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The behavior of soils when exposed by an excavation is becoming more and more important. For example, 300- to 500-ft-high cuts are becoming increasingly necessary to the construction of modern highways with long radii curvature and relatively flat grades. The succeeding chapters are designed to assist the practicing engineer in coping with these and other problems. A sufficient amount of theoretical soil mechanics is presented in order to emphasize the effects of soil properties on the behavior of a soil mass. Several case histories of constructed cut slopes are described, with emphasis on geologic conditions and field observations rather than on theoretical analysis based on laboratory test results.

Descriptions are given on soil and geological conditions in California; however, this manual is not intended as a textbook on Soil Mechanics or Geology.

Earth exploration methods in common use are described, particularly those of the California Division of Highways.

The installation and use of various construction control devices are dealt within some detail to clarify their purpose and application.

Supplemental aids to the execution of exploration programs are discussed, as well as the more important reports and records. Sample forms, used for recording essential information, are included. Emphasis is placed on the need for field personnel to thoroughly and carefully record all facts that will aid designers to visualize field conditions as they are. Although the designer is familiar with the significance of an explorations program, those in the field should always remember that the designer is trying to see the job through the field man's eyes.

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Institute of Transportation
and Traffic Engineering

Course Notes

Slope Stability and Foundation Investigation

Prepared by personnel of the Materials and
Research Department, California Division of Highways

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Preface

These notes have been prepared for the short course on Slope Stability and Foundation Investigation, presented by the Institute of Transportation and Traffic Engineering and University of California Extension. They are intended to acquaint practicing highway engineers with current methods of design, construction, and maintenance of stable cut slopes. It is hoped that they may also constitute a sort of highway foundation manual which will assist the practicing engineer in applying rational methods to soil mechanics problems within the subject area.

The bulk of the material in the notes has been derived from technical publications authored by personnel of the Materials and Research Department of the California Division of Highways. The contributions of these many individuals are acknowledged with great appreciation.

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I. Introduction

The behavior of soils when exposed by an excavation is becoming more and more important. For example, 300- to 500-ft-high cuts are becoming increasingly necessary to the construction of modern highways with long radii curvature and relatively flat grades. The succeeding chapters are designed to assist the practicing engineer in coping with these and other problems. A sufficient amount of theoretical soil mechanics is presented in order to emphasize the effects of soil properties on the behavior of a soil mass. Several case histories of constructed cut slopes are described, with emphasis on geologic conditions and field observations rather than on theoretical analysis based on laboratory test results.

Descriptions are given on soil and geological conditions in California; however, this manual is not intended as a textbook on Soil Mechanics or Geology.

Earth exploration methods in common use are described, particularly those of the California Division of Highways.

The installation and use of various construction control devices are dealt with in some detail to clarify their purpose and application.

Supplemental aids to the execution of exploration programs are discussed, as well as the more important reports and records. Sample forms, used for recording essential information, are included. Emphasis is placed on the need for field personnel to thoroughly and carefully record all facts that will aid designers to visualize field conditions as they are. Although the designer is familiar with the significance of an explorations program, those in the field should always remember that the designer is trying to see the job through the field man's eyes.

A list of "Sources of Supplemental Information" is included as Chapter VIII for readers who are interested in more details of a particular area than are presented herein.

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Fig. II-1 — Geomorphic Provinces of California

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II. The Relation of California Geology to Highway Work

A. General

The purposes of this chapter are: (1) to show the great variety in California geology, and (2) to show that this variety has an engineering significance. The description of the geomorphic provinces is extremely brief, serving only as a basis for discussion of the engineering significance. Some common problems related to highway engineering are presented in order to illustrate suitable procedures for exploration and solution.

The geomorphic provinces of California are shown in Fig. II-1.

B. Klamath Mountains Province

1. Geology

a. The Klamath Mountains have been formed by long continued erosion of an uplifted plateau. The resulting peaks rise 6,000 to 8,000 ft above sea level. The average annual rainfall for this province is generally from 40 to 50 in. per year. In some localities, however, annual rainfall as high as 100 in. per year are common. When the dense vegetation on these rugged slopes is disturbed intense erosion can take place.

b. The rocks that compose the Klamath Mountains are generally older and more highly deformed than rocks of the surrounding provinces.

c. In Paleozoic time sedimentary and volcanic rocks were deposited in the ocean that bordered an ancient land mass. The sedimentary rocks are chiefly shales, sandstones, and conglomerates with local lenses of limestone and chert. The volcanic rocks are mostly basalt or andesite although some rhyolite does occur.

d. Granitic and ultra basic rocks intruded these older sedimentary and volcanic rocks at approximately the same time that the granitic rocks of the Sierra Nevada were emplaced. The Paleozoic rocks were folded and faulted and some were metamorphosed. The uplifted area was rapidly eroded, then near the end of the Cretaceous period, the Klamath region was again uplifted.

e. Tertiary rocks are present in isolated locations within the Klamath Mountains. Some of these sedimentary rocks are continental in origin while others show evidence of marine deposition. In the higher regions of the Klamath Mountains there is some evidence of past glacial activity.

2. Engineering Significance

a. Cut-Slope Design

(1) Cut-slope stability in the Paleozoic rock is greatly affected by the orientation of the discontinuities in the rock mass. Joint patterns and faulting should be analyzed in designing cut slopes in these rocks. Some of the Paleozoic rocks have a definite foliation. The orientation of this foliation should be considered in cut slope design.

(2) To design cut slopes in granitic rock the competency of the rock must be evaluated. Granitic rock ranges from decomposed granite to hard, fresh rock with very few joints. Cuts of 30 ft or less in height in decomposed granite can be designed on steep slopes. Cuts higher than 30 ft in decomposed granite may need to be designed somewhat flatter in order to avoid a total failure of the slope face. Near vertical slopes can be used for hard fresh granitic rock with few joints. For granitic rock between these two extremes, jointing patterns become critical in the slope design. Rockfall is the main problem in designing cut slopes in granitic rock.

(3) Serpentine is present in the Klamath Mountains province and has been intensely sheared in many areas. In designing cut slopes in sheared serpentine the orientation of the road prism to the shearing is very important. If centerline crosses at right angles to the foliation, fairly steep slopes can be used. However, if centerline parallels the foliation, flatter

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slopes will probably be required.

(4) The Tertiary sediments will generally require a moderate slope design.

b. Embankment Foundations

Embankment foundations within the Klamath Mountains province can be a problem. Much of the Klamath Mountains has undergone deep weathering and thick soil covers have developed. Extensive drainage systems including stabilization trenches may be required to provide a stable foundation for embankments.

c. Materials Sources

Roadway materials of suitable quality are generally available in the Klamath Mountains Province. Aggregates to be used for portland cement concrete or asphalt concrete can be obtained from streams within the province. Materials to be used for rock slope protection can be obtained by careful selection within the granitic rock or within the older metamorphic rocks.

d. Excavation Characteristics

(1) The intensely weathered portion of the Paleozoic rock can usually be ripped. The less weathered rock will probably require blasting.

(2) Decomposed granite can be ripped fairly easily. Hard fresh granite will require heavy blasting.

(3) In general, the Tertiary sedimentary rocks can be ripped.

e. Landslide Problems

There are a number of existing landslides within the Klamath Mountains province. These slides are not easy to detect because of the heavy vegetation in some areas and the thick soil cover. Aerial photos and geologic reconnaissance can be used to locate these existing landslides.

f. Economic Mineral Deposits and Mining Claims

Economic mineral deposits and mining claims exist throughout the Klamath Mountains. Any deposits or claims that are located within the proposed right-of-way should be evaluated at an early stage.

C. Cascade Range and the Modoc Plateau

1. Geology

a. The Cascade Range and the Modoc Plateau geomorphic provinces are discussed together because in general they have similar rock types and the engineering significance for these rock types is similar. The Cascade Range is a volcanic mountain range while the Modoc Plateau is a highland area capped by volcanic rock. With the exception of some of the higher peaks such as Mt. Shasta, the average annual precipitation is low for this area, averaging 8 inches per year or less over much of the region.

b. The oldest rocks in the Cascade Range are Miocene in age and are referred to as the Western Cascade series. These rocks consist of lava flows of basalt, andesite and dacite with some interbedded pyroclastic rocks such as tuff and ash. After the Western Cascade series was eroded for an interval, a new series of volcanic rocks referred to as the High Cascade series was erupted. The High Cascade rocks are mainly andesite. The most recent volcanic activity in the Cascade Range was when Mt. Lassen erupted during the summers of 1914-1917.

c. The oldest rocks of the Modoc Plateau are a series of interbedded lava flows, pyroclastic rocks and lake deposits that form some of the block faulted ranges. Rocks of Pliocene age include both volcanic and lake deposits. These later rocks are much less faulted and less tilted than the older group.

2. Engineering Significance

a. Cut-Slope Design

Cut-slope design in volcanic flow rocks can be quite steep for moderate cut heights.

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Joint patterns are important to slope stability in these rocks. However, many times a vertical jointing pattern and a horizontal jointing pattern are present that will allow steep designs to be used. Unconsolidated materials such as ash or cinders will require a moderately flat design. These materials are stable at their angle of repose. Many times both volcanic flow rocks and the pyroclastic rocks such as ash or cinders will be encountered within the same cut. Under these conditions a variable slope design is often used with steeper slopes being designed in the rock and flatter slopes being designed for the unconsolidated material.

b. Embankment Foundations

Foundations for embankments within these provinces are generally adequate. One possible exception may be where the embankment is being constructed over an ancient lake bed. Some settlement can be anticipated because of a considerable depth of compressible material.

c. Materials Sources

Stream gravels are scarce in these provinces. Most of the commercial sources are located at the western or southern boundary. Crushed volcanic rock or cinders are available in many places in the region.

d. Excavation Characteristics

The volcanic flow rocks such as basalts and andesites may require blasting for excavation unless the material is very broken. Pyroclastic materials such as ash and cinders can be excavated easily without blasting. Lake sediments likewise will not require blasting.

e. Lava Tubes

Sometimes in these volcanic regions the top of a lava flow would harden forming a crust while the underlying molten rock would continue to flow. Thus a tunnel or lava tube was formed. After a lava tube has been formed the roof or a portion of the roof may collapse. If a road is to be built across a region that has lava tubes, exploration work must be done to ensure that the road will not cause collapse of the roof of these tubes.

D. Coast Ranges

1. Geology

a. The Coast Ranges are a series of more or less discontinous northwest-trending mountain ranges extending from the Klamath Mountains on the north coast of California to the Transverse Ranges on the south. Elevations range from sea level to over 7,000 ft. Total precipitation varies from almost 100 in. per year in the north coast area to less than 8 in. per year for parts of the southern coast ranges.

b. The structure of the Coast Ranges is exceedingly complex with intense folding and faulting having affected all formations. There is a substantial difference in the basement rock types within the Coast Ranges. Each has had a profound effect on the character and structure of the later sediments. There is a crystalline basement rock composed of gneiss, schist, and marble of the Sur series that has been intruded by granitic rock. These crystalline basement rocks have provided a firm foundation for the later sediments. The other type of basement rock is represented by the Franciscan formation which is composed of sandstone and shale along with associated metavolcanic rock, chert and schist. This less competent basement rock has permitted intense folding to take place in the later sedimentary cover.

c. The Franciscan formation is exposed extensively throughout the Coast Ranges. The material that composes this formation was deposited in a marine environment. The formation has been intruded by serpentine and other ultrabasic rock.

d. Unmetamorphosed sandstone and shale were deposited contemporaneously with the Franciscan formation in part and deposition extended through Upper Cretaceous time. These sediments will be referred to as Upper Cretaceous rock.

e. After late Cretaceous time the seas became more restricted. Therefore there is more variation between rocks of the same age from the north end of the province to the south end. The most extensively exposed formation is called the Monterey formation of the Middle to Late Miocene age. This formation is characterized by siliceous shale, chert and diatomaceous shale although other rocks such as shale and sandstone are also present.

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f. There was considerable volcanic activity in Miocene time. There was also a slight amount of volcanic activity during late Pliocene time.

g. The rocks of the Coast Range have been folded in a complex manner. The dominant structural feature of the Coast Ranges is the northwest trending fault zones. The most well known of these fault zones is the San Andreas fault which extends from the southern end of the province northward heading to sea just north of Point Arena. Other large fault zones within the Coast Range include the Nacimiento fault, the Hayward fault, the Calaveras fault and the South Fork Mountain fault that separates the north end of the Coast Ranges from the Klamath Mountains.

2. Engineering Significance

a. Cut-Slope Design

(1) Cut-slope design within the basement complex of the Sur series and associated granitic rocks presents no unusual problems. Slope designs are usually based on the degree of weathering of the rocks and on the orientation of discontinuities such as joint patterns and fault zones.

(2) Slope design within the Franciscan formation can be a major problem. Many of the natural slopes within the Coast Ranges show evidences of instability. This instability can range from soil creep to massive landslides. Cutting into slopes that are barely stable in a natural condition generally precipitates sliding. Even in areas where the natural slopes appear to be stable, considerable exploration work is necessary for slope design. Competency of the materials in the Franciscan formation varies greatly. For example, an area that has sandstone outcrops may also include a zone of badly sheared shale. Heavy annual rainfall contributes to the instability. Extensive drainage may be required for stable cut slopes. In some areas of the Franciscan formation a slope design similar to that used in soils may be required. In other areas rock may be competent enough that consideration must be given to the joint patterns and the faulting in the area. The design of slopes in serpentine that occurs within the Franciscan formation will be strongly influenced by the orientation of the roadbed to the foliation of the serpentine. Cut slopes at right angles to the foliation can be designed rather steeply, while cut slopes parallel to the foliation will probably require flatter slopes.

(3) Slope design in the Upper Cretaceous rock should be based on the orientation of the road prism to the bedding and the orientation of the discontinuities such as joints and faulting.

(4) Slope design in rocks younger than Upper Cretaceous can vary widely. Some of these rocks are quite competent and can stand on steep cut slopes. Other formations such as the Purisima formation are prone to landsliding. Some formations are prone to gulleying and erosion such as parts of the Aromas formation.

b. Embankment Foundations

(1) The location and design of embankments within the Coast Ranges requires careful study. Many of the foundation problems are associated with the Franciscan formation. Because many of the natural slopes within the Franciscan formation tend to be unstable, the placement of side hill embankments becomes somewhat hazardous. Embankments crossing drainage channels also require careful study. The location of many drainage channels is controlled by fault zones. Since fault zones are areas of weakened rock, the material is eroded more easily. The soils that develop on the rocks in the Franciscan formation contain several clay minerals and some are expansive.

(2) Embankment foundation problems can occur in areas that are not underlain by the Franciscan formation. The Coast Range province consists of a series of ranges. Between these ranges the intermontane valleys may contain compressible material. Embankments built in these areas may have considerable settlement.

(3) Along the coast or within the Bay Area, embankments may be required across slough areas or ancient lagoonal areas. These types of foundations also have settlement problems.

c. Materials Sources

(1) Adequate materials can be found within the Franciscan formation. However, the quality of materials is not as high as in some other geomorphic provinces. The location of

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adequate materials therefore may require considerable exploration. Aggregates for portland cement concrete and for asphalt concrete can be obtained from the stream gravel in the Coast Ranges. The stream gravels derived from the Monterey formation may contain siliceous shales and cherts that would be reactive with the alkalies in cement.

(2) Rock slope protection material may be obtained in the Coast Ranges. However, obtaining adequate size may be a problem. Rock slope protection material can be obtained in selected areas within the Franciscan formation. It can also be obtained from some of the more massive sandstone beds of the Upper Cretaceous. The basement complex can also provide some rock slope protection.

d. Excavation Characteristics

(1) Metamorphic rocks of the Sur series and the granitic rocks that intrude this series are weathered and broken by faulting. Any cut, however, with a depth of 20 to 30 ft or greater is likely to require blasting for excavation.

(2) Much of the rock of the Franciscan formation is deeply weathered and badly broken by faulting. This type of rock can usually be ripped. There are areas, however, within the Franciscan formation that will require blasting.

(3) The thinly laminated beds of the upper Cretaceous can usually be ripped, but the massive sandstone beds will probably require blasting.

(4) The younger rocks that occur in the Coast Ranges usually can be ripped, but there will be some exceptions where both sedimentary and volcanic rocks will require some blasting.

e. Landslide Problems

(1) Existing landslides are prevalent throughout the Coast Ranges. In some areas slides are so frequent that no highway alignment can avoid all of them. This situation is particularly true in the northern Coast Ranges. When the existing slides cannot be avoided, corrective measures must be taken to provide a stable highway. The types of sliding range from surface creep and rather shallow debris slides to deep seated extensive landslides. Thorough exploration may be required to determine which type of sliding is occurring. Corrective measures will also depend upon the type of sliding.

(2) There are many existing slides throughout the Franciscan formation, but sliding is not confined to the older rocks. Younger formations such as the Purisima formation and the Orinda formation have many existing slides.

f. Seismic Activity

(1) A number of active faults exist within the Coast Ranges. The most widely known of these active faults is the San Andreas fault. In Central California the Hayward fault and the Calaveras fault are also active and in the southern part of the Coast Ranges the Nacimiento fault and the Big Pine fault are active.

(2) Movement on these active faults can adversely affect engineering work. Surface rupture along one of these faults would displace any engineering structure crossing it. Therefore, when a highway crosses an active fault, embankments are probably preferable to structures because of the ease of repair. In addition to surface displacement, the earthquakes that accompany movement along an active fault can cause damage. This damage is usually restricted to some rockfall and some settlement in the embankment areas. In extremely active seismic areas cut and embankment slopes may require a less steep design than would normally be used.

E. Great Valley

1. Geology

a. The Great Valley is a large, essentially flat area lying between the Coast Ranges on the west and the Sierra Nevada Mountains on the east. It is more than 400 miles long and averages some 50 miles wide. It is drained from the north by the Sacramento River and is drained from the South by the San Joaquin River. These two rivers join and flow through the Carquinez straits. The average rainfall is more than 36 in. per year in the north end of the Great Valley, decreasing gradually to less than 8 in. per year in the south end.

b. The Great Valley is a northwest trending structural trough whose axis is located

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near the eastern boundary of the Coast Ranges. The trough was formed by the westward tilting of the Sierra Nevada block. Many thousands of feet of sedimentary material have been deposited in this depression. Most of this material is buried deeply and only the more recent deposits are of special engineering interest. In the northern part of the Sacramento Valley reddish colored sedimentary terrace deposits rim the Great Valley. Most of the other material in the northern part of the Great Valley are river formed deposits of various kinds. Considerable variation in grain size for these materials can be expected depending upon how the material was deposited. For example, former stream channels may have coarse gravels while the outer edges of flood plains may have extremely fine-grained material. One distinctive feature of the northern part of the Great Valley is the Marysville Buttes. These Buttes are volcanic in origin and have pushed up through the sedimentary rocks. The southern portion of the Great Valley, that part drained by the San Joaquin River, also has many river-formed features. Large alluvial fans extend out into the Great Valley from both the Sierra Nevada and the Coast Ranges.

2. Engineering Significance

a. Cut-Slope Design

Most cuts that would be required in the Great Valley will be relatively small in size. The cuts will be in poorly consolidated materials for the most part and cut slopes generally will be relatively flat. Special attention should be given to designing cut slopes in the low gently rolling hills along the west side of the San Joaquin Valley. Interbedded sands, silts and clays generally dip toward the valley and sliding can occur along thin clay beds on very flat angles.

b. Embankment Foundations

Embankments constructed in the Great Valley will generally not be of great height; however, there are a few areas of the Great Valley where even low embankments will encounter foundation problems. The groundwater table is high and there are areas of peat in the delta areas where the Sacramento and San Joaquin Rivers meet. Settlement of embankments will be a problem in these areas.

c. Materials Sources

(1) Aggregates to be used for portland cement concrete or asphalt concrete can be obtained from streams that flow into the Great Valley from the Sierra Nevada Mountains. A few streams that flow from the Coast Ranges provide a usable aggregate although the quality is not nearly as good as the Sierra Nevada aggregates.

(2) Rock Slope Protection material can be obtained from some of the massive sandstone beds that exist along the eastern side of the Coast Ranges. Granitic rocks and metamorphic rocks from the Sierra Nevada can also provide rock slope protection.

d. Excavation Characteristics

Excavation of the poorly consolidated sediments of the Great Valley can usually be accomplished with ease.

e. Subsidence

Subsidence is a special problem that must be dealt with along the western edge of the San Joaquin Valley portion of the Great Valley. Some of the soils in this area such as those developed from mud flow deposits have a flocculent structure. When this flocculent structure becomes wet, from some process such as irrigation, the structure collapses and considerable subsidence can result.

F. Sierra Nevada

1. Geology

a. The Sierra Nevada geomorphic province is roughly 430 miles long and 40 to 80 miles wide. It has a general northwest trend. A cross section through the province shows a striking difference between the eastern and western slopes. As a result of relatively recent faulting the eastern slopes are very steep while the western slopes are characterized by a gradual descent from the summit to the Great Valley. Many of the peaks within the southern part of the range exceed an elevation of 12,000 ft.

b. The Sierra Nevada mountain range lies parallel to the Pacific Coast and forms a

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barrier to moisture laden air moving eastward from the Pacific Ocean. Most of the moisture is deposited on the western slopes in the form of rain and snow. Consequently, most of the streams and rivers drain westerly. The streams that do flow east derive most of their water from rain and snowfall at or near the summit. In general, total precipitation decreases from north to south and from the summit toward lower elevations.

c. The geological history of the Sierra Nevada begins some three hundred million years ago when the present site of the Sierra Nevada range was occupied by a shallow ocean trough. Sediments as thick as 30,000 ft along with some volcanic rocks are estimated to have been deposited in this trough. Then possibly 120 to 130 million years ago the sediments in this trough were buckled and warped into a series of low mountain ranges. The roots of the mountain ranges were intruded by granitic rock. A major fault system developed along the eastern edge of the Sierra Nevada Range and the mountain block was tilted upward. Much of the original sedimentary material has been eroded away, exposing extensively the granitic rock that can be seen in the higher elevations today. The older rocks that do remain have been metamorphosed and are exposed in the Sierra Nevada foothills and in roof pendants that are exposed in various locations within the granitic rock. Erosion of these rolling Sierran hills continued to the Eocene epoch. During this time slowly moving streams transported the rock debris, including quartz particles, to a shallow sea. These quartz sand and clay deposits which were formed constitute the Ione formation. During this same period the auriferous stream gravels were also being deposited. Following deposition of the Ione formation, extensive bodies of rhyolite, tuff, and associated gravel of the Valley Springs formation were deposited on the existing Sierra Nevada landscape. However, at this time, there was also renewed uptilting of the Sierra Nevada block, and much of the Valley Springs formation was rapidly eroded away. During Pliocene time there was continued uplift of the mountain range and numerous volcanic eruptions occurred. Much of the volcanic material was in the form of andesitic mud flows which filled the various drainage channels. These mud-flow-type deposits are part of the Mehrten formation. Volcanic activity, along with continued uptilting, continued through the Pliocene time. During the Pleistocene time when the Sierra Nevada had been uplifted to near its present height, glaciers formed that carved out the U-shaped valleys in the high Sierra and deposited glacial debris in the higher altitudes. Today continued earth movements are indicated by earthquakes and minor seismic activity that occur along the east side of the Sierra Nevada geomorphic province.

2. Engineering Significance

a. Cut-Slope Design

(1) In designing cut slopes in the older metamorphic rocks, consideration must be given to the orientation of the road prism in relation to the foliation of these rocks and also to the jointing patterns that exist in the rock. Rockfall is the main problem in slope design in these metamorphic rocks.

(2) The design of cut slopes in granitic rocks requires a careful evaluation of the competency of the rock. Granitic rock can vary from a decomposed granite to a hard fresh rock with very few joints. In general, small cuts in decomposed granite can be stable with steep slopes, while large cuts in decomposed granite require relatively flat cut slopes in order to avoid a failure of the cut face. Slopes that are nearly vertical can be used in hard fresh granitic rock. Jointing patterns become critical in the cut slope design for granitic rock between these two extremes. Rockfall is the main problem in designing cut slopes in granitic rock. Much better results can usually be obtained from cut slope design in granitic rock if controlled blasting methods are used that will provide some insurance against overshooting.

(3) Serpentine is present in the Sierra foothills. If the roadbed crosses the serpentine body at right angles to the foliation, fairly steep slopes can be used. However, if the centerline parallels the foliation of the serpentine body, flatter slopes will probably be required.

(4) Cut slope design in the Ione formation, the Valley Springs formation or the weathered parts of the Mehrten formation need to be relatively flat in order to avoid a failure of the cut face. Slope design in the harder volcanic rocks such as basalt flows can be fairly steep with rockfall again being the main problem. Slope design in glacial material depends on the height of cut and compactness of the material.

(5) Occasionally a highway alignment will be in cut through a talus area. Cuts in talus should be analyzed very carefully because most talus slopes are just barely in equilibrium and

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any disturbance even by a minor cut may cause movement of the talus slope.

b. Embankment Foundations

(1) Embankment foundation problems within the Sierra Nevada province are generally minor in nature. Nevertheless, some exceptions should be mentioned. For instance, it may not be possible to construct sidehill embankments on steep slopes where a rock slope is exposed.

(2) An analysis is required when the embankment crosses some of the marshy areas that are present near the crest of the Sierra Nevada. Settlement may be excessive in some of these meadows.

(3) An embankment foundation problem that is peculiar to the Sierra Nevada occurs when the embankment is placed on the contact between the Mehrten formation and the underlying Valley Springs formation. The Mehrten formation in many places is a fairly pervious material while in general the underlying Valley Springs material is less pervious. When an embankment is placed on this contact and the heavy winter rains saturate the Mehrten material a high hydrostatic head develops that may cause a foundation failure.

c. Materials Sources

(1) Roadway materials of suitable quality are generally available in the Sierra Nevada. For example, subbase material and even base material can often be obtained from roadway excavation particularly in the metamorphic rocks and the granitic rocks. Aggregates for use in portland cement concrete or asphalt concrete can be obtained from the streams that flow from the Sierra Nevada. Most of the standard aggregate sources are at the lower elevations. Some aggregate sources are located on the eastern side of the Sierra Nevada range.

(2) Rock slope protection materials can be obtained by careful selection within the granitic rocks. This is also true to a more limited degree in the older metamorphic rocks. If small amounts of slope protection material are needed selected stream boulders or volcanic flow rocks can sometimes provide the material.

d. Excavation Characteristics

(1) Granitic rock, as mentioned under cut slope design, varies greatly in its competency. Decomposed granite can be ripped easily whereas hard fresh granite will require heavy blasting. A seismic survey is one of the best methods for obtaining information to classify excavation. Information from such a survey can also provide a guide for estimating earthwork factors.

(2) The upper weathered material of the older metamorphic rocks can usually be ripped. As these rocks become fresher the more massive types will require blasting. Some of the thinly plated types may be rippable although the individual pieces themselves become quite hard.

(3) The serpentine that occurs in the foothills can often be ripped. This is particularly true if the rock is intensely sheared. However, occasional massive boulders or pods will require blasting.

(4) The sands and clays of the Ione formation usually can be ripped easily.

(5) Excavation characteristics for the volcanic rocks within the Sierra Nevada vary widely. The rhyolite tuff of the Valley Springs formation generally is rippable. The Mehrten formation is rippable where the formation is deeply weathered. If the formation is not deeply weathered, some blasting may be required. The volcanic flow rocks such as basalt and andesite will usually require blasting unless the rock is quite broken.

(6) Although some of the glacial material is highly compacted it generally can be ripped.

e. Landslide Problems

Naturally occurring landslides are not numerous in the Sierra Nevada. Detection of existing landslides can best be accomplished by using aerial photographs and a geologic reconnaissance.

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f. Sinkholes

An engineering geologic problem of limited extent occurs in the Sierran foothills in the areas underlain by limestone. Limestone is somewhat soluble and groundwater traveling along joints tends to enlarge these joints and under certain conditions will cause the development of underground caves. Collapse of the roof of these caves when they are near the surface causes the development of a sinkhole. Sinkholes that develop within the right-of-way can be quite hazardous.

g. Seismic Activity

As was mentioned earlier there is seismic activity along the eastern front of the Sierra Nevada Range. For the most part this activity has not been intense and damage to highways has been limited to settlement of embankments with some associated cracking and falling rock damage to paved surfaces. Damage to structures can be a more serious problem.

h. Economic Mineral Deposits and Mining Claims

Extensive mining operations have been conducted in parts of the Sierra Nevada in the past. Economic mineral deposits and mining claims that are located within the proposed right-of-way should be evaluated in an early stage, preferably during planning.

G. Transverse Ranges

1. Geology

a. The Transverse Ranges are an east-west trending group of mountains and valleys. The name is derived from the fact that this east-west trend is transverse to the general northwesterly trend of most of the structure in California. There is a variety of topography and a wide range of elevations within the transverse range. Elevations over 10,000 ft are recorded in the San Gabriel Range and the San Bernardino Mountains. The average annual rainfall at the western end of the Transverse Ranges is approximately 7 in. a year, while over 35 in. per year is normal near Big Bear Lake located northeast of San Bernardino.

b. The geology of individual mountain ranges within the Transverse Ranges is quite varied. Most of the San Bernardino Mountains and the San Gabriel Mountains consist of very old metamorphic rocks that have been intruded by granitic rocks. The metamorphic complex consists of Precambrian gneiss, Pelona schist, Paleozoic metasedimentary rocks that include quartzite and limestone, and the Santa Monica slate.

c. The western part of the Transverse Ranges is composed mainly of younger sedimentary and volcanic rocks. These include exposures of Mesozoic age rocks that include the Franciscan formation and Upper Cretaceous sediments. Rocks of Paleocene and Eocene age are mostly sandstone, shale and some conglomerate. However, the Vasquez formation contains some volcanic flow. Miocene age rocks include the Rincon shale, the Vaqueros sandstone, the Topanga formation and shale of the Monterey (Modelo), Puente and Sisquoc formations. Pliocene age rocks include the sandstone and shale of the Pico formation.

d. There are numerous faults throughout the Transverse Ranges, some of which are still active. These include the San Andreas fault, San Gabriel fault, Santa Ynez fault and the Newport-Inglewood fault.

2. Engineering Significance

Engineering work within the Transverse Ranges requires that many geologic problems be solved. Because of the wide variety of materials within the province, a thorough exploration program is needed to obtain the information necessary for engineering design.

a. Cut-Slope Design

(1) Cut-slope design for the rocks of the metamorphic complex should be based upon the orientation of the foliation of these rocks, the orientation of joint patterns and the orientation of faulting.

(2) Granitic rocks vary widely in their competency. Cuts of 30 ft or less in height in decomposed granite can be designed on steep slopes. Cuts higher than 30 ft will need a flatter design to avoid a total failure of the cut face. Near vertical slopes can be used in hard fresh granite, particularly if controlled blasting methods are used. Slope design for granitic rock between

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the two extremes should take into consideration the jointing patterns in the rock. Rockfall is a problem in designing slopes in granitic rock.

(3) Slope designs in Mesozoic age rock such as the Franciscan or Upper Cretaceous formations, need to consider the orientation of bedding of these rocks as well as the joint patterns or fault zones that may be present.

(4) There is a wide variation in the competency of the Tertiary rocks within the Transverse Ranges. Careful exploration is usually required to design slopes in these rocks. The attitude of bedding is extremely important in designing slopes in these rocks.

b. Embankment Foundations

Careful consideration must be given to foundation condition when the embankment is placed on a dip slope. Sometimes within the Transverse Ranges the additional weight of the embankment on a dip slope will start a bedding plane slide.

c. Materials Sources

Sources of aggregate for portland cement concrete and asphalt concrete are limited in the Transverse Ranges. Stream gravels in the western part of the province usually contain siliceous shales that are potentially reactive with the alkalis in cement. Commercial aggregate sources in the central and eastern portion of the province are being encroached upon by urban development. Rock slope protection material of adequate quality is also difficult to obtain in the Transverse Ranges. Considerable exploration is usually required to find a source for this material.

d. Excavation Characteristics

(1) The older metamorphic rocks and the granitic rocks are often deeply weathered, and shallow cuts can be made in these materials without blasting. The deeper cuts, however, will probably require blasting.

(2) As has been stated previously, there is a wide variety in the competency of the sedimentary rocks within the Transverse Ranges. Sometimes harder massive sedimentary rocks are interbedded with softer rocks, thus posing a difficult excavation problem.

e. Landslide Problems

Numerous landslides exist throughout the Transverse Ranges. The Tertiary age rocks are especially prone to sliding. Bedding plane slides are quite common. When large masses of rock have slid intact these existing slides may be difficult to identify.

f. Seismic Activity

Because a number of active faults exist within the Transverse Ranges consideration must be given to seismic forces in engineering design. Surface rupture along one of the faults would adversely affect any engineering structure crossing it. Usually it is preferable to cross active faults on embankments rather than structures because of the ease of repair. Other earthquake damage is usually restricted to some rockfall in cut areas and settlement in embankment areas. In extremely active seismic areas cut and embankment slopes may require less steep designs than would normally be used.

H. Mojave Desert, Colorado Desert, and Basin Ranges

1. Geology

a. The Mojave Desert, the Colorado Desert, and the Basin Ranges are being considered together because all three of these geomorphic provinces have many similar features and engineering considerations for the three provinces are also similar. These three regions are characterized by mountain ranges with intervening alluvial valleys. Desert climatic conditions, interior drainage and saline lakes are characteristic of the entire area.

b. Most of the mountain ranges have been formed by block faulting. Very old metamorphic rocks are exposed in many of these ranges. These metamorphic rocks consist of gneiss, schist, quartzite, marble, and others. Granitic rocks have intruded these older rocks and are exposed extensively throughout the provinces.

c. During the Cenozoic era continental sedimentary deposits and volcanic rocks were

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deposited on the older rocks. The volcanic rocks include lava flows, breccias and tuffs. Their composition ranges from rhyolite through andesite to basalt.

d. Most of the intermontane valleys have contained or contain lakes. These desert valleys contain alluvial sediments, lake bed clays, and saline deposits. In a few places they contain local basalt flows and cinder cones. Alluvial fans are a characteristic feature in these desert regions. Sand dunes are present in some locations.

e. The faulting that has formed many of the features of these provinces has occurred during Cenozoic time. Some of the faults are known to be active while others within the region may or may not be active.

2. Engineering Significance

a. Cut-Slope Design

(1) Generally cut slopes in desert areas can be somewhat steeper than slope designs for similar rock in areas with more rainfall.

(2) The attitude of the foliation, joint patterns, and faulting will greatly affect the stability of cut slopes in the older metamorphic rocks.

(3) The degree of weathering varies greatly in granitic rocks. Low cuts in decomposed granite can be designed on steep slopes. Higher cuts in decomposed granite will need a flatter design to avoid a total failure of the slope face. Near vertical slopes can be used in hard fresh granite particularly if controlled blasting methods are used. Slope designs for granitic rock between the two extremes should take into consideration the jointing patterns in the rock. Rockfall is a problem in designing slopes in granitic rock.

(4) Moderately steep slope designs can be used in Cenozoic sediments for cuts of 30 ft or less.

(5) Slope designs in Cenozoic volcanic rocks will vary. Steep slope designs can be used in volcanic flow rocks such as basalt and andesite. Somewhat flatter slopes will be required for cuts in tuff, ash and cinders. Cut slopes with interbedded flow rocks and pyroclastic rocks may require a variable slope design. Rockfall is often a problem in this type of material.

b. Embankment Foundations

For the most part, embankment foundations within these desert regions are satisfactory. However, when embankments are to be built over lake deposits problems may arise. Sometimes considerable settlement may occur.

c. Materials Sources

Generally satisfactory materials can be produced from most of the desert areas; however, aggregates for portland cement concrete and asphalt concrete are scarce in the Colorado Desert Area. Some materials within these desert areas may include volcanic glass or opal. These materials are potentially reactive with the alkalis in cement. Rock slope protection material can be obtained from selected older metamorphic rocks, selected granitic rocks, or volcanic flow rocks.

d. Excavation Characteristics

Blasting may be required in the older metamorphic rocks and in the granitic rocks. Most of the Cenozoic sediments will not require blasting. Some of the Cenozoic volcanic rocks will require blasting and some will not.

e. Economic Mineral Deposits and Mining Claims

Mining operations have been conducted throughout the desert regions. Economic mineral deposits and mining claims that are located within the proposed right of way should be evaluated at an early stage.

f. Sand Dunes

Special consideration should be given to areas of sand dunes. Often these dunes are drifting and will encroach upon a highway. Unless the areas are avoided by alignment, protective measures may be needed for the road.

g. Geothermal Areas

Geothermal areas exist near the Salton Sea and in the Long Valley-Casa Diablo area.

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Steam escaping from a geothermal area can cause special hazards to a highway.

h. Seismic Activity

The extensive faulting that has formed many of the features of these desert areas is still continuing. Active faults include the Garlock fault, San Andreas fault, San Jacinto fault, and portions of the Sierra Nevada fault. Other faults within the region may also be active. Consideration must be given to seismic forces generated by these active faults.

I. Peninsular Ranges

1. Geology

a. The Peninsular Ranges are bounded on the east by the Colorado Desert and on the north by the Transverse Ranges. The average annual rainfall for much of the province is 10 to 15 in. per year. However, annual rainfalls as high as 30 in. per year are recorded at some of the higher elevations.

b. The geologic history of the province is somewhat similar to that of the Sierra Nevada province. During Paleozoic and early Mesozoic time marine sedimentary and volcanic rocks accumulated. Then in mid-Cretaceous time these rocks were intruded and metamorphosed by granitic rocks. Extensive erosion followed the mountain building caused by the intrusions. As a result granitic rock is exposed extensively in this province.

c. The oldest metamorphic rocks include schist, amphibolite, quartzite, gneiss, and crystalline limestone. A second group of metamorphic rocks predate the granitic intrusion. They are exposed principally on the western side of the province. They consist of slates and argillites of the Bedford Canyon formation and volcanic rocks of the Santiago Peak volcanics. Sedimentary and volcanic rocks that are younger than the granitic intrusion and the metamorphic rocks have been deposited on the eroded surface of these older rocks. From Oceanside south to the Mexican Border the rocks consist of poorly consolidated, flat lying units of sandstone, shale and conglomerate. North of Oceanside these rocks cover a much wider area. Younger volcanic rocks occur near Jacumba, south of Elsinore, and in the Los Angeles basin.

d. Several large northwest trending faults tend to subdivide the province. The San Jacinto, Agua Caliente and Elsinore faults are considered to be active faults.

2. Engineering Significance

a. Cut-Slope Design

(1) The attitude of the foliation, joint patterns, and faulting will affect the stability of cut slopes in the older metamorphic rocks.

(2) The degree of weathering varies greatly in the granitic rocks. Low cuts in decomposed granite can be designed on steep slopes. Higher cuts in decomposed granite will need a flatter design to avoid a failure of the slope face. Near vertical slopes can be used in hard fresh granite particularly if controlled blasting methods are used. Slope designs for granitic rocks between the two extremes should take into consideration the jointing patterns in the rock. Rockfall is a problem in designing slopes in granitic rock.

(3) Consideration should be given to the consolidation of the material and to the attitude of the bedding when designing cut slopes in the younger sedimentary rocks.

(4) The younger volcanic rocks are generally quite broken. Rockfall will be a problem of cut slopes in these rocks.

b. Embankment Foundations

Many lagoons exist along the coast of this province. In some instances a hard thin crust has formed over the lagoon. The soft mud and peat found in these lagoons are unsatisfactory embankment foundations. Special studies are usually needed to design and construct embankments over these areas.

c. Materials Sources

Aggregate for portland cement concrete and asphalt concrete can be found in this province, although locally some difficulty may be experienced in mountain regions. Selected areas of granitic rock provide satisfactory rock slope protection material.

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d. Excavation Characteristics

Blasting may be required in the older metamorphic and granitic rocks. However, there is considerable variation in the competency of these rocks and exploration work probably will be required to determine excavation characteristics. Most of the younger sedimentary and volcanic rocks will not require blasting for excavation.

e. Landslide Problems

Naturally occurring landslides are not numerous in the Peninsular Ranges.

f. Economic Mineral Deposits or Mining Claims

Some mining activity has taken place in the Peninsular Ranges. Extensive utilization of nonmetallic materials has occurred. Economic mineral deposits and mining claims that are located within the proposed right of way should be evaluated in an early stage, preferably during planning.

g. Seismic Activity

There are active faults within this province and consideration must be given to seismic forces generated by these active faults.

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III. Properties of Soils

A. General

Those properties of soils which serve mainly for their identification and classification are commonly called "Index" properties. These properties are only indirectly related to the important physical engineering properties. Index properties include gradation and particle shape, soil consistency, porosity and void ratio, specific gravity, water content, and density or unit weight. The engineering properties of most concern are strength, compressibility, and permeability. The important properties of soils are described below, index properties being discussed first, and then engineering properties.

B. Index Properties of Soils

1. Gradation and Particle Shape

a. Grains larger than about the 200-mesh size can be seen with the naked eye or with the aid of a hand lense. They constitute the coarse fraction of soils. Arbitrarily, the coarser fractions of soils are called gravel, sand, or silt, depending upon their size. While there is no universally accepted definition of size classes and names, those adopted by the U. S. Bureau of Soils in about 1890 are widely accepted. These are as follows:

Particle or Sieve Size	Soil Name
3" - 3/4"	Coarse Gravel
3/4" - No. 4 sieve	Fine Gravel
No. 4 - No. 10	Coarse Sand
No. 10 - No. 40	Medium Sand
No. 40 - No. 200	Fine Sand
0.05 mm - 0.005 mm	Silt
Finer than 0.005 mm	Clay

b. In classifying soils according to grain size, one should bear in mind that the physical properties of the finest soil fractions depend largely on factors other than grain size. It follows that a high degree of refinement in determining the "effective" diameter of very fine fractions is usually not warranted. In employing the above classes, one should keep in mind that the words "silt" and "clay" relate only to to grain size and have no bearing on plastic behavior. When describing soils on this basis, the terms "silt" and "clay" are commonly combined with the word "size"; for example, particles are "clay size", or "silt size." Fine rock flour, for example, exhibits all the common properties of silt but may be finer than 0.005 mm and thus classify as clay size. On the other hand, some clays may consist mostly of particles larger than 0.005 mm, but contain small quantities of extremely fine, colloidal particles, and exhibit definite plasticity associated with true clays.

c. Grain-size distribution of the coarser fractions is determined by shaking the particles through a nest of sieves. The separation by sieve size is called mechanical analysis. The California Division of Highways materials laboratory and many others use the U. S. Standard sieves. The soil which passes through a given sieve size is described by the word "minus," or "passing." Thus, an aggregate is spoken of as having 10% "minus 200" material, or 10% "passing 200" material. The smaller grain sizes are usually separated by wet methods. The California Division of Highways uses the hydrometer method for separating soil fractions finer than the 200-mesh screen. In the wet method, a quantity of the soil is placed in a 1000-cc graduated cylinder. After it is thoroughly shaken to suspend all particles, specific gravity readings are taken from time to time to determine the density of the suspension. Large particles fall out of suspension faster than small particles. By application of Stokes' Law the relationship between grain size, time, and density can be calculated. It is thus possible to determine the amount of particles that have left a given suspension at various time intervals, and indirectly convert density determinations into a grain-size distribution. The resulting mechanical analysis obtained by wet and dry methods is tabulated on standard report forms. If the problem warrants a detailed study of

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grain-size distribution, the data may be plotted and gradation curves drawn. The most useful kind of grain-size distribution plot is one in which the "percent passing" is plotted on an arithmetic scale and the grain sizes on a logarithmic scale.

d. Soil particles may be basically round (bulky); they may be flat (flakey or scalelike); or they may be elongated. Rounded particles, no matter how fine, do not make a soil plastic. Sand grains are bulky particles. They may be angular, subangular, rounded, or subrounded, depending upon the degree of wear they have experienced. Coarse materials, such as sand, when consisting chiefly of quartz, are relatively incompressible. Sometimes they contain large percentages of mica flakes. This makes them relatively springy and yielding.

e. When soils are described according to grain size, if most of the grains are of about the same size, the soils are said to be uniformly graded. On the other hand, if the soil contains a wide range of sizes and the curve on the semilog plot is relatively smooth, the soil is said to be well-graded.

f. Fine soils may be made up of bulky particles or scalelike particles, depending upon the mineral substance from which they are formed. Clay minerals are composed of microscopically and submicroscopically thin sheets; hence, clays usually contain large percentages of scalelike particles. As a rule, the finer the soil, the larger the percentage of flakey constituents.

g. The behavior of very fine soils is governed to a large measure by surface phenomena. As soil grains become smaller and smaller in size, forces that are insignificant in coarse-grained soils become very important. The properties of absorbed water films can be significant for fine-grained particles. Negative electric charges on the surface also can become very important. Ions and cations become significant. Various chemical treatments of soils have been devised that operate on the principle that changes in the electrical charges can change the properties of very fine grained soils.

2. Soil Consistency

a. The physical properties of most fine-grained soils are influenced greatly by the water content. The consistency of a clay may be soft, firm, stiff, or hard, depending upon the amount of water it contains. A direct, quantitative test of the consistency of undisturbed soils is the unconfined compressive strength. The California Division of Highways classifies the consistency of clays in terms of unconfined compressive strength as follows:

Consistency	Unconfined Compressive Strength (kg per sq cm)*
Very soft	Less than 0.3
Soft	0.3 - 0.7
Firm	0.7 - 1.4
Stiff	1.4 - 2.8
Hard	Over 2.8

*Approx. equivalent to tons per sq ft

b. A clay deposit that is being produced by sedimentation at the bottom of a lake or bay has at the beginning the consistency of liquid mud. In this state it will readily flow if its support is removed or a load is placed upon it. As pressure is applied by greater thickness of overlying material the clay will become denser and stiffer and may be described as plastic. Upon further loading or drying the clay will lose its plastic tendencies and reach a semisolid state. Upon further drying, the clay will shrink to the point where it undergoes no further volume change. This point is the transition to the solid state.

c. When clays are remolded over a wide range of water content, the consistency can vary over much the same range described above. The range of water contents that accompany changes in the consistency of fine-grained soils tell something about other properties of the soil. If, for example, a relatively minor reduction in water content reduces a soil from the liquid to the plastic or semisolid states, one can infer that the soil is probably composed of bulky particles, and is relatively nonplastic and noncompressible. In contrast, if a considerable quantity of water must be added to change a soil from the plastic to the liquid state, the soil probably contains a high percentage of very fine scalelike particles or organic matter, and is highly compressible.

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d. The Atterberg "limits" are measured by the water content that corresponds to the boundaries between several arbitrary states of consistency progressing from liquid to solid. These limits tests are performed on that portion of the material that passes the No. 40 sieve.

e. The liquid limit is the water content in percent of the dry weight at which the soil first shows a small but definite shearing strength with a reduction in water content. In the reverse direction, it is the water content at which the soil mass just starts to become fluid.

f. The plastic limit is the water content at which the soil mass ceases to be plastic and becomes brittle or crumbly when rolled into threads one-eighth inch in diameter. The plastic limit is always lower than the liquid limit.

g. The plasticity index is the numerical difference between the liquid and plastic limits and represents the range of moisture over which the soil is plastic. The plasticity index, in combination with the liquid limit, indicates the sensitivity of soils to changes in moisture.

h. Soil consistency as an index of engineering properties of soils is applied only to fine-grained soils, or the fine-grained fractions of aggregates. In the Division of Highways, the Sand Equivalent test was developed as a means of determining the activity of the fine fractions of commercial aggregates. It is a rapid, easy to perform test that has been finding wide acceptance among highway engineers.

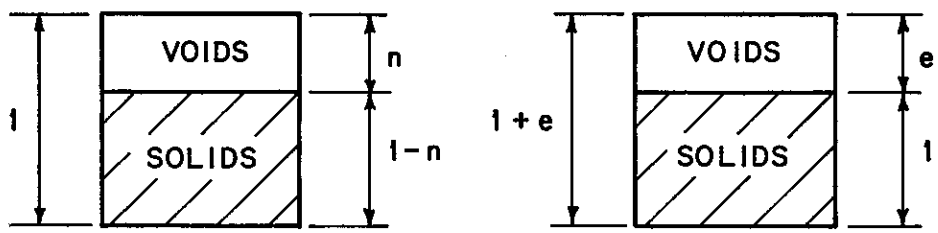
3. Porosity and Void Ratio

a. An important feature of both natural and compacted soils is the amount of solids contained in a given volume, or conversely, the amount of voids. Many of the computations in soil mechanics are greatly simplified by considering the voids rather than the solids. Two expressions are commonly used for defining the void or pore spaces: (1) The porosity, n, is the percentage of the void spaces in relation to the total volume of the mass. Thus, in Fig. III-1a if the total volume is expressed as 1, and the void spaces as n, the porosity = n/1 = n; (2) The void ratio, e, is defined as the ratio of the void spaces to the volume occupied by the solid particles. This definition is clarified by Fig. III-1b, in which the volume of the void spaces is expressed as e and the volume of the solids as 1. This makes the total volume 1+e. By the definition, e = e/1 = e. This definition is one of the most useful tools of soil mechanics. The two terms are related to each other, as follows:

$$n = \frac{\text{vol. of voids}}{\text{total volume}} = \frac{e}{1+e} \qquad \text{(III-1)}$$

$$e = \frac{\text{vol. of voids}}{\text{vol. of solids}} = \frac{n}{1-n} \qquad \text{(III-2)}$$

b. Porosity and void ratio are measures of the looseness or compactness of soil structure. In soils that are composed of mixtures of bulky and flakey particles, with organic matter, porosity and void ratio can vary over wide limits. After an engineer has examined soils in various states and has compared their state with their porosity and void ratio as determined by test, these terms begin to convey a definite understanding to him. They also have definite value in numerous soil mechanics computations.



(a) Definition of Porosity (b) Definition of Void Ratio

Fig. III-1 — Porosity and void ratio defined.

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4. Specific Gravity

a. The computation of soil weight is often necessary in soil mechanics studies. Weight and volume are correlated by the specific gravity of the substances in a soil. Specific gravity is defined as the ratio between the unit weight of a substance and the unit weight of water at 4 deg C.

b. The specific gravity of an earth material can vary somewhat, depending upon the manner in which the test is performed. The true or absolute specific gravity of the mineral constituents is determined on finely pulverized material so that the impermeable pores or voids in the coarser grains are eliminated. The specific gravity determined in this way is the highest that can be determined for a given soil. The apparent specific gravity is determined on the soil particles as they occur naturally. Any voids in the interior that cannot be filled with water are called impermeable voids. Whenever any impermeable voids are present, the apparent specific gravity is smaller than the true specific gravity, due to the buoyancy effect of the voids. This test probably comes closest to duplicating the true condition; hence, the apparent specific gravity is used in most soil mechanics computations involving the specific gravity of the mineral constituents.

c. Bulk Specific Gravity (saturated surface dry) is the specific gravity with the surface or "permeable" voids filled with water. It is smaller than the apparent if any permeable voids are present. The bulk specific gravity (saturated surface dry) is used primarily in concrete mix calculations.

d. Specific gravity is a general index of durability. Materials with high specific gravities as a rule are tougher and more durable than materials of low specific gravity. Specific gravity also gives a clue to other properties. For example, light specific gravity materials often are more susceptible to slope erosion than are heavier materials.

5. Water Content or "Natural Moisture Content"

a. The natural water content of a soil is one of the most useful index properties of a soil. Soil moisture is very important in soil behavior as it is a primary factor causing variations in soil properties. The numerical values of water contents in themselves are not direct measures of engineering properties, but after comparing soils of various consistencies with known water contents the water content becomes significant to the engineer. Moisture is the principle factor subject to change in soils, either naturally, or with the help of man. When used as a construction material for embankments the natural water content may govern the length of construction season and even the suitability for use. Materials that naturally are very dry may require the addition of considerable water to permit compaction. Conversely materials that are much wetter than the optimum may require costly time consuming drying.

b. Water content usually is expressed as the ratio of the weight of water in a soil mass to the weight of solids, because the weight of solids in a given mass remain unchanged. Water content, w, is thus defined as:

$$w = \frac{\text{Weight of water}}{\text{Weight of solids}} \qquad \text{(III-3)}$$

c. The water content as defined here is based upon the amount of water than can be driven off by drying to a given temperature. This water occurs in soil as free moisture and capillary moisture. The free water that occurs in coarse-grained soils can drain downward under the force of gravity. Capillary water is the water that is held in fine-grained soils by the surface tension of water. It resists the force of gravity and remains in the pore spaces between the soil grains. It is reduced, but may not be entirely eliminated, by evaporation at normal air temperatures. It is largely removed in the test procedure for determining water content. Another form of water, "hygroscopic moisture," occurs in soil in the form of thin moisture films coating soil particles. This water is ADsorbed, not ABsorbed. It does not enter soil particles. Hygroscopic moisture is held with tremendous force and cannot be removed by evaporation at normal air temperatures.

6. Density or Unit Weight

a. Another very useful property of soils is the density or the weight of a unit volume of

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the soil. Density can be expressed in a number of ways. The more important densities are: (1) natural (in-place) density, (2) moist density, (3) saturated density, (4) dry density, (5) buoyed density, and (6) maximum test density. The natural (in-place) density and the maximum test density represent a condition of occurrence. The other terms refer to a specific definition of density and are applicable to any condition of occurrence one wishes to apply them to. Thus, the natural (in-place) density can be determined on the moist, saturated, dry, or buoyed basis.

b. Unit weights, water contents, and void ratios (or porosities) are all related. The interrelationships can be computed in several ways. A very simple way that does not require memorizing any formulas is to think in terms of an elemental volume with units expressed in the metric system. Basic relationships become very simple. As unit weights are expressed as specific gravities in the metric system, the weight in pounds per cubic foot is easily obtained by multiplying the specific gravity by the weight of one cubic foot of water (62.4 lb per cu ft).

c. Completely Saturated Condition - The 100% saturated case is shown on Fig. III-2. An elemental volume is represented. Volumes of soil and water are shown at the left of the diagram. Weights of the constituents (soil and water) are shown at the right. This diagram represents all the basic relationships needed for unit weight and moisture content computations for fully saturated soils. By definition,

$$e = \frac{\text{Vol. of Voids}}{\text{Vol. of Solids}} = \frac{e}{1} = e$$

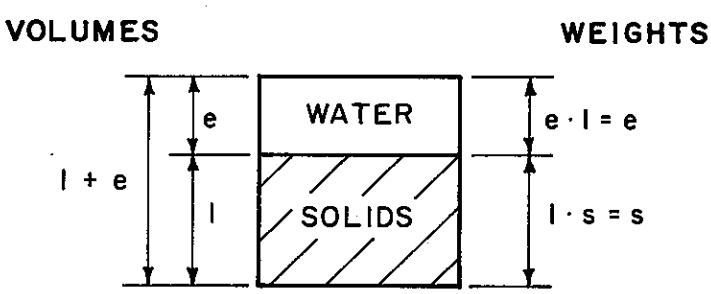

Fig. III-2 — Volume-weight relationships for 100% saturation.

This establishes the volume of the solid and the water phases, as shown at the left, the total volume being 1 + e. The weight of e volume of water equals e · 1 = e units. The weight of unit volume of solids equals 1 · s = s units. The total weight is (s + e) units. From this diagram, saturated water content, and unit weights are determined, as follows:

(1) Water Content

$$\text{Water Content} = \frac{\text{Wt. of water}}{\text{Wt. of solids}} = w$$

$$w = \frac{e}{s} \qquad \text{(III-4)}$$

(2) Dry Unit Weight

$$\text{Dry Unit Weight} = \frac{\text{Wt. of solids}}{\text{Total Volume}} = \gamma_{dry}$$

$$\gamma_{dry} = \frac{s}{1 + e} \qquad \text{(III-5)}$$

(3) Saturated Unit Weight

$$\text{Saturated Unit Weight} = \frac{\text{Wt. of solids + Wt. of water}}{\text{Total Volume}} = \gamma_{sat.}$$

$$\gamma_{sat.} = \frac{s + e}{1 + e} \qquad \text{(III-6)}$$

(4) Buoyed Unit Weight

$$\text{Buoyed Unit Weight} = \frac{\text{Wt. of solids - buoyancy}}{\text{Total Volume}} = \gamma_{buoyed}$$

$$\gamma_{buoyed} = \frac{s - 1}{1 + e} \qquad \text{(III-7)}$$

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Example: Specific gravity, $s = 2.67$
Void ratio $e = 1.0$

(1) Water content, $w = \frac{e}{s} = \frac{1.0}{2.67} = 0.37$

In percent, $w = 0.37\ (100) = 37\%$

(2) Dry Unit Weight, $\gamma_{dry} = \frac{s}{1+e} = \frac{2.67}{1+1} = 1.33$

In lb per cu ft, $\gamma_{dry} = 1.33\ (62.4) = 83.0$ lb/cu ft.

(3) Saturated Unit Weight, $\gamma_{sat} = \frac{s+e}{1+e} = \frac{2.67+1}{1+1} = \frac{3.67}{2} = 1.83$

In lb per cu ft, $\gamma_{sat} = 1.83\ (62.4) = 114.2$ lb/cu ft.

(4) Buoyed Unit Weight, $\gamma_{buoyed} = \frac{s-1}{1+e} = \frac{2.67-1}{1+1} = \frac{1.67}{2} = 0.83$

In lb per cu ft, $\gamma_{buoyed} = 0.83\ (62.4) = 51.8$ lb/cu ft.

Check of 4:

Buoyed unit weight = (Sat. unit weight) - (Unit weight of water)

$= 114.2 - 62.4 = 51.8$ (check)

d. For partially saturated soils a similar procedure can be used by making allowance for the space occupied by the air or gas, but considering that the air or gas contributes no weight.

7. Identification and Classification of Soils

a. Many soils engineers believe, or have at one time believed, that most of their worries will be over if they can identify and classify soils by some satisfactory system. As they gain experience they usually realize that classifications, useful though they may be, do not provide answers to all soils problems. The separating or classifying of soils into widely used groups is useful in giving engineers a common language that can be used in conveying information to fellow engineers. To be of maximum value the groups should have distinctly different engineering properties. Systems that describe soils in very general terms such as "fine sand," "silt," or "sandy gravel" are necessarily limited as mechanisms for conveying information about the probable behavior of soils in engineering construction.

b. In the California Division of Highways, foundation soils are classified according to grain size and physical condition and on all important projects significant laboratory test data are summarized on boring profiles. Designs and construction procedures for projects involving questionable soil conditions are almost always developed with the aid of tests that measure engineering properties such as strength, consolidation and permeability.

c. The California Division of Highways classifies soils by a modified procedure which takes into account many factors which can be observed. The scope of this procedure is illustrated by the outline labeled "Field Classification of Soils" found in the appendix.

C. Engineering Properties of Soils

1. General

a. The index properties aid in identifying and classifying soils. They also give an indication of probable engineering behavior. Sometimes the index tests can provide sufficient information for the development of safe and economical engineering designs for earthwork projects.

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They may demonstrate, for example, that a certain foundation has superior properties for an intended construction, and in such cases more elaborate testing may not be needed. On the other hand, they may indicate that a soil is so weak and compressible to such a great depth that a particular project is not at all feasible. In the majority of cases, however, the index tests do not define the solution so definitely as these two extremes and it is necessary to test the engineering properties of the soil, and adapt the design and construction to these properties.

b. Strength, compressibility and permeability are the primary engineering properties that influence the behavior of large earthwork projects such as cut slopes, foundations, earth dams, etc.

c. The engineering properties of soil and rock, as construction materials, are probably more variable than those of any other construction material. The most variable property of earth materials is permeability, which probably is the most variable property of any construction material. Permeability in soils is the ability to transmit water. It can vary from 100,000 ft a day or more for coarse gravel to 0.000001 ft a day for highly plastic fat clay, or a range of around 10 billion times! Since permeability influences other engineering properties, fairly wide variations in strength and compressibility should be expected.

2. Shear Strength

a. Probably the most difficult and controversial subjects of soil mechanics are those relating to the shear strength of fine-grained soils. The difficulties arise because of the complex nature of soils. Most common construction materials have a comparatively uniform structure throughout. Concrete has a structure of aggregate particles held together by a cement matrix, and steel has an intricate crystalline structure, but these materials are relatively uniform from one point in a structure to another. Their properties are to a large degree under the control of the designer and builder. Earth materials, on the other hand, usually vary substantially over small distances in a mass, and their structure is a mineral framework interspersed with solidified water, free water, gas, and sometimes organic matter in various degrees of decomposition. The whole may be found in any stage of compression and cementation. Identical compositions may have widely differing engineering properties, depending upon their "history."

b. In 1776, a French scientist, Coulomb, observed that the shearing strength of most soils is made up of two parts, one that depends upon the pressure on the shear plane (friction) and one that is independent of that pressure (cohesion). The shearing strength s, is expressed as:

$$s = c + \sigma \tan \phi \qquad \text{(III-8)}$$

where σ is the pressure on the shear plane, $\tan \phi$ is the coefficient of friction, and c is the cohesion.

c. Sand and gravel of other cohesionless soils have little or no cohesion; hence, their shear strength is:

$$s = \sigma \tan \phi \qquad \text{(III-9)}$$

d. Saturated impervious "cohesive" clays respond very slowly to externally applied pressures, and their strength sometimes is expressed as:

$$s = c \qquad \text{(III-10)}$$

e. The terms, noncohesive or cohesionless and cohesive are commonly used in discussing soils. As used above in Equation III-10 the term cohesion represents the apparent cohesion. True cohesion is usually defined as the strength a soil or rock has acquired through application of heat, pressure, or chemical cementation. A soil or rock retains this strength, whether wet or dry. True cohesion is a property of highly compressed clays, hardpan, sandstone, etc. When fine-grained soils are removed from their natural place in the earth the earth pressures are removed and the soil, like a compressed spring, tends to expand. This tendency to expand is restricted by a suction in the soil water that is caused by surface tension of the water as exhibited by capillarity. Any cohesion that is retained by capillary action is called apparent cohesion. This strength disappears if the capillary forces are lost.

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f. When loads are applied to soil masses the soil grains are pressed more tightly against each other, and the volume decreases. The bulky grains of sands and gravels adjust to increased pressures with very little volume change, but soils that are composed of fine grains, particularly with large percentages of flat scalelike particles, loosely arranged, undergo large volume changes under increased loads. If the pore spaces are filled with water, and the soil is highly impervious, the soil particles cannot be pressed closer together until some of the water is squeezed out. Under this condition, which occurs in soft muds, peats, and impervious saturated clays, the added load produces pressures in the pore water. Any pressure that goes to the pore water is reacting against the added load, neutralizing part of the pressure on the shear planes. Not until the excess water drains out can the soil grains press more firmly against each other to carry the added load. This process of flow of water out of saturated clays and other fine-grained soils, and the accompanying decrease in volume is called "consolidation." Deep beds of compressible clays and peats may continue to settle for many decades after loads have been placed on them. The delayed settlement causes bumps at bridge ends, and the uneven ride that often develops several years after roads are built over swamps. These effects of delayed consolidation are highly undesirable. The slow rate at which the strength of these materials is increased is even more critical. Soft muds and clays that occur in foundations over which embankment is placed may at the end of several months of filling operations have little more strength than they possessed originally. When this is the case, the original in-place strength is used in analyzing the embankment cross-section for stability. The unconfined compression test and the vane borer test are widely used for determining the in-place strengths of soft clay soils. When the construction schedule allows a slow controlled rate of loading, the stability is calculated with allowance made for the gain in strength that can be expected over the slow construction period. Sometimes the increase in rate of consolidation by vertical sand drains is analyzed to see if this method is feasible. In these studies, the soil strength is estimated from triaxial shear tests on soil samples that have been consolidated at various pressures.

g. The strength of highly impervious soils may vary so little over a normal construction period that the only safe method may be to rely only on the in-place strength, or in-place cohesion of the soil. This strength which is immediately available is usually referred to as the apparent strength. When the in-place strength of a relatively weak, saturated soil is expressed in terms of shear coefficients, c and ϕ, these coefficients are apparent coefficient which may bear little resemblance to the true shear coefficients which pertain to the soil when it is fully consolidated. Research and careful testing by a number of investigators have shown that the "true" coefficient of friction of soils decreases with increasing plasticity. The following average values are representative of tests run on clay soils from a number of countries. The tests were on undisturbed samples that had been fully consolidated under their overburden loads.

Plasticity as Measured by P. I.	The "true" Coeff. of Friction, ϕ
5	35 deg.
10	33 deg.
20	31 deg.
40	27 deg.
60	24 deg.
80	22 deg.

h. One should be careful to keep in mind that the "true" coefficient of friction of soils, as typified by the above average data, represent the possible strengths under conditions of complete consolidation.

3. Compressibility

a. General - The term compressibility refers to reduction in the volume of soils due to increased loads. Changes in the volume of soil masses cause a great many of the problems of building high quality highways. A number of terms have widespread use in describing volume changes of soil. The meanings of some of the important terms are as follows:

Consolidation - refers to volume changes that occur with the passage of time, usually concurrent with the squeezing of water out of the pores of fine-grained soils.

Compression means volume changes caused by the application of external loads.

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Shrinkage is volume reduction brought about by capillary forces that compress soils during drying.

Compaction relates to the densifying of soils by artificial means such as tamping, rolling, vibrating, or other temporary actions.

b. In contrast to the above terms, rebound, swell, expansion, heave, and a number of other terms pertain to the loosening of soil structure.

c. Usually, phenomena associated with compressibility are caused by changes in the volume of the voids and to little or no extent upon changes in the solid particles. Soils that have voids which are largely filled with air usually compress immediately without appreciable delayed consolidation, but there are some notable exceptions. Windblown loess usually has a relatively loose structure, very lightly cemented. When loess soils are loaded without changes in water content they generally show some immediate compression which may be relatively moderate. If they subsequently become saturated they may settle excessively and destroy the structure that was placed upon them. In California, extensive deposits of loose-structured gravelly silty clay occur along portions of the new "Westside" Freeway alignment. These deposits evidently were laid down as mud flows during short periods of heavy rains. These mud flows have built up in thin layers to total thicknesses of 100 ft or more, without any appreciable penetration of water after they were placed. When these soils become saturated by accidental flooding, irrigation by extended or repeated floodings, etc., their structure is weakened and extensive settlement results. Settlements of 15 to 20 ft have been observed. Obviously engineering construction in these areas is very hazardous.

d. Another type of land settlement is that caused by lowering the permanent ground water level by reclamation or pumping. When the water level is lowered in any soil its effective weight on all layers below it is increased from its submerged weight to its damp or moist weight. If the water table is drawn down several hundred feet (as in the San Joaquin Valley) the added weight may cause very substantial settlement in relatively noncompressible deposits.

e. Settlement as encountered in highway construction usually is associated with the placement of embankments on compressible soil formations.

4. Permeability

a. The property of a soil that permits water to flow through it is called permeability; the process by which water moves through soils is called percolation. Engineers who design earth dams and levees are interested in permeability in its effect on the quantity of water that leaks through dams or levees, or in its effect on hydrostatic pressures effecting the safety of these projects. Highway designers are interested in permeability for a number of different reasons, of which the following are probably the most important:

(1) Its effect on the rate of consolidation and increase in strength of weak, compressible mud and peat foundations.

(2) Its effect on the quantity of ground water that leaks from soil and rock formations into roadbeds or embankment foundations.

(3) Its effect on the ability of "pervious" filter materials to remove water.

(4) Its effect on the stability of cut slopes.

b. The factor that relates permeability with seepage conditions is called the coefficient of permeability.

c. The coefficient of permeability, k, as used in civil engineering, is defined by the following expressions, which commonly is called Darcy's Law:

$$v = ki \qquad \text{(III-11)}$$

or:

$$k = \frac{v}{i} \qquad \text{(III-11a)}$$

In these expressions, v is the discharge velocity, and i the hydraulic gradient. The coefficient of permeability, k, from a practical standpoint, is a constant, although the true coefficient depends to a minor degree upon the density and viscosity of the percolating liquid.

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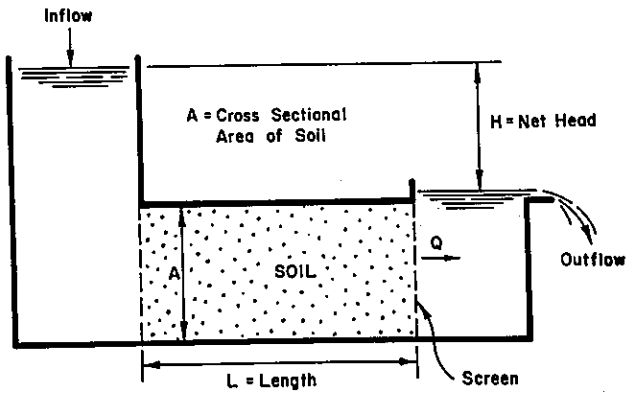

Fig. III-3 — Seepage through soil.

d. The coefficient of permeability usually is determined directly from field or laboratory tests in which seepage conditions are controlled. Equation III-11 relates the discharge velocity (the average velocity in a cross-section) to permeability and hydraulic gradient. A well-established hydraulic principle tells us that quantity of flow is equal to the average velocity times the area, or:

$$Q = vA \qquad \text{(III-12)}$$

If we substitute ki for v (from Equation III-11), we obtain:

$$Q = kiA \qquad \text{(III-13)}$$

This equation is another expression of Darcy's Law and provides the basis for experimental determination of permeability, and for theoretical seepage calculations.

e. If water is forced through soil in a simple permeameter, as shown in Fig. III-3, and the quantity of seepage per unit time, head, H, length of sample, L, and cross-section area, A, are known, the following relationships apply:

$$Q = k \frac{H}{L} A \qquad \text{(III-14)}$$

or by transposing terms:

$$k = Q \frac{L}{HA} \qquad \text{(III-15)}$$

f. The coefficient of permeability can be determined from Equation III-15, in a permeameter with a fixed head. It may be determined in the laboratory with a falling head permeameter, or in the field by pumping into or out of wells or pits, with formulas modified to fit the conditions of the test. The permeability of clays and the other fine-grained soils can be calculated indirectly from measurements taken during laboratory consolidation tests. Attempts sometimes are made to estimate permeability from grain size distribution or other properties; however, these methods are subject to large errors and should be recognized.

g. Coefficient of permeability is one of the most widely used soil properties in soil mechanics and soils engineering. In highway design it enters into studies of the length of time required for the construction of embankments over marshes and swamps, and of the design cross-section for fills on weak foundations. It provides the means for designing drainage systems for roadways that must be depressed beneath the normal groundwater level.

h. Since permeability is subject to such wide variations, computations in which it enters must always be regarded as approximations of expected behavior.

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IV. Subsurface Explorations

Subsurface explorations require considerable judgment in order to analyze foundation conditions from limited factual data. The engineer who lays out the exploration work must decide which areas may be troublesome or questionable for purposes of the proposed project and concentrate the investigation efforts in those areas. The selected methods of exploration will vary with the purpose. The object is to furnish a record of essential information and not omit important items. In a foundation for a high embankment, a thin layer of soft clay that isn't recorded might cause a failure. In a seepage investigation, a thin layer of pervious gravel may be extremely important. Obviously then, placement of the borings, frequency of sampling, and maximum depth sampled are most important to the success of any foundation exploration. A most efficient program is obtained with the assistance of the engineer who is to design the project.

A. Recognizing a Foundation Problem

Existing field conditions frequently camouflage troublesome foundation areas which would require special design or treatment in order to insure construction of a stable roadway.

1. Cut-Slope Stability

a. The forces of an ancient landslide may be in equilibrium after having moved at some time in the distant past. Subsequent weathering of surface materials and growth of vegetation will often conceal the potential foundation failure from the untrained eye.

b. The geological bedding of a rock slope in its existing condition on the surface may suggest that steep cut slopes would be stable. Proposed cut areas should be examined in the field to determine if the excavation would exceed the limits of the good materials or favorable beddings. This is often the case in building highways in California due to the inconsistency of the geology. This is particularly true in recent years with the advent of higher and wider cuts than ever before.

c. Side-hill or "sliver" fills involving cut on one side of the line and fill on the other are quite often sources of foundation problems. These are usually due to the presence of poor materials in the underlying ground and the build-up of hydrostatic pressure when aquifers are "pinched" off by consolidation of foundation materials after the embankment is placed.

2. Embankment Settlement and Stability

a. In considering the matter of embankment foundations it is important to recognize that an embankment which settles may or may not present a stability problem bordering on imminent failure. In most cases embankments constructed over soft fine-grained soil foundations will cause the underlying foundation soil to consolidate. This closer contact of the soil grains contributes to a more stable foundation and decrease in the rate of settlement.

b. Some underlying soft materials may displace outwardly due to plastic flow when the load is applied. This may result in an embankment failure.

c. If weak compressible foundation soils are present in proposed embankment areas, it is probable that the design engineer will include some special treatment and/or construction control procedure in the plans.

d. If the presence of poor foundation soils can be determined during the planning and materials report stages of the proposed project there is usually adequate time for study and recommendation of suitable economic, corrective measures to be applied during construction. Hence, the importance of a foundation exploration program often justifies expenditure of the necessary funds to do the job.

3. Determining the Need for Field Exploration

During early reconnaissance of the proposed line it behooves the materials and design engineers to consult the available informational aids which often prove helpful in identifying

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foundation problem areas. These include geological and soils maps, aerial photographs, previous engineering reports, and finally, reviewing the topography in the field.

B. Surface Indications of Foundation Problem Areas

1. Problem Area Clues

The experienced foundation engineer or geologist can frequently discern problem areas by observing certain clues apparent on the surface.

a. In hilly or mountainous terrain, a landslide which has moved in recent times is usually obvious from the cracks, loose materials, and the disorder of the vegetation at the site. A quiescent slide, however, is frequently obscured by sloughing and weathering of surface materials and the subsequent growth of vegetation. Contour maps and aerial photographs will often identify these areas by clearly indicating a contrast between the orientation of the slide mass with the contours of the surrounding area.

b. Fault zones are potential trouble spots as they often involve poor materials and groundwater. Cuts in or near fault zones should always be given careful consideration.

c. A swamp or marshy area is easily identifiable as a probable poor foundation site. Silts and clays which are commonly found in the relatively flat areas will nearly always consolidate to some degree when loaded with an embankment. The presence of these materials in sufficient quantity to cause significant settlement is not usually apparent on the surface. Undisturbed sample borings, and subsequent laboratory tests are practical means of determining whether the underlying soil will support the proposed embankment and how much settlement can be anticipated.

C. Exploration Methods

1. Subsurface Operations

Subsurface explorations for foundation investigation purposes consist primarily of the following types of operations:

a. Push- or drive-sampling of fine-grained-soils (undisturbed samples).

b. Continuous helical auger flight borings.

c. Bucket rig borings.

d. In-place strength determinations.

e. Rotary coring of harder materials.

f. Installation of inclinometers and slope indicators.

g. Geologic methods.

D. Sample Types

1. Three Types

The choice of the drilling method is determined by the kind of sample needed to provide the desired information. Samples are basically classified into three types:

a. Undisturbed Samples

(1) It is generally agreed among soils engineers that no sample of soil or rock can be cut and brought to the ground surface and remain truly undisturbed.

(2) The so-called "undisturbed sample" is one which has been subjected to as little disturbance as possible in order to preserve its strength, consolidation, permeability characteristics, etc., in order that it may be used in laboratory testing with reasonable assurance that the results will be commensurate with the behavior of that material in the field.

b. Representative Samples

A sample of soil or rock whose physical structure may have been radically changed by the sampling operation, but which is not contaminated by other materials from elsewhere in

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the boring may be considered as representative of the chemical or mineral composition of the material, even though the water content may have changed. These include bucket samples.

c. "Cuttings Samples" or "Wash Samples"

(1) The least dependable sample of the three types, these are merely cuttings or residue as a result of the drilling operations. The chewing up and transporting of these cuttings from the bottom of the boring to the surface, thereby subjecting it to a mixing action with other materials in the same boring, does not afford a very reliable example of the characteristics of the "in-place" material.

(2) Helical auger flight drilling is fast and relatively inexpensive. It serves a useful purpose in foundation exploration work, but the samples of cuttings from this operation are not suitable for testing to determine the nature and behavior of the materials being drilled.

Borings drilled in this manner are an economical means of obtaining groundwater information. The rate of penetration of the flights during the drilling operations can be interpreted by an experienced driller or inspector as to the quality of the materials to some degree. However, a drive sample or core sample provides more reliable information.

E. Field Operations

1. Soil Sampling

a. The method chosen to obtain undisturbed samples necessarily depends upon the material to be encountered. In the case of soft soil a sampler may be driven by means of a hand-operated hammer. Hand operations are least expensive, require less time, and provide a sample of good quality if properly performed.

b. Drill rigs are used for sampling materials from soft fine-grained soils to stiff or hard clay as well as for rotary coring hard rock. The nature of the material also determines the kind of sampler to be used. It is necessary to obtain "undisturbed" samples for laboratory testing of soils for strength and consolidation. Numerous types of samplers have been developed for this purpose.

c. Proper training and skill in the technique of undisturbed sampling for foundation studies is most important to the laboratory testing program. The reliability of the test results depends upon the quality of the sample. Through soils mechanics theory, research projects, and practical application, the following conclusions have been established.

(1) Cutting and removing a soil sample from beneath the ground surface cannot be done without subjecting it to some degree of disturbance.

(2) A "thin wall," piston-type sampler equipped with an extension point when hydraulically pushed into the soil at a rapid rate will provide a sample of better quality than would a "snub-nosed" sampler advanced by hammering.

(3)* If the following basic requirements are satisfied, then a sample suitable for laboratory testing and for all practical purposes considered as "undisturbed" can be obtained:

(a) No disturbance of the soil structure.

(b) No change in water content or void ratio.

(c) No change in constituents or chemical composition.

2. California Sampler (Mod. 1952)

a. In order to obtain undisturbed soil samples for testing the California Division of Highways uses a steel tube lined with 2-in. diameter by 4-in.-long brass tubes. The bottom end "cutting point" is closed by means of a solid steel retractable piston or plug. When the

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*Committee Report of American Society of Civil Engineers on Sampling and Testing Soils Mechanics, November 1949, by M. Juul Hvorsley.

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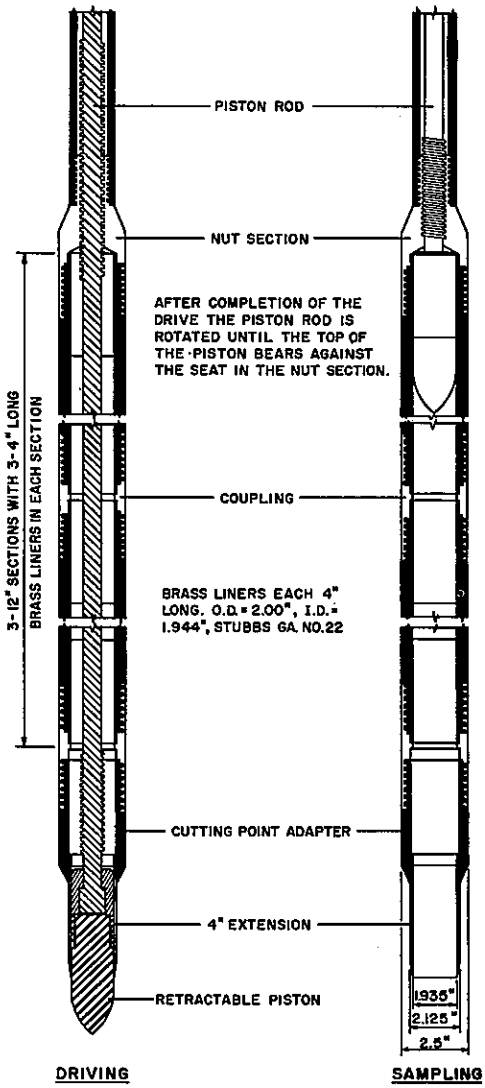

Fig. IV-1 — California type 2-in. soil sampler (Mod. 1952).

desired depth for sampling is reached the sampler is open by means of a "left-hand" thread arrangement on a steel rod which is attached to the steel piston or plug. As the sampling progresses deeper, additional rod sections are added through the hollow drill rod in order to make the smaller piston connected rod accessible at the surface. The open sampler is then hydraulically pushed or driven by means of a free-fall hammer into the soil. This sampler is sturdy, reliable, and simple to operate. It is readily adaptable to hand sampling or drill rig operations. A variety of cutting points facilitates use of this sampler in many types of soils. (See Fig. IV-1.)

b. The Division of Highways does a considerable amount of exploration with a 1-in.-diameter California Type Soil Sampler. The mechanism of this sampler is similar to the 2-in.-diameter sampler described above except that the soil samples are retained in 1-in.-diameter by 6-in.-long tubes. This equipment is used when the investigation does not warrant the field expense of truck-mounted drilling equipment and the laboratory expense of strength and consolidation tests. The sampler is advanced with a 25-lb hand hammer or a hammer activated by a small gasoline engine. The equipment can be packed into hilly locations where it is not practical to put the usual vehicular equipment. The samples can be used for classification tests such as moisture content, density and gradation but are not suitable for the tests necessary for a thorough strength and settlement study. This sampler is quite adequate for many investigations and the necessary soils information can be obtained at a very reasonable cost.

3. "Stationary Piston" California Sampler

a. Modification of the 2-in. regular California Sampler has provided a "stationary piston" type which usually obtains an excellent quality sample when used in very soft, fine-grained soils. Instead of the piston being retracted when the sampling depth is reached, it is merely released and the sample barrel is pushed downward into the soil for a distance of about 2 or 3 ft while the piston is held in a fixed position by clamping the inner rod to the drill rig mast. The sample enters the barrel simultaneously with the progress of the downward thrust.

b. When sampling below the water table or in wet soft plastic clay, retracting the piston allows entry of water and/or soil into the sample barrel prior to the sampling thrust. This is due to excess hydrostatic pressure and existing overburden pressure.

c. When the sampler is brought to the surface, the drill crew carefully disconnects the sections containing the brass tubes filled with soil. The inspector receives 1-ft portions of the sampler containing these tubes, places them in a sample ejector, and carefully pushes them out into a supporting tray or rack where each tube is severed from the adjacent one by means of a coping saw with a piano wire blade.

d. Trimming, patching, or smoothing the ends of these soils samples should be done with extreme care to avoid pressing or remolding the soil and thereby altering the strength characteristics.

e. After the tubes are sealed with tape and dipped in molten wax to protect against

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moisture loss, each one is carefully placed in a padded box for hauling or shipping to the laboratory. DO NOT THROW, DROP, OR OTHERWISE JOLT THESE SAMPLES. THIS IS MOST IMPORTANT.

4. Rock Coring

a. When it is necessary to cut core sample of rock, a truck-mounted rotary drill rig with 2- or 3-in. -diameter double-tube rotary core barrels is used. These rigs are capable of sampling to depths of as much as several hundred feet. The actual cutting of the rock is done with a rotating "core barrel cutting head" set with either tungsten-carbide inserts or industrial diamonds. During the coring operations water is pumped under pressure through the drill rod to the core barrel where it is discharged usually through holes in the cutting head. This cools the bit and also removes cuttings from the hole as more water is pumped in, thereby forcing the excess water containing the cuttings to the surface.

b. As the hole deepens the cuttings may not remain in suspension long enough to reach the surface. Commercially produced "drilling mud" additives are then mixed with the water. This product is basically a bentonitic type of clay. If the boring intercepts fractures, faults, or porous materials where the circulation water is lost, the drilling mud is usually thickened and this will frequently seal off the openings and prevent circulation loss. Experienced core sample drillers use drilling mud of proper consistency as an aid in the coring operation. This slick fluid also acts as a lubricant to reduce the frictional wear during coring. It is an aid to produce a core of good quality, and also lengthens the life of the core cutting head. If the boring wall has a tendency to cave in, thickened drilling mud will also aid in supporting it. A casing is sometimes used to stop circulation loss. This is a steel pipe large enough in diameter to accommodate the drilling bit or core barrel. It extends from the surface to some depth below the circulation water loss area. Sawdust or cottonseed hulls added to the drilling mud will effectively seal off small fractures and cracks, but these have a tendency to plug the holes in the core barrel in subsequent coring attempts.

c. In some locations and types of material it is not advisable to contaminate or saturate the samples with water or drilling mud. In these situations, the cuttings are removed by applying air pressure, developed by a compressor, through the drill rod. Drilling with air is quite successful if the material is relatively dry and the drill hole is above the water table. Difficulties are encountered when drilling in wet materials or below the water table.

d. When the geologic features of a rock formation, such as the fractures and strike and dip of bedding planes, are oriented in a somewhat vertical plane it may be desirable to obtain samples from horizontal borings. These "horizontal" borings are usually drilled at a slope of approximately 5% to 10% above the horizontal. The drilling operations are essentially the same as discussed above, where it is probably inferred that all rock coring is limited to vertical borings. The horizontal borings, in conjunction with vertical borings, will enable the engineer to present a more realistic description of subsurface conditions than would otherwise be possible.

e. Like most technical skills the knowledge of core drilling has to be acquired through on-the-job training and experience. To be really proficient a man usually has had years of experience behind him. The locations where drilling is done is most often in areas of unstable materials, faults, existing landslides, marshes, talus slopes, etc. Unlike water well or oil field drilling where the surface materials are drilled through and usually ignored, highway building is always concerned with the surface mantle of soil and rock. Where unstable conditions exist and a drilling program is warranted, many different kinds of soil and rock are encountered, and it is difficult to obtain representative samples. For this reason a driller with a broad background of experience in undisturbed sampling and coring is a most valuable asset. The Foundation Driller rating has been adopted by the Division of Highways in recognition of the special skill requirements so necessary in representative sampling for foundation explorations. The success or failure of any foundation exploration drilling program is directly related to the ability of the foundation drillers on the job.

5. Logging the Samples

a. The boring record is made by the inspector in charge of the job as the soil sampling progresses. This is the engineering record of the field exploration activities and should be legible, factual, and concisely accurate. The boring record form which is currently in use by the California Division of Highway is the product of practical field use of its prototypes over a

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period of about thirty years. A copy of the boring record is shown in the Appendix, along with instructions, which are mostly self-explanatory.

b. One point which should be reiterated here is the need to adequately complete the boring record at the time the sampling is being done. This is the only engineering record of the boring exploration which will be available to the engineering personnel when the foundation study, testing and analysis begins. REMEMBER: The field exploration will be looked at through the field inspector's eyes! The boring record and the samples should be of the best quality possible and complete to minor details. Something that is thought to be inconsequential at time of sampling may be most important during later studies.

c. The snap-out form contains three carbon copies. It is bound along the left margin to facilitate using a metal snap binder in the field. This not only protects the paper from dirt, grease, and water but also furnishes a hard surface on which to write. A 3-H or other medium hard pencil is recommended in order that the first copy will be clearly legible and also enough pressure can be applied to make the three carbon copies easily readable. Close examination of the attached sample boring record which includes a keyed instruction sheet, will provide answers to most of the questions which may arise when learning to use the form.

d. This set of instructions has been issued in an attempt to standardize the logging of soil samples and rock core samples by the inspectors throughout the State. A mutually accepted system of the materials descriptions and logging procedure is of prime importance for good communication between the inspector in the field and the soils engineer in the laboratory to produce an intelligent report of a foundation investigation. Deviation from a coordinated system causes unnecessary delay of the project and in some instances may contribute to inaccurate conclusions.

e. Some of the more troublesome areas of foundation engineering which are of importance to the soils engineer are as follows:

(1) The groundwater level in areas where borings have been made for consolidation and strength studies is vital to the computations to determine bearing capacity and the amount of settlement. It is also important in the calculations which determine an estimate of the rate of settlement. It is recommended that a representative number of borings be cased with perforated pipe to facilitate subsequent groundwater readings being taken over the several seasons of the year in order to more accurately establish variation in the groundwater table. Raingutter downspouting is recommended for this purpose as it is light-weight, inexpensive, and easy to perforate. A 75- to 100-ft-deep boring can be cased to the bottom in a few minutes by using 10-ft-long sections of downspout fastened with rivets or sheet metal screws. A device has been developed for perforating the downspout from the inside, and when fastened with blind rivets makes a smooth snag-free conduit.

(2) The thickness of the layers of different materials is vital to computations which determine amount and rate of settlement. Loose sand, gravel, or a combination of loose sand and gravel samples do not lend themselves to laboratory testing nor is it necessary to test them as thoroughly as the soft clay samples. The in-place strength and behavior under load of these materials is well known. This is not to imply that sand and gravel should not be sent to the laboratory. Quite the contrary, sufficient jar samples should be made to present a continuous soil profile of the boring and not just the layers of poor material. The thickness of each distinctly different layer of material should be documented by the boring record and verified by the jar samples sent to the laboratory.

(3) The complexity of soil and rock formations and deposits in California is well known. It is most important that the boring for samples which are to be representative of the conditions affecting the foundation study be located in the proper place to determine applicable results.

(4) Soil sample borings for a foundation settlement and strength study should be placed where the maximum load is expected and at any additional locations where settlement may be expected. If the stability of an area is in question, some borings should be made along the toe of the proposed slope. They should be sufficient in number to be representative of the existing conditions. The number and depth of these borings should be on the basis of the materials encountered, and the number of good quality undisturbed samples recovered and processed for laboratory testing. In the case of cut slope design borings, it is desirable to locate them in such a way as to provide samples of the soil or rock which, for the most part, will remain in

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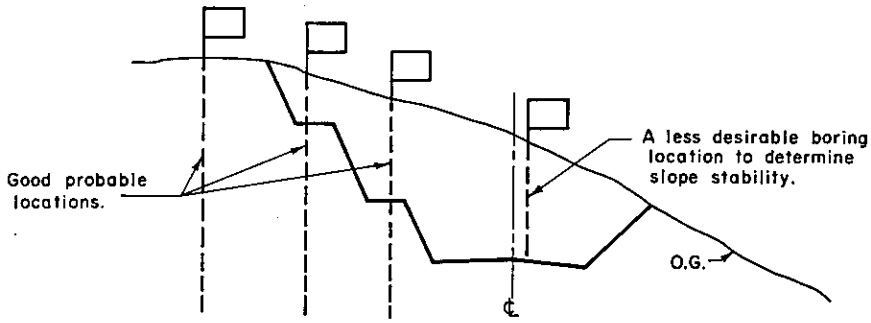

Fig. IV-2 — Recommended locations of borings to obtain data for cut slope design.

the slope after construction and not at a place which will be representative of the material to be excavated. (See Fig. IV-2.)

(5) If the boring is in an occupied neighborhood, it is recommended that the top be covered by some means to preclude vandals from dropping objects in the boring and preventing further water readings. It has been found that a length of 3-in.-diameter galvanized pipe, with a coupling and plug screwed on one end, fitted over the top of the boring will safely protect it for future readings.

6. Processing the Undisturbed Soil Samples

a. After the brass tubes are cut apart, the inspector examines them carefully, notes changes of materials, decides which tubes are worth saving for shipment to the laboratory, and which ones should be put in jars for visual observation. If an extension point is being used, this material makes an excellent jar sample since it is not practical to adequately package it for consolidation or strength tests. If enough material is available, a jar sample should be made of each significantly different material. This enables laboratory personnel to have a visual soil profile of the boring when the jars are lined up as indicated on the Boring Record. This is most helpful when considering what testing, and how much is necessary on the tube samples which have been sent in to the laboratory. In this way it is not necessary to open the sealed tubes until time for them to be tested.

7. Bucket-Rig Borings

a. These bucket samplers are available in many sizes with diameters ranging from several inches to several feet.

b. The bucket drilling operation is accomplished by rotating a steel cylinder-type container which has cutting edges or teeth affixed to the bottom end. The material is gouged out by the rotary motion and the chewing operating of the cutting edges. This loose material is caught in the container much in the manner of a rotating posthole digger. It is then withdrawn from the hole and emptied.

c. The samples are representative of the materials at a particular depth rather than being a mixture of various materials from throughout the depth of the boring as in the case of auger flight cuttings.

d. There is a practical limit to the depth to which these borings can be drilled due to the in and out operation to empty the bucket after drilling a short distance.

8. In-place Strength Tests

a. Strength determinations of materials in-place may be accomplished by means of a device called the "vane borer." The apparatus consists of two paddles or vanes attached to a circular shaft at right angles to each other. This is pushed into the soil to the desired depth for the shearing strength determination. Torque is applied to the rod on which the vanes are fastened. This causes a shear failure as pressure is applied to the soil. The amount of force required to cause failure is converted to shear strength in pounds per square foot. This test is comparable

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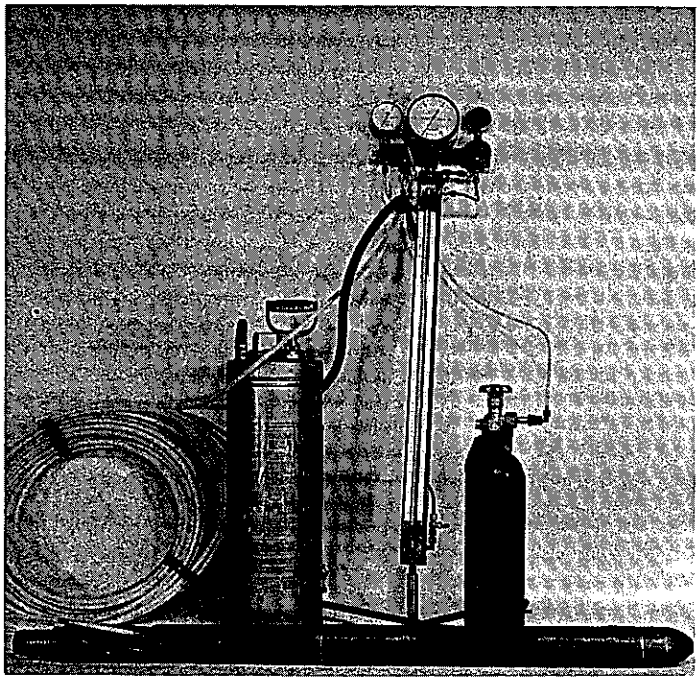

Fig. IV-3 — Menard pressuremeter device.

to the unconfined compression test on a laboratory sample. This method of strength determination is limited to the softer fine-grained soils containing little or no rock or thick layers of sand.

b. The Pressuremeter is a patented device developed by Louis Menard of Paris, France, for the in-place testing of soils. It is handled in the United States by the Geocel Corporation, 16027 West 5th Ave., Golden, Colorado.

(1) The Pressuremeter equipment is shown in Fig. IV-3. It consists of a source of pressure, a source of water, the volumeter, coaxial tubing, and the testing probe as fabricated for insertion into the borehole.

(2) The Pressuremeter is basically a hydraulic cell which is inserted into a borehole and expanded against the side of the hole. A record is made of the increments of pressure versus the volume of fluid required in the expansion of the cell. The test is usually continued until the soil surrounding the probe has failed. This process is repeated at several locations down the borehole. From this information it is possible to estimate the shearing strength of the soil, and to calculate a load-deformation modulus.

c. The Iowa Soil Borehole Direct Shear Device was designed by R. L. Handy, Professor of Civil Engineering, Iowa State University, and Captain N. S. Fox, U. S. Army Corps of Engineers. It is an expanding-split hydraulic cell for determination of shear strength in soil boreholes. The faces of the cell are serrated to firmly engage the soil surface of the borehole wall when a pressure is applied to the cell. Tension is then applied to a cable attached to the cell until shear failure occurs in the soil engaged by the serrations. Increased pressure is applied and the operation repeated. The data and method of analysis are similar to that of the conventional direct-shear test.

d. The Goodman Borehole Plate-Bearing Test Device was developed by Richard E. Goodman, Assistant Professor of Geological Engineering at the University of California at Berkeley. It is a device for high-pressure testing in 3-in.-diameter boreholes in rock. The plate-bearing type cell is inserted to any desired depth, and pressures up to 10,000 psi are supplied by a hydraulic jack. It will simultaneously measure displacements of the boreholl wall to 5 millionths of an inch. An elastic modulus may be developed from the resulting data for use in analysis and design.

e. The California Division of Highways laboratory has used the Menard Pressuremeter at several locations and indications are that the data obtained will provide parameters of angle of friction and cohesion which will compare favorably with the parameters obtained with conventional laboratory equipment. The laboratory has not used the Iowa or Goodman devices.

9. Inclinometers

a. The practice of placing heave stakes along the toe of an embankment as a control check against ground movement during construction has long been used in California highway construction. In recent years an installation known as the inclinometer has been adopted as a supplement to the heave stakes. It consists of a small-diameter plastic tube which is placed in a drilled hole near the toe of the embankment. Various lengths of solid steel rod which are small enough in diameter to pass snugly through the plastic tube are then used as sounding probes and lowered to the bottom of the hole. If ground movement begins to occur as the fill load is increased, the plastic tube is bent from its original vertical position. As subsequent

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checks are made by lowering the probes, this bending of the tubing is immediately apparent as the probes will lodge at the top of the bend. This method of observing ground movement also indicates at what depth the top of the movement is occurring.

10. The Slope Indicator

a. A more sophisticated instrument for measuring horizontal ground movement has been developed by the engineering firm of Shannon and Wilson of Seattle, Washington. This device consists of a pendulum (tiltmeter), a control box, connecting cable, and a slotted plastic casing installed in a boring. The position of the casing slots is oriented with the compass in order to facilitate duplication of the instrument reading process and provide the reference points for description of direction of movement.

b. The slope indicator instrument is housed in a watertight brass cylinder which is about 15 in. long by 2-1/2 in. in diameter. This cylinder is lowered into the plastic casing in the boring by means of the electrical connecting cable. The tip of the pendulum contacts a precision wound resistance coil which subdivides the coil into two resistances forming one-half of a conventional Wheatstone-bridge circuit; the other half, including switches and batteries, is enclosed in a control box at the ground surface. The inclination of the tiltmeter is proportional to the dial reading of a precision potentiometer in the control box, which reads from zero to 1000.

c. The curve obtained by plotting the position of the pendulum at rest at a given depth through a series of subsequent readings will indicate the variance obtained as a result of ground movement.

d. Proper installation and use of the slope indicator makes it possible to accurately determine ground deformation of as little as 1 in. in 100 ft of depth.

11. Geologic Methods

a. A geologic map is a base map on which field observations of important rock types and geologic features have been plotted. Mapping studies have been conducted throughout most of the state. These studies are normally conducted in conjunction with alternate route, cut slope design, materials site, and groundwater studies.

(1) A simplified geologic map is shown in Fig. IV-4. Some of the important data depicted are the dip and strike of rock units with relation to the proposed centerline, unstable zones and landslides, possible groundwater problem areas, and possible materials sites.

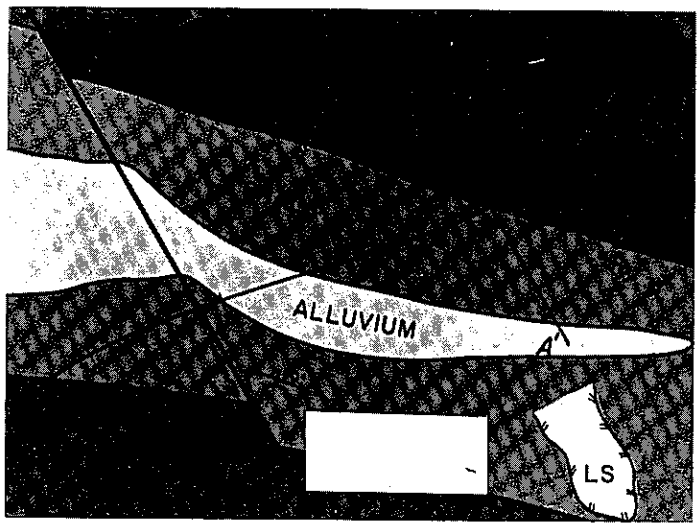

Fig. IV-4 — Example of geologic mapping.

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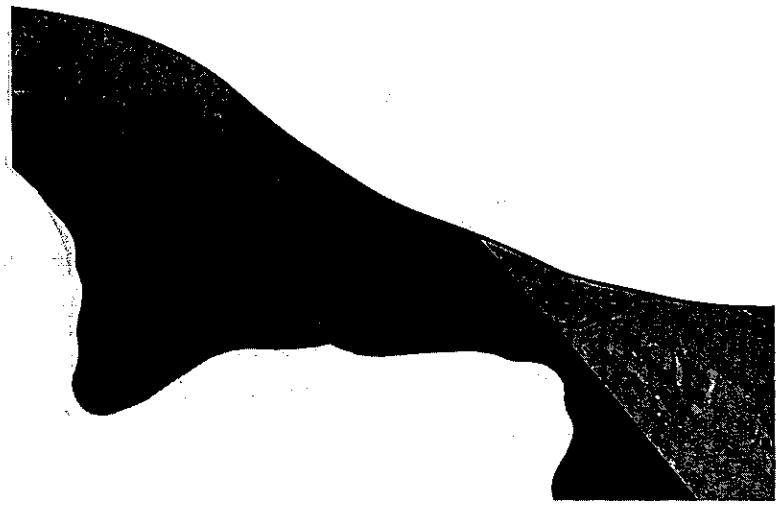

Fig. IV-5 — Cross section of line A-A' of Fig. IV-4.

(2) Fig. IV-5 is a cross-section taken along line A-A' of the geologic map. A study of cross-sections of a geologic map enables the engineering geologist to recommend a cut slope design that conforms to stability and the aesthetic properties of the formations involved.

b. Considerable information is gained from photographs when they are used as a supplement to geologic mapping which is known as photogeology. This is particularly true of aerial photographs. A trained observer can identify such important features as drainage patterns, bedding, joint patterns, faulting, landslides, and various soil types. The study of aerial photographs should be done in the early phases of an investigation. The information gained from the photographs is checked in the field before it is added to the geologic maps.

c. The basic units involved in seismic refraction studies are illustrated in Fig. IV-6. These are the energy source, the geophones, the geophone cables, and an instrument for recording the arrival of seismic energy. The seismic method depends on a wave front expanding outward from the energy source, refraction at the interface between materials with different elas-

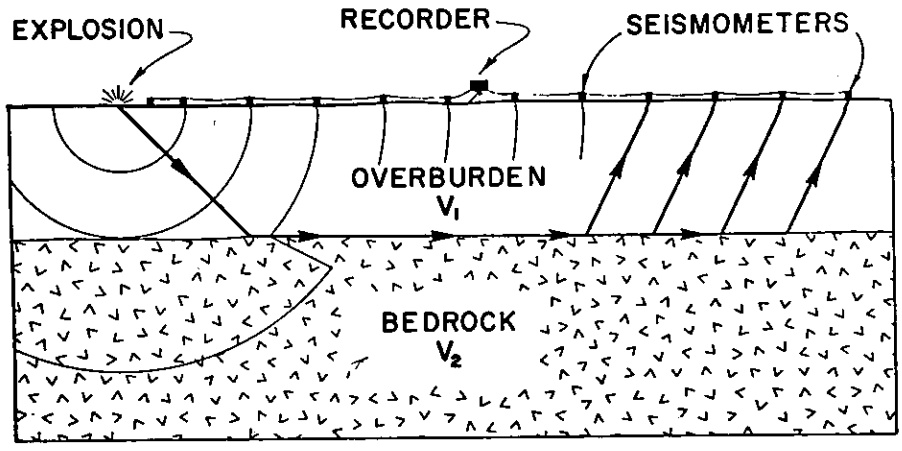

Fig. IV-6 — Basic units of seismic refraction study.

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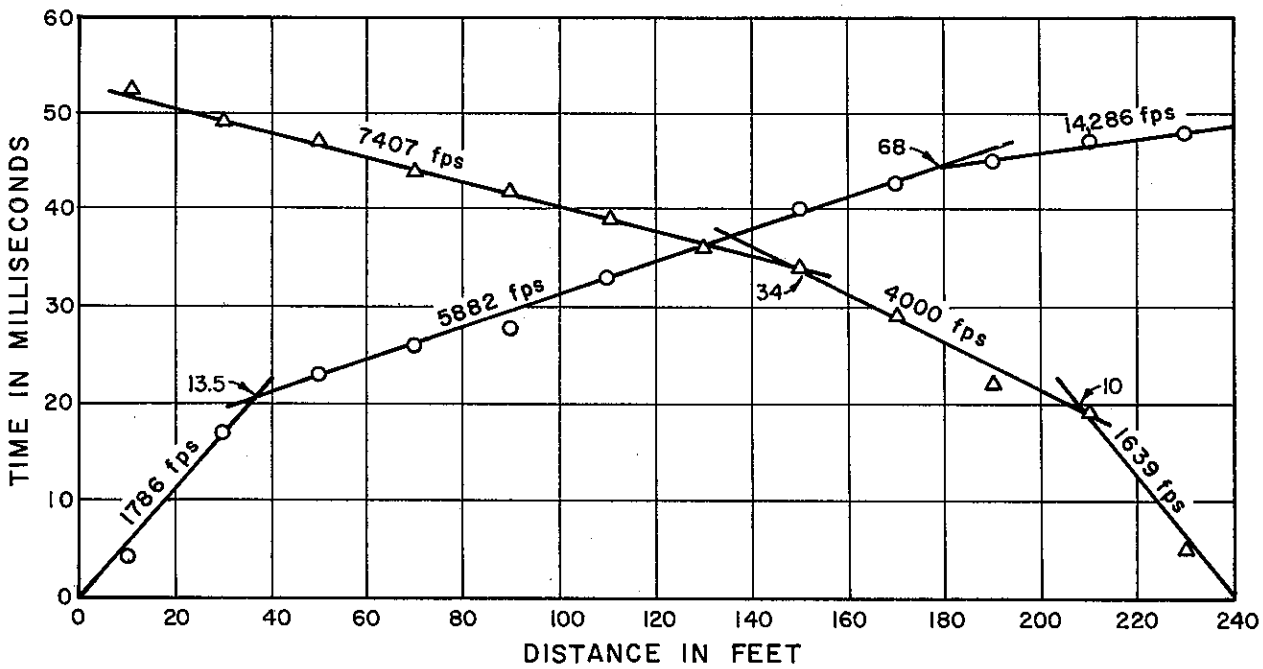

Fig. IV-7 — Time-distance graph developed from seismic investigation.

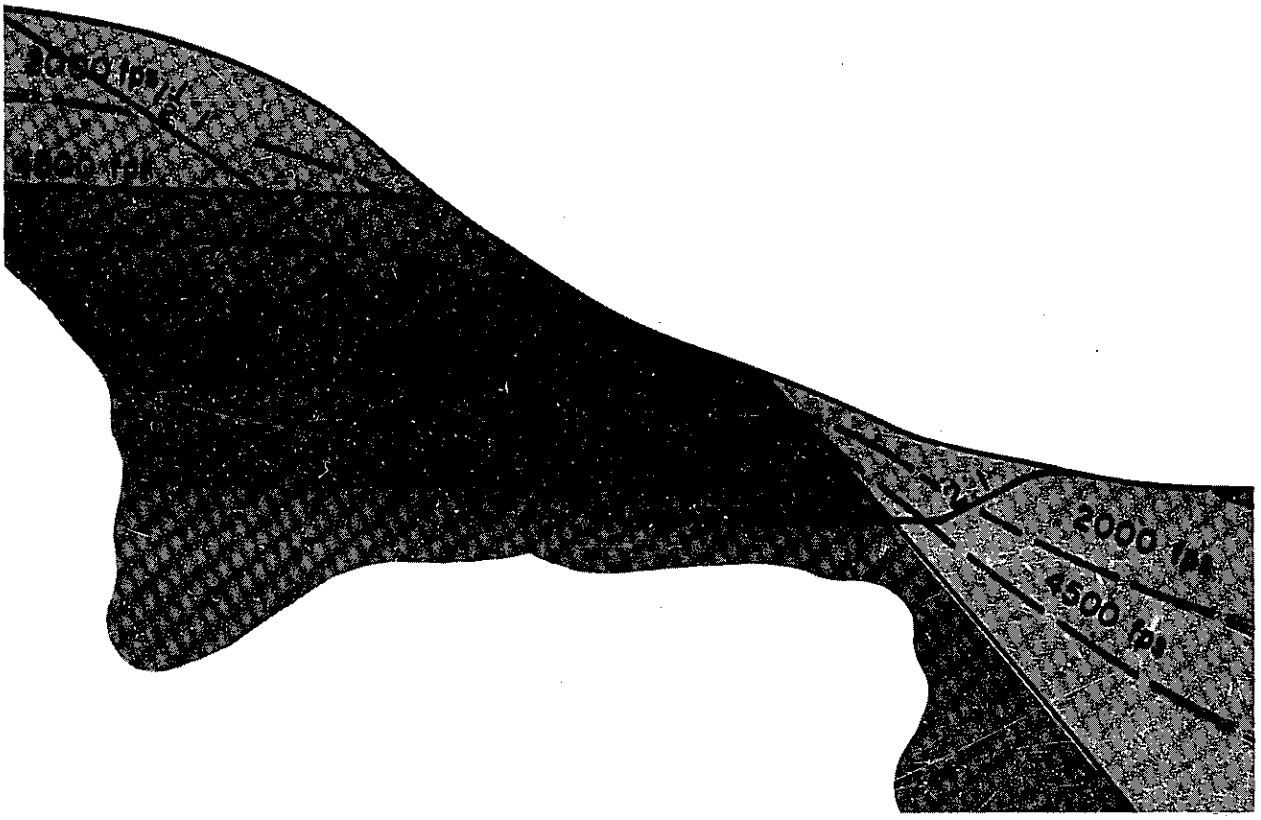

Fig. IV-8 — Seismic information superimposed on geologic data.

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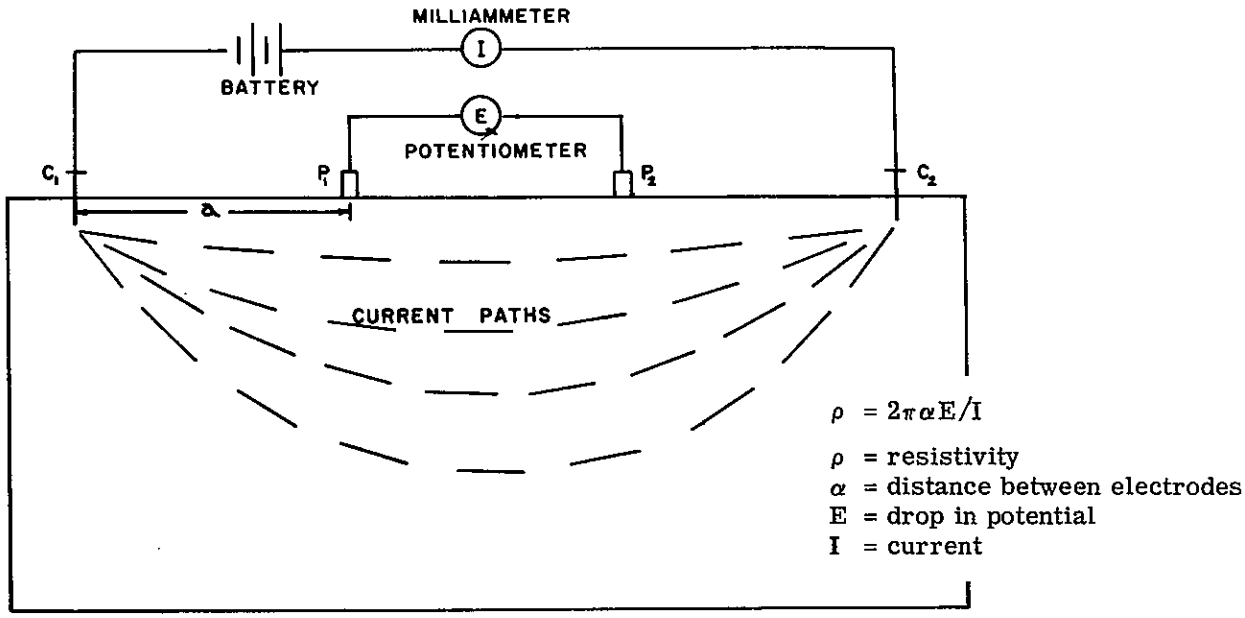

Fig. IV-9 — Basic units of earth resistivity study.

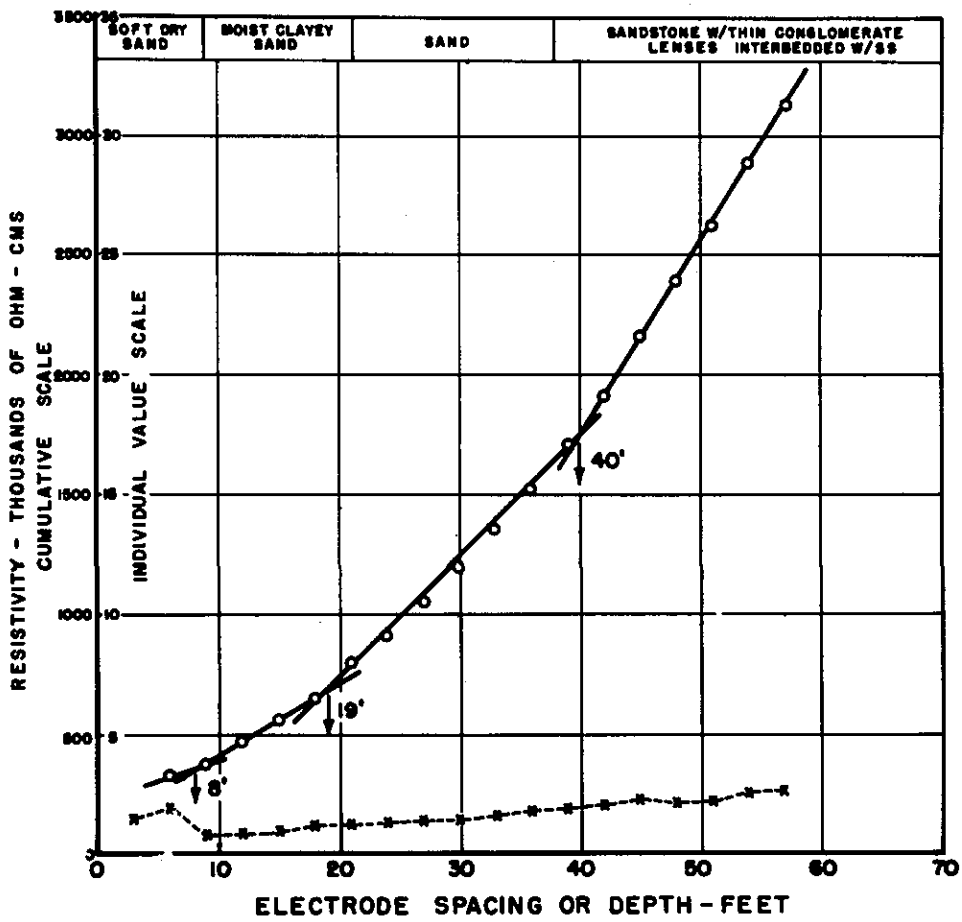

Fig. IV-10 — Plot of cumulative resistivity as a function of electrode spacing or depth.

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tic properties, and return of the energy to the ground surface. In a refraction seismic study only the first arrivals of energy at the geophones are used. The raw data obtained are the travel time from the energy source to the geophones and the distance between the geophones. These data are plotted on time-distance graphs as indicated in Fig. IV-7. Lines are drawn through the points and the inverse of the slope of the lines is equal to the longitudinal wave velocity. Depth calculations are made at changes in slope to higher velocities. Velocities normally increase because of such features as consolidation, groundwater, and changes in material from soil to rock.

(1) Fig. IV-8 shows seismic information added to the same cross-section as shown in Fig. IV-5. A study of the data enables one to determine the depth to rock, the excavation characteristics of the overburden and the rock, some estimate of stability of the cut slopes, and the approximate location of groundwater.

(2) There are certain limitations on the use of a seismic refraction method. A constant or an increasing velocity with depth is required. A plane surface approximately equal to three times the depth of investigation is required. For example, if a depth of investigation of 50 ft is needed, the geophone lines on the surface must be laid out on a plane 150-ft long. The plane need not be horizontal, in fact studies have been conducted on slopes as steep as 40 deg. Changes in slope along the seismic line must, however, be avoided.

(3) One of the limitations mentioned above for the seismic refraction method is the requirement for either a constant or an increasing velocity with depth. This limitation can be overcome with an uphole survey. Small charges of dynamite are detonated at progressively shallower depths in a bore hole and the velocity of the materials are measured from a number of points in the drill hole to a geophone placed on the ground surface. If holes have been previously drilled, this is a method of gaining substantially more information about an area.

d. Earth resistivity is defined as the resistance of the earth to the passage of an electrical current. Fig. IV-9 is a schematic diagram of the equipment used to determine earth resistance. Two electrodes, connected to a battery are used to induce current into the soil. A potential drop is measured between the two inner porous pots. A potential gradient is measured on the surface by taking readings as the distance "a" is increased at a constant rate. There is an empirical relation between the distance "a" and depth. Therefore there is an empirical relation between the potential gradient measured on the surface and a potential gradient with depth.

(1) Fig. IV-10 is a plot of cumulative resistivity values and distance "a." A description of the material is shown at the top of the graph. Note the changes in the slope of the curve with changes in the material.

(2) Resistivity curves must be correlated to drill logs or outcrops or used in conjunction with other geophysical methods. It has been found that earth resistivity values in any given area can change radically with the seasons of the year. This is probably due to changes in the water content of the soil.

(3) A variety of information can be obtained by means of a resistivity survey. It is often possible to identify changes in materials such as from sand to gravel or from soil to rock. Weathering patterns in rock formations can be identified. The depth and horizontal extent of landslides can often be determined. The resistivity method is often used to locate a water table. There is some indication that the resistivity method can be used to locate such hidden features as tunnels and solution cavities.

(4) There are some limitations on the resistivity method. Similar to the seismic method it requires a plane surface approximately equal to three times the depth of investigation required. Also this method is not particularly suitable in areas of highly complex geology nor is it suitable in areas where the surface material is extremely dry.

(5) Electrical resistivity equipment is portable and light weight. Interpretations can be made on the spot in the field when this is necessary or desirable. The method is not limited to zones of increasing hardness. One of the major uses for the equipment is locating soft or saturated zones such as might be anticipated in a landslide at depth.

e. An engineering geologist is usually assigned to the drill crew for rock coring projects. The geologist has been trained and is well qualified to evaluate the cores as they are taken from the ground and before they have been damaged by handling and exposure to the atmosphere.

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f. The Borehole TV Camera is a relatively new method of investigation that is currently being used by this department. In practice it consists of lowering a specially constructed TV camera into a drill hole. This provides a continuous closed circuit television picture of the boring. This tool can be used to evaluate the material and to photograph areas of particular interest within the boring.

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V. Applications to Highway Design and Construction

A. Design of Cut Slopes

1. Basic Considerations

a. Information required for the economical design of cut slopes includes nature and strength of the material to be excavated, groundwater conditions, attitude of beds of sedimentary rocks; degree of weathering, jointing, and fractures; and the presence of landslides, active or ancient. To obtain these data several of the following methods are commonly utilized: visual inspection on ground; study of topographic maps and air photos; geologic survey; geophysical exploration, either seismic or resistivity; exploration by borings, vertical or horizontal; and tests on undisturbed samples or cores.

b. The methods to be employed in an individual cut slope design will vary with the scope of the problem, with the sources of information that are available, and with the physical conditions at a location. Often, the most suitable type of investigation and the best methods to use will not be known in the beginning but must be determined during the progress of the exploration and studies.

c. The normal procedure for investigating a proposed cut is to evaluate all available data, study surface conditions, and develop a working hypothesis. On the basis of this hypothesis an exploration program is started. As data are collected on the properties of the earth masses involved, the exploratory techniques are altered as required to obtain the needed information. If we knew in advance what kind of methods would be most appropriate, where the explorations should be located, and how deep each boring should be made, we would have many of the answers that are being sought by the explorations.

d. If the material to be excavated is a fairly uniform fine-grained soil, shear tests can be made on undisturbed samples, and the stability of various slopes can be calculated by stability analyses. Generally the cut more often contains more or less rock and is heterogeneous. These conditions virtually preclude any determination, by laboratory tests, of the average or effective shear strength of the slope forming material. In such cases the stability analysis by theoretical soil mechanics methods is not possible, and the cut slope design must be determined by application of judgment and experience, after evaluating and taking into consideration all of the factors mentioned above which might affect slope stability.

e. Obviously, for reasons of economy, the cut slopes should be as steep as possible, consistent with stability. Typical slopes for various materials might be somewhat as follows:

(1) In cohesionless sands the slope should be no steeper than 1-1/2:1, since this is about the angle of repose for such material.

(2) In cohesive soils, containing silt and clay along with sand, slope of 1-1/2:1 or flatter are usually in order.

(3) In cemented sediments, such as sandstone, shale, and conglomerate, steeper slopes may be used, depending on the degree of cementation, bedding, jointing and groundwater conditions.

(4) In weathered rock, slopes may vary from 3/4:1 to 2:1, again depending on degree of weathering, groundwater, etc.

(5) In hard fresh rock, slopes as steep as 1/2:1 or 3/4:1 are sometimes possible.

2. Benched Cut Slopes

a. The question of benched cut slopes frequently arises. In the past it has been the opinion of the California Division of Highways that high cut slopes, i. e., 100 ft or more, should always be constructed with at least one bench which would serve to reduce traffic hazard from falling rock and debris or to control erosion of the cut face caused by surface water flowing down such a long slope (unbroken by a bench). In unstable wet slopes which require installation of horizontal drains, benches also provide a working area from which to install the drains. The

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planning manual of the Division of Highways stipulates a bench width of 20 ft minimum. In many cases a narrower bench is useless, as sloughing, erosion, or very minor local slides would soon obliterate the bench. Current practice, particularly in the northern part of the State, is to construct 30-ft-wide benches.

b. The stability of a cut slope against major slides with the head of the slide above the slope line is determined by the effective overall slope from the hinge point to the top of cut or to some point on the slope above the top of the cut. The same overall stability may be achieved by either a uniform slope or an equivalent benched slope. After deciding on the overall effective slope which is considered stable, the cut faces between benches must be designed flat enough to prevent local slope failures between benches. We do not recommend benches for purposes of stability at less than 40-ft vertical spacing and generally 60 ft or more is preferable. In order to retain the desired overall slope the cut faces should be flattened as the vertical distance between benches is increased.

c. It is important in most cases that adequate surface drainage be provided on benches. In general, surface water and groundwater should be handled by separate facilities. To combine them usually decreases the efficiency of one or both. With the advent of more serious consideration of aesthetics in highway design, the use of benches is diminishing in lieu of flatter slopes and widening at grade.

3. Existing Cut Slopes

Cut slope design on new construction is frequently based on the slopes of existing cuts adjacent to or in the vicinity of the proposed excavation, even though the new cuts may be of much greater height than the existing cuts. This design is valid only if two conditions exist: (1) the soil and groundwater conditions in the proposed excavation are the same as in the exposed cut faces; and (2) the stability of the slope is independent of the height of cut. Both assumptions could be hazardous, and a new cut should never be arbitrarily designed with the same slope as some existing cut without careful consideration of all of the factors affecting slope stability. In rare situations, certain types of soil make the stability of cut slopes independent of the height of cut.

4. Surface Indications

Surface indications alone cannot always be used to predict the quality of formations that will be exposed by highway excavations. There have been numerous instances in California highway construction of cuts being made into apparently stable rock formations, in which a hard knob formed a shallow cap over much weaker formations. In such cases, the hard rock exposure gives a false picture of the true condition, and the stability of a proposed cut may be much less than is apparent from surface indications.

5. Relative Stability

Although the shear strength of most cut material cannot be measured by laboratory tests, the relative stability of various slope designs can be compared by assuming or estimating reasonable values of cohesion, c, and angle of internal friction ϕ. Stability analysis can be made using these estimated strengths and the factor of safety can be computed for different slope designs. Also, the safe heights possible for various strengths can be calculated. The results of a study of this kind are summarized on Fig. V-1.

6. Calculated Risk

There is a tendency among design engineers to think in terms of a calculated risk of slope failures or landslides. We object to the use of this term, because the proposed risk is usually indeterminate, sometimes almost certain, but seldom calculated. Furthermore, slope designs are never 100% safe. Almost always there is some risk of slope failures and landslides when cutting through unstable terrain. The use of 1-1/2:1, 2:1 or 3:1 cut slopes does not necessarily assure that the factor of safety is high or that slides will not occur. For example, in Culebra Cut through the Cucaracha formation in Panama, slopes were flattened to 8:1 and 10:1 in some aresa before slide movement was finally controlled.

7. Ground Water Effect

a. Landslides can and do occur without the presence of groundwater, but in the vast majority of the most troublesome landslides and slope failures groundwater is probably the most

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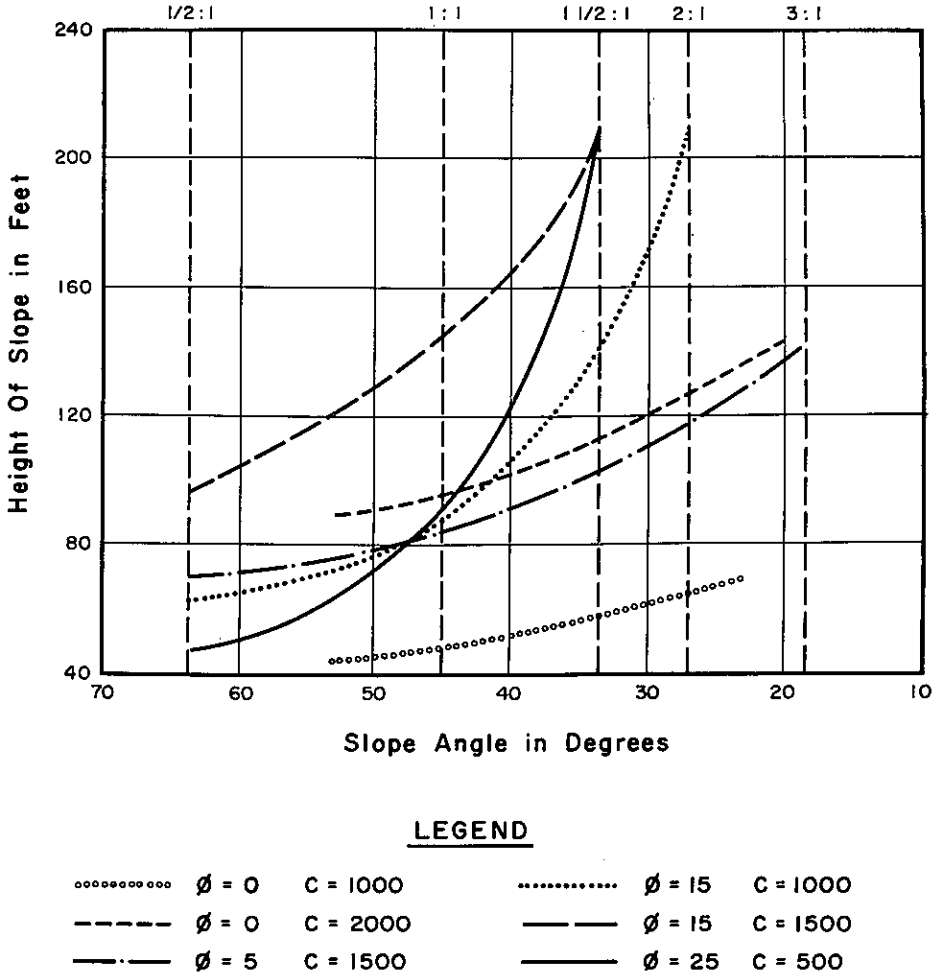

Fig. V-1 — Height of slope vs slope angle for various values of c and ϕ.

important single contributing factor. Accordingly, it is extremely important that groundwater conditions be determined before designing cut slopes in excavation where it is known or suspected that groundwater may be present. This can usually be done only by means of borings. Pumping tests of boreholes may be necessary to ascertain whether the amount of subsurface water is likely to affect the stability of the cut slopes.

b. If it is known that groundwater will be encountered in the excavation of unstable or questionable material in cut areas, the cut slopes should be designed accordingly.

c. If the soil is such that the water can be intercepted and removed by horizontal drains, the design should provide for such drains and a contract item should be included. It may be necessary to install horizontal drains from benches in the cut slope as well as from roadway grade. The location, length, and spacing of the drains will depend on the location of the water bearing soil, character of the soil, height of cut, and topography.

8. Natural Planes of Weakness

Of paramount importance to the stability of cut slopes are the orientation, continuity and dip of natural planes of weakness. Regardless of how carefully all evidence is studied, the overall slide potential of a given earth mass is never truly known until the excavation has been made and the mass has been put to a full-scale test.

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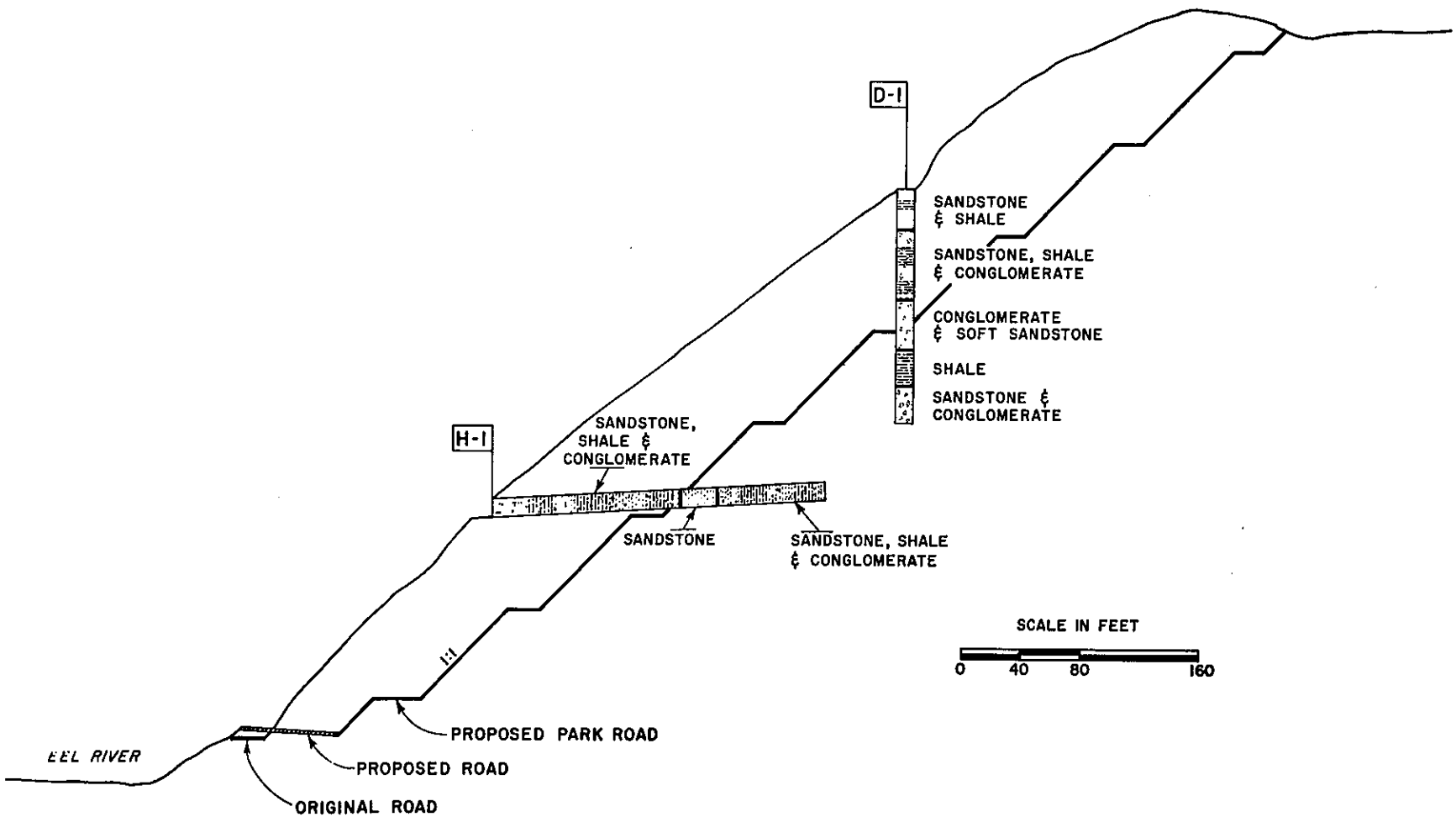

Fig. V-2 — Typical section of Dyerville cut showing benched design and simplified borings.

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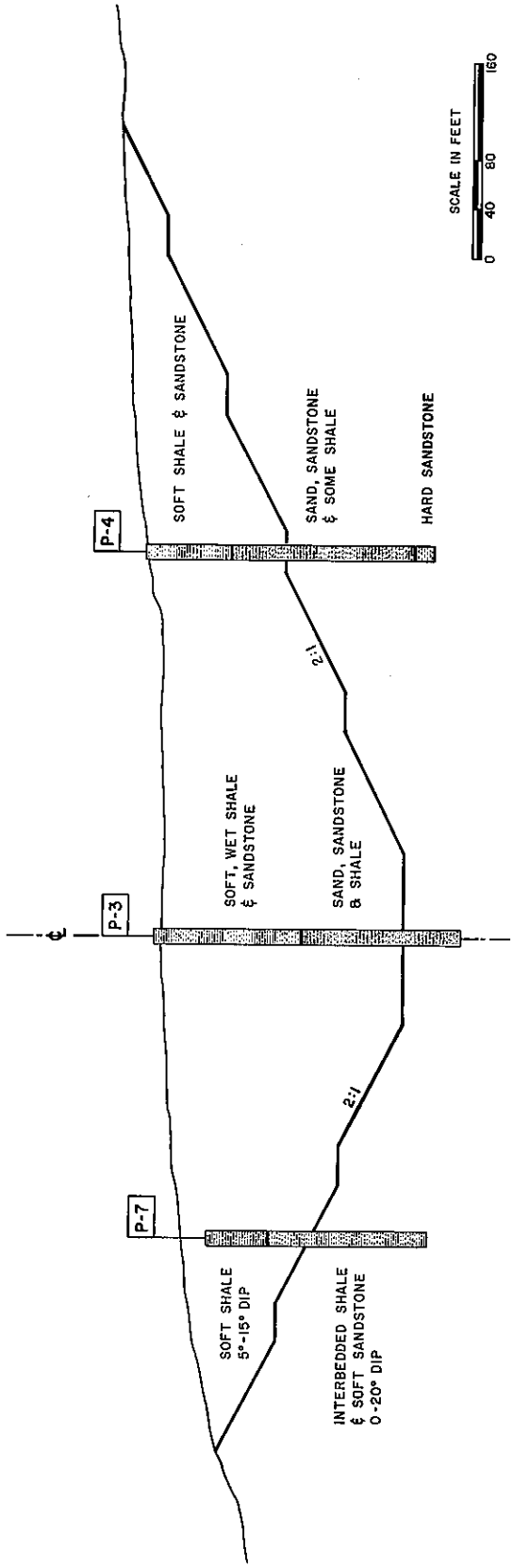

Fig. V-3 — Typical section of Carquinez cut with simplified borings.

9. Complexity of Cut Slope Design

If the impression that cut slope design is neither simple nor an exact science has been created, then the problem has been realistically appraised. Thorough evaluation of a great many factors and conditions is essential; experience and judgment are important ingredients, and prescience would certainly be helpful.

10. Examples of Cut Slope Designs

a. In Fig. V-2 a cross-section is given through the center of the Dyerville Cut which is one of the highest side hill cuts in highway history. This cut, on U. S. Highway 101, is located near the town of Weott in Humboldt County in North Coastal California. The top of the cut stands 480 ft above the grade of the completed road.

(1) At the site of the cut the materials are principally thick, interbedded sandstone and shale, and some conglomerate. The sediments have been subjected to considerable deformation and fracturing. Many of the fracture systems contained heavy accumulations of groundwater. Seepage was evident at the old roadway near the toe of the new cut.

(2) On the basis of a field study and an examination of cores it was concluded that the slope had a high degree of stability. The presence of large redwood trees on the slope with an age of at least 2000 years confirmed this conclusion as they were living proof that no important slide movement had occurred for many centuries.

(3) In view of the great height of this cut the recommended design called for 1:1 slopes with 20-ft-wide benches at 60-ft vertical intervals. This design produced a slope roughly parallel to the slope that existed before this cut was excavated.

(4) This cut was constructed according to plans and the job was accepted as complete on November 12, 1958. Nine winter seasons have passed with no signs of instability.

b. Another major cut is the Carquinez "Big Cut" which was constructed south of the Carquinez Straits in Contra Costa County on Interstate Highway 80. The natural terrain south of the straits was standing on slopes of 3:1 or flatter. Two faults were known to exist in the hill. The high cost of tunnel construction left

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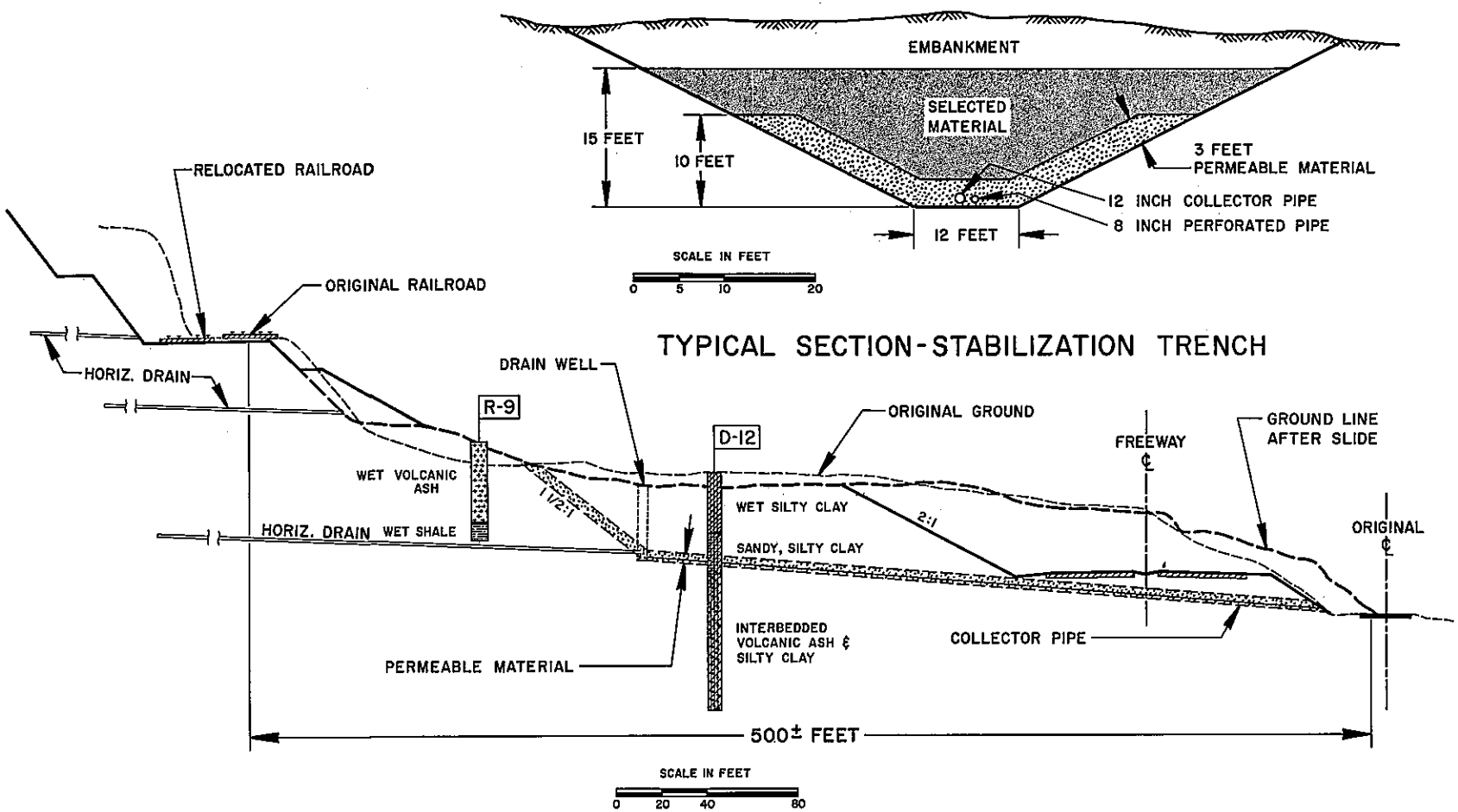

Fig. V-4 — Profile and typical section of stabilization trench at Towle Slide.

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no choice but to construct this section of highway by open-cut methods. A number of deep drill holes revealed soft interbedded shales and sandstones.

(1) The sediments ranged from hard sandstone to soft friable sand, from firm silty shale to soft clay-shale. Interbeds varied from microscopic to visual. Substantial evidence of groundwater was disclosed by the borings. When deposited, the sediments that form this region were uniform and competent, but intensive folding and faulting have greatly weakened the masses.

(2) The typical cross-section reproduced in Fig. V-3 shows approximately the maximum section through the cut, which was designed with slopes of 2:1 and 30-ft-wide benches at 60-ft vertical intervals. This cut was widened about 30 ft on each side at roadway grade to provide protection against the blocking of traffic lanes in case large slides should take place after the road was opened to traffic. A substantial number of horizontal drains were installed at various levels in the cut to lower the groundwater level.

(3) This cut has been stable with the exception of minor sloughing and one slide involving about 2% as much volume as the original cut. Much of the guess-work in the design of this important highway cut was eliminated by the use of soils explorations and geological methods.

(4) Frequently highway alignments must cross landslides, many of which are more or less active. When it is necessary to cross known landslides the design should be developed with a view to improving stability rather than lowering it. In some cases material should be removed from the head of the slide and in others the material should be placed at the toe. Control of drainage is usually necessary.

c. A cross-section through a portion of highway constructed across a very troublesome landslide known as the Towle Slide is given in Fig. V-4.

(1) During the winter of 1957-58, a large slide occurred in the area between U. S. 40 (now Interstate Highway 80) and the double-track of the Southern Pacific Railroad just below the town of Towle in Placer County. The slide extended approximately 1,000 ft along I-80 and the railroad. The head of the slide was at the railroad tracks. Local slipouts occurred beneath the tracks, leaving the tracks in one direction suspended. Borings revealed very unstable soils and weathered light-weight rocky material in the slide.

(2) Though the construction of a longitudinal stabilization trench between the existing highway and the railroad would have been desirable, it was evident that the upper side of the stabilization trench would not be stable, and the railroad would be endangered during construction of the trench. The decision was made to construct a series of transverse stabilization trenches extending from the existing road toward the railroad, recognizing that even the upper ends of these trenches would have to be kept some distance from the railroad during construction. These trenches were constructed as the first order of work on the new freeway. Several wells were drilled through the slide mass and pumped prior to and during construction of the trenches.

(3) Horizontal drains were fanned from the upper ends of these trenches to depths of 100 to 150 ft. The upper ends, bottoms, and lower portion of the sides of the trenches were blanketed with a layer of permeable material to facilitate drainage. A perforated pipe was placed in the lower part of the trench to carry the water flowing into the trench. These stabilization trenches were spaced on 200-ft centers.

(4) Drainage wells were installed along a line connecting the upper ends of the stabilization trenches and roughly parallel to the new freeway. These 30-in.-diameter drain wells were installed on 8-ft centers and were connected at the bottom by hand excavation. A slotted pipe was placed in this tunnel connecting the bottoms of the drain wells, and the remainder of the tunnel was backfilled with permeable material. Vertical perforated pipes were placed in the drain wells, and the area around the pipes backfilled with permeable material. This in reality provided a pervious drain wall or gallery connecting the upper portions of the stabilization trenches. The freeway was subsequently completed and this installation has gone through seven winters without any signs of serious distress.

d. Construction of the San Diego Freeway required an unusually deep and large cut known as the Mulholland Cut which is located in the vicinity of Sepulveda Blvd. and Mulholland Drive, a few miles to the northwest of Beverly Hills, California. At its deepest point, the proposed highway grade was to be 320 ft below the original ground line.

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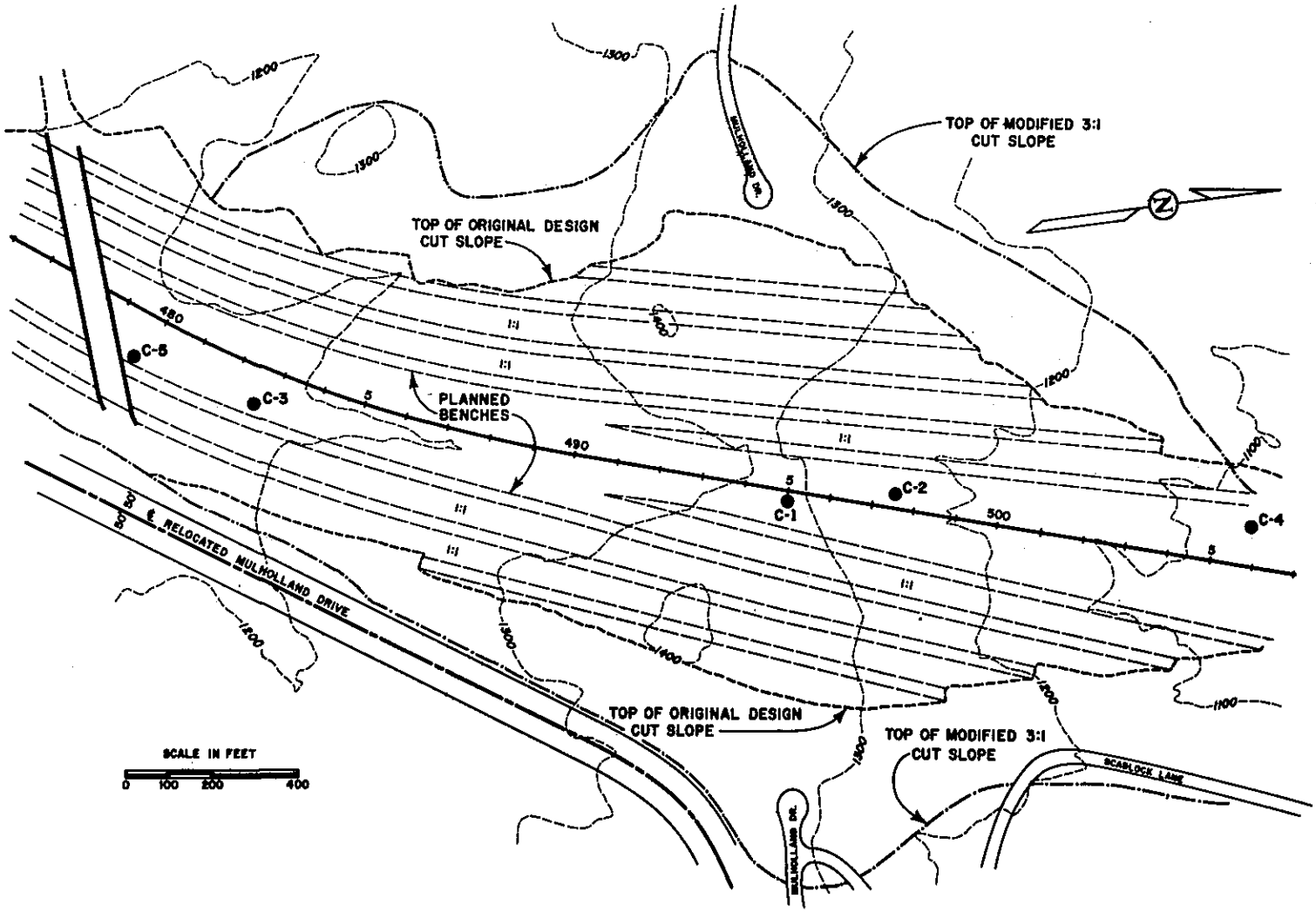

Fig. V-5 — Plan showing original and modified slope design of Mulholland cut.

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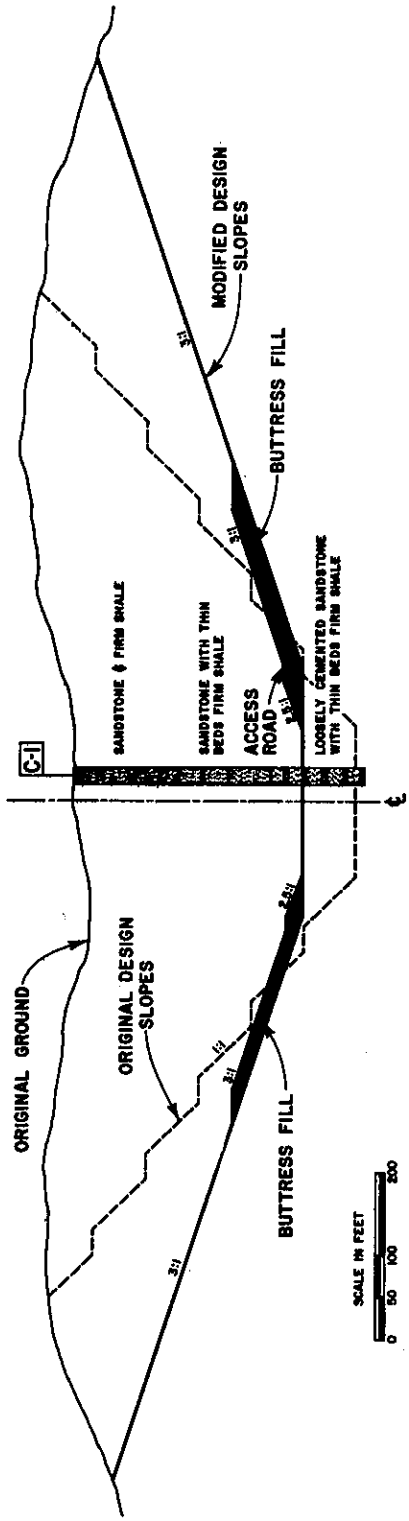

Fig. V-6 — Typical section of Mulholland cut showing original and modified design.

(1) Geological and soils reconnaissance of the area indicated that the formations in the cut probably were largely laminated sandstones and shales. In view of the great depth of the cut and its proximity to urban developments, it was decided that a number of deep borings should be made.

(2) In 1955, starting August 11, five borings were made in this cut area. The locations of the borings are shown on Fig. V-5. Supplementing the data obtained in the actual drilling operations were electric logs of all of the borings. The foundation conditions are illustrated by an abbreviated summary of hole C-1 on Fig. V-6. The upper 100 ft consisted of beds of firm shale of variable thickness and loosely to moderately cemented sandstone. The next 100 ft was made up of loosely to moderately cemented sandstone with an occasional thin bed of firm shale. From 200 ft to the bottom of the boring (333 ft) the materials consisted of laminated firm shale and loosely cemented sandstone. The dip of the formations generally was 15 deg to 20 deg., and the strike was generally normal to the roadway. The water table was recorded 120 ft above the proposed grade in this boring. In general the sandstones were softer than had been anticipated from preliminary investigations, but appeared to be relatively stable. Slight variations in the strike resulted in a slight dip, less than 3 deg., into the roadway prism in the vicinity of Station 501 to 502.

(3) After a review of the boring logs and the geological history of the area, it was recommended that this cut be constructed with 1-1/2:1 slopes with 30-ft benches at 60-ft vertical intervals. Subsequently a large excavation was made in this area to obtain borrow for another highway project. This excavation stood on a 1:1 slope without any serious failures. It was then decided that the Mulholland cut could be designed on 1:1 cut slopes benched at 60-ft vertical intervals.

(4) On July 25, 1960, a contract was let for the construction of a portion of the San Diego Freeway which included this cut. Of the 7,900,000 cu yd of roadway excavation in the contract, the major part was in this single large cut.

(5) Excavation of this cut was started from the north end and taken nearly to grade. The plan was to progressively push the excavation further south into the hill. It so happened that the slight dip of the formations previously noted was toward the excavation that was being made. The major slide which is described here involved the mass to the west of the excavation, which slipped toward the excavation with a rotary motion, most of the movement being longitudinal.

(6) Local slides of a minor nature developed during early progress of this contract, getting progressively worse as the excavations progressed. By October 1961, cracking developed on the east side of the Mulholland cut which forced the owner of an adjacent home to leave his home on Scadlock Lane. Additional cracking in the street and adjacent properties on the west side of the

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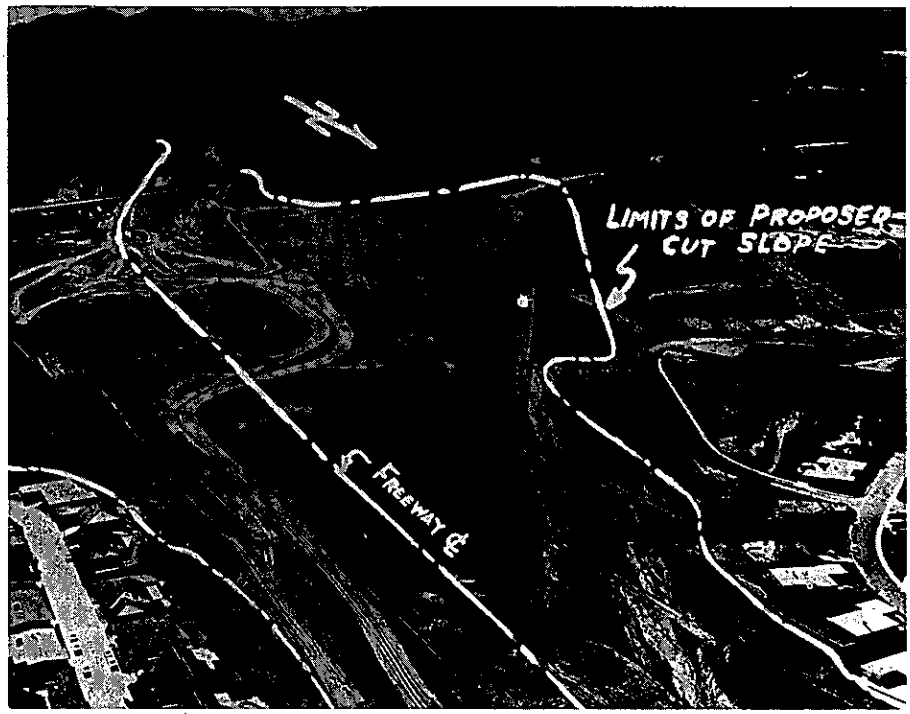

Fig. V-7 — Aerial view of Mulholland showing failure of partially completed cut and limits of modified cut slope.

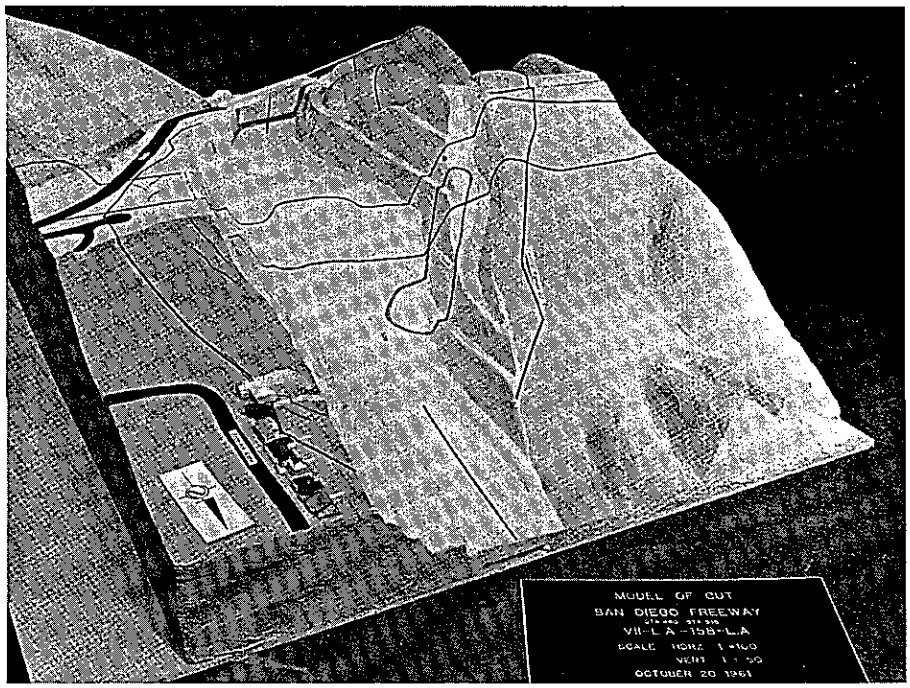

Fig. V-8 — Photograph of Mulholland cut model showing status of excavation at time of failure.

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cut indicated a slide potential of major proportions. Accordingly, the contractor suspended operations in this portion of the cut and moved his equipment to higher ground.

(7) Minor cracking had occurred in the east side of the cut as early as July 1961, but this was not considered serious until early in October when cracks began to develop in the floor slabs of the private dwellings on Scadlock Lane, as noted above, and more definite cracking occurred through the contractor's yard and in the pavement of old Mulholland Drive. The extent of the slide may be seen in Fig. V-7. To aid in analyzing the geological conditions and the slide movement, a geological model was constructed (see Fig. V-8).

(8) A series of investigations by the California Division of Highways District Office and Headquarters Materials and Research Department, and by consulting engineering geologist, Dr. Arthur Cleaves, were conducted late in October 1961. These investigations indicated that the slide movement was in the form of a block glide, the slippage occurring on the gently dipping siltstone bedding planes that slopes toward the excavation. A study of this slide indicates that it probably was caused by a combination of factors, of which the following are believed most important: (a) a slight dipping of the formations into the roadway excavation, (b) the relief of stresses within the earth masses caused by the unloading, and (c) the breakdown of shale into clay by the slide movement, producing very slippery planes. There was nothing observed in the original explorations to indicate that a serious slide potential existed. The stresses set up by the tremendous earth mass evidently exceeded the strength, which caused the movement to start, setting off the cycle described above. Following these investigations a number of alternate corrective treatments were considered including slope flattening, use of buttress fills, raising the grade, shifting the line to the west in conjunction with slope flattening and buttress fills, tunnel construction, and split roadways with steepened grades. Estimated costs of the corrective treatments varied from $2,500,000 to $38,000,000 (the highest being for bored tunnels).

(9) The alternate selected made the following changes (see Fig. V-6):

(a) Grade of road raised 50 to 60 ft through the main cut area.

(b) Grade rate of road steepened from 4-1/2% (design rate) to 5-1/2% for a distance of about one mile.

(c) Slope in slide area flattened to 3:1.

(d) Buttresses constructed at base of slopes to a height of approximately 80 ft and a face slope of 3:1. The base width of the west buttress 75 ft and the east buttress 50 ft.

(10) It is evident from the experience during construction that the cut slopes, as designed, were not stable. The project has been completed, and during the five years since completion the cut has been in a satisfactory condition.

B. Design of Embankment Foundations

1. Basic Considerations

a. The construction of highways over weak, compressible soil presents some of the more difficult problems in Soil Mechanics. When highway construction was relatively small business, rule of thumb and dependence upon individual, personal experience were reasonably adequate; with expansion of highway construction into the biggest construction project of all time, however, existing problems have increased in importance. High-speed, wide freeways, mushrooming in areas where low-speed narrow roadways previously had been adequate, require greater emphasis on the rational soil mechanics methods. The best use of these methods depends upon observing the actual behavior of projects which have been designed and constructed. By reviewing existing projects, the adequacy of the methods that were used can be evaluated and if we are observant, improvements can be made in the use of these methods.

b. If embankments are constructed over foundation soils having insufficient strength to support the added load, shear failure or slipouts may occur, or the underlying soft material may be displaced outward by plastic flow. If the foundation soil is highly compressible excessive settlement of the embankment may occur, resulting in damage to or destruction of the pavement, damage to structures, or hazards to traffic due to distortion of the profile and cross-section of the roadbed. Such settlement may occur even if the strength of the foundation soil is adequate to support the load without shear failures.

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2. Purpose of Soils Investigations

Soils investigations should be made to determine (a) whether the foundation soil will have sufficient strength to support the proposed embankment, and (b) the magnitude and rate of settlement which will result from the embankment load. Following is a discussion of the methods that often are used in designing embankments over unstable foundations.

3. Soil Mechanics Restrictions

A general acquaintance with laboratory tests that measure strength and compressibility may give the impression that stability and settlement analyses can be applied to all embankment foundations, using laboratory test data and soil mechanics methods. Unfortunately this is not always true, as these tests can be performed only on relatively fine-grained soils, and the analyses are predicated on the soil being homogeneous and isotropic within definite layers. Where the foundation soil is highly heterogeneous, contains considerable coarse particles, and where the soil is saturated only in strata adjacent to aquifers, a rigorous analysis based on laboratory test data usually is not possible. On the other hand, where the foundation soil is typical marsh deposits consisting of relatively homogeneous, saturated fine-grained soils, the methods of soil mechanics can be applied with a high degree of confidence. This latter condition will be considered first.

4. Construction Over Sand

If the foundation soil is cohesionless sand, considerable settlement may occur if the soil is in a very loose state, but such settlement usually takes place rapidly as the embankment is constructed, and post-construction settlement generally is negligible. There is seldom any risk of shear failures if the soil is free-draining. Silty sands and extremely fine sands consolidate more slowly, and a surcharge or waiting period may be required to prevent objectionable settlement after structures and paving are completed. The laboratory analyses will indicate whether any such special treatments will be beneficial.

5. Construction Over Weak Soils

a. Where embankments are to be constructed over saturated cohesive soils, both the strength and the settlement characteristics must be investigated. Several alternate methods of construction are considered when the soil is so weak that it will not support the proposed embankment: (1) if the weak soil is less than 15 ft thick, the most positive treatment is to strip the unsuitable material and replace with stable material (if the stripped area cannot be dewatered the backfill should be granular material); (2) when the depth of the weak soil layer is greater than 15 ft or 20 ft, stripping is usually not feasible or economical, but it may be possible to construct the proposed embankment with properly designed berms or toe support embankments; (3) vertical sand drains may be installed to accelerate settlement and produce an increase in strength during construction; (4) if it is not economical to construct embankment by any method it may be necessary to use viaduct or structure, founded on piles, instead of embankment.

b. The most economical and effective berm design can be determined by stability analyses. But this type of construction does not reduce or prevent settlement.

c. Structure is generally more costly than embankment, even if extensive foundation treatment and special design and construction control of the embankment are necessary. Occasionally this is the most practical solution. One such project is discussed later in the section on "Examples of Foundation Designs."

6. Use of Vertical San Drains

a. Proper design of vertical sand drains requires comprehensive testing and analysis. Vertical sand drains are erroneously considered by some personnel as: (1) a panacea for all foundation troubles; (2) entirely ineffective under any or all conditions; or (3) to be used as a last resort when construction by any other method is impossible. Because of these misconceptions a brief discussion of vertical sand drains is given later.

b. Saturated soils can compress or consolidate as loads are applied only as water escapes from the soil pores. The permeability of fine-grained cohesive soils is so low that the pore water escapes very slowly, especially if the drainage path is long, as would be the case when the compressible layer is thick and drainage is possible only in a vertical direction.

c. Installation of vertical sand drains not only shortens the drainage path, but also per-

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mits movement of water horizontally as well as vertically. Most marsh soils are somewhat stratified, with greater permeability in a horizontal direction than normal to the bedding. When an external load is applied to a saturated cohesive soil the load is transmitted to the pore water and excess hydrostatic pressure develops. Under this condition the strength of the soil is very low. As the pore pressure is dissipated by drainage the shear strength of the soil improves. When vertical sand drains are installed in thick layers of compressible soil the rate of consolidation and concomitant increase in shear strength will depend largely on the spacing of the drains and the radial permeability of the soil.

d. The soil permeability in a vertical direction can be determined in the laboratory consolidation test, but the radial permeability as determined by field installation of piezometers is a more reliable indication of the probable actual rate of drainage. Therefore, field permeability measurements generally should be made before designing vertical sand drain installations.

e. The magnitude of total settlement due to consolidation of foundation soil can be calculated with reasonable accuracy, but estimates of the rate of settlement are much less precise. This is due primarily to the inaccuracies in determining actual average permeabilities and lack of knowledge of the effects of smear and disturbance during construction of the vertical sand drains.

f. Careful control of the drain installation and rate of embankment placement are important. On vertical sand drain construction projects, settlement devices, piezometers, and heave stakes are usually installed to observe the behavior of the foundation soils.

g. Actually, vertical sand drains are suitable for only a very small percentage of projects where embankment foundation treatment is required and these are usually confined to flat areas with highly compressible soil.

h. Most embankment slipouts are caused by a zone of weak material in the foundation soil or by the presence of groundwater which produces excessive hydrostatic pressure. There is usually a combination of the two conditions. Prevention of the embankment failures can be effected by removal of the weak foundation soil, by adequate subdrainage or by a combination of the two. For the vast majority of cases where foundation stability is a problem and especially in sloping terrain, a combination of stripping, subdrainage, or stabilization trenches are more appropriate than sand drains.

i. If the zone of weak soil is at the ground surface and is relatively shallow, the most effective treatment is to strip the unsuitable soil. If groundwater is present, or may occur in the future, the stripped area should be blanketed with permeable material and a perforated pipe installed before the embankment is constructed. Underdrains are frequently a part of subdrainage treatment, and the number and spacing of these will depend upon the area to be drained and the slopes of the stripped area.

7. Stabilization Trenches

a. Where the depth to firm material is great, and where groundwater is present, the most commonly used type of foundation treatment is probably the "stabilization trench." Such trenches should have a bottom width of 8 to 12 ft with side slopes as steep as can be maintained during excavation and backfill. The bottom should be founded in firm material below any aquifers. Depending on the terrain, the system may consist of one trench normal to centerline, a longitudinal trench with one or more outlet trenches, or a T, Y or herringbone configuration. A perforated pipe and a blanket of permeable material is placed in the bottom of the trench, and the blanket is also placed on the side slopes and back slope. The remainder of the trench is backfilled with material from roadway excavation, thoroughly compacted.

b. Stabilization trenches serve as drainage trenches and also provide some increased resistance to slide movement due to the "buttress" action of the mass of stronger material in the massive trench section. Most of these trenches have been effective in preventing embankment slipouts, but there have been a few failures where the trenches were not carried down to firm material, or where the trenches were too widely spaced.

8. Subdrainage Methods

a. The foundation soil may have sufficient strength to support the proposed load when the groundwater table is low, but failure of the embankment might occur if the water table rises due to heavy rainfall or becuase the embankment load has changed the natural flow of subsurface

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water. When this occurs, cohesive soils will lose strength because of saturation, and soils possessing internal friction will lose shear strength due to the buoyant effect in the material below the water table. The damming effect of heavy embankments often causes a marked rise in the subsurface water level under the embankment and upslope from the embankment. Stability analyses will demonstrate that a nominal rise in the phreatic line can cause a marked reduction in the factor of safety against sliding. These facts emphasize the importance of adequate subdrainage of the foundation soils under embankments, particularly sidehill embankments constructed over natural ground having moderate to steep transverse slopes.

b. Berms have been mentioned in connection with embankment construction over marsh areas, where the natural ground is usually nearly level. Berms are seldom feasible on sidehills where the transverse slope is steep, unless the berm is constructed agains a hillside so that it acts as a buttress between the embankment and another hill. In such cases a ravine or depression would be filled in by the berm and some type of drainage structure might be required.

c. Stabilization trenches are considered to be one of the most positive methods of treating wet unstable embankment foundations, but under some conditions they are not feasible or economical. The depth of the lower aquifers may be too great to intercept with the trenches or the topography may be such that a suitable outlet trench cannot be constructed. In some cases subdrainage can be accomplished by installing horizontal drains before the embankment is constructed. Frequently a row of drains is installed into the natural slope just outside the slope line on the lower side of the embankment, and another row installed in the gutter line if sidehill section, or above the upper slope line if thorough embankment. (See D-4 for horizontal drains.)

d. If vertical borings show the presence of several aquifers, separated by impervious layers, it may be advisable to install closely spaced vertical drainage wells or a drainage gallery. The drainage gallery would consist of closely spaced vertical wells interconnected by tunnels at the bottom, with suitable outlets, either horizontal drains or pipes placed in trenches. If it is impractical to interconnect the vertical wells they may be drained by intercepting with horizontal drains. Either type of vertical well is backfilled with permeable material, and preferably a perforated pipe is installed vertically in the center of each vertical drainage well.

9. Submerged Embankments

a. It is sometimes necessary to construct embankments which will be subject to submergence and subsequent drawdown. This condition is fairly common where highways are relocated in reservoir areas formed by dams to be constructed by other agencies. Embankments which are constructed of random excavation, with the usually 1-1/2:1 slopes, are likely to become unstable when drawdown takes place after inundation of the embankment. Unless the embankment material is granular and free draining, the reduced shear strength after saturation may result in shear failure when subjected to the seepage forces caused by drawdown of the water surface.

b. The portion of embankments which will be submerged should be constructed of durable rock which is and will continue to be free draining. Design of the embankment with flat slopes will provide greater stability and is essential unless the embankment material consists of good open-graded durable rock.

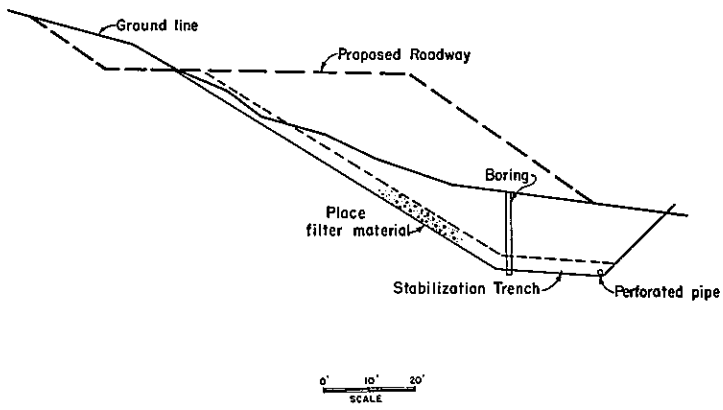

Fig. V-9 — Cross-section of stabilization trench.

10. Examples of Foundation Designs

a. A typical cross-section through a sidehill embankment with a stabilization trench is given in Fig. V-9. A fairly elaborate system of vertical and horizontal drains is shown in Fig. V-10.

b. Examples of four different types of foundations are presented below. The foundation conditions, sampling, testing and analysis, and the recommended foundation treatment are discussed.

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c. Example 1 is Road 10-Sol-37, Sacramento Street Interchange near north city limits of Vallejo and approximately one-half mile east of the Napa River (see Fig. V-11).

(1) A grade separation was proposed to cross a new 4-lane divided roadbed of Route 37 at Sacramento St. A proposed structure lengthof approximately 200 ft will necessitate approach embankments with maximum height of approximately 25 ft.

(2) Four borings were made along the proposed alignment that crosses Route 37, two each at the opposite ends of the structure. Two of the borings were made in areas where the height of embankment is near the maximum, and the other two about 100 ft farther from the proposed structure where the height of embankment would be approximately 17 ft. Two of the borings were made to a depth of 15 ft, and several feet of firm to hard material was penetrated. The other two borings went to depths of 39 ft and 40 ft, and approximately 30 ft of firm to hard material was proven. Unconfined compression and consolidation tests as well as the usual primary classification tests such as gradings, densities, moistures, and Atterberg Limits were performed on representative samples from throughout the borings.

(3) The borings revealed 8 to 10 ft of wet soft compressible peat and silty clay. Several unconfined compression tests gave results in the order of 0.20 T. S. F. It is questionable whether this soil has adequate strength to support the required height of embankment, maximum approximately 25 ft, without a controlled rate of construction or some other means of foundation stabilization. At a depth of approximately 10 ft there is an abrupt change to a much firmer sandy or silty clay, having unconfined compressive strengths 0.80 T. S. F. to 1.95 T. S. F. and even higher strengths at depths of 15 and 20 ft and deeper.

(4) Consolidation test results representing the various layers of soil are as follows:

Depth of Material ft	Consolidation - % of Original Height load (tons per sq ft) of:		
	0.5	1.0	2.0
0 - 7	23	38	50
7 - 13	3	6	9
Below 13	1	2	3

(5) Settlement calculations indicate that the ultimate settlement in the area of maximum height of embankment would be in the order of 4 ft, and most of this settlement would take place in the top 8 or 10 ft. If the upper layer of poor quality soil was stripped, the maximum settlement would be less than 1 ft, most of the settlement would occur during construction, and the embankment above original ground could be built by the ordinary construction methods and the standard specifications. The underlying firmer soil does have sufficient strength to support the required height of embankment.

(6) After the analyses of all pertinent data, it was recommended that the upper 8 to 10 ft of soil, which is soft compressible peat and silty clay, be stripped for the full width of the embankment, and along the line to a point where the existing ground rises and firmer soil is encountered. This would mean stripping for a length of approximately 1000 ft exclusive of the structure. The exact limits of the stripping should be determined in the field during construction. As the groundwater in this area is at the ground surface, fluctuating with the tide, the stripped area should be backfilled with a pervious, granular material. The embankment above original ground could be built by the ordinary construction methods and the standard specifications. The proposed freeway has not yet been constructed through this area (1967).

d. Example 2 is Road 10-Sol-21, Sta. 62 - Sta. 79; proposed 1600-ft length of embankment 24 ft to 30 ft high across marsh land in the Benicia Arsenal (see Fig. V-12).

(1) The area under investigation is reclaimed tidal land surrounded on three sides by hills and on the fourth by the Suisun Bay. Part of the area is used as a storage basin for the surface runoff from the surrounding hills with a drainage canal and tidal gates to control the flow of water into the Suisun Bay.

(2) The proposed line crosses approximately 1600 ft of marsh land, also two railroads, one at each end, and the above mentioned drainage canal. Approximately 70 ft to the east and running parallel to the highway right-of-way is a high level steel and concrete railroad trestle.

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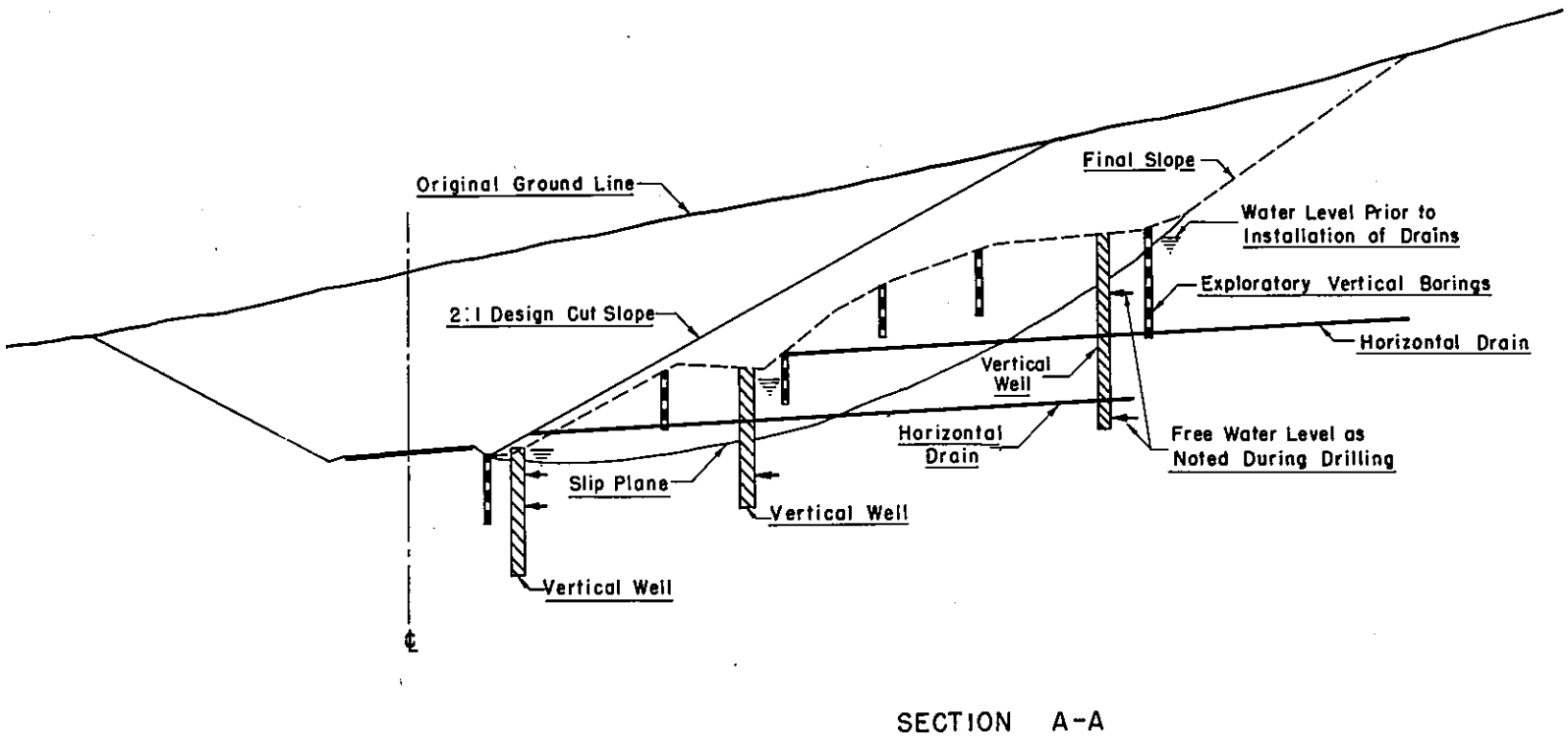

Fig. V-10 -Slide correction by horizontal drains in combination with vertical drain wells.

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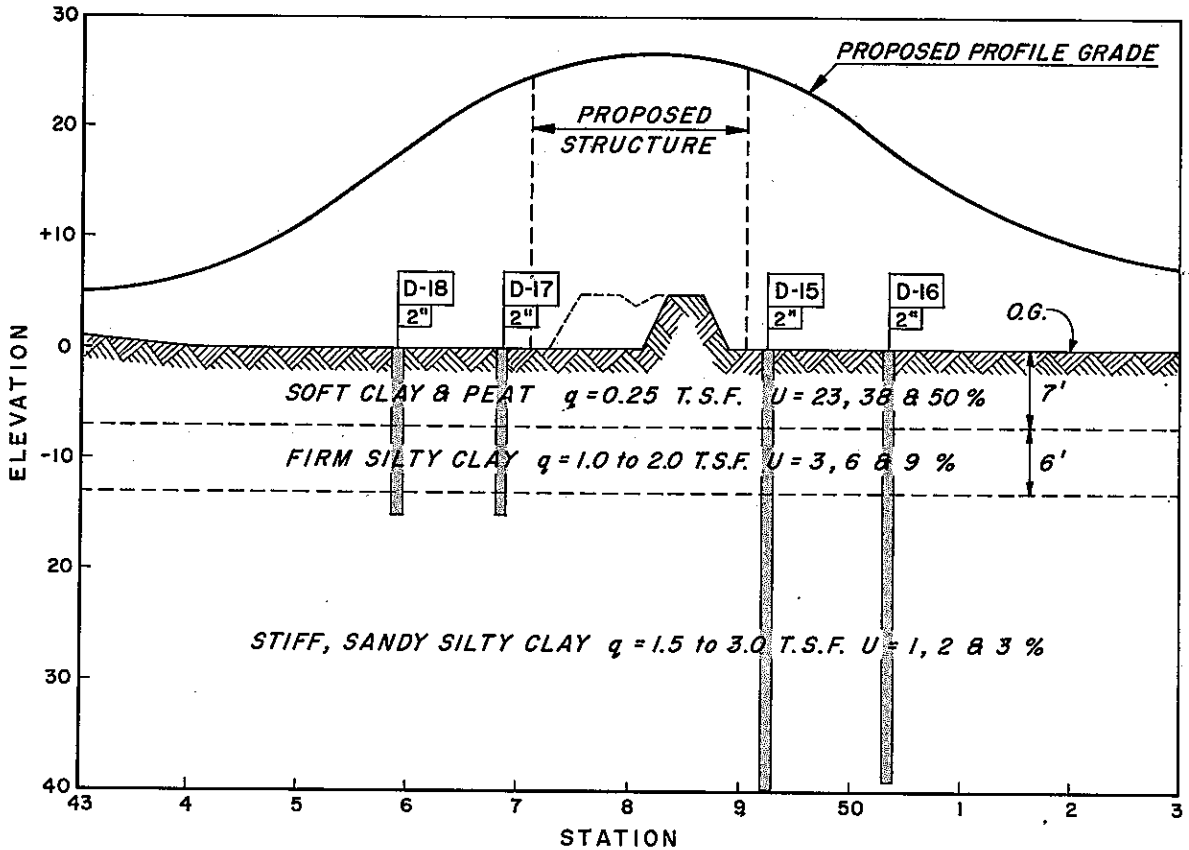

Fig. V-11 — Profile of foundation investigation at Sacramento Street interchange, Road 10-SOL-37.

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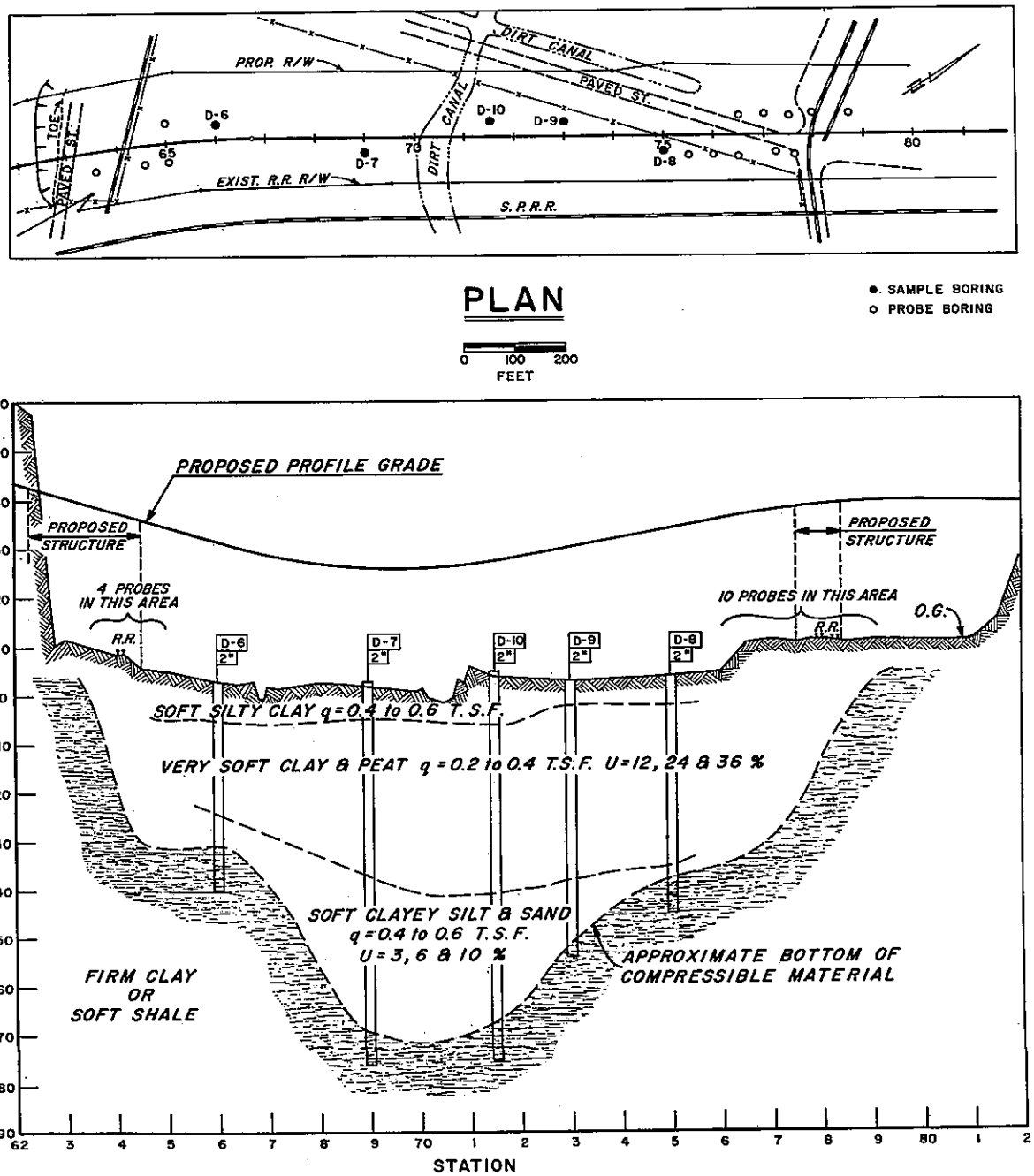

Fig. V-12 — Plan and profile of foundation investigation at Benicia Arsenal, Road 10-SOL-21.

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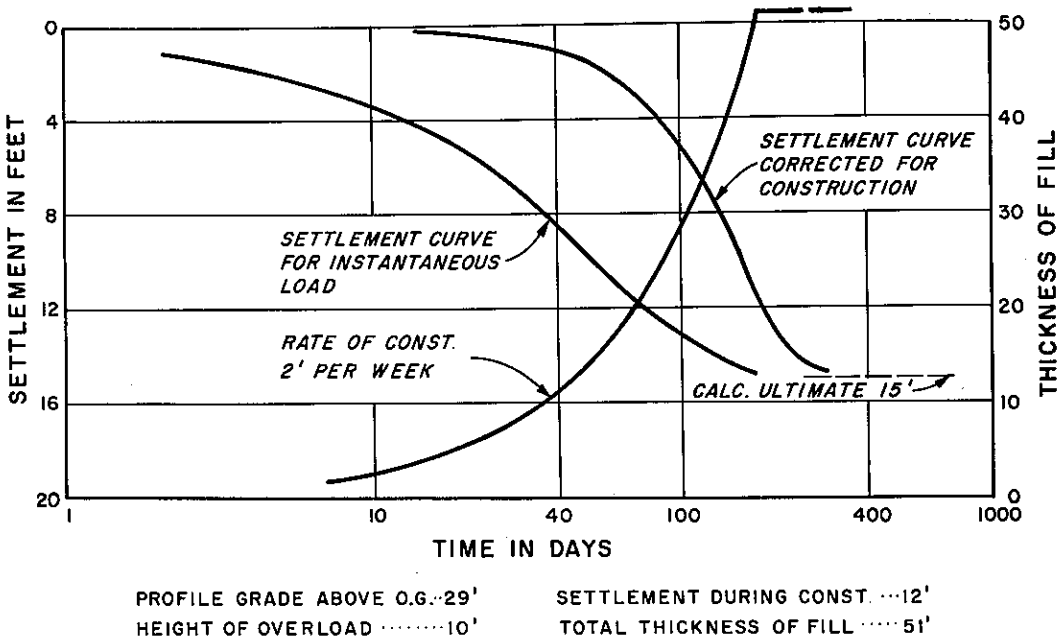

Fig. V-13 — Estimated settlement log time plot for embankment construction in sand-drain area (18-in. sand drains... 8-ft centers) at Benicia Arsenal, Road 10-SOL-21.

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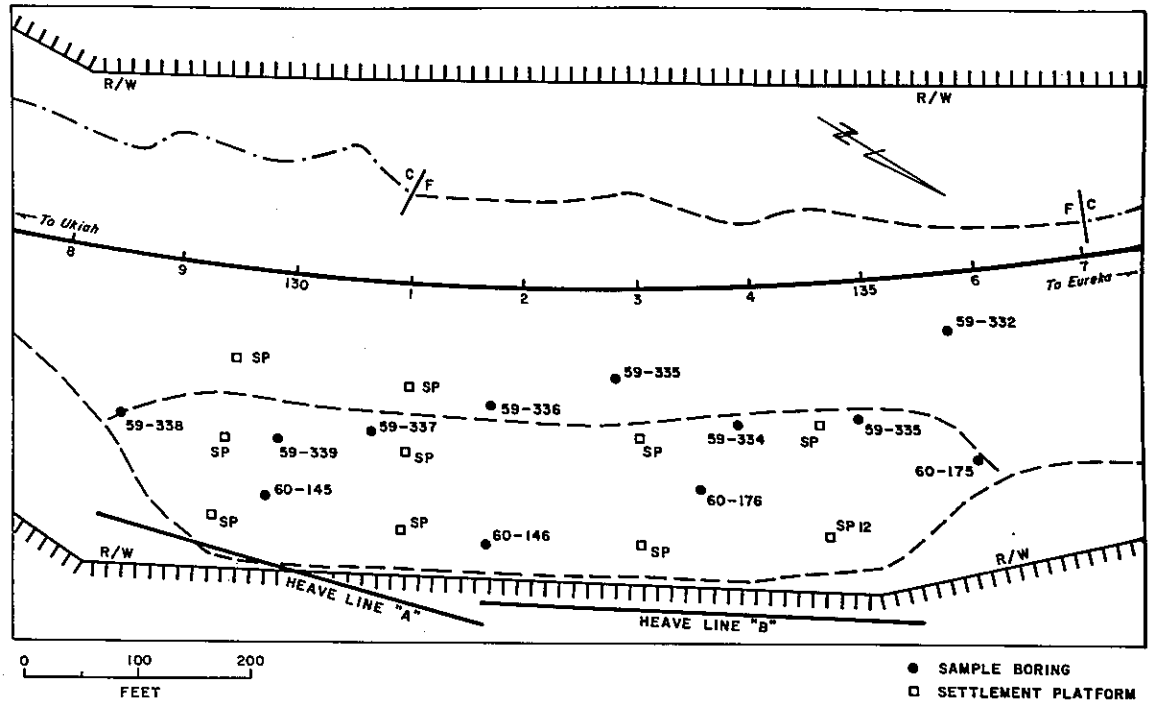

Fig. V-14 — Plan view of foundation investigation on Road 01-HUM-101.

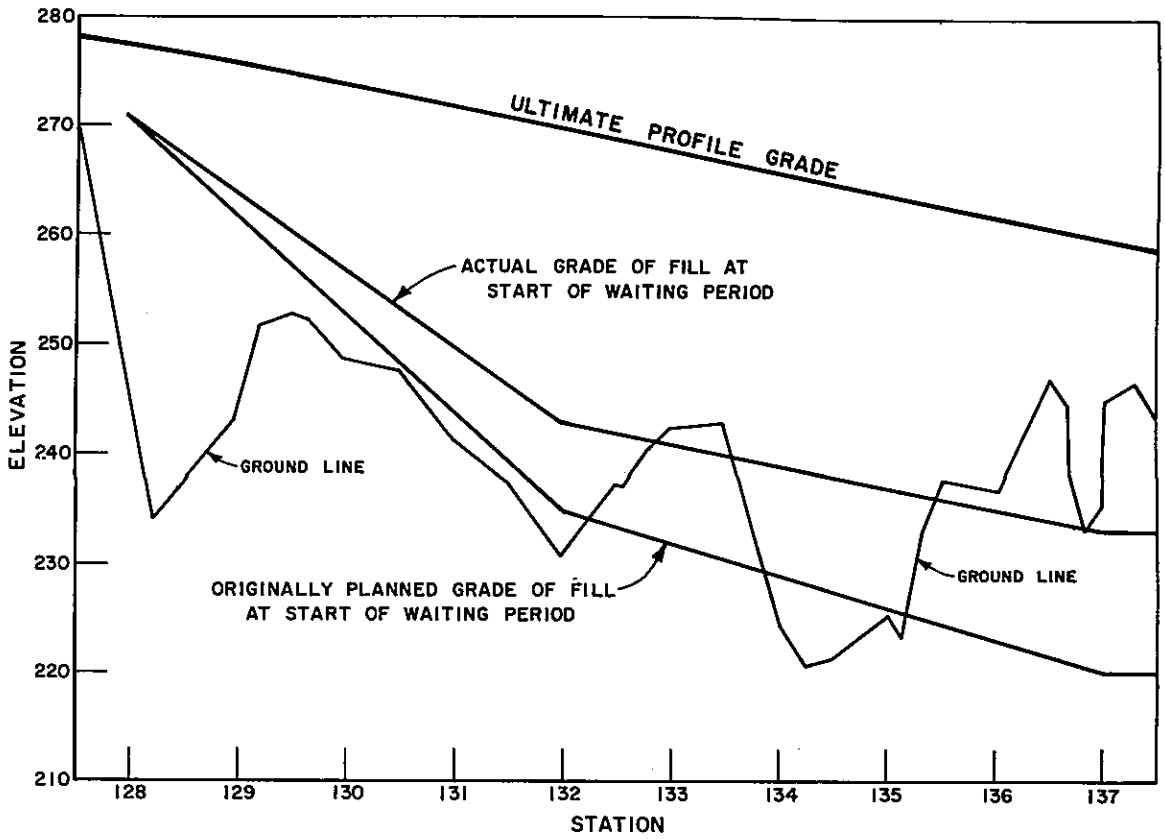

Fig. V-15 — Profile view of foundation investigation on Road 01-HUM-101.

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(3) Five 2-in. sample borings and 14 probe holes were made in the area. It was felt that five borings would furnish sufficient samples for the testing that would be necessary in this investigation. The additional 14 probe holes were made to determine more definitely the limits of the soft compressible material. The borings varied in depth from 40 ft to 80 ft.

(4) The borings and test data show a bed of soft compressible clay or silty clay with scatterings of peat or organic matter in the top 20 ft to 30 ft. Near the ends of the line, as it crosses the marsh, the soil – still soft and compressible but without the organic material — is approximately 30 ft thick and increases in thickness to approximately 70 ft at the center. The material is very soft throughout this depth. Several unconfined compression tests showed values less than 0.20 T. S. F. There were only two tests that showed values above 0.5 T. S. F., most of the figures being between 0.2 and 0.4 T. S. F. Below the soft compressible material, a firm brown shale was found in all the borings. Consolidation figures of 12%, 24% and 36% under loads of 1/2, 1 and 2 T. S. F. are representative of the highly compressible material while the slightly firmer material above the shale have consolidation figures of 3%, 6% and 10% at comparable loads.

(5) Early in the field explorations and testing it became apparent that sand drains would have to be considered in the analyses and foundation treatment recommendations. In addition to the field sampling and probing, piezometers were installed at several locations and depths to determine the field or horizontal permeability of the soil. The field permeability rather than the time factors in the one-dimensional consolidation theory is used in settlement calculations involving sand drains.

(6) Three methods of crossing the marsh were considered in our analysis: (a) the installation of sand drains, (b) total or partial stripping of the compressible material, and (c) a structure. The following is a discussion of the findings.

(7) The material had adequate strength to support an embankment 8 to 10 ft high. An embankment in excess of this height would require some increase in strength of the foundation soil by consolidation. The required height of the embankment varies from 24 ft at the center of the marsh to 30 ft at the extremities. Settlement would be in the magnitude of 14 to 17 ft at the center of the marsh and 8 to 12 ft near the ends. The lesser settlement would occur under the higher part of the embankment as the depth of the compressible material has decreased to approximately 30 ft. Approximately 50 ft of embankment would be necessary to compensate for the settlement and provide a 10-ft surcharge (see Fig. V-13).

(8) In order to provide adequate foundation strength to support the embankment during a construction period of any reasonable length of time it would be necessary to accelerate the rate of settlement by the use of 18-in.-diameter sand drains on 8-ft centers. A construction program which would include a controlled rate of loading of 2 ft per week, two waiting periods, (one during construction and one while the surcharge is in place) and a berm 50 ft wide and 10 ft high on both sides would be necessary if a stable condition is to be maintained in the foundation soils.

(9) There were risks and numerous uncertainties involved in the calculations and analysis to determine: (a) the rate of loading that is compatible with the increase in strength due to consolidation to produce a stable foundation condition, (b) the amount of settlement that will occur during and after construction, (c) the possibility of damaging the railroad trestle, and (d) the difficulty in providing surface drainage for the area.

(10) It was the opinion of the soils engineers that the risks and uncertainties involved were too great to construct an embankment at this location, and it was recommended that the area be spanned with a structure. This structure is now complete and the freeway has been in service for about one year (1967).

e. Example 3 is Road 01-Hum-101, between 1/2 mile south of Sylvandale and Myers Flat (see Figs. V-14 and V-15).

(1) This investigation was conducted to evaluate foundation conditions in the area of a proposed sidehill embankment and to determine the sequence of construction operations necessary to produce a stable roadway. The planned embankment would toe on a flat area lying between the hillside and the bank of the Eel River which extends along the right side of the project. Profile grade called for an embankment 1000 ft in length with a maximum height of some 80 ft above the flat. The original ground slopes toward the right on a grade of about 1-1/2:1.

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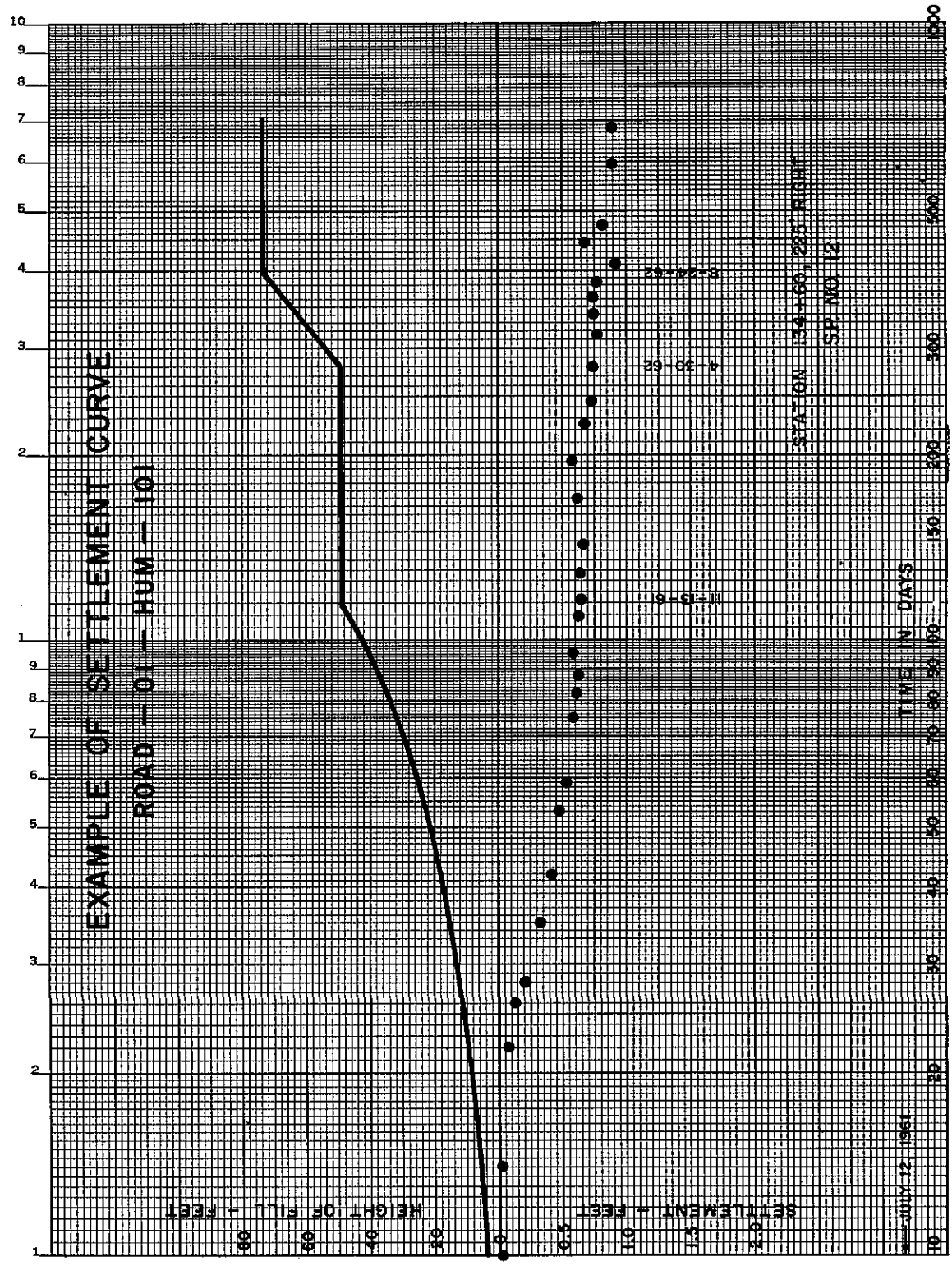

Fig. V-16 - Typical settlement log time plot on Road 01-HUM-101.

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(2) A total of twelve 2-in.-diameter borings were drilled in the embankment area. Eight of the borings were spaced at intervals of approximately 100 ft along the hillside slope 50 to 200 ft right of the proposed centerline. The additional borings were located in the flat area near the toe of the fill. In addition to the 2-in. sample borings a total of eighteen 1-in.-diameter borings were made at scattered points.

(3) The hillside soils consisted of a surface layer of soft to firm clayey silt, 8 to 21 ft thick, underlain by firm weathered shale. Several feet of sand lay between the silt and the shale in some of the borings. The soil in the flat was much softer than the soil on the hillside and consisted of some 5 to 20 ft of soft, organic clayey silt over dense, silty sand and gravel.

(4) The presence of weak soils at the construction site was indicated by the compressive strength tests. Minimum unconfined strengths in the order of 0.3 to 0.5 tons per sq ft were recorded. Although it was recognized that these strengths were inadequate for support of the maximum embankments the test data indicated that the soils could develop the required additional strength if loaded at a controlled rate and given sufficient time to consolidate. Since much of the foundation soil upon which the toe of the sidehill embankment would bear was of marginal strength, the stability of the embankment would be improved by placing a berm or strut along the toe of slope allowing a greater initial loading.

(5) the calculated (theoretical) settlement varied widely from boring to boring. It was estimated that settlements would range from 1/2 to 2-1/2 ft with a differential maximum of 1-1/2 ft in a distance of 100 ft along the line.

(6) Following review of the test data a sequence of construction procedures designed to produce a stable embankment within a schedule of reasonable length was recommended. These recommendations included: (a) employment of a strut (or berm) 25 ft high and up to 100 ft wide along the embankment toe: (b) constructing the embankment in two stages with a waiting period of at least 60 days to be interposed after the embankment reached half its finished height; and (c) use of a controlled rate of loading of not more than 3 ft per week. It was also recommended that reference points (heave stakes), settlement platforms, and piezometers be installed to assist the Engineer in evaluating behavior of the foundation soils during construction. In August 1961, construction of the embankment was begun and halfway level was reached by mid-October. The foundation remained stable as the work progressed. The piezometer installations in the area indicated no significant excess hydrostatic pressure, and a line of reference points established a short distance outside the toe of slope showed little or no lateral movement. In view of the reassurance provided by the data from the control instrumentation, it was decided to increase the height of the first stage of the embankment to about 10 ft over the previously established 40-ft maximum. Following a winter shutdown of several months duration, construction of the embankment resumed, and it was completed in the Spring of 1962.

(7) During the initial 90 days of settlement readings 70% to 90% of the actual settlement occurred. In all instances where data were available, measured settlement amounts proved to be less than the theoretical settlement. At the time of paving, settlement beneath the embankment had virtually ceased with only one-tenth to two-tenths of a foot of additional settlement anticipated (see Fig. V-16).

(8) A complication arose when embankment construction restricted natural drainage from the hillside to the river. Seepage appeared on top of the strut, minor sloughing of material occurred from the main embankment and some of the settlement platforms were flooded. Under separate contract corrective measures to facilitate drainage were taken by installing horizontal drains through the embankment.

f. Example 4 is Road 09-Iny-395, Sta. 542 - Sta. 556; proposed 1600 ft length of embankment over mud at Little Lake (see Fig. V-17).

(1) In 1961 District 09 began the reconstruction of approximately five miles of Route 395 (then Iny-23-G) to bring the route up to modern expressway standards. The project included the crossing of Little Lake in Southwestern Inyo County. Due to possible foundation problems in embankment construction across 0.3 mile of the lake, a foundation investigation was initiated to determine the necessity of special treatment of the foundation soils.

(2) The existing roadway across the lake had been constructed during 1929-30 under Contract No. 29CS1. An embankment supporting tracks of the Southern Pacific Railroad crosses the lake parallel to the highway, about 100 ft to the west. The embankment of the existing road-

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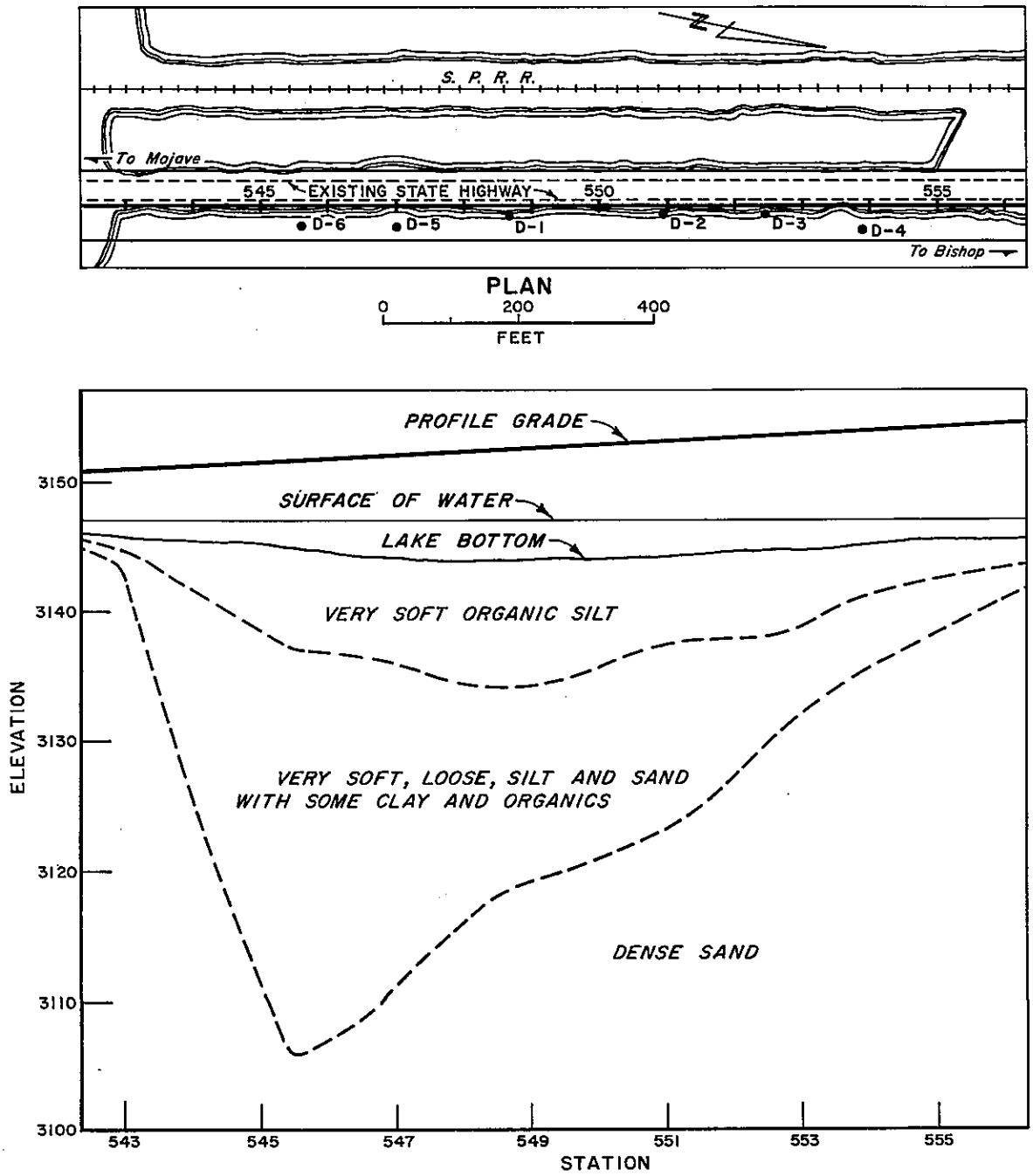

Fig. V-17 — Plan and profile view of foundation investigation on Road 09-INY-395.

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way had been constructed by end-dumping. Some 8 ft of soft sediments were displaced by the rock and earth fill materials near the lake center before stable support was afforded by the underlying soils. The new construction was to widen the existing 35-ft-wide embankment along its east side by approximately three times its original width, providing for two additional lanes (northbound) and extra width for a 16-ft private road outside the right-of-way line.

(3) Foundation exploration for the new embankment began with a series of shallow sample borings drilled in 2 to 4-ft of water from a platform mounted on rowboats. This equipment proved inadequate for obtaining samples to the required depth; and it was supplanted by a more stable barge-mounted Concore drill rig. Subsequently, six 2-in. sample borings, 14 to 41-ft deep were made with the heavier equipment along the line of the new embankment. The material yielded by these borings is classified into three general layers: a very soft, highly compressible, organic, sandy clayey silt comprising the actual lake bed and extending from 2 to 10-ft in depth; a middle layer of the same general soil type but of somewhat higher strength; and an underlying layer of dense, fine to coarse sand located from 8 to 38 ft below the lake bed.

(4) The sediments in the lake bed were very low in strength. Unconfined and triaxial compression tests showed compressive strengths of less than 0.2 ton per sq ft for most of the samples tested.

(5) Experience gained from the previous construction across the lake, augmented by soil analysis and stability determinations, indicated that either of two methods of end-dumping would prove feasible for building a stable embankment. They were: Method One, end-dump in a manner calculated to produce a minimum amount of displacement until a working platform is achieved, then complete the embankment and a surcharge uniformly at a controlled rate. Method Two, end-dump at a very rapid rate to surcharge height, maintaining a shape of fill designed to force maximum displacement of unsuitable material similar to the initial construction in 1930.

(6) Each of these methods had certain disadvantages. Although Method One would create a compacted embankment with a minimum of fill material it entailed a very slow rate of loading and would require a waiting period of from six to nine months to reduce settlement following construction to tolerable limits. Method Two would permit rapid construction since it would force displacement of the weak soils in lieu of loading and consolidating them, and a two to three-month waiting period would probably suffice to effect the required settlement. It was believed, however, that rapid loading would create a mud wave of large proportion, which, extending beyond the right of way, might damage private portions of the lake. In addition, the top 2 or 3 ft of the embankment would require compaction in compliance with Standard Specifications following removal of the surcharge.

(7) It was estimated that if Method One construction was used, about 5 ft of displacement would occur near the center of the lake. Total settlement for the non-displaced foundation soils was estimated as less than 1 ft near the northern side of the lake to a maximum of 3 ft at lake center. It was assumed that if Method Two was employed, about 10 ft of displacement would occur. Based on this assumption settlement would range from 1-1/2 to 2-1/2 ft maximum. The installation of settlement platforms was recommended for measurement of the actual settlement as well as to assist in determining the length of waiting period.

(8) It was decided to construct the embankment using Method One, partly because a source of lightweight material was available for use in this embankment. Construction across the lake was accomplished in two stages. Stage One consisted of building the embankment to profile grade plus a 5-ft surcharge, followed by a waiting period. Stage Two, which involved removing the surcharge, placing the structural section, and paving, was completed after most of the settlement had occurred.

(9) Stage One took place during a five-month period ending on August 16, 1961. It was specified that fill material be progressively end-dumped in 12-ft strips, working outward from the existing embankment, with the inside edge of each successive strip leading the outside edge to avoid entrapment of unsuitable material. The rate of loading was limited to 6 in. per day and was not to exceed 2 ft per week. During the work a mud-wave formed which required removal by dragline before construction could continue. On April 19, 1961, when the partially completed embankment had been built to a height of 9 or 10 ft, four riser-type settlement platforms were installed. The platforms were placed at 300-ft intervals on the embankment between Station 545 and Station 554+50. The embankment was completed to surcharge height on

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May 10, 1961. When Stage Two of the project began on May 7, 1962, settlement had slowed to a rate of a few hundredths of a foot per month. The removal of the surcharge was completed by June 4, 1962.

(10) Measured settlement between the installation date of the platforms and August 15, 1963, ranged from 1 ft to slightly over 5 ft. Some additional settlement occurred during the early stages of the embankment construction before the settlement platforms were installed. This amount was estimated by the Resident Engineer to have been about 2 ft.

g. In summary, applying the theories of soil mechanics to the fine-grained soils in highway foundation problems has shown through experience that the amount of ultimate settlement due to consolidation of foundation soils can be calculated with reasonable accuracy. However, because of the many indeterminate factors used in rate studies, predictions of rates of settlement are less reliable and do not always agree with actual observed settlements. In stability analyses, the principles of soil mechanics can be applied with reasonable surety when representative samples of the critical material can be secured for testing.

C. Solutions to Settlement and Consolidation Problems

1. General Discussion

a. Whenever an excavation is made for a highway, or a mass of earth or rock is placed in an embankment, new patterns of stress are created. The application of stresses to any material leads to the development of strains. Stresses are forces; strains are internal movements that accompany forces.

b. The relationships between stress and strain are called stress-strain relationships. The simplest stress-strain relationships are those which pertain to elastic materials in which stress and strain are proportional and time is not a factor. In some soil problems stresses and strains are approximately proportional. In these cases the elastic theory is a useful tool for the development of theoretical solutions. The equations for stress components in soil masses as developed by Boussinesq typify the possibilities of the theory of elasticity as a tool for the solutions of soil problems. The Boussinesq equations give stress components at any point within an elastic, isotropic, homogeneous soil mass extending infinitely in all directions. They are for point loads. Newmark and others have developed solutions for the stresses produced by loaded areas. In settlement problems we are concerned only with the vertical stresses caused by loads applied to the soil. These stresses are equal to the applied pressure at the upper surface of the soil, and become less at greater depths in the soil mass. In the analysis of the probable settlements of footings, this pressure reduction is significant and should be determined; however, in the analysis of settlement produced by loads that cover large areas, such as large earth embankments, there is very little reduction with depth. In many of these problems the surface stress is applied to the entire depth of compressible soil in calculating settlement.

c. There are two kinds of soil engineering problems in which stress-strain relationships are important: (1) those cases in which there is no danger of the stresses exceeding the soil strength but volume changes may be appreciable and settlement may be important, and (2) those cases in which there is the danger that the shearing stresses may exceed the strength of the soil. The settlement aspect of the problem is being discussed here. Stability is discussed in another section. Elastic theory has been applied to stability problems but these applications are not included here.

2. Application of Loads to Soil Masses

a. When loads are applied to soil masses these loads must eventually be carried by the soil skeleton made up of the solid particles of the soil mass.

b. If a highway embankment is constructed over a gravel or sand foundation the weight of the embankment is transmitted almost instantly to the bulky soil grains. As the pressure builds up at the contacts between the particles slight compression occurs and the entire mass settles. The settlement is usually slight and it occurs rapidly.

c. In contrast, when loads are applied to clays or other saturated, fine-grained soils, part of the load is carried by the water in the pore spaces. As the pressures transfer slowly to the mineral skeleton, the settlement occurs over a long period of time, and it can be very substantial. Countless cases of damage to engineering works by delayed settlement are being experienced in nearly every developed part of the earth.

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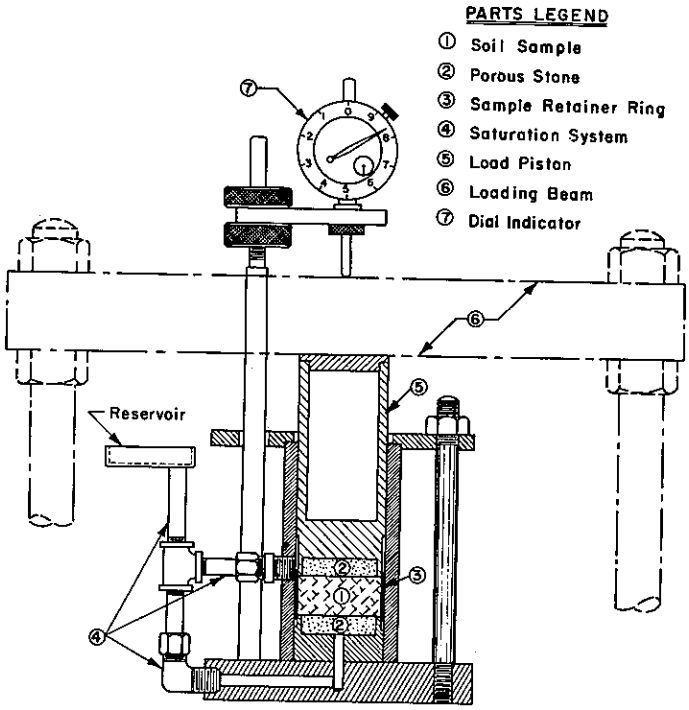

Fig. V-18 — Schematic diagram of one dimensional consolidometer.

3. Determination of Amount and Rate of Settlement

a. In highway work, as in most other construction involving compressible foundations we are concerned with the amount and rate of settlement. The compressibility characteristics of fine-grained soils are determined by testing undisturbed samples in an apparatus called a consolidometer, Fig. V-18. The information furnished by consolidation tests permits an analysis of both the probable amount and rate of settlement of soil foundations. In the consolidation test, soils are allowed to consolidate thoroughly under each of a series of increasing loads. The void ratio, e, is determined for each load by standard laboratory methods, and these values are shown on a pressure-void ratio plot, P-e curve, as shown in Fig. V-19. The change in height of the sample as measured by an Ames dial, or the void ratio is plotted at various time intervals for each load application. A typical time-dial movement plot is given in Fig. V-20.

b. Pressure-void ratio curves provide the basic information needed to calculate the amount of settlement that can be expected in the field due to a given loading condition. The amount of settlement expected is computed from Equation V-1.

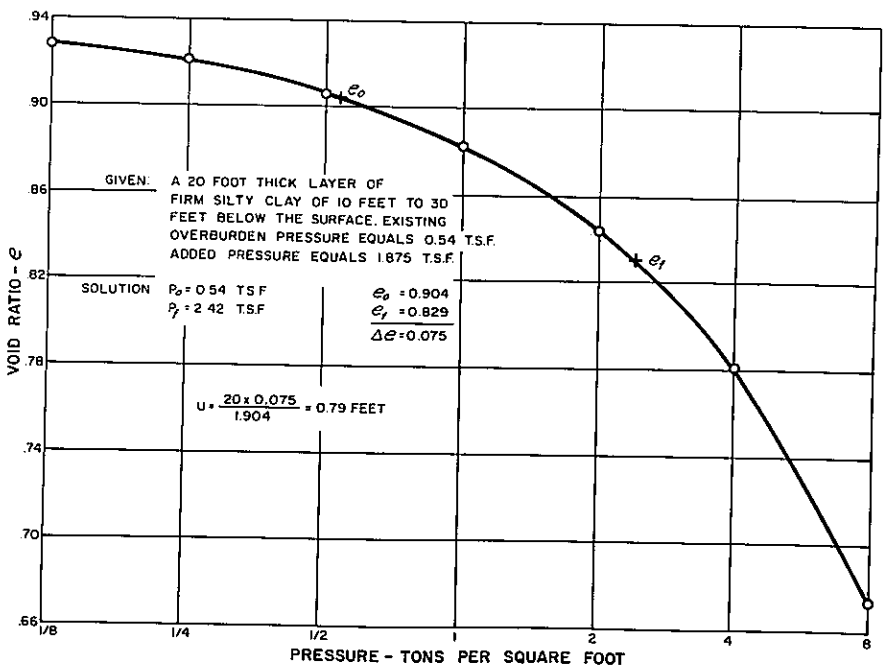

Fig. V-19 — Calculation of estimated ultimate settlement based on typical pressure-void ratio curve.

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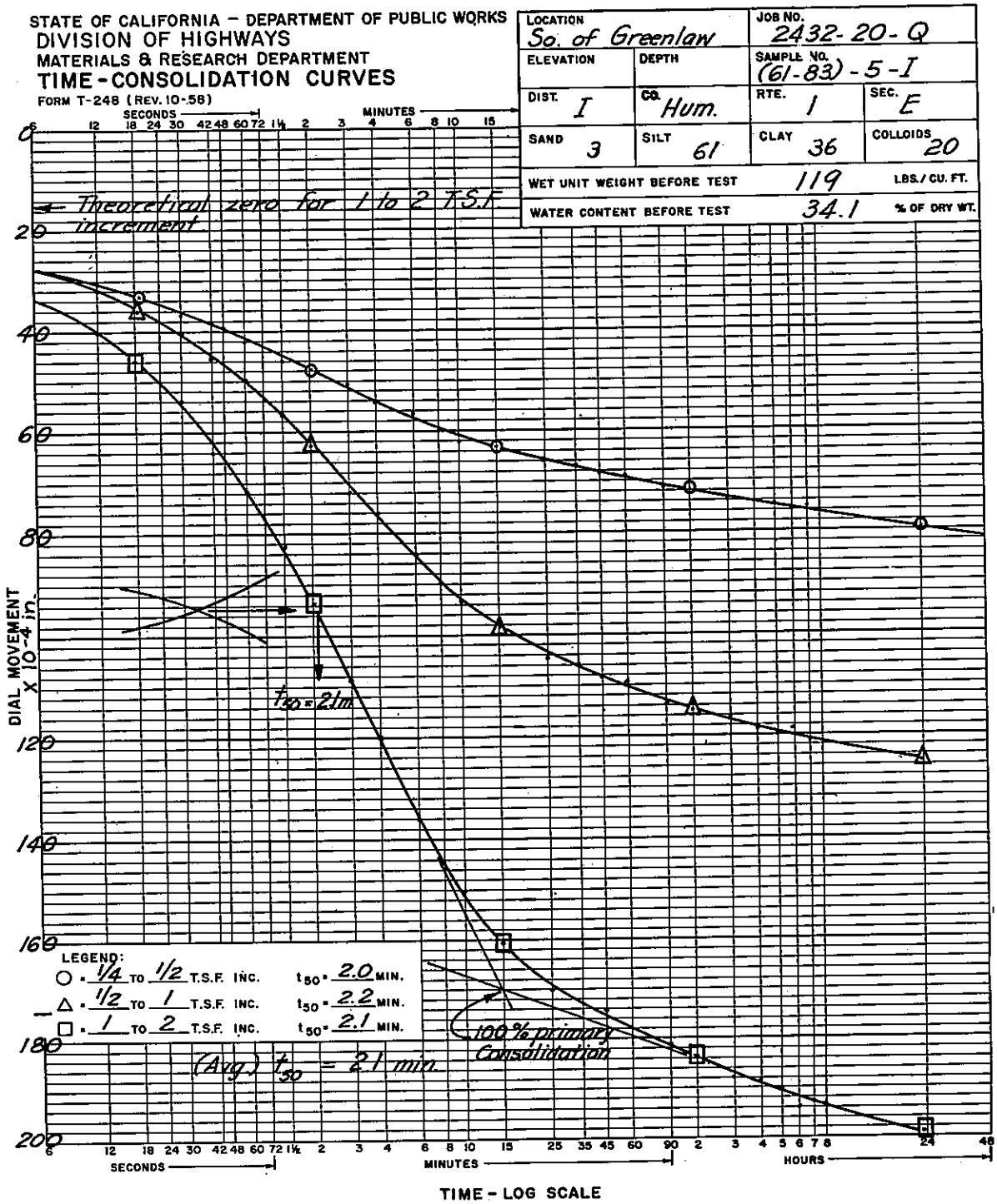

Sample Dial Movement - Time Curves

Fig. V-20 — Typical examples of consolidation as a function of time under different confining pressure.

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$$\Delta H = H_o \frac{\Delta e}{1 + e_o} \qquad \text{(V-1)}$$

In this equation ΔH is the amount of settlement in a soil layer with an initial thickness of H_o. The term Δe is the change in void ratio and e_o is the initial void ratio.

c. In practical problems it is customary to perform consolidation tests on a number of samples taken from the top to the bottom of the compressible layers. If the consolidation characteristics are fairly uniform, an average P-e curve may be applied to the entire layer. If appreciable changes occur in the layer, or if distinctly different characteristics occur, the foundation is separated into two or more layers and the settlement is calculated for each layer. A typical settlement computation is shown on Fig. V-19.

d. To calculate the rate of settlement soils engineers make use of the Terzaghi Consolidation Theory. His mathematical treatment of the flow of water out of loaded saturated soils leads to functions that are analagous to expressions for the flow of heat. The assumptions on which the theory was developed are:

(1) The soil is homogeneous.
(2) Saturation is complete.
(3) There is negligible compression of water and soil grains.
(4) Infinitesimal masses act the same as layer masses.
(5) One-dimensional compression.
(6) One-dimensional flow.
(7) Darcy's Law is valid.
(8) Certain soil properties remain constant during consolidation.
(9) An idealized pressure-void ratio concept is valid.

e. The theory provides a means for applying laboratory consolidation tests to full-size layers of soil in the field. Some of the uncertainties involved in these computations are:

(1) The difficulty of determining the average properties of large soil masses.

(2) The difficulty of determining the distance water must travel to permeable discharge layers.

(3) Variations in properties that occur during the process.

f. By means of various factors developed by the consolidation theory it is possible to analyze the progress of the average percent consolidation of an entire layer, and also variations that may occur within layers. The progress of consolidation is related to a dimensionless term called the time factor, T. The average percent consolidation that has occurred at a given time is called the consolidation ratio, U. How the consolidation ratio varies within a clay layer at various stages of consolidation is illustrated in several textbooks on soil mechanics. *

g. A simplified analysis of rates of consolidation of field layers can be made by applying the principle that the time for consolidation of comparable soil layers of different thickness varies with the square of their thicknesses. Thus:

$$\frac{t_1}{t_2} = \frac{h_1^2}{h_2^2} \qquad \text{(V-2)}$$

In this equation, t_1 is the time for a soil layer with thickness of h_1 to reach a given degree of consolidation. And t_2 is the time for a layer of the same soil with thickness h_2 to reach the same degree of consolidation. To illustrate the use of this relationship assume that a 1-in.-thick consolidation test specimen of a silty clay reaches 90% consolidation in 30 minutes. How much time would be required for a 20-ft-thick layer of the same soil to reach the same degree of consolidation? This time is calculated by substituting the correct numerical values in Eq. V-2, as follows:

*See Fig. 10-9, page 235, D. W. Taylor, Fundamentals of Soil Mechanics.

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$$\frac{0.5 \text{ hours}}{x \text{ hours}} = \frac{(1 \text{ inch})^2}{(240 \text{ inches})^2}$$

The time in hours for the 20-ft layer to reach 90% consolidation:

$$x = 0.5 \text{ hours } \frac{(240^2)}{1^2} = 28{,}800 \text{ hours}$$

$$= 1{,}200 \text{ days}$$

$$= \text{approximately } 3.3 \text{ years}$$

h. Computations of this kind can be very useful in general studies of foundation consolidation. Generally, when theoretical studies are made of rates of consolidation, more sophisticated calculations are made using time factors and other factors relating to the theory of consolidation.

D. Highway Drainage

1. General Comments

a. In a previous portion of this manual, it is stated that soils are accumulations of mineral grains, organic matter, water, and gas; and their strengths are derived largely by the point-to-point pressures that are acting on the mineral skeleton or framework. Moderate amounts of water in soils can contribute to their strength by capillary tension, but when the voids become filled with water the strength of the soil generally is reduced. When the voids of fine-grained soils are filled with free water, part of newly applied load is carried by the water. Since water is relatively incompressible and has no strength, free water in soil masses generally reduces strength. A well-known example of the effect of free water in both coarse and fine-grained soils is buoyancy, which reduces the effective pressure on the soil skeleton to that produced by the submerged weight.

b. Excess water can destabilize soils in three basic ways:

(1) Destroys capillarity, softening compact fine-grained soils, reducing their ability to withstand loads.

(2) Reduces strength by permitting excess pore pressures to exist in the soil.

(3) Produces physical seepage forces.

c. The importance of adequate drainage has been widely recognized as vital to the stability and durability of highways. Following is a brief review of some of the more common, practical methods for controlling and removing soil water. It omits the more specialized methods, such as electro-osmosis, vibroflotation, and chemical treatments.

2. Vertical Sand Drains

a. Vertical sand drains are used for increasing the rate of flow of excess water from saturated clay and peat soils in order to speed up the consolidation process and to shorten the time needed to reach equilibrium. An ideal installation is one in which an embankment settles so completely before paving that no further settlement of any consequence occurs after paving. There have been numerous installations where sand drains have worked remarkably well, and excellent roads have been obtained. There have also been some installations where sand drains have speeded up the process, but consolidation after paving has been rather substantial.

b. Increasing the rate of consolidation will also increase the strength of weak clays and peats and prevent foundation failures that occur when normal methods are employed. When sand drains are installed by forcing a closed hollow pipe or mandrel into soft soils there is a substantial shoving and remolding of the soil which causes a temporary reduction in strength. Usually the reduction in strength that is caused by driving sand drains is regained in a few days to a few weeks. Some notable failures of sand drain installations have occurred when this reduction has been ignored. The remodeling and weakening of soft soils can be largely eliminated by the use of jetting, augering, or comparable methods. Current practice by the California

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Division of Highways prohibits the use of a closed mandrel when installing sand drains, where stability is a problem.

c. Although sand drains can greatly speed up the removal of water from weak, saturated foundations, they should not be considered a cure-all. They should never be used without adequate field explorations and an appropriate soils analysis. They are most effective in draining stratified soils, containing lenses of fairly permeable material that are intercepted by the drains. Sand drains have been installed in more-or-less homogeneous soil foundations of low permeability. Although the consolidation cycle has been reduced to a fraction of what it would have been without drains, it was still so slow that the benefit of the drains was not fully realized. In one case drains were installed in a deep mud which would have continued to consolidate for at least one hundred years. The drain installation reduced the time to 10 or 12 years, but this did not eliminate settlement troubles subsequent to paving.

d. Vertical sand drains have been used quite successfully at numerous locations. Generally, they are used in conjunction with controlled rates of loading and frequently in combination with berms. The loading rate may be limited to 1 or 2-ft per week, and one or more waiting periods may be specified to produce an increase in the soil strength to meet stability requirements. Settlement platforms, heave stakes and piezometers generally are installed in sand drain areas to provide continuous records of the progress of consolidation and to give warning of impending failure so that the rate of loading can be further reduced, if necessary.

e. Vertical sand drains should be used only after their probable effectiveness has been determined by careful soil testing and analysis. The cost of treatment with sand drains should be compared with other methods of construction as economics generally governs the treatment that will be selected. In some cases hazards and uncertainties may rule out this type of construction and require use of structure founded on piles, even though the cost with sand drains may appear favorable.

3. Underdrains and Permeable Blankets

a. Common use of conventional underdrains is for control of groundwater which might cause saturation of the subgrade and structural section, with resulting loss of bearing capacity and damage to the pavement. Underdrains are commonly installed near the hinge point or shoulder area on one or both sides of cuts. The preferred location is 3 to 4 ft inside the hinge point, which allows room to operate backhoes or trenching machines and also avoids undercutting the cut slope face.

b. Where a definite aquifer is evident, underlain by impervious material, the trench should extend down to the impervious layer and thus the underdrain can intercept substantially all lateral flow of water into the roadbed. If no definite aquifers can be identified the underdrain should be of sufficient depth to lower the water table under the roadbed to prevent saturation of the subgrade. If the source of groundwater is several feet below the gutter line an underdrain only 2 ft deep will be completely ineffective, and would be a waste of money. It is believed that in many instances underdrains have not been deep enough. The proper depth usually can be determined only by exploration and visual inspection during both design and construction. The proper depth varies with soil and terrain conditions, and also with the geometrics of the roadbed. Wide roadbeds generally require deeper drains than narrow roadbeds, as can be demonstrated by flow-nets which show a definite rise in water level in areas between widely spaced drains.

c. It should be recognized that underdrains along the gutter line cannot prevent upward movement of water under the structural section in the roadbed. Artesian water, springs, or seeps under the roadbed are not uncommon. Well defined isolated springs can be drained by a system of underdrains under the roadway, with proper outlets into longitudinal conduits along the gutter line. Frequently groundwater seepage or artesian water is not localized, and the subgrade will be wet throughout most of a cut area. When this condition exists a permeable blanket should be placed under the structural section, together with perforated pipe underdrains to collect the water from the permeable blanket. The underdrains are usually required along both sides of the cut, and on four-lane or other wide sections an additional line of underdrain often is required in the median strip.

d. There is evidence that the movement of water through a permeable layer of the type often specified is too slow to protect the subgrade from saturation even if a thick blanket is provided. The permeability of the material could be increased by removal of fines, but a permeable

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material which is open enough to drain the water rapidly from wide sections of a blanket may be so coarse and open that it might become clogged by migration of the subsoil into the voids. This could be corrected by placing a layer of sand over the subgrade, then a layer of coarse material. This kind of double-layer construction is disliked by many highway engineers because of the increased care that is required for its placement. Generally an attempt is made to use a single layer which is fine enough to prevent soil intrusion and coarse enough to remove the incoming water. Achieving both objectives with a single material is a difficult task. Very strict specifications are required if the grading and permeability are to be kept within reasonable limits. The California Division of Highways has made considerable progress in this area with a resultant change in the grading requirements of permeable material, effective with the 1964 Standard Specifications.

e. On the older highway construction "French drains" often were used in lieu of modern underdrains. They commonly consisted of a ditch or V-trench backfilled with broken rock, and contained no pipe conduit. Such drains generally are not approved nowadays, as clogging can readily render an entire installation ineffective in a short time, and pipe collectors are considered a vital part of drains.

4. Horizontal Drains

a. Engineers have known for years that high groundwater in hillsides can trigger slides in cuts and slipouts in embankments. Failures most generally occur during or shortly after periods of heavy precipitation. Cut slopes and embankments that have stood through several mild winters may suddenly collapse during a winter of more than normal precipitation. The cause of these failures is the destabilizing action of free water in the soil mass. A slope may be just barely stable under a given hydrostatic head, but when additional water enters the soil the head is raised to the point where the slope can no longer withstand the pressures exerted by the water, and failure occurs.

b. Horizontal drains have been used for years by the California Division of Highways for the stabilization of slopes. On the whole, this work has been rather successful; however, present practice is to try to predict where failures are likely to occur and stabilize during construction. When this can be done much expense and inconvenience to the public are eliminated.

c. A horizontal drain is a small-diameter well drilled on approximately a 5% to 10% grade into a slope, and fitted with a perforated pipe for the removal of water that enters the drain. Pipes generally are provided for carrying the collected water away to a safe point of discharge.

d. Like other drainage methods, horizontal drains are no cure-all. They should not be installed until investigation has been made to determine the probable effectiveness and economy of this type of treatment. This investigation may consist of vertical borings, visual inspection of the terrain, geologic study, or study of the behavior of a slope or landslide. The first few drains installed at any location may be somewhat exploratory in nature, particularly where the area has not previously been explored with vertical borings. Grades, depths, and location of the later drains should be predicted on the information obtained from the first few drains.

e. At locations where the effectiveness of horizontal drains is questionable a few exploratory drains usually will indicate whether this type of treatment is likely to be effective.

f. At some locations horizontal drains have been used in conjunction with vertical wells. The vertical wells serve to interconnect porous joints or seams that have not been penetrated by horizontal drains, and under some conditions can accelerate drainage. At locations where distinct aquifers are not interconnected the vertical wells generally should be interconnected at the bottom by tunneling or belling. In this manner a combined system will be most effective in removing water and reducing hydrostatic pressures.

5. Seepage Analysis Methods

a. General Discussion

(1) An old-time highway engineer is reported to have said, "There are just six problems that make highway construction difficult, and all of them are drainage." If he was thinking of a stable roadway, he wasn't far from the truth. Water can be blamed (and frequently is) for most of the slipouts, slides, and unstable conditions that plague the highway engineer. In proper quantities, water is an essential ingredient in the construction of highways, but too much water in the wrong places can do great harm. Prevention of the accumulation of water in harmful

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amounts, or the removal of naturally occurring water from soil masses is the function of "highway subdrainage."

(2) Just as with all other fields of engineering, practical experience is the primary ingredient of highway design and construction. Knowledge of soil mechanics principles can be very useful in analyzing and understanding the behavior of soils, but is of limited value without practical know-how.

(3) We have said that the study of the shearing strength of clays is perhaps the most difficult aspect of soil mechanics. Now we say that seepage and drainage are probably among the least understood aspects of soil behavior. These conclusions appear to be true even though the fundamental relationships that govern the flow of water through soils are comparatively simple. The exceedingly slow rates that are encountered in the field are of such small magnitude that they are hard to visualize. In fact, the rates that are possible through some of the "permeable" aggregates that are widely used for highway drainage are much slower than is generally recognized.

(4) The development of the California Water Plan necessitates the relocation of portions of our highways. In some locations, the terrain and alignment are such that the highway embankment may serve as part of the dam or minor fills around a reservoir. Some portions of our highways contain depressed sections, i.e., profile grade is depressed below the existing level terrain. In some cases the natural groundwater level is near the ground surface. An example of this design is where the new Interstate Highway 5 will go through the City of Sacramento between 2nd and 3rd Sts. The highway will be within approximately 200 ft of the Sacramento River, and at times the river will be 20 ft or more above the highway. These are areas where seepage velocity and flow nets are a necessary part of the study for the design and construction of stable roadways. The following notes briefly review the fundamentals of seepage.

b. Darcy's Law and the Flow Net

(1) In 1856, H. Darcy published a treatise describing a concept, which has come to be known as Darcy's Law. This law is expressed by the formula:

$$v = ki = k\frac{h}{l} \qquad \text{(V-3)}$$

The term, k, is commonly called the coefficient of permeability. It has the dimension of a velocity, cm per sec, ft per min, etc. In the above equation, v, is called the discharge velocity. The permeability represents the discharge velocity for a hydraulic gradient ($i = h/l$) equal to unity.

(2) According to Darcy's Law, for laminar flow, the quantity of water which will flow per unit of time through the sample of soil shown in Fig. V-21 is equal to:

$$Q = Av = kiA \qquad \text{(V-4)}$$

This is the more common form in which Darcy's Law is usually expressed.

(3) Darcy's Law is one of the most useful tools of the soils engineer. He can use it for estimating both quantities of seepage and velocities of flow. When estimating velocities he must remember that the product of permeability and hydraulic gradient, ki, is the discharge velocity, which is the imaginary velocity that would act over the full cross-section area of the soil, A, to produce the flow that is actually produced by the average forward velocity in the void spaces. This velocity is called the seepage velocity. It is equal to the discharge velocity divided by the "effective" porosity, or

$$v_s = \frac{ki}{n_e} \qquad \text{(V-5)}$$

The seepage velocity is always greater than the discharge velocity, because the porosity is always less than 1.0. The effective porosity is determined by the amount of pore space through which there is flow of water. If the effective porosity is 50%, for example, the seepage velocity is two times the discharge velocity. If the effective porosity is 20%, the seepage velocity is five times the discharge velocity.

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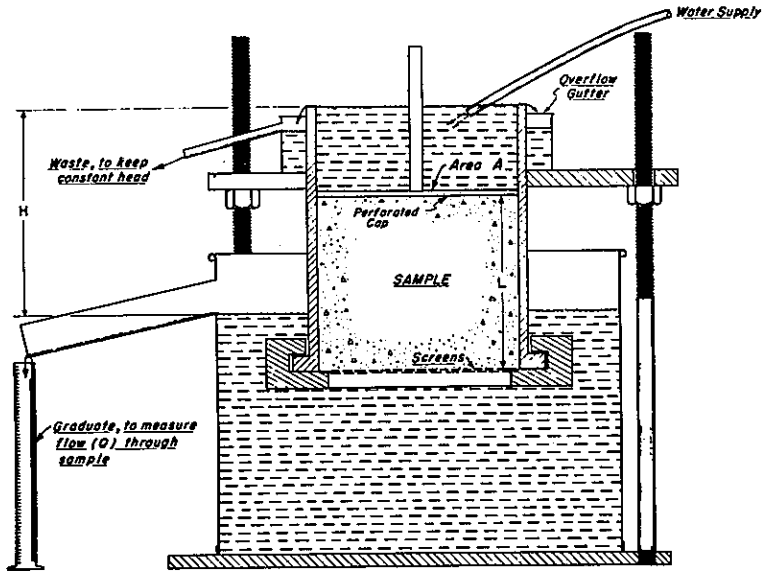

Darcy's law for flow rates under laminar conditions is written:

$$Q = kiA$$

where

Q = rate of flow.
k = coefficient of permeability.
i = hydraulic gradient or head loss (H) over a given flow distance (L).
A = gross cross-sectional area of soil.

Fig. V-21 — Schematic diagram of apparatus for constant-head permeability.

(4) The above relationships can be used for solving many seepage problems. For example, assume that a highway has been constructed through a wet cut in a mountainous area. A blanket of permeable material has been placed beneath the structural section to collect and remove harmful groundwater. This drainage aggregate has been tested in the laboratory at the same density that exists in the roadway, and has been found to have a permeability of 10 ft per day. The porosity of the blanket is 20%. If the drainage blanket lies on a slope of 5%, how fast can water drain down the grade in this blanket to be removed from the roadway?

(5) From Eq. V-5 the possible seepage velocity for the conditions described above is computed as follows:

$$v_s = \frac{ki}{n_e} = \frac{(10 \text{ ft/day})(0.05)}{0.2} = 2.5 \text{ ft/day}$$

Thus, by inserting appropriate values in this equation, one can establish limiting degrees of performance that can be expected for highway subdrainage facilities working under known basic conditions. These solutions are limited to cases involving simple flow conditions. More general solutions involving more complex flow conditions can be developed with the "flow net," which will be described below.

(6) A flow net is a diagram consisting of two sets of intersecting curves: (a) the flow lines, which represent possible paths water can taken in seeping through a soil mass, and (b) the equipotential lines, which show the manner in which the total head is dissipated in the soil mass. At all points on an equipotential line the water would rise in observation wells to a common elevation. Fig. V-22 is an illustration of a simple flow net. It depicts seepage through a homogeneous sand under a row of sheet piles.

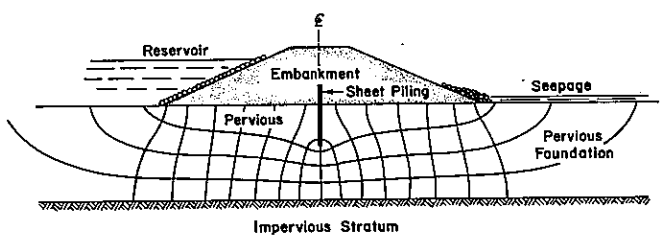

Fig. V-22 — Example of typical flow net as influenced by sheet piling.

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(7) A flow net permits determination of: (a) the head-distribution throughout a soil mass in which water is flowing, (b) the hydraulic gradients throughout the soil mass, (c) the shape factor which influences the quantity of water that can flow through the soil cross-section, and (d) a visual picture of the seepage pattern.

(8) One should not fail to recognize that flow nets can be obtained only for cross-sections composed of idealized soil masses. Flow nets can accurately depict seepage within masses having the properties and dimensions assumed in preparing the soil cross-sections, but real soil masses seldom are as simple and homogeneous as we must assume. If one does not expect unreasonable accuracy from flow nets, they can be of considerable value to the engineer who must analyze seepage problems.

(9) Flow nets are solutions to second degree LaPlacian differential equations. They can be solved mathematically, although this process becomes exceedingly complex, even for simple cross-sections. Flow nets can be obtained by models of several varieties, or they can be obtained by trial-and-error sketching methods. Those who have developed skill in the sketching methods generally find other methods much too cumbersome for general use. Models, however, are very useful devices for depicting the nature of seepage through soil masses.

(10) Darcy's Law and the flow net are two of the most useful tools at the disposal of the soils engineer. When used with discretion and judgment, they can be invaluable assets.

(11) A comprehensive and practical treatise on the subject of drainage has recently been authored by Harry R. Cedergren of the California Department of Water Resources.* It is an excellent guide for the highway engineer in his design of subdrainage facilities.

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*Cedergren, Harry R., Seepage, Drainage, and Flow Nets, John Wiley and Sons, Inc., 1967.

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VI. Construction Control Devices in Soft Foundation Areas

A. General

The construction of highway embankments over marshy or so-called swampy lands presents several serious problems. The soils normally associated with this type of terrain are high in silt and clay content, often high in organics, and saturated with water. When these soils exist in poorly consolidated layers, they are weak and highly compressible. As a consequence embankments built over marshes, without special treatment or control, can result in costly slipout failures during construction and/or settlements which are destructive to the structural section after paving.

There are a number of methods which can be utilized in the design and construction of embankments to obviate these difficulties. Such items as the use of berms, sand drains, controlled rate of constructing the embankment, waiting periods, surcharge, etc., are very effective in overcoming the detrimental characteristics of marshy soils. It must be realized, however, that these methods of treatment are rather costly and will only be used judiciously. Therefore, it follows that the success of a method or combination of methods, used in a given situation, is heavily dependent upon the correctness of many assumptions made on the basis of preliminary foundation investigations.

This leads to the need for the measurement of the effects of embankment load upon the foundation soil, during construction, to assure that the design assumptions are valid and provide a means of anticipating trouble. The following discussion will cover the details of the construction control methods used by the California Division of Highways to accomplish this purpose.

B. Construction Control

1. Four Monitoring Instruments

a. There are four primary instruments utilized in monitoring the settlement and stability of embankments. They are (1) settlement platforms, (2) piezometers, (3) heave stakes, and (4) inclinometers. In the California Division of Highways, Materials and Research Department install these devices at the start of construction, utilizing specially trained and equipped personnel. The installations are made after a written or verbal request from the District or as specified by the contract special provisions.

b. Reading of the instrumentation is normally performed by the resident engineer's forces, at regular intervals, the data being recorded on special manifold forms which are furnished by the Department. All necessary calculations are completed on the project and copies are sent to the Foundation Section of the Department. Plotting of the data is essential to effective, up-to-date analysis. The analysis of data is normally undertaken by the Foundation Section, although the resident engineer may make any plots or studies he feels advisable.

c. Following is a discussion of each of the devices, taken individually, with emphasis upon description, principle of operation, calculations, and analysis. Techniques of installation are not detailed here.

2. Sealed Fluid Level Settlement Platform

a. There are two types of settlement platforms currently used for routine construction control purposes. They are the sealed fluid level and the riser type devices.

b. Fig. VI-1 illustrates a typical installation of a fluid-level settlement platform. The device consists of a standpipe attached to an 18 x 18-in. wood or metal platform which is placed on either the original ground or the "working table" at or near the centerline of the embankment. The standpipe is connected by 3/8-in. plastic tubing to a transparent sight tube mounted vertically in a protective box outside the limits of the embankment. A fluid, usually water*, fills the standpipe, connecting tube and sight tube.

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*Under severe freezing conditions, a light oil or kerosene may be used.

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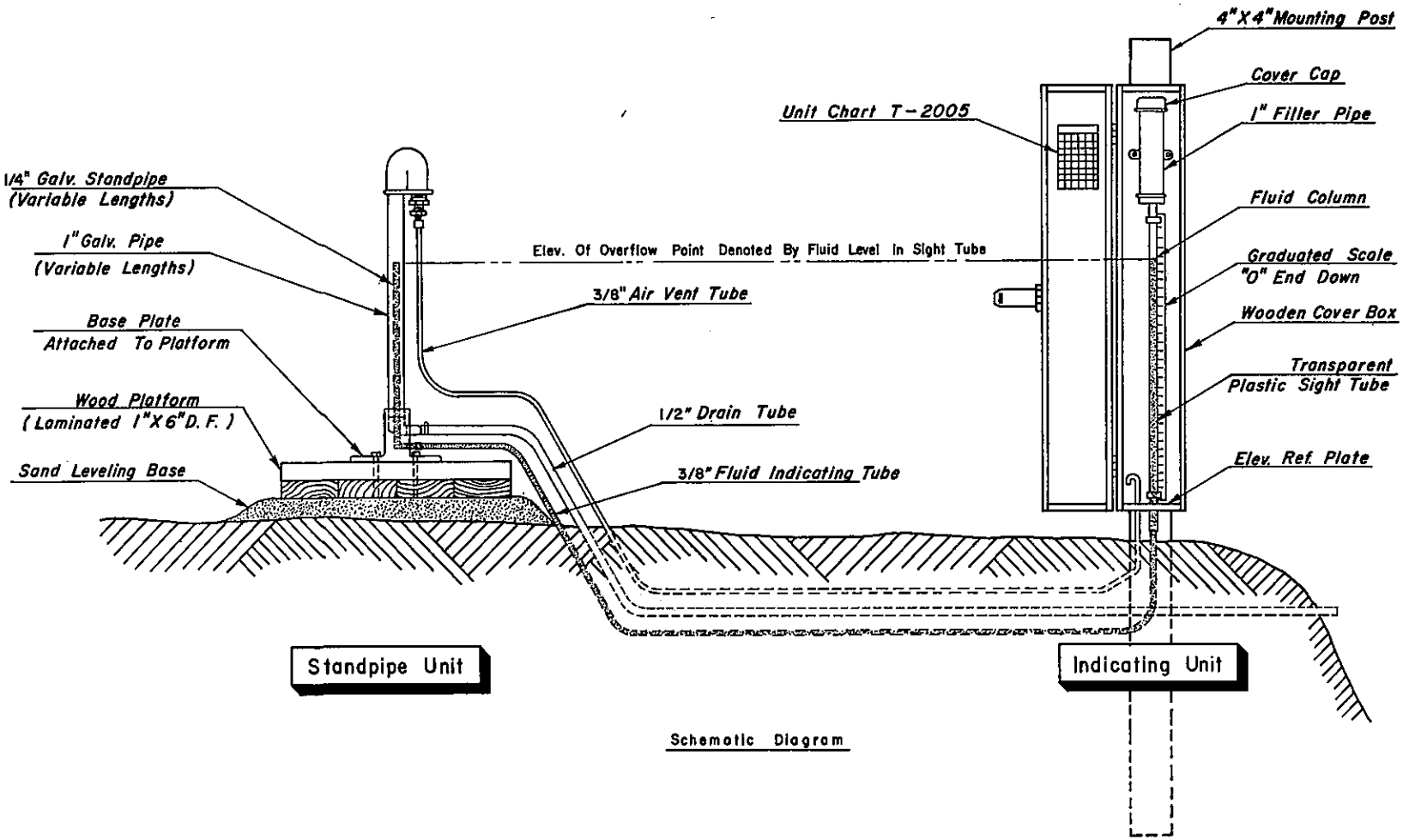

Schematic Diagram

Fig. VI-1 — Settlement indicating device — sealed fluid level type.

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TO HEADQUARTERS LABORATORY

STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
Materials & Research Department

SETTLEMENT DATA

FORM T-2006 (REV. 3-57)

Water Level Type
Settlement Platforms

JOB NO.	DATE	PAGE
	1-24-64	2

DIST.	CO.	RTE.	SEC.
IV	Mon	56	J

CONTRACT

LOCATION: Watsonville Bypass

NUMBER OR STATION	WATER TUBE READING	REFERENCE NAIL ELEVATION	RISER PIPE: ELEVATION TOP OF PIPE	RISER PIPE: LENGTH OF PIPE	SETTLEMENT PLATFORM: ORIGINAL ELEVATION	SETTLEMENT PLATFORM: PRESENT ELEVATION	SETTLEMENT IN FEET	FILL HEIGHT: SURFACE ELEVATION	FILL HEIGHT: FILL ABOVE ORIGINAL GROUND	ELAPSED TIME DAYS	UNIT WEIGHT OF FILL LB/CU. FT. / REMARKS
SP 7	1.77	20.29	22.06	5.53	16.89	16.53	.36		5	9	
SP 8	1.53	17.61	19.14	5.55	13.89	13.59	.30		5		
SP 9	1.63	22.48	24.11	5.65	18.96	18.46	.50		6		
SP 11	1.80	22.73	24.53	5.25	19.55	19.28	.27		6		
SP 13	1.35	19.90	21.25	5.35	16.09	15.90	.19		2		
(Col. 1)	(Col. 2)	(Col. 3)	(Col. 4)	(Col. 5)	(Col. 6)	(Col. 7)	(Col. 8)	(Col. 9)	(Col. 10)	(Col. 11)	

Plotted 1-28-64 BTS

Fig. VI-2 — Settlement data sheet for water level type settlement platform.

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c. Operation of the system is based upon the principle that a liquid seeks its own level. When the ground upon which the platform is resting, settles, the standpipe overflows and the level in the sight tube lowers correspondingly. With the graduated scale attached to the sight tube, a measurement of the amount of settlement of the platform relative to the indicating unit can be made.

d. If the indicating unit remains at constant elevation then the measurement reflects the absolute settlement of the embankment. On the other hand, if the indicating unit also settles or, as sometimes happens, raises because of heave, then a correction must be applied to the fluid scale reading in order to determine absolute settlement of the embankment. The correction is made from elevation changes of the indicating unit as determined by performing differential levels at periodic intervals.

e. It is noted in Fig. VI-1 that the standpipe is enclosed in a 1-in. galvanized pipe equipped with an "air vent" at the top and an overflow "drain" connection at the bottom. Plastic tubing is run from these two connections to the indicating unit where the ends are sheltered to keep them open and free. This completes the sealing of the standpipe unit to prevent flooding in the event of a rising groundwater table and minimize air locking of the device which could prevent the standpipe from overflowing properly.

f. An example of fluid level type settlement platform data and calculations is illustrated on Form T-2006 in Fig. VI-2.

g. The fluid-level type of installation is very effective at locations where it is possible to establish the indicator boxes in readily accessible places at the toe of the embankment. However, in areas where the embankment is being built on land which is covered by water or the "mud displacement" type of construction is being used, then it is necessary to utilize the riser type of settlement platform.

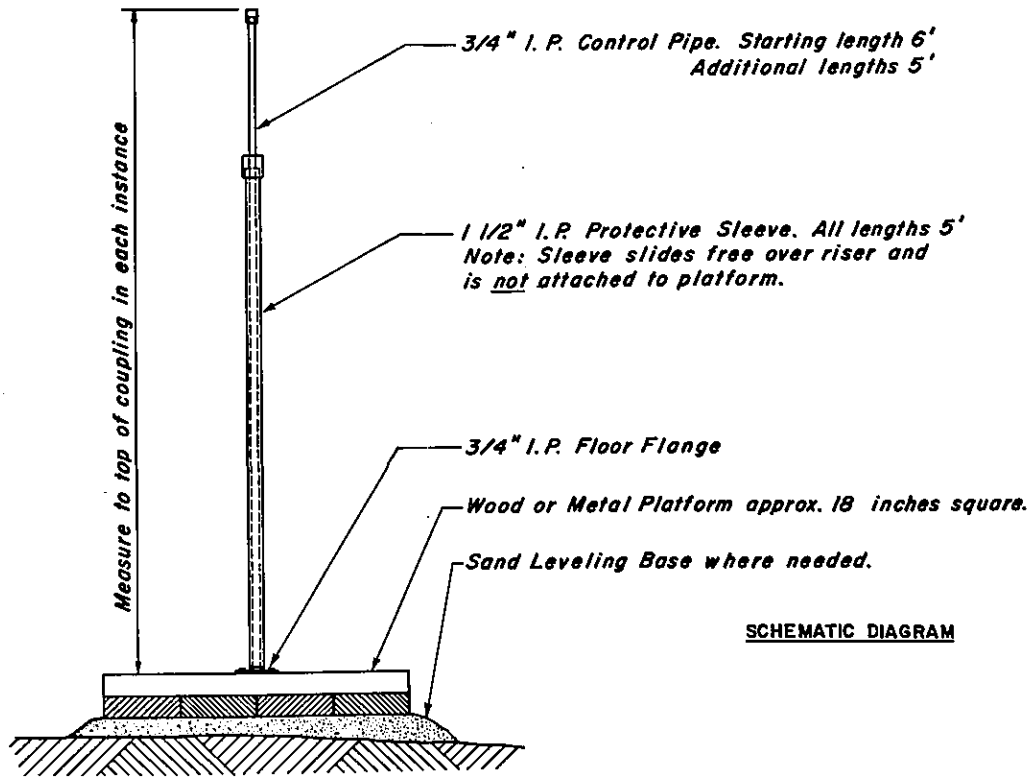

Fig. VI-3 — Settlement indicating device — riser pipe type.

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TO HEADQUARTERS LABORATORY

STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
Materials & Research Department
SETTLEMENT DATA
FORM T-2006 (REV. 3-57)

Riser Type
Settlement Platforms

JOB NO.	DATE	PAGE
29	12-18-64	18

DIST.	CO.	RTE.	SEC.
10	Sol	21	

CONTRACT: 65-10MAC649004
LOCATION: Benecia to Cordelia

NUMBER OR STATION	WATER TUBE READING	REFERENCE NAIL ELEVATION	RISER PIPE: ELEVATION TOP OF PIPE	RISER PIPE: LENGTH OF PIPE	SETTLEMENT PLATFORM: ORIGINAL ELEVATION	SETTLEMENT PLATFORM: PRESENT ELEVATION	SETTLEMENT IN FEET	FILL HEIGHT: SURFACE ELEVATION	FILL HEIGHT: FILL ABOVE ORIGINAL GROUND	ELAPSED TIME DAYS	UNIT WEIGHT OF FILL LB/CU. FT. / REMARKS
SP-17	(Riser Type S.P.)		30.29	21.20	9.75	9.09	0.66	22.3	20.5	~~+22~~	197~± Ult. &
-18	"	"	26.38	16.09	11.01	10.29	0.72	20.6	18.2	"	199~± " "
-19	"	"	26.27	15.94	10.50	10.33	0.17	18.0	13.5	"	201+50± " "
-20	"	"	11.40	5.37	6.15	6.03	0.12	6.4	7.1	"	201+50 Rt. Fr. Rd.
-21	"	"	11.63	5.38	6.63	6.25	0.38	6.6	4.7	"	199~ " " "
-22	"	"	11.80	5.37	6.61	6.43	0.18	7.2	5.2	"	197~ " " "
-23	"	"	13.99	10.60	3.98	3.39	0.59	11.6	8.3	~~+47~~	310~. Ult. &
-24	"	"	13.82	10.59	3.82	3.23	0.59	13.1	8.4	"	312~ " "
-25	"	"	14.99	10.64	4.54	4.35	0.19	13.7	8.5	"	314~ " "
-39	"	"	9.01	5.35	4.40	3.66	0.74	6.8	4.8	"	310+25 Rt. Fr. Rd.
-40	"	"	9.35	5.37	4.50	3.98	0.52	6.3	4.4	"	312~ " " "
-41	"	"	9.67	5.36	4.52	4.31	0.21	6.1	3.9	"	314~ " " "
-42B	"	"	20.15	15.86	4.44	4.29	0.15	15.9	14.0	~~+57~~	321~ Ult. &
-44	"	"	18.97	10.67	8.51	8.30	0.21	15.0	13.4	~~+65~~	323+50 " "
-45B	"	"	19.60	15.89	3.78	3.71	0.07	14.8	13.0	~~+57~~	326~ " "
-72	"	"	13.79	10.66	3.20	3.13	0.07	10.7	6.9	~~+65~~	328~ " "
-65	"	"	11.25	5.35	6.02	5.90	0.12	8.5	1.0	~~+77~~	372+90± Rt. Fr. Rd.
-62	"	"	11.74	5.35	6.42	6.39	0.03	7.8	5.0	~~+77~~	374+75± " " "
-46	"	"	18.02	16.00	2.86	2.02	0.84	11.4	7.4	~~+97~~	394+95 Ult. &
-47A	"	"	20.04	15.88	-0.51			17.5	7.4	~~+7~~ 189	400+40± " "
											Plotted 12-29-64

Fig. VI-4 — Settlement data sheet for riser type settlement platform.

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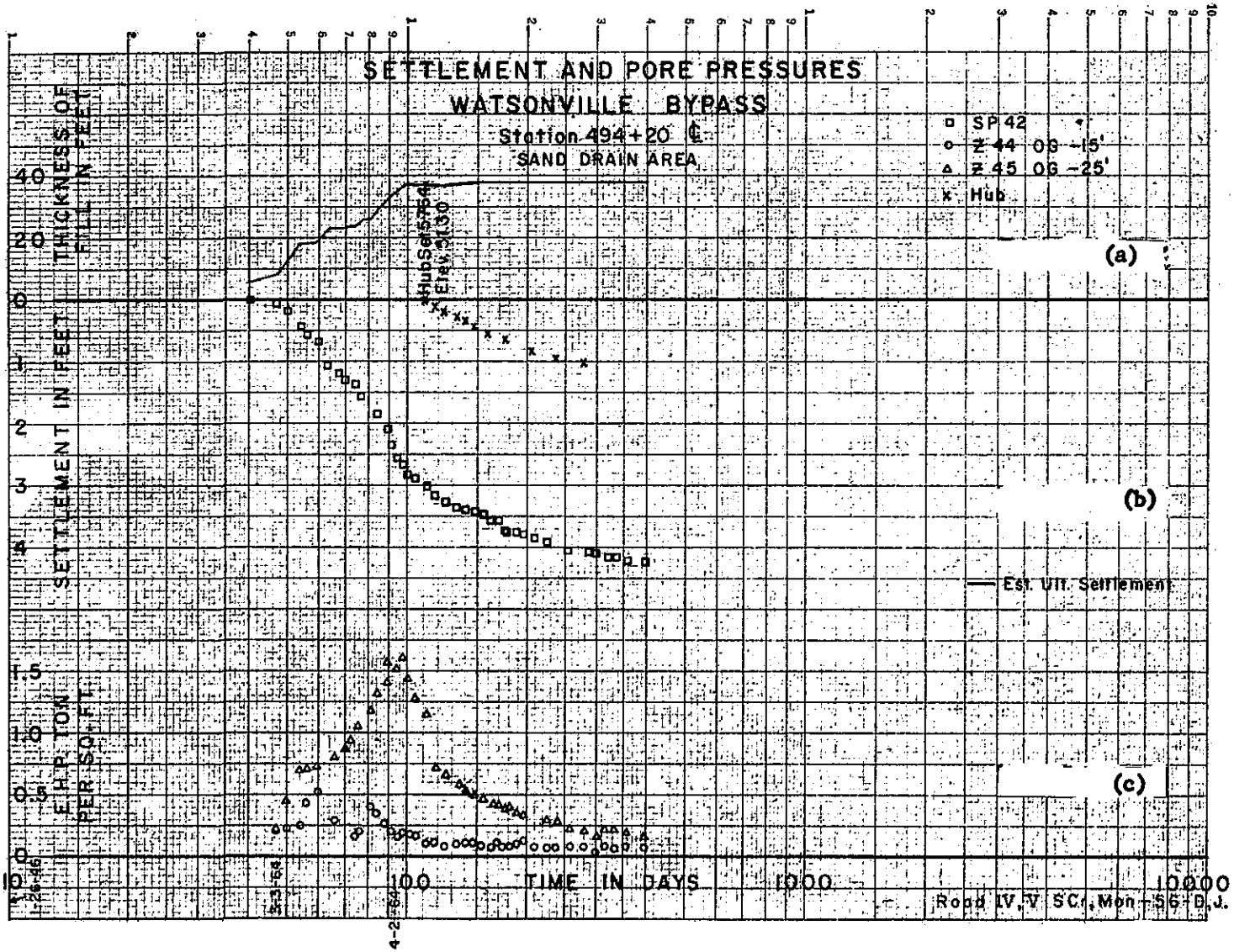

Fig. VI-5 — Settlement and pore pressures.

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3. Riser Settlement Platform

a. The riser type of settlement platform consists simply of an 18 x 18-in. wooden or metal platform which has a 3/4-in. galvanized pipe attached to the center by a pipe flange as shown in Fig. VI-3. When embankment heights in excess of 10 ft are involved, a 1-1/2-in. pipe sleeve is placed around the 3/4-in. riser to prevent pull-down by the embankment on the riser. The platform is set on either the natural ground or a working table. As the embankment is constructed over the platform, the 3/4-in. riser pipe (and 1-1/2-in. sleeve) is extended and maintained above the surface of the embankment. On divided roads the settlement platform is normally placed near the centerline so that the riser extends through the median. In the case of two-lane roads the platform is arranged so that the riser coincides with the shoulder.

b. Measurement of settlement with the riser type device is accomplished by normal survey leveling methods. Care must be exercised that suitable bench marks are accurately established and adequately referenced for this purpose. Fig. VI-4 illustrates the use of Form T-2006 for recording and calculating data obtained from this type device.

c. Data is usually plotted on 3 or 4 cycle x 20 to the inch semilogarithmic paper. Figs. VI-5(a) and (b) show embankment heights and corresponding settlements measured on the indicated number of days after the unit is installed. The plot of pore pressure, Fig. VI-5(c) is discussed under the section on piezometers.

4. Piezometers

a. The piezometer is a device for measuring the water pressure within a soil mass at a given point. It consists of a hollow cylindrical porous stone, normally 1 ft long and 1-1/2-in. O. D. by 1 in. I. D., which is connected to a plastic tube. The stone is embedded in the soft mud layer, where pressure determinations are desired, and the plastic tube extends aboveground.

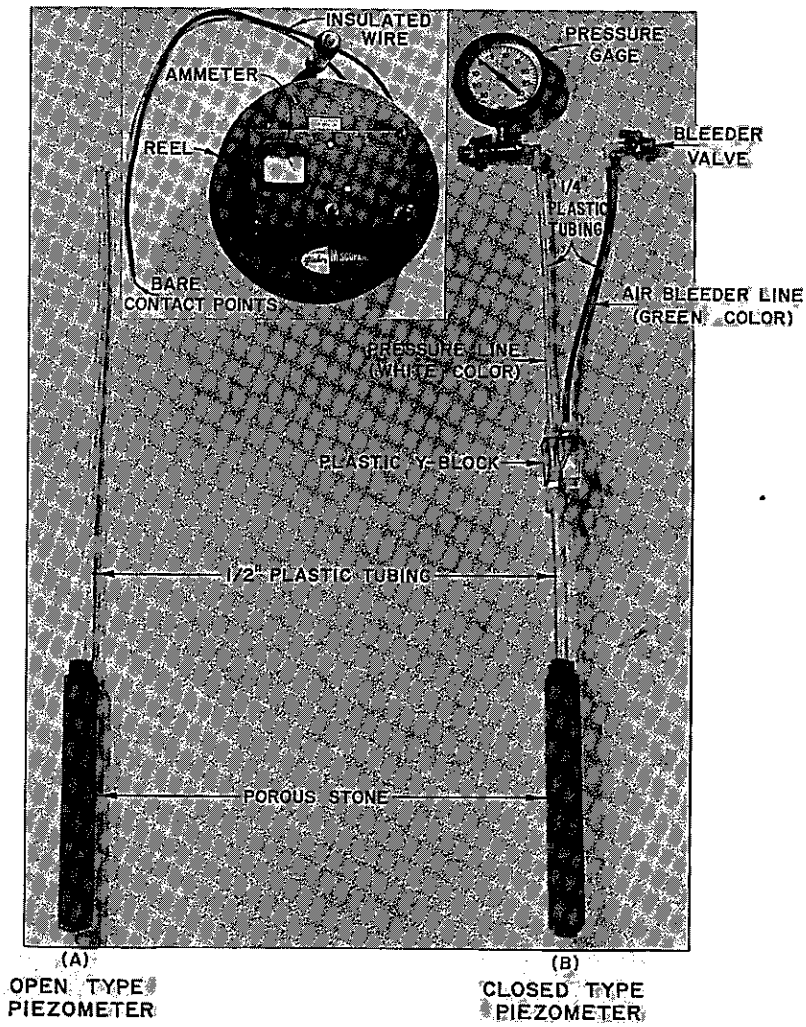

Fig. VI-6 — Piezometer systems.

b. There are two types of systems used in measuring hydrostatic pressure. They are the open and closed systems. In the open system the plastic tubing is brought straight up directly above the porous stone to the ground surface and the elevation of the water level inside the tubing is determined. In the closed system a pressure gage (capable of reading both positive pressure and vacuum) is connected to the plastic tubing and the entire system filled with water. Fig. VI-6(a) and (b) illustrate the basic details of both systems.

c. In order to amplify the meaning of pore pressure and its relation to fill construction, consider a typical soft, marshy soil in its natural condition. The soil is both saturated and highly impermeable. In this state the soil particles are rather loosely contacting each other. When an external load is applied, such as in the construction of an embankment, part of the stress is transmitted through the water and registers as pore pressure. Since water has no shear strength (for all practical purposes), the soil is initially relatively weak. An increase in strength is attainable through the process of consolidation which brings

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TO HEADQUARTERS LABORATORY

STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
Materials & Research Department

EXCESS HYDROSTATIC PRESSURE DATA

FORM T-2004 (REV. 3-57)

Closed Type System

Job No.	Date	Page
2 9	12-11-64	

Dist.	Co.	Rte.	Sec.
04	Mon-SC	56-67	J-D-Wats-A

Contract	Location
64-471 3C32F	Watsonville Bypass

Piezometer Number	1 Water Table Elevation	2 Elevation Piezometric Level	3 Distance From Water Table-Feet	4 Pressure of Water Col. (3 X .021)	5 Gauge Reading	6 Reading T/S.F. (5 X .072)	7 Pore Press. T/S.F. (4 + 6)	8 Height of Fill Placed Feet	9 Loading Due to Fill T/S.F.	Elapsed Time Days	Unit Weight of Fill Lb/Cu. Ft. / Remarks
Z-46	10.1'	17.3	7.2	0.22	-3.0	-0.11	0.11	38 ft	✓	331	
Z-38		16.8	6.7	0.21	-2.0	-0.07	0.14	18 ft	✓		
Z-44		17.4	7.3	0.23	-4.5	-0.16	0.07	✓ ✓	✓		
Z-45		17.8	7.7	0.24	0.0	0	0.24	✓ ✓	✓		
Z-39		16.8	6.7	0.21	-2.2	-0.08	0.13	✓ ✓	✓		
Z-29		17.4	7.3	0.23	-2.1	-0.08	0.16	47 ft	✓		
Z-40		16.6	6.5	0.20	5.0	+0.36	0.56	18 ft	✓		
Z-12		19.9	9.8	0.30	-2.2	-0.08	0.22	✓ ✓	✓		
Z-14		18.4	8.3	0.26	-4.8	-0.17	0.09	26 ft	✓		
Z-10		19.5	9.4	0.29	-1.0	-0.04	0.25	16 ft	✓		
Z-53	12.2	18.2	8.0	0.19	-1.5	-0.05	0.14	3.2 ft	✓		
Z-54		19.0	8.8	0.21	-1.1	-0.04	0.17	✓ ✓	✓		
Z-55		19.0	8.8	0.21	-2.8	-0.10	0.11	✓ ✓	✓		
Z-56		18.4	8.2	0.19	-2.9	-0.10	0.09	✓ ✓	✓		
Z-58	11.2	20.3	9.1	0.28	-0.5	-0.02	0.26	✓ ✓	✓		
Z-59		19.5	8.3	0.26	0.5	-0.02	0.24	✓ ✓	✓		
Z-60		21.3	10.1	0.30	-4.1	-0.15	0.15	34 ft			
Z-61		19.5	8.3	0.26	-4.0	-0.14	0.12	✓ ✓			Plotted 12-18-64 JR

Fig. VI-7 — Excess hydrostatic pressure data sheet — closed type system.

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STATE OF CALIFORNIA - DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
Materials & Research Department

EXCESS HYDROSTATIC PRESSURE DATA

FORM T-2004 (REV. 3-57)

Open Type System

Job No.	Date	Page
2464-21-5	8-27-62	26

Dist.	Co.	Rte.	Sec.
XI	SD	2	Nat. C

Contract	Location
62-1V13C17-I	National City

Piezometer Number	1 Water Table Elevation	2 Elevation Piezometric Level	3 Distance From Water Table-Feet	4 Pressure of Water Col. (3 x .031)	5 Gauge Reading	6 Reading T/s.f. (5 x .072)	7 Pore Press. T/s.f. (4 + 6)	8 Height of Fill Placed Feet	9 Loading Due to Fill T/s.f.	Elapsed Time Days	Unit Weight of Fill 120 Lb/Cu. Ft. Remarks
8	0.5	6.13	5.6	.174	—	—	.17	6.8	.41	68	
9	0.5	8.06	7.6	.236	—	—	.24	7.2	.43		
13	0.5	3.66	3.2	.099	—	—	.10	0	0		
14	0.5	1.98	1.5	.047	—	—	.05	0	0		
#38	W.T	0.51	—	—	—	—	—	—	—	68	

Fig. VI-8 — Excess hydrostatic pressure data sheet — open type system.

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the soil particles into more intimate contact and consequently transfers more of the load from the water to the soil particles. This in turn reduces the pore pressure. However, for consolidation to take place, water must be squeezed from the pores of the soil. Since most marshy soils are highly resistant to the movement of water, the transfer of pressure from water to soil does not take place rapidly. Pore pressures remain relatively high until water passes from the soil over a period of time. Consequently, the factors of strength gain and settlement are time dependent functions of pore pressure.

d. Piezometer measurements of pore pressure may be used for monitoring both settlement and stability. For settlement studies, the piezometers are normally placed near centerline and near the settlement platforms. These installations are especially useful for determining when primary settlement is nearly complete so that paving may be undertaken. It is possible to do some stability analysis from centerline piezometers. However, this is highly theoretical and since many asumptions must be made, the accuracy of this method is sometimes debatable.

e. A more direct method for monitoring stability is to place several piezometers at the toe of the embankment. Under normal circumstances a small amount of pressure will be registered on these devices during construction. Any sharp and sudden change in pressure, either in the positive or negative direction, foretells of a shear failure occurring in the foundation soil.

f. An example of piezometer data and calculations on Form T-2004 is illustrated in Fig. VI-7, for a "closed" system.

An example of piezometer data and calculations on Form T-2004 is illustrated in Fig. VI-8 for an "open system" installation. A typical example demonstrating the plotting of piezometer data is shown in Fig. VI-5(c).

5. Heave Stakes

a. Heave stakes are surface movement indicating devices which are used when the stability of the fill foundation is considered to be a problem. A row of stakes, usually consisting of 2 x 2-in. x 18-in. hubs (or larger) are placed from 10 to 25 ft outside of and generally parallel to the toe of the embankment. The hubs may be spaced on centers ranging from 10 to 100 ft, depending on circumstances; the most common spacing is 50 ft. It is not advisable to extend heave line points much beyond a distance of about 1000 ft as sight distance begins to affect the accuracy of measurements.

b. Survey tacks are set on each hub in line using a transit. Sight targets for line are placed on some solid support (e.g., tree, power pole, building, etc.) at each end of the line. It is advisable, wherever possible to double target each end of the line on independent supports for reference.

c. Differential levels are run on the hubs for elevations. It is desirable to have a bench mark (B. M.) located on solid ground near each end of the line so that levels can start at one B. M. and close to the other. However, for lines under 500 ft long, one good bench mark may be used for both starting and closing. Error of closure should never exceed 0.10 ft. Errors between 0.05 and 0.10 ft should be redistributed among heave points.

d. Fig. VI-9 shows an example of heave stake data recorded on Form T-2081. Fig. VI-10 illustrates the manner in which both vertical and horizontal movement are plotted for each individual heave stake.

e. In analyzing the heave stake data, it must be borne in mind that, it is not the total magnitude of movement but rather the rate of change which is important. The faster the horizontal and vertical displacement, the more critical the stability situation becomes.

6. Inclinometers

a. The inclinometer is a device for sensing subsurface movement of soil masses. It consists of 10-ft lengths of 3/4-in. I. D. plastic tubing joined together to form a suitable length. The tubing is installed vertically through the soft mud layer to firmer material at a distance of from 5 to 15 ft outside the toe of the embankment. Extreme care is exercised to insure that the tubing is initially installed straight.

b. When an unstable condition begins to develop and the mud flows outwardly away from the embankment, the inclinometer tube will be bent or deflected in a manner similar to that

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REPORT OF HEAVE STAKES

Lab. Auth. _____ Dist. 05 Co. MON Rt. 0056 Sec. J

Location Watsonville By-pass Contract 64-04T 12C 32 F

Date 4/17/64 Recorder Pugnier party Fill Height on ℄ _____ ft.

Line No. Rt. of "J" 490 to 497+50

	Station	Vertical Movement				Horizontal Movement		Remarks
		H.I.	Rod Reading	Present Elevation	Original Elevation	Left	Right	
	BM#9 497+50		-8.92 -9.20	14.18 13.91	14.18 * 13.91	✓	0.00	
	497~		-8.84	14.26	14.30	✓	0.05	
	496+50		-9.41	13.69	13.79	✓	0.06	
	496~		-9.99	13.11	13.14	✓	0.10	
	495+50		-9.75	13.35	13.38	✓	0.15	
	495~		-10.34	12.76	12.77		0.14	
	+60		-10.50	12.60	12.61	✓	0.08	
	494+50		-9.58	13.52	13.53	✓	0.07	
	494+05		-9.08	14.02	14.02	✓	0.03	Mark on H.W.
+5.80	T.P.	23.10	-12.47	17.30				
+11.44	B.M.	29.77	-11.44	18.32	18.33			
	493+40		-9.66	20.10	20.11	✓	0.01	
	493~		-10.57	19.19	19.19	✓	0.01	
	492+50		-9.55	20.21	20.21	✓	0.02	7/10/08
	492~		-5.24	24.52	24.50	✓	0.04	4-21-69
	+80		-3.68	26.08	26.05	✓	0.03	BTS
+0.29	+50	29.76	-13.25	29.47	29.46	✓	0.04	
	491+25		-11.24	31.48	31.47	✓	0.03	
	490+00		-2.00	41.72	40.71	✓	0.01	
+0.69	B.M.#8	42.72		42.03				

Form T-2081 (Orig. 10-61)

Fig. VI-9 — Heave stake data sheet.

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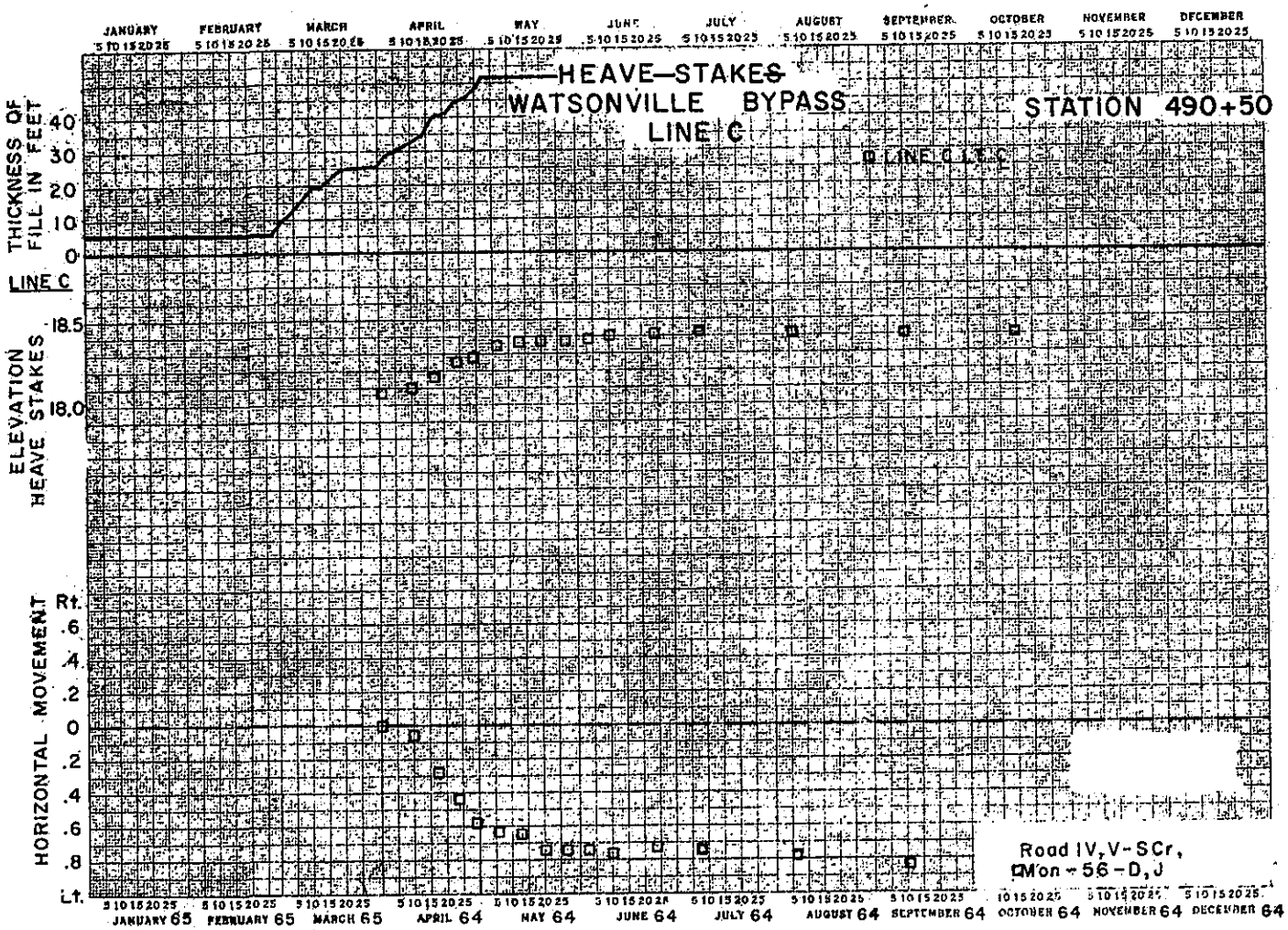

Fig. VI-10 — Plot of embankment thickness versus vertical and horizontal heave.

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Not to Scale

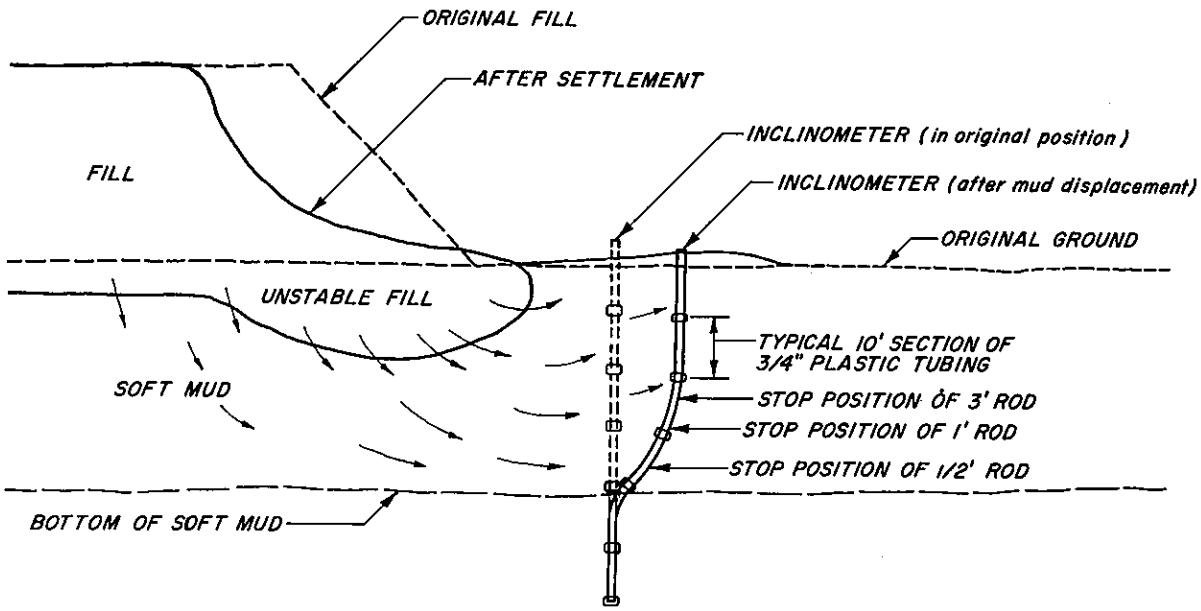

Fig. VI-11 — Typical installation showing inclinometer movement during displacement.

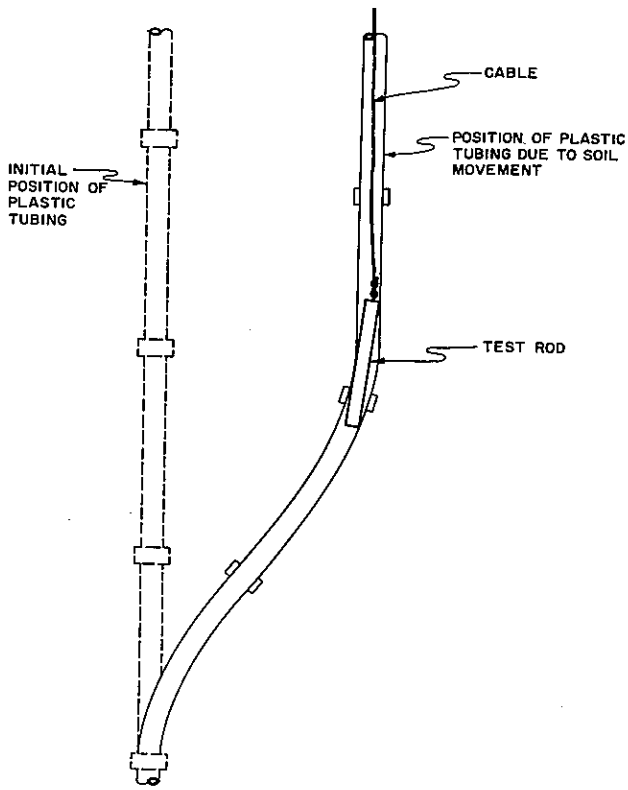

Fig. VI-12 — Inclinometer device.

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ROOT INCLINOMETER READINGS

LAB. AUTH. 2Q DIST. 10 CO. Sol RT. 21 SEC.
LOCATION Benicia to Cordelia
DATE 1-12-65 OPERATOR Leavitt

NUMBER	DEPTH IN FEET	DEPTH PROBE DROPS TO 3 FOOT	1 FOOT	½ FOOT	REMARKS
I - 37	30.10	6.0	20.0	22.5	Mud Heave
34	30.00	11.2	16.7	16.8	" "
33	26.63	4.5	21.4	22.8	" "
32	21.35	14.2	14.4	19.8	
31	22.10	5.0	19.0	19.0	
28	21.95	6.1	14.1	18.5	
48	17.1	4.6	15.2	17.0	* See below
49	20.0	8.0	19.9	19.9	* " "
50	15.50	4.0	15.5	15.5	* " "
55	16.10	15.9	15.9	15.9	
57	20.10	10.6	19.9	19.9	* " "
74	14.70	14.7	14.7	14.7	
73	6.10	5.9	5.9	5.9	
54	9.95	4.4	9.8	9.8	
53	12.45	4.4	12.4	12.4	
* Delta caused by erosion from last storm in vicinity of					
Settlement Platforms has apparently deflected Inclinometer					
near Surface.					
					Plotted 2-9-65
					R

Form T-2078 (ORIG. 9-61)

Fig. VI-13 — Root inclinometer data sheet.

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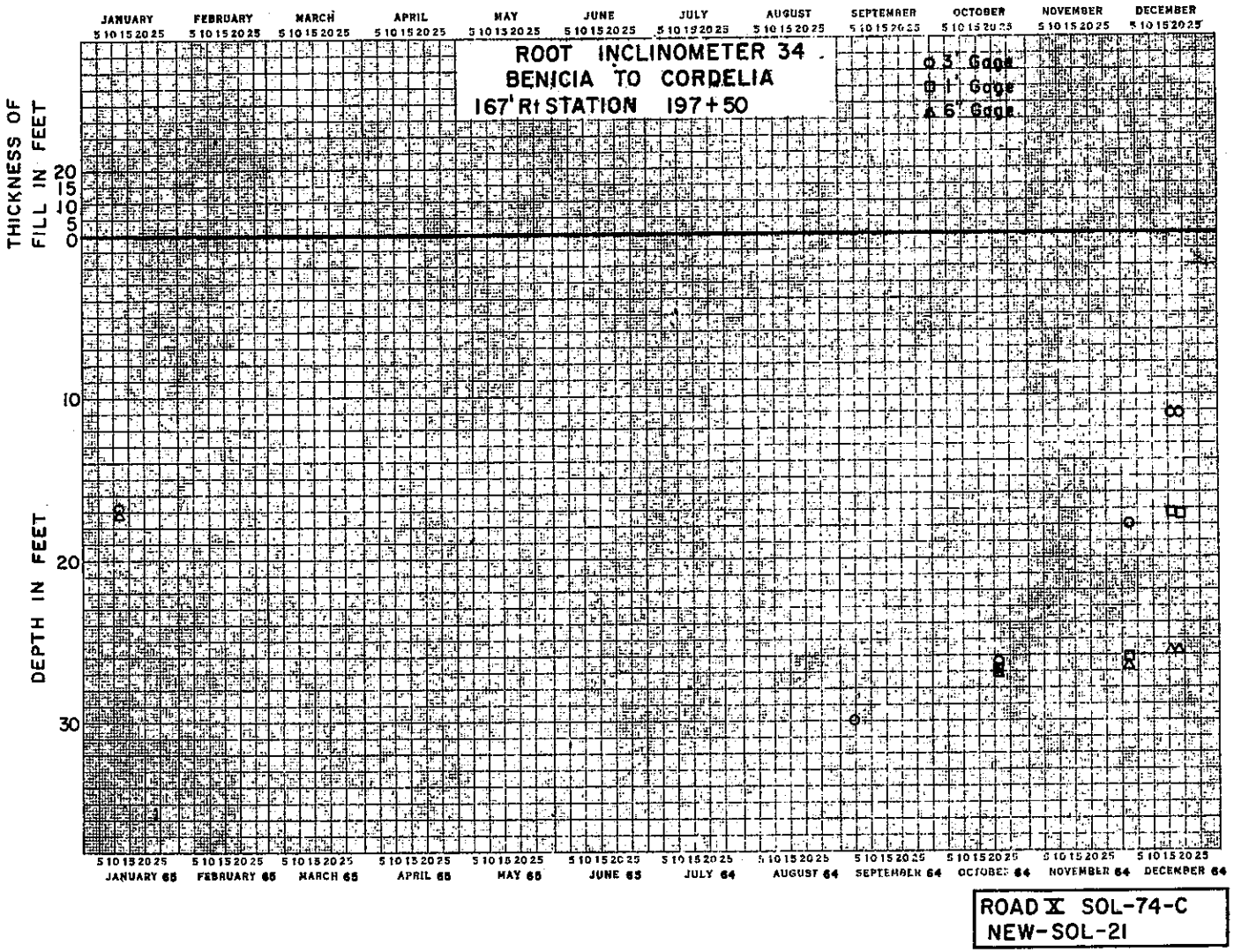

Fig. VI-14 — Plot of inclinometer data versus time and embankment thickness.

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demonstrated in Fig. VI-11. Detection of this bending is accomplished by lowering a 0.675-in.-diameter test rod into the 3/4-in. tubing, on the end of a line, until the rod binds in the bend as shown in Fig. VI-12. This procedure is performed with three rods having lengths of 3 ft, 1 ft, and 1/2 ft, and depths to refusal are determined in each case. If on the first trial, the 3-ft rod goes to the bottom of the tube then it is not necessary to try the 1-ft and 1/2-ft rods. Inclinometer data is recorded on Form T-2078 as shown in Fig. VI-13 and plotted in the manner illustrated in Fig. VI-14.

c. The inclinometer will provide only a very general concept of the measurement of the magnitude of underground movement, but it can give a fair indication of the depth at which displacement is occurring. The significance of this field test, with regard to stability, is not only how much the refusal points change, for the various rods, but also how rapidly they change. The faster the positions change the more critical stability becomes.

7. Summary of Use of Monitoring Instruments

a. The two principal items of concern, when building embankments over soft foundation soils are (1) the stability of the embankment foundation and (2) the magnitude and rate of settlement. The location of the control devices in or on the embankment foundation is dependent upon the type of problem involved. When only settlement is of concern, a settlement platform and several piezometers, at various depths, are installed on or near centerline. On the other hand, where stability is a problem, piezometers are installed at the toe of the embankment along with an inclinometer (5 to 15 ft out from toe) and heave lines (10 to 25 ft out from toe). Fig. VI-15 illustrates the instrument positioning for both stability and settlement.

b. The frequency of reading the devices will vary according to circumstances involved in individual projects. In general, readings will range from one to three times a week during construction of the embankment. If stability is critical, readings may be taken two or three times a week. On the other hand, if the problem only involves settlement then once a week will usually suffice. After the embankment is completed, and during a waiting period before paving, the frequency of readings may be reduced to once or twice a month. The number of readings will not only depend upon the criticalness of stability, but also the rate of settlement. If the rate of settlement is relatively rapid, then more frequent readings are in order. This will aid in making predictions as to when settlement will be nearly complete so that surcharges may be removed and/or the construction of the structural section may be satisfactorily undertaken.

c. In summarizing field control operations, it may be said that the effectiveness of the instrumentation is no better than the care and conscientiousness with which the measurements are made. Effective use of construction control methods, when dealing with difficult marshlands, will lead to less expensive and better riding highways for tomorrow's traffic.

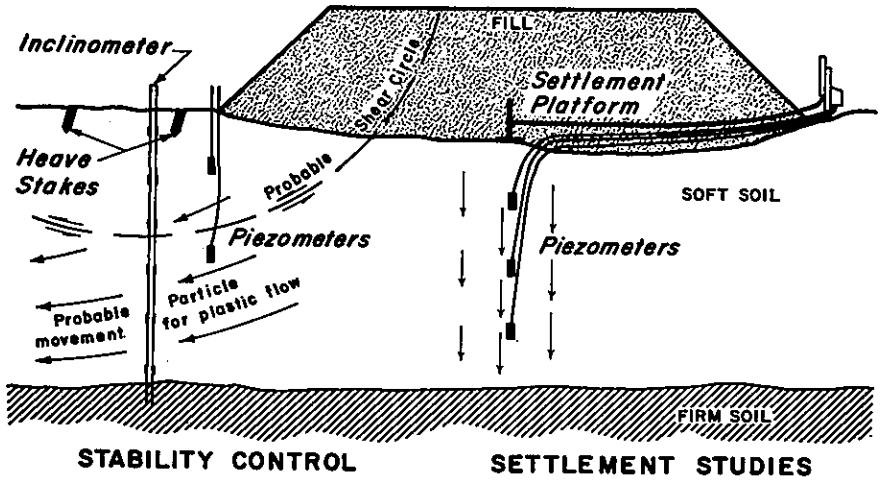

Fig. VI-15 — Placement of construction control devices.

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VII. Relative Compaction by the Area Concept Utilizing Nuclear Gages

A. Introduction

During the past 10 years the California Division of Highways has conducted several comprehensive research studies, both in the laboratory and in the field, in an attempt to substitute nuclear compaction control of moisture and density for the California standard sand volume test.

The earlier studies, 1954 through 1962, revealed many inconsistencies in the use of the gages. Use, however, of the knowledge gained during these earlier studies coupled with technical improvement by the manufacturers of the gages made the use of nuclear methods for moisture and density determinations more encouraging.

In 1965 an extensive research program was developed for field testing of the nuclear gages and the area concept on eleven construction projects in ten districts. Test Method No. Calif. 231 was specified in the special provisions as the method to be used in compaction control of earthwork. The projects selected for this study represented a broad range of various soil types, terrain, climatic conditions and construction operations.

In general, the results of this statewide study have been very encouraging. A discussion of the advantages and disadvantages in applying the nuclear method in this study are detailed in an interim report by W. G. Weber titled Nuclear Testing for Control of Earthwork Moisture and Density, dated March 9, 1966.

As a consequence of the favorable outcome of this study, nuclear compaction control is being specified for use on about 45 additional construction projects covering the 11 highway districts. Several of these projects are now underway and the balance are slated for contracts during 1967.

B. Density by Transmission-Type Gage

1. Apparatus

Density is determined when using the transmission technique by directly measuring the gamma rays which pass through a soil. The transmission gage (Fig. VII-1(a)), is constructed with a probe which may be extended into a hole formed in the soil. The extended probe will contain the detector tube. For any given extension of the probe there is a fixed distance between the source and the detector tube.

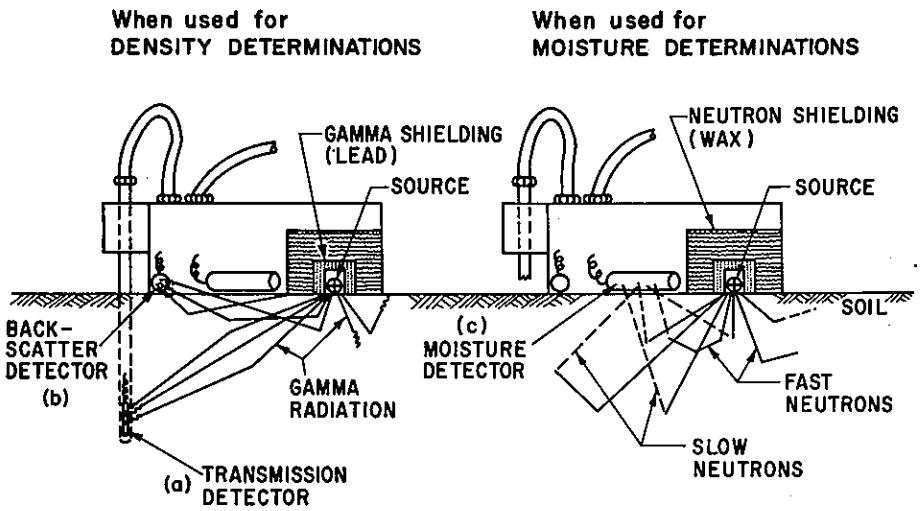

Fig. VII-1 — Transmission type nuclear density-moisture probe.

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2. Gage Operation

The relative amount of gamma radiation received by the detector tube depends on the density of the soil between the detector and the source. A soil of low density will absorb less of the radiation than will a soil of high density. Consequently the detector will reflect a high count for the low density soil, and a low count for the high density soil.

3. Calibration Curves

Calibration curves are established for the count in a fixed period of time versus known soil densities. Separate calibration curves are determined for the separate probe positions due to the difference in distances between probe and detector tube. These calibration curves are then used in the field to determine the wet density of the soil.

C. Density by Backscatter or Compton-Type Gage

1. Apparatus

When using the backscatter method, soil density is determined by measuring the gamma rays which are returned at lower energy from a soil after irradiation. The backscatter gage is constructed with both the source and detector tube housed in the same unit (Fig. VII-1(b)). Suitable shielding is placed between the two so that radiation cannot be directly transmitted from the source to the detector tube.

2. Gage Operation

The gage is placed on a soil and the gamma rays from the source enter the soil. Some of these rays are deflected back to the detector tube after a loss of energy. Much of the radiation energy is entirely absorbed by the soil materials. The soils of high density will absorb more of the radiation than soils of low density. Thus, the relative amount of radiation returned from the soil may be used as an index to the soil density.

3. Calibration Curves

A calibration curve of relative count for a fixed period of time versus soils of known density is established. This calibration curve is then used to determine the wet density of the soil in the field. Some soils require different calibration curves from others, and it is necessary to check the calibration when using the gage on several soils to be sure that the operator is using an appropriate curve.

D. Soil Moisture by Neutron Moderation

1. Definition

Soil moisture is determined by measuring the relative amount of slow neutrons produced by the water in a soil after irradiation with fast neutrons.

2. Fast, Slow, and Thermal Neutrons

A neutron is a heavy nuclear particle with no electrical charge. Depending on their speed or energy they are classed as fast, slow, or thermal neutrons. Thermal neutrons are free neutrons with only the energy of the ambient temperature. A slow neutron is one which can be absorbed by the nucleus of certain atoms, producing an isotope of that atom and releasing radiation. A fast neutron is one at high speed with an energy so great that it has a low probability of absorption. It usually loses its energy by elastic collisions with the nucleus of atoms until it becomes a slow neutron, at which time it can be absorbed by certain atoms. If the fast neutron collides with a nucleus of large size it merely changes direction with little loss of energy. The case is similar to that of a marble hitting a billiard ball. However, if the fast neutron collides with a nucleus of similar size such as that of the hydrogen atom, it will lose half of its energy in the single collision. It is seen that water, a large portion of its matter being hydrogen, would produce a large number of these slow neutrons after exposure to radiation by fast neutrons.

3. Nuclear Moisture Gage

A source of fast neutrons is used in the nuclear moisture gage with a detector tube that is senstive to slow neutrons, but does not respond to the fast neutrons (Fig. VII-1(c)). When the moisture gage is placed on a soil, the higher the moisture content the higher will be the

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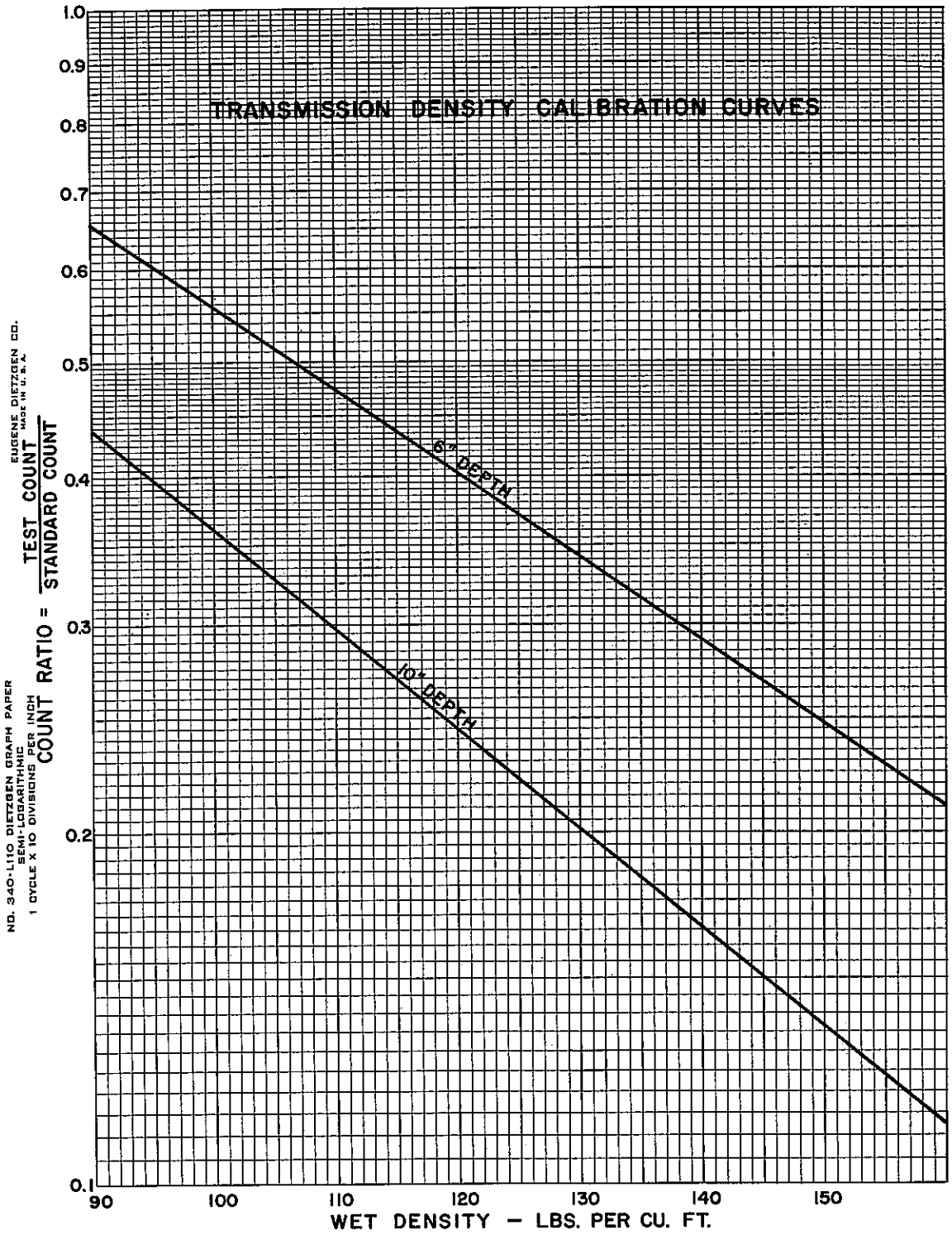

Fig. VII-2 — Transmission density calibration curves.

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count recorded by the gage. This is due to the transformation of the fast neutrons to slow neutrons by the hydrogen atoms in the soil water.

4. Calibration Curve

A calibration curve of relative count versus water content in pounds per cubic foot for a fixed period of time is established for the particular gage. The calibration curve may then be used to determine the moisture content of soils in the field.

E. Reading Count Ratio and Calibration Curves

1. Nuclear Readings at Test Site

The procedure to prepare the surface of the test site is given in Sections D-1 and D-2 of Test Method No. Calif. 231. The objective is to minimize the effect of voids and surface roughness beneath the gage. These irregularities can seriously affect the the test count. Nuclear readings are taken as directed in Sections D-3 and D-4 of Calif. 231. The procedure outlined for taking the count will provide a satisfactory reading with a minimum effort. Many new operators tend to take an unnecessary number of readings which are time consuming and do not significantly increase the accuracy of the test.

2. Count Ratio

a. All of the gages include a self standard as part of the supplied equipment. This provides a method of periodically taking a check count to test the operation of the gage. Any unusual variation in this check count requires immediate investigation as the gage may not be used until it is corrected. The reading on the self standard is called the standard count.

b. Unless otherwise specified, the accepted average count for the site moisture or density is converted to an index called the count ratio. The test count is divided by the standard count to yield the count ratio. This practice helps to compensate for a certain amount of drift with time which is normal for electronic equipment such as the nuclear soil gages.

3. Calibration Curves

The calibration curves are usually in terms of count ratio versus density or moisture, but may occasionally be in count only. The moistures and densities indicated by the calibration curves supplied with the gage are to be compared with field data obtained by the conventional sand volume and oven drying. The calibration curves may be corrected if necessary as directed in Section C of Calif. 231. The field moisture or density is then determined from the calibration curve as illustrated in Fig. VII-2.

F. Field Density Control: Example

1. In the example in Fig. VII-3, the field inspector has determined to his satisfaction that the 600 ft of roadway is of essentially the same soil type. (Section E-1 of Test Method No. Calif. 231). It is very important that the test area be determined with care for the reason given in the test method. The area has been divided into three approximately equal sub areas (see Fig. VII-3).

2. The results of the wet soil densities are presented in Fig. VII-3. The average of the site densities is 126 lb per cu ft. The maximum laboratory test density is performed on a composite sample made of equal representative portions of soil from each nuclear test site (Section E-4 of Calif. 231). An adjusted wet density of 139 lb per cu ft is obtained from the impact test curve. The required compaction is 90% or 125 lb per cu ft.

3. It is seen that the area average of 126 lb per cu ft would pass, and that not more than two or one-third of the field tests are below this required test value. However, the test average of Sub area 1 is below the required compaction. The Resident Engineer will accept the entire area.

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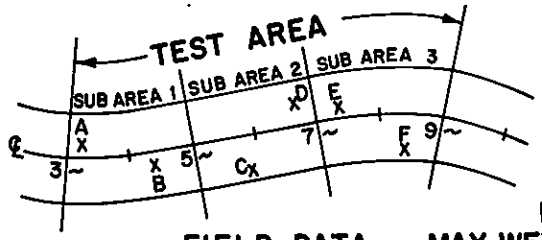

SPECS:
MIN R.C. 90%

FIELD DATA

LAB TEST:
MAX WET DEN. 139 pcf

SUB AREA	SITE	SITE WET DENSITY	RELATIVE COMPACTION
1	A B	119 pcf 128	86% 92
2	C D	131 125	94 90
3	E F	124 129	89 93
AVERAGES		126	91

ANALYSIS: SINCE SITE AVG. = 91 > 90 SPEC. AND NOT MORE THAN 1/3 OF THE SITES HAVE RC VALUES < 90 SPEC. THE AREA PASSES.

Fig. VII-3 — Example of field density control.

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Appendix

Field Classification of Soils*

Table A — Minimum Description for Classification of Soil In Place as Sampled 102

Additional Descriptions for In-Place Condition of Soil. 103

Boring Records*

Abbreviations for Use on Boring Records 103

Fig. A — Illustrative Boring Record 105

Instructions for Filling Out Boring Record. 107

*As prepared and used by the California Division of Highways.

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TABLE A — MINIMUM DESCRIPTION FOR CLASSIFICATION OF SOIL IN PLACE AS SAMPLED

Property	(Size)	Field Identification
1. MOISTURE		Dry-Moist-Wet-Very Wet (very wet means free water evident).
2. a. COMPACTNESS Coarse-grained soils		Loose-Dense-Very Dense.
b. CONSISTENCY Fine-grained soils		Very Soft — Exudes between fingers when squeezed. Soft — Easily molded in fingers. Firm — Can be molded by strong pressure in fingers. Stiff — Cannot be molded in fingers. Hard — Brittle or very tough.
3. COLOR		Indicate range of color: e. g., brown to reddish brown. Use qualifying adjective to indicate cast, as reddish.
4. SOIL TYPES		
a. COARSE-GRAINED, NONCOHESIVE:		
Broken rock	(+3")	Angular fragments of rock.
Stone		Crushed or naturally angular rock particles.
Coarse	(-3" + 1")	
Medium	(-1" + 3/8")	
Fine	(-3/8" + #10)	
Boulders	(+6")	Rounded fragments of rock.
Cobbles	(-6" + 3")	Rounded.
Gravel		Rounded rock particles.
Coarse	(-3" + 1")	
Medium	(-1" + 3/8")	
Fine	(-3/8" + #10)	
Sand		Particles visible to naked eye. No cohesion when dry.
Coarse	(-#10 + #40)	
Fine	(-#40 + #200)	
b. FINE-GRAINED, COHESIVE:		
Silts	(-#200 + 5 Micron)	Particles mostly -200; barely visible to naked eye. Some plasticity, and exhibits marked dilatancy (shaking test shows quick or "livery" condition). Dries moderately quickly and can be dusted off the fingers. Dry lumps possess cohesion, but easily powdered in fingers.
Loams		Contain sand, silt and clay sizes; possess moderate plasticity.
Sandy Clay		Moderate to high plasticity and dry strength. No dilatancy.
Silt-Clay		Fine grained (-200). No dilatancy. Appreciable plasticity.
Clay	(-5 Micron)	Smooth to touch and plastic, no dilatancy. Sticks to fingers and dries slowly. Shrinks appreciably on drying. Dry lumps can be broken but not powdered. They disintegrate under water. Lean clays show the above properties to low degree. Fat clays show the above properties to high degree.
c. ORGANIC		
Peats		Fibrous organic matter, usually brown or black in color: Firm — Fibers compressed together. Spongy — Very compressible and open structure.
5. PLASTICITY		Nonplastic, slightly plastic; moderately plastic; highly plastic. Inherent property of soil. Moisten as necessary to evaluate plasticity.

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Additional Descriptions for In-Place Condition of Soil

Origin

Alluvial	Deposited by water
Aeolian	Wind borne
Coluvial	Gravitational deposition (slides and talus).
Residual	Not transported

If known, record for each bed or deposit; not required for every sample in a boring.

Particle Shape

Rounded; subangular; angular

Particle Size Distribution

One size	Particles mostly one size.
Gap-grading	Intermediate sizes missing.
Graded	Appreciable amounts all particle sizes.

Cementation

Weakly cemented - firmly cemented-indurated.

Consistency

Tenacious	Refers to cohesion.
Sticky	Refers to adhesion.
Tough	Resistant to rupture.
Mellow	A consistency softer than friable; soil particles weakly adhered.
Friable	Soil structure readily ruptured and can be crushed by mod. force.
Cheesy	More or less elastic, deforming considerably without rupture, yet broken by little force.

Color

Mottled; marbled; spotted, streaked; speckled; variegated.

Structure

Homogeneous	Varved	Fissured
Stratified	Laminated	Hardpan
Lenticular	Vesicular	Claypan

Texture

Gritty	Silty	Dirty (applies to sand or gravel)
Sandy	Peaty	

Others

Micaceous	Organic	Calcareous	Alkaline

Abbreviations for Use on Boring Records

Alphabetical

adb	adobe	blk	black
alv	alluvium	btl	brittle
arg	argillaceous	chs	cheesy
arn	arenaceous	cht	chert
bas	basalt, basaltic	cl	clay, clayey
bdd	bedded	com	compact
bl	blue, bluish	cong	conglomerate
bld	boulder	crs	coarse

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dk	dark
dns	dense
frb	friable
fsl	fissile
gr	gray, grayish
grl	granular
grn	green, greenish
grv	gravel
hd	hard
hvy	heavy
ign	igneous
lam	laminated
lm	loam
lse	loose
lt	light
mel	mellow
mod	moderately
mrb	marbled
mtl	mottled
pbl	pebble, pebbly
qtz	quartz
rd	red, reddish
sch	schist, schistose
sd	sand, sandy
sed	sedimentary
sh	shale, shaly
si	silt, silty
sl	slight, slightly
srp	serpentine
ss	sandstone
st	stone, stony
sti	stiff
stk	sticky
str	streak, streaked
strf	stratified, stratification
ten	tenaceous
tgh	tough
thk	thick
tls	talus
tr	trace
vsc	vesicular
yl	yellow, yellowish

Alphabetical by Characteristic

GEOLOGICAL AND SOIL TYPE

adb	adobe
arg	argillaceous
arn	arenaceous
bas	basalt
bdd	bedded
bld	boulder
cl	clay, clayey
cong	conglomerate
lam	laminated
pbl	pebble, pebbly
qtz	quartz
sch	schist, schistose
sd	sand, sandy
sed	sedimentary
sh	shale, shaly
si	silt, silty
srp	serpentine
st	stone, stony
strf	stratified
tls	talus

DEGREE

mod	moderate
sl	slight
tr	trace
vy	very

COLOR

bl	blue, bluish
blk	black
br	brown, brownish
dk	dark
gr	gray

COLOR (contd)

grn	green, greenish
lt	light
mrb	marbled
mtl	mottled
rd	red, reddish
str	streaked

STRUCTURE AND TEXTURE

chs	cheesy
crs	coarse
dns	dense
fsl	fissile
grl	granular
lse	loose
vsc	vesicular

CONSISTENCY

btl	brittle
frb	friable
hd	hard
mel	mellow
pls	plastic
sti	stiff
stk	sticky
ten	tenaceous
tgh	tough

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STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS and RESEARCH DEPT.

BORING RECORD

1- Location DEADWOOD GULCH
2- Dist. II Co. SIS Rt. 59 Sec. C. Cont. W.O. 407-72
3- (a) 110 Ft., ~~Right~~, Left (b) PROP ℄ Line Sta. 159+65

Sample No.	Sampled	Depth	Elev.	Core Catcher Used	% Rec.	Retained Sample Core Box No.	No. Tubes	No. Jars
12	From	13	14	15	16	17	18	19
	To						Core Length	27
	From							
	To						Core Length	
	From							
	To						Core Length	
	From							
	To						Core Length	
	From							
	To						Core Length	

Rig Used 28
Type of Sampler 29
Type of Point 30 — Regular — Extension
Sample Nos.
Type of Core Barrel 31
Type Bit Used 32 — Number — From — To

GROUND WATER DATA

Time	33 DEPTH TO WATER	BOT. BOR.	Time	DEPTH TO WATER	BOT. BOR.

Boring Flushed 34 Depth
Boring Bailed 35 Depth
Boring Cased 36 Depth
Dia., & Type P.M.P. Casing
Type of Casing cap 37

At completion of boring

REMARKS

Weather: 38

Show time schedule of day's operation

Show additional location data with sketch if necessary

Show any other pertinent data not listed elsewhere in log.

Inspector 39 R.D. Lane Foreman 40 S.S. Stone

FORM T-203 REV. 1-61

TO LABORATORY

ELEV.	Datum 4 USGS B.M.	Dia. of Sampler 6 2"	Date 8 3-3-61	Page 10 1
	Surface 5 85.3	Wt. of Hammer 7 350 #	Job No. 9 2438-20-T	Boring No. 11 D-1

20 Depth	21 Per Ft.	22	23 Tubes	Wet Wt. per cu. ft. 24	Moist. % dry wt. 25	26 LOG OF MATERIAL
	2/100					Symbol: Sampler driven closed (with plug lowered)
1	4/125					
2						
				110		Tube sample weighed & discarded
	4/125		V	112		
3			VI	116		Sample retained (core length)
	6/125	1	III	114		Distance sampler driven open (with plug raised or released if stationary piston sampler)
			II	117		
			I	112		
4			A	No		
	9/200					Tube sample—weighed, capped, sealed & stored
5						
6	7/200					
7	20/200					Time in seconds & P.S.I. per ft.
			B	No		
	22		IV	120		Hammer blows per ft.
8			III	120		
		2	II	122		
	47			No		Tube sample discarded, weight not obtainable.
9			I	128		
			A	129		Jar sample, tube weighed & pushed into glass jar
	100					
10			3"			Change in diameter of sample
	20/200					
11						Symbol: Ground water elev.
	22/200					Same (Indicates that material in sample #3 is similar to #2)
12						
	19/200	3				No. of box in which core is stored
13						
	48/250	60°				Change (indicates approx. location of material change)
14						
	127/250					Slipplane dip 60° (approx.)
15						
16						Symbol for jetting
17						
18						Symbol for augering
19						Symbol for rotary drilling
20						Log of material should include the following in the order listed below
	69/200					
21						A. Moisture, from dry to very wet
	71/200	4				B. Consistency, soft, stiff, hard, loose, dense, friable, etc.
22						
	68/200					C. Color, mottled brown etc.
23						D. Grain size, fine, medium etc.
	180/200					E. Soil or rock type
24						F. Plasticity S.P, M.P, H.P
	280/200					G. Structure, laminated, fractured
25						H. Texture and/or shape, such as diameter and shape of pebbles
						I. Other pertinent characteristic such as weathering, broken & mineral comp., etc.
						End of day
						Bottom of hole (Date)

Core box #1

Fig. A — Illustrative Boring Record.

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STATE OF CALIFORNIA — DIVISION OF HIGHWAYS
MATERIALS and RESEARCH DEPT.

BORING RECORD

Location DEADWOOD GULCH
Dist. II Co. 515 Rt. 59 Sec. C Cont. W.O. 4QT72
110 Ft. ~~Right~~, Left PROP ℄ Line Sta. 159+65

Sample No.		Sampled Depth	Sampled Elev.	Core Catcher Used	% Rec.	Retained Sample Core Box No.	Retained Sample No. Tubes	Retained Sample No. Jars
1	From	2	670	No			5	1
	To	5	640				Core Length	28"
2	From	7	620	No				2
	To	10	590				Core Length	32"
3	From	10	590	Yes	100	1		
	To	15	540				Core Length	60"
4	From	20	490	Yes	80	1		
	To	25	440				Core Length	48
	From							
	To						Core Length	

Rig Used Joy 22 HD
Type of Sampler 2" Mod. Vent. Calif.
Type of Point — Regular — Extension
Sample Nos. 2 — 1

Type of Core Barrel LONGYEAR

Type Bit Used	Number	From	To
Insert type corehead	741	10	15
4½" Rock bit	OW972	17	20
Insert type corehead	741	20	25

GROUND WATER DATA

Time	Depth to Water	Depth to Bot. Bor.	Time	Depth to Water	Depth to Bot. Bor.
1200	Dry	10'			
1600	23.2'	25'			

Boring Flushed Yes — Depth 20'
Boring Bailed Yes — Depth 22'
Boring Cased Yes — Depth 25'
Dia., & Type P.M.P. Casing 2" Downspout
Type of Casing cap Tin Can

REMARKS

Weather: Cold & Cloudy

0730 - 0800	Travel to job
0800 - 1040	Sampling
1040 - 1200	Repair mud pump
1200 - 1230	Lunch
1230 - 1410	Rotary Coring
1410 - 1500	Rotary Drilling
1500 - 1600	Rotary Coring
1600	End of day

TO LABORATORY

Inspector R.D. Lane Foreman S.S. Stone

FORM T-205 REV. 1-61

ELEV. Datum USGS B.M. Surface 69.0 | Dia. of Sampler 2" | Wt. of Hammer 350 # | Date 3-3-61 | Job No. 2438-20-T | Page 2 | Boring No. D-1

Depth	Sec. Per Ft.		Tubes	Wet Wt. per cu. ft.	Moist. % dry wt.	LOG OF MATERIAL
8	2/100					
1	4/125					
2			X	110		Dry, dense, brown,
	4/125		V	112		sandy silt
3			IV	116		
			III	114		
	6/125	1	II	117		
4			I	112		
			A	No		
	9/200					
5	7/200					
6	20/200					
7			B	No		
	22 blows		V	117		Same
8			IV	120		← Change
		2	III	120		Damp, stiff, gray, silty
	47		II	122		clay M.P.
9			X	No		
			I	128		
	100		A	129		
10				3"		← Change
	20/200		(RPM)			Moist, firm, dark, gray
11						laminated, weathered
	22/200					shale
12		3		Core Box #1		
	19/200					
13						
	48/250	60°				
14						
	127/250					
15						
16						
17						
18						
19						
20						
	60/200					← Change
21						Dry, hard, gray-blue,
	71/200					fine-grained, slightly broken
22		4				slightly weathered sandstone
	69/200			Core Box #1		with some small angular
23						fragments of black shale
	180/200					and some stringers of
24						calcite
	280/200					
25						
						End of day
						Bottom of hole (Date)

Fig. A — Illustrative Boring Record (continued).

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Instructions for Filling Out Boring Record

This form is used by the field inspectors for logging sample boring materials and for recording other pertinent data and observations.

It is handwritten in the field and produces an original and three carbon copies. Check all of the carbon copies for legibility before tearing apart for distribution. The usual distribution of these copies is as follows:

Original (white) and duplicate (yellow) to M. & R. Dept. Foundation Sec. or District Lab.

Triplicate (yellow) to laboratory with samples or cores when they are delivered.

Quadruplicate (pink) retained by inspector.

The following explanation of the various phases of the form is referenced to the attached copy:

1. Location: geographical location i. e., the name of a creek, canyon, highway junction, street intersection, etc.
2. (a) District number, (b) County, (c) Legislative route number, (d) section of the route, (e) work order number.
3. Boring Location: Usually referenced to a line either proposed or established.
   (a) Distance right or left of the line. If no line is staked or can conveniently be established, use some permanent landmarks to tie in the borings such as power poles, buildings, trees, fences, etc. A sketch with measurements shown on the boring log in the "Remarks" column is helpful.
   (b) Identity of the line being used, i. e., "P", "D."
   (c) Station on the line to which the boring is referenced.
4. Datum: The datum used to arrive at the surface elevation of the boring. If a USGS, City, County, or State bench mark is in the vicinity, it should be used to determine the elevation of the boring. If no bench mark is available, we frequently use approximate elevations arrived at from a contour map which has been prepared from an aerial photo. These are usually correct within from 2 to 5 ft unless the terrain is particularly steep. If Division of Highways datum is established in the area along with others it should be used in preference to the others.
5. Elevation: The elevation of the ground surface where the boring is made.
6. Diameter of Sampler: The inside diameter measurement of the drive sample barrel or core barrel used to take the sample.
7. Weight of Hammer: The approximate weight of the hammer if used to advance the sampler.
8. Date: Specify month, day, and year the boring is being made.
9. Job No.: Laboratory Authorization — job number.
10. Page No: Number the page of the boring logs. These numbers should be continuous starting with 1 throughout the length of the job. Do not start again with 1 when beginning another boring. If there are two or more rigs on a job, it will be necessary for the inspectors to coordinate the boring numbers and log page numbers in order to avoid repetition.
11. Boring No.: Number of the boring preceded by a letter i. e., "D-1" if the beginning sample operation is a drive sample. A-auger, R-Rotary (see Standard Legend Sheet).
12. Sample No.: The number of the sample. Number each sample attempt whether there is recovery or not.
13. Depth: Overall depth of the sample attempt whether the entire length is recovered or not.
14. Elevation: Elevation corresponding to beginning and end of sample run (entire length, not just retained portion).

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15. Core Catcher Used: "Yes" or "No". This applies to drive sampling procedure when some device such as a slotted tube is inserted in the lower end of the sampler as a means of retaining soft or loose materials which have a tendency to fall out of the sample barrel.

    (a) An alternate method which is sometimes used to recover "lost" samples is to open the sampler a foot or two off of bottom and "resample" a portion of the previous sample run.

    Materials recovered by using core catcher devices or by opening the sampler off of bottom and "resampling" a "no-recovery" portion of the boring are disturbed and are not suitable as samples for testing purposes. When recovery is made by any of these methods it is important that the proper notations be made on the log in order that this material will not be selected for testing. A positive method of avoiding this happening is to place this material in jars for visual study only. The question arises as to how to show this "repeat" operation on the boring log. The suggested method is as follows:

    Indicate an interruption in the normal logging procedure by inserting a double "wavy" line or "break" in the sample number column.

    Repeat the footage depths resampled then continue with the routine logging procedure.

16. % Recovery: Length of sample run divided into length of sample recovered x 100.

17. Core Box Number: Number of box into which core is placed. These numbers should be continuous starting with 1 throughout the length of the job.

18. No. Tubes: The number of tubes from each sample run that are processed.

19. No. Jars: Number of jars prepared from each sample run for shipment to the lab.

20. Depth: Depth of hole beginning with "0" at the ground surface and numbered to correspond with depths sampled.

21. _____ per ft. Add "seconds" or "blows" whichever applies. If sampler is hydraulically pushed include pressure reading. Record this information for each foot of the boring as work progresses.

22. Blank Column: Used to fill in with symbols indicating such things as length of sample run, length of core or sample recovered, type of operation between samples such as "pushed closed," rotary drilled, etc. (See attached copy of boring log Sheet No. 1 for additional explanation).

23. Tubes: Use Roman numerals beginning with I. identifying the first tube to be processed starting at the lower end of the sampler. Letters of the alphabet with the sample run number as a prefix for jars to be sent to the lab.

24. Wet Wt. per Cu. Ft.: After weighing carefully trimmed tube, consult conversion chart and record wet weight of 1 cu ft of the material in this column. This is done for each tube saved and processed.

25. Moist % of Dry Wt.: For use of testing section.

26. Log of Material: Inspector's description of the material such as wet, soft, brown silty clay highly pastic. Refer to Table A.

27. Core Length: Length of core retained in inches.

28. Rig Used: Name of rig, e.g., 22-Joy, 250-Joy

29. Type of Sampler: Record type of sampler regardless of type. If a California sampler is used record whether it is the stationary piston, modified vented 2" California sampler, or heavy duty 2" California Sampler.

30. Type of Point: Indicate type of point used for each sample run by entering sample No. in appropriate column.

31. Type of Core Barrel: Identify the core barrel used, i.e., NX, Longyear, Reed.

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32. Type Bit Used: Whether diamonds, tungsten carbide insert, etc., indicate serial number of bit used and depth.

33. Groundwater: This is most important. Record time of the readings and also depth to water from ground surface and the depth to the bottom of the boring from ground surface. If the boring is dry make a notation to that effect.

34 to 37 TO BE COMPLETED ON LAST PAGE FOR EACH BORING

34. Boring Flushed: State "yes" or "no". If water is pumped into a boring to thin and remove drilling mud, the depth to which the flushing is done should be indicated here.

35. Boring Bailed: State "yes" or "no". Note the depth from ground surface down to which a boring is bailed.

36. Boring Cased: State "yes" or "no". Specify type, i. e., 2" downspout, and the depth to which the boring is cased.

37. Type of Casing Cap: i. e., 3" galv. nipple, coupling and plug, or "tin-can."

38. Remarks: Use time of day for sequence and related happenings pertinent to day's operations. Record any delays in normal operating routine and the duration thereof.

39. Inspector: Signature of the inspector in charge.

40. Foreman: Name of Foreman.

COMMENTS

Do NOT repeat information in the daily diary, which has been detailed on the boring log.

Use of "core catcher" type devices to recover soft or loose materials should be avoided, if possible. Suggestions for improving recovery before resorting to catcher devices are as follows:

1. Change the clearance on the extension cutting point by swaging inward or outward (depending on the type of material) a few one-thousandths of an inch. This swaging should be done with a proper tool or jig designed for the purpose in order to avoid deformation of the cutting edge.
2. Slow the rate of penetration.
3. Shorten the length of the sample run.

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