects to perform major work. In the event one of the above holidays falls on a Sunday, the following Monday shall be considered as a holiday. The above provision relative to Sunday and holiday work may be waived but only with the written approval of the Engineer.

The above provisions relative to Sunday, and Holiday work do not preclude the repairing of equipment or the performing of other minor work which requires no inspection by the Engineer.

Except as provided below, working days shall be determined only upon weather conditions or upon conditions on the project caused by the weather. Days during which prosecution of the work has been suspended, due to unsuitable weather conditions, storms, floods or acts of Providence, shall not be counted as working days. Days required after the event of unsuitable weather conditions, storms, floods, or acts of Providence to attain the approximate condition of work before such event shall not be counted as working days.

Days on which work is delayed or suspended, due to acts of the Company or the Engineer, shall be counted as "Companies Delay" and shall not be counted as working days. Saturdays shall be counted as a working day. Sundays and holidays shall not be counted as working days unless the Contractor is permitted to perform major work, in which cases, Saturdays, Sundays and holidays will be charged as working days as hereinafter prescribed.

Additional working days may be granted for additional work added to the Contract. Such additional working days shall be computed with due regard to the nature of the additional work.

Additional working days may be granted to compensate for working days lost due to causes entirely beyond the Contractor's control or for partial Contract time lost due to causes that hampered the normal prosecution of the work and that were entirely beyond the Contractor's control, provided that the following provisions are complied with:

The Contractor shall, as soon as practicable after such delays as mentioned in the preceding paragraph occur, and prior to the expiration of the Contract time period, make written request to the Engineer for an extension in the number of working days, stating the number of working days lost or the amount of partial Contract time lost due to causes beyond his control and submitting sufficient proof to establish his claim.

Working days will be computed for the Contract as a whole and not proportioned for separate divisions of the work. If weather conditions are such that the Contractor cannot work more than forty (40) percent of the normal working day, no time shall be charged. If he can work more than forty (40) percent but less than eighty (80) percent of the day, a half day shall be charged. If he can work eighty (80) percent or more of the day, a full day shall be charged. Working days shall not be computed in lesser fractions than one-half days.

Miscellaneous work, that in itself does not constitute a major item or accomplishment of the project, may be performed without time being charged against the Contract on those days designated as non-working days by the Engineer.

The Contractor or his authorized representative shall at the end of each week be notified in writing by the Engineer as to the number of working days charged in that week. Should the Contractor disagree as to the number of working days charged in that week, he or his authorized representative shall notify the Engineer in writing within five (5) days of such notice and the differences shall be settled at the earliest day possible.

SP-1.5 Program of Construction. The Contractor shall submit to the Engineer in writing, within five (5) days of Notice to Proceed, his program of construction, which shall show the dates at which the various operations and parts will be started and completed, in sufficient detail to enable the Engineer to judge the adequacy of such program. This program shall be modified from time to time as may be necessary in order to secure such results.

The Contractor shall schedule his operations around the installation of instrumentation by the Engineer.

SP-1.6 Contractor's Methods. The Contractor's procedure and methods of construction may be of his own selection except as specified herein provided they will, in the judgment of the Engineer, secure results which satisfy the requirements of the Plans and Contract Documents. Permission by the Engineer to use any particular device or method of construction shall not relieve the Contractor from full responsibility for any failure resulting therefrom. Approval of details of construction, or of lists or bills of material, shall not relieve the Contractor from full responsibility to secure results in conformity with the

Plans and Contract Documents. It is expressly understood that the Contractor is in all respects an independent contractor for this work, notwithstanding under certain conditions he is bound to follow the directions of the Engineer, and is in no respect an agent, servant, or employee of the Company.

SP-1.7 Cooperation of the Contractor. Instrumentation will be installed by the Engineer in the embankment as placement proceeds and after placement of the embankment has been completed. At times directed by the Engineer the Contractor will be required to stop construction of the embankment at the location of the main instrument arrays and to conduct operations elsewhere on the project. Close cooperation of the Contractor with the Engineer is required to schedule operations. The Contractor shall not proceed with construction of the embankment at a main instrument array until instruments have been installed, checked out and approval given for the Contractor to proceed.

SP-1.8 Damage to Instrumentation. Instrumentation damaged or destroyed by the Contractor's operations shall be repaired or replaced by the Contractor at no expense to the Company. The Contractor shall take whatever precautions necessary to protect the instrumentation from damages due to construction operations. If in the opinion of the Engineer, protective measures are inadequate, the Engineer shall direct the Contractor to take appropriate protective measures.

SP-1.9 Inspection. The Engineer will provide a full time staff to observe construction and to sample and test each lift of the embankment materials in place and in a field laboratory. The embankment will be tested for water content, unit weight, degree and uniformity of compaction, plate bearing resistance and other properties as necessary to thoroughly document the embankment material.

SP-1.10 Information to be Furnished by Contractor. Delete Section 1.6.4 of the Standard Specifications.

The contractor shall furnish a program of construction within five (5) days of Notice to Proceed as provided elsewhere in these Special Provisions.

The Contractor shall furnish with his proposal a list of equipment which he expects to place on this project and he shall quote rental rates per hour on each unit of equipment for use in paying for Force Account Work. Such rental rates shall be fully operated rates and include machine, operator, supplies, fuel, repairs, taxes, overhead, etc. Rental will be paid only for time machines are engaged in doing Force Account Work. The list of equipment shall be provided on the prescribed format included with the contract documents.

The Contractor shall promptly furnish the Engineer when requested, a force report and rates of pay.

The Contractor shall prepare whatever detailed working drawings are necessary to enable him to construct all parts of the work in conformity with the plans and specifications. For all general construction, working drawings shall illustrate details and methods of construction which are left to the Contractor's choice or which are not shown on the plan drawings. Working drawings shall show the project name and location, the Contractor's and Company's names, and the location of the parts of the work. They shall be signed or initialed to indicate they have been checked, and shall be submitted to the Engineer for review in triplicate. One set will be returned to the Contractor approved, or showing changes or corrections required, and copies in triplicate shall be resubmitted after correction until they are approved. If additional copies have been specified in the plans or in these specifications they shall be furnished.

The Contractor shall report any errors found in the plans to the Engineer, who will make or approve the necessary corrections. The Company shall not be responsible for errors on the Contractor's drawings, even though approved, or for minor discrepancies of the plan drawings. Payment for working drawings, revisions thereof, and for copies furnished shall be included in the amounts bid for materials or work. The Contractor shall furnish as many sets of prints of working drawings as the Company may need for the work.

SP1.11 <u>Office and Utilities</u>. The Contractor shall maintain an office, convenient to the work, where all communications from the Engineer may be received and acknowledged by said Contractor or his authorized agent. The office may be on Company property subject to the site being approved by the Engineer. The Contractor shall procure and maintain at his own expense all utilities required for his operations such as water, electricity, telephone, etc. The Contractor shall provide adequate facilities at the site and public nuisance on Company property will not be permitted.

The Contractor shall provide a temporary building or trailer containing 500 square feet suitable for office and storage for the Engineer. The contractor shall provide a minimum of four desks or equivalent and associated office furniture and procure and maintain at his own expense all utilities such as water, 110v electricity, telephone, heat, etc. The facility shall be subject to approval by the Engineer and shall be operational within ten (10) days of Notice to Proceed.

SP-1.12 <u>Subsurface Conditions</u>. The plans show a portion of certain geologic information obtained from subsurface investigations in the vicinity of the project alignment and in possible borrow areas. The complete logs of borings and test pits are not included with these contract documents but are available for review by prospective bidders, upon request, from Chief Engineer, Eastern Lines, ATSF, Topeka, Kansas. Rock cores obtained from certain borings are available at the ATSF Depot, Cassoday, Kansas, for inspection by prospective bidders upon appointment.

The information shown on the plans and on the logs of borings and test pits are not warranted to show actual sub-surface conditions. The Contractor agrees that he will make no claim against the Company, if in carrying out construction he finds that actual conditions do not conform with those indicated by said investigations.

SP-1.13 <u>Alterations of Plans or Change in Amounts of Work.</u> The Engineer shall have the right, at any time during the progress of the work, to make such increases or decreases in quantities and such alterations in the details of construction, including alterations in the grade or alignment of the embankment or structure or both, as may be found necessary or desirable. Such increases or decreases and alterations shall not invalidate the Contract nor impair the bond or release the sureties thereof, and the Contractor agrees to accept the work altered, the same as if it had been a part of the original work.

Unless increases and decreases in quantities and alterations in Plans materially change the character of the work to be performed or the cost thereof, the altered work shall be performed as a part of the Contract and will be paid for at the same contract prices as for other parts of the work. The term "materially change", above, for purposes of intent under the Contract shall be construed to apply only to the following circumstances:

(1) When the character of the work as altered differs in kind or nature from that involved or encountered in the original proposed construction.

(2) When the total amount increase or decrease in quantities of a major Contract bid item affected by the work as altered varies from those same individual items in the Proposal by more than twenty (20) percent.

If the character of the work or the unit costs thereof are "materially changed" as above defined, an allowance will be made on such basis as may have been agreed to in advance of the performance of the work involved, or in case no such basis has been previously agreed upon, then an equitable adjustment in the contract price will be made.

No claim shall be made by the Contractor for any loss of anticipated profits because of any such alteration, or by reason of any variation between approximate quantities and the quantities of work as done.

When the accepted quantities of work vary from the quantities in the Proposal the Contractor shall accept as payment in full, so far as Contract items are concerned, payment at the original Contract unit prices for the accepted quantities of work done. If the altered or added work is of sufficient magnitude as to require additional time in which to complete the project, the Chief Engineer may extend the time of completion by the amount of additional time which, in his judgment, is required.

SP-1.14 Cross Sections. Field cross sections have been taken and plotted. Prints are available for Contractors use in bidding and for construction purposes, upon request, from the Chief Engineer, Eastern Lines, ATSF, Topeka, Kansas.

SP-1.15 Blasting. Heavy blasting will not be permitted. Light charges shall be used and each shot shall be covered with cable mats prior to detonation.

SP-1.16 Fair Employment Practice. The Contractor shall not discriminate against any applicant for employment under this Contract or against any employee performing work under this Contract because of the race, color, religion, ancestry, or national origin of such person. The Contractor shall notify all labor unions which represent employees working under this Contract of its policy of non-discrimination in employment and shall publicize his policy of non-discrimination to all his supervisors, foremen and employment agencies which refer appplicant for employment under this Contract to the Contractor. The Contractor shall include the provisions of this Paragraph SP-1.16 with the substitution of the word "Subcontractor" for the "Contractor" in all his subcontracts hereunder.

The Contractor and Subcontractors shall execute the Fair Employment Practices Certification included with the contract documents and furnish any other evidence of compliance with the Fair Employment Practices requirements as may be required by the Engineer.

SP-1.17 Traffic Delays Certain operations by the Contractor may be delayed by railroad traffic. The Contractor shall take these delays into account in preparation of bid prices and shall agree that he will make no claim for extra compensation due to such delays.

SP-1.18 Clearing of Track. The Contractor shall provide sufficient personnel and equipment to promptly clear the tracks of debris from blasting, hauling across the tracks, or any other cause, and shall keep the tracks clear.

SP-1.19 The contract prices shall include full compensation for all costs incurred under this section.

SPECIAL PROVISIONS

SECTION SP-2 - EARTHWORK

Modifications, amplifications and additions to Section 2 - Earthwork of the Standard Specifications.

SP-2.1 The provisions of the Standard Specifications Section 2 shall apply except as modified and supplemented herein.

SP-2.2 General. Delete Article 2.1.1 of the Standard Specifications. The work covered by these specifications shall be as designated by the plans, special provisions and invitation to bid, and shall consist generally of the necessary clearing and grubbing; excavation; removal of existing culverts, placing of embankment including backfill around existing bridges; construction of surface ditches, channels, dikes, berms, signal offsets, motor car set-offs, and grading road crossings including approaches, etc., as may be designated by the Engineers.

SP-2.3 Clearing and Grubbing.

SP-2.3.1 Disposal. Modify Article 2.2.4 of the Standard Specifications. Stripping shall not be deposited in berms outside of fill slopes of the test embankment.

SP-2.4 Grading.

SP-2.4.1 Definition. The term "Grading" in these specifications includes all excavations and embankments required for the formation of the roadway, surface ditches, channel and road changes, dikes, berms, signal offsets, road crossings and approaches including removal of existing culverts, backfilling around existing bridges and all similar work pertaining to the construction of the test embankment.

SP-2.4.2 Roadbed. Modify Article 2.3.2 of the Standard Specifications. The "Roadbed or subgrade" shall be a plane surface 10 feet wide.

SP-2.4.3 Roadway. Modify Article 2.3.5 of the Standard Specifications. The "Roadway" dimensions shall be as shown on the plans.

SP-2.4.4 Slopes. Modify Article 2.3.7 of the Standard Specifications. The side slopes of excavations into the existing main line embankment shall be 1.5 to 1 and as shown on the plans.

SP-2.4.5 Standard Section. Delete Article 2.3.8 of the Standard Specifications. The embankment, excavations and ditch sections shall be as shown on the plans.

SP-2.4.6 Berms. Delete Article 2.3.10 of the Standard Specifications.

SP-2.4.7 Disposal of Material. Delete Article 2.3.11 of the Standard Specifications. It is the intent of these specifications that all suitable materials from roadway, cut ditch, channel and other excavation be used so far as practicable in forming embankments, dikes or similar facilities.

Excavation in excess of that needed to make the embankment according to the plans and specifications or unsuitable material shall be used to widen embankments outside the limits of the test section, flatten slopes outside the limits of the test section, or to be deposited in spoil areas, at locations and in height and form designated by the Engineer. Payment for such excess material placed outside of the limits of the staked embankment section shall be in excavation only.

SP-2.4.8 Borrow. Modify Article 2.3.12 of the Standard Specifications. Borrowing beyond the limits of the designated and staked excavation section and other borrow excavation shall be measured as excavation of the proper classification.

SP-2.4.9 Waste. Delete Article 2.3.13 of the Standard Specifications. At the discretion of the Engineer and with his written consent, the Contractor may waste unsuitable excavated materials in a manner and location approved by the Engineer. Unsuitable materials may be wasted east and west of the test embankment adjacent to the main line embankment and on the Railroad right-of-way between the track and Highway K-177.

Waste disposal areas adjacent to the existing embankment shall be stripped of vegetation before material is placed. Waste material shall be placed in 12 to 18-inch thick lifts and compacted systemically by routing hauling equipment over the embankment.

If the Contractor should desire to waste or borrow off the right-of-way, he may do so only with the written approval of the

Engineer. Before entering on the ground, he must obtain easement from the land owner and release from the tenant in form acceptable to the Engineer. Such easement and release shall be at the Contractor's expense.

Waste material removed from approved areas will be measured as excavation of the proper classification.

SP-2.4.10 Common Excavation. Add to Article 2.3.17 of the Standard Specifications. Common excavation shall include both required excavation and borrow.

SP-2.4.11 Excavation Below Subgrade. Delete Article 2.3.20 of the Standard Specifcations. Rock shall not be used for embankment except as herein provided.

SP-2.4.12 Surface Ditches. When ordered by the Engineer, surface ditches shall be made. The cross section and location of such ditches shall be as directed and staked by the Engineer.

SP-2.4.13 Select Material. Delete Article 2.3.25 of the Standard Specifications.

SP-2.4.13.1 Select Material A. Select Material A shall consist of CLAY obtained from required excavations and authorized borrow. Select Material A shall be free of organic material and shall not contain over two (2) percent rock fragments by dry weight, and shall not have rock fragments over two (2) inches in dimension. Select Material A shall not include silts.

Whenever possible Select Material A shall be hauled directly from excavation and placed in compacted embankment.

SP-2.4.13.2 Select Material B. Select Material B shall consist of silt, clay or mixtures of silt, clay and rock fragments obtained from required excavation or authorized borrow. Select Material B shall be free of organic material and shall not contain more than forty (40) percent rock fragments by dry weight, and shall not have rock fragments over four (4) inches in dimension.

Whenever possible Select Material B shall be hauled directly from excavation and placed in compacted embankment.

SP-2.4.14 Roadway Embankments. Delete Article 2.3.28 of the Standard Specifications.

SP-2.4.15 Compaction of Embankment. Delete Articles 2.3.29, 2.3.30 and 2.3.31 of the Standard Specifications. Embankments shall be constructed in layers by spreading Select Material A or Select Material B in uniform lifts, but in no case shall any layer exceed six inches after compaction. Each layer shall be moisture conditioned by adding water (sprinkling) or by aerating to produce a water content between optimum and optimum plus three (3) percent. Optimum water content shall be determined by ASTM Designation D 1557 test procedures.

Sprinklers used for the application of water shall consist of tank trucks, pressure-distributors, or other construction equipment designed to apply water uniformly and in controlled quantities to variable widths of surface. Tank trucks shall be equipped with positive shutoff valves so that no leakage will result from the nozzles when the construction equipment is not operating. Leaks shall be repaired immediately and any material rendered too wet as determined by the Engineer shall be removed or reconditioned.

Each layer or lift shall be compacted in an orderly, systematic and continuous manner to achieve uniform density with eight passes of a tamping roller. The use of the test embankment as a haul road will be restricted to minimize over-compaction of the embankment.

Tamping roller shall consist of a heavy duty double drum unit with a drum diameter of not less than 60 inches and an individual drum length of not more than 72 inches. The drums shall be water or sand and water ballasted. Each drum shall have staggered feet uniformly spaced over the cylindrical surface such as to provide approximately three tamping feet for each two square feet of drum surface. The tamper feet shall be 7 to 10 inches in clear projection from the cylindrical surface of the roller and shall have a face area of not less than 5 nor more than 7 square inches. The roller shall be equipped with cleaning fingers, so designed and attached as to prevent the accumulation of the material between the tamping feet, and these cleaning fingers shall be maintained at their full length throughout the periods of use of the roller. The weight of the roller shall not be less than 4,000 pounds per linear foot of drum length weighted. The design and operation of the tamping roller shall be subject to approval. The Engineer shall have the right at any time during prosecution of the work to direct such repairs to the tamping feet, minor alterations in the roller, and variations in the weight as may be found necessary to secure optimum compaction of the earthfill materials. The roller shall be towed by a crawler-type tractor of sufficient power to operate the roller at a speed of approximately 3-1/2-miles per hour under all conditions.

The embankment shall be protected at all times against excessive accumulation of moisture due to precipitation. The embankment surface shall be maintained in a relatively smooth, level condition that will readily drain and a rubber tired roller shall be provided for sealing the surface of the embankment to promote efficient drainage.

The removal of excessively wet embankment material may be restricted by the presence of buried instrumentation at the main instrument arrays.

No frozen material shall be placed in embankments.

SP-2.4.16 Test Embankment. It is the intent of the specifications to achieve a uniform embankment with a dry unit weight of approximately 95 percent compaction based on a maximum density determined by ASTM 1557-66T test procedures. At the start of construction the Contractor will be directed to place a section of embankment approximately three feet in thickness and 200 feet long within the project embankment to verify the adequacy of specified compaction procedures. Based on these test results the Engineer may direct that additional or fewer passes of the tamping roller (adjusted rolling) be made or that the limits of moisture conditioning be varied to more readily achieve a uniform and adequately compacted embankment.

SP-2.4.17 Instrumentation. Certain instruments and material will be placed in the embankment at the locations of the main instrument arrays at various stages (levels) during construction of the embankment. At the location of the main instrument arrays the embankment will be over-built six inches and then fine graded to the elevation at which the instruments will be placed as directed by the Engineer. Grading operations shall not be conducted in the vicinity of the main instrument array during placement of instrumentation.

It will be the responsibility of the Contractor to protect the instrumentation against damage due to construction operations.

SP-2.4.18 Compaction of Rock Fills. Delete Article 2.3.32 of the Standard Specifications. Rock Excavation will be wasted.

SP-2.4.19 Embankments on Swampy Ground. Delete Article 2.3.33 of the Standard Specifications. Where the embankment foundation is soft and incapable of supporting hauling and compaction equipment, the excavation shall be deepened to remove unstable materials as directed by the Engineer.

SP-2.4.20 Compacting Excavation Base. Modify Article 2.3.35 of the Standard Specifications. The Contractor shall scarify and compact the full width of embankment.

SP-2.4.21 Soils Tests. The Engineer will take soils samples and perform gradation and moisture content tests to ascertain that the Contractor is performing the work in compliance with these specifications. The Engineer will in addition, conduct plate bearing tests, density and other tests on the fill and related laboratory testing. Tests will be made as frequently as the Engineer will consider necessary. The Contractor shall remove surface material and render such assistance as necessary to enable sampling and testing. The Contractor shall take such precautions as necessary to protect personnel from injury due to the Contractor's operations while they are performing testing operations on the embankment.

The Engineer may direct that inspection trenches or test pits be cut in the embankment to determine that the Contractor has complied with these specifications. Such trenches or pits will be of limited depth and size. Trenches shall be backfilled with the material meeting the requirements of that size. Backfill shall be compacted to a density at least equal to the contiguous embankment.

SP-2.5 Measurement and Payment.

SP-2.5.1 Measurement of Excavation. Delete Article 2.5.3 of the Standard Specifications. Excavation will be measured by the cubic yard of proper classification within the designated and staked excavation sections and within designated and approved borrow areas. Materials excavated outside of designated sections or approved borrow areas will not be measured. Excavations for removal of culverts will not be measured.

SP-2.5.2 Measurement of Adjusted Rolling. Adjusted rolling will be measured by the actual number of hours of additional rolling. No deduction will be made for specified reduced rolling.

SP-2.5.3 Payment for Adjusted Rolling. Adjusted rolling shall be paid for at the contract unit price per hour. This price shall be full compensation for furnishing all labor, materials, tools, equipment, supplies, supervision and incidentals necessary to perform adjusted rolling.

SPECIAL PROVISIONS

SECTION SP-3 - CORRUGATED METAL PIPES

SP-3.1 The provisions of the Standard Specifications Section 9, CORRUGATED STEEL AND STRUCTURAL PLATE PIPES shall apply except as modified and supplemented herein.

SP-3.2 Structural Plate Pipe. Delete Articles 9.3.4, 9.5 and 9.6 of the Standard Specifications.

SP-3.3 Measurement and Payment. Delete Article 9.7.

SP-3.3.1 Measurement. Corrugated Metal Pipes of the various types and sizes will be measured, along the pipe centerline, by the lineal foot in place.

SP-3.3.2 Payment. Corrugated Metal Pipes shall be paid for at the contract unit price per lineal foot of the various types and sizes in place. This price shall include full compensation for excavation, furnishing, unloading, storing and transporting the pipe; fine grading the foundation; placing the pipe; and backfilling.

SPECIAL PROVISIONS

SECTION SP-4 - MISCELLANEOUS

SP-4.1 Riprap.

SP-4.1.1 Extent. This work shall consist of selecting, dumping and/or hand placing riprap at designated culvert entrances, channel and embankment locations.

SP-4.1.2 Hand Placed Riprap. Suitable limestone rock shall be selected from required rock excavation, stockpiled if necessary, and hand placed in horizontal layers around designated culvert entrances, channel and embankment locations. The rock shall be durable and hard and shall have a 12-inch minimum dimension in the horizontal plane.

SP-4.1.3 Dumped Riprap. Suitable limestone rock shall be selected from required rock excavation, stockpiled if necessary, and dumped and track rolled at designated channel and embankment locations.

SP-4.1.4 Measurement. Measurement of hand placed and dumped riprap shall be by the number of cubic yards of riprap placed in accordance with the plans and as directed by the Engineer.

SP-4.1.5 Payment. Hand placed and dumped riprap shall be paid at the contract unit price per cubic yard in place. This price shall be full compensation for furnishing all labor, material, tools, equipment, supplies, supervision and incidentals necessary for selecting suitable rock from required excavation, stockpiling if necessary, excavating, placing rock, and backfilling.

SP-4.2 Road Surfacing.

SP-4.2.1 Extent. This work shall consist of furnishing and placing gravel surfacing on the access road, public roads, and private roads at designated locations along the test embankment.

SP-4.2.2 Subgrade. The soil subgrade shall be compacted in accordance with Articles 2.3.34 and 2.3.35 of the Standard Specifications. Rock subgrade shall be smoothed by grading and/or choking irregularities with rock spalls and track rolling with a dozer. Subgrade preparation shall be accomplished incidental to required excavation and placement of embankment and no separate payment will be made therefor.

SP-4.2.3 Gravel.

SP-4.2.3.1 General. Gravel shall be crushed limestone rock or crushed gravel meeting the following requirements.

SP-4.2.3.2 Soundness. The loss ratio of aggregate from each individual source for surfacing or resurfacing shall be not less than seventy-hundredths (0.70) after 25 cycles of the freezing and thawing test.

SP-4.2.3.3 Wear. The wear loss of aggregate from each individual source shall not exceed forty-five (45) percent when tested by the Los Angeles Abrasion Test Method.

SP-4.2.3.4 Deleterious Substances. The deleterious substances in this aggregate shall not exceed the following percentages by weight:

Soft friable material	*5.0
Material passing No. 200 sieve (wash)	*3.0
Sticks (wet)	2.0
Mud balls (wet, No. 4 sieve)	2.0

*Note - When such material is available which conforms to all of the requirements for "Road Surfacing" except those marked *, this material may be used with the consent of the Engineer if the Contractor furnished one and one-half (1-1/2) percent additional material for each one (1) percent over the allowable amount of soft friable material and material passing No. 200 sieve specified, but in no case shall the total amount of soft friable material and material passing No. 200 sieve be greater than ten (10) percent.

No one of the percentages listed above shall be exceed when taken separately. In addition any combination of the above deleterious substances shall not exceed seven (7) percent.

SP-4.2.3.5 <u>Size Requirements</u>. Grading after removal of deleterious substances shall conform to the following:

Size	Percent Retained
1 inch	0-45
3/8 "	45-100
#30 sieve	95-100

SP-4.2.4 <u>Placement</u>. Gravel shall be spread over designated roadways to the specified thickness, watered and compacted.

SP-4.2.5 <u>Measurement</u>. Road surfacing (gravel) shall be measured by the ton in the vehicle at the time and place of unloading or at other points of designation. Deductions will be made for all moisture in the material. Moisture content will be determined by a method approved by the Engineer.

SP-4.2.6 <u>Payment</u>. The amount of completed and accepted work, measured as provided above, shall be paid for at the Contract unit price bid per ton for "Road Surfacing" which price shall be full compensation for furnishing all materials, for all labor, equipment, tools, and incidentals necessary to complete the work.

SP-4.3 <u>Guard Fence</u>.

SP-4.3.1 <u>Extent</u>. This work shall consist of furnishing materials and installing a guard fence at designated locations along the project access road.

SP-4.3.2 <u>Posts</u>. Posts shall consist of ASA Schedule 40, three (3) inch diameter galvanized steel pipe four and one-half feet in length. The posts shall be capped and pipe shall be new or in good used condition which is acceptable to the Engineer.

SP-4.3.3 <u>Cable</u>. Cable between posts shall be a continuous, one-half inch diameter, three strand galvanized steel cable. The cable shall be new or in good used condition which is acceptable to the Engineer.

SP-4.3.4 <u>Placement</u>. Posts shall be set in two (2) foot deep drilled holes and grouted into place with cement-sand grout. Posts shall be set vertically at the required spacing, in a continuous line. Tilted or offset posts shall be removed and replaced at the Contractor's expense.

SP-4.3.5 <u>Measurement</u>. The guard fence shall be measured by the number of lineal feet of fence installed.

SP-4.3.6 Payment. The amount of completed and accepted work as measured above, shall be paid for at the Contract unit price per lineal foot for "Guard Fence", which price shall be full compensation for furnishing all materials, labor, equipment, tools and incidentals necessary to complete the work.

SP-4.4 Barricades.

SP-4.4.1 Extent. This work shall consist of furnishing materials and installing barricades at designated locations on the embankment and access road.

SP-4.4.2 Posts. Posts shall consist of ASA Schedule 40 four (4) inch diameter galvanized pipes seven (7) feet in length. The posts shall be capped and the pipe shall be new or in good used condition which is acceptable to the Engineer.

SP-4.4.3 Chain. Chain shall consist of heavy duty galvanized steel chain. One end shall be permanently attached to one post and other end shall be free.

SP-4.4.4 Lock. A suitable bracket shall be provided on one post to attach the free end of the chain along with a heavy duty, combination padlock. The bracket shall provide weather protection for the lock.

SP-4.4.5 Installation. Posts shall be installed in drilled holes, three (3) feet deep and backfilled with cement-sand grout. Posts shall be set vertical at designated locations.

SP-4.4.6 Measurement. Barricades shall be measured by the installed unit which shall include two posts, chain, brackets and lock.

SP-4.4.7 Payment. The amount of completed and accepted work, measured as provided above, shall be paid for at the Contract unit price per barricade unit which price shall be full compensation for furnishing all materials, for all labor, equipment, tools, and incidentals necessary to complete the work.

SP-4.5 Highway Easement Seeding.

SP-4.5.1 Extent. This work shall include furnishing, drilling seed, fertilizing and mulching in designated areas of Highway Right-of-way where vegetation was removed or destroyed as a result of authorized construction operations.

SP-4.5.2 Seed. Seed shall consist of certified native grass as follows:

Alfalfa	6 pounds/acre (to be drilled separately)
Bromegrass	12 pounds/acre
K-31 Fescue	10 pounds/acre
Italian Rye grass	10 pounds/acre
Mixed native grasses	15 pounds/acre

SP-4.5.3 Planting. Seed shall be drilled in with a commercial drill, fertilized and mulched. Fertilizer shall be 12-24-12 at a rate of 400 pounds per acre. Mulching shall be at a rate of two tons per acre.

SP-4.5.4 Measurement. Highway Easement Seeding shall be measured by the pound of the various kinds of seed furnished and planted.

SP-4.5.5 Payment. The amount of seed furnished and planted shall be paid for at the Contract unit price per pound for the various kinds of seed, which price shall be full compensation for preparation of the ground, for furnishing and planting all seeds including fertilizing and mulching and for all labor, tools, equipment and incidentals necessary to complete the work.

U. S. DEPARTMENT OF TRANSPORTATION

EXPERIMENTAL TEST TRACK EMBANKMENT

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

INDEX OF DRAWINGS

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS
DATED Sept. 12, '70 APPROVED Chief Engineer

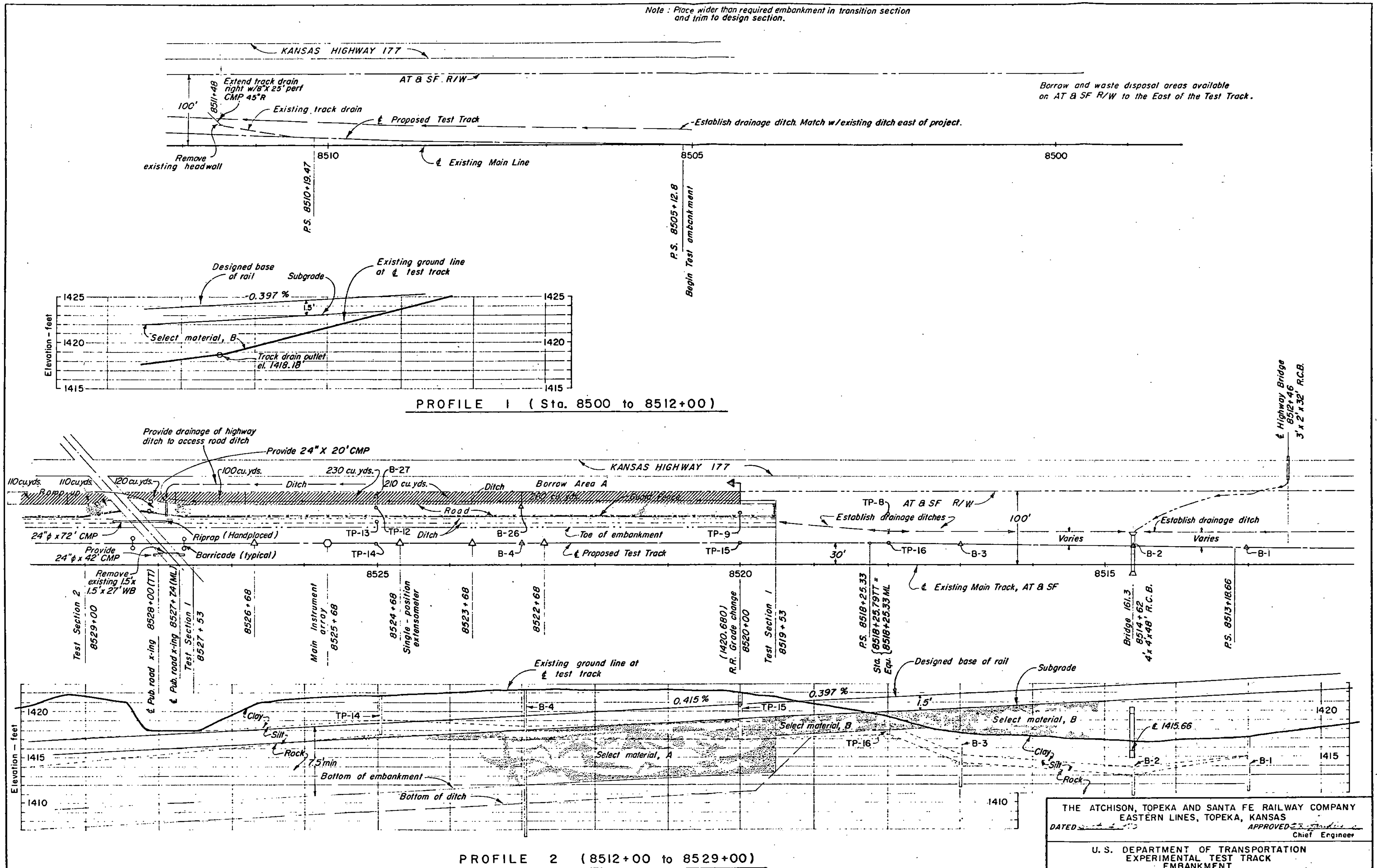
U.S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

PLAN AND PROFILE OF TEST TRACK EMBANKMENT

SHANNON & WILSON INC.

Rev.

2/12



NOTES:

1. Subsurface conditions between indicated borings or test pits may vary from that shown.
2. All stationing relative to main line (ML) stationing.
3. Track drains between Sta. 8488+20 and 8511+48 and between Sta. 8517+50 and 8545+67 are not to be disturbed.
4. Indicated borrow quantities represent estimates of usable borrow.

0 50 100 150 200
Scale - feet
Horizontal

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

DATED ______ APPROVED ______
Chief Engineer

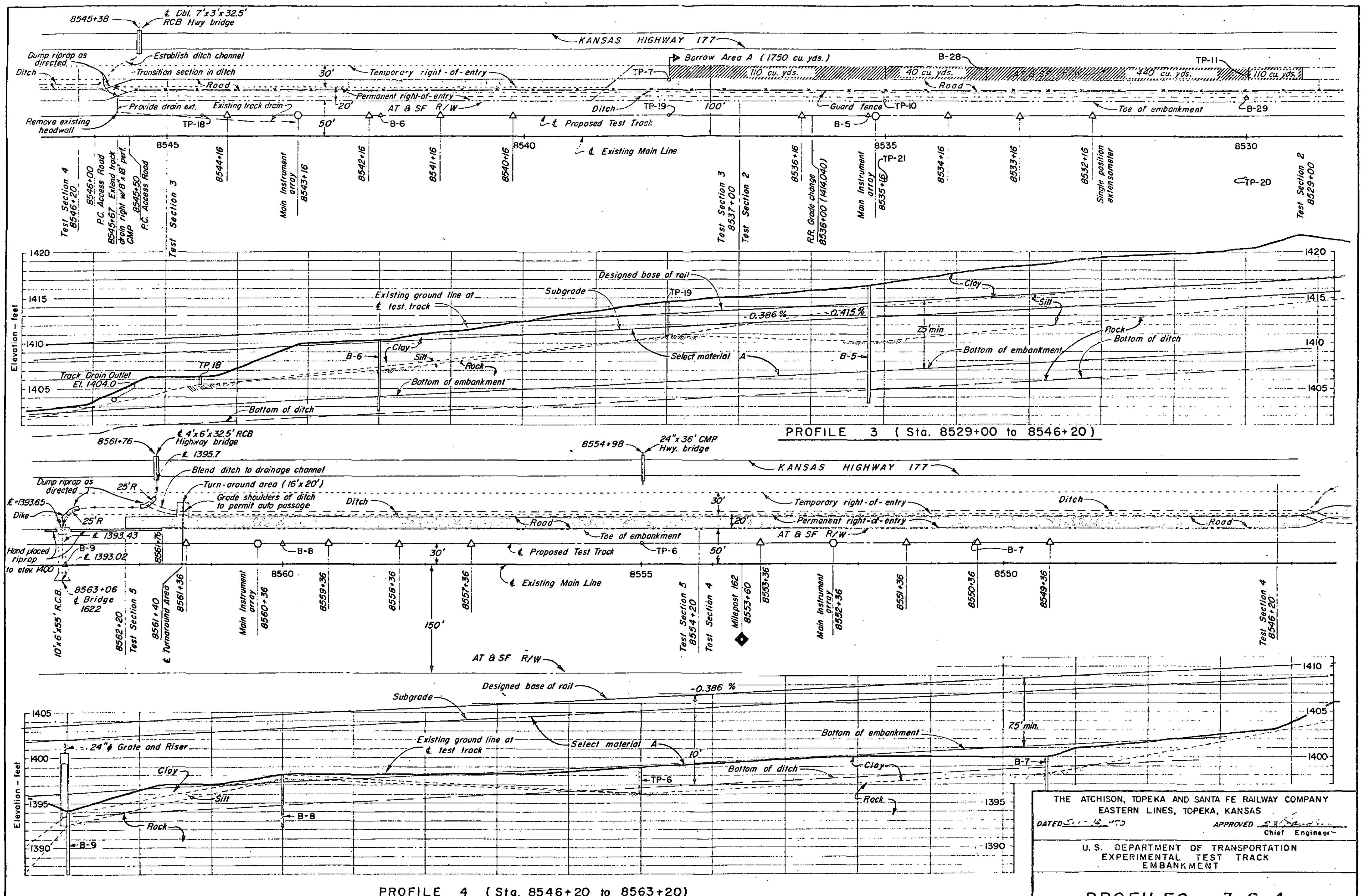
U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

PROFILES 1 & 2

Shannon and Wilson
Soil mechanics and foundation engineers

Rev. 3

12



NOTE : 1. Refer to drawing No. 3 for general notes.
2. Track drains between Sta. 8517+50 and 8545+67 are not to be disturbed.

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

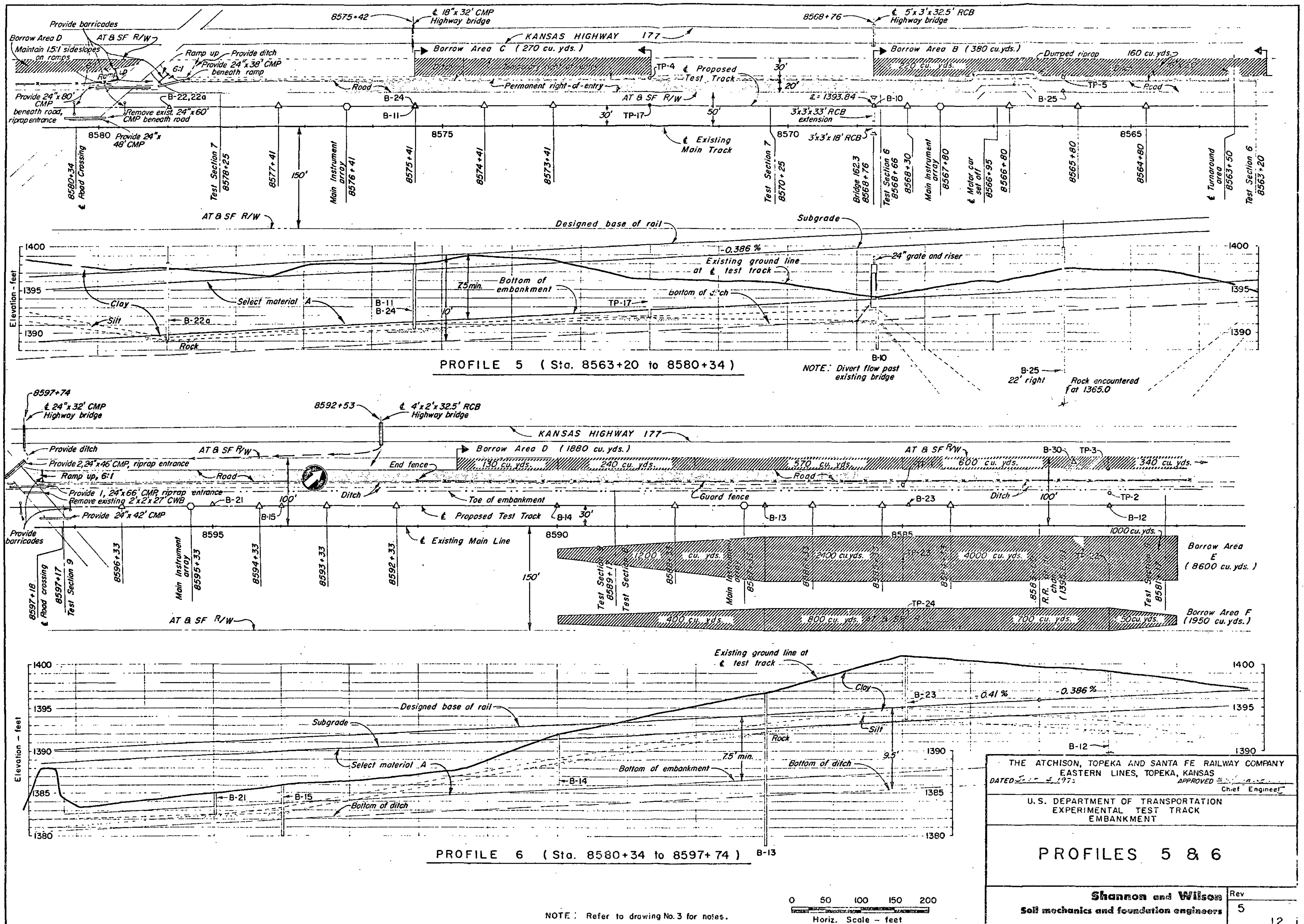
DATED ____ APPROVED ____ Chief Engineer

U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

PROFILES 3 & 4

Shannon and Wilson
Soil mechanics and foundation engineers

Rev.	
4	12



THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

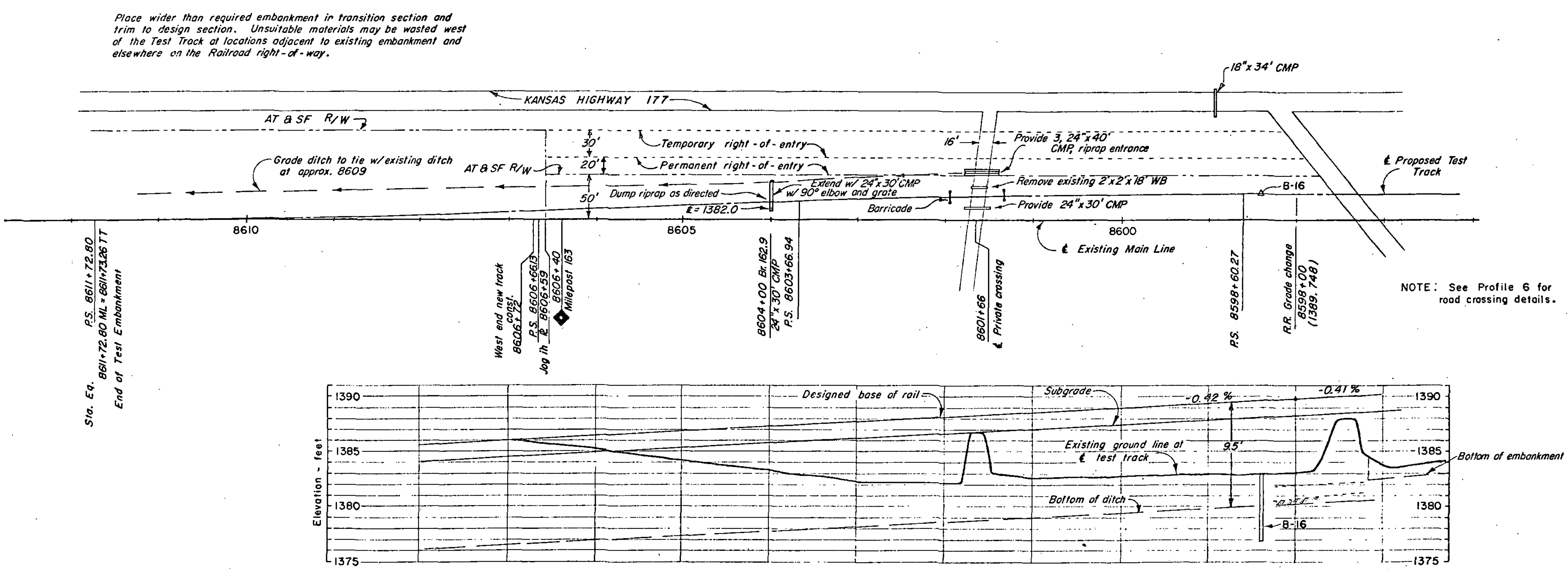
DATED ... APPROVED ... Chief Engineer

U.S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

PROFILES 5 & 6

Shannon and Wilson
Soil mechanics and foundation engineers

Rev
5



PROFILE 7 (Sta. 8597+74 to 8611+72.80)

0 50 100 150 200
Horiz. Scale - feet

NOTE: Refer to drawing No. 3 for notes.

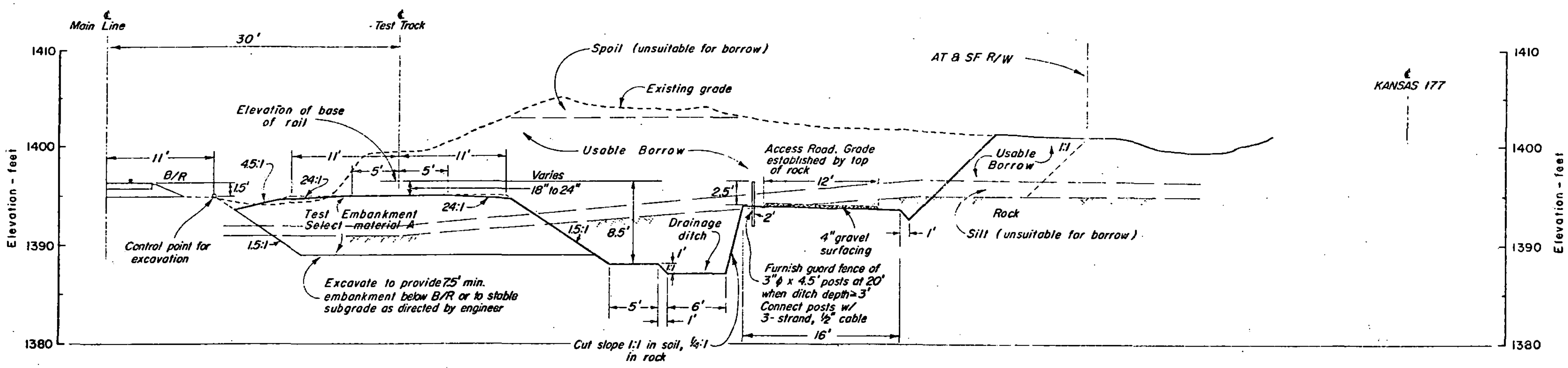
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS
DATED ______ APPROVED ______ Chief Engineer

U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

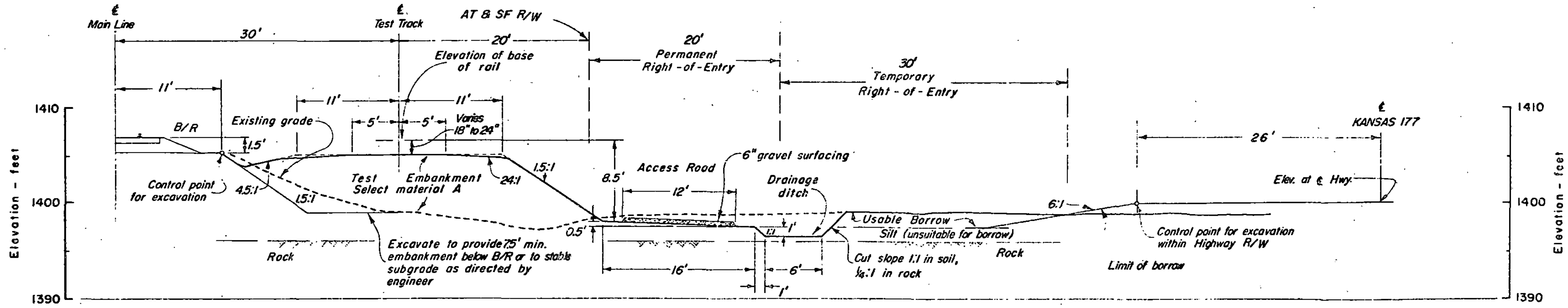
PROFILE 7

Shannon and Wilson
Soil mechanics and foundation engineers

Rev. 6 | 12



TYPICAL EXCAVATION SECTION
(STA. 8582+00)



TYPICAL EMBANKMENT SECTION
(STA. 8555+00)

0 5 10 15 20
Horiz. Scale - feet

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

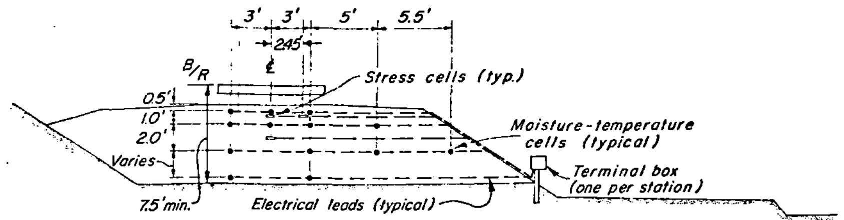
DATED ________ APPROVED ________ Chief Engineer

U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

TYPICAL EXCAVATION & EMBANKMENT SECTIONS

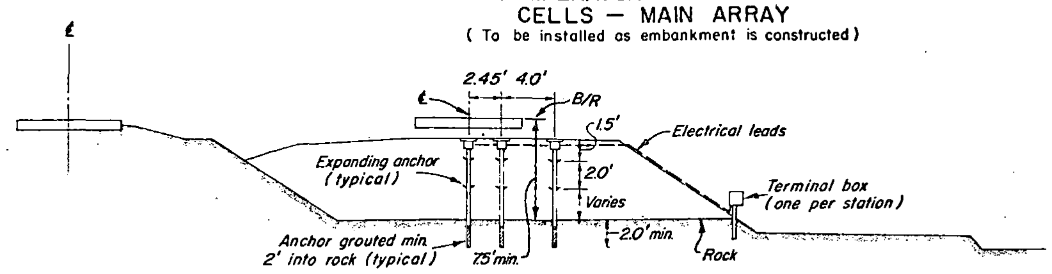
Shannon and Wilson
Soil mechanics and foundation engineers

Rev. 7 / 12



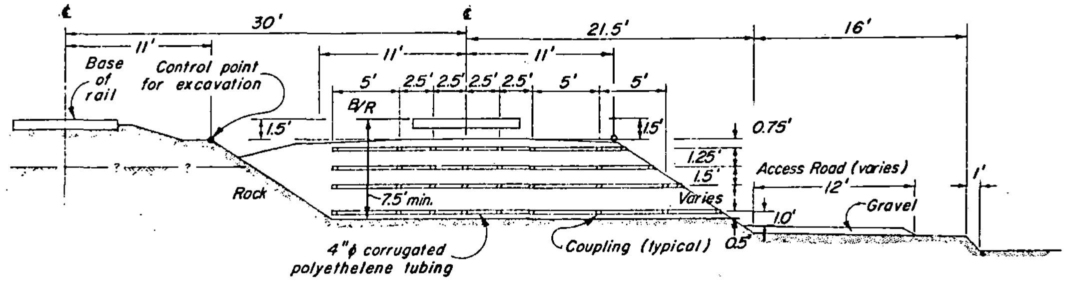
MOISTURE - TEMPERATURE CELLS AND STRESS CELLS — MAIN ARRAY

(To be installed as embankment is constructed)



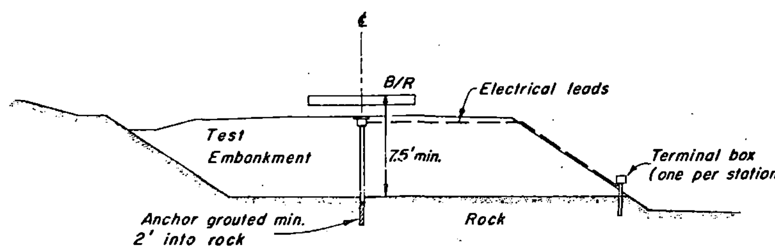
VERTICAL EXTENSOMETERS - MAIN ARRAY

(To be installed after completion of embankment)



HORIZONTAL TUBING - MAIN ARRAY

(To be installed as embankment is constructed)

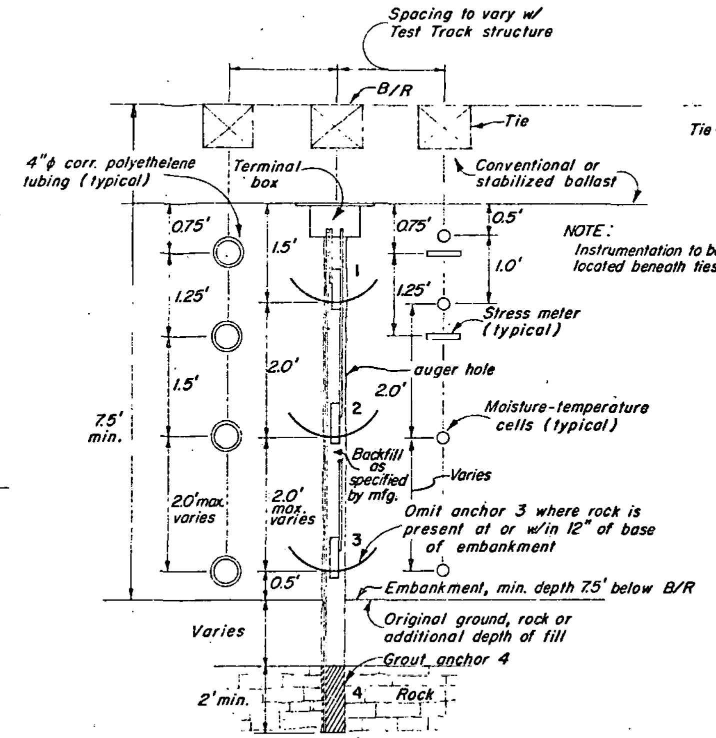


SINGLE - POSITION EXTENSOMETER

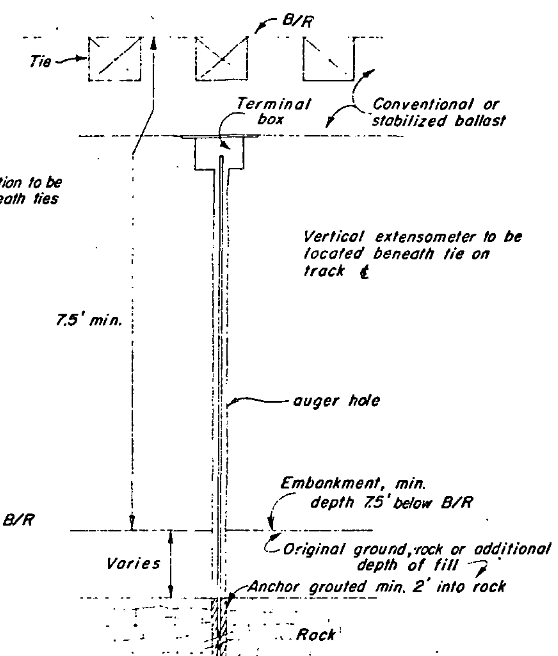
(To be installed after completion of embankment)

0 5 10 15

Scale - feet

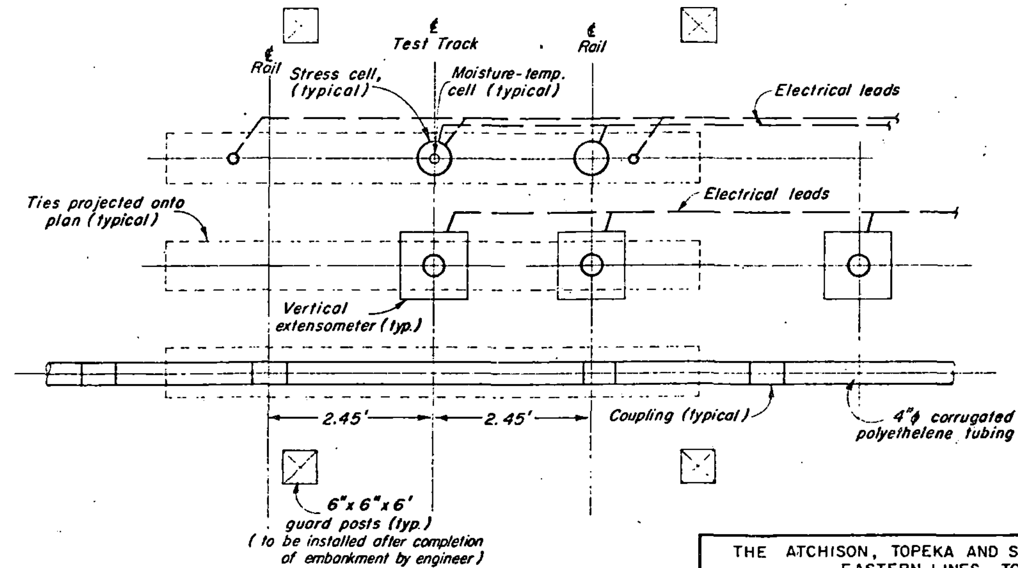


LONGITUDINAL SECTION
INSTRUMENTATION - MAIN ARRAY



LONGITUDINAL SECTION
SINGLE - POSITION EXTENSOMETER

(To be installed after completion of embankment)



PLAN
INSTRUMENTATION - MAIN ARRAY

0 10 20 30 40

Scale - feet

NOTES:

1. All instrumentation to be furnished and installed by the engineer.
2. Moisture - temperature cells, stress cells and horizontal tubing to be installed as the embankment is placed. Extensometers (vertical) to be installed after embankment is completed.

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

DATED [illegible] APPROVED [signature] Chief Engineer

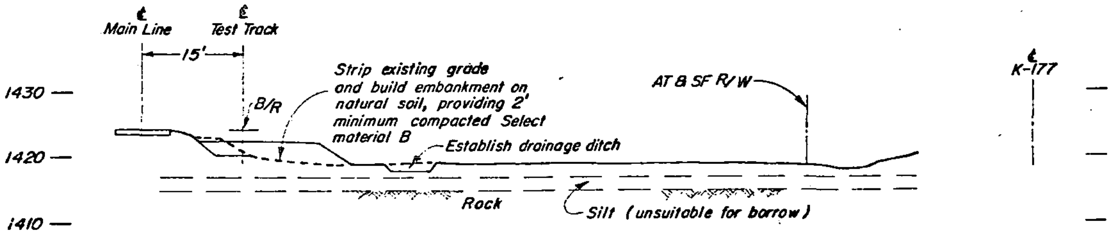
U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

TYPICAL INSTRUMENTATION DETAILS

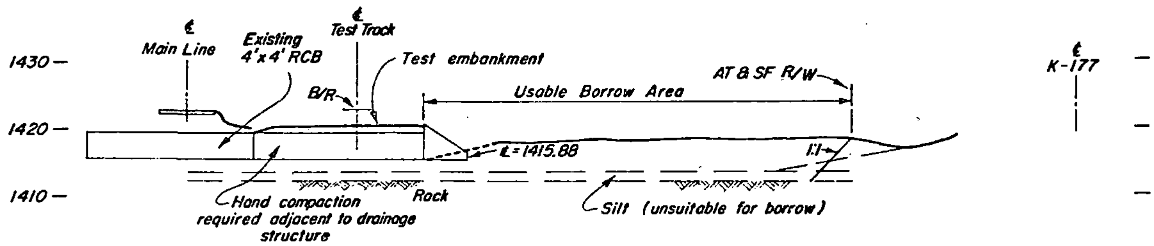
Shannon and Wilson
Soil mechanics and foundation engineers

Rev.

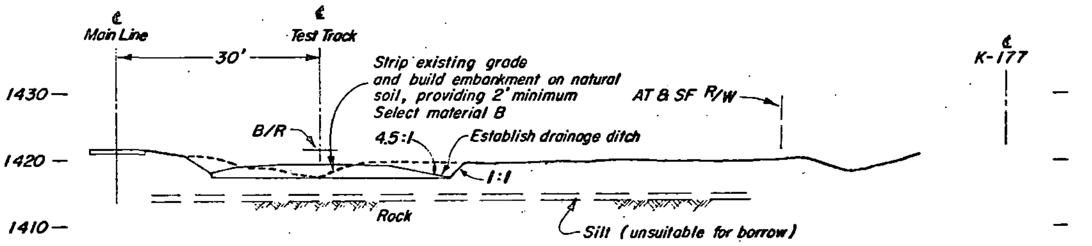
8 / 12



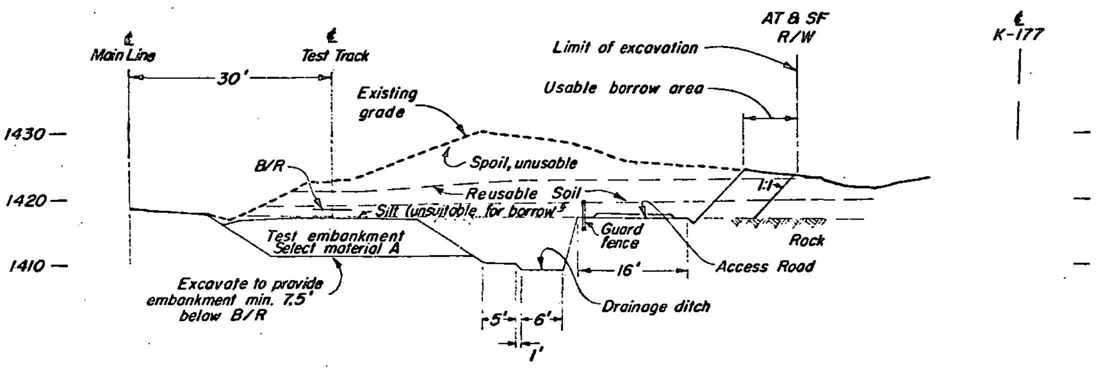
EMBANKMENT SECTION, TRANSITION
STA. 8511 + 51



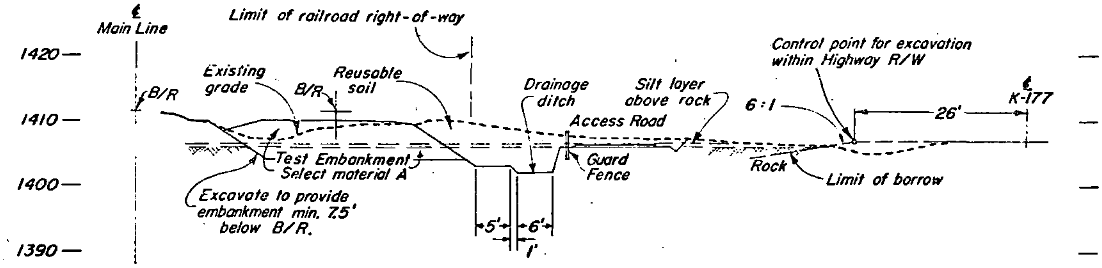
BRIDGE 161.3, TRANSITION ZONE
STA. 8514 + 62



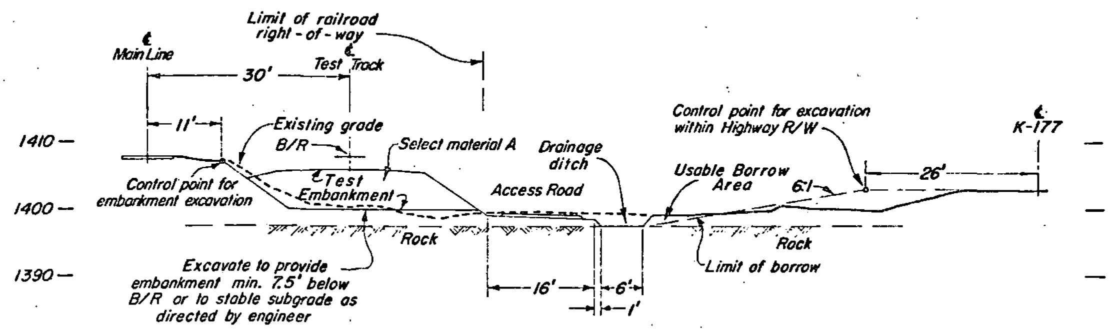
EXCAVATION SECTION, TRANSITION ZONE
STA. 8517 + 35



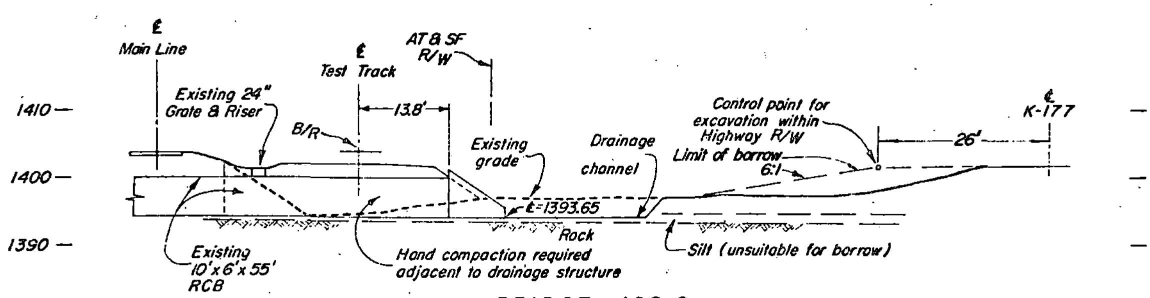
EXCAVATION SECTION
STA. 8525 + 00



EXCAVATION SECTION
STA. 8543+75



EMBANKMENT SECTION
STA. 8552+00



BRIDGE 162.2
STA. 8563+06

NOTES:

1. Subsurface conditions are shown for illustrative purposes. Actual subsurface conditions may vary from that shown.
2. Areas of reusable or unsuitable soil are shown for illustrative purposes. The determination of usable and unsuitable material will be made by the Engineer in the field.

0 10 20 30
Scale – feet

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS

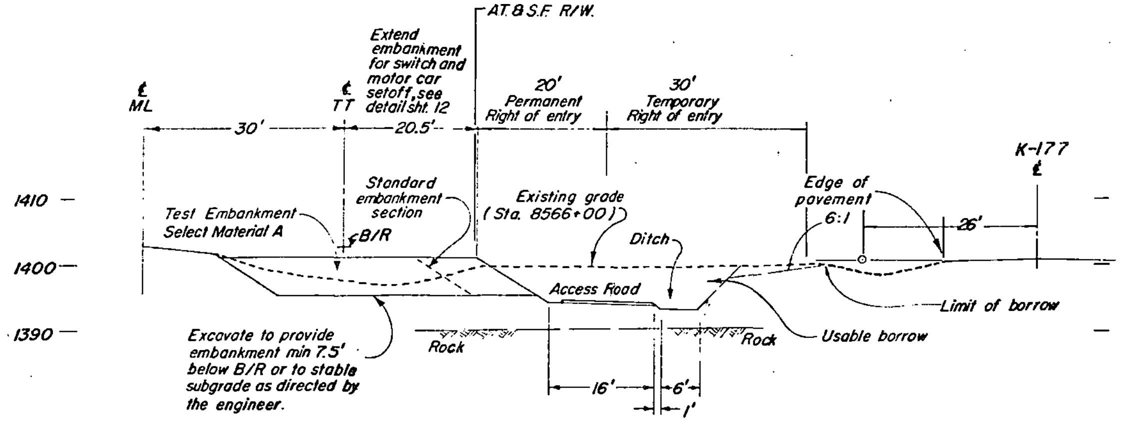
DATED APPROVED
Chief Engineer

U.S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

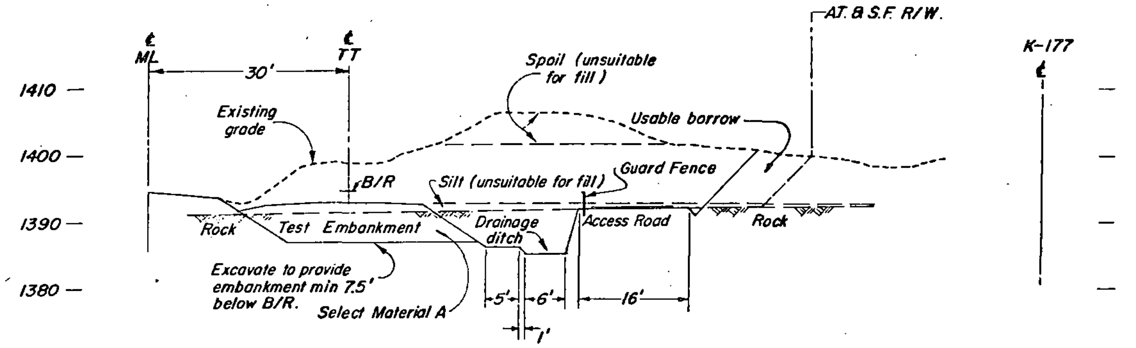
TYPICAL SECTIONS
STA. 8511+51 to STA. 8563+06

Shannon and Wilson
Soil mechanics and foundation engineers
Seattle

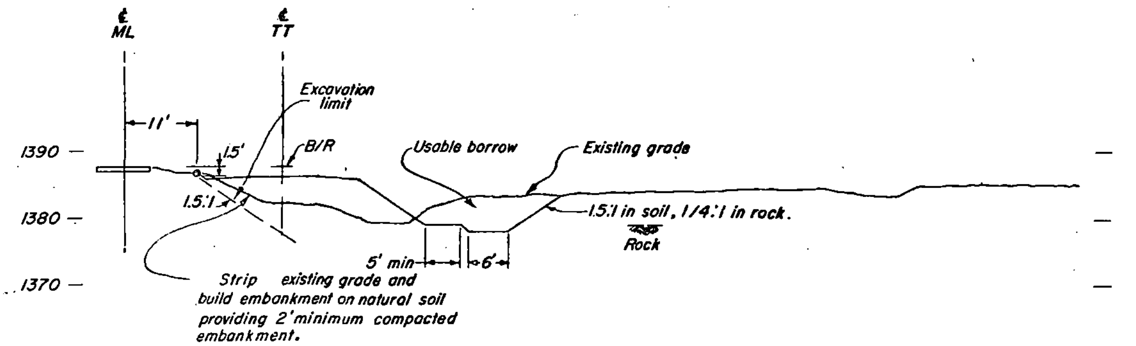
Rev.
9/12



SECTION THROUGH RAMP ROAD AND PLATFORM
FOR SWITCH AND MOTOR CAR SETOFF
AT STA. 8566+00

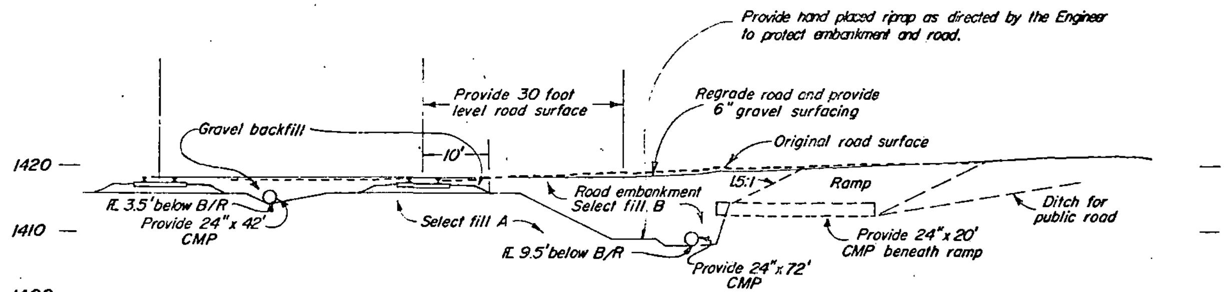


EXCAVATION SECTION
AT STA. 8585+00

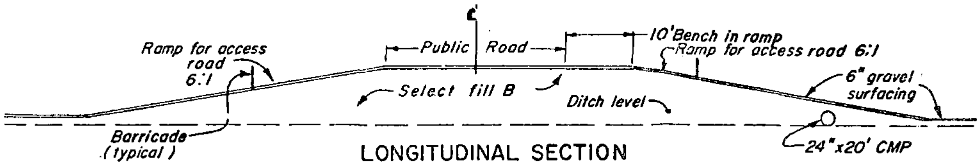


TRANSITION SECTION
STA. 8603+00

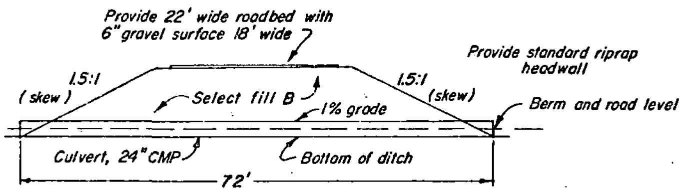
NOTE: See drawing



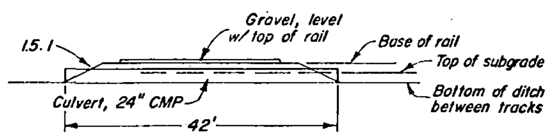
SECTION THROUGH PUBLIC CROSSING
AT STA. 8527+74



LONGITUDINAL SECTION
THROUGH ACCESS ROAD RAMPS
AT STA. 8527+74



CULVERT BENEATH PUBLIC ROAD
AT STA. 8527+74



CULVERT BETWEEN TRACKS
AT STA. 8527+74

9 for general notes.

0 10 20 30
Scale - feet

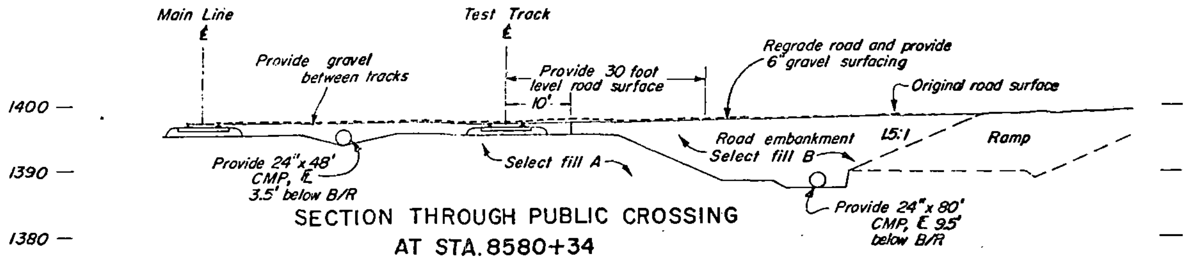
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS
DATED 3-14-470 APPROVED
Chief Engineer

U.S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

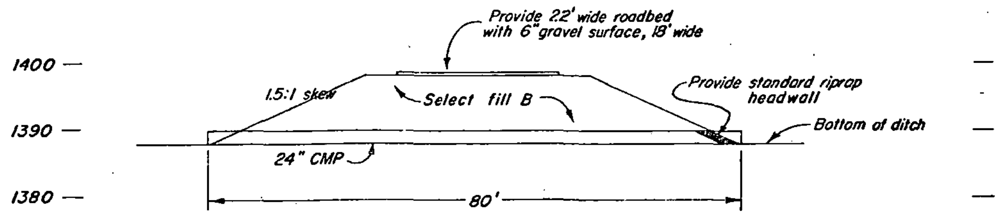
TYPICAL SECTIONS STA. 8566 to 8603 AND PUBLIC ROAD CROSSING, STA. 8527+74

Shannon and Wilson
Soil mechanics and foundation engineers

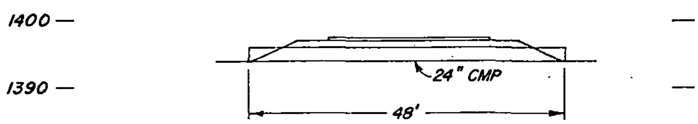
Rev. 10
12



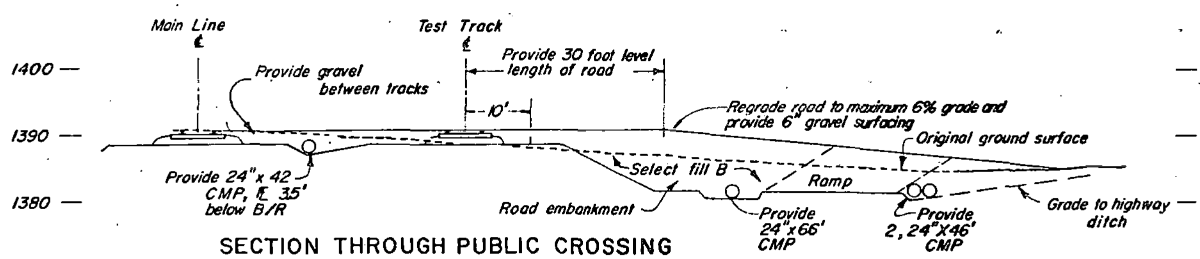
SECTION THROUGH PUBLIC CROSSING
AT STA. 8580+34



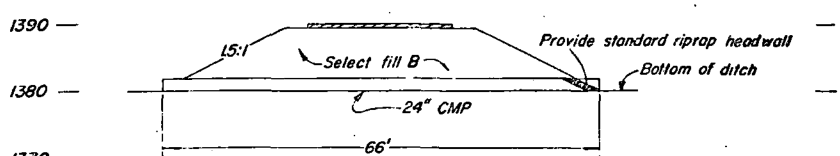
CULVERT BENEATH PUBLIC ROAD
AT STA. 8580+34



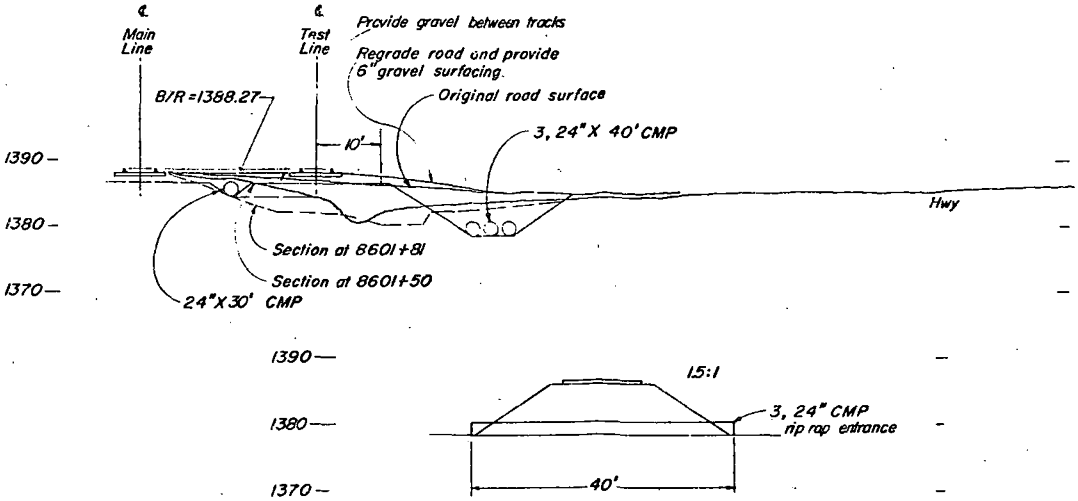
CULVERT BETWEEN TRACKS
AT STA. 8580+34



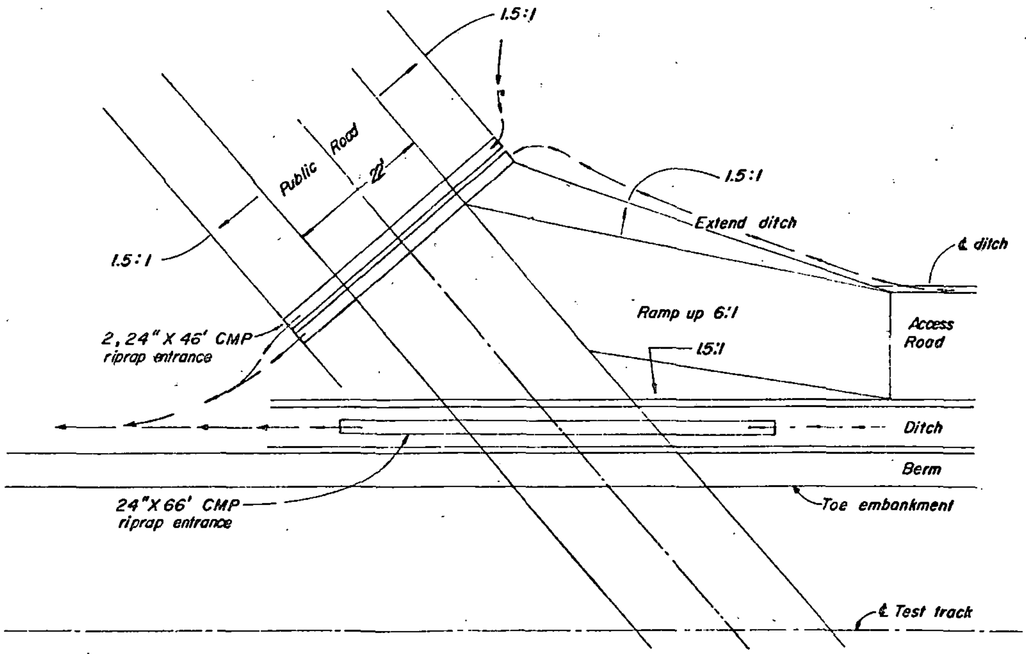
SECTION THROUGH PUBLIC CROSSING
AT STA. 8597+18



CULVERTS BENEATH PUBLIC ROAD
AT STA. 8597+18



CULVERTS BENEATH PRIVATE ROAD CROSSING
AT STA.8601+66



PLAN AT ROAD CROSSING
AT STA.8597+18

0 10 20 30 40 50
Scale – feet.

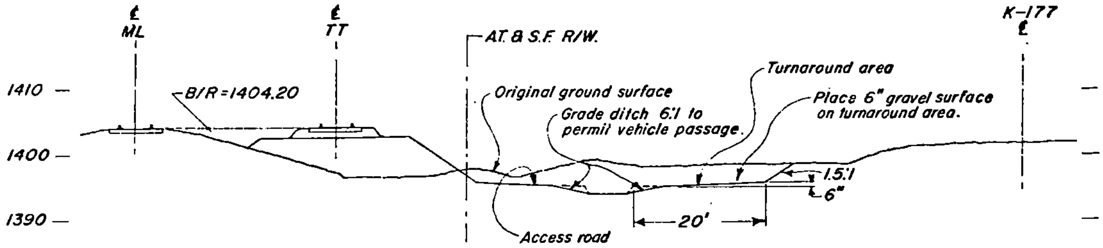
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA, KANSAS
DATED Sept 14, 1970 APPROVED [illegible]
Chief Engineer

U. S. DEPARTMENT OF TRANSPORTATION
EXPERIMENTAL TEST TRACK
EMBANKMENT

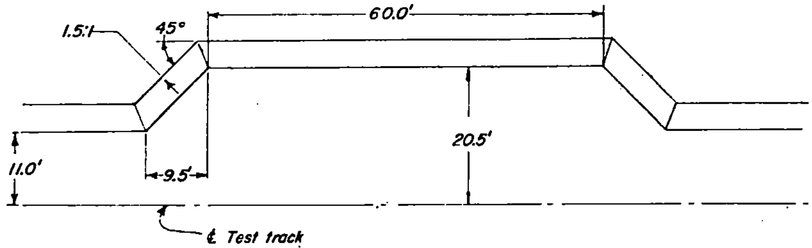
SECTIONS THROUGH ROAD CROSSINGS AT STA. 8580+34, 8597+18, AND 8601+66

Shannon and Wilson
Soil mechanics and foundation engineers

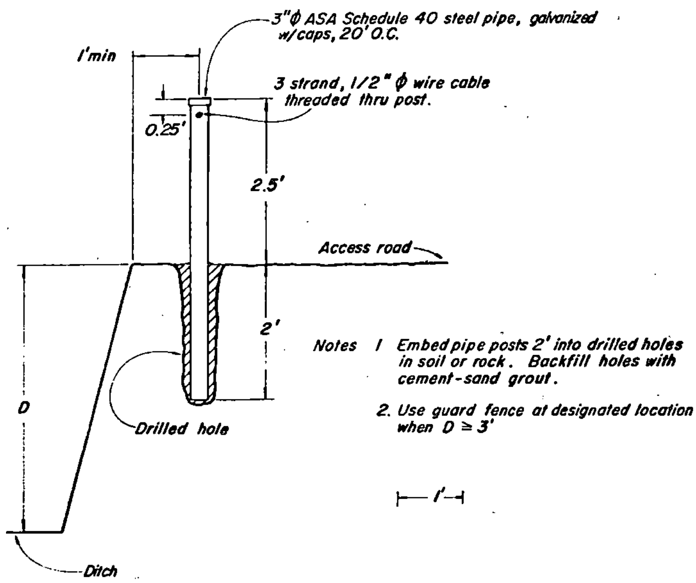
11 / 12



SECTION THROUGH TURNAROUND AREA
STA. 8561+40

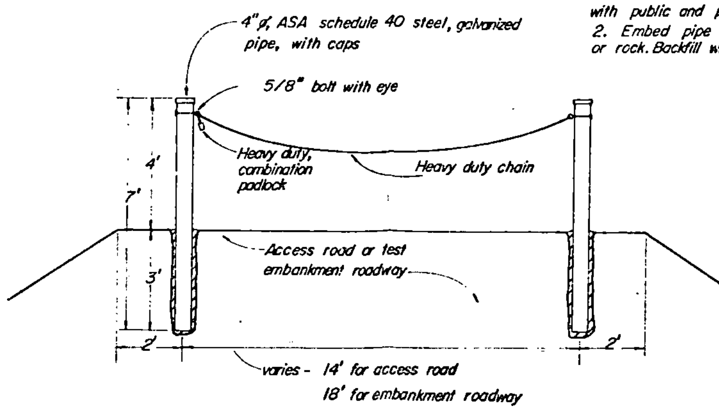


EMBANKMENT FOR SIGNAL AND MOTOR CAR FACILITIES



GUARD FENCE DETAIL

Notes:
1. Place barricades at designated locations on access roads and embankment roadway at junctions with public and private roads
2. Embed pipe posts 3' into soil or rock. Backfill with sand-cement grout.



Scale: ⊢/⊣

BARRICADE DETAIL

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY EASTERN LINES, TOPEKA, KANSAS DATED Sept. 14, 1970 APPROVED [illegible] Chief Engineer
U. S. DEPARTMENT OF TRANSPORTATION EXPERIMENTAL TEST TRACK EMBANKMENT
DETAILS
Shannon and Wilson **Soil mechanics and foundation engineers** — Rev. 12 / 12

GPO 917-301