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'CHNICAL GUIDELINES FOR EXPANSIVE SOILS ... HIGHWAY SUBGRADES

June 1979
Final Report

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DEPARTMENT OF TRANSPORTATION • UNITED STATES OF AMERICA •

Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Materials Division
Washington, D.C. 20590

FOREWORD

This report provides technical guidance for application of pre- and postconstruction treatment alternatives for minimizing volume change of expansive soils, and practical design, construction, and maintenance recommendations for minimizing moisture infiltration into an expansive soil subgrade. The report will be of interest to chief highway administrators and other policy makers, as well as to engineers at the operating level.

The technical guidelines presented in this report resulted from FCP Project 4D study, "An Evaluation of Methodology for Prediction and Minimization of Detrimental Volume Change of Expansive Soils in Highway Subgrades," conducted by the U.S. Army Engineer Waterways Experiment Station (WES). The research was performed by the WES for FHWA under Purchase Order No. 4-1-0195, during the period July 1, 1974, to June 30, 1979.

Acknowledgment is given to the following advisory group members who provided consultation to the program:

Mr. Bud Brakey (Colorado)
Mr. Paul Teng (Mississippi)
Mr. Gene Morris (Arizona)
Mr. Malcolm Steinberg (Texas)
Mr. Gene McDonald (South Dakota)

and the following States for actively participating in the program:

Mississippi
Texas
Colorado
Montana
South Dakota
Arizona

Although clay soils can be found throughout the United States, expansive clays are not a nationwide problem. Accordingly, sufficient copies of the report are being distributed by FHWA Bulletin to provide a minimum of one copy to each FHWA Regional Office, one copy to each FHWA Division Office, and one copy to each State highway agency. Additional copies are being sent to States and division offices in those geographical areas that are significantly affected by expansive soils which exhibit a wide spectrum of engineering behavior.

Charles F. Galambos/for
Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

U.S. Federal Highway Administration.

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16. Abstract

Volume change of expansive soil subgrades resulting from moisture variations causes an estimated annual damage of $1.7 billion to streets and highways. Minimization of the detrimental damage to pavements on expansive soils was the subject of a 4-yr research study conducted by the U. S. Army Engineer Waterways Experiment Station. The results of that study that should be implemented are presented in this report. Technical guidelines are presented on: the location of potentially expansive soil areas using occurrence and distribution maps, as well as alternative sources of information; field exploration and sampling of expansive soils; identification and classification of potentially expansive soils using index and soil suction properties; testing of expansive soils and prediction of anticipated volume change; selection of appropriate treatment alternatives; and presentation of design, construction, and maintenance recommendations for new and existing highways. Appendixes to the technical guidance report describe the soil suction test procedure, a standard procedure for odometer swell tests, procedure for calculating the Potential Vertical Rise (PVR), a bibliography on treatment alternatives, and standards for field monitoring data.

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PREFACE

The study of the methodology for prediction and minimization of detrimental volume change of expansive soils in highway subgrades is a 4-year investigation funded by the Department of Transportation, Federal Highway Administration, under Intra-Government Purchase Order No. 4-1-0195, Work Unit No. FCP 34D1-132.

The work was initiated during June 1974 by the Geotechnical Laboratory (GL) of the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi. Dr. Donald R. Snethen, Research Group, Soils Mechanics Division (SMD), GL, was the principal investigator during the period of this report. The work reported herein was performed by Dr. Snethen; Dr. Frank C. Townsend and Dr. Lawrence D. Johnson, Research Group, SMD, GL; and Dr. David M. Patrick, Research Group, Engineering Geology and Rock Mechanics Division, GL. The Technical Guidance Report was prepared by Dr. Snethen. The investigation was accomplished under the general supervision of Mr. C. L. McAnear, Chief, SMD, and Mr. J. P. Sale, Chief, GL.

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

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
angstroms	0.001	micrometres
Atmospheres (normal = 760 torr)	101.325	kilopascals
Atmospheres (technical 1 kgf/cm^2)	98.0665	kilopascals
Fahrenheit degrees	5/9	Celsius or Kelvin degrees*
feet	0.3048	metres
gallons (U. S. liquid)	3.785412	cubic decimetres
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6894.757	pascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (force) per square foot	95.76052	kilopascals

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9)(F - 32). To obtain Kelvin (K) readings, use: K = (5/9)(F - 32) + 273.15.

TECHNICAL GUIDELINES FOR EXPANSIVE SOILS IN HIGHWAY SUBGRADES

PART I: INTRODUCTION

Background

1. Volume change of expansive soil subgrades resulting from moisture variations frequently cause severe pavement damage. Highways constructed in the Southwest, Western Mountain, Central Plains, and Southeast geographical areas are particularly susceptable to these types of damage. A 1972 survey[1] of the highway departments in the 50 states, District of Columbia, and Puerto Rico indicated that 36 states have expansive soils within their geographical jurisdiction. Expansive soils are so areally extensive within parts of the United States that alteration of the highway routes to avoid the material is virtually impossible. The annual cost of damage to streets and highways caused by expansive soils was conservatively estimated in 1973 to exceed $1.14 billion.[2] Simply applying the cumulative sum of the annual rate of inflation[3] since that estimate was made (1972-78, cumulative sum = 49.6 percent) and without consideration of additional construction the current annual cost of damage to streets and highways is approximately $1.705 billion.

2. In an effort to minimize the detrimental volume change of expansive soils in highway subgrades and the associated pavement damage the U. S. Army Engineer Waterways Experiment Station (WES) has conducted a 4-year study which had as its major purpose to evaluate and make recommendations concerning the major aspects of the expansive soils in highway subgrades problem; namely, description of the occurrence and distribution of expansive soils on a physiographic area basis, definition and verification of the roles of the microscale mechanisms that cause volume change, evaluation of expedient methodology for identification and classification of potentially expansive soils, evaluation of testing methodology for expansive soils and prediction of anticipated volume change,

evaluation and development of recommendations for appropriate treatment alternatives for pre- and postconstruction applications, and preparation of recommendations for practical procedures for the design and construction of new pavements and maintenance of existing pavements. The results of the research study have been reported in four interim reports[4-7] and a final report.[8,9]

Purpose

3. The purpose of this technical guidance report is to present in abbreviated form the results of research study in a format suitable for implementation by the various State Transportation Agencies that encounter problems with expansive soils in highway subgrades. The technical guidance will be in the form of recommendations or criteria covering the identification, quantification, and treatment of expansive soils in highway subgrades to minimize detrimental volume change.

Scope

4. The technical guidance provided within this publication is designed to minimize the volume change characteristics of expansive soils resulting from moisture content variations. Volume change problems resulting from chemical or biological related (i.e. sulfate precipitation or pyrite oxidation) phenomena and frost heave are specifically excluded from consideration.

Applicability

5. The technical guidance is applicable for use by State Transportation Agency personnel including geotechnical engineers, geologists, highway planning and design engineers, and agency management personnel involved in design, construction, and maintenance of highways on expansive soil subgrade.

PART II: RECOGNITION OF POTENTIAL SWELL PROBLEM AREAS

6. The first important decision in the design and construction sequence for a highway is the route selection. Route selection is often influenced by local social, economic, environmental, and/or political considerations prevalent at the time of design. Oftentimes the geologic materials (and the associated problems) traversed by the selected route are not considered until the collection of parameters for the pavement design. For expansive soils, it is important to recognize the existence of the problem and have a qualitative indication of the extent of the potential swell problem as early in the design and construction sequence as possible. This part of the report provides the technical guidance necessary to determine the location of potentially expansive soils and qualitatively estimate the extent of the potentially expansive soil problem. A major task in the research program involved a survey of the manifestations of expansive soils based on physiographic areas. The results of that research task were a series of occurrence and distribution maps[5] that provide a basis for identifying potential expansive soil problem areas.

Occurrence and Distribution Maps

7. Beneficial use of the occurrence and distribution maps is based on an understanding of the information used to develop the maps and accompanying discussions that add to the effectiveness of the maps. To provide this basis for understanding, selected sections of the Task B report[5] are presented in the following paragraphs along with the maps and tabular summary which help to explain the basis for the maps.

Basis for classification

8. The categorization and classification methods used to develop the occurrence and distribution maps are subjective and are based primarily upon the estimated volume change of argillaceous materials within

the geologic unit, the presence of montmorillonite, geologic age, and reported problems due to expansive materials. The approach used is essentially geologic in that stratigraphy and mineralogy are considered to be key elements. Pedology on the other hand is not considered to be as important regionally, although it may have local significance.

9. The distribution of expansive materials is categorized by geologic unit on the basis of (a) degree of expansiveness and (b) expected frequency of occurrence. The degree of expansiveness relates to the expected presence of montmorillonite, whereas the frequency of occurrence involves the amount of clay or shale in the geologic unit. Three major sources of information formed the bases for classificational decisions:

- a. The reported occurrences of expansive materials as indicated in published literature or other sources of data which revealed actual problems or failures due to expansive materials.[10] These sources were not necessarily limited to highway subgrades.
- b. Materials maps provided summaries of illustrated earth material properties pertinent to this study.[11] These materials maps were used to delineate areas of argillaceous materials, and the soils surveys were used to substantiate suspected occurrences of expansive materials.
- c. Geologic maps and cross sections were used to identify and delineate areas of argillaceous rocks and sediments that were believed to possess expansive properties.[12,13,14]

10. These three general sources were combined to produce four mapping categories that reflect the degree of expansiveness in terms of volume change and expected frequency of occurrence. The four categories are as follows:

- a. High. Highly expansive and/or high frequency of occurrence.
- b. Medium. Moderately expansive and/or moderate frequency of occurrence.

c. Low. Generally of low expansive character and/or low frequency of occurrence.

d. Nonexpansive. These areas are mainly underlain by materials that, by their physical makeup, do not exhibit expansive properties and that, upon weathering, do not develop expansive soils.

11. The following premises guided the map categorization:

a. Any area underlain by argillaceous rocks, sediments, or soils will exhibit some degree of expansiveness.

b. The degree of expansiveness is a function of the amount of expandable clay minerals present.

c. Generally, the Mesozoic and Cenozoic rocks and sediments contain significantly more montmorillonite than the Paleozoic (or older) rocks.

d. Areas underlain by rocks or sediments of mixed textural compositions (e.g., sandy shales or sandy clays) or shales or clays interbedded with other rock types or sediments are considered on the basis of geologic age and the amount of argillaceous material present.

e. Generally, those areas lying north of the glacial boundary are categorized as nonexpansive due to the cover of glacial drift. Whether the drift itself is expansive is a function of drift texture and the mineralogy of the source material. The till deposited in Montana and the Dakotas is partially composed of material derived from expansive, Cretaceous shales in this region; thus, this till may show considerably more expansive properties than tills in other regions. Also, the argillaceous sediments deposited in Pleistocene lakes may be of such texture and mineralogy that they also possess limited expansive properties.

f. From a regional standpoint, those soils derived from the weathering of igneous and metamorphic rocks are considered nonexpansive. Such soils may contain some expansive clay minerals but their concentration and the general soil texture preclude appreciable volume change. Also, in temperate areas such soils are usually limited in thickness.

g. The categorization does not consider climate or other environmental aspects.

h. Argillaceous rocks or sediments originally composed of expandable-type clay minerals do not exhibit significant volume change when subjected to tectonic folding, deep burial, or metamorphism.

i. Volcanic areas consisting mainly of extruded basalts and kindred rocks may also contain tuffs and volcanic ash deposits that have devitrified and altered to montmorillonite.

j. Areas along the glaciated boundary may have such a thin cover of drift that the expansive character of the materials under the drift may predominate.

12. The twenty first-order physiographic provinces[12] are shown in Figure 1. The occurrence and distribution of expansive material by FHWA regions are shown in Figures 2-6, and the potentially expansive geologic units are summarized in Table 1. A narrative description of expansive materials within each of the physiographic provinces is presented in subsequent paragraphs.

Distribution of expansive materials by physiographic province

13. In the following paragraphs the twenty first-order physiographic provinces are discussed in terms of the potentially expansive materials within them. The general lithology, geologic age, stratigraphic association, and mineralogy (if known) are presented and the relative degree of expansiveness estimated. The narrative descriptions should be used in conjunction with Figures 2-6 to qualitatively indicate the extent of potentially expansive materials. For those State Transportation Agency personnel not interested in detailed geologic descriptions as provided in paragraphs 14 through 45, procedures for recommended usage of the occurrence and distribution maps begin with paragraph 46, page 34. The detailed geologic information included in paragraphs 14 through 45 may be used as additional reference material in conjunction with utilization of the maps.

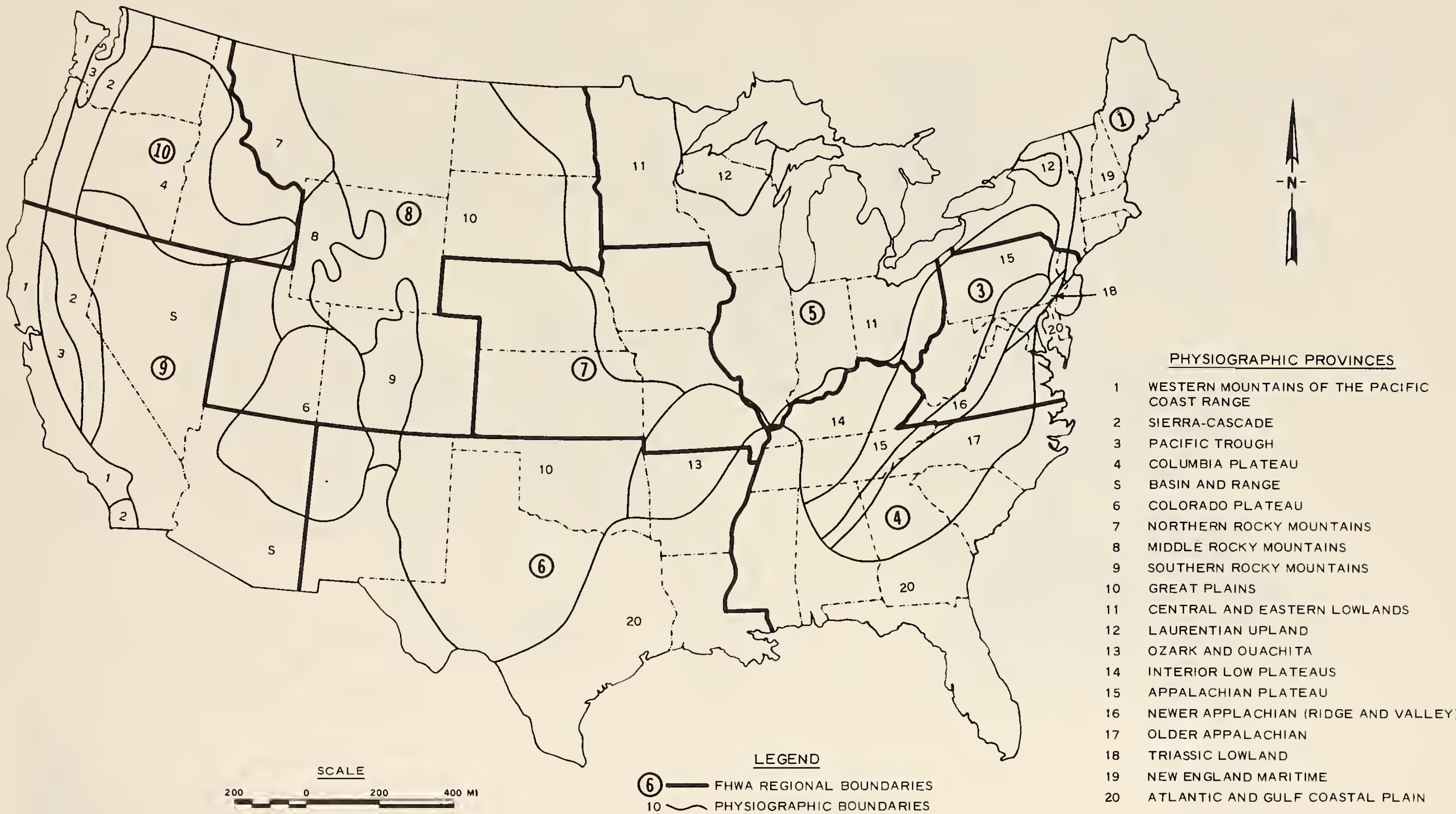

Figure 1. First-order physiographic provinces within the continental United States

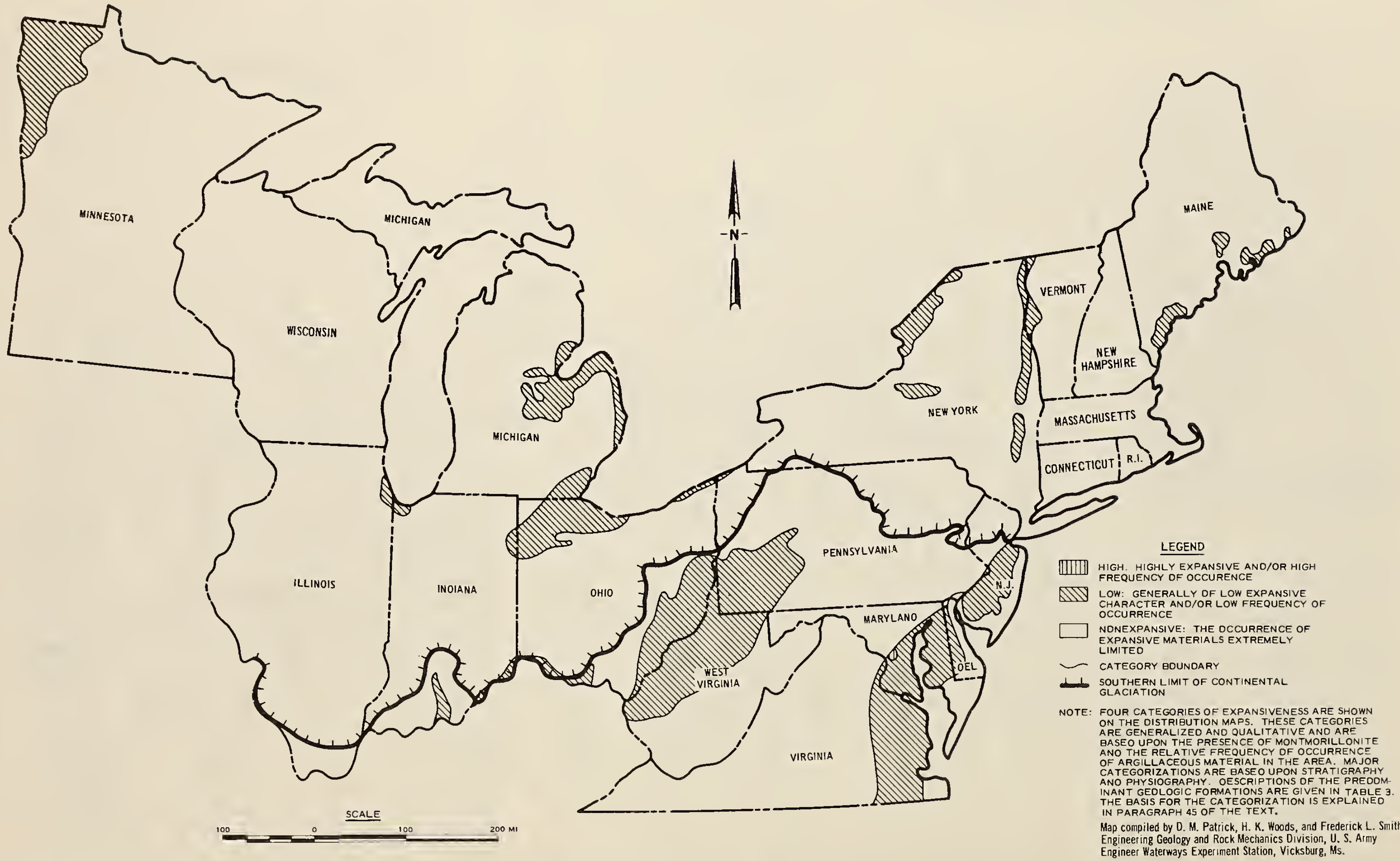

Figure 2. Distribution of potentially expansive materials in the United States; FHWA Regions 1, 3, and 5

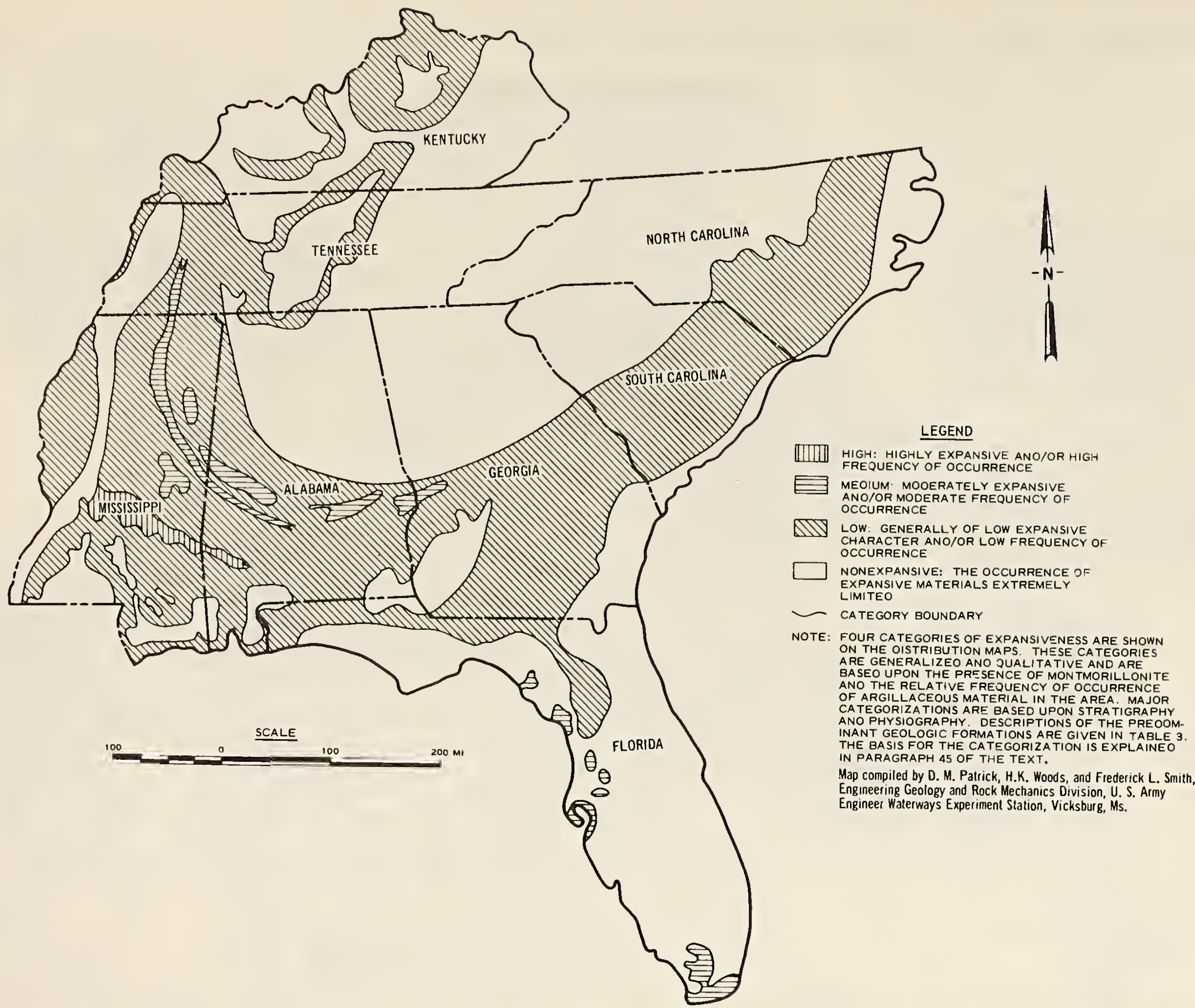

Figure 3. Distribution of potentially expansive materials in the United States; FHWA Region 4

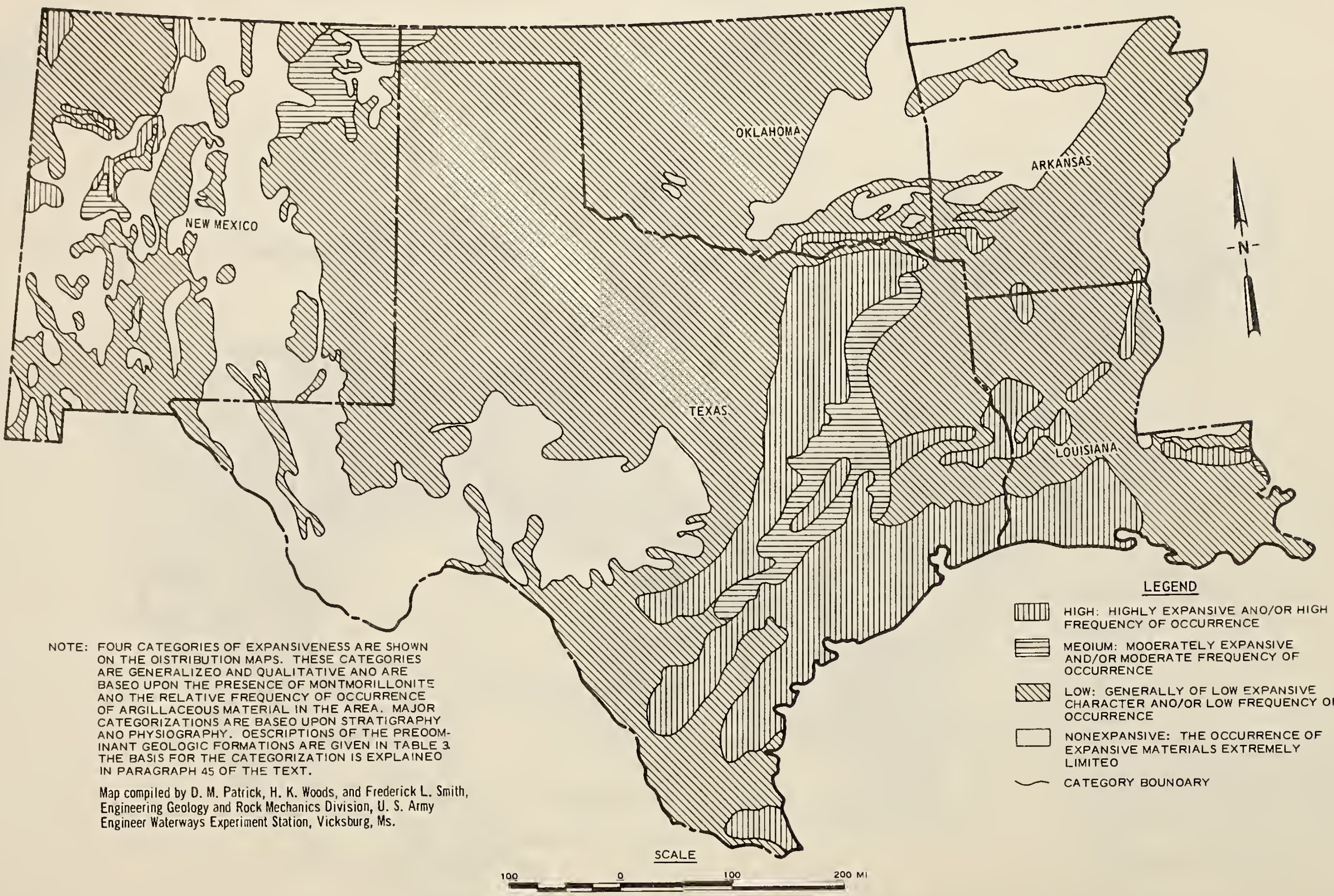

Figure 4. Distribution of potentially expansive materials in the United States; FHWA Region 6

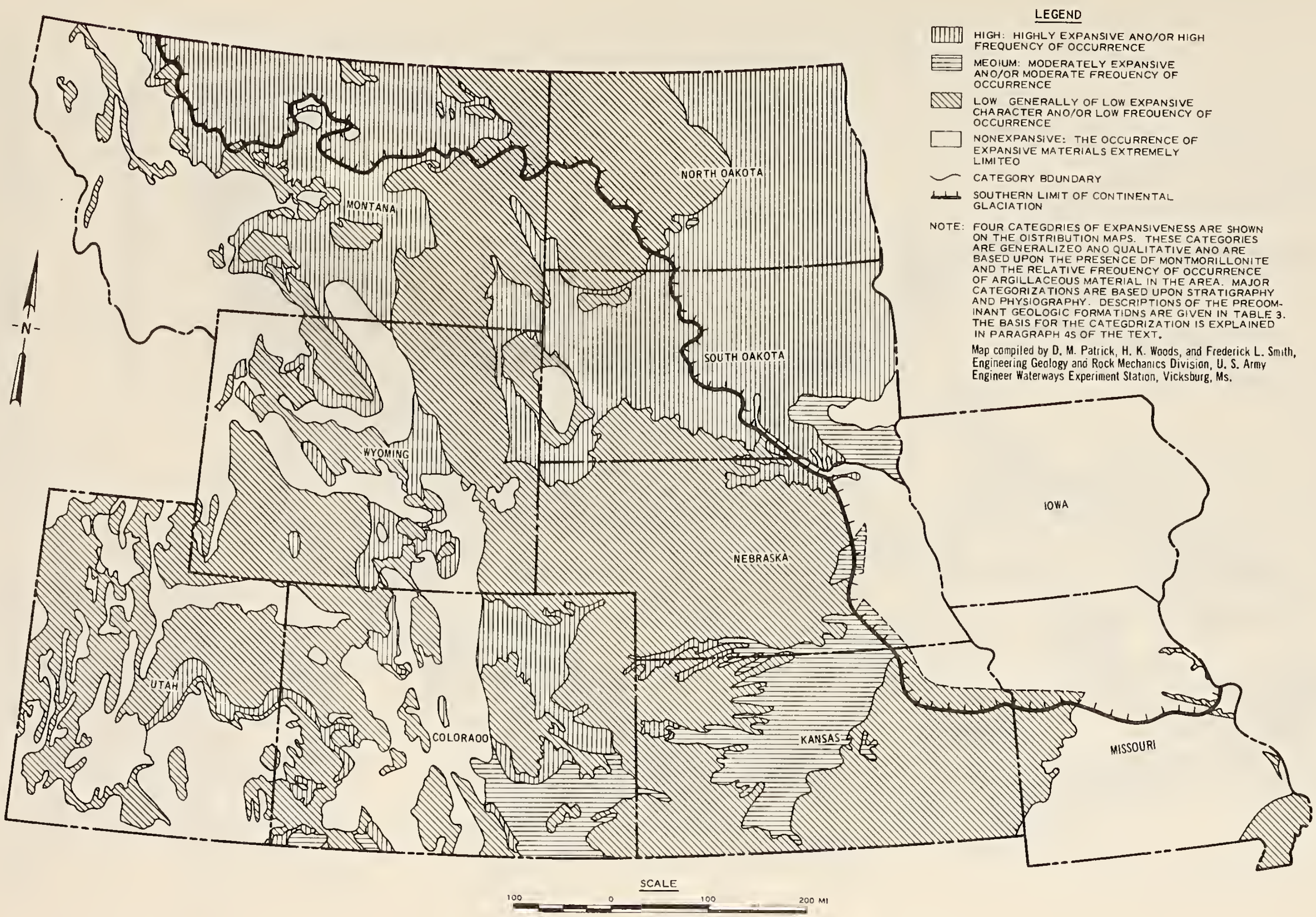

Figure 5. Distribution of potentially expansive materials in the United States; FHWA Regions 7 and 8

Figure 6. Distribution of potentially expansive materials in the United States; FHWA Regions 9 and 10

Table 1

Tabulation of Potentially Expansive Materials in the United States

Physiographic Province No.*	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map** Category	Remarks
1	Western Mountains of the Pacific Coast Range	Reefridge	Miocene	CA	1	The Tertiary section generally consists of interbedded sandstone, shale, chert, and volcanics
		Monterey	Miocene	CA	1	
		Rincon	Miocene	CA	1	
		Tembler	Miocene	CA	1	
		Umpqua	Paleocene-Eocene	OR	3	
		Puget Gp	Miocene	WA	3	Interbedded sandstones and shales with some coal seams
		Chico Fm	Cretaceous	CA	1	
2	Sierra Cascade	Cascade Gp	Pliocene	OR	4	Predominate material is volcanic
		Columbia Gp	Miocene	WA	4	Interbedded sandstones and shales may occur throughout, particularly in western foot hills
		Volcanics	Paleozoic to Cenozoic	NV	4	
		Volcanics	Paleozoic to Cenozoic	CA	4	
3	Pacific Trough	Troutdale	Pliocene	WA	3	Great Valley materials characterized by local areas of low-swell potential derived from bordering mountains. Some scattered deposits of bentonite
		Santa Clara	Pleistocene	CA	3	
		Riverbank	Pleistocene	CA	3	
4	Columbia Plateau	Volcanics	Cenozoic	WA, OR, ID, NV	4	Some scattered bentonites and tuffs
5	Basin and Range	Valley fill materials	Pleistocene	OR, CA, NV, UT, AZ, NM, TX	3	Playa deposits may exhibit limited swell potential. Some scattered bentonites and tuffs
		Volcanics	Tertiary	OR, CA, NV, UT, AZ, NM, TX		
6	Colorado Plateau	Greenriver	Eocene	CO, UT, NM	3	
		Wasatch	Eocene	CO, UT, NM	3	
		Kirkland shale	Upper Cretaceous	CO, UT, NM, AZ	2	Interbedded sandstones and shales
		Lewis shale	Upper Cretaceous	CO, UT, NM, AZ	2	
		Mancos	Upper Cretaceous	CO, UT, NM, AZ	1	
		Mowry	Upper Cretaceous	CO, UT, NM, AZ	1	
		Dakota	Jurassic-Cretaceous	CO, UT, NM, AZ	3	Interbedded sandstones and shales
		Chinle	Triassic	NM, AZ	1	

(Continued)

* Refer to map of physiographic provinces, Figure 1.
** Numerical map categories correspond as follows: 1 - high expansion, 2 - medium expansion, 3 - low expansion, and 4 - nonexpansive.

Table 1 (Continued)

Physiographic Province						
No.	Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category	Remarks
7	Northern Rocky Mountains	Montana Gp	Cretaceous	MT	1	Locally some sandstone and siltstone
		Colorado Gp	Cretaceous	MT	2	Locally some siltstone
		Morrison	Jurassic	MT	3	Shales, sandstones, and limestones
		Sawtooth	Jurassic	MT	3	
8	Middle Rocky Mountains	Windriver	Eocene	WY, MT	3	
		Fort Union	Eocene	WY, MT	3	
		Lance	Cretaceous	WY, MT	1	
		Montana Gp	Cretaceous	WY, MT	1	
		Colorado Gp	Cretaceous	WY, MT	2	
		Morrison	Jurassic-Cretaceous	WY, MT	3	
9	Southern Rocky Mountains	Metamorphic and granitic rocks	Precambrian	WY	4	Montana and Colorado Gps may be present locally with some Tertiary volcanic and minor amounts of Pennsylvania limestone (sandy or shaly).
		Metamorphic and granitic rocks	Precambrian	CO	4	
		Metamorphic and granitic rocks	Precambrian to Cenozoic	NM	4	
10	Great Plains	Fort Union	Paleocene	WY, MT	3	
		Thermopolis	Cretaceous	WY, MT	1	
		Montana Gp	Cretaceous	WY, MT, CO, NM	1	
		Colorado Gp	Cretaceous	WY, MT, CO, NM	2	
		Mowry	Cretaceous	WY, MT, CO, NM	1	
		Morrison	Jurassic-Cretaceous	WY, MT, CO, NM	3	
		Ogallala	Pliocene	WY, MT, CO, NM, SD, NE, KS, OK, TX	3	Generally nonexpansive but bentonite layers are locally present
		Wasatch	Eocene	MT, SD	3	
		Dockum	Triassic	CO, NM, TX	3	
		Permian Red Beds	Permian	KS, OK, TX	3	
		Virgillian Series	Pennsylvanian	NE, KS, OK, TX, MO	3	
		Missourian Series	Pennsylvanian	KS, OK, TX, MO	3	
		Desmonian Series	Pennsylvanian	KS, OK, TX, MO	3	
11	Central and Eastern Lowlands	Glacial lake deposits	Pleistocene	ND, SD, NM, IL, IN, OH, MI, NY, VT, MA, NE, IA, KS, MO, WI	3	Some Paleozoic shales locally present which may exhibit low swell

(Continued)

Table 1 (Continued)

Physiographic Province No.	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category	Remarks
12	Laurentian Uplands	Keweenawan	Precambrian	NY, WI, MI	4	Abundance of glacial material of varying thickness
		Huronian	Precambrian	NY, WI, MI	4	
		Laurentian	Precambrian	NY, WI, MI	4	
13	Ozark and Ouachita	Fayetteville	Mississippian	AR, OK, MO	3	May contain some montmorillonite in mixed layer form
		Chickasaw Creek	Mississippian	AR, OK, MO	3	
14	Interior Low Plains	Meramac Series	Mississippian	KY	3	
		Osage	Mississippian	KY, TN	3	
		Kinderhook	Mississippian	KY, TN	3	
		Chester Series	Mississippian	KY, IN	3	Interbedded shale, sandstone, and limestone
		Richmond	Upper Ordovician	KY, IN	3	
		Maysville	Upper Ordovician	KY, IN	3	
		Eden	Upper Ordovician	KY, IN	3	
15	Appalachian Plateau	Dunkard Gp	Pennsylvanian-Permian	WV, PA, OH	3	Interbedded shale, sandstone, limestone, and coal
16	Newer Appalachian	See Remarks	See Remarks	AL, GA, TN, NC, VA, WV, MD, PA	4	A complex of nonexpansive Precambrian and Lower Paleozoic metasedimentary and sedimentary rocks
17	Older Appalachian	See Remarks	Paleozoic	AL, GA, NC, SC, VA, MD	4	A complex of nonexpansive metamorphic and intrusive igneous rocks
18	Triassic Lowland	Newwark Gp	Triassic	PA, MD, VA	4	
19	New England Maritime	Glacio-marine deposits	Pleistocene	ME	3	Pleistocene marine deposits underlain by nonexpansive rocks. Local areas of clay could cause some swell potential
20	Atlantic and Gulf Coastal Plain	Talbot and Wicomico Gps	Pleistocene	NC, SC, GA, VA, MD, DE, NJ	4	Interbedded gravels, sands, silts, and clays
		Lumbee Gp	Upper Cretaceous	NC, SC	3	Sand with intermixed sandy shale
		Potomac Gp	Lower Cretaceous	DC	3	Sand with definite shale zones
		Arundel Fm	Lower Cretaceous	DC	1	
		Continental and marine coastal deposits	Pleistocene to Eocene	FL	4	Sands underlain by limestone, local deposits may show low swell potential

(Continued)

Table 1 (Concluded)

Physiographic Province No.	Physiographic Province Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category	Remarks
20	Atlantic and Gulf Coastal Plain (Cont'd)	Yazoo Clay	Eocene	MS, LA	1	A complex interfacing of gravel, sand, silt, and clay. Clays show varying swell potential
		Porters Creek Clay	Paleocene	MS, AI, GA	1-3	
		Selma Gp	Cretaceous	MS, AI, GA	2-3	
		Loess	Pleistocene	LA, MS, TN, KY	4	A mantle of uniform silt with essentially no swell potential
		Mississippi alluvium	Recent	LA, MS, AR, MO	3	Interbedded stringers and lenses of sands, silts, clays, marl, and chalk
		Beaumont-Prairie Terraces	Pleistocene	LA, MS, TX	1	
		Jackson, Claiborne, Midway	Paleocene-Oligocene	LA, MS	1-3	
		Navarre, Taylor, Austin	Upper Cretaceous	TX	1-2	
		Eagleford, Woodbine	Upper Cretaceous	TX	3	
		Washita	Lower Cretaceous	TX, OK	1-3	
		Fredricksburg	Lower Cretaceous	TX	3	
		Trinity	Lower Cretaceous	TX	4	

14. <u>Western Mountains of the Pacific Coast Range Province (1).</u> This physiographic province extends along the Pacific coast from Washington State to California and consists mainly of a series of mountains termed the "Coast Ranges." These mountains are made of folded and faulted Tertiary rocks lying on a Mesozoic and older basement. The lithologies represented by these Tertiary materials include most sedimentary rock types and some igneous and metamorphic material. The Tertiary sequence in the Coast Ranges of Washington State consists of more or less alternating sandstones, shales, and volcanics. The relatively subordinate amount of shale suggests the classification as low. The Tertiary materials in Oregon that may pose limited problems due to expansiveness are confined mainly to Paleocene and Eocene age strata. This sequence consists of shale and sandstone with some interbedded volcanics and is classified as low. In California, the potentially expansive materials in this province are mainly restricted to the Upper Cretaceous Chico formation and portions of the Eocene and Miocene series. Significant Eocene and Miocene units are, respectively, the Anita and Cozy Dell shales and the Reef Ridge and Rincon shales. The intermontane Quaternary alluvial deposits may also exhibit expansive properties. The Chico formation is classified as high and Tertiary and Quaternary materials are classified as low.

15. <u>Sierra-Cascade Province (2).</u> This physiographic province lies to the east of the Coast Ranges in Washington and extends south through Oregon into California where it terminates at the Transverse Ranges (an extension of the Coast Ranges). The province continues south of the Transverse Ranges and continues into Baja, California. The materials in the province are considered to be mainly nonexpansive and include a wide variety of intrusive and extrusive igneous, metamorphic, and deformed sedimentary rocks.

16. <u>Pacific Trough Province (3).</u> This physiographic province is a large, noncontinuous, alluvium-filled valley between the Coast Ranges and the Sierra-Cascade Province. The valley extends south from Puget Sound to Eugene, Oregon. In California, the valley's northern end is at the Klamath Mountains and extends south to the Transverse Ranges. In

Washington and Oregon, the Quaternary valley materials are mainly granular and are classed as low. The Great Valley of California also contains Quaternary alluvial gravel but the gravels are underlain by Jurassic and Cretaceous shale and sandstone, which outcrop along the margins of the valley. The alluvial fill material in the valley consists predominantly of sand and gravel, but finer grained silts and clays may be locally abundant. The alluvium is classifed as low and the Cretaceous rocks are classified high.

17. Columbia Plateau Province (4). This physiographic province occupies portions of Washington State, Oregon, Nevada, and northwestern Wyoming. The rocks in these areas consist primarily of volcanic igneous materials of Tertiary and Quaternary age. The volcanics are classifed as nonexpansive. These volcanics are locally overlain by Quaternary clastic material of alluvial origin, which may contain sufficient fine-grained interbeds or volcanically derived material to warrant classification as low.

18. Basin and Range Province (5). This physiographic province occupies most of Nevada and portions of Oregon, California, Utah, Idaho, Arizona, New Mexico, and Texas. The region is characterized by uplifted, fault-bounded mountain blocks separated by valley areas containing lacustrine and alluvial materials derived from erosion of the adjacent mountains. The mountain blocks consist of a diverse lithologic assemblage of igneous, sedimentary, and metamorphic rocks. Probably the most common surface rocks, in decreasing frequency of occurrence, are Tertiary volcanics; Paleozoic carbonates; Mesozoic carbonates, sandstone, and shale; and Precambrian igneous and metamorphic rocks. Although the volcanic rocks may locally contain some tuffs and related glassy rocks that may be montmorillonitic, the mountain block areas are classified as nonexpansive. The materials in the intermontane valleys are mainly Quaternary in age and consist primarily of sand and gravel with subordinate amounts of clayey silts and occasional bentonite deposits. The finer-grained material usually occurs either in alluvial deposits, considerably beyond the peripheries of the mountain blocks where coarser material is deposited, or in lacustrine areas. The bentonite is usually restricted

to lacustrine enviornments. The occurrence of the fine-grained material may locally be quite extensive, such as in the depositional areas of the former Pleistocene Lake Searles in southern California, Boneville in Utah, and Lahontan in Nevada and California.[11] These lake deposits may, however, contain carbonates and evaporites as well as active clays. The overall classification of these alluvial and lacustrine deposits is low.

19. Colorado Plateau Province (6). The Colorado Plateau Physiographic Province encompasses southeast Utah, southwest Colorado, northwest New Mexico, and northeast Arizona. Structurally, the strata are relatively undeformed and flat lying. The greater proportion of the exposed stratigraphic section ranges in age from Permian to Tertiary, and the units that possess expansive properties are Triassic, Cretaceous, and Tertiary in age. The Permian units are predominantly limestone and sandstone and very minor occurrences of shale and limestone. The Permian and Jurassic areas are classified as nonexpansive. Triassic units mainly occur in Arizona and Utah. The sequence bears some relationship to the Permian "Red Bed" association in Texas and Oklahoma in that red shales and reddish coarser clastics are conspicuous. These Triassic rocks consist of the progressively older Wingate sandstone, Chinle shale, Shinarump conglomerate, and Moenkopi formation. The Chinle shale is a red to grey, highly argillaceous mudstone or shale containing some sandy facies.[11] This shale is mapped as highly expansive throughout the Colorado Plateau although the most serious problems with it have occurred in eastern Arizona. The lower Cretaceous Dakota formation and the youngest portion of the upper Cretaceous consist mainly of coarse clastics with some shale. These units are mapped as low. However, the lower upper Cretaceous Mancos formation, which is roughly equivalent to the Colorado Group of the Great Plains Province, is locally highly argillaceous and is classified as highly expansive.[11] The Mancos shale contains some sand facies and extends around the north side of the Colorado Plateau in Utah and Colorado. Tertiary units on the Colorado Plateau consists of a diverse sequence of coarse clastics, clays, limestone, coal, and volcanics (including some bentonites). These materials are mainly alluvial and lacustrine deposits derived from the erosion of

uplifted areas within but mainly peripheral to the province. Their origin and nature are quite similar to the Tertiary units in the Great Plains Province. The Tertiary on the Colorado Plateau is classified as low on the basis of the predominant occurrence of sand and gravel. However, some Tertiary units are considerably argillaceous, particularly the Eocene Green River Formation on the north flank of the Plateau in Utah and Colorado, and thus locally may be classified as high.

20. Northern, Middle, and Southern Rocky Mountain Provinces (7,8,9). These three provinces together comprise the Rocky Mountain System or chain that extends from the Canadian border in Washington, Idaho, and Montana into northern New Mexico. The terrane of the Northern Rocky Mountain Province consist primarily of deformed Precambrian metasedimentary and intrusive and extrusive igneous rocks. These materials are classified as nonexpansive. Small areas of Quaternary alluvial material, however, are present, and these are considered to be low. The Middle Rocky Mountain Province is located in western Wyoming and northeastern Utah. This region is characterized by the occurrence of broad, usually elongated, domal uplifts. The uplifted areas consist mainly of Precambrian igneous and metamorphic rocks although younger, Paleozoic and Mesozoic strata are present on the peripheries of the uplifts. These younger materials usually dip away from the uplift and are occasionally deformed. The rocks in the core of the uplifts and the peripheral to the core may generally be classified as nonexpansive. The peripheral rocks, if relatively undeformed, may exhibit expansive properties. The areas in between the uplifts are included in the Great Plains Province. The rocks peripheral to the uplift also occur on the Great Plains and thus the potential for expansion around the uplifts may be determined from the same rocks on the plains depending on the deformation. The lithology and character of the Southern Rocky Province bears a certain degree of similarity to the Middle Rocky Mountain Province with the possible exception of more volcanic rocks in the former. Generally, the rocks intimately associated with the uplifted mountain areas will exhibit low potential for expansion, while those strata farther removed from the uplift will exhibit properties similar to the same material occurring in

the adjacent Great Plains and Colorado Plateau Provinces.

21. Great Plains Province (10). The Great Plains Physiographic Province is a region of gently undulating topography occupying portions of Texas, New Mexico, Oklahoma, Colorado, Kansas, Missouri, Nebraska, Wyoming, Montana, and North and South Dakota. The potentially expansive materials in this large area are diverse with respect to composition, source, distribution, and age. Also, geologic units present in this province occur elsewhere in adjacent provinces, but there is no genetic relationship between the source of the expansive materials and this particular province. The ages of these materials range from Pennsylvanian to Tertiary. Generally, the Mesozoic and Tertiary units contain the highest frequency of occurrence of expansive materials. Mississippian and younger rocks present are much less expansive as a whole although individual units may locally pose problems. Structural deformation of rocks and sediments in this province is minor, and the materials are relatively undeformed. Local uplifts within the province and uplifted areas near or at the province borders have produced some degree of deformation that may affect the degree to which these materials exhibit expansive properties.

22. Argillaceous rocks of Pennsylvanian age occur throughout this province; however, the most extensive areas of occurrence are in Oklahoma, Kansas, Missouri, and Texas. The rocks of this age that occur in the other states of this province are generally restricted to areas of uplift where they occupy narrow bands around the uplifted area; those occurring in north central Texas comprise a sequence of interstratified limestone and shale with some sandstone. In Oklahoma, Kansas, and Missouri, the Pennsylvanian units are somewhat similar to those in Texas except for some increase in the amount of sandstone and coal. These units are classified as low throughout the province on the basis of amount of shale present in the stratigraphic section and potential amount of montmorillonite present in the shales.

23. Permian units are widespread in Texas, New Mexico, Oklahoma, Kansas, and southern Nebraska. The overall lithology of the Permian is mixed compositionally although two associations are predominant. One is

the "Red Bed" association that consists of red shale, sandstone carbonate, and evaporites. The other is an association of evaporites, carbonates, and shale. The "Red Bed" association extends from Texas through Oklahoma and Kansas into Nebraska. The relatively large proportion of shale, the potential for montmorillonite occurrence, and the clay in the carbonates are the bases for classifying the "Red Bed" association as low. On the other hand, the evaporite-carbonate association, occurring mainly in New Mexico, is classified as nonexpansive.

24. Triassic units occur in Texas, New Mexico, Colorado, Wyoming, Montana, and South Dakota. These units consist primarily of shale and sandstone and bear some similarity to the "Red Bed" association of the underlying Permian. Stratigraphic names applied to these units are: Dockum Group (Texas), Dockum or Lykins formation (New Mexico and Colorado), Spearfish formation (Montana and South Dakota), and Chugwater (Wyoming). Aside from New Mexico and Texas where these Triassic units are relatively widespread, they are more or less restricted in other areas to the peripheries of uplifts and are classified as low.

25. Units of Jurassic age occur throughout the province although they are not particularly extensive. These units are predominantly shale, siltstone, and sandstone and, although they may locally be troublesome, are classified as low. Commonly used stratigraphic names include the Morrison and Sundance formations.

26. Probably the most widespread and troublesome units in this province are those of Cretaceous age. These units consist mainly of shales and limestones, of which many are argillaceous.[11] The reason for the predominance of these lithologies is the widespread inundation of the continental interior that resulted in the deposition of clay and limestone. Also, during the Cretaceous, volcanoes were active along the borders of these Cretaceous seas. Volcanic debris was carried by air currents and by streams toward the sea basins where it was deposited along with material derived by weathering in the source area. The volcanic debris was subsequently altered to montmorillonite by diagenesis. The amount of volcanic debris deposited in a particular location was dependent upon the location, extent, and duration of vulcanism and

various sedimentological aspects of the ocean basin. In any event, under favorable conditions, thick, relatively pure accumulations of volcanic debris were deposited. These accumulations, after diagenesis, resulted in the formation of bentonite. The bentonite may occur as thick (3 ft*) relatively pure montmorillonite; as thinly bedded laminae (a few tenths of an inch); or as disseminations mixed with other sediments. Regardless of purity, size, etc., the overall mode of origin of these deposits is the same. Generally, the Cretaceous units are subdivided into an upper series and a lower series. The upper Cretaceous includes the thick, extensive, and argillaceous Pierre formation and an overlying sequence of sandstones and siltstones. These units are sometimes called the "Montana Group." Included in the upper series and underlying the Pierre formation is a sequence of alternating shale and limestone or marl. This sequence consists of the progressively older Niobrara limestone, Carlile shale, Greenhorn limestone, and Graneros shale and is referred to as the "Colorado Group." Locally, the base of the Graneros shale is the base of the upper series and overlies a sequence of lower Cretaceous sandstones and shales. In other areas, there is no boundary between upper and lower Cretaceous that conforms to either top or bottom of a stratigraphic unit; thus, the boundary occurs within a formation. Also, this formation itself may be time-transgressive. This time-transgression of units, the facies changes within units, and the different stratigraphic names applied to formations of the same age in different locations require that the discussion be conducted by region.

27. The lower Cretaceous of Texas occurs in the region to the north and west of the border between the Great Plains and the Atlantic and Gulf Coastal Plain Physiographic Provinces. The materials consist of limestone with subordinate amounts of sandstone and shale. These materials are classified as nonexpansive, mainly on the basis of the amount of limestone. The lower Cretaceous in New Mexico and Colorado occurs in the eastern part of these states and consists of sandstone and shale

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page vi.

(Dakota and Purgatoire formations). The highly argillaceous Mowry shale overlies the sandstone and shale and transgresses the lower Cretaceous and upper Cretaceous boundary. These units are mapped as medium in these states. In Kansas and Nebraska, the Cretaceous is subdivided into three groups: the progressively older Montana, Colorado, and Dakota Groups. The Montana and Colorado Groups are upper Cretaceous, and the Dakota Group is lower Cretaceous at the base and upper Cretaceous at the top. The lower Cretaceous portion of the Dakota Group consists of the Kiowa shale and the Cheyenne sandstone and is classified as medium. The lower Cretaceous in Montana, Wyoming, and the Dakotas is locally highly argillaceous but also contains appreciable sandstone. Highly argillaceous units are the Mowry and Newcastle formations that contain commercial grade bentonite. The classification is considered to be medium.

28. Upper Cretaceous units in this province occur in New Mexico, Colorado, Nebraska, Kansas, Wyoming, Montana, and the Dakotas. The limestone-shale sequence of the Colorado Group has generally been mapped as medium in New Mexico, Colorado, Kansas, and Nebraska on the basis of the presence of relatively large proportions of limestone or marl. However, the time-equivalents to the Colorado Group in Wyoming and Montana contain appreciably more clay and less limestone and, in these last two states, are classified as highly expansive. The highly argillaceous, time-equivalent to the Colorado Group in Wyoming and Montana is the Cody formation. The rocks are classed as highly expansive and consist predominantly of shale with subordinate strata of sandstone and siltstone. The shale units are the Pierre formation (Dakotas, Colorado, New Mexico, Kansas, and Nebraska), the Bearpaw shale (Montana), and the Lewis shale (Wyoming). Although classified as highly expansive, these units exhibit facies variations that include the presence of sandstone or sandy shale, particularly in Montana and Wyoming. Even the highly argillaceous Pierre and Bearpaw shale are locally sandy. Also, the distribution maps include the youngest Cretaceous materials overlying the Montana Group in the highly expansive category. The uppermost Cretaceous units are mainly granular and include the Fox Hills sandstone, the Laramie formation, and the time-equivalents of these formations.

29. The Tertiary materials in the Great Plains Physiographic Province consists of a rather mixed lithologic association of coarse gravel, sand, silty clay, coal, limestone, and bentonite. These deposits are all continental in origin and their source area was the uplifted mountain areas to the west of the province or, in some cases, the uplifted domal areas within the province. Generally, the coarser clastics were deposited by meandering streams flowing off of the newly formed mountains. Depressions lying upon these alluvial areas were the loci of deposition of finer clastics, coal, limestone, and bentonite. Vulcanism along the Pacific Coast, as well as in New Mexico and Wyoming, produced volcanic debris that became incorporated within the sediments and, after diagenesis, altered to bentonite. Continental deposits are somewhat more difficult to summarize than marine deposits. This stems from the heterogeneous nature of the former, whereas marine deposits are more uniform and continuous over larger areas. This great quantity of coarse clastics, not withstanding the frequent occurrence of clay and bentonite in these Tertiary units, was the primary basis for classifying these materials as low.

30. Generally, the Quaternary deposits in this province consist of Pleistocene glacial deposits (till, outwash, and stratified ice-contact material) and alluvial material of nonglacial origin and alluvial and eolian deposits of Holocene age. The extent to which these materials may exhibit expansive properties is dependent upon the incorporation of the older, Tertiary or Cretaceous, materials within them and the amount of volcanic debris that they contain. Glaciers overriding areas of expansive materials are likely to deposit tills that are also composed of expansive clays derived from the underlying rocks. Likewise, Pleistocene or Holocene age streams draining areas of expansive materials may redeposit these expansive clays. Depressions in alluvial valleys may be depositional sites of volcanic debris that may alter to montmorillonite. The glacial till in the Dakotas and Montana contains expansive clay derived from the Pierre shale and from other formations. However, the till is not nearly as expansive as the older source rocks. The parts of Montana and the Dakotas overlain by till are classified on the basis of the nature and composition of the Tertiary or Cretaceous bedrock.

The basis for this classification was the variable thickness of the till in these areas as well as the relatively expansive character of the till itself. Pleistocene lacustrine deposits in the Dakotas and Minnesota may exhibit some degree of expansive properties and are classified as low.

31. Central and Eastern Lowlands Province (11). This physiographic province comprises a large area of the northern United States between New York State and the Dakotas and consists of a relatively large variety of rock types, environments, and geologic structures. Perhaps the most common characteristics throughout the region are the lack of structural deformation, the preponderence of Paleozoic rocks, and the extensive cover of glacial drift overlying the bedrock in most of the area. Aside from areas of expansive glacial materials, the surface exposures of pre-glacial expansive material are limited to areas where the glacial drift is thin or areas near the southern limit of glaciation. The oldest rocks that may exhibit some degree of expansion are shales and shaley carbonates of upper Ordovician age. These rocks occur in southern Indiana and Ohio and in northern Kentucky and consist of alternating layers of shale and shaley limestones. These materials are classified as low and are discussed in the section on the Interior Low Plateau Province. The Maquoketa shale of upper Ordovician age, occurring in northeast Missouri and mentioned in the section on the Ozark and Ouachita Province, is also classified as low. The Maquoketa shale in northeastern Missouri is partially covered with glacial drift. The upper Ordovician shales in southern Indiana and Ohio may also be so covered. The influence of the glacial material is thin to nonexistent near the glacial boundaries, depending upon the age of the glacial material, the amount of erosion that has occurred, and the topography in the area.* In any event, the upper Ordovician and possibly the Silurian, Mississippian, and Pennsylvanian outcrop areas may contain some shales that might exhibit a degree of expansiveness; however, they would be classified as low. The Pleistocene glacial material in this province is

* Note that the classification of materials north of the glacial boundary reflects the bedrock material under the glacial cover in Montana and North Dakota (Great Plains Province).

considered to be nonexpansive even though some till deposits are quite clayey. A significant exception to this observation is to be found in the Dakotas where the till is highly argillaceous due to the incorporation of the overridden Pierre shale. The distribution map shows the area north of the glacial boundary to be high, principally on the basis of the underlying bedrock but also on account of the clayey till. Areas that are underlain by fine-grained sediments deposited in former Pleistocene lacustrine and marine deposit may, under some circumstances, exhibit expansive properties but would be classed as low. This material occurs in North and South Dakota, Minnesota, along the southeastern shores of Lakes Michigan and Huron, and the southern and southwestern shores of Lake Erie. Similar materials are also found in New York State.[11]

32. Laurentian Upland Province (12). This physiographic province includes portions of Minnesota, Wisconsin, Michigan, and New York and is a terrane of Precambrian igneous and metamorphic rocks that are not considered expansive.

33. Ozark and Ouachita Province (13). The Ozark and Ouachita Physiographic Province comprises portions of southern Missouri, northwestern Arkansas, and eastern Oklahoma. The Ozark region consists of relatively undeformed strata dipping away from the Ozark Dome centered in the St. Francois Mountains of southeastern Missouri. The rocks comprise a sequence of lower Paleozoic carbonates (Cambrian through Devonian) followed by a more clastic sequence of Mississippian and Pennsylvanian rocks. The Ouachita region of Arkansas and Oklahoma is a terrane of highly folded and faulted, generally clastic rocks ranging in age from Ordovician to Pennsylvanian. Generally the province should not be expected to exhibit extensive expansive materials; however, some goelogic units pose problems. The Maquoketa shale of upper Ordovician age, occurring on the northeast and southeast flank of the Ozark Dome as well as farther north in the Central and Eastern Lowlands Province, is a source of potential and actual expansion and is classified as low. The lower Pennsylvanian, Atokan, and Morrowan series and the upper Mississippian Chesterian series contain considerable shale and may exhibit some degree

of expansive properties. The stratigraphic names of these shales are: Atoka formation (contains considerable sand), Morrowan, Bloyd shale, and Chesterian and Fayetteville shale. The Atoka formation occurs extensively on the north flank of the Ouachita Mountains and on the south flank of the Ozark Dome. This material is locally mapped as low. The Bloyd and Fayetteville shales are also mapped as low. These occur in an east-west band across northern Arkansas.[11]

34. Interior Low Plateaus Province (14). The Interior Low Plateaus Physiographic Province comprises most of Kentucky; the unglaciated, southern parts of Indiana and Ohio; and that part of Tennessee between the Coastal Plain and the Appalachian Plateau Province. The terrane consists of an essentially nondeformed, gently dipping sequence of predominant carbonates, sandstones, shales, and coal. Ages range from Cambrain to Pennsylvanian. This sequence of rocks is not believed to be appreciably expansive; however, the sequence does contain certain shales and shaley carbonates. The upper Ordovician series consists of interbedded shaley limestone and shale and occurs in north central Kentucky, southeast Indiana, and southwest Ohio. The shales may contain some mixed-layer illite-montmorillonite and are classified as low. Silurian shales such as the Crab Orchard and Waldron formations may also pose limited problems. Mississippian shales belonging to the Osagean and Kinderhookian are locally highly argillaceous and are mapped as low in their outcrop area around the Nashville Dome in central Tennessee. Included in this category is the outcrop area of shales belonging to the Mississippian Chester series in northwestern Kentucky and adjacent parts of Indiana.

35. Appalachian Plateau Province (15). This physiographic province comprises portions of eastern Kentucky and Tennessee, eastern Ohio, most of West Virginia, western and northern Pennsylvania, and southern New York State. The region lies directly northeast of the Newer Appalachian Province and is characterized by a sequence of relatively undeformed sandstones, shales, and carbonates ranging in age from Devonian to Permian. Pennsylvanian and Permian rocks, however, are most abundant. Although shales occur throughout this sequence, they are somewhat more

common and continuous in the Permian Dunkard series located mainly in West Virginia and adjacent portions of Ohio and Pennsylvania. This series is classified as low and is shown on the distribution map. The Pennsylvanian shales, though not shown, would also be low.

36. Newer Appalachians (Ridge and Valley) Province (16). The rocks in this physiographic province are predominantly carbonates and sandstone with some shale and range in age from Cambrian to Devonian. These older rocks, which have been subjected to considerable folding and which do not consist of appreciable shale, are classified as nonexpansive.

37. Older Appalachians Province (17). The Older Appalachians Physiographic Province, lying to the southeast of the Newer Appalachians and northwest of the Coastal Plain, consists of a heterogeneous assortment of igneous, metasedimentary, and metamorphic rocks. The province is divided into two subprovinces, the Blue Ridge to the northwest consisting of Precambrian igneous rocks, and the Piedmont to the southwest, predominantly a Paleozoic, metamorphic, and igneous terrane. The Blue Ridge Subprovince is classified as nonexpansive because of the absence of argillaceous rocks and the thinness of pedologic soils. The Piedmont Subprovince consists locally of thick residual soils developed upon many different kinds of igneous and metamorphic rocks. Generally these soils would not be troublesome with respect to expansion, with the exceptions, of the soils developed upon slates, phyllites, and some schists and soils developed upon basic igneous rocks. These metamorphic areas may pose problems due to the fine-grained nature of these soils, and the basic igneous areas may have soils that contain some montmorillonite. These situations are believed to be quite local in extent; thus, the areas of older rocks are classified as nonexpansive.

38. Triassic Lowlands Province (18). This physiographic province consists of isolated, elongated areas of Triassic age rocks lying in fault-bounded basins generally within the Older Appalachians Province. The rocks consist predominantly of sandstones with shales and basic igneous rocks. These areas are considered nonexpansive.

39. New England Maritime Province (19). The New England Maritime Province, consisting primarily of igneous and metasedimentary terrane,

is not considered to be an area of significantly expansive materials and is therefore mainly mapped as nonexpansive. However, areas of southern Maine are underlain by Pleistocene marine terraces composed of silt and clay. These fine-grained deposits may exhibit some degree of expansiveness and are classified as low on the distribution map.[11]

40. Atlantic and Gulf Coastal Plain Province (20). This physiographic province is underlain by a sequence of sedimentary rocks and nonlithified sediments of varying composition, ranging in age from Cretaceous to Holocene. For the most part, these materials are nonlithified of these componenets. Lithified sandstones and limestones also occur but occupy less area than the nonlithified sediments. These sediments and rocks that have been subjected to only minor folding and faulting and generally exhibit low dip angles and minimal diagenetic effects and may highly weathered.

41. The oldest expansive material on the coastal plains is Lower Cretaceous in age and extends noncontinuously from Texas to Washington, D. C. Materials of this age are generally of mixed lithologies but contain potentially expansive clay, shale, and marl.

42. The expansive surficial soils of Texas have been described by Carter[15] on the basis of montmorillonite content. The stratigraphic and lithologic approach used in the present report also recognizes the expansive properties of the Washita and underlying Fredericksburg Groups. Both groups are predominantly marl and limestone with interbedded clay or shale. Both groups are assigned to the low category in Texas due to the predominant carbonate component in the stratigraphic section. The Trinity Group that underlies the Fredericksburg is not particularly expansive. The boundary between the Atlantic and Gulf Coast Plain Province and the Great Plains in Texas runs approximately along the contact between the Lower and Upper Cretaceous rocks. However, the approximate position of the boundary has resulted in the presence of Lower Cretaceous rocks in both provinces. In Oklahoma, the Washita Group is highly clayey and is classified as highly expansive. This unit is more clay-rich relative to the Washita in Texas, although it too contains considerable limestone or marl. The underlying Fredericksburg consists of clay, sand,

and limestone and is classified as low.[11] The Lower Cretaceous Fredericksburg and Trinity Groups are exposed in southwestern Arkansas. Here these materials are mainly sands with some marl and area assigned to the low category. The more expansive Washita Group is not extensively exposed in Arkansas.[11] Lower Cretaceous materials occur in a narrow band between Fredericksburg, Virginia, through the Washington, D. C., and Baltimore areas to a point northeast of Trenton, New Jersey. The sequence consists of silty clays, clays, and sands of the Patasco, Arundel, and Patuxent formations, respectively. These units are classified as low in swell potential due to the generally coarse-grained nature of the materials. However, the argillaceous character of the Arundel formation has resulted in the classification of a small area including northeast Washington as highly expansive.

43. The rocks and nonlithified sediments of Upper Cretaceous age occurring on the Gulf and Atlantic Coastal Plain are somewhat similar to the underlying Lower Cretaceous materials. The Upper Cretaceous is represented by clays, marls, sands, and units of mixed lithologies. However, in Texas and Mississippi, for example, this sequence of rocks and sediments contains highly expansive argillaceous marls that have contributed significantly to poor subgrade performance in these states. The oldest Upper Cretaceous rocks on the Texas coastal plain belong to the Woodbine and Eagle Ford Groups. These two units have mixed lithologies and consist of sand, clay, marl, and limestone. Although locally highly expansive, the generally low clay content of these units leads to classification as low in expansive character. The successively younger Austin, Taylor, and Navarro Groups are generally highly argillaceous and are mapped as highly expansive. These units also contain appreciable carbonate, especially the Austin Group, and, therefore, may be classed as marl or chalk. The Upper Cretaceous unit exposed in Oklahoma is the Woodbine Group. This unit is sandy and is classified as low. In Arkansas, the exposed Upper Cretaceous is considered to be of low expansive character with the exception of the Taylor and Navarro Groups, which are clayey and presumed to be highly expansive.[11] The Upper Cretaceous units in Tennessee and Kentucky are predominantly granular materials

with minor clay and marl and are thus classified as low. The Upper Cretaceous units in Mississippi, Alabama, and Georgia are compositionally diverse and contain various proportions of sands, clays, and marls. Generally, they lack extensive concentrations of clay and are therefore classified as low. A notable exception is the Selma Group that, although carbonate rich, contains significant amounts of clay and is classified as medium.[11] Bentonites occur locally in the Eutaw formation but are not mapped separately. The Upper Cretaceous units in North and South Carolina are mainly sands with small amounts of fine-grain sediments admixed and/or occurring as infrequent members. This outcrop area is categorized as low.[11]

44. The Tertiary units in the Gulf and Atlantic Coastal Plain Province consist of a diverse assemblage of lithologies representing varying degrees of expansiveness. The Tertiary is characterized by considerable facies changes that may be gradual over long distances or quite abrupt over short distances. These facies changes make the summary statements on the degree of expansiveness of a particular unit difficult. Thus, some units may be mapped as highly expansive in one area and medium or low in other areas. Tertiary units in Texas are classified no lower than low. The categorization in this state is based primarily upon the work of Belcher[11] and Carter.[15] The Paleocene and Eocene are represented by sandy and clayey units and are not considered to be as expansive as the underlying Cretaceous or the younger Tertiary units. The Paleocene and Eocene are mapped as low, medium, or high. The variability is no doubt due to the variable clay content and gradational nature of these units. Oligocene, Miocene, and Pliocene units in Texas are highly argillaceous and have generally been mapped as highly expansive. These units are also gradational and locally may contain appreciable sand, thereby reducing the potential for expansion. The Tertiary of Louisiana mainly consists of sands and clays with minor amounts of marl and units containing admixtures of these components. Generally the Tertiary is classified as low; however, on the basis of Belcher's identification of highly clayey surface materials, the highly

expansive category was assigned to some areas.[11] Potentially highly expansive Tertiary units in Louisiana include: Porter's Creek clay (Paleocene), Logansport (Paleocene), Yazoo (Eocene), and clay members in the Fleming and Catahoula formations. The Tertiary in Arkansas is somewhat similar to that in Louisiana and has been classified as low. The Porter's Creek clay in southwest Arkansas has, however, been rated as highly expansive. Tertiary units in Mississippi consist of sand, clay, marl, limestone, sandstone, and admixtures of these components. Generally, the argillaceous nature of many of these units requires that most of these units be classified no lower than low. Certain units, however, contain appreciably more clay, some of which is montmorillonitic and thus must be classified higher. The Porter's Creek clay, a bentonite in the Midway Group (Paleocene), is locally high in montmorillonite and is classified as medium. Some areas underlain by Wilcox Group (Paleocene and Eocene) and Claiborne Group (Eocene) units also consist of clays and are also classified as medium. The most troublesome Tertiary unit in Mississippi is the Yazoo Clay of the Jackson Group (Eocene) which is classified as high.[11] Problems with expansive clays have not posed extensive problems in Alabama and Georgia. Generally, the Tertiary strata in this area are predominantly sandy with minor amounts of marl and clay and therefore are classified as low. Kaolinite may be more common than montmorillonite. The prevalence of limestone and marl throughout Florida leads to the classification of most of this area as nonexpansive. Argillaceous materials do occur locally and require a higher classification. The Ocala formation (Eocene) is classified as low and the Tampa Group (Miocene) as medium. These units may weather to moderately clayey soils.[11] The Tertiary units in Delaware, Maryland, Virginia, and the Carolinas are predominantly granular and as such are classified as low. Marls and clays do occur but are subordinate in amount.

45. The Quaternary units in this physiographic province are compositionally diverse and consist of granular material on the Atlantic coast (generally nonexpansive); sands, clays, marls, etc., in Florida

(also nonexpansive); fine-grained terrace deposits in Texas and Louisiana (highly expansive); and compositionally mixed, although mainly fine-grained, alluvial deposits along the Mississippi Valley and in the delta region of Louisiana. In Texas, the Pleistocene consists of terraces along the Gulf coast of fine-grained materials with appreciable clay. These materials are mapped at the successively younger Bently, Montgomery, and Beaumont terraces. The quantity of clay present requires that these outcrop areas be classified as highly expansive.[11] Pleistocene terraces (Prairie) in southwestern and northeastern Louisiana are fine-grained and contain montmorillonite. These materials are similar to previously discussed materials in Texas and are classified as highly expansive, although some granular material is also present. The fine-grained materials in the Louisiana delta and in the Mississippi alluvial valley are locally quite clayey and thus are classified as low on the basis of montmorillonite content.[11] The fine-grained materials occurring in swamps and bogs in southern Florida are classified as medium.[11]

Recommended usage

46. The goal of the information presented in the previous paragraphs is to provide the user with a qualitative concept for identifying problem areas involving potentially expansive soils. A secondary but significant goal involves planning of field exploration and sampling programs. As with any discussion of this nature in which subjective information is presented, caution should be used when analyzing the results to avoid misuse of the indicated data. To minimize the hazard of misuse of the information, a recommended decision process is provided and discussed in detail in the following paragraph. It should be kept in mind that although the scale of the included maps is relatively small, it does not preclude their use for short distances; however, the maps are best suited to regional planning needs within a particular state or corridor planning through several states. The following description provides a procedure for narrowing the information to specific locations.

47. Once a tentative route and a basic design have been selected the occurrence and distribution maps are consulted to identify the categories of potential volume change that will be traversed by the

route. This provides a first approximation of the potential problem with expansive soils. The "narrowing" process for potentially expansive soil areas begins with the definition of the physiographic province and the predominant potentially expansive geologic unit within the province. Narrative descriptions for all of the physiographic provinces and, in some cases, predominant geologic units are included to provide an insight into the geologic environment. The process continues by locating and defining the distribution of the predominant geologic units using published U. S. and State geologic maps complemented with information from U. S. Department of Agriculture (USDA) soil surveys. With the areal limits of the potentially expansive materials reasonably defined, experience with existing highways and structures within the area should be reviewed to verify the indicated degree of expansivity. The planning of the field exploration and sampling program for the selected route is the next and final step dealing with this phase of the overall decision process. The next step involves actual identification and classification of the expansive materials based on samples obtained in the exploration program. In geologic units characterized as low or nonexpansive, deviations from routine sampling programs will not be necessary to adequately define the properties of the subgrade materials. For the moderate and high categories, additional exploration should be conducted to define in detail the extent and variability of the potentially expansive materials.

Alternative Sources of Information

48. The occurrence and distribution maps provide the basis for identifying potentially expansive soil areas and were developed in response to a specific research task requirements. However, other sources, similar in nature, are available to provide additional information for problem area identification. These other sources include:

- a. Map for assessment of expansive soils within the United States.[16,17]
- b. USDA Soil Conservation Service (SCS) County Soil Surveys.[18-29]

c. U. S. and State Geological Survey Natural Hazard Maps.

49. The expansive soil assessment map developed by Krohn and Slosson[16,17] is the same scale as the occurrence and distribution maps (1:5,000,000); however, the basis for the map categories is similar to that used in the SCS county surveys. The map uses three categories with regard to severity of the expansive soil problem corresponding to the shrink-swell guidelines set forth by the SCS.[18] The SCS guidelines use the Coefficient of Linear Extensibility (COLE) for categorization of the shrink-swell behavior of potentially expansive soils. The map categories used by Krohn and Slosson[16] use the following criteria:

a. High. Generally includes soils, high in clay, that are made up of a large percentage of montmorillonitic minerals. These soils have a COLE value usually greater than 6 percent.

b. Moderate. Generally includes soils containing moderate amounts of clay that also contain some montmorillonitic minerals. COLE values for these soils vary between 3 and 6 percent.

c. Low. Generally includes soil containing some clay; however, the clay consists mainly of kaolinite and/or other low shrink/swell clay minerals. These soils have COLE values generally lower than 3 percent.

The map provides a very useful source of complementary information for the occurrence and distribution maps.

50. The SCS County Soil Surveys are the most detailed soil maps available for use by engineers. The major limitation to their use is that not all of the United States are mapped, and many of the existing maps are 10 or more years old. Recent soil surveys (i.e., early 1970's to present) contain information on engineering test data, estimates of soil properties significant in engineering and interpretations of engineering properties of soils for each of the major soil series within the county. Also included are engineering test data and classification of the soil series under the Soil Taxonomy.[19,20] In Soil Taxonomy, the Vertisol order includes all of the expansive soils. Within the Vertisol order, the major suborders are Torrerts, Uderts, Usterts, and

Xererts. The system is based on several formative elements that provide individual meaning to the total descriptive term. In the Vertisol order, the basic formative element is "ert;" therefore, when these three letters appear in a soil descriptive name, that material should be considered potentially expansive.

51. The U. S. Geological Survey (USGS) is currently preparing a group of natural hazard maps for the United States, and one of the natural hazards being mapped is expansive soils. The scale will be comparable to the occurrence and distribution maps and will likewise use geologic data as a basis for map categorization. In some areas, the USGS has prepared quadrangle maps[30,31] that show areas containing swelling soils. These maps, although limited in number, are quite detailed in their presentation and provide an excellent source of information. Some State Geological Surveys have prepared maps similar to the USGS maps; however, these are also limited in number. An excellent example of such a map was prepared by Hart[32] for the Front Range Urban Corridor in Colorado. Other states have been or are currently involved in development of these maps, and it is to the State Transportation Agency's advantage to maintain contact with such agencies as State Geological Surveys or other similar agencies, so they can be aware of maps and publications that can provide additional information for identification of potentially expansive soil areas.

Demonstration of Methods for Recognition of Potential Swell Problem Areas

52. To better understand the usefulness of the occurrence and distribution maps and other sources of information for identifying potential expansive soil areas, a practical demonstration will be conducted and described in the following paragraphs. In previous reports,[6,8,9] a number of sampling sites were described from which samples were taken to provide material for laboratory testing within the various tasks of the research project. For demonstration purposes, consider sampling site 12 located in west-central Kansas approximately 5-1/2 miles east of

Hayes, Kansas. The scenario for the demonstration involves the design and construction of a section of I-70 east of Hayes, Kansas.

Occurrence and distribution maps

53. Exact location of the specific site is not possible on the occurrence and distribution maps because of the scale; however, the general area east of Hayes, Kansas, is located approximately 160 miles east and 80 miles south of the northwest boundary corner of the state (Figure 5). The map category in this area is "Moderate" indicating moderately expansive materials and/or a moderate frequency of occurrence. A geologic map of the area of interest indicates that the Blue Hill shale member of the Caslile Formation, Colorado Group, will be encountered. The geologic map[14] provides approximate areal limits of the material, and the narrative summaries provide some additional information on the geology of the material. With this basic information and an experienced geologist, an adequate field exploration program can be planned to specifically locate the limits of the formation in question and obtain samples to positively identify and classify the potential swell.

Krohn and Slosson hazard map

54. According to the Krohn and Slosson map the area east of Hayes, Kansas exhibits a "high" potential swell, which like the occurrence and distribution map indicates a potential problem.

SCS county survey

55. Since the SCS county survey maps provide much more detail than the previous maps, it is necessary for the purpose of the demonstration to be more specific about the location. Therefore, further consideration will be based on the exact location of sampling site 12. The site is located in Ellis County, Kansas (NW1/4 of SE1/4, section 33, T13S, R17W). The exact site location is shown on Figure 7. The site is located in the Bogue series. Table 2 provides excerpts form the various tables of information presented in the Ellis County Soil Survey[33] for the Bogue series. From these tables of engineering properties, estimates of the properties of the soils in the Bogue series can be made.

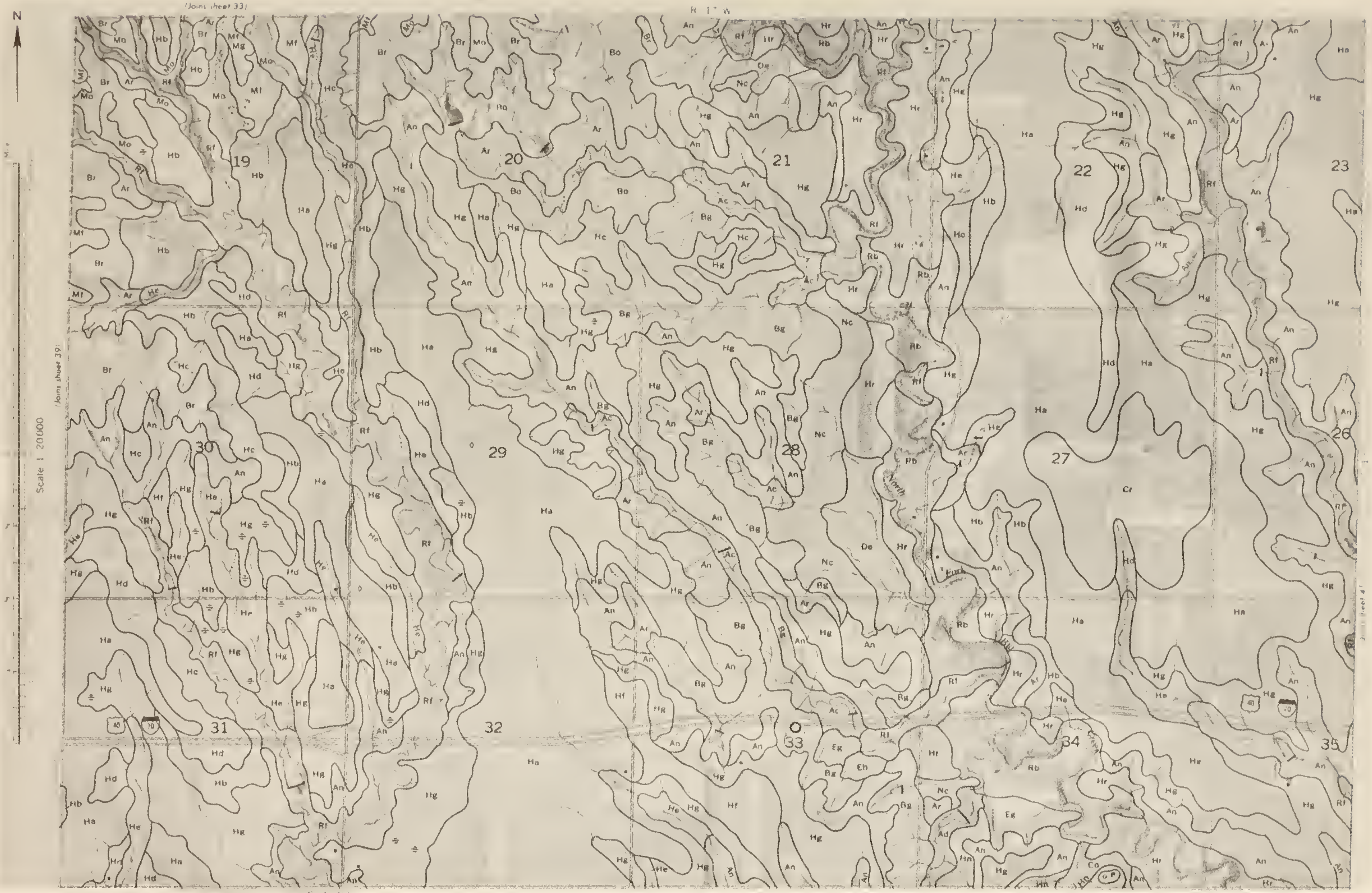


Figure 7. Aerial photograph of sampling site 12, from Ellis County, Kansas soil survey

Table 2

Excerpts from Various Tables of Engineering Properties in the Ellis County, Kansas, Soil Survey (from Reference 33)

TABLE 5.—*Estimates of soil properties significant in engineering*

[An asterisk in the first column indicates that at least one mapping unit in this series is made up of two or more kinds of soil. The soils in such mapping units may have different properties and limitations, and for this reason it is necessary to follow carefully the instructions for referring to other series that appear in the first column of this table. The symbol < means less than; the symbol > means more than]

Soil series and map symbols	Depth to bedrock	Depth from surface	Classification: USDA texture	Classification—Con.: Unified	Classification—Con.: AASHO	Percentage less than 3 inches passing sieve—No. 4 (4.7 mm)	No. 10 (2.0 mm)	No. 40 (0.42 mm)	No. 200 (0.074 mm)	Permeability	Available water capacity	Reaction	Shrink-swell potential
	Feet	*Inches*								*Inches per hour*	*Inches per inch of soil*	*pH*	
*Bogue: Bg, Bo. For Armo part of Bo, see Armo series.	2–3	0–17	Clay	CH	A-7	100	100	95–100	90–100	<0.06	0.11–0.14	6.6–8.4	High.
		17–23	Clay	CH	A-7	100	100	95–100	90–100	<0.06	0.11–0.14	4.5–6.0	High.
		23–32	Weathered clay shale	CH	A-7	100	100	90–100	90–100	<0.06	0.11–0.14	3.5–5.8	High.
		32	Clay shale.										

TABLE 6.—*Interpretations of engineering properties of the soils*

[An asterisk in the first column indicates that at least one mapping unit in this series is made up of two or more kinds of soil. The soils in such mapping units may have different properties and limitations, and for this reason it is necessary to follow carefully the instructions for referring to other series that appear in the first column of this table]

Soil series and map symbols	Suitability as source of—Sanitary landfill cover material	Topsoil	Sand and gravel	Road subgrade [1]	Road fill [1]	Soil features affecting—Highway location [1]	Pond reservoir areas	Soil features affecting—Con.: Embankments, dikes, and levees	Terraces, diversions, and waterways	Irrigation	Degree and kind of limitation for—Septic tank absorption fields	Sewage lagoons	Shallow excavations	Foundations for low buildings	Sanitary landfill [2] (trench and area types)
*Bogue: Bg, Bo. For Armo part of Bo, see Armo series.	Poor: slopes of 3 to 25 percent; clay texture; shale at a depth of 20 to 40 inches.	Poor: very firm if moist; slopes of 3 to 25 percent; clay texture.	Unsuited	Poor: high shrink-swell potential.	Poor: poor shear strength.	Slopes of 3 to 25 percent; shale at a depth of 20 to 40 inches.	Very slow permeability; slopes of 3 to 25 percent; shale at a depth of 20 to 40 inches.	Poor stability; poor shear strength; poor workability; high shrink-swell potential.	Slopes of 3 to 25 percent; shale at a depth of 20 to 40 inches; clay texture; very firm if moist.	Slopes of 3 to 25 percent; shale at a depth of 20 to 40 inches; clay texture; very slow permeability.	Severe: very slow permeability; slopes of 3 to 25 percent; shale at a depth of 20 to 40 inches.	Moderate where slopes are less than 7 percent. Severe where slopes are 7 to 25 percent: shale at a depth of 20 to 40 inches.	Severe: shale at a depth of 20 to 40 inches; clay texture; slopes of 3 to 25 percent.	Severe: shale at a depth of 20 to 40 inches; slopes of 3 to 25 percent; high shrink-swell potential.	Severe: shale at a depth of 20 to 40 inches; clay texture; slopes of 3 to 25 percent.

TABLE 7.—*Engineering test data*

[Tests performed by the State Highway Commission of Kansas under a cooperative agreement with the Bureau of Public Roads (BPR) in accordance with standard procedures of the American Association of State Highway Officials (AASHO)]

Soil name and location	Parent material	SCS sample No.	Depth	Moisture-density [1]: Maximum dry density	Optimum moisture	Mechanical analysis [2]: Percentage less than 3 inches passing sieve—No. 10 (2.0 mm)	No. 40 (0.42 mm)	No. 200 (0.074 mm)	Percentage smaller than—0.05 mm	0.02 mm	0.005 mm	0.002 mm	Liquid limit	Plasticity index	Classification: AASHO	Unified [3]
			In	*Lb per cu ft*	*Pct*								*Pct*			
Bogue clay: 1,550 feet E. and 450 feet S. of NW. corner of sec. 17, T. 12 S., R. 16 W. (modal).	Clay shale.	S-68-Kans-26-1-2	0–6	93	23	100	100	97	96	90	78	66	65	41	A-7-6(20)	CH
		26-1-4	10–17	95	22	100	100	98	97	91	78	63	64	38	A-7-6(20)	CH
		26-1-6	23–32	94	25	100	100	98	97	91	78	63	64	38	A-7-6(20)	CH

[1] Based on AASHO Designation T99-57, Method A (*1*), with the following variations: (1) all material is ovendried at 230° F. and crushed in a laboratory crusher, and (2) no time is allowed for dispersion of moisture after mixing with the soil material.

[2] Mechanical analysis according to AASHO Designation T88-57 (*1*), with the following variations: (1) all material is oven-dried at 230° F. and crushed in a laboratory crusher, (2) the sample is not soaked prior to dispersion, (3) sodium silicate is used as the dispersing agent, and (4) dispersing time, in minutes, is established by dividing the plasticity index value by 2; the maximum time is 15 minutes, and the minimum time is 1 minute. Results by this procedure may differ somewhat from results obtained by soil survey procedure of the Soil Conservation Service (SCS). In the AASHO procedure, the fine material is analyzed by the hydrometer method and the various grain-size fractions are calculated on the basis of all the material, including that coarser than 2 millimeters in diameter. In the SCS soil survey procedure, the fine material is analyzed by the pipette method and the material coarser than 2 millimeters in diameter is excluded from the calculations of grain-size fraction. The mechanical analysis data used in this table are not suitable for naming textural classes for soils.

[3] SCS and BPR have agreed to consider that all soils having plasticity indexes within two points of the A-line are to be given a borderline classification. Examples of borderline classifications obtained by this use are ML-CL and SM-SC.

For example, the Bogue soil is classified as a CH or A-7-6, with 90+ percent passing the No. 200 sieve and 60+ percent smaller than 2 micrometers, an approximate liquid limit and plasticity index of 65 and 40, respectively, and a high shrink-swell potential. Although the properties listed in the tables are for specific samples tested for the county survey, with prudent judgment they can be used to form composite description of the soils that are likely to be encountered. Table 3, reprinted from the Ellis County Soil Survey, shows the classification of the various soil series under the Soil Taxonomy system. As indicated the Bogue series is a Vertisol; therefore, the potential for expansion is high.

56. In summary, it is apparent from the demonstration that the soil at sampling site 12 is potentially expansive and that the field exploration and laboratory testing program should concentrate on determining the areal extent and magnitude of the potentially expansive soil problem. All of the data discussed in the previous paragraphs will not be available in many situations; therefore, it may be necessary to obtain the information needed to make a judgment on the potential swell problem from another reliable source, experience. In locations where existing roads and buildings have suffered damage from expansive soils, a survey of these structures in the vicinity of proposed construction will provide an excellent source of information on the occurrence and relative magnitude of volume change. Even if all of the maps and county surveys are available, a localized survey of structures in close proximity to the proposed construction will provide additional verification of suspected problem areas.

Table 3

Classification of Soil Series
(from Reference 33)

Series	Family	Subgroup	Order
Anselmo	Coarse-loamy, mixed, mesic	Typic Haplustolls	Mollisols.
Armo	Fine-loamy, mixed, mesic	Typic Haplustolls	Mollisols.
Boel	Sandy, mixed, mesic	Aquic Haplustolls	Mollisols.
Bogue	Very fine, montmorillonitic, mesic	Udorthentic Pellusterts	Vertisols.
Brownell	Loamy-skeletal, carbonatic, mesic	Typic Haplustolls	Mollisols.
Campus	Fine-loamy, mixed, mesic	Typic Calciustolls	Mollisols.
Canlon	Loamy, mixed, calcareous, mesic	Lithic Ustorthents	Entisols.
Carlson	Fine, montmorillonitic, mesic	Typic Argiustolls	Mollisols.
Corinth	Fine, mixed, mesic	Typic Ustochrepts	Inceptisols.
Crete	Fine, montmorillonitic, mesic	Pachic Argiustolls	Mollisols.
Detroit	Fine, montmorillonitic, mesic	Pachic Argiustolls	Mollisols.
Eltree	Fine-silty, mixed, mesic	Pachic Haplustolls	Mollisols.
Harney	Fine, montmorillonitic, mesic	Typic Argiustolls	Mollisols.
Heizer	Loamy-skeletal, carbonatic, mesic, shallow	Lithic Haplustolls	Mollisols.
Holdrege	Fine-silty, mixed, mesic	Typic Argiustolls	Mollisols.
Hord	Fine-silty, mixed, mesic	Pachic Haplustolls	Mollisols.
Inavale	Mixed, mesic	Typic Ustipsamments	Entisols.
McCook	Coarse-silty, mixed, mesic	Fluventic Haplustolls	Mollisols.
Mento	Fine, montmorillonitic, mesic	Typic Argiustolls	Mollisols.
Munjor	Coarse-loamy, mixed, calcareous, mesic	Typic Ustifluvents	Entisols.
New Cambria	Fine, montmorillonitic, mesic	Cumulic Haplustolls	Mollisols.
Nibson	Loamy, carbonatic, mesic, shallow	Typic Haplustolls	Mollisols.
Penden	Fine-loamy, mixed, mesic	Typic Calciustolls	Mollisols.
Roxbury	Fine-silty, mixed, mesic	Cumulic Haplustolls	Mollisols.
Wakeen	Fine-silty, carbonatic, mesic	Typic Haplustolls	Mollisols.
Wann	Coarse-loamy, mixed, mesic	Fluvaquentic Haplustolls	Mollisols.

PART III: FIELD EXPLORATION AND SAMPLING OF EXPANSIVE SOILS

57. The proper type and amount of samples and associated field observations provide the engineer with data describing the type of materials to be encountered, the vertical and horizontal boundaries of the different materials, and following laboratory testing an estimate of the strength and deformation properties of the materials required for design purposes. Adequate field exploration programs to achieve these goals cannot be dictated by a rigid set of procedures, although certain basic requirements must be satisfied in each investigation. Both the detail and extent of a field exploration program will vary depending on the nature of the suspected problem, the type of project under construction, and the allowable risk of failure. The purpose of this part of the technical guidance report is not to establish rigid guidelines; instead, recommendations will be made on planning, sampling, and sample storage procedures when expansive soils are encountered along the selected route.

Planning Field Exploration and Sampling Programs

58. The planning of a field exploration and sampling program should be a combined effort between the geotechnical staff (soils engineers and geologists) and the design personnel. Each State Transportation Agency has a basic policy on field exploration, which specifies maximum horizontal distances between borings and the depth of borings; however, when expansive soils are expected, it will be necessary to obtain more detailed information. The coordinated effort should begin with the selected route (including proposed grade line) and the various sources of information previously described in Part II. The location of the borings should be made according to agency policy (i.e. no farther apart than 500 ft in continuous cut sections and no farther apart than 1000 ft under any circumstances). Additional borings should be drilled whenever variations in the geological conditions are noted on published

information or in the field and whenever changes in the highway cross section (i.e., cut, or fill) occur. To provide basic information concerning the material properties at changes in the highway cross section, boring locations as shown in Figure 8 should be considered. Borings should also be located at any discontinuities in the highway cross section, such as culverts, pipelines, and utility crossings.

59. Ideally, for expansive soils the depth of the borings should coincide with the depth of active zone, which is variable depending on the soil type and local climatic conditions. Generally, the depth of active zone will vary between 5 to 10 ft. Confirmation of the depth of active zone is not an easy task and requires some specific studies to bracket the range; therefore, to assure at least a representative measure of the soil properties in a majority of the active zone, borings should extend to a minimum depth of 5 ft below the final grade in all cut section and 5 ft below the foundation surface (natural ground surface minus depth of clearing and grubbing) beneath shallow fills (i.e., height less than approximately 20 ft). Throughout the depth of the borings each change in soil type should be carefully noted and described.

60. The amount and type of samples taken from the borings will depend on the purpose of the field exploration program. For example, for reconnaissance soil surveys, which are conducted to identify and delineate subsurface geological conditions, auger (bucket or continuous flight) borings will provide adequate samples for routine classification tests. For preliminary or special subsurface soil surveys, which are conducted to provide analysis and design data, some type of undisturbed samples should be taken to adequately define the in situ properties of the natural soil. Whether reconnaissance, preliminary, or special soil surveys are being conducted, samples should be taken for each different soil encountered through the depth of the boring. To provide adequate samples to use the soil suction or odometer swell testing procedures, which will be described in a subsequent part of this report, at least one undisturbed sample of each suspected expansive soil should be obtained in addition to the auger boring samples for classification.

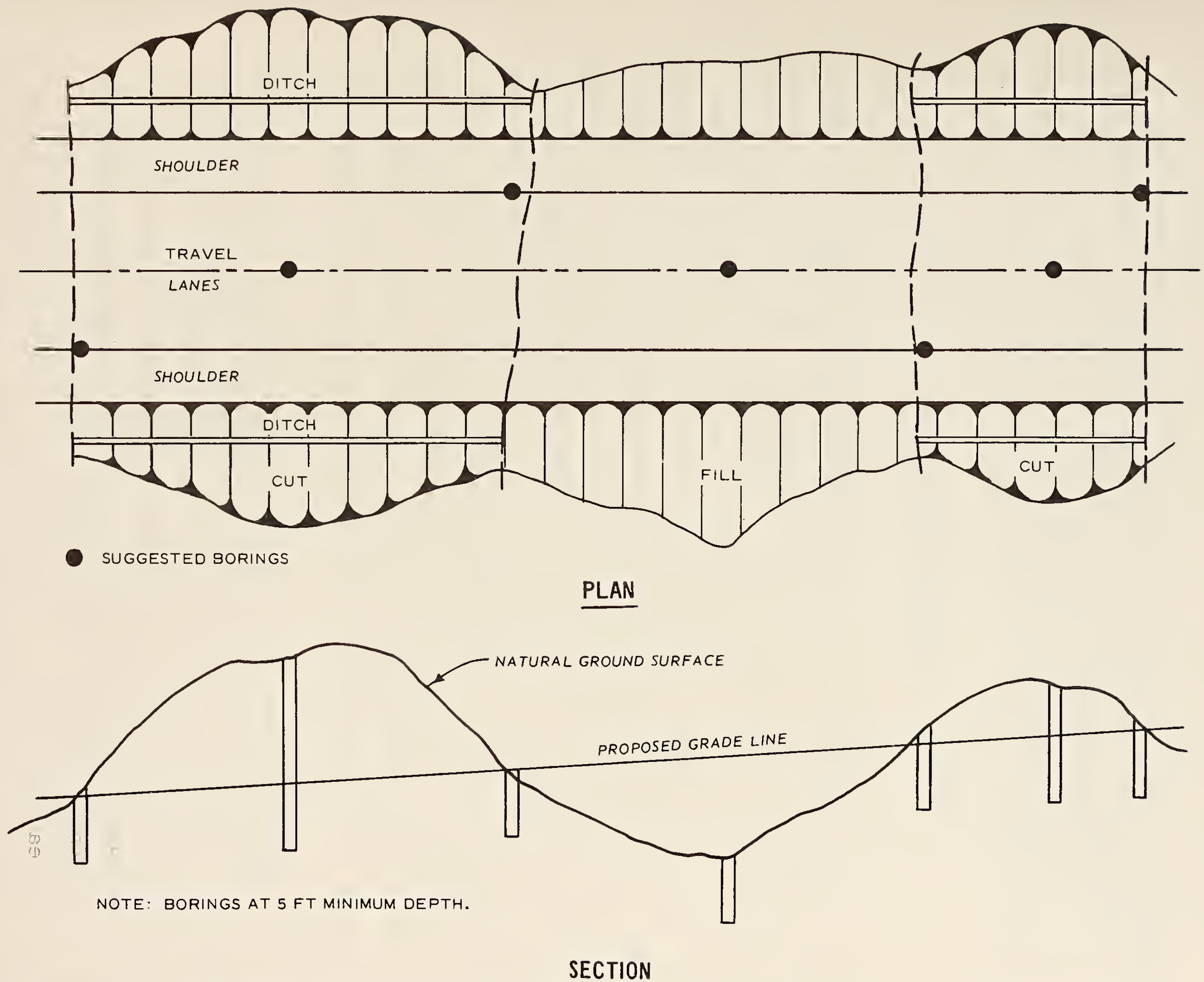


Figure 8. Suggested boring locations for cut and fill sections in expansive soils

61. Other pertinent information that should be noted during the field exploration program which will affect the behavior of expansive soil subgrades includes: the existence of, depth to, and variations of a groundwater table; the general terrain and surface drainage characteristics; and the extent and type of vegetative cover. Detailed information and guidelines on collection and use of subsurface exploration data are available elsewhere[34-37] and should be consulted to provide uniform guidance for field exploration and sampling programs.

Sampling Techniques

62. Proper sampling of expansive soils is particularly important because of the nature of the materials involved. Expansive soils vary from medium to firm materials, such as the Prairie Terrace formations of Louisiana, to very hard rock materials, such as the Pierre or Mancos shales of the Northern and Central Plains areas. Complementary to the varying degrees of firmness are large variations of in situ moisture content, i.e., from less than 5 percent for some of the shale materials to near or above the plastic limit for some of the softer materials. Finally, such structural discontinuities as fissures, slickensides, and bedding planes can make sample recovery a near impossible task. With such a variety of field-related problems and the ever-present requirement for minimal disturbance, it is obvious that a variety of sampling techniques must be available to the engineer to obtain good undisturbed samples.

63. The application of currently available sampling techniques is dependent on the variables discussed in previous paragraphs as well as the type of tests that are planned. For simply delineating the subsurface conditions, classification testing (i.e., specific gravity, grain-size distribution, Atterberg limits) and for physicochemical testing, auger (bucket or continuous flight) borings will provide the necessary type and amount of sample. For compaction tests and test methods for defining effects of soil stabilizers that require larger sample quantities, large borings or pit samples can provide the required amount.

For tests to define the in situ characteristics of expansive soils such as soil suction or swell and swell pressure tests, undisturbed samples will be required.

64. Undisturbed sampling techniques generally used in expansive clays and shales include push-tube and rotary core barrel samplers. Push-tube samplers consist of thin-walled, seamless, stainless steel tubes (2- to 5-in. ID) that are advanced into the soil by hydraulic or falling weight systems. Variations of the push-tube samplers involve the use of pistons within the sampling tube to take advantage of the vacuum created during sampling. The simplest form of push-tube does not have a piston; instead, the driving head is affixed to the sampling tube and has a pressure release valve (ball type) to bleed off the compressed air and to close and form a vacuum on the sample during withdrawal of the sampler. A second variation is the free piston or semifixed piston push-tube sampler in which the piston is held at the lower end of the sampling tube during insertion of the sampler and allowed to rest on the sample during the push. In this way, the vacuum is again used only during the withdrawal of the sampler. The third variation is the fixed piston push-tube sampler in which the piston is connected or fixed to the drill rig during the push and the vacuum assists during the pushing of the sampler as well as during the withdrawal. Push-tube sampler are best suited for medium-stiff or stiff clays that are free of gravels or small rocks which could damage the leading edge of the tube. Rotary core barrel samplers may be categorized as double-barrel or single-barrel. The double-barrel type, such as Denison, Pitcher, or WES samplers, consists of an outer barrel with a cutter shoe to advance the sampler and an inner barrel with a cutter edge to fine-trim and contain the sample. A single-barrel rotary core sampler is simply a core barrel with a cutter shoe, usually with a diamond head, to advance and contain the sample. The double-barrel samplers are best suited for hard soils and soils containing gravel. Single-barrel samplers are best suited to sampling rock.

Sample Preservation and Storage

65. Minimization of sample disturbance during and after drilling is crucial to the usefulness of an undisturbed sample. This is particularly true for expansive soils since minimal changes in the moisture content and/or soil structure will adversely affect the properties measured from the test specimens. Drilling equipment and technique affect disturbance during sampling and can be minimized by proper maintenance of equipment and training of personnel. Disturbance following drilling is influenced by the way the sample is preserved, transported and stored. For auger borings, pit or other disturbed samples, the sampled material should be thoroughly sealed in waterproof containers so that the moisture content can be accurately measured if desired. Each container should be clearly labeled and stored under conditions that minimize large temperature and humidity variations. Undisturbed samples of expansive soils may be stored in the sampling tubes or extruded and preserved then stored. Preference should be given to storing the samples in the sampling tubes, then extruding the samples as needed for testing. This helps to minimize the stress relief a sample might undergo if not adequately confined following sampling. If stored in the sampling tube an expanding packer with a rubber O-ring should be used in both ends of the tube to minimize moisture loss. If the expansive soil sample is to be extruded and stored, then the sample should be removed immediately after sampling and thoroughly sealed to minimize stress relief and moisture loss. Containers for storage may be either cardboard or metal and should be approximately 1 in. greater in diameter and 1.5 to 2.0 in. greater in length than the sample to be encased. Three-ply, wax-coated cardboard tubes are available in various diameters and lengths and may be cut to desired lengths. Wax-coated cardboard tubes with metal bottoms, similar to those commonly used for preparing concrete specimens may be used. Undisturbed samples preserved in cardboard tubes should be completely sealed in wax. A good wax for sealing expansive soils consists of a 1 to 1 mixture of paraffin and microcrystalline wax. This mixture adequately seals the sample and does not become brittle when

cold. The temperature of the wax should be approximately 20°F above the melting point when applied to the expansive soil sample, since wax that is too hot will penetrate pores and cracks in the sample and render it useless. If the wax-coated cardboard tubes with metal bottoms are used, prior to placing the sample in the tube a small amount of wax (depth $\approx$ 0.5 in.) should be placed in the tube and allowed to partly congeal. The sample should then be placed in the tube and completely surrounded and covered with the hot wax, then allowed to cool before moving. The wax and cardboard provide an excellent seal against moisture loss and enough confinement to minimize stress relief and particle reorientation. The sealed samples should be carefully labeled, transported, and stored in an environment with minimal temperature and humidity variations.

PART IV: IDENTIFICATION AND CLASSIFICATION OF POTENTIALLY EXPANSIVE SOILS

66. The purpose of a reconnaissance or equivalent level soil survey is to identify the different types of materials that will be encountered along the selected route and to establish the vertical and horizontal limits of the various materials. It is at this time that an effort must be made to identify and classify potential problem areas. The obvious purpose of identification and classification methodology for expansive soil is to provide the engineer with a semiquantitative estimate of the problems that can be expected, so he can (a) take additional samples to more adequately define the problem and/or (b) assign samples for quantitative testing so that estimates of anticipated volume change can be made. Combining field observations made at the time of sampling with basic classification test data and good judgment, the engineer can semiquantitatively estimate the severity of the potentially expansive soil problem. The following paragraphs provide technical guidance for the use of field observations and laboratory test data to identify and classify potentially expansive soils.

Field Observations

67. Although expansive soils occur in many different geologic, geographic, and climatic zones of the United States there are some relatively consistent and useful visual indicators of their presence. The following criteria may be useful in visually identifying expansive soils:

- a. Exposed surfaces of expansive soils, when dry, exhibit an irregular or pebbly texture resembling popcorn. Also, on dried, exposed surfaces, desiccation cracks will be evident particularly during dry times of the year. The more frequent and deeper the desiccation cracks the greater the potential swell. The dry strength of the exposed surface material is generally very high.

b. When moist, the plasticity of expansive soils is evident by attempting to roll a small piece of the soil into a thread. The easier it is to roll the thread, the higher the plasticity and generally the higher the potential swell.

c. Fissures and slickensides are abundant in freshly exposed surfaces of most expansive soils.

d. When wet, expansive soils have a very slick, cohesive texture and will adhere to shoes or tires of vehicles.

e. Distortions or tell-tale damage to adjacent structures will be evident.

Laboratory Classification Data

68. The evaluation of expedient methodology for identification of potentially expansive soils[7] conducted as a task of the research project indicated that the Atterberg limits (specifically, the liquid limit (LL) and plasticity index (PI)), Bar Linear Shrinkage (BLS) and the natural soil suction (τ_{nat}) were the most consistent indicators of potential swell. Using the results of the evaluation, technical guidelines were developed for identifying and classifying potential swell based on post-reconnaissance soil survey test data. The technical guidelines are discussed in the following paragraph.

69. Following the field exploration and sampling program, routine testing of the samples will provide a majority of the data necessary to identify and qualitatively classify the potential swell of the soils (i.e., LL, PI). The remaining factor, soil suction, should be determined, and then the WES classification of potential swell, as shown in the following tabulation, may be used:

LL, %	PI, %	τ_{nat}, tsf	Potential Swell Classification
>60	>35	>4	High
50-60	25-35	1.5-4	Marginal
<50	<25	<1.5	Low

The above classification system may be used without the soil suction criteria; however, it should be noted that the accuracy and conservatism of the system are reduced when the LL and PI are used without the soil suction criteria. For soils that exhibit a low classification of potential swell, the pavement design may be completed using basic strength parameters with the confidence that expansion problems will be minimal and normal construction procedures will further minimize the limited problems. For soils that exhibit a high classification of potential swell, all locations should be tested (undisturbed samples) and an estimate of the anticipated volume change made. Specific testing and prediction techniques are discussed in detail in Part V. For soils that exhibit a marginal classification of potential swell, additional testing at that location should be judged on the conditions at the specific site. The marginal category is indicative of a moderate to high capacity for volume change, but characteristic properties preclude the development of swell. For example, a marginal classified soil that has a relatively high natural water content (i.e., greater than the PL) and a low natural density will most likely not cause serious problems and therefore not require testing. However, if conditions are such that the properties (water content or density or both) are likely to change during and following construction, then additional testing and an estimate of the volume change corresponding to the new conditions are necessary.

Demonstration of Methodology

70. To continue the previous demonstration initiated in Part II it will be necessary to assume that a reconnaissance soil survey has been conducted at sampling site 12 near Hayes, Kansas, and the results of the laboratory classification and soil suction tests[7] for a sample near the final grade are:

$$\begin{aligned} LL &= 75 \\ PI &= 51 \\ \tau_{nat} &= 1.8 \text{ tsf} \end{aligned}$$

Using the WES classification of potential swell, the materials at sampling site 12 will classify in the high potential swell category based on LL and PI and marginal based on τ_{nat} . For analysis and design purposes, the material will be considered to have a high potential swell, and additional tests will be assigned to characterize the expansive soil and estimate the anticipated volume change.

PART V: TESTING EXPANSIVE SOILS AND PREDICTION OF ANTICIPATED VOLUME CHANGE

71. Once an expansive soil has been identified and classified in the marginal or high categories of potential swell, quantitative characterization of the expansive soil becomes a necessity in order to accurately estimate the magnitude of anticipated volume change. Accurate estimates of volume change are a requisite for the selection of effective treatment alternatives or preparation of adequate designs. Techniques available for quantitative characterization of expansive soils and prediction of anticipated volume change fall into three groups, namely, soil suction tests, odometer swell (or swell pressure) tests, and empirical techniques. One of the research project tasks involved an evaluation of testing and prediction methodology. The results of the evaluation of testing and prediction techniques showed that the soil suction concept and associated testing and prediction procedures provide a better characterization of the behavior of expansive soils and a more reliable estimate of anticipated volume change for selected conditions based on comparisons with measured field behavior. The measurement of soil suction using thermocouple psychrometers is a simple, inexpensive, accurate, and reliable testing procedure, which can be easily implemented. From a practical point of view, the soil suction testing procedure is much less time-consuming than odometer procedures, and the measured data are applicable to a wider range of moisture conditions. The procedure suggested for estimating the depth of the active zone based on the soil suction versus depth profile provides a reasonable estimate of the depth of active zone, which is consistent with reported experience. The depth of active zone is useful in establishing the details of the testing program and providing limits for applying the prediction technique. A major requirement of any prediction technique involves an estimate of the final or equilibrium conditions. The suggested procedures based on assumed final soil suction profile, particularly the saturated moisture content profile, provide a reliable and generally conservative estimate of final conditions. Using these results an

abbreviated step-by-step description of methods for testing expansive soils and predicting anticipated volume change includes:

a. Adequate definition of the soil profile and collection of good undisturbed soil samples. Frequency of sampling will depend on the selected alignment and the variability of the geologic materials along the alignment.

b. Selection of soil specimens from representative soil samples and the collection of soil suction data using thermocouple psychrometers. Following reduction of the data, the soil suction versus water content data and specific volume versus water content data should be plotted and the required parameters defined.

c. Estimation of the depth of active zone and final soil suction profile, then calculation of the anticipated volume change for each layer in the profile and summation to obtain the total surface movement.

In the absence of soil suction testing equipment, the odometer swell test procedure provides reasonable and generally conservative estimates of volume change based on comparisons with measured field behavior. The Overburden Swell (OS) test is consistent with the definition of potential swell and therefore satisfies most of the field simulation requirements; however, it is much more time-consuming. Empirical prediction techniques, such as the Potential Vertical Rise (PVR), provide estimates of anticipated volume change that are actually more accurate than the OS test. In the absence of soil suction or odometer data, the PVR method is quicker and simpler than both test procedures, providing classification data are available. The major reason for the improved accuracy of this technique is that it requires adequate profile definition and considers the influence of overburden on the predicted heave.

72. Each of the categories of testing and prediction methods consist of a testing method and an associated prediction procedure. The remaining portion of this part of the report will be concerned with describing in detail the testing methods and prediction procedures. Each

testing method and prediction procedure will be demonstrated at the end of the major section.

Soil Suction Test and Prediction Procedure

73. To briefly review, soil suction is the force exerted by a soil mass responsible for soil-water retention,[6,8,9] or more simply the pulling force exerted on soil-water by the soil mass. Soil suction can be measured by a number of procedures; however, the use of thermocouple psychrometers is one of the simplest and most reliable procedures available. In simple terms, the psychrometer measures the relative humidity of a soil specimen in a sealed container. Soil suction is directly related to relative humidity; when calibrated for various relative humidities (or soil suctions), the psychrometer output can be easily converted to soil suction in any convenient units.

Test method

74. The soil suction laboratory testing procedure for measuring soil suction with thermocouple psychrometers is the same as that described by Johnson[38-42] and Johnson and Snethen.[43,44] Details of the thermocouple psychrometer testing procedure, including equipment used, calibration procedure, sample preparation and testing procedure, and data reduction and interpretation are described in Appendix A. In general terms, the procedure involves cutting a section of an undisturbed sample into 9 or more 1.5-in. (approximate side dimension) cubes. The soil cubes are placed in individual sample containers (1-pt metal paint cans with interiors coated with wax to prevent corrosion) which act as environmental chambers when sealed. Of the 9 or more specimens, two are tested at their natural water content, and the remainder, depending on their natural water contents, are either dried (at room temperature) for varying lengths of time or wetted by adding distilled water. The purpose of the wetting and drying process is to establish a range of moisture contents over which the soil suction can be measured. The

natural water content and wetted specimens are sealed in the sample containers immediately after cutting and adding water. The dried specimens are sealed after their respective drying time. The sample containers are sealed with rubber stoppers (No. 13-1/2) through which a thermocouple psychrometer had been placed. When sealed the psychrometer extends approximately 1 in. beyond the bottom of the stopper into the sample container, thus allowing it to measure the relative humidity within the sample container. All of the sample containers are then placed in a polystyrene temperature chest, which minimizes temperature variations around the sample containers. The specimens are allowed to equilibrate for approximately 48 hr, after which the soil suction and temperature readings (voltage output) are taken for each specimen using a psychrometric microvoltmeter. To determine the compressibility factor, α, which will be discussed in a subsequent section, the natural moist density of each specimen is determined using the volume displacement method. The water content of the specimens is then determined. The thermocouple voltage output (millivolts) values are converted to temperature (°C) using the following conversion:

$$\text{Temperature, }^{\circ}\text{C} = \frac{\text{Output in millivolts}}{0.0395\ \text{millivolts}/^{\circ}\text{C}} \tag{1}$$

The psychrometer voltage output (microvolts) at the measured temperature is converted to microvolt output at 25°C (calibration temperature) by

$$E_{25} = \frac{E_T}{0.325 + 0.027T} \tag{2}$$

where

T = measured temperature, °C

E_T = voltage output at T°C, microvolts

The psychrometer output is then converted to soil suction in tons per square foot using the calibration line for the specific psychrometer. A typical example of a calibration line for the thermocouple psychrometers used to collect the soil suction data in the research project is

$$\tau^{o} = 2.82\ E_{25} - 4.4 \quad (3)$$

where

τ^{o} = total soil suction at atmospheric pressure, tsf

E_{25} = microvolt output at 25°C

Data reduction and presentation

75. Following reduction of the soil suction data and determination of the corresponding water contents, the data are plotted on a semilog plot with soil suction on the log scale. Generally, three-cycle semilog paper is sufficient to accommodate all the data points. By keeping track of the points representing natural conditions, all of the data points are used to plot a straight line through the points. If some variation occurred at the upper or lower end of the curve, the data points between soil suction of approximately 2 and 20 tsf are used to establish the line. Examples of soil suction versus water content relationship for Hayes, Kansas (sampling site 12), are shown in Figure 9 and may be described using the equation

$$\log \tau_{m}^{o} = A - Bw \quad (4)$$

where

τ_{m}^{o} = matrix soil suction without surcharge pressure (i.e., atmospheric pressure), tsf

A,B = constants (y-intercept and slope of soil suction versus water content curve, respectively)

w = water content, percent

The slope, B , of the line is determined by calculating the inverse of the change in water content over one cycle (i.e., 1 to 10 tsf) of soil suction. The intercent, A , is calculated by applying Equation 4 at soil suction equal to 1 tsf.

76. Contrary to data obtained using equipment and procedures, such as the pressure plate or pressure membrane devices, no hysteresis was indicated in any of the soil suction versus water content relationship.[8,9] Although no experimental verification was made concerning the

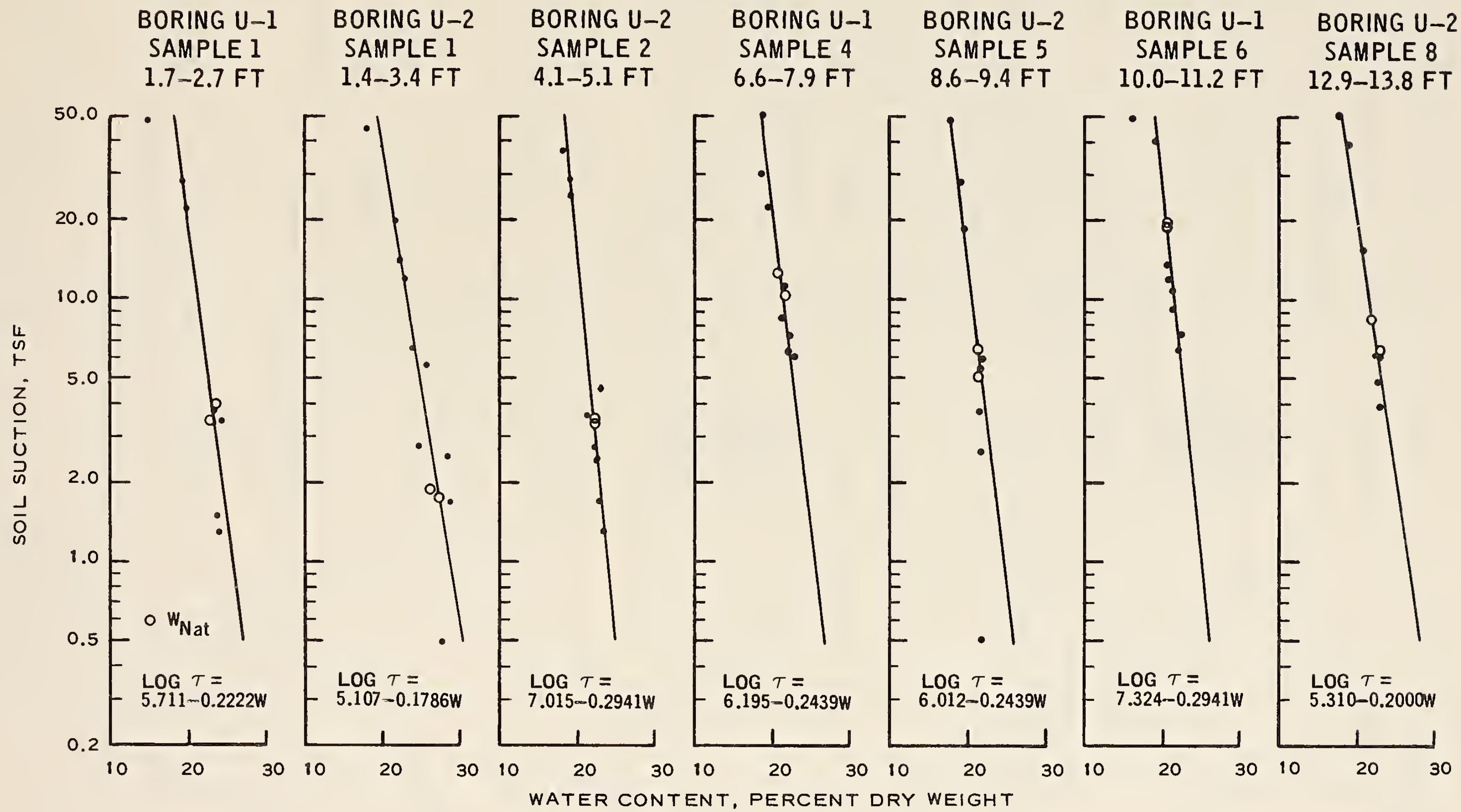


Figure 9. Soil suction versus water content relationships for Blue Hill shale, Hayes, Kansas

existence of hysteresis, it is likely that the test procedure inhibits its development. For example, the fact that the test specimens are dried or wetted from natural condition, rather than completely air-drying, and then wetting or completely saturating, and then drying appears to provide more reliable data. In addition, the procedure is more representative of in situ behavior; that is, moisture content varies from some equilibrium value in response to such factors as moisture infiltration, and evaporation and transpiration.

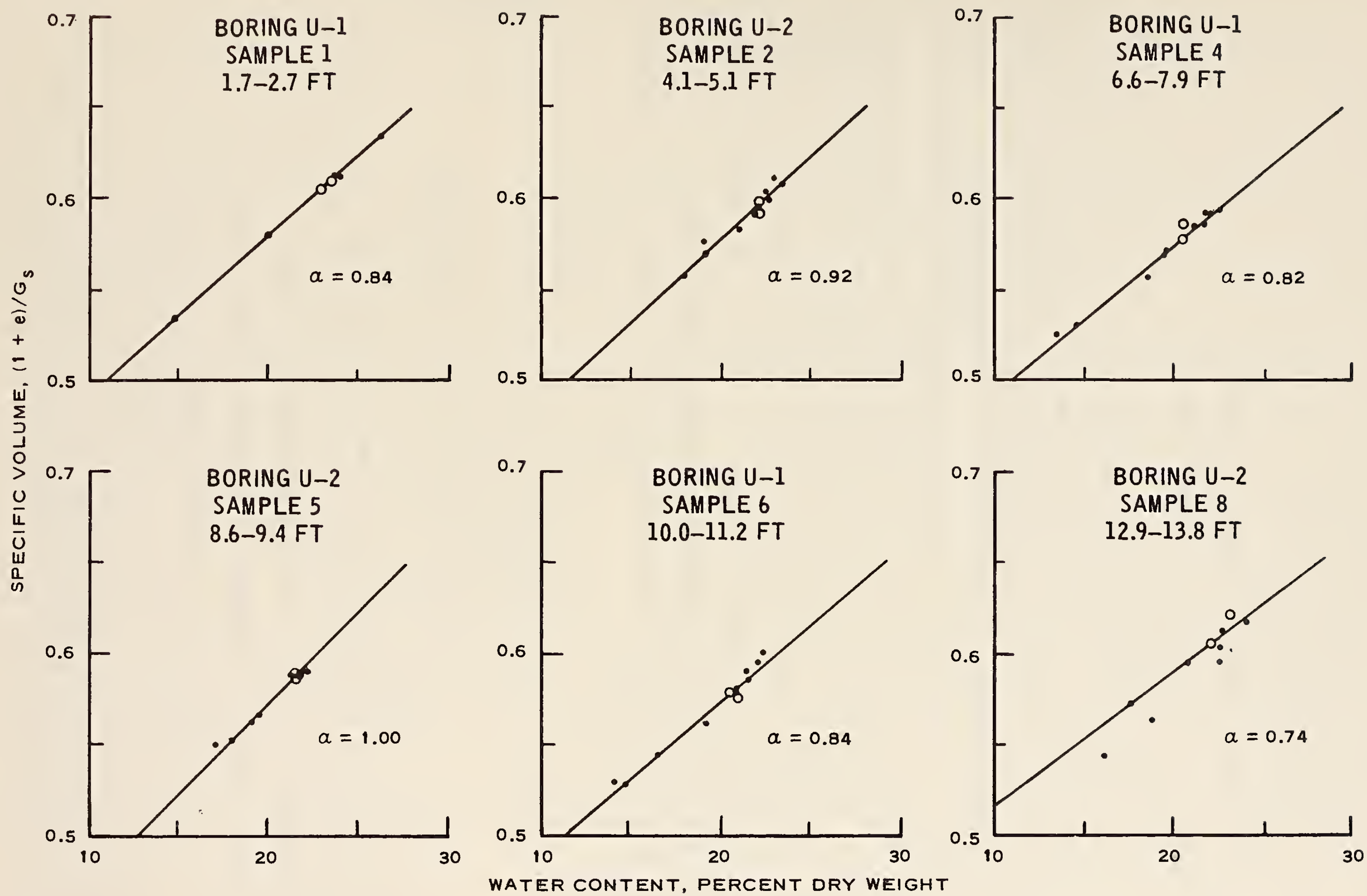
77. As described in the explanation of Equation 4, the soil suction versus water content relationship represents the matrix soil suction of the sample. The osmotic soil suction can be estimated using psychrometer techniques. If the soil suction becomes stable (i.e., does not further decrease) with increasing water content and the soil suction versus water content relationship becomes horizontal, then that value of soil suction represents the osmotic component. However, for the materials and range of moisture contents tested in the research project, the osmotic component was not evident at 19 of the 21 sampling sites and only a slight indication was noted at the remaining 2 sites.[6,8,9] Therefore, for all practical purposes the soil suction versus water content relationship represents the total soil suction.

78. In the equation for prediction of volume change using soil suction data, which will be discussed in a subsequent section, a volumetric compressibility factor, α , is included that relates the change in volume to the corresponding change in water content. The value of α is determined by calculating the slope of the specific volume (inverse of dry density) versus water content relationship. Examples of the specific volume versus water content relationships for Hayes, Kansas (sampling site 12), are shown in Figure 10.

Details of heave prediction

79. Using the soil suction data to characterize an expansive soil as previously described, the volume change of an expansive clay stratum may be estimated using the following equation:

$$\frac{\Delta H}{H} = \frac{C_\tau}{1 + e_o} \left[(A - Bw_o) - \log (\tau_{mf} + \alpha\sigma_f)\right] \qquad (5)$$



NOTE: DATA NOT AVAILABLE FOR BORING U-2, SAMPLE 1, 1.4–3.4 FT.

Figure 10. Specific volume versus water content relationships for Blue Hill shale, Hayes, Kansas

where

H = stratum thickness, ft

C_τ = suction index, $\alpha G_s/100B$

e_o = initial void ratio

w_o = initial moisture content, percent

τ_{mf} = final matrix soil suction, tsf

α = compressibility factor

σ_f = final applied pressure (overburden plus external load), tsf

The suction index, C_τ, reflects the rate of change of void ratio with respect to soil suction and can be calculated as shown above. The laboratory data necessary to apply Equation 5 include G_s, e_o, A, B, w_o, and α, all of which (except G_s) can be easily determined in the soil suction test procedure. The remaining two variables, τ_{mf} and σ_f, are functions of the assumed depth of active zone and the assumed final soil suction profile, both of which will be discussed in subsequent paragraphs. The compressibility factor for CH clays is commonly set equal to one, because the voids of these soils are filled with water within a wide range of moisture contents (quasi-saturated). In the absence of measured data, the compressibility factor may be roughly estimated from the PI by[45]

$$PI < 5 \quad \alpha = 0 \tag{6a}$$

$$PI > 40 \quad \alpha = 1 \tag{6b}$$

$$5 < PI < 40 \quad \alpha = 0.0275\ PI - 0.125 \tag{6c}$$

80. The equations described above provide predictions of in situ volume change of a soil stratum with respect to field conditions of soil composition, structure, initial and equilibrium moisture profiles, and confining pressures. Vertical rise at the ground surface may be estimated by summing the volume change of each stratum in the soil profile.

81. The depth of active zone or depth of desiccation has been defined as the thickness of the layer of soil in which a moisture deficiency exists. The depth of active zone is a transient value influenced

by the soil type, soil structure, topography, and climate. The depth of seasonal moisture variation may be equivalent to the depth of active zone if the material responds to changes in the climate relatively fast. However, most materials do not respond rapidly enough, so the depth of active zone is generally greater than the depth of seasonal moisture variation. As far as climate is concerned, the depth of active zone generally reflects the past arid extremes of the climatic history. No universally applicable rules exist for establishing the depth of active zone. For the research project, the soil suction versus depth profiles for the 21 sampling sites were used to estimate the value.[8,9] Examination of the soil suction versus depth profiles for the sampling sites revealed that no two sites were identical; however, some general trends were prevalent. The existence of these general trends allowed for the establishment of some "rules of thumb" that when tempered with sound engineering judgement, provide a reasonable estimate of the depth of active zone. The "rules of thumb" to use are:

<u>a</u>. For soil suction versus depth profiles that exhibited a relatively constant value with depth at the lower levels, the depth of active zone was set at the upper end (depth) of the constant range.

<u>Example</u>: Sampling site 1, Jackson, Mississippi, Figure 11: the soil suction became constant below a depth of 7.7 ft, and the depth of active zone was set at 8.0 ft.

<u>Example</u>: Sampling site 4, Lake Charles, Louisiana, Figure 12: the soil suction became constant below a depth of 5.0 ft and the depth of active zone was set at 6.0 ft.

<u>b</u>. For soil suction versus depth profiles that exhibited S- or Z- shaped curves with depth, the depth of active zone was set below the first major change in magnitude of the soil suction, i.e., high to low or low to high.

<u>Example</u>: Sampling site 12, Hayes, Kansas, Figure 13: the soil suction increased to 11.2 tsf at 6.8 ft then decreased to 6.2 tsf at 8.7 ft, which constitutes the major change in magnitude described above, and the depth of active zone was

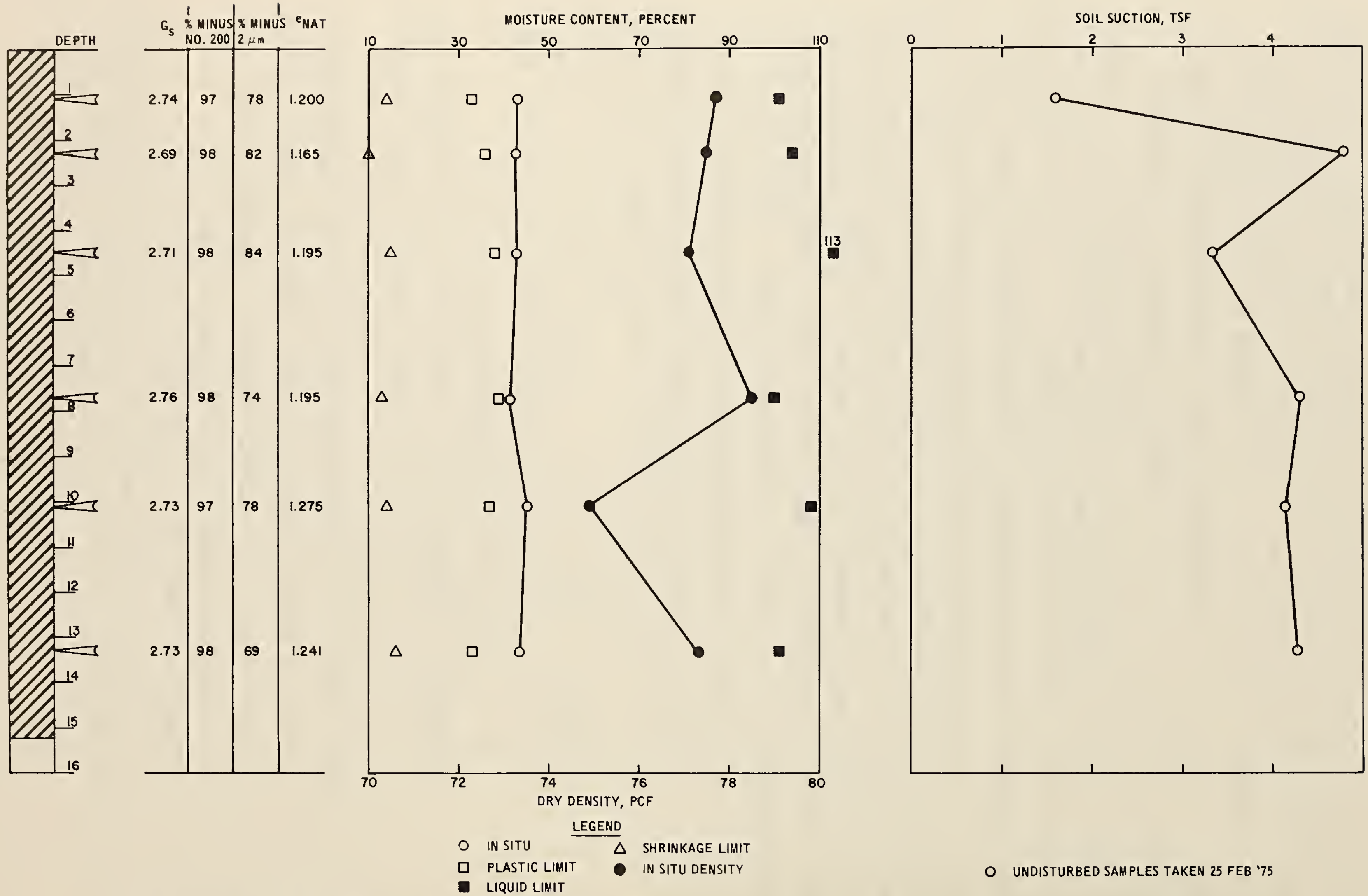


Figure 11. Boring log and moisture content, dry density, and soil suction profiles for sampling site 1, Jackson, Mississippi

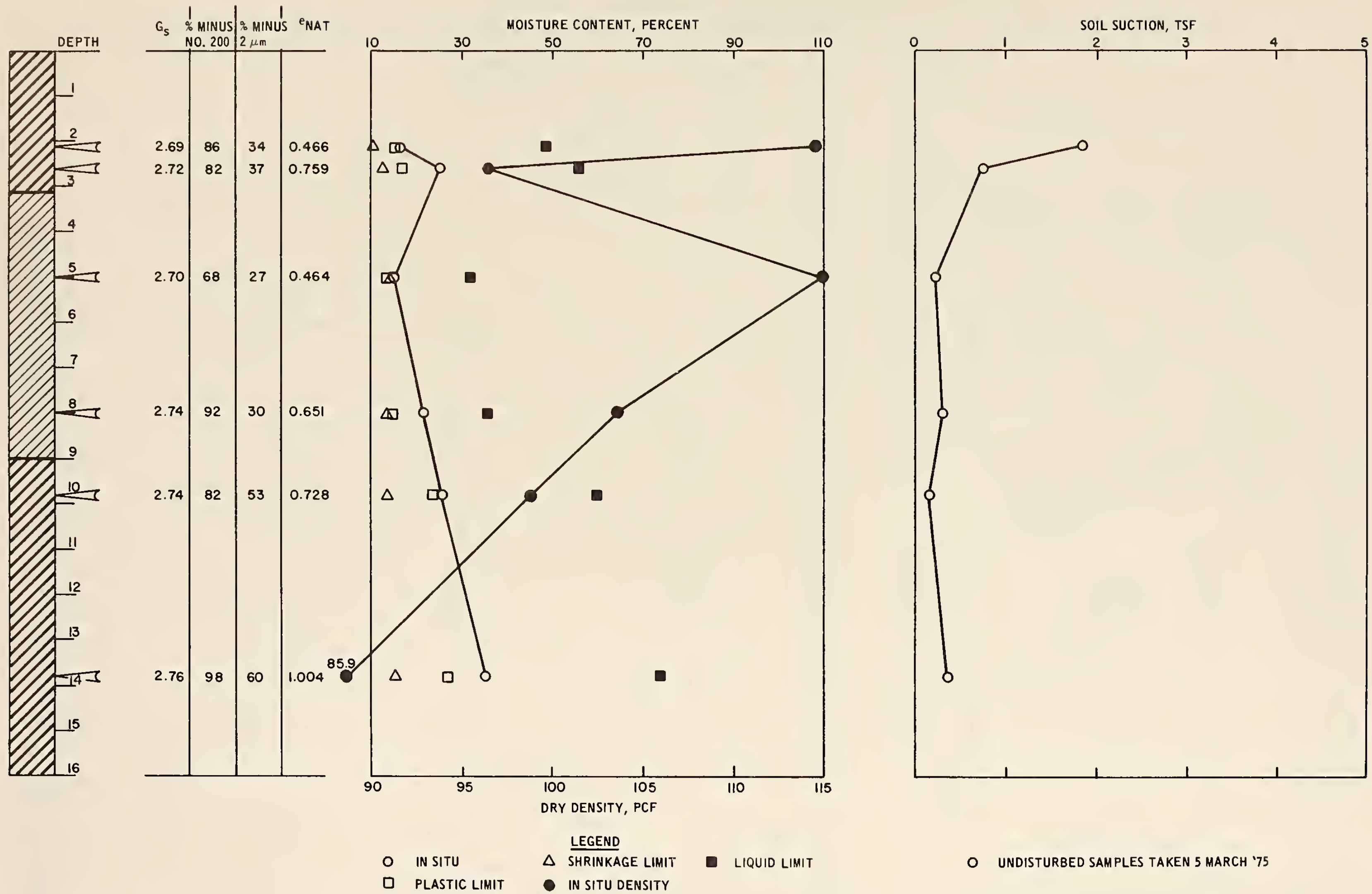


Figure 12. Boring log and moisture content, dry density, and soil suction profiles for sampling site 4, Lake Charles, Louisiana

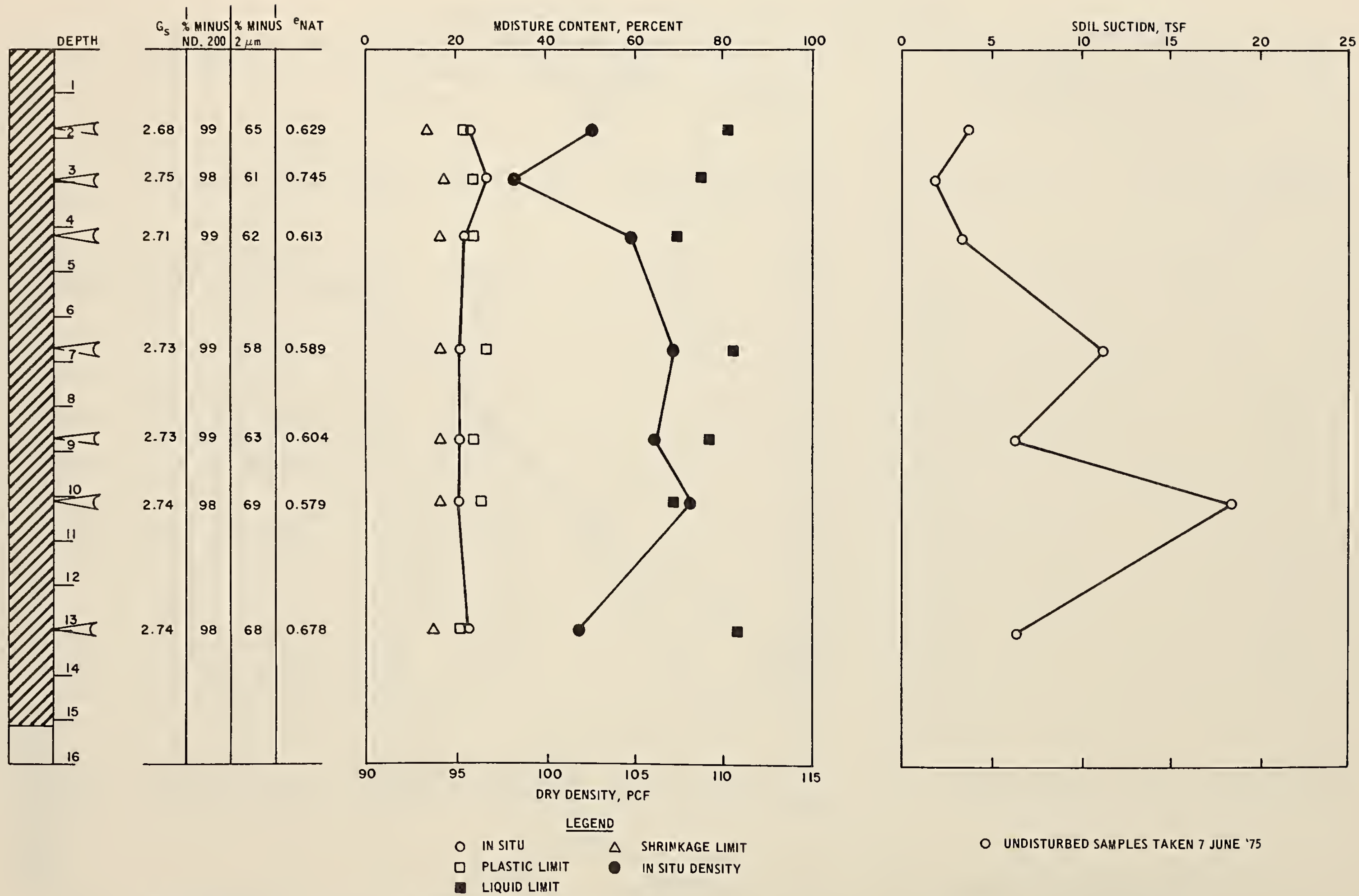


Figure 13. Boring log and moisture content, dry density, and soil suction profiles for sampling site 12, Hayes, Kansas

set at 8.0 ft (approximate midpoint of two depths involved was used because of the relatively large difference in the magnitudes of soil suction).

<u>Example</u>: Sampling site 17, Newcastle, Wyoming, Figure 14: the soil suction decreased to 0.2 tsf at 6.4 ft, then increased to 7.8 tsf at 9.1 ft, and the depth of active zone was set at 7.0 ft (next whole foot below major change was used because of the relatively large difference in the magnitudes of soil suction).

Although the "rules of thumb" do not provide exact determinations of the depth of active zone, they do provide reasonable estimates that are consistent with reported experience.

82. The final soil suction profile is a very important factor in estimating the magnitude of anticipated volume change. The accuracy of the prediction of potential heave hinges on the ability to estimate future equilibrium moisture conditions for a given profile. Russam,[45,46] Richards,[47] and Johnson[40-42] have all prepared guidelines for estimating final suction profiles and thus the final moisture conditions. In their simpliest terms, all of the recommended guidelines may be summarized as three assumptions, namely, saturated profile, negative hydrostatic, and constant at some equilibrium value. An additional assumption in which the final soil suction is assumed to be zero (i.e., $\tau_{mf} = 0$) may be used in the prediction technique. Figure 15 shows the soil suction versus depth profile at sampling site 12 and the four assumptions that can be used in the prediction technique. Assumption 1 requires that in the final soil suction profile, the soil suction is zero throughout the depth of the active zone. This assumption is extremely conservative and generally unrealistic relative to reported field behavior; however, it was included to provide a maximum limit to which a material is expected to heave. Assumption 2 requires a hydrostatic (i.e., triangular-shaped) relationship with zero soil suction at ground surface and increasing with depth to the actual profile value at the depth of active zone. This profile is also generally conservative but to a much smaller degree

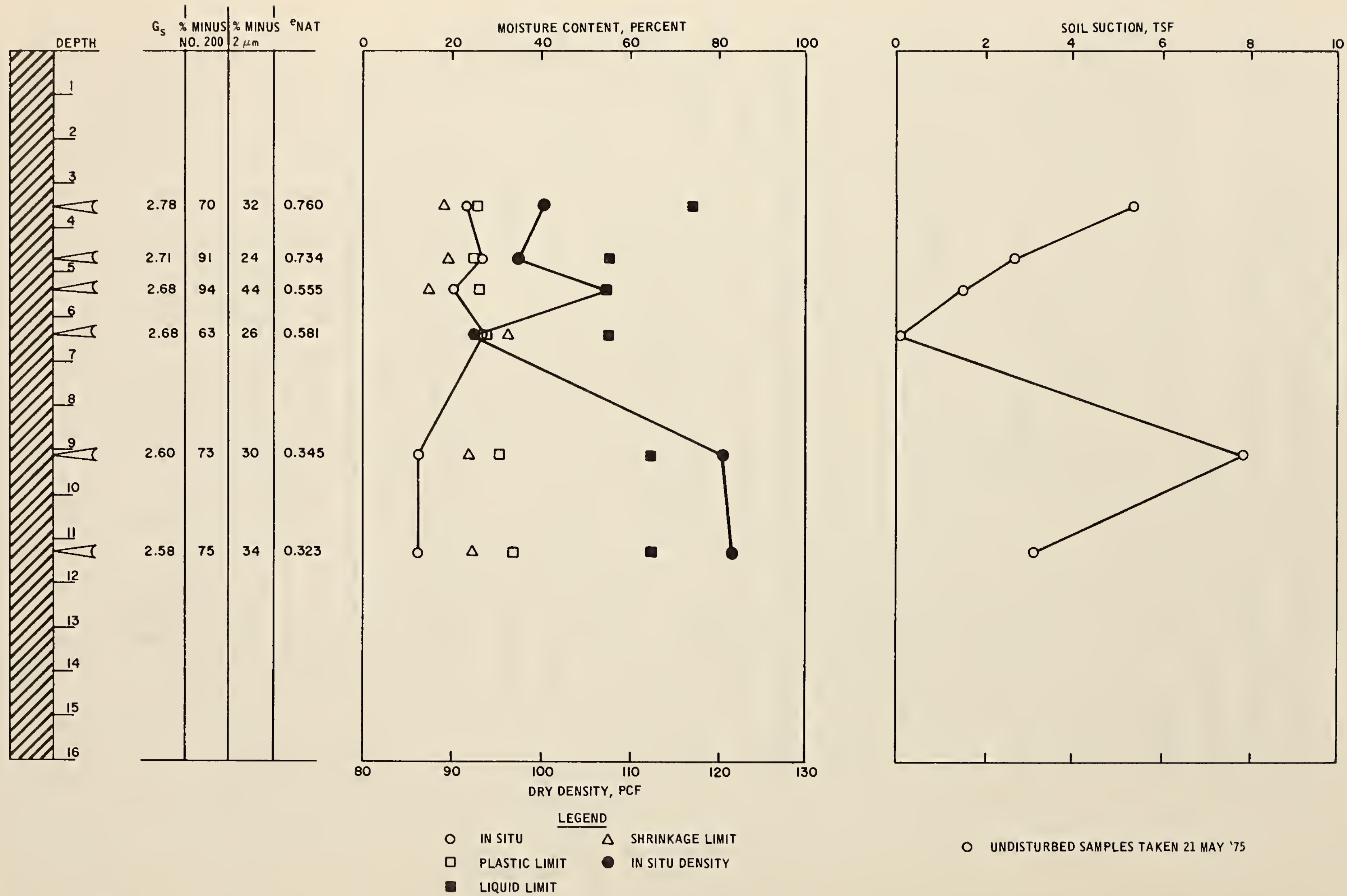


Figure 14. Boring log and moisture content, dry density, and soil suction profiles for sampling site 17, Newcastle, Wyoming

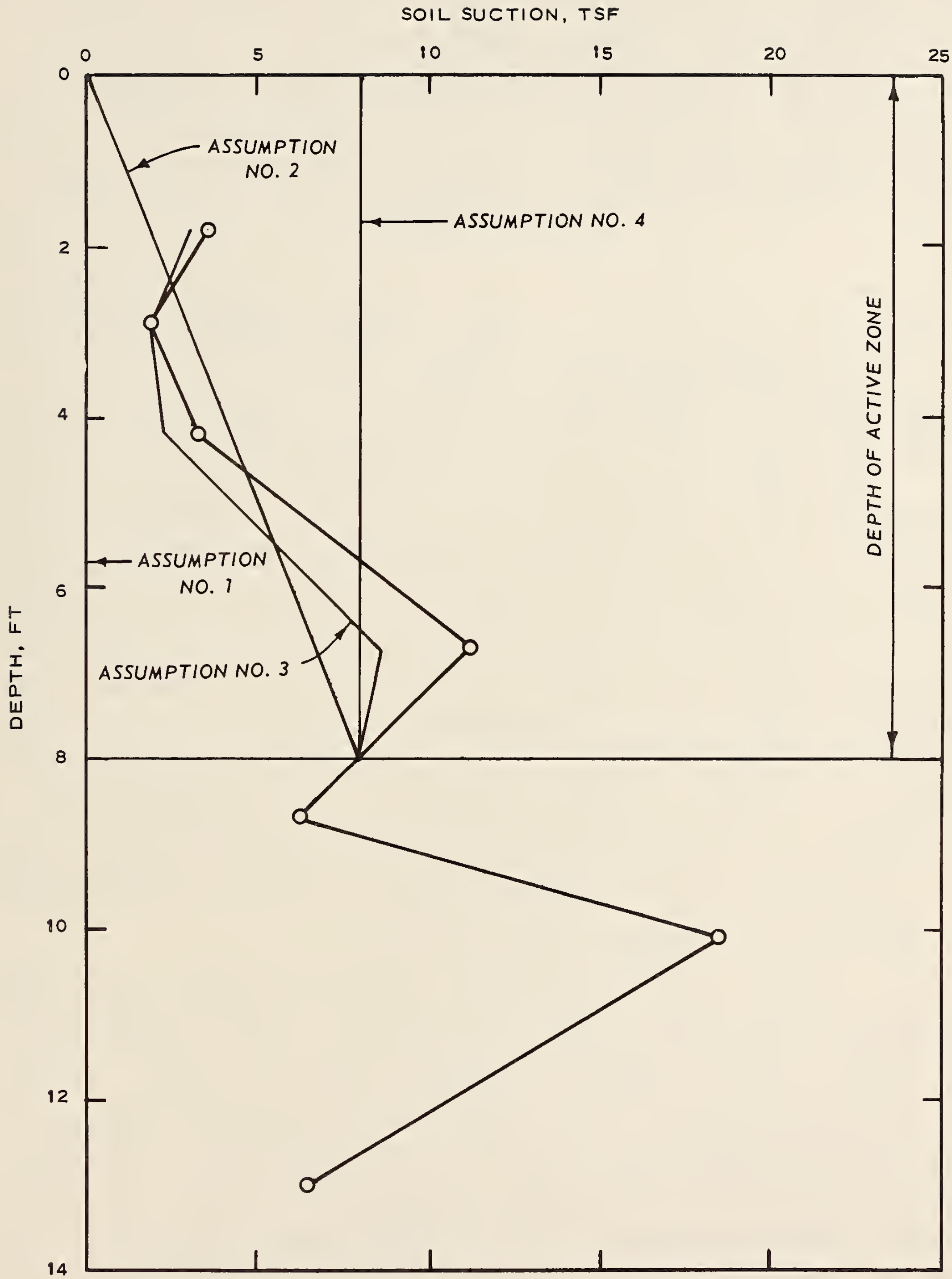


Figure 15. Soil suction versus depth at sampling site 12, Hayes, Kansas, with various assumptions for final soil suction profile shown

than assumption 1 simply because experience has shown that moisture contents beneath covered areas generally increase to some equilibrium value dependent on the local climate and soil type. Assumption 3 requires that the saturated water content be used in Equation 4 to estimate the final suction value. This is probably the most realistic value for estimating potential heave since it involves measused physical properties of the soils rather than assumed relationships. Assumption 4, which does not apply for site 12 but is shown to convey the concept, requires that the final soil suction profile be a constant value based on an equilibrium value beneath the depth of active zone. For example, at sampling site 4, Lake Charles, Louisiana, the average soil suction beneath the depth of active zone was approximately 0.25 tsf.[8,9] Therefore, assumption 4 required a constant soil suction value of 0.25 tsf through the depth of active zone. This assumption is limited to soil suction profiles such as the one exhibited at sampling site 4 for prediction of heave and may be more realistic for this type of profile since the moisture content will eventually increase in the upper zones causing the volume change. These assumed final soil suction profiles (four) are not necessarily the only profiles that can be used in estimating volume change. If field experimentation and experience have shown limits for subgrade moisture variations, then the moisture content plus some portion of the moisture variation range can be used in Equation 4 to estimate the final soil suction profile.

83. The major limitation to the prediction of volume change involves the fact that there is no way to effectively and accurately predict subgrade moisture regime changes, so to compensate for the void in technology, assumptions must be made that balance practicality, reality, and accuracy. The evaluation of the prediction procedure using the soil suction data indicated that assumption 3 provided the most accurate and reliable comparisons with measured field behavior. This stands to reason since the change in soil suction profile (initial to final) more closely approximates reported actual changes in the moisture regime with time. The important thing to remember about selecting a final soil suction

(or moisture content) profile is that consideration must be given to the initial profile, the expected sources of moisture infiltration, and the influence climate and seasonal change will have on the moisture regime. Where experience and field subgrade moisture variation data are available, the final soil suction profile assumptions may be modified to incorporate the observed variations.

Application of soil
suction prediction procedure

84. To demonstrate the use of the soil suction data for prediction of anticipated volume change, sampling site 12, Hayes, Kansas, will again be used as an example. The boring log and initial soil suction profile for site 12 are shown in Figure 13. Based on the "rules of thumb" concerning the depth of active zone, the value for site 12 is approximately 8.0 ft. Physical and engineering properties for all of the samples tested from site 12 are given in Table 4. Since the material within the depth of active zone is relatively uniform and four samples were tested within the same depth, the calculations will be made using four layers. Using the data from Table 4 and Equation 5, the surface heave can be calculated as shown in Table 5. The calculated surface heave is 8.1, 0.3, and 0.3 in. for assumptions 1, 2, and 3, respectively. As indicated previously, assumption 1 is excessively conservative and its highly unlikely that the excessive conservatism will ever be warranted; however, the assumption should be kept in mind for establishing maximum upper limits for volume change. Assumptions 2 and 3 provide the same answers because of the similar nature of the assumed final soil suction profiles. In this case, the assumption 2 profile and the initial profile exhibited the same general trend (i.e., increase with depth). For assumption 3, the initial and final profiles will always exhibit the same trend or shape since the only difference is the offset resulting from the change in water content from the initial value to the saturated value.

Table 4

Summary of Physical and Engineering Properties for Soil Samples from Sampling Site 12, Hayes, Kansas

Boring No.	Sample No.	Sample Depth ft	G_s	% -200	% -2 μm	Liquid Limit %	Plastic Limit %	Plasticity Index	Shrinkage Limit %	Shrinkage Ratio	Bar Linear Shrinkage %	Natural Water Content %	Natural Void Ratio e_{nat}	Natural Dry Density pcf	Natural Soil Suction tsf	A	B	α
U-1	1	1.7- 2.7	2.68	99	65	81	22	59	13.6	1.88	21.0	23.2	0.629	102.7	3.60	5.711	0.2222	0.84
U-2	1	1.4- 3.4	2.75	98	61	75	24	51	17.6	1.75	20.2	27.2	0.745	98.4	1.77	5.107	0.1786	--
U-2	2	4.1- 5.1	2.71	99	62	69	24	45	16.3	1.79	18.8	22.1	0.613	104.9	3.28	7.015	0.2941	0.92
U-1	4	6.6- 7.9	2.73	99	58	82	27	55	16.6	1.89	16.0	21.1	0.589	107.3	11.19	6.195	0.2439	0.82
U-2	5	8.6- 9.4	2.73	99	63	77	24	53	16.5	1.79	16.8	21.4	0.604	106.2	6.20	6.012	0.2439	1.00
U-1	6	10.0-11.2	2.74	98	69	69	26	43	16.5	1.80	16.8	20.6	0.579	108.3	18.43	7.324	0.2941	0.84
U-2	8	12.9-13.8	2.74	98	68	83	21	62	14.8	1.89	19.4	22.5	0.678	101.9	6.45	5.310	0.200	0.74

Table 5

Example of Heave Calculations Using Soil Suction Data at Sampling Site 12, Hayes, Kansas

Depth ft	Layer No.	Layer Thickness ft	Physical Properties G_s	Physical Properties W_o %	Physical Properties e_o	Physical Properties S %	Soil Suction Parameters τ^o_{mo} tsf	Soil Suction Parameters A	Soil Suction Parameters B	Soil Suction Parameters α	Suction Index C_τ*	Final Applied Pressure σ_f** tsf	Final Soil Suction Profile, Assumption No. 1 τ_{mf} tsf	2 τ_{mf} tsf	3 τ_{mf} tsf	4† τ_{mf} tsf	Predicted Percent Swell and Heave for Each Layer, Assumption No. 1 Swell %	1 Heave in.	2 Swell %	2 Heave in.	3 Swell %	3 Heave in.	4† Swell %	4† Heave in.
	1	2.4	2.68	23.2	0.629	98.8	3.6	5.711	0.2222	0.84	0.101	0.08	0	1.8	3.1	-	10.8	3.1	1.8	0.5	0.3	0.1	-	-
2.4																								
	2	1.2	2.75	27.2	0.745	100.4	1.8	5.107	0.1786	0.88	0.135	0.19	0	3.0	1.9	-	8.0	1.1	-2.0	-0.3	-0.5	-0.1	-	-
3.6																								
	3	1.9	2.71	22.1	0.613	97.7	3.3	7.015	0.2941	0.92	0.085	0.28	0	4.3	2.3	-	5.8	1.3	-0.8	-0.2	0.6	0.1	-	-
5.5																								
	4	2.5	2.73	21.1	0.589	97.8	11.2	6.195	0.2439	0.82	0.092	0.43	0	6.7	8.6	-	8.7	2.6	1.2	0.3	0.6	0.2	-	-
8.0																								
Total Heave for Depth of Active Zone, in.																		8.1		0.3		0.3		-

* $C_\tau = \alpha G_s/100B$.
** Overburden pressure to midpoint of layer.
† Assumption No. 4 is not applicable to this side because of the variability of the soil suction below the depth of the active zone.

NOTE: Minus sign indicates shrinkage (or consolidation).

Overburden Swell Test and Prediction Procedure

85. Odometer tests involve the collection of swell (percent) or swell pressure data from samples inundated in distilled water and allowed to sorb water until an equilibrium condition is reached. In its simpliest form, the odometer swell test involves preparation of a test specimen (remolded or undisturbed), which is placed in an odometer and loaded to some specified load. The specimen is then inundated and allowed to expand until an equilibrium moisture content is obtained. The specimen may then be consolidated back to the original void ratio (alternative definition of swell pressure) and rebounded or removed and the final moisture content determined. Swell pressure is determined from an odometer test specimen using an identical procedure up to and including inundation of the specimen. At the point that volume change begins to occur, load is applied to maintain constant void ratio (i.e., constant volume swell pressure test). When an equilibrium state is reached the sample is rebounded and may then be removed to determine moisture content or reconsolidated to determine reload characteristics of the soil.

86. Estimates of anticipated volume change are made using odometer test data and applying a reverse consolidation theory. The applied load and structural rigidity generally determine which of the odometer tests (swell or swell pressure) should be used in the design of a specific structure. If applied loads are light and the structure is relatively flexible (i.e., pavement), then the deformation or swell should be quantified. If the applied load is large and the structure rigid (i.e., bridge abutment), then stress or swell pressure should be quantified. As indicated, pavements are lightly loaded and relatively flexible structures; therefore the deformation characteristics are most important.

Factors influencing odometer test procedures and results

87. Odometer test and prediction procedures described in the literature[8,9] are varied in their approaches to the problem of quantifying the characteristics of expansive soils; however, some common factors are

apparent. For example, they all involve a swell or swell pressure test, and nearly all rely on the reverse consolidation theory for quantification relationships. The fact that all of the tests are odometer-type tests leads to another common factor; that is, procedural factors that directly affect the testing procedure and, in turn, influence the results of the prediction equation. Fredlund[48] describes these procedural factors which should be of concern when testing expansive soils as:

a. Loading procedure. Specifically, the load increment ratio and duration of load. For example, the time required to obtain swell equilibrium is dependent on the amount of load and length time the load is applied.

b. Friction in components of odometer. Friction in the load application portion of the testing apparatus may result in actual load felt by the specimen being less than the applied load (i.e., weight or air pressure). The significance of the friction factor becomes greater for the lighter loads, particularly for the rebound curve.

c. Compressibility of the components of the odometer. Compressibility of the apparatus affects the measurement of swell pressure and the slope of the compression and rebound curves. Experience[49] has shown that deformation as small as 0.1 percent during a constant volume swell pressure test can result in a 10 percent error in measured swell pressure.

d. Compressibility of filter paper. Results of the use of filter paper are similar to those that occur from apparatus compressibility. In addition, the effect of filter paper compressibility is a time-dependent function as well as load dependent. In other words, filter paper compressibility does not occur instantaneously as does the apparatus compressibility. In fact, the magnitude of the filter paper compressibility may be as much as 2.5 to 5 times as great as the apparatus compressibility, depending on the load.

e. Seating of porous stone and soil specimen. The surfaces of porous stones and soil samples are rough surfaces that can interface with one another in the presence of water and applied load. The result is a deflection that is not reflected in dial gage readings. The net influence on measured swell or swell pressure is comparable to the compressibility problems previously described.

88. All of these procedural factors are inherent in odometer testing procedures, whether consolidation or swell properties are being measured. However, with careful consideration of the magnitude of the procedural problems, their influence can be significantly reduced. For example, in odometer swell or swell pressure tests, loads should be applied and maintained until an equilibrium condition is reached. Any procedure that specifies time limitations on volume change measurements should not be considered for routine use. Friction in the odometer apparatus can be checked by replacing the soil specimen with a load cell and verifying the amount of load actually being applied to the specimen. The influence of the friction can then be either minimized through appropriate maintenance or adjustment or corrected through calibration factors. Compressibility of the apparatus can be determined and appropriate calibration factors applied to correct the problem. Filter paper should not be used in odometer swell or swell pressure tests, thus eliminating the problem with filter paper compressibility. Porous stone-soil sample seating problems can be minimized by using finely ground porous stones. The solutions to the procedural problems are simple but often overlooked because of either minimal laboratory inspection and calibration or the lack of a standard odometer test for expansive soils.

89. Two other factors that influence the results of odometer testing and prediction procedures are lateral sample confinement and method of water application. These factors fall in the catagory of inconsistencies in the simulation of field conditions rather than equipment problems. In addition, very little can be done to eliminate their influence on the results of odometer test procedures. It is unlikely that an expansive soil will ever be completely inundated or confined

laterally in an in situ state. As far as odometer testing is concerned, these two factors must be compromised to obtain a practical measurement and estimate of anticipated volume change.

Test method

90. The Overburden Swell (OS) test being recommended is a deformation test consistent with the definition of potential swell presented elsewhere.[6] The loading sequence used for the OS test is shown in Figure 16a. A detailed description of the OS test procedure is presented in Appendix B;[50] the following general description is provided here for continuity. After careful preparation of the test specimen, it is placed in the odometer and a light seating load (0.01 tsf) applied. The load remains at this value for approximately 5 min. The odometer ring and water container are covered with cellophane wrap to minimize moisture change by evaporation. After 5 min, the applied load is increased to the in situ overburden pressure. The specimen is inundated 30 min after application of the overburden pressure unless significant deformation occurred. In which case, inundation is delayed until essentially all of the deformation has occurred. As swelling occurs, a deformation versus log of time curve is maintained. When the deformation versus log time curve levels off, the consolidation portion of the test is begun. The specimen is consolidated until the void ratio at the overburden pressure is reached, thus providing an alternate definition of swell pressure; then the specimen is rebounded in decrements to the seating load. At each load in the consolidation and rebound portion of the curves, the deformation versus log time plot is maintained to assure that equilibrium is reached as close as possible under a given load. The specimen is then removed for water content determinations.

91. In the event swell pressure data is required, the Constant Volume Swell Pressure (CVSP) tests can be conducted on undisturbed or remolded specimens. The loading sequence of the CVSP test is shown in Figure 16b and is identical to the OS test up to and including inundation of the specimen. At this point, when the specimen begins to expand, small increments of load are applied to maintain constant void ratio. When no

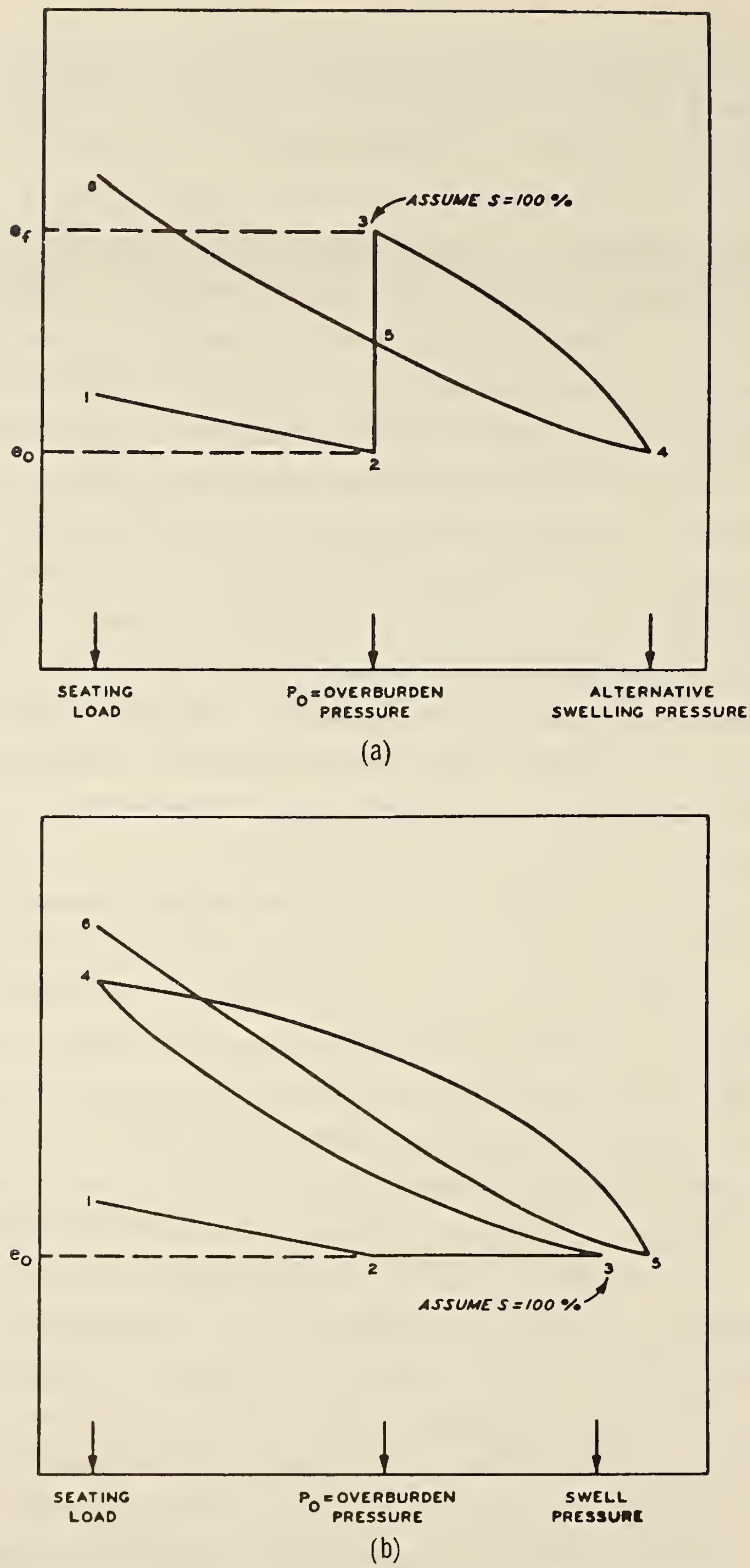


Figure 16. Loading sequence for (a) overburden swell test and (b) constant volume swell pressure test

further deformation is noted and the specimen is in equilibrium with the applied load, the swell pressure is defined. The specimen is then unloaded in decrements to the seating load, consolidated to the void ratio at the overburden pressure, and again unloaded in decrements to the seating load. The specimen is then removed for water content determinations.

Details of
heave prediction

92. Prediction of heave using OS test data involves the simple relationship:

$$\frac{\Delta H}{H} = \frac{e_f - e_o}{1 + e_o} \tag{7}$$

where

e_o = initial void ratio corresponding to overburden load conditions

e_f = final void ratio corresponding to 100 percent saturation

$\frac{\Delta H}{H}$ = percent swell

The predicted heave is the product of the percent swell and layer thickness represented by the soil test specimen. The initial void ratio is determined from weight-volume relationships for the sample in its initial condition (i.e., overburden load). The final void ratio is measured in the OS test and corresponds to the equilibrium condition of the specimen under the applied load. To apply reverse consolidation theory, the final void ratio is assumed to correspond to a 100 percent saturation condition. This is one major disadvantage inherent in the OS test when overconsolidated clays and shales are tested, since their initial degree of saturation is relatively low and the time to acheive 100 percent saturation is very long if it can be attained at all.

93. As with the soil suction prediction technique, the OS test procedure and prediction technique requires an estimate of the depth of active zone that will limit the depth of required testing as well as the depth to which the prediction technique is applied. Estimates of the

depth of active zone may be made based on local experience (i.e., subgrade moisture variation studies), if available; otherwise, the value will have to be assumed. A reasonable range for assumed values would be between 5 and 8 ft.

94. Arguments for and against the use of the OS test based on accuracy could continue indefinitely. Therefore, the decision to use the OS test will have to be based on its application to specific conditions and the time required to obtain data versus time available prior to utilization of the data. If the OS test is used, then it should be conducted according to the loading sequence specified here; thus, comparison between organizations will be enhanced.

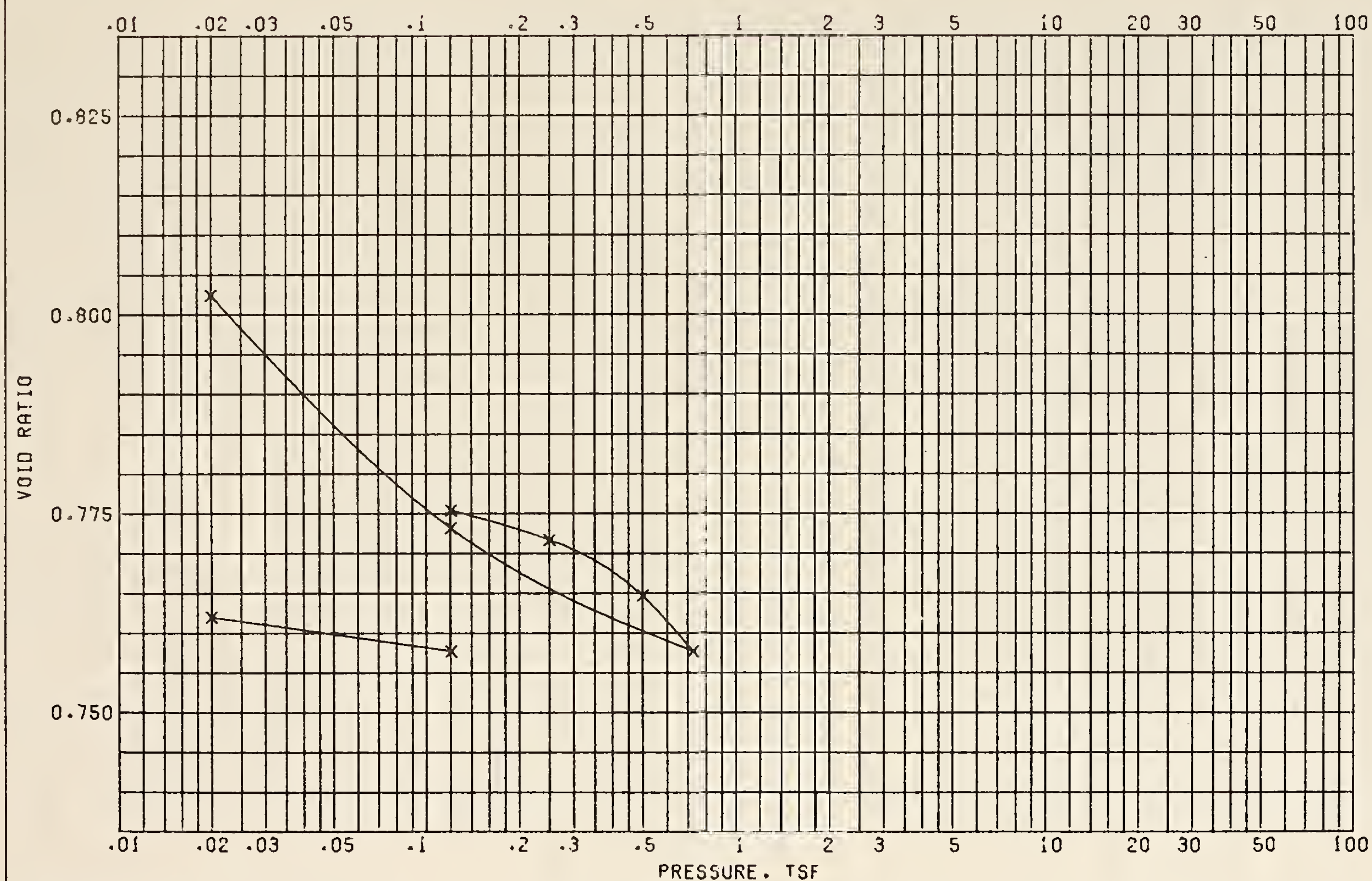
Application of OS test
prediction procedure

95. To demonstrate the use of the OS test data for prediction of anticipated volume change, sampling site 12, Hayes, Kansas, will again be used as an example. One specimen was tested using the OS test procedure, and the results are shown in Figure 17. Ideally, specimens should be tested from samples representing the various layers in the soil profile; however, for the purpose of this demonstration the single specimen tested will be assumed to represent the soil within the depth of active zone (i.e., 8.0 ft) as previously determined. Using the data presented in Figure 17 and Equation 7, the anticipated swell may be calculated as follows:

$$\Delta H = (8.0\ \text{ft})\ \frac{0.776 - 0.758}{1 + 0.758}$$

$$\Delta H = 0.08\ \text{ft}\ (\approx 1.0\ \text{in.})$$

The prediction procedure is very simple and direct; however, the results are limited in their application since the test results represent a single initial and final condition under the selected overburden pressure. To estimate the anticipated volume change for other loading conditions will require additional testing.



PROJECT	FHWA EXPANSIVE CLAYS
	IN HIGHWAY SUBGRADES
SITE	HAYES, KS (RERUN)
	U-2-1 1.4-3.4 FT

Figure 17. Void ratio versus log of pressure for undisturbed overburden swell (OS) test specimen, sampling site 12, Hayes, Kansas

Empirical Prediction Procedures

96. The soil suction and OS test and prediction procedures provide methods for characterizing expansive soils and quantifying anticipated swell for expansive soils in highway subgrades and should be applied to interstate and primary highways that encounter expansive soils. For secondary highways, which may tolerate more swell and have less funds available for exploration and testing, it will not be necessary to conduct the testing programs previously described. In lieu of these testing and prediction procedures, the Potential Vertical Rise (PVR)[51-54] may be used to estimate the anticipated swell. The PVR is an empirical procedure based on correlations between odometer swell test results on undisturbed and remolded samples and index properties. The PVR or surface heave is estimated using a family of curves that relate the PI, volumetric change, surcharge pressure (overburden), and PVR of an expansive soil stratum. The total PVR is the sum of the PVR's for the various strata within the depth of active zone. The Texas Department of Highways and Public Transportation standard method[51] for determining the PVR is presented in Appendix C. The required data and calculation details for determining the PVR are described in Appendix C.

97. Site 12, Hayes, Kansas, will again be used to demonstrate the calculation of the PVR. Obtaining the required data from Table 4 and using the procedures outlined in Appendix C, the PVR can be calculated as shown in Table 6.

98. The use of the testing and prediction procedures previously described is obviously a function of the availability of appropriate testing equipment and the time and funds to adequately sample and test the anticipated expansive soils within the selected route, as well as the priority of the highway (i.e., interstate, primary, secondary). Every effort should be made to sample and test suspected expansive soils as often and thoroughly as possible. As more information is collected and experience gained, particularly with the soil suction test and prediction procedure, more confidence can be placed on the sampling and

Table 6

Calculation of the Potential Vertical Rise for Sampling Site 12, Hayes, Kansas

Depth ft	Wet Density pcf	Avg Load psi	LL %	Dry 0.2 LL +9	Wet 0.47 LL +2	W_{Nat} %	Dry Avg Wet	PI	Volumetric Swell* %	Free Swell** %	PVR, in. Top of Layer	PVR, in. Bottom of Layer	Difference	PVR for Layer in.
0-2.4	126.5	2.1	81	25.2	40.1	23.2	Dry	59	17.6	21.4	0	2.05	2.05	2.05
2.4-3.6	125.2	3.1	75	24.0	37.3	27.2	Avg	51	11.7	15.1	1.19	1.63	0.44	0.44
3.6-5.5	128.1	4.8	69	22.8	34.4	22.1	Dry	45	13.4	16.9	2.03	2.81	0.78	0.78
5.5-8.0	129.9	7.1	82	25.4	40.5	21.1	Dry	55	16.4	20.1	3.66	4.43	0.77	0.77

PVR = 4.04

* Figure 1, Appendix C.
** Free Swell, % = (% Volumetric Swell) (1.07) + 2.6.

testing program for characterizing the expansive soil and quantifying anticipated heave.

Rate of Heave

99. Thus far the discussions have concentrated on estimating the magnitude of volume change for measured or assumed final conditions, i.e., ultimate heave. The rate of volume change or the rate at which the heave occurs is also important but much less significant than the magnitude, since it is much simpler to design a pavement or select a treatment alternative based on an estimated maximum volume change than to include both magnitude and rate of occurrence. The rate of heave is a function of the permeability and the amount of structural discontinuities present in the soil mass. The calculation of the rate of heave is solely dependent on the characterization of the soil's permeability, which is one of the most difficult properties to quantify, particularly for overconsolidated clays and shales. The problem with a dry expansive soil is that as water moves into the soil and volume change occurs, the permeability decreases across the wetting front. Therefore, the permeability is not only difficult to measure but also is a time-dependent variable that varies with distance from the wetting front. When appropriate laboratory data are available, the rate of heave can be estimated using time-settlement procedures from consolidation theory applied in reverse. More recently efforts have been made to use the diffusion theory[42,55,56] to predict rate of heave. The major problem with use of the diffusion theory is the determination of appropriate permeability values, which essentially eliminates the practical usefulness of the procedure. With all of the problems inherent in the measurement of permeability and prediction of rate of heave, the most practical approach to the expansive soil problem, considering the current state-of-the-art, is to concentrate on testing and predicting the magnitude of volume change.

PART VI: SELECTION OF TREATMENT ALTERNATIVES FOR EXPANSIVE SOIL SUBGRADES

100. Once an expansive soil has been characterized using the testing procedures previously described and an estimate of the anticipated volume change has been made giving consideration to the environmental conditions that influence volume change, a decision must be made whether to treat the subgrade soil to reduce the anticipated heave, thus minimizing possible damage to the pavement, or to forego treatment of the subgrade and repair the pavement damage as it occurs by remedial maintenance. Factors, in addition to estimated volume change, which must be considered in making this decision include: size of the project, construction funding, use of the highway (i.e., interstate, primary, secondary, farm-to-market, etc.), and comparative economics of the various alternatives. However, with construction costs continually increasing and past experience resulting in high maintenance costs, the decision is biased toward selection and use of preconstruction treatment alternatives based on sound engineering judgment tempered with experience from successful application of the best performing treatment alternatives. The purpose of this part of the report is to define and describe technical guidelines for systematic and logical selection of treatment alternatives or combinations of treatments that effectively minimize volume change and the associated damage to pavements.

101. The options available for minimizing volume change and pavement damage can be grouped into three categories, namely:

- a. Avoid the expansive material by route relocation or alteration.
- b. Mechanically or chemically alter the expansive material to reduce its potential volume change.
- c. Control subgrade moisture conditions by maintaining in situ moisture contents or by increasing the moisture content to an equilibrium condition.

All three of these categories are applicable to preconstruction conditions, while only certain alternatives within categories b and c are applicable to postconstruction conditions. Discussions in the remaining portions of this part of the report will be developed around the major categories with secondary discussions describing pre- and postconstruction applications of the treatment alternatives. A bibliography listing publications that describe the evaluation of and experiences with the various treatment alternatives is included as Appendix D. Some suggested guidelines for field monitoring data (type and frequency) to evaluate the effectiveness of the various treatment alternatives are given in Appendix E.

Avoid the Expansive Soil

102. Avoiding the expansive soil in lieu of more favorable subgrade conditions is a viable alternative only in limited situations, since route selection is generally based on local social, economic, environmental, or political considerations prevalent at this point in the design sequence. Furthermore at this time in the design sequence, generally very little information concerning subgrade soil conditions is available, which would be necessary to relocate or alter the selected route. In recent years, much more information has become available to help planners make better judgments concerning the suitability of a selected route. For example, occurrence and distribution maps, natural hazard maps as prepared by Krohn and Slosson, recent Soil Conservation Service county soil surveys, and U. S. and State Geological Survey publications-all of which map potentially expansive soils and were described in Part II. The availability of this information combined with State Transportation Agency experience, makes the option of avoiding the expansive soil problem a viable alternative in many situations.

Mechanically or Chemically Alter the Expansive Soil

Mechanical alteration

103. Mechanical alteration is the term used to describe treatment alternatives, which include ripping, scarifying, or otherwise remolding

the expansive soil to disturb the soil structure, then compacting the soil with moisture content and/or density control. Also included in this category is subexcavation and replacement, which can be achieved using granular material, nonswelling cohesive material, chemically treated material, or in situ material remolded and compacted to strict moisture content-density specifications. The obvious distinction between ripping or scarifying and subexcavation is that ripping or scarifying does not usually require removal of the material to be treated, while subexcavation does. In addition, the application depth of ripping or scarifying is limited to approximately 1.5 to 2.0 ft, while subexcavation has been applied to a maximum depth of 6.0 ft.

104. A third alternative which may be considered under this category, is surcharge loading or, more precisely, the use of fills over expansive soils to counteract the anticipated volume change with an applied load. This is a viable alternative provided a source of non- or low-expansive soil is available or some type of alteration of the soil is done if the expansive soil is used. Generally, the thickness of the fill is based on the load required to minimize deformation to specified limits (i.e., equal to swell pressure if design calls for zero deformation).

105. Ripping or scarifying. Reported experience with ripping or scarifying expansive soil subgrades to minimize volume change is limited at best; therefore, information from which technical guidelines can be defined is also limited. Ripping or scarifying the subgrade may be considered a minimal effort treatment alternative, since the depth of influence is not great (i.e., generally less than 2 ft) and the actual alteration of the soil is limited unless considerable effort is expended on mixing or remolding the soil. Ripping or scarifying is best suited for application to secondary highways since they can normally tolerate larger deformations and because the depth limitations of this treatment alternative preclude extensive alteration of the soil. However, this treatment may be considered for application to primary highways where the subgrade soils exhibit low potential swell and the uniformity of the final grade is of concern. If initial in situ moisture contents are low, then the ripping or scarifying should be followed by an application

of water (i.e., irrigation sprinkler, water truck, etc) to increase the moisture content for volume change reduction purposes and to facilitate compaction. Compaction should be controlled so that the placement conditions minimize the eventual occurrence of volume change. Typical placement conditions for at least the upper 6 in. of the ripped or scarified layer should be 92-95 percent maximum dry density and optimum moisture content minus 2 percent or greater (AASHTO T-99).

106. Subexcavation. Subexcavation and replacement involves a greater effort with respect to treatment of expansive subgrade soils since the minimum depth of application is generally not less than 2 ft with an average depth of application varying between 4 and 6 ft. Subexcavation and replacement is best suited for high potential swell materials that have a low initial moisture content (i.e., less than AASHTO T-99 optimum moisture content by approximately 10 percent or more) and a high initial density (i.e., greater than AASHTO T-99 maximum dry density by approximately 10 percent or more). These are not mandatory conditions for the use of subexcavation since other conditions may control (i.e., subgrade uniformity and elimination of fissures and fractures); instead, they are guidelines to consider when selecting an appropriate treatment alternative. Subexcavation and replacement requires removal and replacement of the expansive subgrade soil, and the material being put back should not cause problems with respect to the in situ material. For example, granular soils should never be used as backfill for subexcavation and replacement projects. Granular soils provide access for surface water to the in situ materials, and in some climates they provide the conditions necessary for hydrogenesis to occur. Backfill material should be cohesive and preferably nonswelling (i.e., silts, clayey silts, silty clays, or some clays) and be placed so that moisture intrusion is essentially eliminated. The soil being removed can be used as backfill provided it is either chemically or mechanically altered to reduce the swell potential. Chemical alteration, usually lime modification, will be discussed in a subsequent section of this part of the report, and mechanical alteration simply involves removal, remolding, and replacement of the natural soil to increase the moisture content, decrease

the dry density, and enhance the uniformity of the subgrade. Backfill material, particularly remolded in situ soil, should be replaced and compacted with careful moisture and density control. Typical placement conditions should be 92-95 percent maximum dry density and optimum or greater moisture content (AASHTO T-99). The lateral limits and depth of application of subexcavation and replacement are dependent on the potential swell of the subgrade soil. A typical example of the use of subexcavation and replacement for an interstate highway[57] is shown in Figure 18. The main-line subexcavation for the top 3 ft extended the full width of the divided four-lane roadway (i.e., shoulder slope to shoulder slope). The backfill material for the top 3 ft was select subgrade material. The subexcavation between a depth of 3 and 6 ft was confined to the subgrade shoulder lines with the backfill consisting of the remaining subexcavated material placed at a higher moisture content and lower density. The depth of application of subexcavation and replacement can be selected on the basis of experience within a given area, determination of the depth of active zone and the prediction of anticipated swell based on the modified properties of the backfill material (i.e., higher moisture content and lower density), or the following guidelines developed and used by the Colorado Department of Highways.[58] For interstate and primary highways, the depths of subexcavation and replacement and moisture density control[58] are:

Plasticity Index	Depth of Treatment, ft
10-20	2
20-30	3
30-40	4
40-50	5
>50	6

A slightly different set of guidelines are used for secondary and state highways:

Plasticity Index	Depth of Treatment, ft
10-30	2
30-50	3
>50	4

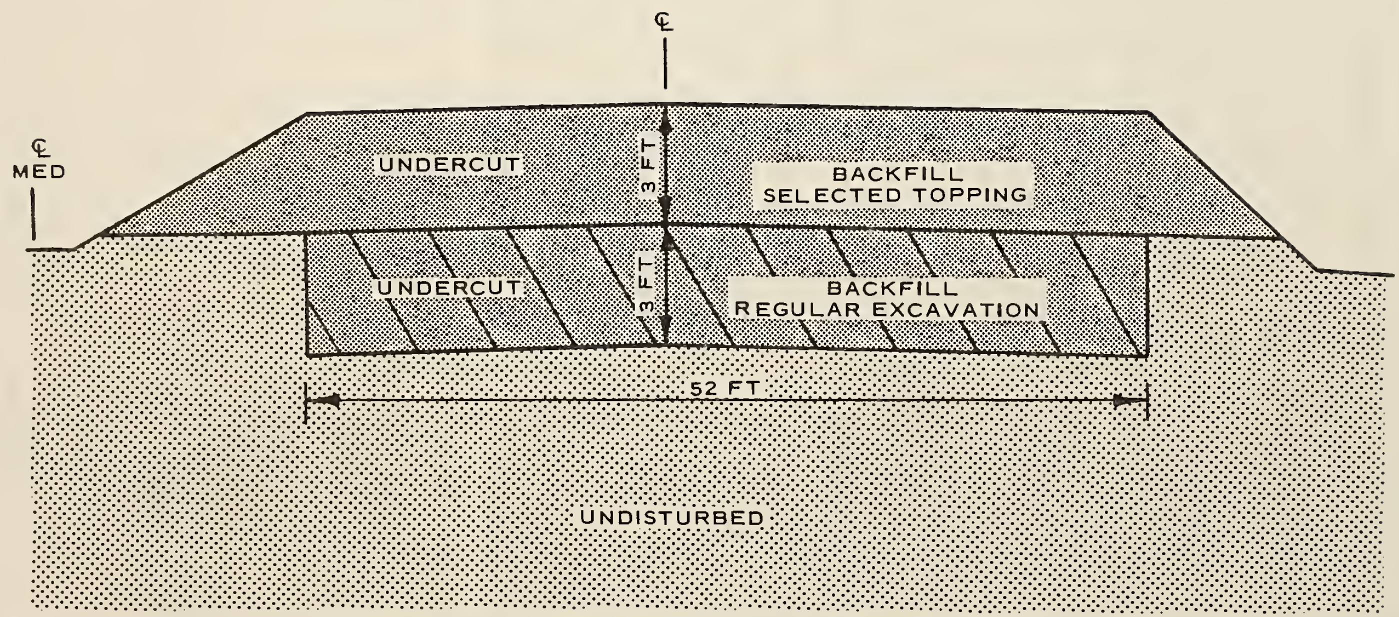


Figure 18. Typical example of subexcavation and replacement for an interstate highway

107. Surcharge. The obvious objective of surcharging an expansive soil is to load the soil with a sufficient amount of fill material to counteract the anticipated volume change. The practical application of this treatment alternative is limited to soils exhibiting low swell potential because of the generally large loads required to minimize deformation of highly expansive soils. In this content, surcharging would be considered a minimal effort treatment alternative best suited for secondary highways or low swell potential soils. However, if a source of low- or nonswelling material is available and fills can be incorporated into the final grade with a minimum of additional expense, the alternative should be considered. The fill thickness can be determined by the equivalent thickness of overburden to balance the measured swell pressure or by using the soil suction technique minimizing the deformation by applying additional load.

Chemical alteration

108. Chemical alteration refers to the addition of chemical compounds that alter the characteristics of the clay mineral or clay-water combination which, in turn, reduces the potential expansiveness. Literally hundreds of chemicals have been tried. Cementation by lime, lime-flyash, and cement have been tried. Ion exchange (addition of divalent or trivalent salts), cation fixation in expanding lattice clays (with potassium), deactivation of sulfates (with calcium chloride), waterproofing (with silicones or asphalts), cementation (silicates, carbonates, lignins, phosphoric acid), and alteration of permeability and wetting properties (surface active agents) have all been used to attempt to reduce expansive characteristics. However, due to mixing problems, economics, effectiveness, and practicality, none of these "exotic" compounds are recommended for large-scale routine treatment of swelling soils.[59] Lime continues to be the most widely used and effective additive for modification of expansive clays.

109. Although lime has been shown to be the most effective and reliable chemical stabilizer for expansive clays, the major limitation to its routine use is the application of the chemical to sufficient depth. Conventional mix-in-place techniques generally limit the depth

of treatment to approximately 8 to 12 in., efforts have been made with some success to extend that depth to 2 ft or more using specialized application equipment and procedures.

110. Preconstruction applications. The use of lime modification as a preconstruction treatment alternative to minimize anticipated volume change is well suited for fill construction using potentially expansive soils and to chemically alter backfill material in conjunction with subexcavation and replacement since the soil will be remolded during construction. For example, for fill construction application the lime can be applied and mixed in the borrow area, and for subexcavation and replacement the lime can be applied in the backfill stockpiles. Lime modification can be used successfully for normal gradeline treatment, providing the depth of treatment is sufficient to limit a large portion of the volume change. The effective depth is influenced by the potential expansiveness of the soil and the time and money available to apply the treatment alternative.

111. Before comparing lime modification with other treatment alternatives on a construction time and economics basis, some basic questions must be answered. The questions included:

- a. Is the soil lime-reactive?
- b. How much lime (in percent) is required to achieve the desired volume change reduction?
- c. How much influence on soil properties should be required to produce the volume change reduction?

The answers to questions a and b involve the selection of a lime content, or more specifically the "lime modification optimum" (LMO), which minimizes the volume change. By definition the LMO corresponds the lime percentage that maximizes the reduction in plasticity.[8] The LMO can be accurately estimated using the lime-pH test suggested by Eades and Grim.[60] The lime-pH test provides an estimate of the percent lime required to effectively reduce the plasticity of a soil. The procedure suggested by Eades and Grim is summarized in Table 7. The procedure is very simple and can be completed in less than 2 hr. Basically the procedure involves mixing the dry soil and lime, adding water,

Table 7

Suggested pH Test Procedure for Silt-Lime Mixtures
(from Reference 60)

Materials:

1. Lime to be used for soil stabilization

Apparatus:

1. pH meter (the pH meter must be equipped with an electrode having a a pH range of 14).
2. 150-ml (or larger) plastic bottles with screw-top lids.
3. 50-ml plastic beakers.
4. CO_2 - free distilled water.
5. Balance.
6. Oven.
7. Moisture cans

Procedure:

1. Standardize the pH meter with a buffer solution having a pH of 12.45.
2. Weigh to the nearest 0.01 g representative samples of air-dried soil, passing the No. 40 sieve and equal to 20.0 g of oven-dried soil.
3. Pour the soil samples into 150-ml plastic bottles with screw-top lids.
4. Add varying percentages of lime, weighed to the nearest 0.01 g, to the soils. (Lime percentages of 0, 1, 2, 3, 4, 5, 6, and 8, based on the dry soil weight, may be used).
5. Thoroughly mix soil and dry lime.
6. Add 100 ml of CO_2 - free distilled water to the soil-lime mixtures.
7. Shake the soil-lime and water for a minimum of 30 sec or until there is no evidence of dry material on the bottom of the bottle.
8. Shake the bottles for 30 sec every 10 min.
9. After 1 hr, transfer parts of the slurry to a plastic beaker and measure the pH.
10. Record the pH for each of the soil-lime mixtures. The lowest percent of lime giving a pH of 12.40 is the percent required to stabilize the soil. If the pH does not reach 12.40, the minimum lime content giving the highest pH is that required to stabilize the soil.

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thoroughly mixing the combination, and determining the pH of the mixture. The results can be plotted on arithmetic scales to aid in determining the lime percentage at a pH of 12.4. Figure 19 shows the pH versus lime content (left) and the influence of lime on the Atterberg limits (right) for samples from site 12, Hayes, Kansas. From Figure 19 the LMO is approximately 4 percent. Also from Figure 19, it may be noted that as the lime content increases, the liquid limit decreases and the plastic limit increases with the net result being reduced plasticity index. The maximum reduction in plasticity occurs at 4 percent as indicated by the lime-pH test. The use of the Atterberg limits test in conjunction with the pH test is recommended since it provides verification of the lime percentage and a measure of the affect lime has on the given soil (question c). The Atterberg limits need not be run at all lime percentages used in the pH test. It is generally sufficient to run the Atterberg limits at the LMO and LMO plus and minus two percent.

112. The Atterberg limits, specifically the PI, provide a method of establishing the effectiveness of lime on reducing swell as well as plasticity. In Reference 7, the PI was shown to be one of the best indicators of potential swell for natural soil. From a lime treatment point of view, this is also true. If a 50 percent reduction in the plasticity index is not obtained at the LMO then the practicality of using lime to minimize swell is essentially nonexistent. For low and marginal potential swell soils, i.e. PI of 35 or less, the reduction should be to 15 or less. Alternatively, the influence of lime on the expansive soil can be determined by using the previously described testing and prediction procedures applied to remolded, lime-treated samples.

113. Postconstruction applications. Two lime-treatment procedures have been used as postconstruction treatment alternatives, namely, drill-hole lime and lime slurry pressure injection (LSPI). Both procedures are controversial with respect to their mechanisms and successful performance. Although conclusive data needed to clear the controversies, were not obtained during this study, the following descriptions of the mechanisms and the nature of the controversies are included to inform

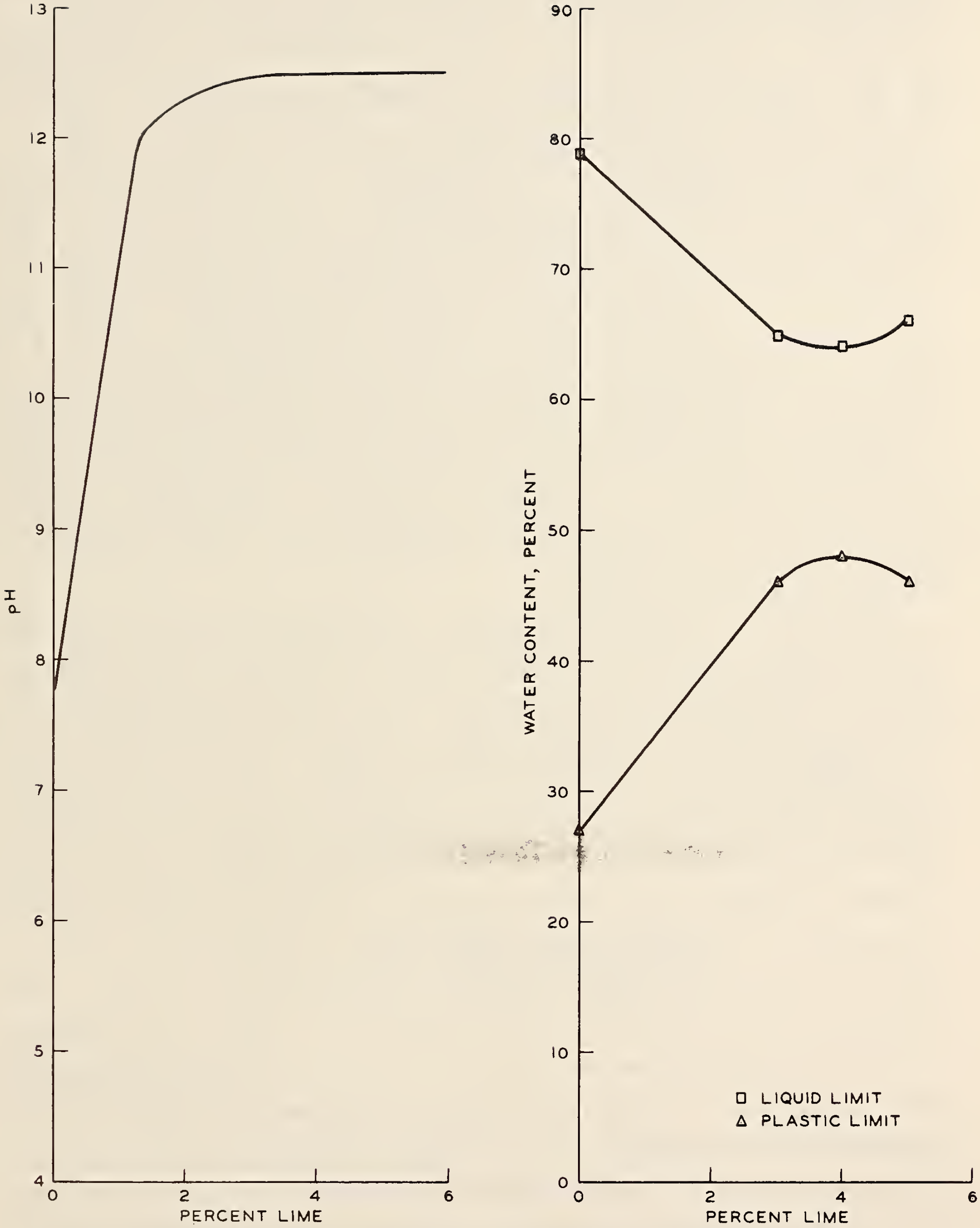


Figure 19. pH and water content versus percent lime for samples from sampling site 12, Hayes, Kansas

interested State Transportation Agency personnel of the existence of the procedures and precautions that should be taken before using them. Descriptions of experiences with the two procedures are given in Reference 8, and a selected bibliography on the subjects is included in Appendix D.

114. The drill-hole lime technique consists of drilling holes through the pavement and into the subgrade and backfilling the hole with a lime slurry or lime slurry-sand mixture. Once placed in the holes, the lime migrates or diffuses into the expansive soil and reacts (ion exchange) with the clay minerals. This diffusion process is quite slow and extensive time may be required before a substantial quantity of the soil is affected unless a system of cracks and fissures extend away from the hole. A more logical explanation of the mechanism and performance of the drill-hole lime technique involves lateral stress relief resulting from the borings and its effect on vertical deformation; this is combined with the wetting of the surrounding soil as water migrates outward from the lime slurry.

115. The LSPI technique consists of pumping lime slurry under pressures of up to 200 psi, depending on soil conditions, through hollow injection rods into foundation soils. The injection rods penetrate the soil at approximately 1-ft intervals, and the slurry (2.5-3.0 lb of lime per gallon of water) is injected until refusal. The center of the controversy over the LSPI has been the mechanism of movement into and reaction with fine-grained soils. Early proponents suggested that the slurry diffused "through" the soil mass with the aid of the injection pressure. In reality, the movement of the slurry through fine-grained or clay soils is through the cracks, fissures, or other discontinuities. Furthermore, actual diffusion of the lime into the soil is very limited; most of the calcium reaction occurs at the surface of the crack or fissure that the lime slurry flows through. Proponents of the LSPI suggest that this calcium reaction on the surfaces of cracks and fissures effectively forms a layer around the individual clods that helps minimize moisture content fluctuations. There has been no indisputable verification that this does occur. The two important points that these discussions emphasize is that first of all, the soil must be reactive,

and second, the soil must be capable of accepting the injected slurry (i.e., must be fractured sufficiently to allow movement through the soil mass).

116. At present, no experience has been reported in which the LSPI was used as a postconstruction treatment alternative for highway pavements on expansive soil. The majority of the experience has been the result of railroad track foundation stabilization to increase the strength of the foundation soil; however, in one situation an expansive soil was effectively stabilized with regard to volume change.

117. It is difficult to deny the potential that the LSPI has for certain applications in the expansive soil problem area, but by no means is it a cure-all for expansive soil subgrade problems. Although no standard analysis procedure is available for estimating the influence of LSPI on a given soil, some simple engineering tests can help provide sufficient information so that a practical assessment can be made concerning the selection decision. First of all, it should be determined whether the soil is reactive with lime. Techniques described in paragraph 111 should provide the answer to this question. The second question that must be answered is whether the lime slurry can be injected in the soil. This can be done by a simple injection pumping test. The details of the test need not be complicated; in fact, the simpler the better. If water can be forced through the soil, then it is likely that the lime slurry can also be injected.

Control Subgrade Moisture Conditions

Maintaining in situ moisture conditions

118. Since volume change of expansive soils is the result of variations in subgrade moisture content, it is obvious that if an expansive subgrade soil can be isolated from any moisture changes, then the volume change can be reduced or minimized. In this context, waterproofing membranes can be used to successfully minimize subgrade moisture variations and the associated volume change of expansive soils in highway subgrades.

The preconstruction application of the waterproofing membrane concept can be achieved several ways, namely:

- a. Continuous sprayed asphalt membrane over the entire subgrade and ditches (i.e., verge slope and a specified distance up the back slope).
- b. Full-depth asphalt pavement with a sprayed asphalt or synthetic fabric membrane beneath the ditch.
- c. Full-depth asphalt pavement with paved ditches in cut sections.
- d. Vertical synthetic fabric membrane cutoffs.

119. Waterproofing membranes, particularly continuous sprayed asphalt (catalitically blown, emulsified, or asphalt-rubber) membranes, have been used successfully in many states. Asphalt membranes perform best when applied over the entire subgrade section, down the verge slopes, and up the back slope a specified distance (i.e., equivalent vertical distance of approximately 1.5 ft above the ditch invert, or 6 to 12 in. above the finished pavement grade elevation). Membranes can be applied on different soil types and profiles as well as in various climatic zones. Although membranes have been used successfully in humid climates, they perform best in situations where: the soil profile is relatively dry; the moisture content profile is relatively uniform with depth; the groundwater table is at a sufficient depth and has no influence on near surface behavior; and the climate is dry-subhumid or drier (semiarid or arid). This stands to reason since these conditions generally describe a situation in which the major influence or subgrade moisture is from surface infiltration and the membranes, properly applied, essentially eliminate the surface moisture ingression. Variations of the waterproofing membrane concept, such as full-depth asphalt pavement with sprayed asphalt or synthetic fabric membranes beneath the verge slopes and ditches, provide a useful treatment alternative for minimizing subgrade moisture variation that should be limited to use on high priority roads (interstate and primary) over low potential swell soils and on low priority roads (secondary and state) for low, marginal, or high potential swell soils. In other words, for interstate and primary roads

on which the waterproofing membrane concept is used, a continuous sprayed asphalt membrane (back slope to back slope) should be applied for all cut sections. For fill sections, the membrane should extend a sufficient distance down the fill slope to assure that moisture cannot reach the compacted subgrade material (i.e., vertical of 1.5-2.0 ft below the finished pavement grade). If physically and economically feasible, the median in divided four-lane highways should be included in the membrane application (i.e., the median is less than two highway lanes in width). Typical examples of the continuous sprayed asphalt membrane and full-depth asphalt pavement with sprayed asphalt membrane beneath the ditches are shown in Figure 20. For the full-depth asphalt pavement with synthetic fabric membrane application, which would be similar to the upper diagram in Figure 20, care should be taken to assure that the membranes are properly placed and the lap-joints are sealed with some type of waterproofing compound (i.e., asphalt). For the full-depth asphalt pavement with paved ditches, which is also similar to the upper diagram in Figure 20, the membrane and loose soil cover are replaced with a paved layer consisting of an asphalt base material laid down and compacted to a finished thickness of approximately 2 in. This alternative should likewise be limited to the same situations described for the full-depth asphalt pavement with sprayed asphalt or synthetic fabric membranes.

120. Vertical membrane cutoffs, constructed of synthetic fabric membranes placed in narrow vertical ditches at the edge of a pavement and backfilled, have not been used extensively primarily because of construction problems and availability of a strong enough fabric to withstand placement and service requirements. Equipment capable of excavating a narrow trench (i.e., width less than 6 in.) to depths as great as 10 ft (depending on the soil type) and synthetic fabrics with significantly increased strength and durability are currently available for application of this treatment alternative. An example of a vertical membrane cutoff applied to a two-lane road is shown in Figure 21. Ideally, the depth of the vertical membrane cutoff should extend to the depth of the active zone; however, if the type of soil and the practicality and economics of the installation are not favorable, then the depth

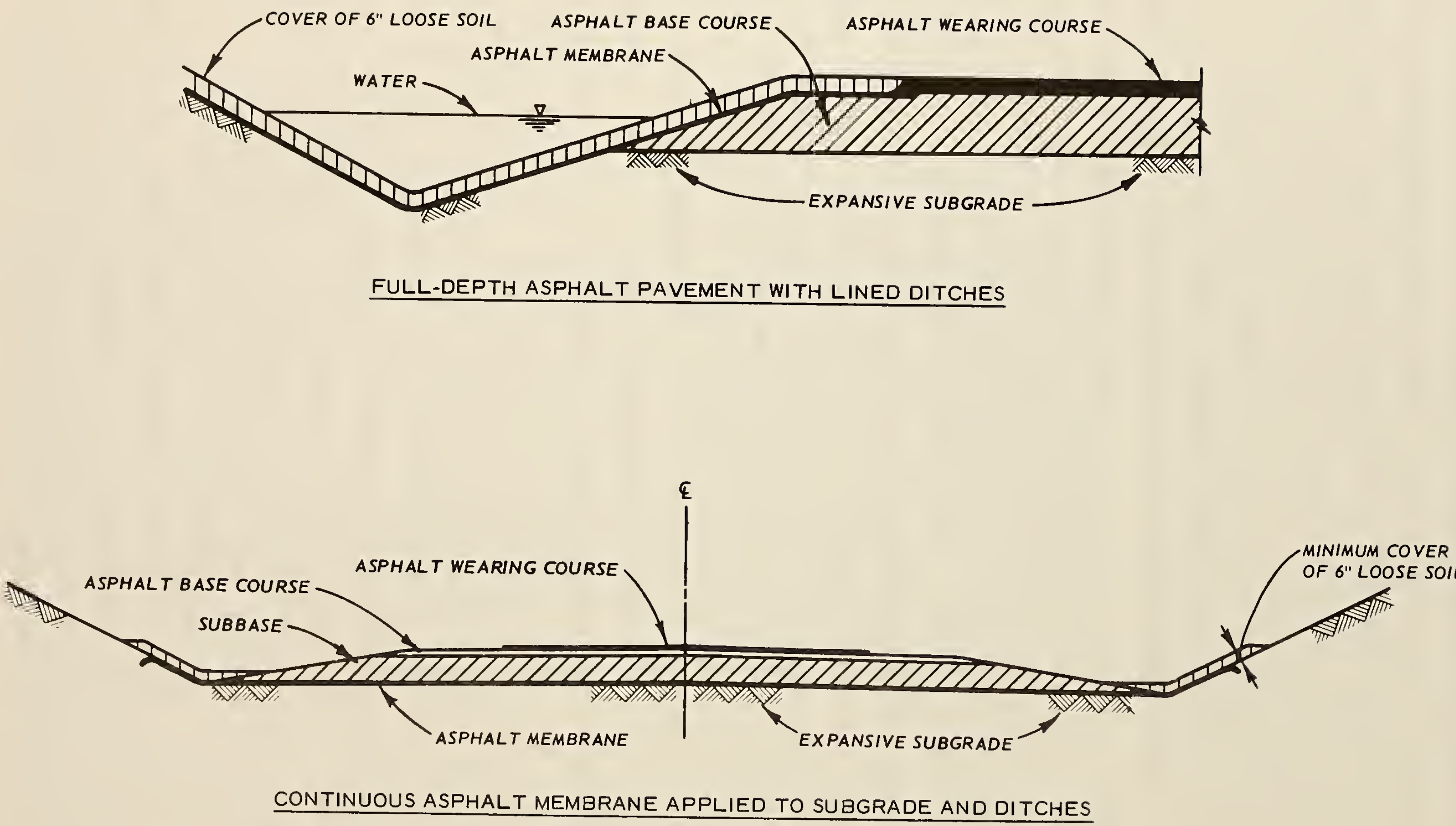
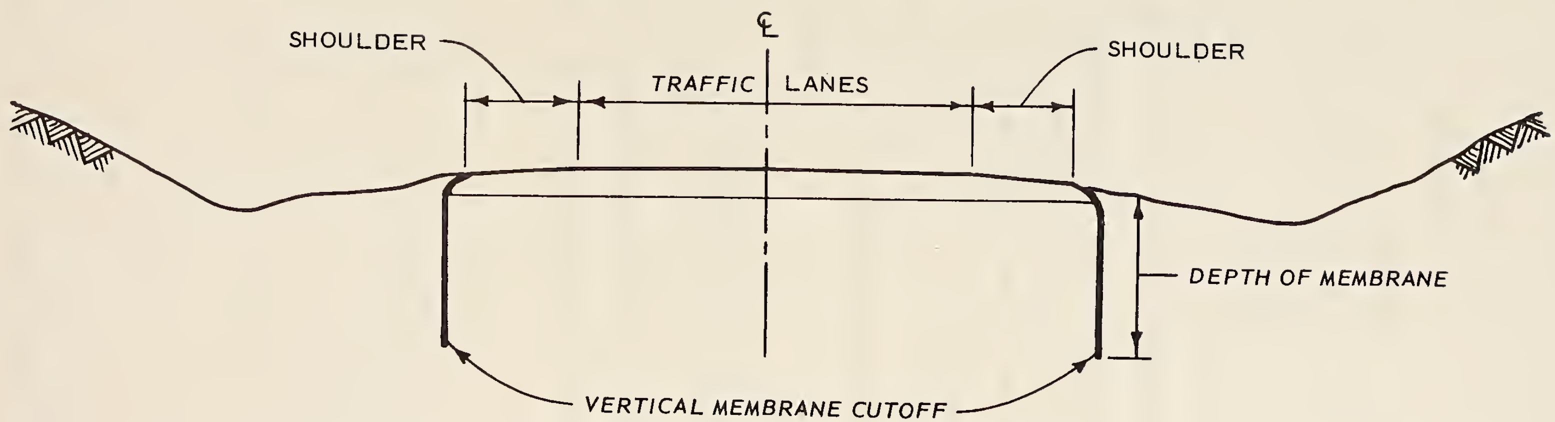


Figure 20. Typical sprayed asphalt membrane applications to minimize subgrade moisture variations from surface infiltration



NOTE: THE DEPTH OF THE MEMBRANE CUTOFF SHOULD EXTEND TO THE DEPTH OF ACTIVE ZONE; HOWEVER, IF THE SOIL CONDITIONS AND THE PRACTICALITY AND ECONOMICS OF THE INSTALLATION ARE NOT FAVORABLE THEN THE DEPTH SHOULD BE REDUCED TO FACILITATE INSTALLATION.

Figure 21. Example of vertical membrane cutoff construction

may be reduced. Membrane depths less than 2 to 3 ft will generally not provide enough positive influence to warrant their application. An important consideration in the construction of a vertical membrane cutoff is the type and placement of the backfill material used to fill the trench. The material should be as impermeable as feasible with the upper 18-24 in. of the backfill well compacted and the surface sealed (i.e., sprayed asphalt). If the trench is deep (i.e., greater than 5 ft) granular backfill material may be used in the lower portions of the trench, but the upper 18-24 in. should be cohesive material compacted to minimize the permeability and sealed with a sprayed asphalt.

121. Any of the previous waterproofing membrane concepts can be used as postconstruction treatment alternatives; however, nearly all of the reported experience has been with continuous sprayed asphalt rubber membranes. The treatment procedure involves placing a level-up course over the damaged pavement to reduce surface distortions and facilitate membrane application. Following any required drainage improvement work, the asphalt rubber membrane is applied to the pavement, shoulders, and verge and back slopes. Finally, a wearing surface course is placed. The asphalt rubber membrane provides two functions: first, it acts as a stress absorbing interlayer that helps reduce reflection cracking; and second, it provides a membrane that minimizes surface moisture infiltration. Example specifications for asphalt rubber membranes are given in Appendix F.

Increasing in situ moisture conditions

122. Increasing the in situ moisture conditions is achieved by prewetting or ponding the expansive soil subgrade with water. The objective of prewetting or ponding is to provide a continuous source of water so that a desiccated expansive soil can obtain a higher and hopefully more stable equilibrium moisture condition prior to constructing the pavement. In other words, the objective is to dissipate the potential swell prior to construction of the pavement. The higher initial moisture content achieved by ponding results in preswelling of the expansive subgrade and thus minimizes the long-term volume change and

associated pavement damage. Based on the evaluation of treatment alternatives in the research program, ponding appears to perform best in soils that are fractured or fissured and have subgrade moisture content profiles which are higher and more stable at depths (i.e., below a depth of 8-10 ft) and lower and more variable in the shallow depths. The best time to apply the alternative is during the dry season when the natural cracks and fissures are open due to desiccation. Surface earthwork should be minimized so that the ponding water can use the desiccation cracks as avenues into the soil mass. Boreholes with sand backfill have been used to expedite the saturation process. Determination of the depth that the ponding will influence and the time required to achieve that depth is still a major question requiring judgement. The logistics of applying the ponding alternative such as construction scheduling, construction of retaining dikes, water supply, drainage after completion, and lime treating the saturated surface, are the major points of consideration for the selection and use of ponding as a preconstruction treatment alternative.

Miscellaneous Postconstruction Treatment Alternatives

123. Thus far, discussions have centered on preconstruction treatment alternatives with subsequent discussions on postconstruction application for those alternatives that can be applied to reduce further damage to existing highways. From an experience point of view, the postconstruction alternatives already discussed make up the majority of the available selection. However, one relatively unique postconstruction treatment alternative that has been used with success in treating expansive soil subgrades is electroosmosis. The other miscellaneous item which merits mentioning as a postconstruction option is remedial maintenance.

Electroosmosis and base exchange

124. The concept of electroosmosis and base exchange with clay minerals is referred to as electrochemical soil treatment. The concept

involves placing electrodes (anode and cathode) in a soil mass and applying a current to initiate moisture movement. A stabilizing agent is then introduced into the soil mass (i.e., injection wells), and the electric current "moves" the solution into the soil mass so that the stabilizer can react with the soil. Based on a detailed study of electro-osmosis and base exchange, O'Bannon and Mancini[61] concluded that: electrochemical soil treatment can be successfully completed by maintenance personnel using the field procedure established in their study; the lower the initial moisture content and higher the percentage of montmorillonite the more effective the electrochemical treatment; and electrochemical soil treatment is most effective on a highly localized clay mass with a high swell potential. Electrochemical soil treatment is an effective but relatively expensive treatment alternative, which should be considered for localized expansive soil problems, such as highways in urban areas where right-of-way is limited, and for roadways beneath overpasses where maximum clearance is important.

125. In the recommendations in their final report, O'Bannon and Mancini[61] recommend that electrochemical soil treatment be implemented using the following generalized procedure:

a. Execute a preliminary sampling program.

b. Identify the degree of potential swell and estimate the depth of treatment based on the swell pressure of undisturbed sample.

c. Estimate volume of soil to be treated and determine amount of potassium chloride (KCl) required using a design figure of 1.6-2.0 percent per unit weight of soil.

d. Prepare selected site for electrochemical treatment, using 6-in.-diam auger holes (solution wells) approximately 6 ft deep and on 4- to 8-ft centers (exact spacing will depend on results of laboratory solution movement tests on soil samples). Place steel pipe sleeve into auger holes to a depth equal to the thickness of the pavement (surface and base courses) plus 6 in.

e. Excavate longitudinal trenches along pavement, place horizontal anodes and cathodes, and make the necessary electrical connections.

f. Fill solution wells twice a day for 30 days; then apply a voltage gradient of 0.2 volt/cm and continue to fill the solution wells for 3 to 4 weeks depending on electrode polarization pheonomena.

g. Collect sufficient posttreatment samples to determine the effectiveness of the electrochemical treatment.

Remedial maintenance

126. Remedial maintenance refers to maintenance techniques such as isolated overlays (patches) to level up distortions, removal and replacement of sections of the pavement and portions of the subgrade to remove the distortions, or mudjacking to level up distorted concrete slabs. These techniques are cosmetic repairs that improve the ride quality but do nothing to minimize the swelling characteristics of the subgrade soils. Although these techniques are superficial in nature, they form an integral part of a State Transportation Agency's maintenance program and will continue to do so. No set guidelines are available for application of remedial maintenance techniques. For isolated incidents in which the problems with distortions are not frequent and widespread, remedial maintenance techniques should be sufficient to maintain ride quality. However, when the distortions become more frequent in both distribution and occurrence, one of the postconstruction treatment alternatives previously discussed should be considered.

PART VII: SPECIAL DESIGN, CONSTRUCTION, AND MAINTENANCE RECOMMENDATIONS

127. The pre- and postconstruction treatment alternatives discussed in the previous part of this report constitute the major options for modifying or altering (mechanically or chemically) the characteristics of an expansive soil or controlling the subgrade moisture conditions (membranes or ponding) in order to minimize the volume change in expansive highway subgrades. This part of the report deals with practical design, construction, and maintenance recommendations, which have as their primary function minimization of moisture infiltration into the expansive subgrade. These recommendations do not connote changes to actual design criteria or procedures; instead, they are practical alternatives to be considered during the design of and implemented during the construction of a new highway or used as part of a maintenance or rehabilitation program for existing highways. To facilitate subsequent discussion, the design and construction recommendations will be considered in two categories:

- a. Drainage (surface and subsurface).
- b. Pavement cross section features.

Likewise, the maintenance recommendations will be discussed using two categories:

- a. Drainage (surface and subsurface).
- b. Routine maintenance and repair.

Since no specific research programs have been conducted in the concepts being considered in this part of the report, the major sources of information used to obtain the supporting data were State Transportation Agency experience as gathered primarily through personal contact and secondarily through State Transportation Agency publications.

Design and Construction Recommendations

Drainage

128. Surface drainage. The lack of proper drainage is probably the most significant factor leading to volume change of expansive

subgrade soils and thus damage to the overlying pavement. In a majority of the situations in which damage has resulted to a pavement placed on expansive soils, the cause of the problem can be directly related to the lack of proper drainage. Some obvious signs that adequate drainage is not available are ponded water in the ditch, soft spots in the ditch or verge slope, or the presence of plants or trees that grow best in saturated or submerged environments (i.e., willows, cattails, etc). The assurance of proper surface drainage is initially the responsibility of the design engineer since it is his design that will control the highway cross section. Figure 22 schematically represents some practical design and construction recommendations to help assure adequate surface drainage is maintained. Several of these recommendations are standard practice in some State Transportation Agencies; however, in many states these details are overlooked especially when expansive soils are present. Figure 22a shows a typical highway cross section. To assure adequate surface drainage and thus minimize surface infiltration, the verge slope should be between 1V:6H and 1V:8H. Generally, the 1V:6H slope is the maximum allowed under safety guidelines. In addition, the ditch invert should be approximately 25 ft from the shoulder's edge. One State Transportation Agency specifies a verge slope and a minimum of 3 ft vertically between the edge of the shoulder and the ditch invert, which is comparable to the 1V:8H slope requirement. The same verge slope requirements apply to fill sections. In the longitudinal direction, it is important that surface water collected in the ditches be removed as quickly as possible. This may be accomplished by establishing minimum ditch or median invert slopes. Figure 22b shows a sloping grade in a cut. AASHTO guidelines[62] for maximum grade vary with type of highway, design speed, and type of topography. If the situation shown in Figure 22b occurs in rolling or hilly terrain and the highway grade (slope 1) is greater that approximately 2 percent, then a sufficient gradient is available to remove the surface water. If the terrain is rolling or hilly and the grade is less than about 2 percent, then precautions should be taken to assure proper drainage. For example, slope 2 in Figure 22b should be greater than slope 1 by approximately 0.1 to 0.3 percent depending on the relative

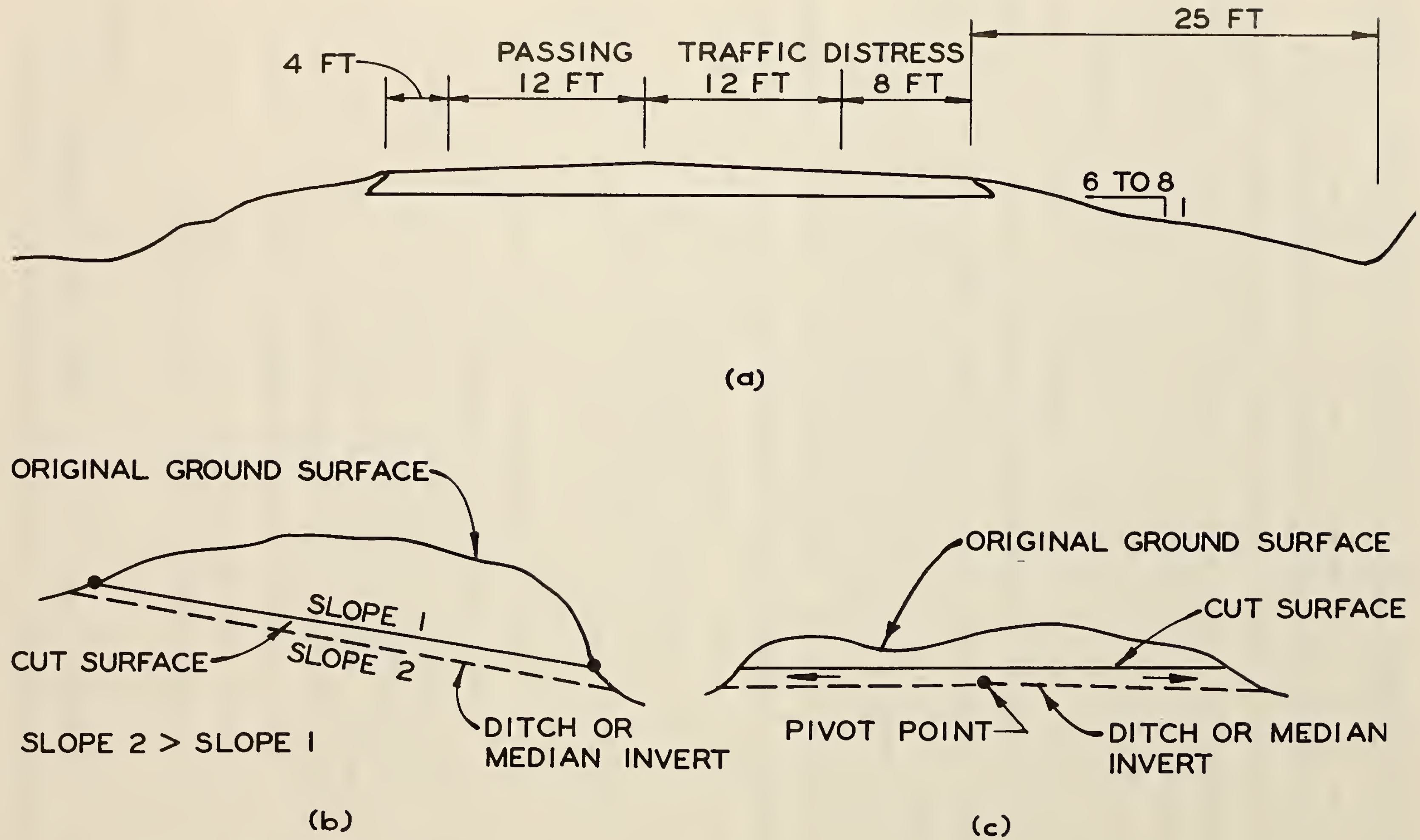


Figure 22. Diagram of surface drainage design and construction recommendations for (a) transverse and (b) and (c) longitudinal highway sections

grade (i.e., increase in slope 2 should be greater for smaller slope 1 values). If the terrain is flat to gently rolling, then the situation shown in Figure 22c would be more appropriate, particularly in long, shallow cuts. In this situation, a pivot point marks a change in slope of the ditch invert. The use of two slopes shortens the distance that water must travel to exit cut sections and thus reduces the amount of water that may accumulate in the case of sloughing of the cut slope. The amount of invert slope from the pivot point should provide adequate drainage without causing excessive earthwork or very deep ditches. Values between 0.1 and 0.5 percent should be sufficient.

129. Subsurface drainage. Subsurface water movement is generally a less severe problem in expansive soils because of the very low permeability of the material; however, in situations where the soil is fractured or fissured and a source of water (i.e., groundwater table, or a pond, lake, or other impoundment adjacent to a roadway) is available, precautions should be taken to insure proper subsurface drainage. Experience with subsurface interceptor drains in expansive clays and shales is somewhat limited. Whether a subsurface drain is needed, how deep and long the drain must be, and the drain component design are all factors dependent on specific situations. Guidance on these factors are provided in numerous publications[63-68] and will not be discussed in detail in this report. Some general guidelines concerning the use of interceptor drains in cut sections suggest that the drains be located adjacent to or just under the edge of the shoulder and preferably as deep as the seasonal active zone of the expansive soil.

Pavement cross section

130. Efforts to minimize surface moisture infiltration and pavement distortion can be significantly enhanced by the use of selected highway and pavement cross-section features. Some of the more practical features are discussed in the following paragraphs.

131. Avoid cut sections. Avoiding cut sections, particularly deep cuts, not only reduces the necessity of establishing new subgrade moisture equilibrium conditions beneath the new grade line, but it also reduces the elastic rebound component of volume change, which can be

significant in some shale materials. Avoiding cut sections also reduces the number of cut/fill transition zones, which in many states are the major cause of loss of ride quality. This recommendation does not advocate elimination of all cut sections; instead, elimination or reduction in depth of those cuts in high potential swell soils.

132. Uniformity at subgrade discontinuities. Particular care should be taken to assure that the physical characteristics are uniform at discontinuities such as cut/fill transitions, culverts, utility trenches, pipeline crossings, and any other situations that require a limited section of the subgrade be removed and replaced. At cut/fill transitions, significant differences can exist in the moisture content, density, and structure of the soils involved. Minimization of the difference in physical characteristics is the simplest approach to reducing the localized distortions. The subgrade in the cut section should be ripped or scarified (water added if required) and compacted to conditions comparable to the fill. The depth of scarification will depend on the properties of the natural soil and difference between them and the fill material; however, a minimum depth of 12 in. should always be considered. Preferably the depth will be between 18 and 24 in. Around culverts and in utility or pipeline trenches, the problem is the same (i.e., difference in physical properties); however, the areal extent is limited. Oftentimes, these types of discontinuities are backfilled with granular material, and moisture accumulation and significant local pavement distortions result. Granular soils should never be used as backfill material in expansive soil subgrades. Ideally, the backfill material should be a nonexpansive cohesive soil compacted to sufficient degree to minimize moisture infiltration into the trench. If the ideal material is not available, then the natural soil may be used providing the placement conditions are comparable to those discussed in the previous part of this report or subexcavation and replacement (i.e., thoroughly remolded and compacted with higher moisture content and lower density). For high potential swelling soils, consideration should be given to lime-treating the soil before using it as a backfill. Care should be taken to assure

that the backfill is not placed at a high initial density since this generally results in larger volume changes for similar moisture conditions.

133. Pavement type and feature. Full-depth asphalt pavements generally perform better when placed over highly expansive soils than their concrete counterparts. Full-depth asphalt pavements provide several advantages when expansive soils are encountered, namely: (a) the pavement provides a "membrane" that fullfills the requirement of the waterproofing membrane concept and thus minimizes surface moisture infiltration; (b) if volume change occurs, the "flexible" nature of the pavement allows it to accommodate more distortion before significant pavement failure; and (c) the remedial repair of a damaged asphalt pavement can be completed simplier and quicker whether removing and replacing or using specialized equipment such as heater-planners. Following the identification and classification procedures outlined in the early parts of this report, full-depth asphalt pavements should be used in areas of low swell potential if no subgrade treatment is included. For the marginal and high potential swell categories (i.e., from Part IV), full-depth asphalt pavement should also be considered providing some type of subgrade treatment is used. In other words, in the marginal and high potential swell areas, full-depth pavements are not sufficient without additional preconstruction treatment. For remedial repair involving removal and replacement, the same criteria should be used in selecting the type of pavement. When concrete pavement is used, it is recommended that an asphalt-treated base course be placed beneath the pavement and that it be continuous from shoulder to shoulder.

134. Another pavement cross section feature that can be helpful in reducing moisture infiltration is the paved shoulder. AASHTO guidelines[62] suggest a 10-ft right shoulder (distress lane) and a 4-ft left (median) shoulder. The further the infiltration wetting surface can be maintained from the travel and passing lanes, the less likelihood of damage to the pavement.[8] With this in mind, the AASHTO recommendations for a 10-ft right shoulder is sufficient; however, the 4-ft left shoulder recommendation should be considered an absolute minimum with a preferred width of 6 to 8 ft. The left shoulder width may vary depending on median

drainage characteristics, but the additional likelihood of severe damage to the pavement should be seriously considered if the width is reduced to less than 4 ft.

135. Special construction for membranes. When sprayed asphalt membranes are used as a pre- or postconstruction treatment alternative, then special attention should be placed on details during construction. For example, the surface to be sprayed should be smooth and free of debris (i.e., clods, sticks, etc.) so that holes in the membrane will not result. Care should be taken to insure that the membrane is continuous around culverts and overpasses. Following construction, structures such as guardrail, traffic sign, and reflector posts, should be sealed to minimize leaks through the membranes.

Maintenance Recommendations

Drainage

136. The comments in paragraphs 128 and 129 on surface and subsurface drainage recommendations would apply directly to maintenance or other postconstruction requirements. For example, in removal and replacement of damaged pavements, the criteria depicted in Figure 22 for surface drainage should likewise be applied as part of the reconstruction program. Guidelines[63-68] for subsurface drainage are equally applicable to postconstruction situations. The important recommendation within this category is maintenance of proper drainage. It is recommended that cut sections in problem soils be routinely monitored to remove the sloughs and drain any ponded water in the ditches. The same type of maintenance philosophy should be adopted for subsurface drainage systems, e.g., maintenance of outlet markers, clearing of obstructions at the outlet, and providing surface drainage away from the outlet.

Routine maintenance and repair

137. Recommendations within this category include the development of and adherence to a judicious routine maintenance program, which should involve regular surveilance of pavement condition for roadways

in expansive soils areas. Sealing of cracks and joints in the pavement is an integral part of any maintenance program to minimize moisture infiltration. Several State Transportation Agencies have implemented crack and joint sealing programs that incorporate the use of elastomeric compounds such as asphalt rubber. These compounds have proven to be more resilient under various load and climate conditions than other sealing compounds and tend to adhere better to the crack surfaces.

138. Where sprayed asphalt membranes have been used as pre- or postconstruction treatment alternatives, special efforts should be expended to maintain the integrity of the membrane. For example, when traffic or maintenance vehicles penetrate the topsoil covering and puncture or tear the membrane, the opening should be cleared, repaired, and covered with an adequate thickness of topsoil. The same type of maintenance should be applied when guard rail, traffic sign, or reflector posts are routinely moved or destroyed by traffic.

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APPENDIX A

SOIL SUCTION TEST PROCEDURE USING THERMOCOUPLE PSYCHROMETERS

1. Soil suction is a quantity that can be used to characterize the effect of moisture on the volume and strength properties of soils; that is, soil suction quantitatively describes the interaction between soil particles and water, which determines the physical behavior of the soil mass. Total soil suction is the force responsible for soil water retention. Soil suction, expressed in units of pressure, is a measure of the pulling force exerted on water by the soil mass.

2. The total soil suction is the sum of the matrix and osmotic components. The matrix suction is the result of clay mineral surface attractive forces for water and cations (i.e., ion hydration) and the surface tension effects of water in soil. The matrix suction is both water content and surcharge pressure dependent. The osmotic suction arises from the presence of soluble salts in the soil's pore water and is the result of the difference in the ion concentration between the pore water and the clay particles double-layer water. Osmotic suction is independent of water content and surcharge pressure.

3. Table 1 of the Statement of the Review Panel in the preprint volume of the Proceedings of the First (1965) International Research and Engineering Conference on Expansive Soils describes the range of measurement and sensitivity in normal use of eleven techniques available for the measurement of soil suction. One of the better techniques with a wide measurement range and good sensitivity is the psychrometer. The thermocouple psychrometer measures the relative humidity in the soil by a technique called Peltier cooling. By causing a small direct current of about 4 to 8 milliamperes to flow through the thermocouple junction for about 15 sec in the correct direction, this junction will cool and water will condense on it when the dew point temperature is reached. Condensation of this water inhibits further cooling of the junction and the voltage difference developed between the thermocouple and reference

junctions can be measured using a microvoltmeter. With proper calibration the thermocouple psychrometer output in microvolts can be converted directly to soil suction in convenient units of pressure. Typical thermocouple psychrometer output voltages vary from less than one microvolt for relative humidities close to 100 percent or total soil suctions less than 1 tsf to about 25 microvolts for relative humidities of about 95 percent or total soil suctions of about 60 tsf.

Equipment

4. The equipment required to perform the soil suction test includes:

- a. Psychrometric microvoltmeter (WESCOR Model MJ-55).
- b. Eighteen thermocouple psychrometers (WESCOR Model PT51-10).
- c. Polystyrene thermal containers.
- d. Eighteen metal sample containers (1-pt paint cans).
- e. Eighteen rubber stoppers (size 13-1/2).
- f. Switches (4), switch box (1), and electrical connectors (18).
- g. Stopwatch.
- h. Specimen cutting equipment (i.e., wire saw, knife, bandsaw, etc.).
- i. Tare containers.
- j. Balance, sensitive to 0.1 g.
- k. Laboratory equipment for determination of dry density of the specimens by the volume displacement method.
- l. Calibration standards (WESCOR Osmolality Standards).

5. The equipment is set up by first drilling a hole (0.25-in. diameter) through the center of the rubber stoppers. The thermocouple psychrometer wires are fed through the hole so the psychrometer tip extends approximately 1 in. from the bottom (small diameter end) of the rubber stopper. The protective sheathing around the psychrometer tip should form an airtight seal in the hole in the rubber stopper. The electrical connectors, are affixed to the psychrometer wires for easy connection to the switch

box. The rubber stoppers are placed in the metal sample containers, which are placed in the thermal containers to minimize temperature variations. The switches are wired so that the output voltages (temperature and soil suction) can be monitored on each of the 18 psychrometers in turn. The equipment should be kept in a room where ambient temperature variations are minimal.

Calibration

6. Calibration of the equipment involves normal operation of the equipment with standard solutions, which result in known relative humidities, placed in the sample containers. The different relative humidities result in corresponding retention forces or soil suction values. Several standard solutions are tested, and the resulting microvoltmeter output, when converted to a standard temperature of 25°C, yields a linear calibration line for the individual thermocouple psychrometer.

7. The calibration begins by placing a small piece of filter paper (type and grade variable) in the bottom of each sample container along with 3 mℓ of the calibration standard. A minimum of three, preferably four, calibration standard concentrations should be used to adequately define the calibration line (i.e., 290, 1000, and 1800 mOs/kg). The equivalent moisture retention force or soil suction, in tons per square foot, is calculated by multiplying the concentration by 2.62×10^{-2} (i.e., 1800 mOs/kg × 0.0262 = 47.2 tsf). After sealing the sample containers with the rubber stoppers and placing them in the thermal containers, allow the temperature to equilibrate for approximately 24 hr. Begin taking temperature and soil suction output readings at least three times per day until the output readings stabilize. The time to stabilization varies with concentration of the calibration standard but will generally be in the range of 7 to 10 days.

8. The thermocouple voltage output (millivolts) is converted to temperature (°C) using the following conversion:

$$\text{Temperature, }^{\circ}\text{C} = \frac{\text{output in millivolts}}{0.0395 \text{ millivolts/}^{\circ}\text{C}} \tag{1}$$

The psychrometer (soil suction) voltage output, E_T (microvolts) is converted to the equivalent output at the calibration temperature of 25°C, E_{25}, by

$$E_{25} = \frac{E_T}{0.325 + 0.027T} \tag{2}$$

9. When at least three stable output readings are obtained, the average of the three readings is plotted versus the corresponding moisture retention force or soil suction on arithmetic scales as shown in Figure 1. A convenient scale for plotting the calibration line for the range of indicated calibration standard concentrations is 2.5 tsf/cm for the ordinate and 2.5 microvolts/cm for the abscissa. Typical thermocouple psychrometer calibration lines are linear and can be expressed using the following equation:

$$\tau = mE_{25} - n \tag{3}$$

where

τ = soil suction, tsf

m = slope of the calibration line

n = y-intercept of the calibration line

The slope will always be positive, and the y-intercept should be equal to or less than zero. The calibration line is good for the useful life of the thermocouple psychrometer; however, under normal use an annual check of the calibration by at least one point will assure that the equipment is operating properly.

Test Procedure

10. The soil suction test procedure is applicable to the sample condition (i.e., undisturbed or remolded) that controls the design for

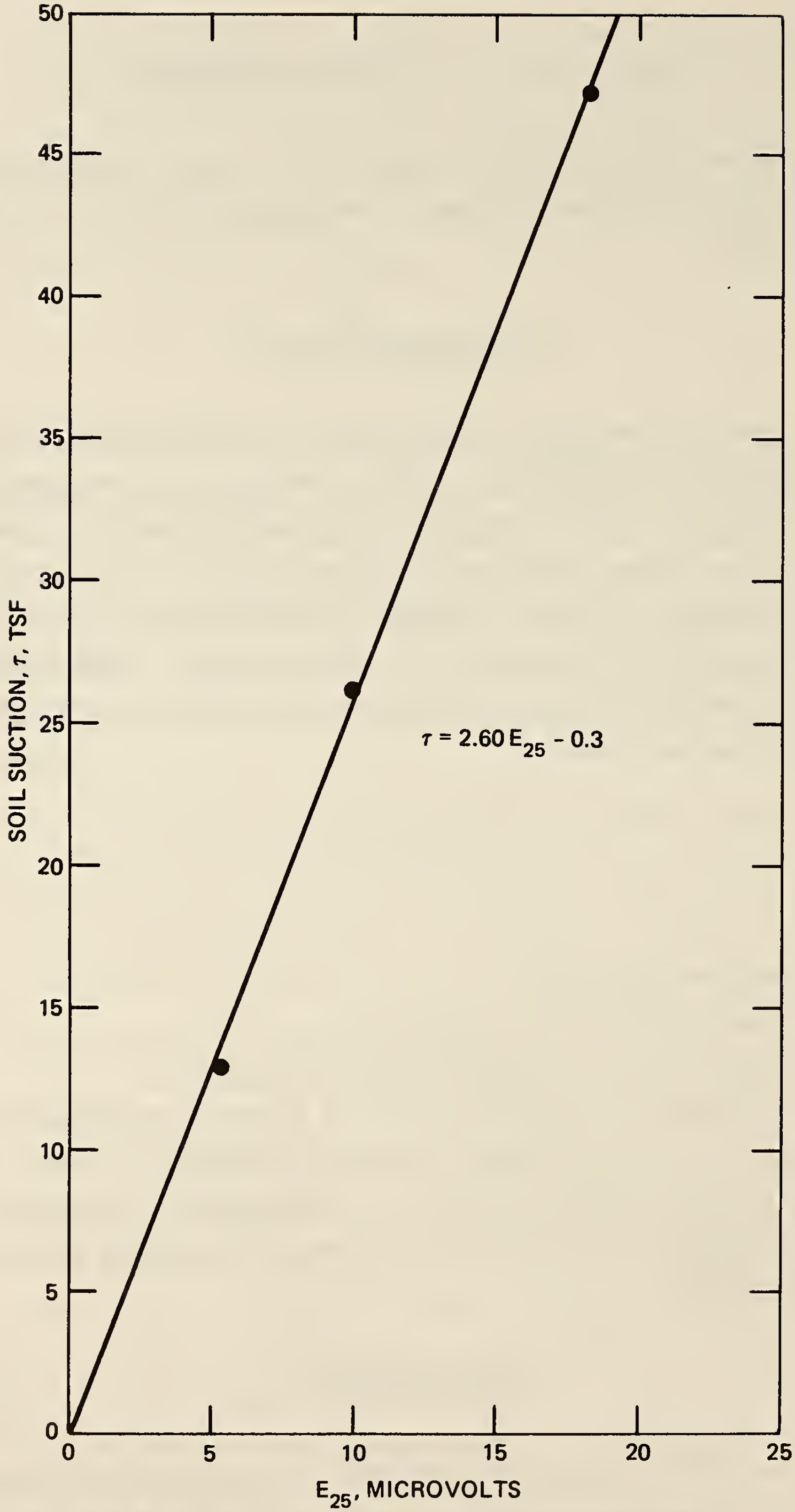

Figure 1. Typical thermocouple psychrometer calibration line

a specific situation; therefore, in the following discussions the term sample refers to the material being tested with the understanding that the condition of the material is dependent on the specific situation. The test procedure begins by cutting the sample selected for testing into nine approximately cubic specimens (approximate side dimensions of 1.5 in.). Two of the specimens are placed directly into the metal sample containers, sealed with the rubber stoppers, and placed in the thermal containers. These two specimens represent the natural conditions. The remaining seven specimens, depending on their natural water contents, are either wetted with varying amounts of distilled water or dried at room temperature for varying lengths of time to establish a range of water content conditions. For example, consider a soil that appears relatively dry (i.e., well below the plastic limit); to establish the necessary water content range, three of the specimens should be dried at room temperature (i.e., 1, 2, and 4 hr, respectively, would be reasonable times), and the remaining four specimens should be wetted with distilled water (i.e., 0.5, 1, 2, and 4 mℓ, respectively, would be reasonable amounts). The wetted specimens, already in the metal sample containers, should be sealed with the rubber stoppers and placed in the thermal containers immediately after wetting. Following the respective drying times, the dried specimens should likewise be sealed and placed in the thermal containers. The specimens sho ld be allowed to equilibrate in the sealed containers (usually approximately 48 hr will be sufficient), then they can be tested.

11. The actual test sequence using the previously described equipment involves:

- a. Selecting a thermocouple psychrometer using the appropriate switch and reading the temperature output in millivolts.
- b. Changing the switch from thermocouple to psychrometer, setting the meter to zero, applying a cooling current for 15 sec, and reading the psychrometer output in microvolts.
- c. Repeating a and b for each of the thermocouple psychrometers in the equipment setup.

After completing the test sequence, the specimens are removed, and the dry densities (volume displacement method) and water contents are determined for each. A suggested data sheet that assures correct collection of the required data is shown in Figure 2.

Data Reduction and Interpretation

12. The soil suction data is reduced by first converting the thermocouple output (millivolts) to temperature (°C) using Equation 1. The psychrometer output (microvolts) is converted to an equivalent output at the calibration temperature using Equation 2. The soil suction of the individual specimens is determined by substituting the equivalent psychrometer output into the psychrometer calibration line equation. The data is then plotted versus water content on a semilog plot to establish the log soil suction versus water content relationship, Figure 3, which is linear and has the form

$$\log \tau = A - Bw \tag{4}$$

where

A = y-intercept

B = slope

w = water content, percent

Generally, three-cycle semilog paper is sufficient to accommodate all of the data points. A convenient scale factor for the abscissa (water content) is 10 percent per inch. By keeping track of the points representing natural conditions, all of the data points are used to establish the $\tau - w$ relationship. If some variation occurs at the upper or lower end of the curve because the limits of the measurement range are approached, the data points between soil suction values of 2 and 20 tsf should be used to establish the $\tau - w$ relationship. The slope, B, of the line is determined by calculating the inverse of the change in water content over one cycle of the log scale. The intercept, A, is calculated by applying Equation 4 at soil suction equal to 1 tsf.

SOIL SUCTION, WATER CONTENT, AND SPECIFIC VOLUME

PROJECT ______ BORING/SAMPLE/DEPTH ______ DATE ______

SOIL SUCTION	PSYCHROMETER NO.												
	SAMPLE CONTAINER NO.												
	WATER CONTENT INCREMENT (0, +, −)												
	THERMOCOUPLE OUTPUT	t, MILLIVOLTS											
		T,* °C											
	PSYCHROMETER OUTPUT	E_T, MICROVOLTS											
		$E_{25°C}$,** MICROVOLTS											
	SOIL SUCTION†, TONS/FT^2		τ										
WATER CONTENT	TARE NO.												
	WEIGHT IN GRAMS	TARE PLUS WET SOIL											
		TARE PLUS DRY SOIL											
		WATER	W_w										
		TARE											
		DRY SOIL	W_s										
	WATER CONTENT, PERCENT		w										
WEIGHT-VOLUME RELATIONS	TEST TEMPERATURE OF WATER, °C												
	WEIGHT IN GRAMS	WET SOIL AND WAX IN AIR											
		WET SOIL	W										
		WAX											
		WET SOIL AND WAX IN WATER											
		DRY SOIL††	W_s										
	SPECIFIC GRAVITY OF SOIL		G_s										
	VOLUME IN CC	WET SOIL AND WAX‡											
		WAX											
		WET SOIL	V										
		DRY SOIL = W_s/G_s	V_s										
	DENSITY PCF	WET DENSITY = (W/V) 62.4	γ_m										
		DRY DENSITY = (W_s/V) 62.4	γ_d										
	VOID RATIO = $(V - V_s)/V_s$		e										
	POROSITY, % = $[(V - V_s)/V] \times 100$		n										
	DEGREE OF SATURATION, % = $[V_w/(V - V_s)] \times 100$		S										
	SPECIFIC VOLUME = $1/\gamma_d$		V_T										

* T °C = t/0.0395

** $E_{25} = E_T/(0.325 + 0.027T)$

† SEE INDIVIDUAL PSYCHROMETER CALIBRATION LINE

†† IF NOT MEASURED DIRECTLY, MAY BE COMPUTED AS FOLLOWS: $W_s = \dfrac{W}{1 + 0.01\,w}$

‡ VOLUME OF WET SOIL AND WAX = $\dfrac{\left(\text{WEIGHT OF WET SOIL AND WAX IN AIR}\right) - \left(\text{WEIGHT OF WET SOIL AND WAX IN WATER}\right)}{\text{DENSITY OF WATER AT TEST TEMPERATURE}}$

VOLUME OF WAX = $\dfrac{\text{WEIGHT OF WAX}}{\text{SPECIFIC GRAVITY AT WAX}}$

Figure 2. Suggested data sheet for collection of soil suction data

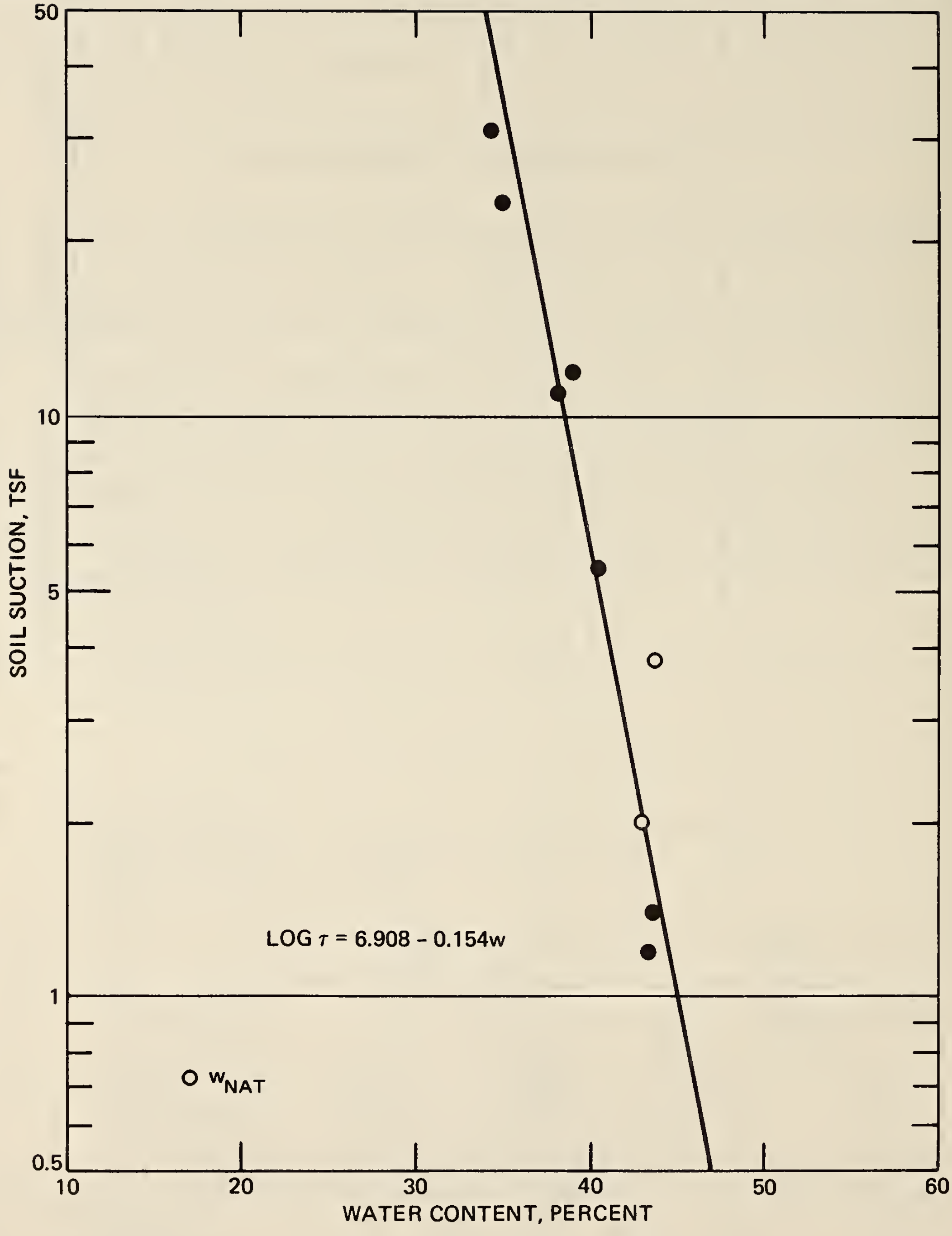

Figure 3. Typical soil suction versus water content relationship

13. Besides the A and B parameters, the prediction of volume change using soil suction data, a volumetric compressibility factor, α, is required that relates the change in volume to a corresponding change in water content. The value of α is determined by calculating the slope of the specific volume versus water content relationship (Figure 4). Convenient scale factors for the specific volume versus water content relationship are 0.25 units per inch for the ordinate (specific volume) and 5 percent per inch for the abscissa (water content). Occasionally, the specific volume versus water content data may indicate an α greater than one. In these limited situations, α should be taken as one since the compressibility factor cannot be greater than one.

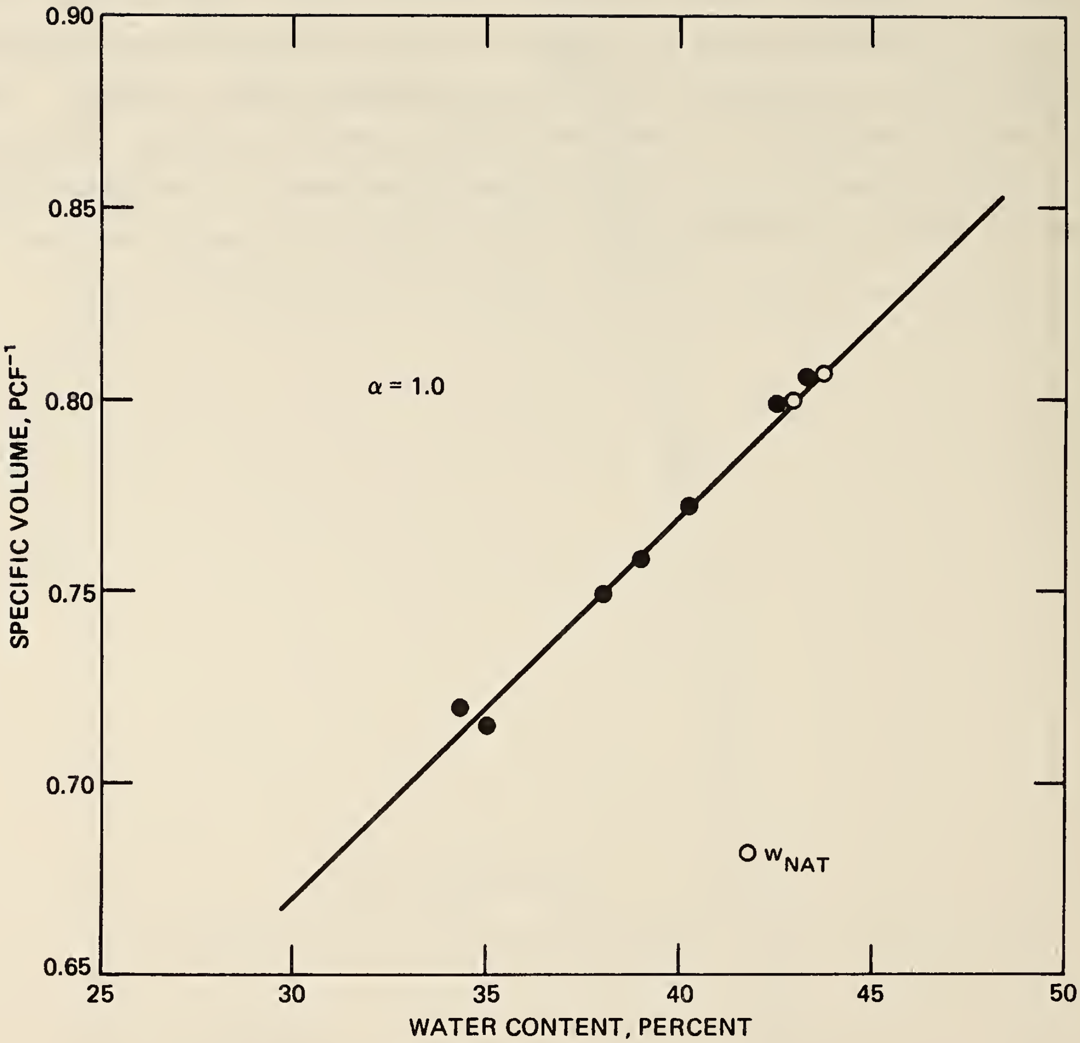

Figure 4. Typical specific volume versus water content relationship

APPENDIX VIIIA:

SWELL AND SWELL PRESSURE TESTS

1. INTRODUCTION. Swell is the process of imbibing available moisture to cause an increase in soil volume until the pore water pressure increases to an equilibrium determined by the environment. The amount of swell to satisfy the new pore pressure equilibrium depends on the magnitude of the vertical loading and soil properties that include soil composition, natural water content and density, and soil structure. The rate of swell depends on the coefficient of permeability (hydraulic conductivity), thickness, and soil properties. Soils that are more likely to swell appreciably include clays and clay shales with plasticity indices greater than 25, liquid limits greater than 40, and natural water contents near the plastic limit or less.

The presence of capillary stress or negative pore water pressure arising from molecular forces in swelling soils causes available moisture to be absorbed. The vertical confining pressure required to prevent volume expansion from absorbed moisture is defined as the swell pressure.

The swell and swell pressure are generally determined in the laboratory with the one-dimensional consolidometer (Appendix VIII, CONSOLIDATION TEST). Swell is determined by subjecting the laterally confined soil specimen to a constant vertical pressure and by giving both the top and bottom of the specimen access to free water (usually distilled) to cause swell. The swell pressure is determined by subjecting the laterally confined soil specimen to increasing vertical pressures, following inundation, to prevent swell.

2. APPARATUS, CALIBRATION OF EQUIPMENT, AND PREPARATION OF SPECIMENS. The apparatus is essentially the same as that listed in paragraph 2, Appendix VIII, CONSOLIDATION TEST. Smoothly ground porous stones should be used in the consolidometer to minimize seating displacements. Filter

paper should not be used because it is compressible and contributes to displacements. The equipment is calibrated and the sample prepared in the same manner as described for the consolidation test.

3. PROCEDURE.

a. Swell Test. (1) Record all identifying information for the specimen, such as project number, boring number, and other pertinent data, on the data sheet (Plate VIII-1, p VIII-18, is a suggested form); note any difficulties encountered in preparation of the specimen. Measure and record the height and cross-sectional area of the specimen. Record weight of specimen ring and glass plates. After the specimen is prepared, record the weight of the specimen plus tare (ring and glass plates), and from the soil trimmings obtain 200 g of material for specific gravity and water content determinations. Record the weight of the material used for the water content determination on the data sheet.

(2) Fit an air-dried, smoothly ground porous stone into the base of a dry consolidometer. Place the ring with the specimen therein on top of the porous stone. If the fixed-ring consolidometer is used, secure the ring to the base by means of clamps and screws.

(3) Place the top air-dried, smoothly ground porous stone and loading plate in position. The inside of the reservoir should be moistened to promote a high-humidity environment. The reservoir and loading plate should subsequently be covered with a sheet of impervious material such as plastic film or moist paper towel to inhibit loss of moisture.

(4) Place the consolidometer containing the specimen in the loading device.

(5) Attach the dial indicator support to the consolidometer, and adjust it so that the stem of the dial indicator is centered with respect to the specimen. Adjust the dial indicator to allow for both swell and consolidation measurements.

(6) Adjust the loading device until it just makes contact

with the specimen. The seating load should not exceed about 0.01 ton per sq ft.

(7) Read the dial indicator and record the reading on a data sheet (Plate VIII-2, p VIII-19, is a suggested form). This is the initial reading of the dial indicator.

(8) Depending on the particular design considerations, a specific load (e.g. overburden plus design load) is applied to the specimen. After a period of at least 5 min but less than 30 min (to avoid shrinkage from drying), record the dial reading on the data sheet (Plate VIII-2) and inundate the specimen.

(9) Inundate the specimen by filling the reservoir, within the inundation ring that surrounds the specimen, with water (distilled, tap, or field pore water, actual or reconstituted). Cover with the plastic film, and moist paper towel or equivalent. If a fixed-ring device is used, a low head of water should be applied to the base of the specimen and maintained during the test by means of the sandpipe.

(10) Observe and record on the data sheet (Plate VIII-2) the deformation as determined from dial indicator readings after various elapsed times. Readings at 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 min, and 1, 2, 4, 8, 24, 48, and 72 hr are usually satisfactory. A timing device should be located near the consolidometer to ensure accurately timed measurements. Allow the load increment to remain on the specimen until it is determined that the primary swell is completed. Time to complete the primary swell of heavy clay soils and clay shales often requires three or more days. Plot the dial reading versus time data on a semilogarithmic plot as shown in Plate VIIIA-1 to ascertain the completion of primary swell. The completion of primary swell is arbitrarily defined as the intersection of the tangent to the curve at the point of inflection, with the tangent to the straight-line portion representing a secondary time effect as shown in Figure 1. This is similar to the procedure in paragraph 5j, Appendix VIII, CONSOLODATION TEST.

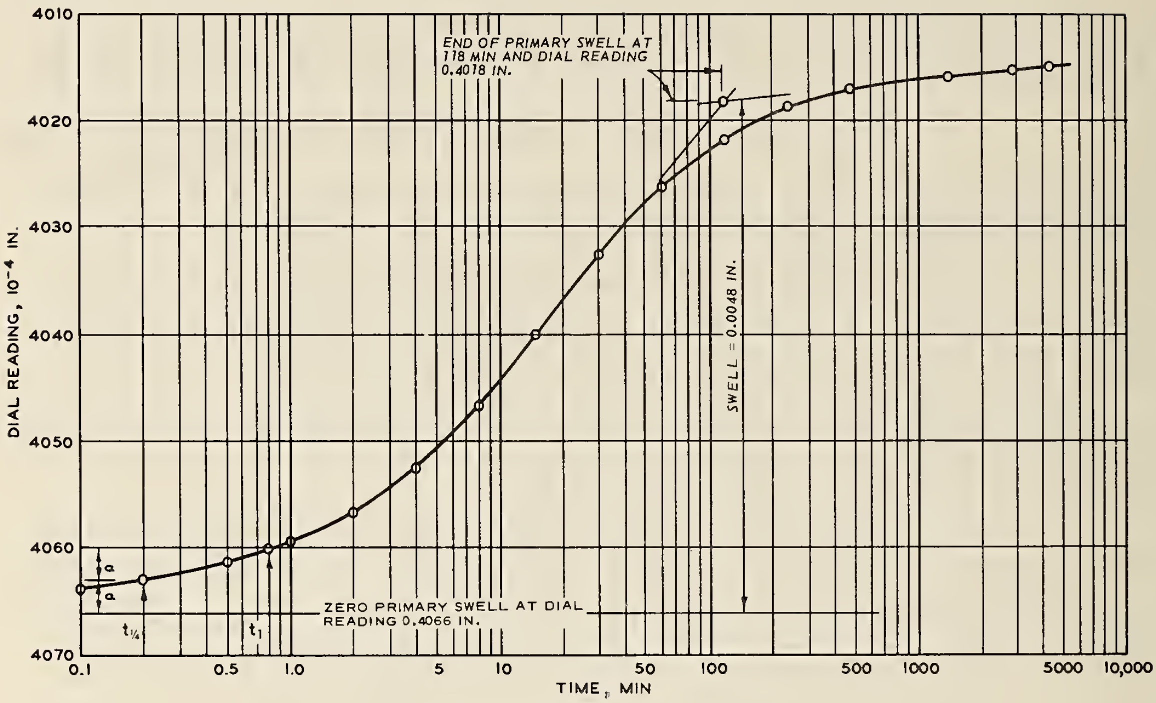

Figure 1. Time-swell curve

(11) Although a falling-head permeability test may be performed at this point of the swell test (see paragraph 8, Appendix VII, PERMEABILITY TESTS), the specimen may not be fully saturated and the permeability results consequently affected. After primary swell is complete,† the load should be removed in decrements according to the procedure in paragraphs 5l and 5m, Appendix VIII, CONSOLIDATION TEST. The final load should be the seating load.

b. Swell Pressure Test. The procedure of this test is identical with the preceding swell test through (9). Following (9), increments

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† After primary swell is complete, the loading pressure may be increased to consolidate the specimen until the void ratio is less than the initial void ratio under the overburden pressure; thereafter, loads may subsequently be removed to determine rebound characteristics. This procedure permits an alternative measure of the swell pressure, defined as the total pressure required to reduce the void ratio to the initial void ratio.

of load are applied as needed to prevent swell. Variations from the dial reading at the time the specimen is inundated with water should be kept preferably within 0.0002 in. and not more than 0.0005 in. Following 24 hr after the specimen exhibits no further tendency to swell, a falling-head permeability test may be performed (see paragraph 8, Appendix VII, PERMEABILITY TESTS) and the final load (which is the swell pressure) should be removed in decrements according to the procedure in paragraphs 5l and 5m, Appendix VIII, CONSOLIDATION TEST. The final load should be the seating load.

4. COMPUTATIONS. The computations for the swell tests are similar to those presented in paragraph 6, Appendix VIII, CONSOLIDATION TEST.

5. PRESENTATION OF RESULTS. The results of the swell tests shall be summarized on report forms, Plates VIIIA-1 and VIIIA-2. The data shall be shown graphically in terms of time-swell curves on the form shown as Plate VIIIA-1 and in terms of void ratio-pressure curves on the form shown as Plate VIIIA-2. To obtain the void ratio-pressure curve, the void ratio, e, is plotted on the arithmetic scale (ordinate) and the corresponding pressure, p, in tons per square foot on the logarithmic scale (abscissa) as shown in Figures 2 and 3. The overburden pressure, p_o, swell pressure, p_s, swell index, C_s, and swell at p_o, $\Delta H/H$, shall be determined and shown on the report form (Plate VIIIA-2). The determination of the overburden pressure is normally made by design engineers.

The swell pressure, p_s, is determined as in Figure 3. The swell pressure by the alternative definition in the footnote of paragraph 3 may also be determined from the results of the swell test (Fig. 2) and recorded on Plate VIIIA-2. The initial void ratio is the void ratio determined following placement of the overburden pressure.

The swell index is defined by the equation

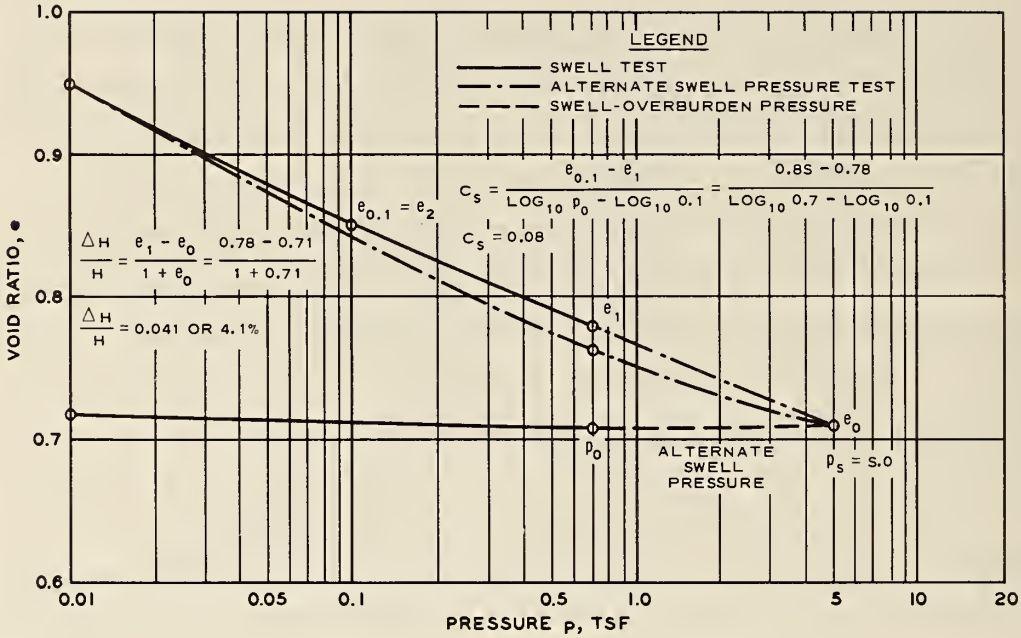


Figure 2. Void ratio-pressure curve of swell test

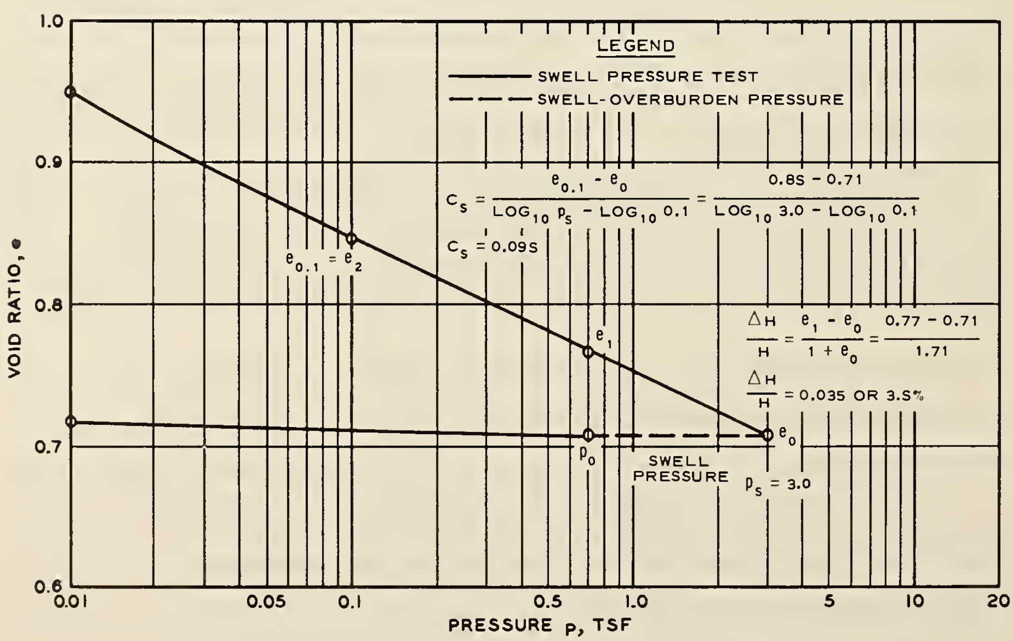


Figure 3. Void ratio-pressure curve of swell pressure test

$$C_s = \frac{e_2 - e_1}{\log_{10} p_1 - \log_{10} p_2}$$

where p_1 and p_2 are selected pressures from a straight-line portion of void ratio-pressure rebound curve, and e_1 and e_2 are the corresponding void ratios. The swell index is a measure of the ability of the soil to swell. Example computations of C_s are shown in Figures 2 and 3.

The swell is defined by the equation

$$\frac{\Delta H}{H} = \frac{e_1 - e_0}{1 + e_0}$$

where e_1 is the void ratio following swell, and e_0 is the void ratio prior to swell. Example computations are shown in Figures 2 and 3 for swell at the overburden pressure p_o .

If permeability tests are performed in conjunction with the swell tests (see Appendix VII, PERMEABILITY TESTS), the coefficient of permeability determined for each void ratio during rebound shall also be plotted in the form shown as Plate VIIIA-2.

A brief description of undisturbed specimens should be given on the report form. The description should include color, approximate consistency, and any unusual features (such as stratification, fissures, shells, roots, sand pockets, etc.). For compacted specimens, give the method of compaction used and the relation to maximum density and optimum water content.

6. POSSIBLE ERRORS. In addition to the possible errors discussed in paragraph 8, Appendix VIII, CONSOLIDATION TEST, the following may also cause inaccurate determination of swelling characteristics:

a. Displacements caused by seating of the specimen against the surface of the porous stones may be significant, especially if swell displacements and loading pressures are small. Thus, smoothly ground porous stones are recommended.

b. Filter paper is highly compressible and contributes to the observed displacements and hysteresis in displacements on loading and rebound. Filter paper is consequently not recommended.

c. The compressibility characteristics of the consolidometer and the test procedures influence the swell pressure results. Because very small expansions in volume greatly relieve swell pressures, the stiffness of the consolidometer should be as large as possible, and variations in displacements that occur during determination of the swell pressure should be as small as possible.

d. Swelling characteristics determined by consolidometer swell tests for the purpose of predicting heave of foundation and compacted soils are not representations of many field conditions because:

(1) Lateral swell and lateral confining pressures are not simulated.

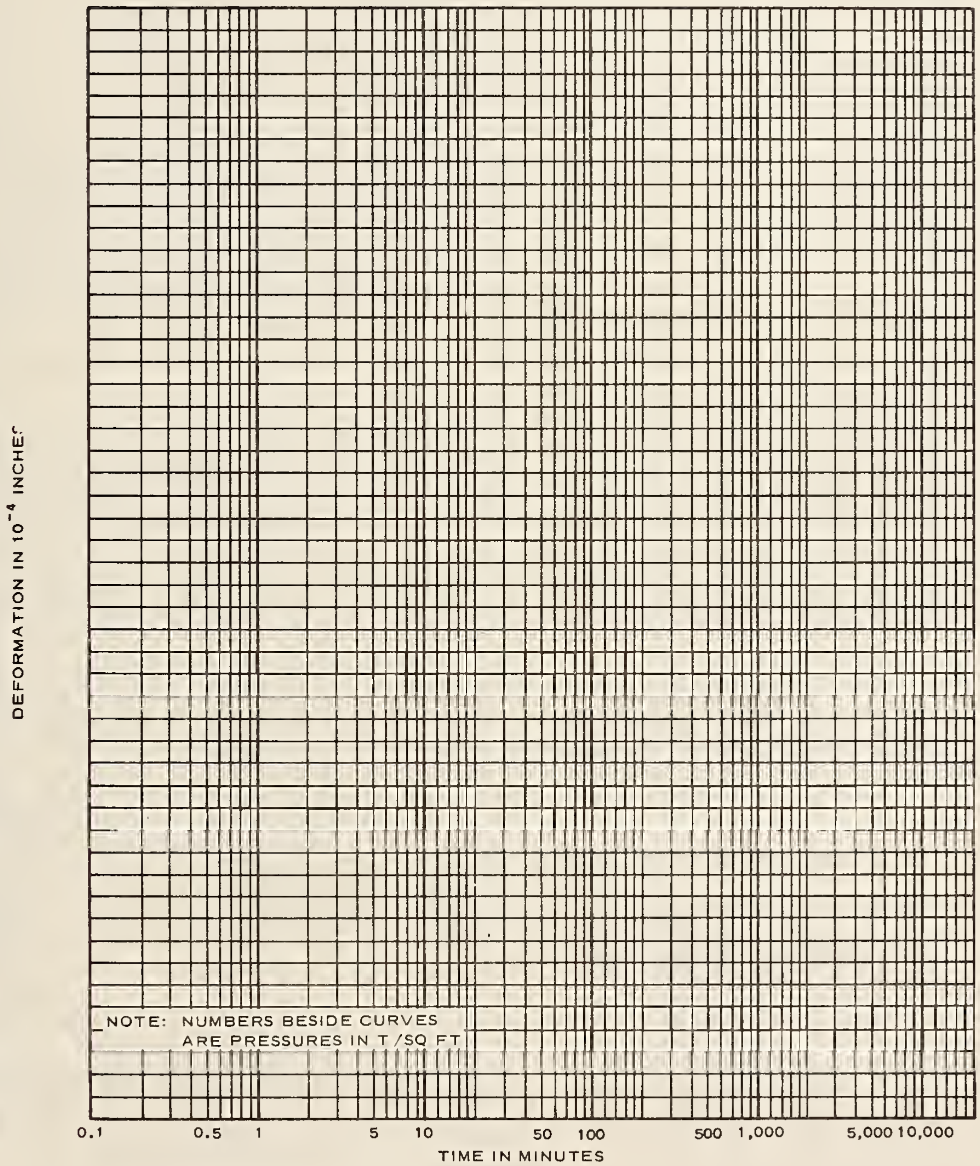
(2) The actual availability of water to the foundation soils may be cyclic or intermittent. Field swell usually occurs under constant overburden pressure depending on the availability of water. The swell index, in contrast, is evaluated by observing swell due to decreases in overburden pressure while the soil specimen is inundated with water.

(3) Rates of swell indicated by swell tests are not reliable indicators of field rates of swell due to fissures in the mass soil and inadequate simulation of the actual availability of water to the soil.

(4) Secondary or long-term swell, which is not evaluated by these test procedures, may be significant for some clays and clay shales. These soils may not be fully saturated at the conclusion of the swell test.

(5) Chemical content of the inundating water affects results; e.g., when testing shales, distilled water may give radically different results than natural or reconstituted pore water.

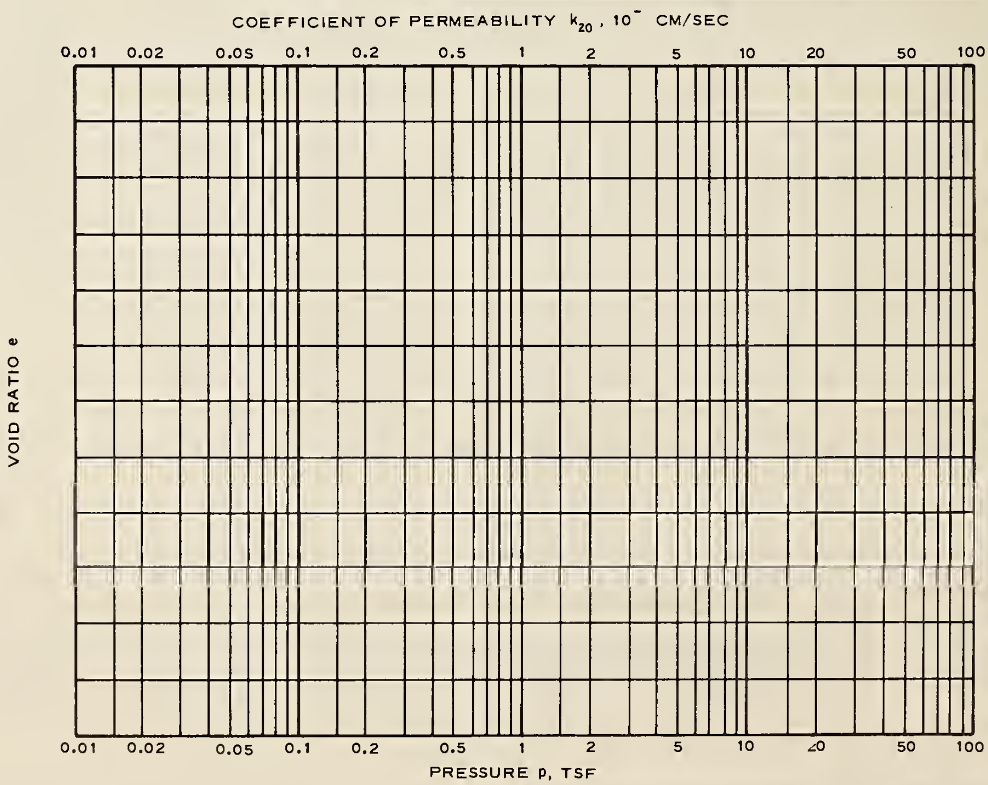
SWELL TEST-TIME CURVES (TRANSLUCENT)		Date
Project		
Boring No.	Sample No.	Depth El



ENG FORM 1 MAY 78 4663

PLATE 1

SWELL TEST REPORT				Date
Project				
Boring No.	Sample No.		Depth El	
Type of Specimen		Type of Swell Test		
Diam in.	Ht in.	Before Test	After Test	
Overburden Pressure, P_o TSF		Water Content, w_o	w_o	
Swell Pressure, p_s TSF		Void Ratio, e_o at p_o %	e_f	
Swell Index, C_s		Saturation, S_o %	S_f	
Swell at p_o, $\frac{\Delta H}{H}$ %		Dry Density, γ_d lb/ft^3		
Classification		k_{20} at p_o = x 10^- cm/sec		
LL	G_s			
PL	D_{10}			
Remarks				



ENG FORM 1 MAY 78 4664

PLATE 2

APPENDIX C
(from Reference 51)

Test Method Tex-124-E
Rev: January 1, 1974

Texas Highway Department

Materials and Tests Division

METHOD FOR DETERMINING THE POTENTIAL VERTICAL RISE, PVR

Scope

This procedure provides a means for the determination of the PVR in a soil strata, such as may be encountered in the placement of a roadway, bridge, or building foundation. Many imperfections in highway structures could be prevented if vertical rise could be avoided or predicted with sufficient accuracy that their effects could be anticipated and minimized.

Definitions

1. Potential vertical rise, PVR, expressed in inches, is defined as the latent or potential ability of a soil material, at a given density, moisture and loading condition, when exposed to capillary or surface water, to swell and thereby increase the elevation of its upper surface along with anything resting on it.

2. Liquid limit, plasticity index, etc.: These terms are often used and are tested as set out in other Texas Test Methods.

3. Overburden is the layer or layers of soil above the layer being investigated. For example, a clay layer covered with ten feet of sand would have ten feet of overburden on it.

4. Layer: The term layer denotes a horizontal soil structure of uniform or near uniform material. When material changes due to moisture, density, or composition, then a new layer is considered.

5. Loading: This term denotes the unit load from both the structure and overburden of each layer involved.

Apparatus

1. Apparatus listed in Texas Test Methods as follows: Tex-101-E, Part I, Tex-103-E, Tex-104-E, Tex-105-E, and Tex-106-E.

2. Supply of paraffin, small cutting knives, etc., for preserving cores, if taken.

3. Sampling device, core drilling rig equipped to take disturbed or undisturbed samples of the material in place.

Note: Undisturbed cores are not absolutely necessary if an approximation of the wet density is known.

Test Record Forms

1. Record soil constants on Form 476-A.

2. Drilling Report and Log Form 513.

3. Field moisture determination forms.

Sampling of Materials

Exploration and sampling is accomplished in accordance with the Foundation Exploration and Design Manual of the Bridge Division, except that greater emphasis must be placed on sampling of top strata. Determine by core or auger borings the soil strata layering to a depth of thirty (30) feet in most cases, and as much as fifty (50) feet when very highly expansive clays are encountered. In some instances, the presence of rock, gravel, or sand substrata will eliminate the necessity for drilling a large number of deep exploration holes. In accomplishing sampling, all holes should be logged and moisture contents determined.

Thicknesses of soil layers, especially clay layers, existing below the structure should be determined. In the case of massive clay layers, the maximum depth to investigate will depend on the position and amount of load proposed and the expansive characteristics of the clay. Secure cores or cuttings to represent these layers. Record data in Table I.

PVR Determination Procedure

1. It is necessary to know the moisture content of each layer sampled. If cuttings only were taken during sampling, then the moisture samples must have been secured at that time and the moisture contents of each layer determined according to Test Method Tex-103-E. If core samples were taken and paraffined up for moisture preservation, then samples may be taken from the cores subsequently for this determination. It is highly preferable to take moisture samples for each layer at the time of sampling, regardless of whether cores or cuttings were taken during sampling.

2. In the case of core sampling, determine the wet densities of the cores representative of each swelling layer. For cores, trim cores into right circular cylinders using knives or other convenient hand tools. Determine the height and diameter and calculate the volume of the core in cubic feet.

$$\text{Volume of core} = \frac{\pi D^2 h}{4 \times 1728}$$

Where: D = diameter in inches

h = height in inches

Weigh the wet core in pounds to the nearest estimated 0.001 pounds and determine the wet density by dividing the wet weight, just obtained, by the volume of the core in cubic feet. When cuttings only are taken during sampling, then use a wet density of 125 pounds per cubic foot, which is usually a reasonable value.

Note: Other accepted methods for determining density of cores, such as set forth by paraffin coatings in Test Method Tex-207-F, may be used, if desired.

3. From representative portions of the cuttings or cores, determine the liquid limit, P. I., and percent soil binder in the soil layers. Record these test results on the appropriate forms given in Test Method Tex-104-E, 105-E, 106-E, and Tex-101-E, Part I, and on Table II.

4. Beginning with the top layer at the surface of the ground from the logging data (from Table I), start compilation of Table II. Determine whether the layers are "wet", "dry", or average.

Note: It has been determined that .2 LL + 9 is the "dry" condition from which little shrinkage is experienced, but where volumetric swell potential is greatest. It is the minimum moisture content swelling clays usually dry to.

0.47 LL + 2, or "wet" condition, corresponds to the maximum capillary absorption by laboratory tests on specimens molded at optimum moisture and surcharged with 1 psi load. This is also analogous to moisture contents found beneath old pavements and other lightweight structures. This is the "optimum" condition.

Consider the layer "average" moisture if the moisture content is closer to the average of the "wet" or "dry" conditions.

5. Examine the test record forms and enter the percent soil binder (% - No. 40) and the P. I. or the layers.

6. Using Figure 1 and the wet, dry, or average moisture condition, find the P. I. of the first soil layer on the abscissa. Move vertically upwards to the appropriate swell line (dry, average or wet) and read the percent volumetric change on the ordinate. This percent volumetric change was determined for 1 psi surcharge.

Note: The PVR vs. Load Curves in Figures 2 and 3 are for free swelling clays under no load and are based on a wet density of soil of 125 pounds per cubic foot. In order to use the curves in Figures 2 and 3, it has been determined that under the conditions of free swell and the percent volumetric swell at 1 psi surcharge given in Figure 1, the following relationship exists:

% Vol. Swell @ No Load =
(% Vol. Swell @ 1 psi) (1.07) + 2.6

Example: From PI chart (Fig.1) the swell at 1 psi = 10

% at no load or free swell
= 10 (1.07) + 2.6
= 10.7 + 2.6 = 13.3

These curves may have to be penciled in on Figures 2 or 3 for accurate readings.

7. In calculating the potential vertical rise, it is convenient or preferable to use two foot elements or layers provided the moisture contents and the log of the hole will permit. The use of two foot layers and the assumption of 125 pounds per cubic foot wet density, which is usually a reasonable wet density, makes the tabulation simpler. The modification caused by using 125 pounds per cubic foot rather than 144 pounds, for 1 psi per foot, has already been incorporated into the curves on Figures 2 and 3. Where wet densities vary from 125 pounds per cubic foot and greater accuracy is desired a modification factor should be applied to that layer equivalent to 125 divided by the actual wet density.

Note: In the two foot layer at the surface, the "average" load in the layer is 1 psi; like wise, in the 2 to 4 foot layer, the psi load is 2 psi for the top two feet plus one half of the 2 to 4 foot layer or 3 psi total. Therefore, the average load in any two foot layer is the average depth of the layer. (Subject to the correction factor as described above.)

8. Using the percent - No.40 column, all PVR subsequently determined shall be modified as follows:

a. Use zero swell where the % - No. 40 is less than 25 percent.

b. Multiply the swell obtained for the layer by the percent - No. 40 when the percentage exceeds 25%.

9. Then, using Figure 1, determine the percent volumetric swell in the first layer. (0-2 feet) Since this swell is determined using 1 psi surcharge it must be modified for free swell, or no surcharge, as given in the note after paragraph 6. Using Figures 2 or 3, and the percent free swell curve, just determined, begin to compile the swell in the layer.

a. In the 0-2 foot layer read the ordinate (PVR) at 1 psi load from the swell curve and record on Table II as "bottom of layer".

b. From the curve read the "top of the layer" load, of zero in the case of this layer, and record on Table II.

c. The difference in these two readings is the PVR in the first two foot layer, subject to modification for density correction from paragraph 7, above, and for soil binder (- No. 40) correction from paragraph 8, above.

10. Next, take the 2-4 foot layer and determine the percent volumetric swell by modifying the value determined from Figure 1. On this percent volumetric swell curve, or a sketched in penciled curve where the line is not actually on Figs. 2 or 3, read the PVR on the ordinate corresponding to 3 psi (bottom of layer) and record on Table II. Read the ordinate corresponding to 1 psi (top of layer) from the same curve and record. The difference in the two readings is the swell in the 2-4 foot layer subject to any density or soil binder (-No. 40) modifications.

11. Continue determination of PVR in each two foot layer until each swelling layer has been loaded out as determined by the curves on Figures 2 and 3 leveling out horizontally and indicated by no difference when PVR is read from that curve. (Actually the swell is negligible or zero anywhere beyond the end of any given curve as shown on these two figures.) Thicker layers may be used in this calculation where they consist of uniform soil having similar PI and moisture contents.

12. Check each layer for modifications for density factor and soil binder.

13. Add the PVR in all layers to obtain the total PVR for the site.

Note: Table II has been calculated for no loading due to the structure. When unit loadings due to the structures are known, then simply add it in "Average Load, psi" and increase each figure in the column by the amount of the structure unit load, but note that the swell will be reduced because of increased loading.

Reporting Test Results

To report the test results, submit a copy of Table II with appropriate job and site identifications.

Note 1: Often, during design, it is necessary to estimate PVR without knowing moisture contents anticipated at time of construction. In cases of this kind, the design and planning of the job should influence the choice of line on Figure 1 to be selected for use. If the project exists in an arid to semiarid climate and the plans and specifications do not provide for moisture-density control nor preservation of moisture it is suggested that the line for .2 LL + 9 be used. If the plans and specifications do require moisture-density control and moisture preservation, the average line may be used.

In the high rainfall areas, the average line may be used where moisture preservation is provided for, but if moisture-density control and moisture preservation are provided for, the lower line (.47 LL + 2) on Figure 1 may be used.

The term "Moisture Preservation" refers to the use of "Blanket Sections" with wide shoulders consisting of granular materials, stabilized soils, or where asphalt membranes are applied for this purpose.

Note 2: The determination of PVR in deep cut sections or deep side hill cuts present a special case from this test method. In the case of these two conditions, the material is surcharged in such a manner that the movement from swell is mostly in one direction, and in some high rainfall areas could be greater than that obtained by use of these procedures.

Note 3: When layers of expansive clays of less than two feet exist (Example 4-4.6 ft.) it is preferable to enter the abscissa on the proper swell curve at 4 and 4.6, respectively, and use the difference in the respective ordinate readings as the unmodified swell in the 0.6 foot thick layer.

Note 4: At optimum conditions the following relationships are valid from Figure 1:

1. Percent Volumetric swell at 1 psi surcharge = .217 (P.I.) - 2.9

2. Percent free swell = .232 (P.I.) - .5

Note 5: For average conditions up to Plasticity indices of about 60, the following relationships are valid from Figure 1:

1. Percent Volumetric swell at 1 psi surcharge = .294 (P.I.) - 2.9

2. Percent free swell = .314 (P.I.) - .5

TABLE I

Texas Highway Department
Form 573
Rev. 4-63

Sheet 1 of 1

DRILLING LOG

County Williamson — Structure Warehouse — District No. 14
Highway No. 29 — Hole No. 1 — Date April 10, 1970
Control 716-3 — Station 715 + 30 — Grd. Elev. 635
IPE — Loc. from Centerline Rt. 300 ft. Lt. — Grd. Water Elev. 595

ELEV. (FT.)	†	LOG	THD PEN. TEST NO. OF BLOWS 1st 6"	2nd 6"	DESCRIPTION OF MATERIAL	Depth	METHOD OF CORING	*
635 0						0		
					Sand, f. p grd., tan, loose		D. Bbl.	
					Clay, dark brown, wet		P. Bbl.	
					Clay, dark brown, firm		P. Bbl.	
					Clay, red and yellow, firm		P. Bbl.	
					Clay, red and yellow, soft, wet		P. Bbl.	
10					Gravel, coarse to fine, some moist yellow clay	10	*	
20					Clay, yellow, soft	20	P. Bbl.	
					Clay, yellow, firm		P. Bbl.	
30						30		
600								
40					P. Bbl. Push Barrel	40		
50					*No good cores obtained due to large percentage of gravel.	50		
60						60		
70						70		

*REMARKS:

Driller John Doe — Logger John Smith — Title Engineering Assistant II

†Indicate each foot by shading for core recovery, leaving blank for no core recovery, and crossing (X) for undisturbed laboratory samples taken.

NOTE: Refer to Foundation Exploration and Design Manual for directions in filling out this form. For distribution, forward one copy to the Bridge Division (D-5) and one copy to the Materials and Tests Division (D-9) if samples are submitted and make a note of same on D-5 copy.

TABLE II

Depth, ft.	Avg. Load PSI	L.L.	Dry .2 LL + 9	Wet .47 LL + 2	% Mois-ture	Dry Avg. Wet	% - No. 40	P.I.	% Vol. Swell (Fig.1)	% Free Swell	PVR, In. Top of Layer	PVR, In. Bottom of Layer	Diff.	Mod. -40 Factor	Mod. Density Factor*	PVR in Layer In.
0 - 2	1	21	----	----	3.1	Dry	100	4	0.0	0	0.00	0.00	0.00	1.00	1.00	0.00
2 - 4	3	60	21.0	30.2	29.7	Wet	100	38	5.5	8.5	0.41	0.88	0.47	1.00	1.00	0.47
4 - 6	5	60	21.0	30.2	20.9	Dry	100	38	11.0	14.5	1.55	2.20	0.65	1.00	1.00	0.65
6 - 8	7	75	24.0	37.3	24.4	Dry	100	45	13.5	17.0	2.81	3.41	0.60***	1.00	1.00	0.60
8 - 10	9	75	24.0	37.3	36.5	Wet	100	45	7.0	10.0	1.69	1.85	0.16	1.00	1.00	0.16
10 - 12	11	65	22.0	32.6	8.5	Wet	15	40			Less than 25% - 40			0.00	1.00	0.00
12 - 14	13	65	22.0	32.6	8.5	Wet	15	40			Less than 25% - 40			0.00	1.00	0.00
14 - 16	15	65	22.0	32.6	8.5	Wet	15	40			Less than 25% - 40			0.00	1.00	0.00
16 - 18	17	65	22.0	32.6	8.5	Wet	15	40			Less than 25% - 40			0.00	1.00	0.00
18 - 20	19	85	26.0	42.0	41.5	Wet	100	60	10.2	13.5	3.54	3.62	0.08	1.00	1.00	0.08
20 - 22	21	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	4.88	5.00	0.12	1.00	1.00	0.12
22 - 24	23	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	5.00	5.11	0.11	1.00	1.00	0.11
24 - 26	25	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	5.11	5.20	0.09	1.00	1.00	0.09
26 - 28	27	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	5.20	5.27	0.07	1.00	1.00	0.07
28 - 30	29	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	5.27	5.33	0.06	1.00	1.00	0.06
30 - 32	31	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	5.33	5.34	0.01	1.00	1.00	0.01
												Total PVR =2.42"				
**20 - 32	19 to 31	80	25.0	39.6	33.9	Avg.	100	54	12.6	16.0	4.88	5.34	0.46	1.00	1.00	0.46

* 125 pounds per cubic foot wet density assumed for all layers. When greater accuracy is desired use $\dfrac{125}{\text{Actual wet density of soil, pcf}}$ as the modifier.

** Note: Since the 12 foot layer from 20 to 32 feet is uniform, the PVR may be determined in one reading by using the "top of the layer" as 19 psi (as in 2 ft. layers) and reading the "bottom of the layer" at 31 psi load as in the 30 to 32 ft. layer. Readings of 4.88"and 5.34", respectively, or a difference of 0.46", will be obtained which is a summation of increments (difference) as shown above for the bottom 12 feet.

When layers of expansive clays of less than two feet exist (Example 4-4.6 ft.) it is preferable to enter the abscissa on the proper swell curve at 4 and 4.6, respectively, and use the difference in the respective ordinate readings as the unmodified swell in the 0.6 foot thick layer.

***See example on Figure 2.

INTERRELATIONSHIP OF P.I. AND VOLUME CHANGE
(Specimens Subjected to Swell Under Avg. of 1 Psi Surcharge)

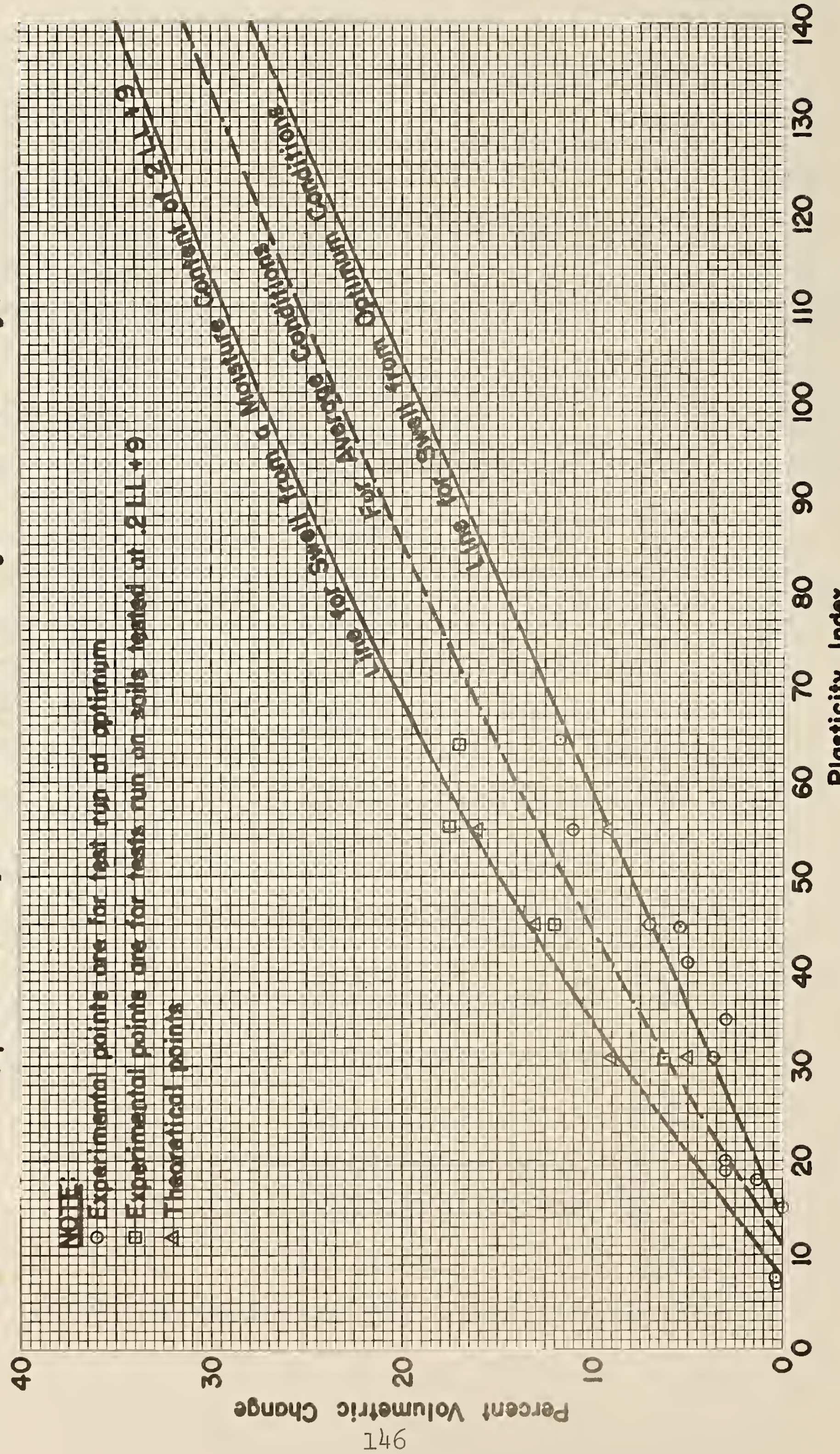


Fig. 1

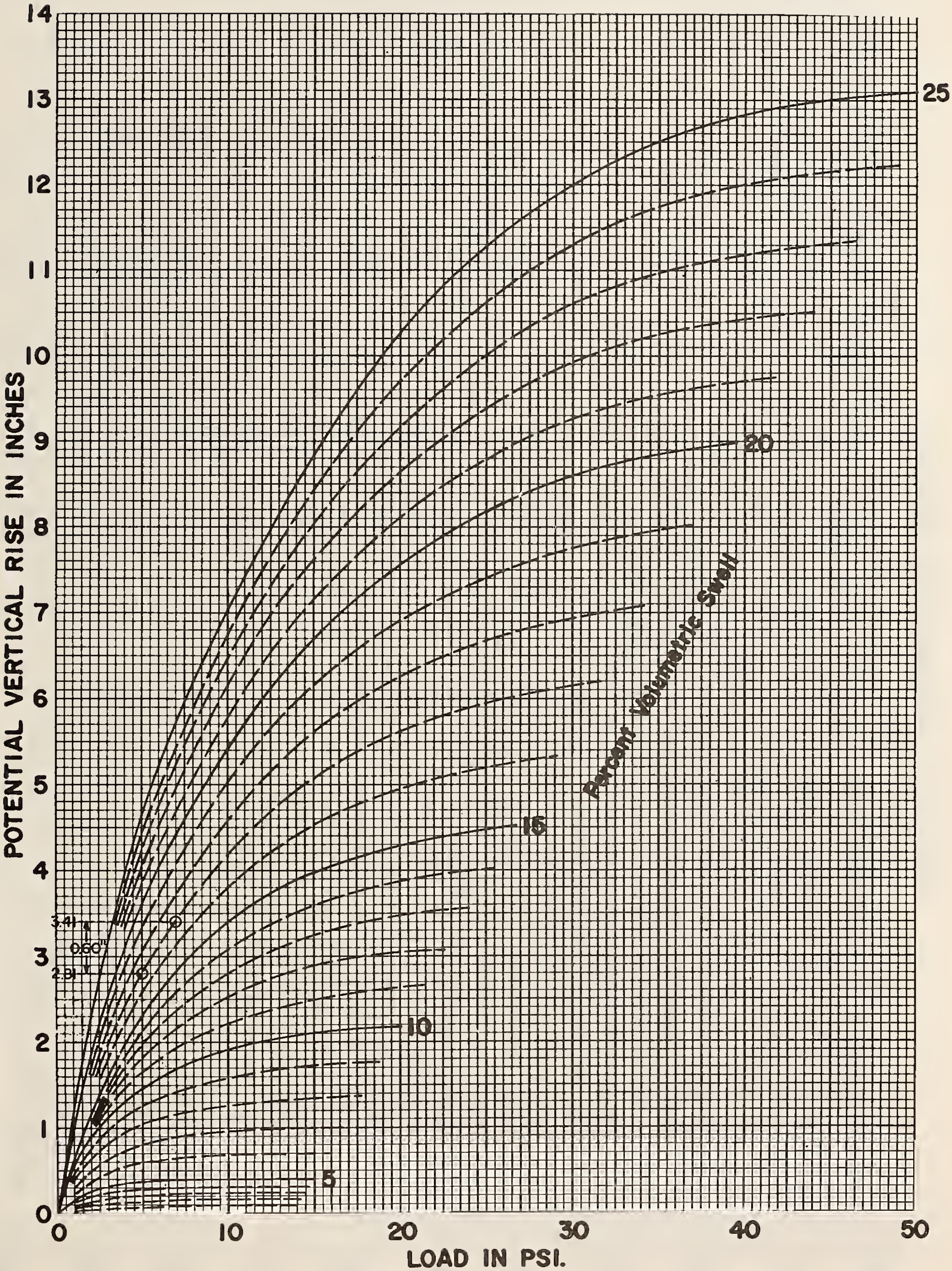

POTENTIAL VERTICAL RISE IN INCHES
LOAD IN PSI.
Percent Volumetric Swell
25
20
15
10
5
3.41
0.60"
2.81
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
0
10
20
30
40
50

Figure 2

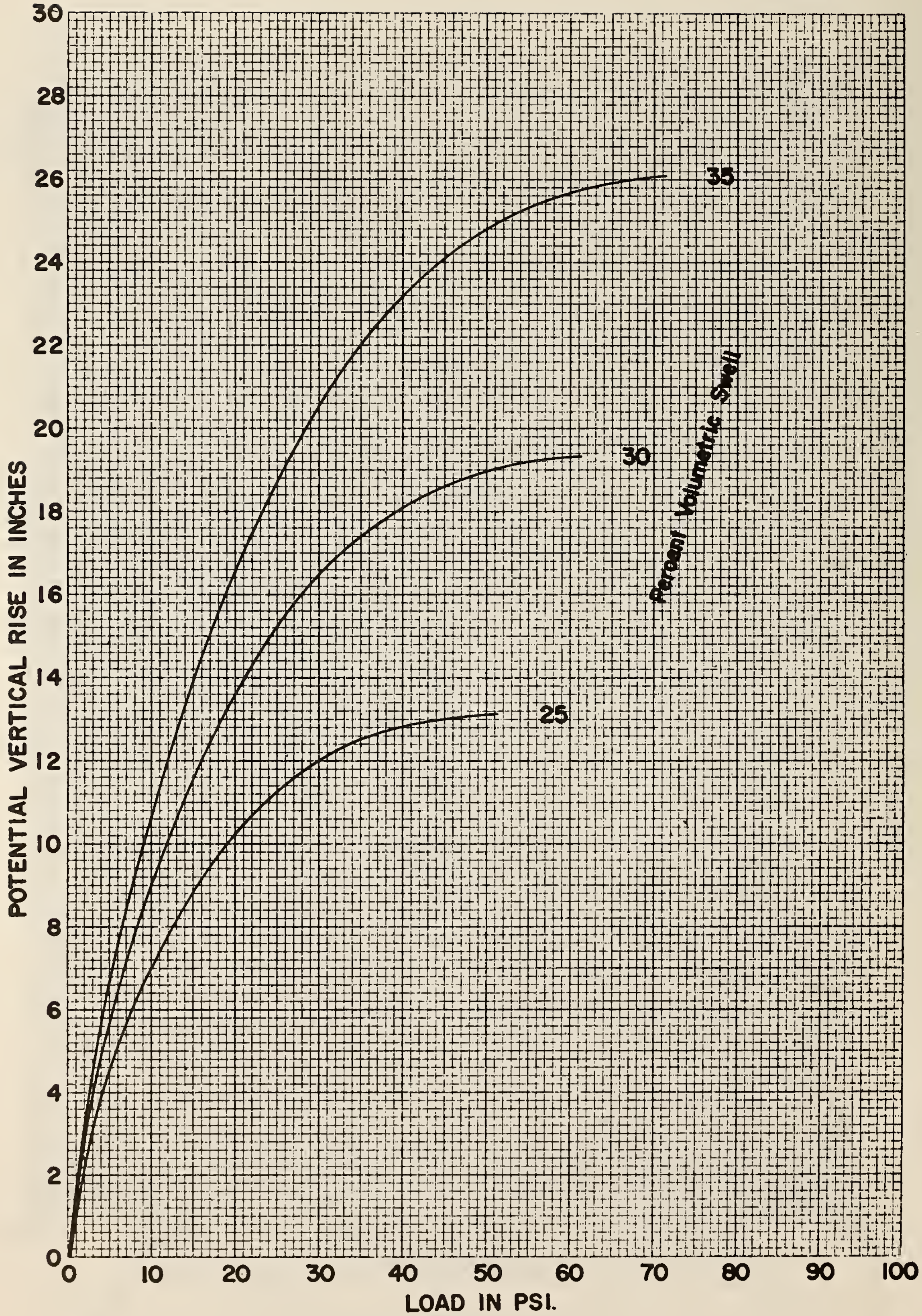

POTENTIAL VERTICAL RISE IN INCHES
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2
0
0
10
20
30
40
50
60
70
80
90
100
LOAD IN PSI.
35
30
25
Percent Volumetric Swell

Figure 3

APPENDIX D

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APPENDIX E

GUIDELINES FOR FIELD MONITORING DATA FOR EVALUATING THE EFFECTIVENESS OF TREATMENT ALTERNATIVES

1. Evaluation of the effectiveness of treatment alternatives for minimization of detrimental volume change of expansive soils in highway subgrades is oftentimes achieved by simple, direct observations by State Transportation Agency personnel rather than systematic engineering measurements. With construction costs continuing to climb and the need to try treatment alternatives different from those previously used by a given State Transportation Agency, it is imperative that treatment alternatives, whether applied through routine or experimental construction, be evaluated on a logical engineering basis so that the decision on the effectiveness of the treatment is based on comparative data instead of personal opinion. Also, treatment alternatives evaluated using sound engineering procedures provide valuable data to other organizations considering the treatment alternatives (i.e., enhances technology transfer). The need for systematic engineering evaluation of treatment alternatives requires that some minimum standards for collection of field monitoring data be established. The following paragraphs describe in general terms some minimum standards for field monitoring data.

2. The measure of the effectiveness of a treatment alternative should be based on the extent the alternative reduces or minimizes subgrade moisture variations and the associated volume changes, whether by altering the properties of the subgrade soil, preventing access of water, or achieving a new equilibrium moisture content. An equally important measure of treatment alternative effectiveness is the ride quality as reflected in the measured performance parameters, since the desired end result is a smooth, comfortable ride for a majority of the design life. Therefore, to systematically measure the effectiveness of treatment alternatives, at least the first three, preferably all four, of the following items must be measured:

   a. Subgrade moisture conditions.

   b. Subgrade strength.

c. Pavement performance.

d. Surface deformation.

For all experimental construction, comparative measurements between the treatment and routine construction should be made. In other words, test and control conditions should always be measured.

3. The measurement of subgrade moisture conditions, whether by nuclear depth gages or direct sampling, should consider the amount and rate of variation between the treated and untreated sections. In addition, the fluctuations with seasonal changes and the influence the treatment has on minimizing the seasonal influence should likewise be considered in the evaluation of effectiveness.

4. Subgrade strength data should be collected to assist in the validation of the influence of moisture on the pavement behavior, as well as assure the engineer that the treatment alternative has no adverse affect on the strength properties of the pavement system.

5. Pavement performance, whether measured in terms of serviceability index or surface roughness, is a more accessible and valuable measure because of the pavement management systems currently being adopted by many State Transportation Agencies. Comparisons using pavement performance data should consider differences between treated and untreated sections and the rate of deterioration of performance with time. This affords evaluation of the treatment alternative, as well as an indication of potential maintenance requirements.

6. Surface deformation, as measured by grid level surveys, is important data, but the time required to obtain the data versus its usefulness may not warrant its consideration. The usefulness of the data can be greatly enhanced through the use of statistical comparisons (i.e., mean, average difference, and variance of the measured data for treated and untreated sections) provided a large enough data base is available.

7. The guidelines presented here are simply suggestions on the type of data that should be collected and how it should be considered in the evaluation of treatment alternative effectiveness. No indication of the amount and frequency of monitoring data collection is made because these factors involve fiscal responsibilities of the State

Transportation Agencies. In addition, the type of equipment for collection of the various monitoring data varies from one State Transportation Agency to another. It is recommended that the personnel responsible for planning of field monitoring programs consult FHWA-RD-79-49 and FHWA-RD-79-50 to become acquainted with the various types of data and frequency of collection that are available to provide a systematic engineering evaluation of treatment alternative effectiveness.

APPENDIX F

EXAMPLE SPECIFICATIONS FOR ASPHALT RUBBER MEMBRANES

FROM ARIZONA DEPARTMENT OF TRANSPORTATION
(from Reference 69)

ITEM - ASPHALT-RUBBER STRESS ABSORBING MEMBRANE (INTERLAYER) (SAMI)

The work under this item consists of placing an asphalt rubber stress absorbing interlayer across the full roadway width.

Asphalt Rubber Materials:

The asphalt shall conform to the requirements of Table 705-1 of the Supplemental Specifications for Asphalt Cement AR-1000.

The granulated rubber shall meet the following requirements:

When the mixing procedure involves the intimate contact between the hot asphalt and rubber for a period of five minutes or more, 95 percent of the granulated rubber shall pass the No. 16 mesh sieve and no more than 10 percent shall pass the No. 25 mesh sieve. Where the contact period is less than five minutes, 98 percent of the granulated rubber shall pass the No. 25 sieve. The sieves shall comply with AASHTO Designation M-92.

The specific gravity of the material shall be 1.15 ± 0.02 and shall be free of fabric, wire, or other contaminating materials, except that up to 4 percent of calcium carbonate may be included to prevent the particles from sticking together.

Mixing Asphalt and Rubber:

The material shall be intimately combined as rapidly as possible for such a time and at such a temperature that the consistency of the mix approaches that of a semi fluid material. The temperature of the asphalt shall be between 350 degrees F. and 450 degrees F.

The method and equipment for combining the asphalt and rubber shall be so designed and accessible that the engineer can readily determine the percentage, by weight, of each of the two materials being incorporated into the mixture.

The proportions of the two materials, by weight, shall be 75 percent ±2 percent asphalt and 25 percent ±2 percent granulated rubber. After the full reaction described has occurred, the mix shall be cut back with Kerosene. The amount of Kerosene used shall be 5-1/2 percent to 7-1/2 percent, by volume, of the hot asphalt-rubber composition as required for adjusting the viscosity for spraying or better "wetting" of the cover material.

The Kerosene shall have a boiling point of not less than 350 degrees F. and the temperature of the hot asphalt-rubber shall not exceed 350 degrees F. at the time of adding the Kerosene.

After reaching the proper consistency, application of the material shall proceed immediately and in no case shall the material be held at a temperature over 330 degrees F. for more than one hour after reaching the proper consistency.

Construction Details:

The existing pavement shall be cleaned in accordance with the requirements of subsection 404-3.01 of the Standard Specifications.

After cleaning and prior to the application of the membrane seal, the existing pavement surface shall be treated with a tack coat.

The hot asphalt-rubber mixture shall be applied at a minimum rate of .60 of a gallon per square yard. A rate of .75 of a gallon per square yard should be used for estimating purposes (based on 7-1/2 pounds per hot gallon). The distributor should be capable of spreading the asphalt rubber uniformly.

All transverse joints shall be made by placing building paper over the end of the previous application, and the joining application shall start on the building paper. Once the application process has progressed beyond the paper, the paper shall be disposed of as directed by the engineer.

All longitudinal joints shall be lapped approximately 4 inches.

Cover Material (Special):

Immediately after the asphalt-rubber membrane has been placed Cover Material (Special) should be applied, primarily as a blotter. The rate of application should be only the amount necessary to protect the membrane from construction equipment required for placement of the asphaltic concrete. If traffic is to be carried over the membrane it will be necessary to increase the rate of application to maintain integrity of the asphalt rubber membrane.

For estimating purposes only the rate of application should be 25 pounds per square yard (dry weight). A sample of the cover material shall be submitted for approval at least two weeks before it is to be used and the engineer will then determine the exact rate of application.

The cover material should be at least as dry as material dried in accordance with the requirements of Section 4.2 of AASHTO T 85 at the time of application.

The cover material (Special) should comply with the following gradation:

Sieve size	% Passing
3/8"	100
#4	30 - 60
#8	0 - 20
#200	0 - 4

At least 50% by weight of the material retained on the #4 sieve should have at least one rough angular surface produced by crushing.

Rolling:

The cover material shall be rolled with pneumatic tired rollers carrying a minimum of 5,000 pounds on each wheel and a minimum air pressure of 100 pounds per square inch in each tire.

Sufficient rollers shall be furnished to cover the width of the spread with one pass. It is imperative that the first pass be made immediately behind the spreader and if the spreading is stopped for any reason, the spreader shall be moved ahead so that all cover material spread may be immediately rolled. The rolling shall continue until four complete coverages have been made. Final rolling shall be completed within two hours after the application of the cover material.

Removing Loose Cover Material:

The power broom used in removing loose cover material shall be a combination air jet and rotary sweeper type.

Excess loose cover material should be removed prior to placement of the asphaltic concrete. Care should be taken to maintain the broom pressure so that only the loose material will be removed and there will be a minimum dislodgement of imbedded cover material.

Prior to placement of asphaltic concrete a tack coat should be applied if the asphalt-rubber membrane has been subjected to traffic.

Weather Limitations:

Placement of the asphalt rubber stress absorbing membrane (1) shall not be made when the ambient air temperature is less than 50 degrees F, (2) shall not be placed on other than an absolutely dry pavement, and (3) material shall not be placed if wind conditions are such that a satisfactory membrane is not being achieved.

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.