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AN EVALUATION OF METHODOLOGY FOR PREDICTION AND MINIMIZATION OF DETRIMENTAL VOLUME CHANGE OF EXPANSIVE SOILS IN HIGHWAY SUBGRADES

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FOREWORD

This report summarizes the results of a 4 1/2-year research study on expansive soils in highway subgrades. The major research results reported previously are summarized and details of the remaining tasks given. The report will be of interest to other researchers in the field and to engineers from States with significant occurrences of expansive soils.

The technical guidance resulting from this report and four previous volumes is contained in the sixth and final volume of this series.

This six-volume set presents the results of FCP Project 4D study, "An Evaluation of Methodology for Prediction and Minimization of Detrimental Volume Change of Expansive Soils in Highway Subgrades." The program was conducted by the U.S. Army Engineer Waterways Experiment Station for FHWA under Purchase Order No. 4-1-0195 during the period of June 19, 1974, to July 1, 1979.

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

Mr. Bud Brakey, Colorado Department of Highways
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

Distribution of this report will be limited to those States routinely affected by the problem, advisory group members, and other expansive soils researchers.

Charles F. Scheffey
Charles F. Scheffey
Director, Office of Research

NOTICE

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16. Abstract

This report concludes a 4-1/2-year study involving an evaluation of methodology for prediction and minimization of detrimental volume change of expansive soils in highway subgrades. The purpose of the study was to evaluate and make recommendations concerning the major aspects of expansive soil in highway subgrade problems: namely, describe (on the basis of physiographic areas) the occurrence and distribution of expansive soils, define and verify the roles of the microscale mechanisms that cause volume change, evaluate expedient methodology for identification and classification of potentially expansive soils, evaluate methodology for testing and prediction of anticipated volume change, evaluate and recommend appropriate treatment alternatives for new and existing highways, and recommend practical procedures for design and construction of new pavements and maintenance of existing pavements. Four interim reports have been published that cover the research efforts in the first three of these tasks. The Final Report summarizes the major research results reported in the interim reports and presents the details of the research efforts in the remaining tasks. Volume I presents the text and summary figures relevant to the discussion of the results of the research tasks. Volume II presents the laboratory data collected on samples from 22 field sampling sites and the monitoring data from eight field test sections located in five different states. Conclusions drawn from the research results provide: better criteria for identifying and classifying potentially expansive soils, more accurate and reliable procedures for characterizing and predicting the behavior of expansive soils, guidelines 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.

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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, MS. 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 Final 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: $K = (5/9)(F - 32) + 273.15$.

AN EVALUATION OF METHODOLOGY FOR PREDICTION AND MINIMIZATION OF DETRIMENTAL VOLUME CHANGE OF EXPANSIVE SOILS IN HIGHWAY SUBGRADES

PART I: INTRODUCTION

1. The purpose of this study was to evaluate and make recommendations concerning the major aspects of the expansive soil in highway subgrade problem; namely, describe on the basis of physiographic areas the occurrence and distribution of expansive soils, define and verify the roles of the microscale mechanisms that cause volume change, evaluate expedient methodology for identification and classification of potentially expansive soils, evaluate methodology for testing and prediction of anticipated volume change, evaluate and recommend appropriate treatment alternatives for new and existing highways, and recommend practical procedures for the design and construction of new pavements and maintenance of existing pavements. A state-of-the-art report[1] was prepared that reviewed current, as well as past, practices for each of the aspects of the expansive soil in highway subgrade problem. In addition, three subsequent reports have been published that cover the occurrence and distribution of expansive soils and shales,[2] verification of the natural microscale mechanisms,[3] and evaluation of expedient identification methodology.[4] The purpose of this report is to present the results of the research on the remaining aspects of the expansive soil in highway subgrade problem. Parts II, III, and IV of this report provide summaries of the topics previously reported to maintain a continuity in the presentation.

2. Once a potentially expansive soil has been identified and qualitatively classified, the next step in the logical design process would be to quantitatively estimate the amount of anticipated volume change, recognizing the soil properties and environmental conditions that influence volume change. In order to do this, an appropriate test must be conducted and an applicable theory used to make the estimate. Within the realm of appropriate tests, three major categories are available, namely, odometer, soil suction, and empirical. The research effort involving the testing and predicting anticipated volume change included a review of the techniques currently used (published) for quantification of expansive soil behavior, a laboratory testing program to provide data to evaluate the published techniques, and a field monitoring program to provide measured volume change data for comparisons of the various techniques. A portion of this report provides the results of the research effort.

3. After quantitatively estimating the anticipated volume change, a decision must be made as to whether to treat the subgrade material to minimize the volume change, design the pavement to withstand the volume

change, or do nothing, i.e., use standard procedures without consideration of the expansive soil problem. Besides avoiding the expansive soil, the options available for treating expansive soil subgrades include maintaining in situ soil moisture conditions through the use of membranes, increasing the in situ moisture conditions to a stable equilibrium value (i.e., prewetting) or altering (mechanically, physically, or chemically) the characteristics of the expansive soil. The research effort to establish recommendations for the use of pre- and postconstruction treatment alternatives included a threefold effort. First, appropriate tests were conducted to characterize the influence of the treatment alternative on expansive soils. Second, the laboratory data were complemented with measured field performance data to verify the characterization of treatment influence as well as provide information on the environmental conditions for which the various treatment alternatives perform best. Third, analytical techniques, particularly the prediction techniques from the previous tasks, were used to compare and evaluate the laboratory and field data and extend the trends to situations involving different environmental conditions. The results of the research provide guidelines for selecting pre- and postconstruction treatment alternatives and estimating the effectiveness of the treatments for given soil properties and site (environmental) conditions.

4. Design and construction of new pavements and maintenance of existing pavements on expansive soil subgrades require some attention to practical details that are sometimes overlooked or incorrectly applied which may adversely affect the performance of the pavement. The research effort to establish practical design, construction, and maintenance recommendations involved collection and evaluation of published information complemented with data from State Highway Agency contacts to determine the quantitative limits for the recommendations. The design and construction recommendations were categorized according to the major application; the categories used were: positive surface drainage, positive subsurface drainage, and cross section features. Likewise, the maintenance recommendations were placed in three categories, namely, improved surface drainage, improved subsurface drainage, and routine maintenance and cosmetic repair. The results of the research provide guidelines for practical recommendations that will help minimize subgrade moisture variations and volume change.

5. Throughout the research study, the major aspects of the expansive soil in highway subgrade problems were carefully studied. In most cases, adequate solutions to the problems were obtained; however in certain situations, insufficient time and data were available to fully investigate the details of the solution. In these limited cases, recommendations for future research and field experimentation were prepared.

PART II: OCCURRENCE AND DISTRIBUTION OF EXPANSIVE SOILS

Geology of Expansive Soils and Rocks

6. Expansive soils and rocks are natural earth materials that, because of certain intrinsic and extrinsic characteristics, are subject to volume change with changes in their ambient environment. The change in volume consists of expansion during water intake and shrinkage during drying. The term "expansive" is used to describe these materials; however, it should be kept in mind that shrinkage is a directly related and important phenomenon which may also be detrimental to pavements and other structures.

7. Expansive materials comprise two distinct classes: sedimentary rocks and soils. Expansive sedimentary rocks include shales, claystones, some siltstones, and other materials that are fine-grained, exhibit some degree of lithification, and possess the extrinsic and intrinsic properties that cause volume change.[1,2,5] Expansive soils may be either transported or residual. Transported expansive soils are, geologically, fine-grained, unlithified, argillaceous sediments having expansive properties. Residual expansive soils are soils having expansive properties and have originated by the in situ weathering of a parent material that may or may not have any expansive properties itself.

8. The intrinsic properties that affect the expansiveness of soils and rocks include type and amount of clay minerals, fabric, grain size distribution, cementation, and effects caused by deep burial, diagenesis, folding, and metamorphism. The clay mineralogy, grain size, and cementation are intrinsic properties that are important in affecting the volume change of both soils and rocks; whereas burial, folding, and metamorphism primarily affect the expansiveness of rocks.

9. The extrinsic properties are climate, particularly amount and distribution of rainfall; topography; depth; soil moisture; fluctuation of groundwater level; and the effects of man including compacted density, type of structure, and the drainage features related to the structure.

Mineralogy

10. The key to the distribution of expansive materials is the distribution of the clay mineral, montmorillonite. This three-layer clay mineral possesses a structural configuration and chemical makeup that permit the sorption of large quantities of water in interlayer and peripheral positions on the clay crystallite. The other clay minerals, kaolinite, illite, vermiculite, and chlorite, also exhibit some degree of expansiveness but the amount is much less than that of montmorillonite.

11. Generally, the occurrence of montmorillonite is limited to rocks or transported soils that are of Mesozoic age or younger.[7] This is explained by the fact that montmorillonite is diagenetically unstable and tends to alter with time and deep burial to other less expansive clay minerals such as illite and chlorite or mixed-layer types. Thus, the Paleozoic and older rocks that have been buried or covered for many millions of years usually contain minor amounts of this mineral. Exceptions do occur such as the Ordovician K-bentonites of the Appalachain region and various Pennsylvanian and Permian lithologic units in the mid-continent and elsewhere. In any event, the presence of montmorillonite in rocks or transported sediments older than the Mesozoic is usually limited, and the clay mineral itself generally occurs in an interlayer relationship with other clay minerals such as illite. Table 1 presents an estimate of the distribution of expansive clays, including mixed-layer types, with geologic time.[8]

Table 1

Estimates of the Percentage of Expandable Clay Present in Precambrian Through Pliocene Age Rocks (from Reference 8)

Age	Percent
Pliocene	65
Miocene	65
Oligocene	50
Eocene	40
Upper Cretaceous	40
Lower Cretaceous	20
Turassic	20
Triassic	20
Permian	40
Pennsylvanian	30
Upper Mississippian	40
Lower Mississippian	5
Devonian	5
Ordovician	15
Cambrian	~5
Precambrian	~5

12. The occurrence of montmorillonite in residual soils is controlled by the nature of the parent material and by climate. Ordinarily, climatic and drainage conditions that allow for the retention of silica and bases favor the formation of montmorillonite and other three-layer clay minerals.[9] Also, montmorillonite is the primary clay mineral constituent of residual soils formed upon parent material containing appreciable montmorillonite. The expansive effects of montmorillonite in residual soils are dependent upon the amount of

montmorillonite present, the thickness of the soil, and the elevation of the particular structure with respect to the soil profile.

Petrology

13. Petrology involves the internal relationships, texture, mineralogy, and origin of rocks or, in some cases, partially lithified sediments (transported soils).[10] Certain aspects of the petrology of expansive materials have been discussed,[1,2] including internal spatial relationships and texture.

14. Hardness, durability, and strength determine whether argillaceous earth material should be classed as rock or soil. Generally, material that requires coring and does not disintegrate in contact with water is classed as rock. Weaker material that can be sampled by push-type tubes and slakes freely in water is classed as soil. The expansive sedimentary materials studied in this project included both types, although most of them would be classed as soils by engineers. Geologists, on the other hand, would call these materials "sediments" or possibly "weakly cemented, sedimentary rocks."

15. Sediments or sedimentary rocks that exhibit expansive properties are mainly clastic and fine-grained. This categorization would include materials composed principally of silt- and clay-size particles and would exclude rocks or sediments containing appreciable particles coarser than silt size and carbonate or evaporitic components. Thus, expansive sedimentary materials would be classed as shales, claystones, siltstones, etc., if rocklike, and as argillaceous, clayey, or silty sediments if poorly lithified.

16. The exhibition of expansive properties is undoubtably related, to a certain degree, to the extent that the materials are lithified. Lithification either binds or cements the mineral constituents together and either prevents access of water or inhibits expansion if water is able to enter the material. Fine-grained, sedimentary rocks such as shales and siltstones, may be lithified by the cementitious effects of mineral cements, such as calcite, hematite, or silica, or the lithification may be due to interparticle bonds that developed during diagenesis. These interparticle bonds are called diagenetic bonds. Clay shales are most often lithified by diagenetic bonding, whereas the more permeable siltstones owe their lithification to mineral cements that have been introduced into the material. The mineral cements are also a result of diagenesis. This distinction between cement and diagenetic bonding has resulted in the use of the terms "cementation shales" and "compaction shales" in some classification schemes, wherein the modifier "compaction" relates to materials that have developed diagenetic bonds by the loading and compaction effect produced by the overlying sediments.

17. The expansive sediments and sedimentary rock owe their origin to the accumulation of detrital sedimentary particles in a depositional basin. The sedimentary environments of deposition were controlled by local physiographic and tectonic conditions existing at the time of deposition. These expansive materials occur in both marine and nonmarine sedimentary environments.

18. The detrital sedimentary particles, such as the montmorillonite that is the source of the volume change problem, are partially the result of weathering and climatic conditions in the source area and the depositional characteristics in the basin. The detrital particles may also have been affected by diagenesis, metamorphism, and tectonism after deposition.

19. Another source for the montmorillonite is through the diagenetic alteration of volcanic glass particles or shards that were deposited in the basin along with the material derived from weathering in the source area. This volcanic glass originated during volcanic eruptions either in or beyond the source area and was subsequently carried by streams or by air currents to the depositional basin. After deposition the unstable, amorphous glass particles are altered by diagenesis to montmorillonite. This process is responsible for the commercial deposits of montmorillonite (bentonite) that occur in South Dakota, Wyoming, Montana, Mississippi, and many other states.

20. It may be concluded that the presence of montmorillonite in expansive rocks and sediments (transported soils) is a function of

a. Weathering and parent material in source area.

b. Transportation to and within the basin of deposition.

c. Vulcanism.

d. Diagenesis.

e. Metamorphic and tectonic influences on the sediments in the basin.

21. Also, these influencing factors relate more closely to environmental conditions at the time of deposition than to present environmental conditions. Thus, present climatic conditions are not usually useful indicators for determining the presence of montmorillonite in rocks or transported soils. Present environment is, however, an extrinsic factor that may affect the amount of expansion exhibited by the material and that also controls present-day weathering.

Categorization of Expansive Materials

22. Any classification scheme for categorizing the distribution of expansive materials should be based upon a combination of intrinsic and extrinsic properties that may be related to observable field performance of structures and to actual amount of expansiveness exhibited by the material in a particular region. Such a classification, based upon useful properties, must take into account the wide extent of these materials throughout the United States and the fact that they are found in various climatic zones and are associated with many diverse tectonic, geologic, physiographic, and geographic regions. Further classificational complications arise when field performance and amount of expansion are considered. These result from a general lack of uniformity in methods of measuring foundation performance and testing expansive materials.

23. Witczak[11] discussed the occurrence and distribution of the expansive soils and expansive geologic units in the continental United States and included an invaluable list of published and unpublished references. Witczak presented his information on expansive materials in terms of physiographic provinces but did not develop any casual or genetic relationships between occurrence and the physiographic province in which the materials are found.

24. Witczak[11] presented distribution maps that show the areal distribution of these materials in terms of a geologic basis, Figure 1, and a pedological basis, Figure 2. Information from these two map types were incorporated with climatic data to form a third map, Figure 3, which delineates areas that are susceptible to expansion problems. The susceptible areas were subdivided into five categories, based upon degree or likelihood of expansion problems to occur. These subcategories were mainly associated with physiographic provinces.[12] It should be emphasized here that Witczak's maps were based on geologic units upon which problems had occurred and on pedological soil classification data. Thus, expansive geologic units for which there was no engineering experience may have been omitted, and the significance of the pedological soil (which may be quite thin) was not given. Nevertheless, Witczak's maps are a useful first approximation of the distribution and relative expansiveness of these materials.

25. Classification and categorization schemes, such as Witczak's and the one presented here, include a certain degree of bias or subjectivity because of several inherent uncertainties of classification. Probably the most significant uncertainty arises because of the lack of uniformly developed data on the amount of expansion exhibited by various expansive soils and rocks. Related to lack of swell data are questions concerning type and extent of damage and the type of structure that may be susceptible to a given amount of expansion. Also, in some areas there may be no experience on the expansiveness of the local rocks and

Figure 1. Distribution of general high volume change soil areas; geologic analysis (from Reference 11)

Figure 2. Distribution of general high volume change soil areas; pedologic analysis (from Reference 11)

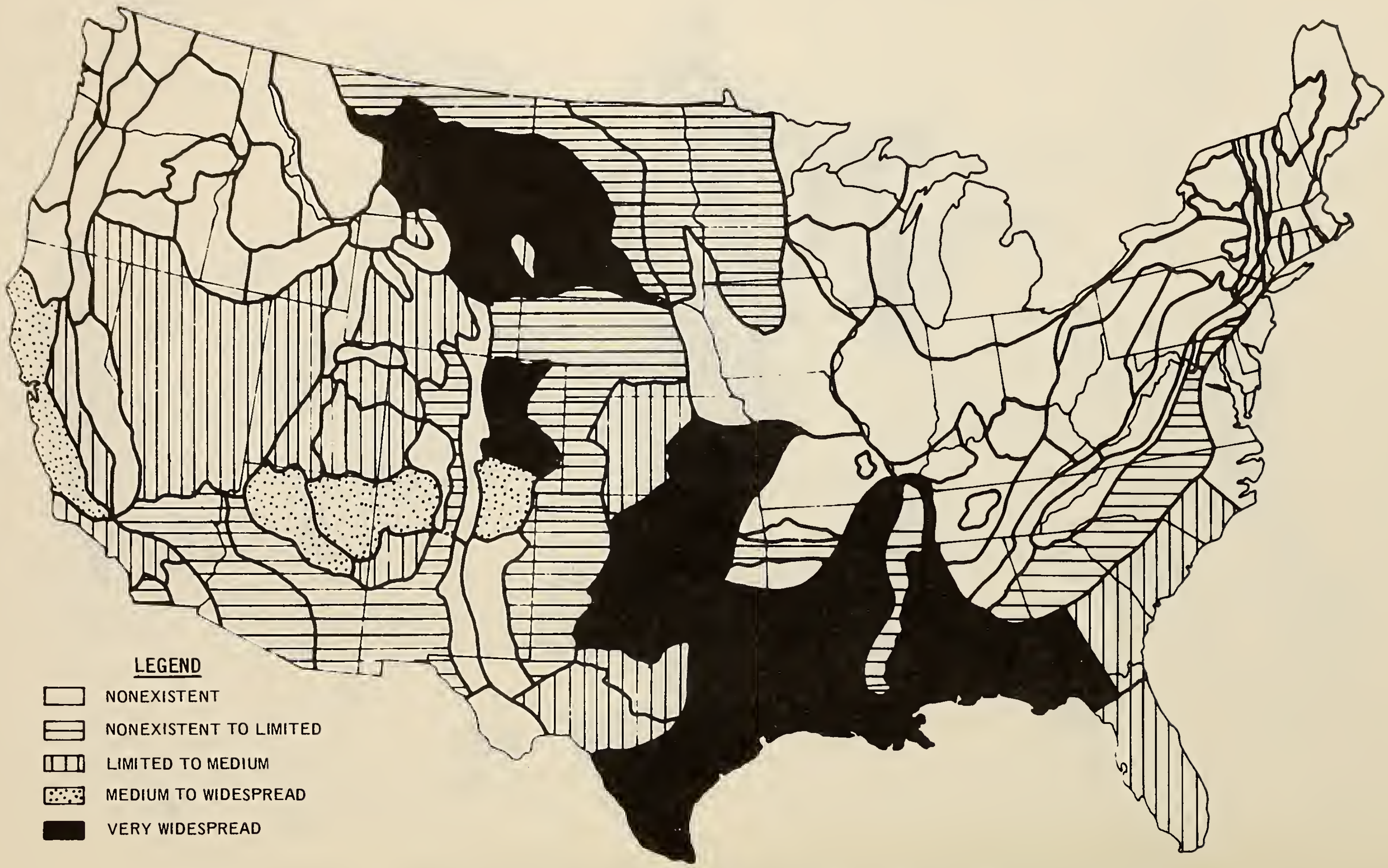


Figure 3. Estimated final adjusted frequency of occurrence rating of high volume change soils, by physiographic unit (from Reference 11)

soils where population densities are low or where there has been no construction of Interstate Highways or other primary roads.

26. The problems of classification and categorization may become more evident by briefly examining three specific materials of more or less known character and comparing them and relating each to other associated materials in each respective area. The Pierre shale, Chinle Formation, and the Yazoo clay represent highly expansive materials all of which have been the cause of considerable pavement damage and have also been responsible for some structural damage in their respective areas. These three geologic units occur in widely separated parts of the country and are associated with different climatic zones and comprise more or less different geological environments. Table 2 lists significant characteristics of these three geologic units.

27. An examination of the data given in Table 2 reveals the diversity of general climatic indicators. The geological characteristics are also somewhat diverse, although clay mineralogy and perhaps depositional environment are common to the three units. In view of the diversity and complexity of the geologic, climatic, and performance characteristics of expansive materials, there appear to be no clear-cut factors to relate these three units.

28. Another approach to identifying common factors is to examine these three materials from a regional standpoint to determine if these highly expansive materials are related spatially, chronologically, or genetically to other expansive materials in the respective regions. In the event that there is any relationship, it would then be possible to classify specific areas as to the extent and origin of expansive material, and possibly, to rate these areas with respect to field performance. There are several ways in which specific regions can be identified and subdivided. Possible bases for subdivision are: climatic zones, major depositional or structural provinces, or physiographic provinces as Witczak has used.

29. The use of climatic zones for identifying and classifying expansive materials would appear to be useful in that such a classification scheme would provide limits on one of the most important extrinsic factors, the moisture regime. On the other hand, this scheme possesses the disadvantage of not placing any limits on the material itself since a potentially expansive material may occur in several different climatic zones.

30. Depositional and/or structural provinces provide a basis for segregating materials on genetic bases, which include source material, depositional environment, diagenesis, and tectonic history. Such a classification may, however, yield little information on climatic influences.

31. Classificational schemes that utilize physiography as a basis

Table 2

Characteristics of Three Expansive Materials

Characteristics	Chinle	Pierre	Yazoo
Geologic:			
Location	AZ, NM	SD, CO, MT	MS, LA
Age	Triassic	Cretaceous	Tertiary (Eocene)
Type of Material	Mudstone	Clay shale	Silty, sandy clay
Clay Mineralogy	Calcium montmorillonite	Sodium, Calcium-montmorillonite	Calcium-montmorillonite
Depositional Environ	Deltaic (mixed, continental, and marine)	Marine	Marine
Physiographic Province	Colorado Plateau	Great Plains	Gulf Coast
Tectonic Province	Colorado Plateau	Williston Basin	Gulf Coast
Climatic:			
Mean Annual Precipitation	8-16 in.	8-16 in.	50 in.
Mean Annual Pan Evaporation	64-80 in.	48-80 in.	~60 in.
Average Temperature (August)	~70 deg F	70-80 deg F	>80 deg F
Average Temperature (January)	~30 deg F	20-30 deg F	~50 deg F

for area subdivision provide some restriction on climate and on the structural and depositional aspects of the materials within the province. This results from the fact that most physiographic province boundaries are based indirectly upon geologic structure and regional depositional patterns, and that some are small enough that the individual areal extent comprises a more or less distinct climatic zone.

32. Most existing physiographic land classifications are based upon differences in landform or topography in different areas. The differences in topography result from climatic differences between provinces and from the effects of different climates on different rock types and geologic structures and on different depositional environments. Thus, a physiographic province is defined as "a region all parts of which are similar in geologic structure and climate and which has consequently had a unified geomorphic history; a region whose pattern of relief features or landforms differs significantly from that of adjacent regions."[13]

33. Generally, it may be concluded from the preceding discussion that expansive materials in a particular area may be conveniently examined and categorized on a physiographic basis and subsequently compared with other expansive materials occurring in other areas. However, this conclusion should not be construed to necessarily mean that all of the rocks or soils in a particular physiographic province would be expected to exhibit the same degree of expansiveness or the same subgrade performance.

34. The first-order physiographic provinces, Figure 4, containing the Yazoo, Chinle, and Pierre units are, respectively, the Atlantic and Gulf Coastal Plains Provinces, Colorado Plateau, and Great Plains. These provinces also contain varying amounts of other units that are also expansive, though to a lesser degree. Genetic relationships between these other expansive units and the highly expansive ones mentioned above result from the following conditions:

a. Similar environments.

b. Similar source areas.

c. Similar geologic histories.

35. Large areas of the Great Plains Province, including the Pierre outcrop areas and the outcrop areas of other units, exhibit expansiveness because of the widespread marine environments that occurred during the Upper Cretaceous time, the nearness to volcanic areas to the west, and the minimal amount of tectonism during subsequent geologic time. These areas contain relatively nonexpansive material (including nonexpansive Pierre) because of the different environmental situations at the time of deposition. Flat topography and the characteristic low-dip angles of these units also contribute to their widespread occurrence.

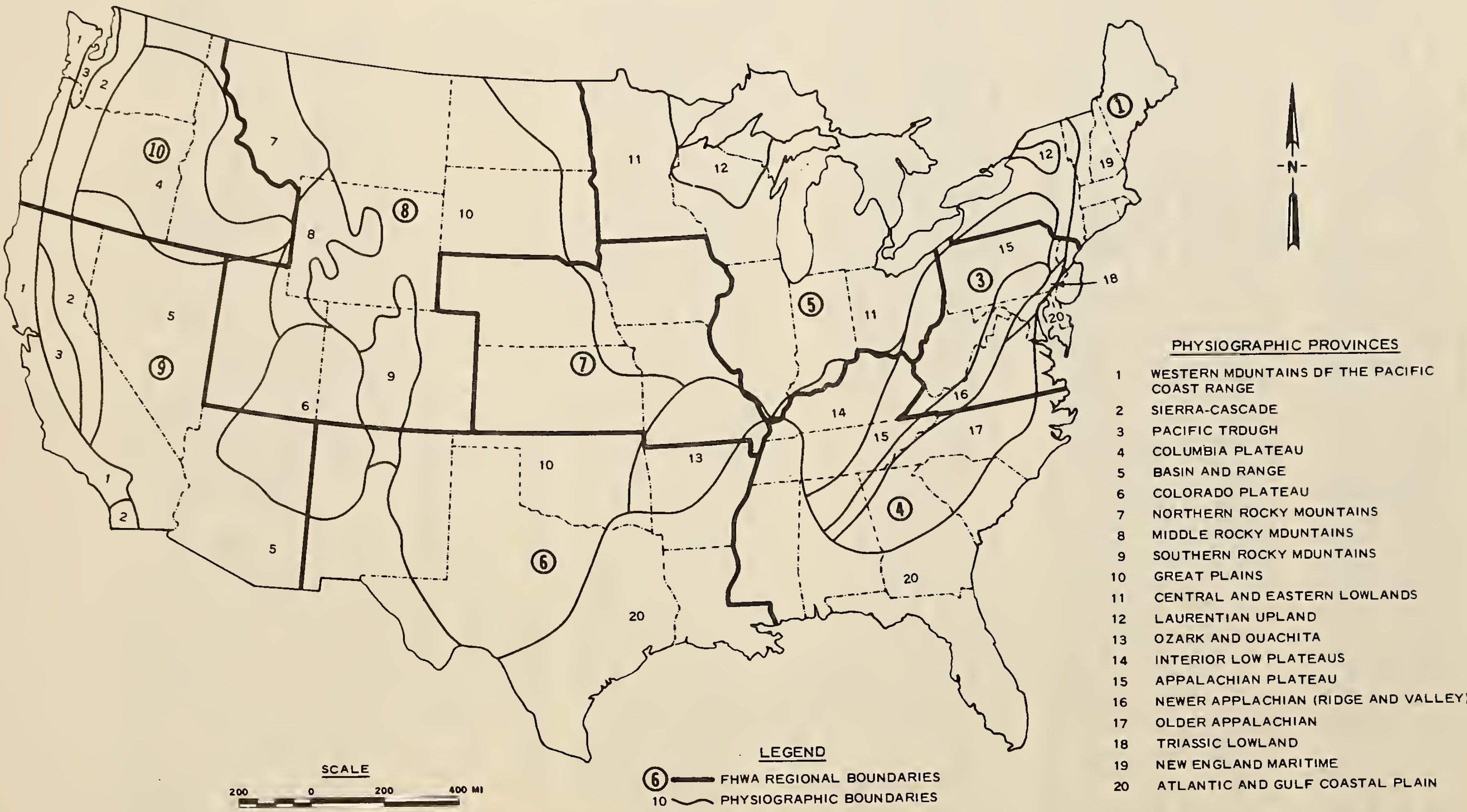


Figure 4. First-order physiographic provinces within the continental United States (from Reference 12)

36. Although the Colorado Plateau Province contains the troublesome Mancos and Chinle units, the region as a whole is not particularly expansive. Environmental conditions are responsible for this since the area contains considerable thickness of granular, continental deposits. The Mancos unit is related in time to the shales of the Colorado Group (which includes the Pierre) in the Great Plains Province, which indicates the possibility of environmental similarities between the Mancos unit and the Colorado Group. The Chinle unit does not have extensive expansive time-equivalents in other provinces to which it can be compared, and within the plateau area the Chinle itself exhibits sand and carbonate facies. The localization of expansive Chinle shale along the southern part of the plateau is due to a distinct sedimentary environment during the Triassic Period.

37. In the Atlantic and Gulf Coastal Plains Provinces, depositional environments and volcanic activity also have a dominant influence on the presence of expansive materials. Environmental controls determine the areas of expansive behavior of the Yazoo clay, which is highly argillaceous with some carbonate to the west and becomes more carbonate-rich to the east. The presence of commercial bentonites in the Vicksburg Group indicated the influence of volcanic activity in this area. Generally, those areas that are much less expansive exist because of the presence of higher energy environments. The sediments found in this province are relatively young and undeformed, thus favoring the preservation of montmorillonite.

Basis for Classification

38. The categorization and classification methods used in this portion of the report are the same as those described elsewhere.[1,2] The methods are subjective and are based 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.

39. 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 revealed actual problems or failures due to expansive materials.[11] These sources were not necessarily limited to highway subgrades.

b. Materials maps provided summaries of all illustrated earth material properties pertinent to this study.[14] The 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.[15,16]

40. 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, upon weathering, do not develop expansive soils.

41. 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 composition (i.e., 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 orginally 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.

42. The twenty first-order physiographic provinces are shown in Figure 4. The occurrence and distribution of expansive material by the Federal Highway Administration (FHWA) regions are shown in Figures 5-9, and the potentially expansive geologic units are summarized in Table 3.

Stratigraphic and Environmental Synopsis

43. Reference 2 presents a detailed discussion of the geology and the distribution of potentially expansive materials using physiographic delineation and shows that their occurrence is closely related to

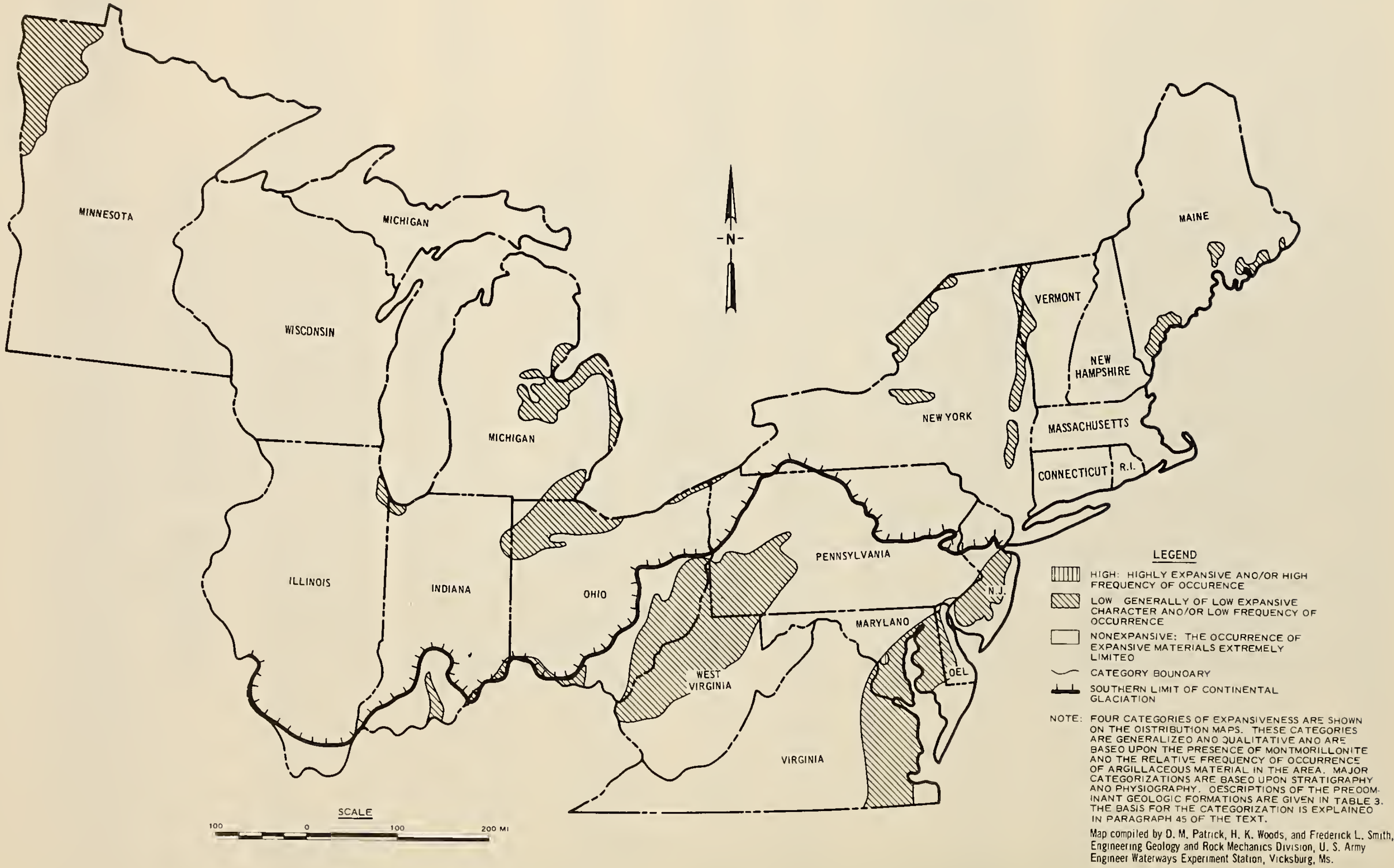


Figure 5. Distribution of potentially expansive materials in the United States; FHWA Regions 1, 3, and 5

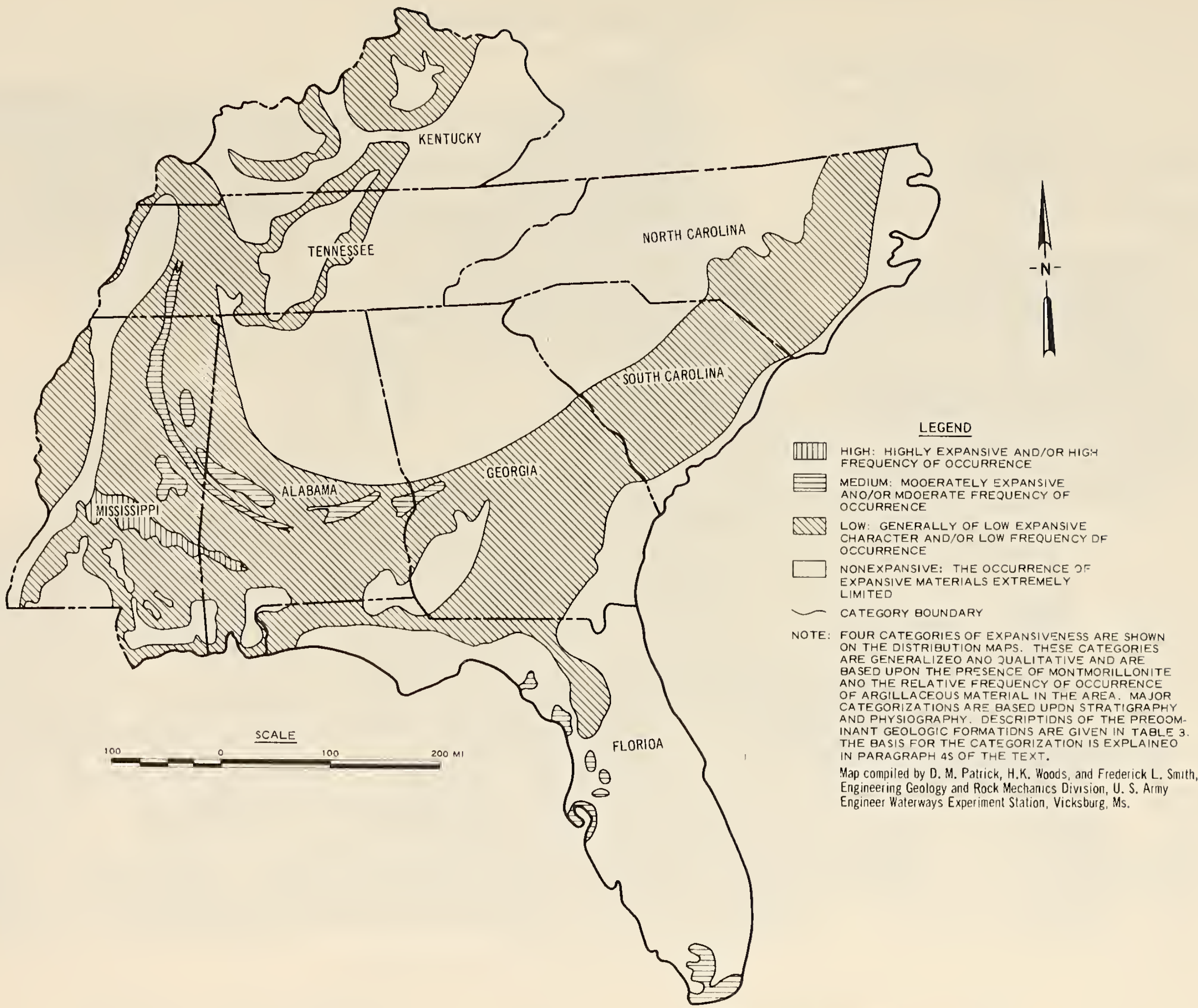


Figure 6. Distribution of potentially expansive materials in the United States; FHWA Region 4

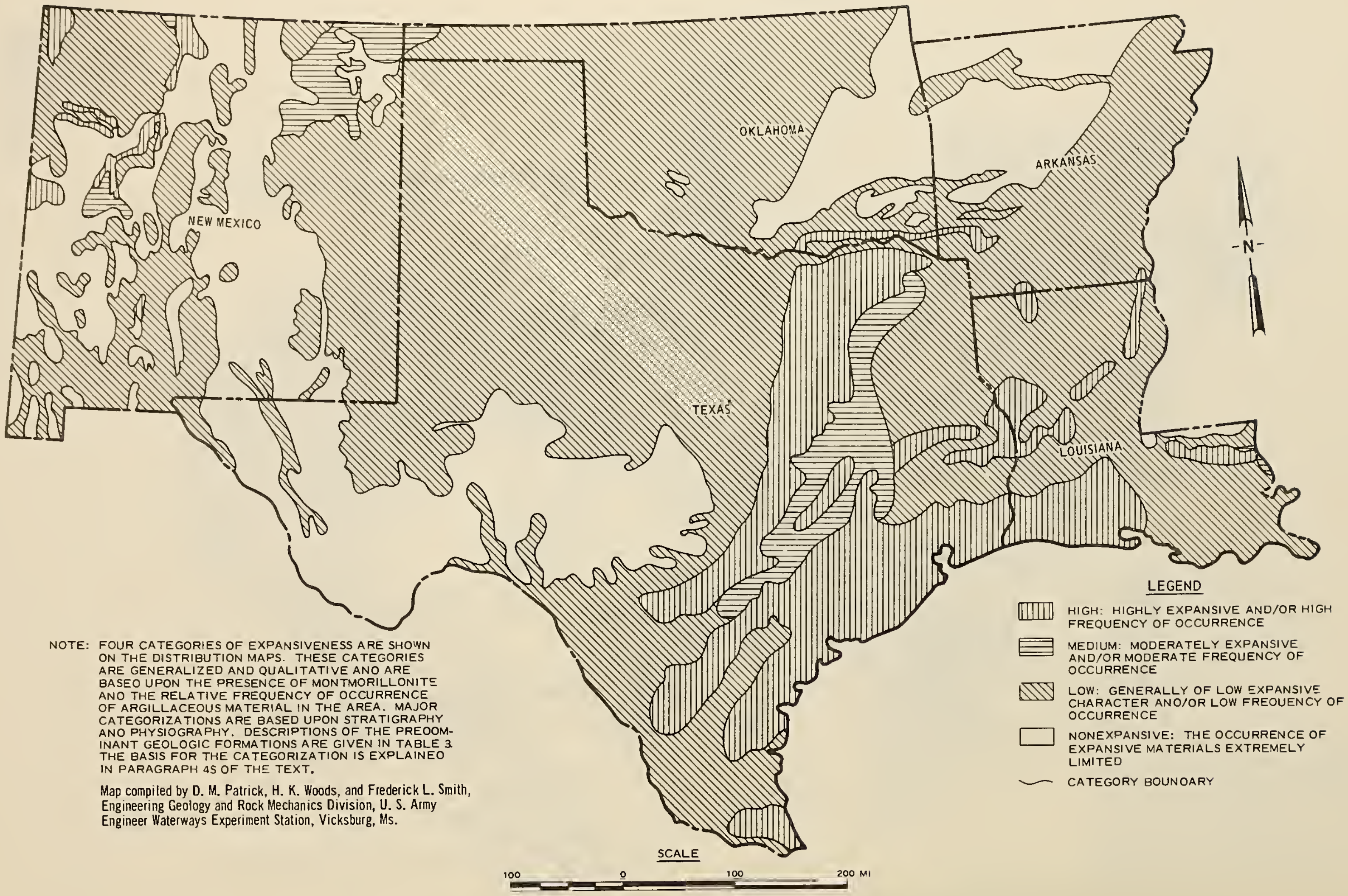


Figure 7. Distribution of potentially expansive materials in the United States; FHWA Region 6

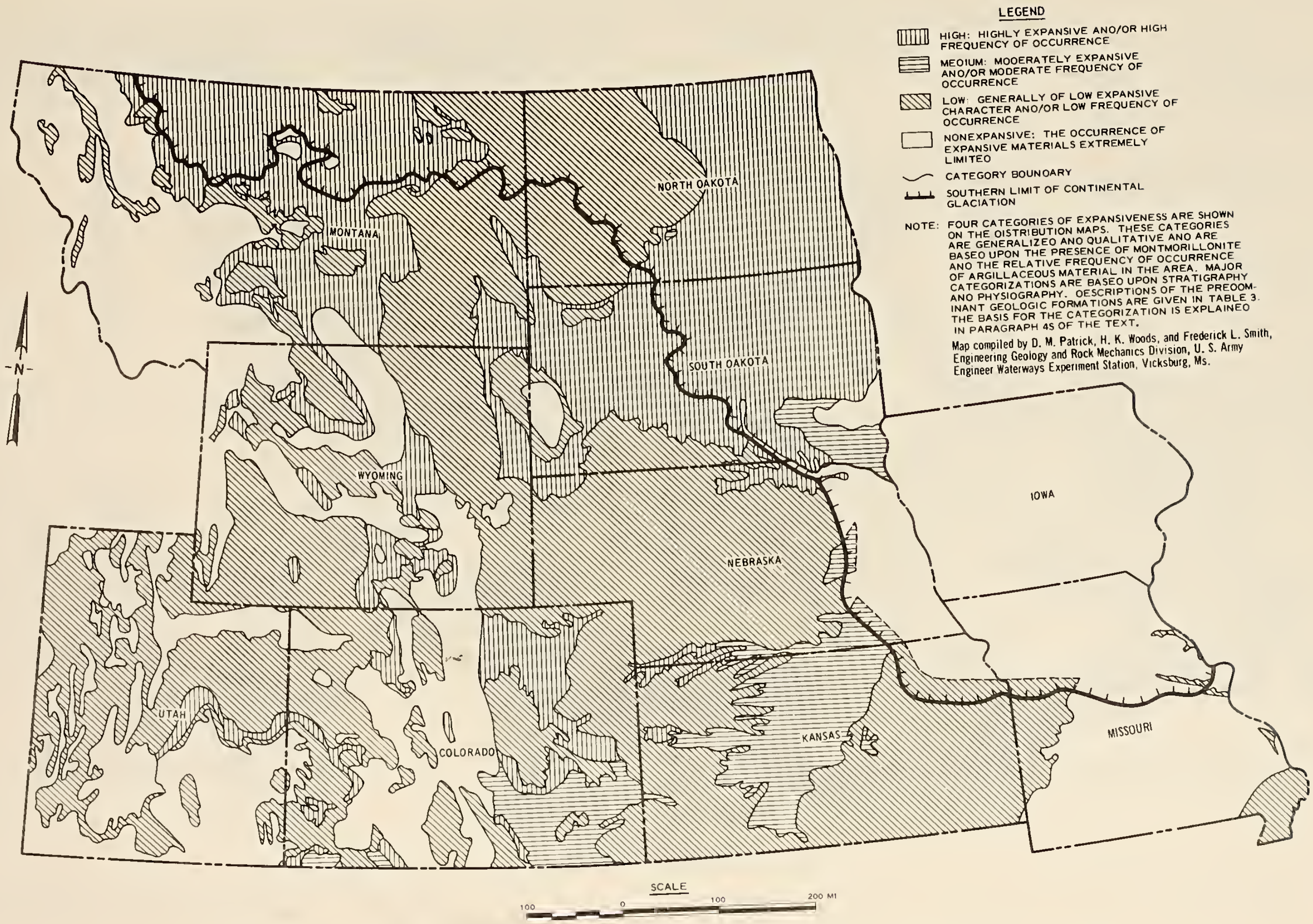


Figure 8. Distribution of potentially expansive materials in the United States; FHWA Regions 7 and 8

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

Table 3

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 4.
** Numerical map categories correspond as follows: 1 - high expansion, 2 - medium expansion, 3 - low expansion, and 4 - nonexpansive.

Table 3 (Continued)

Physiographic Province No.	Physiographic Province 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 3 (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 3 (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	

particular sedimentary rock units and to particular episodes of geologic time. Moreover, it is believed that the stratigraphy and historical geology have a more direct bearing on the occurrence of expansive materials than with relationships between the particular material and the physiographic subdivision in which the material is located. Extrinsic properties also are important, but these mainly influence the degree of expansion. Therefore, the following paragraphs attempt to summarize the occurrence of expansive materials with respect to geologic age, stratigraphy, historical geology, and geologic environment.

Precambrian

44. Precambrian rocks are mainly nonexpansive; these rocks do not contain montmorillonite because this mineral is usually destroyed after deep burial and the resulting effects of diagenesis and metamorphism. Residual soils developed upon rocks of this age may exhibit some degree of expansiveness where weathering favors development of montmorillonite.

Cambrian

45. Rocks of Cambrian age are also nonexpansive. This results from the fact that carbonates rather than shales constitute the dominant lithology during this period. Also, those shales that do occur do not contain appreciable montmorillonite.

Ordovician

46. Considerable amounts of shale occur interbedded with carbonates in the Ordovician Period. These marine shales may contain some mixed-layer illite-montmorillonite and thus pose limited problems. Upper Ordovician units, such as the Maquoketa shale of Missouri and the Kope Formation of Kentucky, Indiana, and Ohio, are examples of Ordovician units that may be troublesome. The occurrence of thin meta- or K-bentonite zones in Ordovician sequences in the Appalachians and in the Midwest support the contention that the occurrence of expansive materials and vulcanism are related.

Silurian

47. This period is represented by coarse clastics and carbonates; thus areas underlain by rocks of this age are generally nonexpansive. Possible exceptions are the marine Waldron and Osgood Formations in Indiana, Kentucky, and Tennessee, which would be classified as nonexpansive to low.

Devonian

48. The Devonian Period consists of considerably more shale than the Silurian or Ordovician Periods. Devonian shales are marine or

deltaic, and although they are not extensive enough to be mapped on the distribution maps, they would be classified as nonexpansive to low.

Mississippian

49. The Mississippian stratigraphic section consists of shale interbedded with sandstone and carbonates. The shales are mainly marine, and although not particularly extensive areally, they may contain appreciable montmorillonite. Note that Weaver[8] (see Table 1) estimated that the Upper Mississippian rocks were the oldest rocks that contain a sizeable proportion of montmorillonite among the clay mineral suite. Mappable areas underlain by Mississippian shales occur in the outcrop areas of the Fayetteville shale (Upper Mississippian) in Arkansas and the Lower Mississippian of Tennessee and Kentucky. The classification of these areas is low.

Pennsylvanian

50. The Pennsylvanian sequence consists of interbedded sandstone, shale, coal, and carbonates in the Eastern and Central Lowlands, Interior Low Plateaus, Appalachian Plateau, and Great Plains Provinces. The shales in this sequence may contain mixed-layer illite-montmorillonite, include both marine and deltaic environments, and are classed as low.

Permian

51. Permian shales of the "Red Bed" sequence of Oklahoma, Texas, and Kansas and the Dunkard Series of West Virginia represent ancient, arid continental environments that have resulted in materials of low expansiveness. Vulcanism in Idaho during this time may have provided volcanic ash to the Great Plains area.

Triassic

52. The conditions that typify the Permian were also present during the Triassic. Much of the Triassic record represents continental or deltaic conditions. Considerable shale was deposited during this time. The shales of the Chinle Formation represent some of the most expansive materials in the United States. Considerable vulcanism occurred in California during this time and may have resulted in ash accumulation and the development of montmorillonite.

Jurassic

53. During Jurassic time, there was deposition of clay and sand in the western United States; vulcanism occurred in British Columbia. Expansive materials are not abundant although the Morrison Formation exhibits some expansive properties in the western United States.

Cretaceous

54. One of the maximum inundations of the continent by marine waters occurred during the Cretaceous Period. Although sands and carbonates were also deposited, highly expansive clays are conspicuous during this period. Deposition of this material occurred throughout the Gulf and Atlantic Coastal Plain and in large areas of the western United States. Vulcanism occurred in the Nevadian Orogenic Belt and produced volcanic ash that was subsequently deposited in these marine basins. The shales of the Montana and Colorado Groups in the west and possibly the clays and marls in the Navarro, Taylor, and Austin Groups in Texas owe their expansiveness to the volcanoes in the Nevadian Orogenic Belt. Their widespread occurrence results from their marine environments.

Tertiary

55. The Tertiary is represented by both marine and continental shales along the Pacific coast, by continental deposits in the Great Plains, Basin and Range, and Colorado Plateau Provinces, and by mainly marine deposits in the Atlantic and Gulf Coastal Plain. Vulcanism was common in the western United States following the Laramide Orogeny, and Tertiary bentonites are found in Mississippi, Texas, Oklahoma, New Mexico, Colorado, Wyoming, Montana, and the Dakotas. The Tertiary, Yazoo clay of west central Mississippi represents one of the most expansive of these Tertiary materials.

Pleistocene

56. Generally, those Pleistocene deposits which possess expansive properties contain material which had originated in older deposits, were eroded out of the older deposit and transported and redeposited, often by glaciers during the Pleistocene. Common examples are the tills of the Dakotas and the terrace deposits of southwest Louisiana.

Holocene

57. The expansive material occurring in Holocene deposits also owes its origin to older formations from which it was derived. For example, the montmorillonite in the clay mineral suite of Mississippi River deposits was derived from montmorillonitic formations in the Great Plains Province. The degree of expansiveness of such deposits is in part controlled by the relative amount of montmorillonite being deposited versus the amount of other clays derived from other sources. Thus, the clay mineral suite found in fine-grained Mississippi River deposits is diluted with nonexpansive illite and chlorite derived from the Ohio River Valley.

Environmental considerations

58. The geologic environments in which expansive materials are found are variable, and it is possible to find these materials under diverse geological conditions.[17,18] The previous discussion has shown that expansive clays occur in marine environments (Montana, Colorado, Navarro, Austin, Taylor Groups, etc.), in mixed environments such as deltas (Chinle Formation), and on continental environments such as the Tertiary of the Great Plains. The marine environment is the most significant, perhaps, since these conditions are most suitable for the widespread distribution of a particular deposit. On the other hand, continental environments, such as lacustrine, fluvial, or alluvial deposits, tend to be more restricted with respect to size. Deltaic environments may be somewhat intermediate in size. Thus, any type of depositional basin may be appropriate for the accumulation of detrital, expansive clays or for volcanic particles that may alter to montmorillonite after diagenesis.

59. The preservation of detrital montmorillonite is a function of the age of the deposit and its geologic history including loading by supra adjacent sediments and tectonism. Volcanic debris (ash, glass, shards, etc.) is diagenetically altered under appropriate chemical conditions to montmorillonite, the preservation of which is then controlled by subsequent geologic history.[19]

60. The volcanic debris, which usually consists of amorphous glass or shards, will alter to montmorillonite under most conditions. However, under special conditions the shards may alter to other clay minerals such as illite or kaolinite, or to other silicates such as zeolites. The alteration process consists of devitrification of the highly unstable volcanic glass and the crystallization or neoformation of the new mineral, montmorillonite. This process requires the removal of chemical constituents from the system but does not generally require chemical additions.

61. The length of time required for the devitrification of the volcanic material may be variable and dependent upon the environment of deposition. The younger Pleistocene and Tertiary continental ash deposits of the Great Plains Province may locally contain appreciably more ash than montmorillonite, indicating that the devitrification process is incomplete. On the other hand, the marine bentonites in the Mowry Formation (Colorado Group) resulted from devitrification that began shortly after deposition on the sea floor.[20]

62. The rhyolitic ash commonly has silica contents of approximately 70 percent silicon dioxide, whereas the silica content of montmorillonite is approximately 57 to 60 percent silicon dioxide. Therefore, in order for a reaction to occur and montmorillonite to form, there must be the appropriate conditions for leaching of silica from the system. This involves a chemical environment suitable for the

solution of silica from the volcanic ash fragments and the removal of the soluble silica beyond the site of alteration. Silica is somewhat soluble at neutral pH conditions but is increasingly soluble under more alkaline conditions. The solubility requirements, with respect to pH, are usually met in most geologic environments. For effective removal, however, there must be sufficient permeability for the water carrying the dissolved silica to move through the system.

63. Alumina, being much less soluble under typical geologic pH conditions, remains to form the octahedral layer of the montmorillonite. Magnesium also remains in the system and will occupy octahedral or interlayer positions in the clay. The alkalis, potassium and sodium, are selectively removed or remain depending upon the chemical character of the aqueous environment. Generally, potassium is removed; if not, the resulting mineral would be illite rather than montmorillonite. The sodium and calcium are selectively removed, and that which remains occupies interlayer positions on the montmorillonite.

64. The amount of expansion exhibited by montmorillonite is strongly influenced by the cation occupying interlay positions, sodium clays being considerably more expansive than clays having either calcium or magnesium in interlayer positions. Therefore, relationships between geologic environments and interlayer cations might provide useful information on expected expansiveness of a particular deposit.

65. It would seem reasonable to assume that volcanic ash deposited in marine environments would tend to retain sodium on the neoformed clay since the marine environment contains abundant sodium. Continental environments, unless evaporitic, usually contain more calcium than sodium, and the clay forming here would exhibit calcium on interlayer positions. Deltaic environments would contain both sodium and calcium, and the neoformed clay would also exhibit both cations in interlayer positions.

66. Unfortunately, the interlayer cations are rather easily exchanged, resulting in the removal of the original ion and the substitution of another. This cation exchange could occur during later stages of diagenesis or during present-day (or earlier) weathering cycles. Further, complications arise with respect to concentrations of calcium carbonate in marine environments. Such conditions may result in the dominance of calcium over sodium on the clays of this environment. Thus, the determination of the environment of a particular deposit does not mean that the interlayer cation on the constituent clay minerals is known.

Summary

67. This part has reviewed the details presented elsewhere,[2]

specifically it provided an insight into the geologic influence on the occurrence and distribution of expansive materials. Some of the more important points concerning this geologic influence and related topics are summarized in the following paragraphs.

68. Expansive earth materials consist of expansive, argillaceous, residual, and transported soils and expansive, argillaceous sedimentary rocks exhibiting varying degrees of lithification. The degree of expansiveness exhibited by the soil or rock is controlled principally by the constituent mineralogy. External factors modify the effects of the mineralogy. Expansive sedimentary rocks and transported soils are believed to be considerably more expansive than residual soils, particularly if the parent material of the residual soil is not expansive.

69. Sedimentary rocks older than Mississippian are not generally expansive because of the instability of montmorillonite and its modification in these older rocks. The most highly expansive materials occur in rocks that are Triassic or younger.

70. The distribution of expansive materials is influenced by stratigraphy and geologic history and modified by external factors. The distribution and degree of expansiveness can be determined for various areas by examining the amount of argillaceous material present and the age of material in the area in question.

71. Physiographic delineation is a useful method for discussion and analysis of expansive materials but has no direct genetic implications beyond first-order subdivision.

72. The source of the montmorillonite present in expansive materials is mainly volcanic ash that has been transported by winds, surface streams, and ocean currents from a volcanic region and deposited in a sedimentary basin. After deposition, the volcanic ash alters to montmorillonite. Detrital montmorillonite may also be an important constituent in some deposits.

73. Geologic environment does not appear to be a useful criterion for the recognition of potentially expansive materials. Marine, continental, and deltaic environments may contain expansive clays. Marine environments are significant in that these environments cover extensive areas; this results in the widespread occurrence of expansive materials.

PART III: VERIFICATION OF NATURAL MICROSCALE MECHANISMS THAT CAUSE VOLUME CHANGE IN EXPANSIVE SOILS

Review of Microscale Mechanisms

74. The expansive soils that most State Highway Agencies deal with are good foundation materials, provided changes in the ambient conditions do not cause a change in the availability of water to the material. For expansive materials to expand, three criteria must be satisfied: namely, (a) an available source of water, (b) a driving force to move the water, and (c) a mechanism or group of mechanisms that actually cause the volume change. Both the sources of water and modes of moisture transfer were discussed in some detail in Reference 1. With this in mind, the discussion will concentrate on verifying the microscale mechanisms causing volume change. Of the six mechanisms listed in Table 4, three are not actually microscale phenomena, namely, capillary imbibition, Van der Waal forces, and elastic relaxation.

75. Capillary imbibition listed in Table 4 as a mechanism is not really a microscale mechanism in the sense of physically causing expansion. Instead, it is actually one of the modes of moisture transfer and, as such, provides a source of water for the true mechanisms. Capillary imbibition will influence both the magnitude and rate of volume change but has little direct bearing on the sorption process and will not be considered in these discussions.

76. Van der Waal forces, including London forces and hydrogen bonding, are weak, attractive electrical forces that develop on the surfaces of clay minerals. These forces exist between adjacent clay mineral surfaces and tend to bond the surfaces together. An example of this kind of bonding can be observed in the coarse-grained micas in which thin sheets can be cleaved from the material. Similar forces occur in clay minerals except that the small particle size and large surface area of clays result in these forces being somewhat more important. The forces are in effect in both the dry and wet conditions and control the sorption of water. After sorption begins, the forces play a small part in bonding the water molecules to the clay surfaces. Van der Waal forces include three types of weak electrical attractive forces, namely:

a. Dipole-dipole attraction. These forces develop between polar molecules having permanent moments and are at least partly responsible for the orientation of water molecules and their bonding to the clay mineral surface.

b. Induction effects. These forces are similar to dipole-dipole attraction and occur between polar molecules; however, the induction effects occur between unoriented molecules by the interaction between one dipole and the polarized electrons of another dipole.

Table 4

Natural Microscale Mechanisms Causing Volume Change in Expansive Soils

Mechanism	Explanation	Influence on Volume Change
Osmotic repulsion	Pressure gradients developed in the double-layer water due to variations in the ionic concentration in the double layer. The greatest concentration occurs near the clay particle and decreases outward to the boundary of the double layer	The double-layer boundary acts as an osmotic membrane when exposed to an external source of free water; that is, it tries to draw the water into the double layer to reduce the ionic concentration. The result is an increase in the double-layer water volume and the development of repulsive forces between interacting double layers. The net result is an increase in the volume of the soil mass
Clay particle attraction	Clay particles possess a net negative charge on their surfaces and edges which result in attractive forces for various cations and in particular for dipolar molecules such as water. This makes up the major "holding" force for the double-layer water	In an effort to satisfy the charge imbalance, the volume of water in the double layer will continue to increase until a volume change of the soil mass occurs
Cation hydration	The physical hydration of cations substituted into or attached to the clay particle	As the cations hydrate, their ionic radii increase, resulting in a net volume change of the soil mass
London-van der Waal forces	Secondary valence forces arising from the interlocking of electrical fields of molecule associated with movements of electrons in their orbits. The phenomenon frequents molecules in which the electron shells are not completely filled	The interlocking of electrical fields causes a charge imbalance which creates an attractive force for molecules such as water
Capillary imbibition	Movement of water into a mass of clay particles resulting from surface tension effects of water and air mixtures in the pores of the clay mass. Compressive forces are applied to the clay particles by the menisci of the water in the pores	As free water becomes available to the clay mass, the pore water menisci begin to enlarge and the compressive forces are relaxed. The capillary film will enlarge and result in a volume change or supply water for one of the other mechanisms
Elastic relaxation	A readjustment of clay particles due to some change in the diagenetic factors	Volume change results from particle reorientation and/or changes in soil structure due to changes in the diagenetic factors

c. London forces. These forces, which are also termed "dispersive" forces, occur in all molecules or extremely small particles including nonpolar (zero dipole moment) varieties. The forces originate from the development of an instantaneous, nonpermanent dipole moment between two particles as they come into close proximity to one another.

Under conditions where water is attracted to the clay mineral, the influence of Van der Waal forces is rapidly overcome by the development of the double-layer water. In addition, this phenomenon is very hard to measure and interpret physically and, in cases where it can be evaluated, provides little or no practical information for the engineer. Accordingly, this phenomenon is also dismissed.

77. Elastic release could actually be considered a special type of volume change rather than a cause of it since it is a particle reorientation resulting from swelling or unloading. Other factors, such as diagenetic bonding and cementation, significantly influence elastic release, generally on the side of volume change reduction. On the other hand, elastic release will influence capillary imbibition, one of the modes of moisture transfer, since particle reorientation will generally result in increased pore sizes.

78. The preceding discussions have effectively reduced the number of microscale mechanisms responsible for volume change to three: clay particle attraction, cation hydration, and osmotic repulsion. Several attempts have been made to isolate and verify the mechanisms of volume change, and in most cases, these three mechanisms are given primary responsibility for expansion.

79. In a study of swelling of compacted clays, Ladd[21] concluded that for samples compacted wet of optimum water content, swelling is caused by osmotic repulsive pressures, and for samples compacted dry of optimum, swelling is influenced by factors in addition to osmotic pressures. These factors include cation hydration and attraction of the clay particle surface for water, London-Van der Waal forces, elastic rebound, particle orientation, and pressence of air. Ladd admitted that the relative importance of the additional factors was not known. An interesting point about Ladd's work is that, in a very general sense, he was pointing out a type of categorization of the range of applicability of the basic mechanisms. In other words, his concept of osmotic repulsion controlling volume change above optimum water content and cation hydration and clay particle attraction plus the other less significant factors having greater influence below optimum water content was a major step toward a better understanding of the volume change phenomenon.

80. Low[22] described five possible mechanisms of soil-water interaction that would, in turn, influence volume change behavior. These include hydrogen bonding, hydration of exchangeable cations, attraction

by osmosis, dipole attraction, and Van der Waal forces. Hydrogen bonding and dipole attraction are specific terms for the clay particle attraction mechanism. Attraction by osmosis is the complementary term to osmotic repulsion. In other words, the mechanism of volume change is repulsion, but the repulsion is the result of osmotic attraction of water into the influence of the double-layer water.

81. Additional references concerning the role of mechanisms may be consulted--Ingles,[23,24] Quirk,[25] and Low[26]--and the results will be the same. That is, the three basic microscale mechanisms will be discussed along with a few relatively inconsequential mechanisms such as Van der Waal forces.

Definition of Mechanisms

82. Previous discussions have indicated that, at the microscale level, the mechanisms causing volume change in expansive soils are clay particle attraction, cation hydration, and osmotic repulsion. In reality, the individual influence of each of these mechanisms is hard, if not impossible to separate from its counterparts. However, to adequately define the mechanisms, their individual roles will first be discussed, and then in summary, the interrelationships of the mechanisms will be addressed.

Clay particle attraction

83. Surface attractive relationships that exist between clay minerals, between clay minerals and water, and between clay minerals and cations are a result of the shape and internal crystallographic structure of the clay mineral. Clay minerals occur as tiny platelets having two types of exposed surfaces--edges and faces. The edges are generally more irregular in shape, exhibit a smaller surface area, and possess both positive and negative charges, primarily due to broken bonds. The faces are generally flat, exhibit the majority of the particles' surface area, and, in the case os smectites and illites, possess an electron-rich surface and negative charge due to the presence of oxygen atoms in the tetrahedral layer. The magnitude of these electrostatic charges and the resulting attractive forces is intensified because of the extremely small size of the clay mineral platelets. These smectites, particularly montmorillonite, are characterized by the substitution of divalent magnesium for trivalent aluminum in the octahedral layer. The substitution results in a net negative charge imbalance that may be satisfied by cations situated on interlayer positions (faces) and to a lesser extent on the platelet edges. The process of attracting and holding water molecules is achieved through the processes of hydrogen bonding of the water molecules to the clay mineral surface and dipole-dipole attraction of the water molecules for one another. A schematic representation of the process is shown in Figure 10a. Because the clay mineral surfaces are usually composed of either exposed oxygens or

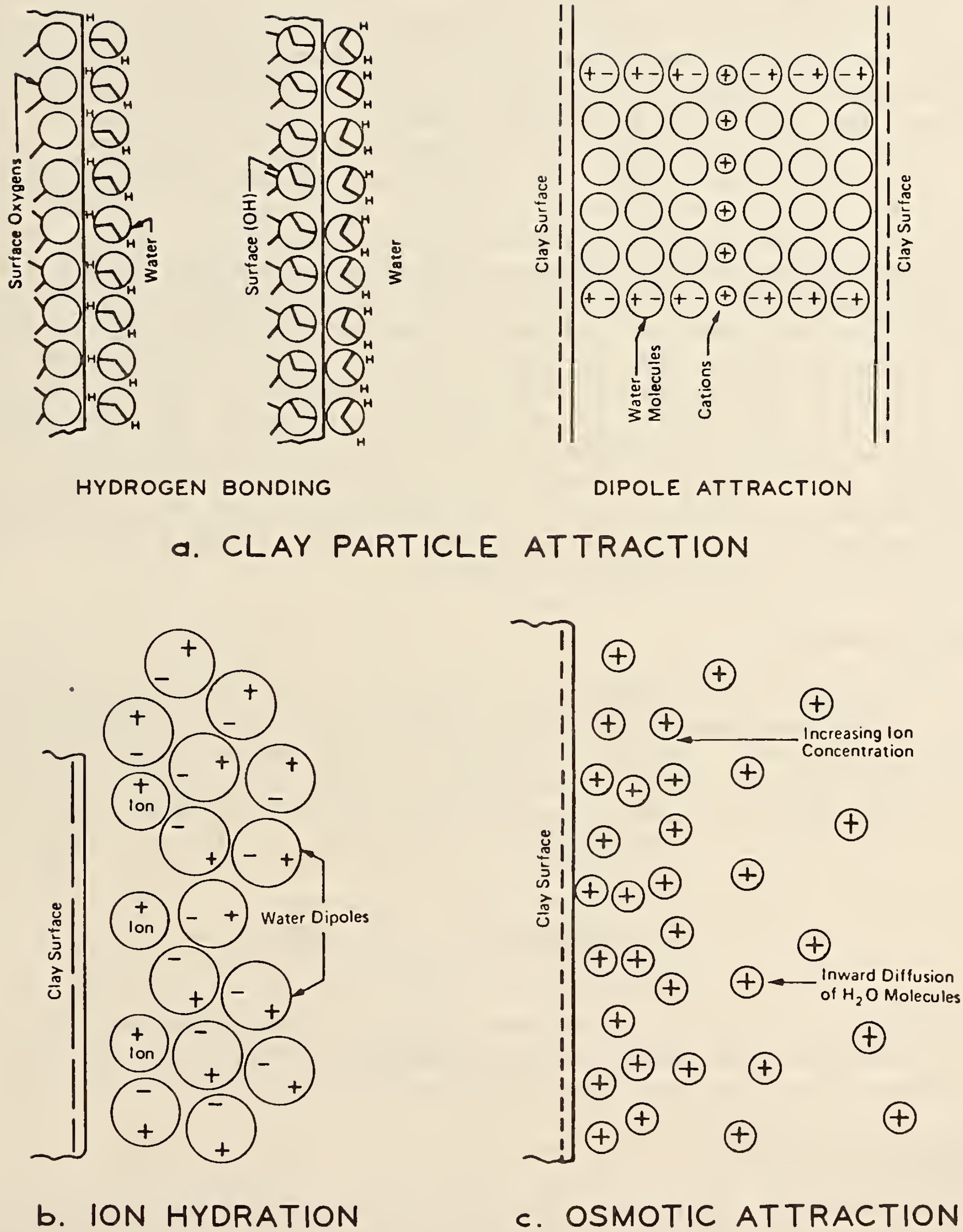


Figure 10. Mechanisms of water adsorption by clay surfaces (from Reference 27)

hydroxyls on faces and/or other positively charged ions on edges, the hydrogen bonds may develop by either the oxygens attracting and bonding with the positive side of the dipolar water molecule or the hydroxyls attracting and bonding with the negative side of the water molecule. The hydrogen bonding of water molecules to the clay mineral surface provides the basic building blocks for the double-layer water that, in reality, is the moving force in soil expansion.

84. Charged surface-dipole attraction in soils involves the surface attractive forces discussed in the previous paragraph; however, the influence of these forces decreases with distance from the clay mineral surface. Water is a dipolar substance, that is, it has two centers of charge--one positive and one negative. To form an analogy,[27] the clay mineral surfaces (interior or exterior between particles) are analogous to negatively charged condenser plates with an electric field strength that decreases with distance from the surface. Water dipoles can orient themselves with their positive poles directed toward the negative surfaces with the degree of orientation decreasing with distance. The only flaw in this analogy is that at the midpoint between the plates or clay minerals, there would be a structural disorder because like poles would be adjacent to one another. Ingles[24] suggests that this may not be far from correct because due to the high hydration number and energy of aluminum in the clay structure, water can be strongly attracted to the surfaces and interposes itself between the surfaces and the counterions, thus removing the counterion as far away as possible or, more precisely, to the midpoint. Such a configuration is shown at the right side of Figure 10a.

Cation hydration

85. The cation hydration mechanism is more easily understood if it is considered as a special case of the clay particle attraction previously discussed. Instead of the clay particles being surrounded by water molecules to balance the negative charge deficiency, suppose that cations, such as calcium, magnesium, sodium, or potassium, are attracted to the clay surface, rendering it neutral from a charge deficiency sense. There still exists a considerable force for the attraction of water in the form of hydration of the cations. This results from the charge of the cation not being fully neutralized. The influence of cation hydration involves both attraction force for water molecules, Figure 10b, and a physical increase in size (ionic radii) following hydration. In reality, there are no purely clay-water or clay-cation systems; rather there is generally a clay-water-cation combination that gives rise to the third and final microscale mechanism, osmotic repulsion.

Osmotic repulsion

86. As explained earlier, the holding (electrostatic attractive) forces for water molecules and ions are greatest at the clay mineral surface and decrease with distance from the surface. Therefore, the

concentration of cations will be greatest at the clay mineral surface and decrease with distance from the surface. When the soil-water-cation combination is exposed to a pore fluid having a different (lesser) ion concentration, the double-layer water acts as a semipermeable membrane allowing water to enter in order to bring the two differing ion concentrations into balance. This is generally denoted as osmotic attraction, Figure 10c. The result of the osmotic attraction is a buildup of the double-layer water. The net result of the double-layer water buildup will be volume change (increase) of the soil mass. The precise influence of osmotic repulsion on volume change is not well understood; however, it is generally accepted that the mechanism has its greatest influence at higher moisture contents (i.e., at greater than optimum moisture content for compacted soils).

87. The previous discussions have been directed toward explaining the microscale mechanisms and their influence on volume change from the standpoint that they act individually. In reality, the three major mechanisms do not act individually; instead, they are quite dependent upon one another. For example, the sorption process cannot begin without clay particle attraction. Ion hydration cannot be significant without clay particle attraction, and osmotic repulsion cannot influence volume change without the particle attraction for water and cations and the variation of cations within close proximity to the clay mineral. The importance of the interdependence is most evident when attempts are made to isolate and verify the direct influence of the individual mechanisms. Even the most sophisticated soil chemical analysis procedures do not provide the needed practical information to relate the mechanisms to volume change behavior. To overcome these deficiencies in available information, the engineer recognizes their existence and then measures the final result (volume change). The soil scientist's end result measurement involves the soil's affinity for water (soil suction). A combination of the two approaches is the basis of this research program; however, the need exists for a better definition of the terminology used by the soil scientist faction, which is included in the following paragraphs.

Definition of Soil Suction

88. Soil suction is a quantity that can be used to characterize the effect of moisture on the volume and strength properties of soils;[28-31] 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. Suction is a pressure term that is a measure of the pulling force (tension) exerted on the water. Tension is also a term used to indicate the force of soil water retention and can be used interchangeably with suction; however, soil moisture suction or simply suction is generally preferred.

89. The total soil suction may be alternatively defined as the free energy present in soil water with respect to a pool of pure water located outside of the soil at the same elevation. This energy is the source of work that is done when the pool of pure water comes in contact with the soil. At equilibrium following contact with the pure water, the free energy is dissipated in the form of work that is done to pull the pure water into the soil, counter the friction, and expand the soil lattice. The effect that the dissipation of free energy or total suction has on the physical properties of the soil depends on the soil composition (type and amount of clay mineral) and the cation environment of the soil.

90. The total soil suction is the sum of the matrix and osmotic components. The matrix suction is comprised of the surface attractive forces for water and cations and the surface tension effects of water in soil, thus representing two of the three major mechanisms. The matrix suction is both water content and surcharge pressure dependent. The osmotic suction arises from the presence of soluble salts in the soil water and is identical in context with the osmotic attractive forces previously described. The osmotic suction will cause a physical change in the soil when contact is made with free water, provided the type and concentration of the salts in the soil differ from that of the free water. Osmotic suction is independent of water content and surcharge pressure.

Evaluation of Soil Suction

91. All processes and chemical reactions of which the natural microscale mechanisms are a part use energy. Evaluation of soil suction is essentially an evaluation of the level of energy available to the natural microscale mechanisms that cause soil volume change. Changes in soil suction reflect the amount of energy in the pore water of the soil used by the mechanisms to cause swell.

Approaches

92. Two approaches are currently being used for describing the effects of water in soil: the mechanistic and energy approaches.[32] The mechanistic approach is based on measurements of negative pore water pressure in specimens using special consolidometers and pressure membrane devices. For these measurements[28] membranes with small pore sizes are used to prevent cavitation of water in the membrane. Air pressures may therefore be applied to the soil specimen to increase the positive or decrease the negative pore water pressure without loss of air or loss of control of the volume of water entering or leaving the specimen. Water will be forced from the soil if air pressures are sufficient to cause positive pore water pressures. Water will be imbibed into the soil if the pore water pressures are negative. Applied air pressures

that lead to no flow of water into or out of the specimen are denoted as the negative pore water pressures or soil suctions.

93. Evaluation of soil suction by the mechanistic approach is encumbered with conceptual and measurement problems. Olson and Langfelder[30] observed that the force fields of the clay minerals, which are responsible for the microscale mechanisms that cause swell, very likely cause the actual pore water pressure to be positive near the surfaces of the clay mineral particles. The mechanistic approach evaluates an equivalent negative pore water pressure or soil suction that is needed to pull the pore water out of the soil. The equivalent pore pressure measurement is performed with a water content that changes during the measurement because water is forced into or out of the specimen. The equivalent pore pressure also neglects much of the contribution to soil suction from the concentration of ions in the pore fluid if the ions are able to pass through the membrane of the apparatus. The ions will pass through membranes made of porous stones or ceramic plates.[28] Hysteresis is also observed in the soil suction-water content relationships determined for a single specimen with the pressure membrane device.

94. Determinations of soil suction using the mechanistic approach must be corrected with various calibration factors. A correction is needed to adjust void ratio computations for deformations of the apparatus. Another correction is needed to adjust water content computations for the accumulation of air beneath the porous plate that results by diffusion of air from the applied air pressure through the soil specimen.[28] Minute voids or fissures in the seal between the porous plate and ring surrounding the plate contribute to the loss of volume control of the air. The volume of air adds to the volume of water that is forced from the specimen. In general, test procedures using the mechanistic approach are often tedious and time consuming, and data reduction is laborious.

95. Evaluation of soil suction by the energy approach is a more general method based on the principle of thermodynamics. Soil suctions evaluated by both the mechanistic and energy approaches can lead to similar magnitudes on similar specimens[28,29,33,34], but the two concepts are different. The method adopted herein for characterizing swell behavior by soil suction is based on the energy approach where soil suction is evaluated from measurements of relative humidity in soils determined with thermocouple psychrometers.

Energy concept

96. The most fundamental expression of the state of water in soil is the relative free energy of the soil water. The force that causes available water to move into soil is expressed quantitatively in terms of the free energy of the soil water relative to the available water outside of the soil. The free energy (Δf) needed to move free pure

water into the pores of soil containing the soil water is[31]

$$\Delta f = RT \log_e \frac{p}{p_o} \qquad (1)$$

where

R = ideal gas constant, 82.06 cc-atm/K

T = absolute temperature, K

p = vapor pressure of the pore water in the soil, atm

p_o = vapor pressure of free pure water, atm

p/p_o = relative humidity

The change in free energy due to movement of the free pure water into the pore water is usually given in terms of an equivalent total soil suction

$$\tau^o = \frac{1.058RT}{v} \log_e \frac{p}{p_o} \qquad (2)$$

where

τ^o = total soil suction, tsf

v = volume of a mole of liquid water, 18.02 cc/mole

The superscript "o" after τ means the soil is not subject to any confining pressure, except for atmospheric pressure. Total soil suction has been defined for convenience as the sum of osmotic τ_s^o and matrix τ_m^o components,[31]

$$\tau^o = \tau_s^o + \tau_m^o \qquad (3)$$

The osmotic suction by definition[31] is due entirely to the concentration of soluble salts in the pore water, and it is consequently related mostly to the osmotic repulsion mechanism. The osmotic suction is expressed by

$$\tau_s^o = \frac{1.058RT}{v} \log_e \frac{p_s}{p_o} \qquad (4)$$

where p_s is the vapor pressure of the free pore water solution, atm.

The osmotic suction can increase as water evaporates from the soil because the concentration of ions in the remaining soil water can increase. The osmotic suction does not change with confining pressure.

97. The matrix suction in swelling soils is related mostly to forces arising from clay particle attraction and cation hydration in addition to surface tension effects. The matrix suction is expressed by

$$\tau_m^o = \frac{1.058RT}{v} \log_e \frac{p}{p_o} \tag{5}$$

and can be evaluated directly from the relative humidity of the soil when the chemical composition of the pore water contributes negligible osmotic suction. The matrix suction will decrease with increasing confining pressure.

Evaluation using
<u>thermocouple psychrometers</u>

98. The thermocouple psychrometer measures the relative humidity in the soil by a technique call Peltier cooling.[35] 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 developed between the thermocouple and reference junction is measured by a microvoltmeter.

99. The voltage outputs of the psychrometers are calibrated by tests with salt solutions, such as potassium chloride, that produce a given relative humidity for known concentrations.[36] The relative humidities are converted to osmotic soil suctions by Equation 4, which are also total soil suctions of these solutions. The resultant calibration curve of the commercial psychrometers is linear. A typical example of a calibration equation is

$$\tau^o = 2.82E_{25} - 4.4 \tag{6}$$

where

τ^o = total soil suction, tsf

E_{25} = microvolt output at 25°C

Typical voltages vary from less than 1 microvolt for relative humidities close to 100 percent or total soil suctions less than 1 tsf to about 25 microvolts for humidities of about 95 percent or total soil suctions of about 60 tsf. The reliable lower limit of these soil suction evaluations is about 1 tsf. Equipment was developed elsewhere using a special constant temperature bath and psychrometers with more than one junction that can be used for measurement of suctions less than 1 tsf with reliability of about 10 percent.[33]

Field Sampling and Laboratory Testing Programs

100. The field sampling program was initiated through contacts with 11 State Highway Agencies in which agency representatives were requested to recommend appropriate geologic formations and site locations based on the simple criterion that the material be representative of an areally extensive deposit of expansive soil which poses problems as defined by the State Highway Agency. As a result, 20 field sampling sites were selected in 11 states. Table 5 provides a summary of the 20 site locations, geologic formations sampled, samples tested (first phase), natural moisture contents and densities, compaction properties, and soil classifications. Detailed site location information, site description, site geology, sample description, description of climate, and summary of climatic data are available in Reference 3.

101. The laboratory program was conducted to determine pertinent physical, physicochemical, and mineralogical properties so that the role of the microscale mechanisms in causing volume change could be inferred from the measured parameters' influence on the magnitude and rate of volume change. In other words, a phenomenological approach was used in which the influence or role of the microscale mechanisms was inferred using known relationships between measured laboratory parameters and microscale mechanisms. Details of the laboratory testing program, testing procedures, and tabular summaries of the laboratory test results are given in Reference 3. Scanning electron microscope photographs of undisturbed specimens were obtained to identify the clay microfabric and classify the samples on the basis of their internal particle arrangement.[3]

Analysis of Laboratory Test Data

102. The phenomenological approach to the verification of the natural microscale mechanisms can be briefly described as an attempt to define the role of the microscale mechanisms by observing the macroscale volume change behavior and correlating it with measurable parameters that generally reflect basic characteristics of the mechanisms. For example, the amount of expansion is dependent on the composition of the soil or more precisely on the amount and type of clay mineral present. The total cation exchange capacity (CEC) is a measurable parameter that generally correlates with the type of mineral present and reflects a characteristic of the clay particle attraction mechanism since it is a

Table 5

Summary of General Site Information and Soil Classifications

Site No.	Site Location	Geologic Formation	Boring/ Sample No. Depth, ft	Natural Moisture Content %	Natural Dry Density pcf	Optimum* Moisture Content %	Maximum* Dry Density pcf	Soil Classification USCS**	AASHTO†
1	Jackson, Miss.	Yazoo	U-2/1/1.0-3.2	43.6	77.5	26.8	78.7	CH	A-7-5 (81)
2	Hattiesburg, Miss.	Hattiesburg	U-2/1/1.0-2.9	26.6	95.2	21.3	101.0	CH	A-7-6 (36)
3	Monroe, La.	Alluvial Material	U-1/1/1.0-2.8	46.0	71.3	30.8	84.5	CH	A-7-5 (68)
4	Lake Charles, La.	Prairie Terrace Material	U-2/1/1.0-3.1	27.3	96.5	15.6	110.8	CH	A-7-6 (33)
5	San Antonio, Tex.	Taylor	U-2/4/3.5-5.1	24.0	95.4	19.6	101.2	CH	A-7-5 (35)
6	Vernon, Tex.	Vale	U-1/4/4.8-7.2	13.8	119.7	17.4	108.9	CL	A-6 (11)
7	Durant, Okla.	Washita	U-2/2/3.5-4.7	15.4	117.9	18.9	105.5	CL	A-7-6 (30)
8	Hennessey, Okla.	Hennessey	U-1/2/3.5-5.6	12.5	124.9	19.4	109.5	CL	A-7-6 (25)
9	Holbrook, Ariz., No. 1, I-40	Chinle	U-2/1/2.5-4.2	9.4	111.5	21.3	97.9	CL	A-6 (10)
10	Holbrook, Ariz., No. 2, SH 180	Chinle	U-2/1/2.0-4.3	16.5	103.6	21.3	97.9	CH	A-7-6 (24)
11	Price, Utah	Mancos	U-1/1/1.2-3.2	6.6	121.2	19.4	106.2	CL	A-7-6 (27)
12	Hays, Kans.	Blue Hill	U-2/1/1.4-3.4	26.2	98.4	26.4	94.7	CH	A-7-6 (58)
13	Ellsworth, Kans.	Graneros	U-2/1/2.0-4.3	16.7	112.8	28.6	85.8	CL	A-7-6 (22)
14	Limon, Colo., No. 1, I-70	Pierre	U-1/5/4.2-6.3	26.4	98.2	22.2	97.7	CH	A-7-6 (35)
15	Limon, Colo., No. 2, I-70	Laramie	U-1/1/3.4-5.0	38.2	78.8	31.8	83.4	CH	A-7-6 (44)
16	Denver, Colo.	Denver	U-3/4/5.7-7.8	14.8	101.1	25.1	95.9	CL	A-6 (10)
17	Newcastle, Wyo., No. 1, SH 16	Mowry	U-2/1/3.0-5.2	26.4	97.5	25.2	96.1	CH	A-7-6 (31)
18	Newcastle, Wyo., No. 2, US 85	Pierre	U-2/1/1.6-3.8	15.8	107.7	22.2	98.5	CH	A-7-6 (25)
19	Billings, Mont.	Bearpaw	U-2/1/3.1-4.6	17.6	110.8	21.1	102.3	CH	A-7-6 (50)
20	Reliance, S. Dak.	Pierre	U-2/1/1.7-3.9	31.4	83.6	38.5	78.5	CH	A-7-5 (42)

* AASHTO T-99, Method A.
** Unified Soil Classification System.
† American Association of State Highway and Transportation Officials.

measure of the clay's ability to hold cations and molecules such as water. The number of relationships between microscale mechanisms, measured variables, and macroscale volume change behavior on which at least a partial understanding has been developed are limited. Examples of these relationships are summarized in Table 6. As indicated in Table 6, the measured variables are limited to physicochemical properties. The laboratory testing was designed to gather as many physical, physicochemical, and mineralogical parameters as feasible for comparison with measured volume change and suspected correlations with microscale mechanisms. Because of the large amount of data available on samples from the 20 sampling sites, it was necessary to apply some statistical comparison procedures to help define the better correlations.[3]

Results of Statistical Comparisons

103. Statistical comparisons between the macroscale volume change characteristics and the three soil suction parameters, i.e., soil suction at natural water content (τ_{nat}) and the A (y-intercept) and B (slope) parameters from the soil suction versus water content relationships, did not correlate when all 20 sampling sites were analyzed as one group. However, when grouped according to physiographic or climatic subgroups, five of the six subgroups had r values (correlation coefficients) greater than 0.7 with at least one of the three parameters. The correlation coefficients with swell as the dependent variable are as follows:

	τ_{nat}	A	B
Atlantic and Gulf Coastal Plains (N = 5)	0.94	0.89	--*
Colorado Plateau (N = 3)	0.73	0.99	0.96
Great Plains (N = 12)	0.77	--*	--*
Humid (N = 4)	0.96	0.94	--*
Moist Subhumid (N = 5)	--*	0.71	0.82
Dry Subhumid (N = 8)	0.51	--*	--*
Semiarid (N = 3)**	0.73	0.99	0.96

* Values $-0.5 < r < +0.5$ not recorded.
** Same as Colorado Plateau.

The value of N indicates the number of data points in the subgroup. The correlation coefficients using swell pressure as the dependent variable are approximately the same with the trend being toward slightly smaller values.

104. The significance of these correlations between macroscale volume change and soil suction and the A and B parameters is twofold: (a) soil suction appears to be a measurable variable that correlates with macroscale behavior and combines the influence of the microscale

Table 6

Relationships Between Microscale Mechanisms,

Measured Variables, and Volume Change

Microscale Mechanism	Measured Variable	Influence on Volume Change
Clay particle attraction	Cation exchange capacity (CEC)	CEC is a measure of the clay particle's surface reactivity or affinity for attracting and orienting water (i.e. double layer water). Increasing CEC generally indicates the presence of active clay minerals (Montmorillonite, etc.) thus more volume change
Cation hydration	Cation exchange capacity	Same basic influence as described above; however, the type of cation present is of more concern
	Exchangeable cations	Measure of the type and amount of cations present on the clay mineral. The thickness of the water layer and the orientation of the water molecules vary with the type of cation. For sodium the thickness is large with decreased orientation. For calcium the thickness is less than for sodium but the orientation is better
Osmotic repulsion	Exchangeable cations and pore fluid cations	The greater the difference of cation concentrations in the double layer water and the pore water the greater the volume change from osmotic repulsion

mechanisms as previously described; and (b) volume change and soil suction are interrelated with physiography and climate.

Results of Comparisons with Soil Suction Data

105. It was pointed out earlier that the total soil suction consists of two components, namely, matrix and osmotic suctions. In addition, the literature review has shown that the matrix component represents the influence of the clay particle attraction and cation hydration mechanisms and the osmotic component represents the osmotic repulsion mechanism. It should be pointed out that no method exists for qualitatively or quantitatively separating the combined influence of the clay particle attraction and cation hydration mechanisms since they are so interdependent on one another. From a practical standpoint, the two mechanisms essentially act as a single influence. Finally, it has previously been described that the thermocouple psychrometer technique generally measures the matrix component of soil suction when developing the soil suction versus water content relationship; however, an osmotic component may be evaluated if the soil suction versus water content relationship becomes horizontal. The osmotic component tends to be dominant at high water contents and may be estimated as the soil suction at the higher water contents when the matrix suction becomes negligible. When an osmotic component is indicated in the soil suction versus water content relationship, the matrix suction may be obtained by subtracting the osmotic component from the soil suction value shown for a specific water content.

106. Examination of the total soil suction versus water content curves for the 20 sampling sites shows that the osmotic component of soil suction is usually very small (less than 1 tsf) or essentially nonexistent. This is evidenced by the fact that the smallest measured soil suction values are less than 1 tsf (generally less than 0.5 tsf) using the psychrometric technique. Two exceptions to this trend are sites 11 and 14 for which the smallest soil suctions measured were 9.2 and 3.5 tsf, respectively. Therefore, the observed curves reflecting the measured matrix suction actually represent the total soil suction.

107. The fact that there was no significant osmotic component of soil suction, with the exception of the two sites mentioned above, agrees with the physicochemical analysis of the specimens. Results of the analysis indicate that 15 of the 20 sampling sites had ratios of the summation of pore fluid cations to the summation of exchangeable cations (Σ PFC/Σ EC) less than one. The five sampling sites with ratios greater than one were sites 11 and 14, as mentioned above, plus sites 2, 6, and 17. The physicochemical testing results are significant from the double-layer theory standpoint since movement of cations into the double layer will not normally occur, which is the cause of osmotic repulsion, unless the pore fluid cation concentration is greater than the cation concentration in the double-layer water (exchangeable cations).

108. Since no significant osmotic component of soil suction was measured for a majority of the samples tested and since the samples represent a diverse cross section of typical expansive materials in the United States, the obvious conclusion concerning the osmotic repulsion mechanism is that it is basically insignificant for these materials and the range of moisture contents tested. If the materials from sites 11 and 14 were located in a climatic region or ambient environment more conducive to higher natural moisture contents, then volume change resulting from osmotic repulsion would be of more consequence.

109. The previous discussions concerning osmotic repulsion and the inherent difficulty in obtaining the individual influence of clay particle attraction and cation hydration essentially reduce the number of microscale mechanisms that cause volume change to one combined influence, namely clay particle attraction and cation hydration. In other words, for the materials tested, the clay particle attraction mechanism plays the major role in determining the volume change behavior. Second in order of influence, but actually showing a combined effect with the clay particle attraction, is the cation hydration. The mechanism showing the least influence on the materials was the osmotic repulsion mechanism.

Summary

110. This part has summarized the basic principles involving the causes of volume change (microscale mechanisms). Specifically it defined the basic definitive relationships associated with soil suction, evaluated an extensive laboratory testing program, and provided an insight into the relative roles of the microscale mechanisms in determining volume change characteristics. Some of the more important points concerning the research task and the roles of the mechanisms are summarized in the following paragraphs.

- a. Although some interesting trends may be inferred using the approach, it is still not successful to infer the microscale mechanism influence from observed macroscopic volume change behavior.

- b. Total soil suction, which is the combination of the matrix and osmotic components, reflects the influence of the three major microscale mechanisms. The matrix suction represents the clay particle attraction and cation hydration mechanisms, which in reality act as one influence since they are so interrelated. The osmotic suction represents the osmotic repulsion mechanism.

- c. The measurement of soil suction using the thermocouple psychrometer technique is simple, quick, and accurate.

The results of the measurements represent a characteristic of the material that can be related to volume change using relatively simple relationships.

d. The results of the laboratory testing program indicated that no significant osmotic component of soil suction was measured on 18 of the 20 samples. In the two exceptions (sites 11 and 14), the osmotic component became significant at the higher moisture contents; i.e., 140 and 125 percent, respectively, of the natural moisture content.

e. Analysis of the results indicates that for the materials and range of moisture contents tested, the clay particle attraction and cation hydration microscale mechanisms play the greatest role in causing volume change. At higher moisture contents and higher cation concentration environments, the osmotic repulsion mechanism provides secondary influence on volume change behavior.

PART IV: EVALUATION OF EXPEDIENT METHODOLOGY FOR IDENTIFICATION OF POTENTIALLY EXPANSIVE SOILS

Review of Identification Techniques

111. The purpose of an identification and/or classification technique for expansive soils is to qualitatively characterize the potential volume change behavior of suspected problem soils. The obvious need for qualitative characterization of potential volume change is two-fold. First, it should serve the purpose of forewarning the engineer during early planning stages of potential problems with expansive soils. Second, it should provide a basis to aid in the decision-making process for dealing with expansive soils in highway subgrades. In other words, the identification/classification technique should provide the necessary information to answer the questions:

a. Do problems with potential expansion exist and, if so, what relative magnitude is expected?

b. Is further testing necessary to use the quantitative prediction techniques? If so, what samples should be tested?

112. In Reference 1, a three-fold categorization of identification and testing techniques was described:

a. Indirect techniques in which one or more of the related intrinsic properties are measured and complemented with experience to provide indicators of potential volume change. These may be grouped according to soil composition; physicochemical, physical, and index properties; and currently used soil classification systems.

b. Direct techniques that involve actual measurement of volume change in an odometer-type testing apparatus. These are generally grouped into swell or swell pressure tests depending on the need for deformation or stress-related data.

c. Combination techniques in which data from the indirect and direct techniques are correlated either directly or by statistical reduction to develop general classifications with regard to probable severity.

Definition of Potential Swell

113. Few, if any, of the published indirect or combination techniques appear to provide a universally applicable technique for several reasons. One of the most obvious inconsistencies in the various methods involves the definition of potential swell. As an example, in one procedure[37] the potential swell is defined as the swell (deformation) of an undisturbed specimen from air dried to saturation under 1-psi surcharge, while in another procedure[38] it is defined as the swell (deformation) of a remolded specimen (optimum moisture content and maximum dry density) under 650 psf (4.5 psi) surcharge. Surcharge pressures generally vary from zero (actually a small seating load) to 1000 psf (7 psi). Initial specimen conditions are either undisturbed or remolded with the remolded specimens having different initial moisture contents and/or dry densities specified. However, for two of the published procedures,[38,39] the specimens tested were molded of artificially prepared soils using mixtures of commercial clay minerals and sand. Another shortcoming of certain published techniques[40,41,42] require time limits or more precisely, the potential swell is defined as the deformation after a set time period of inundation. Add to all these factors the variations in the ambient environmental conditions (soil profile, climate, etc.), which influence the initial condition of the sampled materials as well as the rate of volume change development, and the task of evaluating identification/classification techniques becomes somewhat difficult.

114. The basic definition of potential swell, whether it is for the purpose of identifying and/or classifying the expansive material or for estimating the amount of anticipated volume change, should provide the best simulation of in situ conditions practical. At a minimum, the definition should specify the initial conditions of the specimen, such as water content, dry density, fabric, and structure, as well as the stress conditions relative to the specimen, such as vertical stress and lateral confinement conditions. Ideally, the amount and rate of water applied to the specimen should simulate actual conditions such as groundwater influences and surface infiltration. The current state of the art allows for simulation of most of these conditions, except compromise is required for the lateral confinement and water application conditions. With this in mind, the definition of potential swell that satisfies the largest portion of the field simulation requirements and represents the extreme or worst condition is:

> Potential swell is the equilibrium vertical volume change or deformation from an odometer-type test (i.e., total lateral confinement), expressed as a percent of original height, of an undisturbed specimen from its natural water content and density to a state of saturation under an applied load equivalent to the in situ overburden pressure.

For estimating anticipated volume change, the above definition should be amended to reflect the final stress conditions such as applied load from the pavement or structure. However, for the purpose of evaluating identification/classification techniques, the definition will be used as stated; that is, the volume change from natural conditions to saturation under actual overburden stresses.

Summary of Indirect Techniques

115. In Reference 1, Table 5 described five indicator groups that comprise the indirect techniques for identification/classification of potentially expansive soils. The indicator groups are soil composition, physicochemical properties, physical properties, index properties, and soil classification. The combination techniques are solely dependent on correlation of index properties with measured volume change. In fact, the combination techniques are simply extensions of the index property indicator group so that more factors could be considered. This point is made to simplify subsequent discussions.

Soil composition indicator group

116. The soil composition indicator group is totally concerned with the determination of the type and, to a lesser extent, amount of clay mineral present in the soil. The identification of the clay mineral is positive indication of the potential problem; however, the clay mineralogy does not present an adequate correlation with respect to the amount of volume change that is likely to occur. A point that is of more concern from the practical operational standpoint is that most of the methodology (i.e., equipment and trained personnel) is not readily available on a cost-effective basis for routine use. In summary, the determination of the clay mineralogy is a reasonably accurate indicator that problems could exist; however, very little can be determined about the relative magnitude of the problem. This lack of correlation with potential swell and the logistics problems with routine use have hampered and will continue to hamper the use of the methods included in the soil composition indicator group.

Physicochemical property indicator group

117. The physicochemical property indicator group provides simpler testing procedures than the soil composition group; however, the simplicity is offset by poor correlations with measured volume change. The published relationships show, at best, a general qualitative indication of potential swell. In other words, the magnitude of potential swell generally increases with increasing CEC and is influenced by the type and amount of cation present.

Physical property indicator group

118. The physical property indicator group provides several properties that have significant influence on the volume change characteristic, but individually the properties provide little or no indication of potential swell from a relative magnitude standpoint. The colloidal content has been used in combination with other index properties[37] to categorize potential swell. The other properties--specific surface area, soil fabric, and soil structure--influence the amount and rate of volume change. The influences are explained in more detail elsewhere.[1]

Index property indicator group

119. The most widely used indicator group for identification/classification of expansive soils is the index properties group. The major reasons for the popularity of this group are (a) the practicality from the standpoint that most of the properties involved are routinely determined by all State Highway Agencies and (b) experience showing that potential swell correlates reasonably well with several of the simple index properties. In most cases, the index property group variables, i.e., Atterberg limits or shrinkage properties, are correlated with past experiences and used individually to categorize the potential problem from expansive soils. Examples of these simple categorizations are given in Table 5, Reference 1. On occasions where measured potential swell values are available, the index properties are used individually or combined to correlate with potential swell. The result is several multiproperty categorizations of relative magnitude of potential volume change. This constitutes the combination category previously discussed. This property indicator group includes the largest number of published identification/classification techniques that will be considered in the evaluation of methodology.

Soil classification indicator group

120. The soil classification system indicator group is another group that deals in simple properties of the soils but does not provide more than a general indication that a problem might exist. In the American Association of State Highway and Transportation Officials (AASHTO) classification system, the A-6 and A-7 subgrade groups constitute a majority of the potentially expansive soils. In the Unified Soil Classification System (USCS), the CL and CH categories generally cover the range of potential expansivity with the possibility of some MH soils showing expansive characteristics to a lesser degree. The Soil Conservation Service Classification System or Soil Taxonomy[43-46] as it is now called, is the most detailed classification system in current use. Soil taxonomy uses many of the basic properties as well as temperature and moisture regimes and climate to describe soils. 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

which 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 is generally considered to be potentially expansive.

Evaluation of Identification Techniques

121. The information presented in Reference 1 and discussions in the preceding section indicate that the index property indicator group most effectively balances simplicity and practicality with reasonable accuracy for identification/classification purposes. Within the index property group and the combination category, which is merely an extension of the index property group, 17 published techniques or criteria were selected for evaluation using the data collected for this study. Since it was impossible to duplicate the conditions of development of each of the techniques, the evaluation process is based on applying the techniques to data collected during this study and comparing the relative magnitudes with those determined using the previously presented definition of potential swell. Additional published techniques were omitted from the comparisons because of imposed time limits placed on potential swell measurements.[40-42]

Selection of Identification Techniques

122. The identification/classification techniques represented by the index property indicator were selected generally on the basis of their simplicity and the indicated correlations with measured volume change. With regard to field experience, several of the techniques have been used considerably with reasonable success. The techniques selected for evaluation are described in detail in Reference 4 and are tabulated below along with appropriate reference, in parenthesis, and alphabetic characters to simplify subsequent discussions.

Louisiana Department of Transportation (47)	LDOT
Kansas Highway Commission (48)	KHC
Raman (49)	Raman
Sowers (50 and 51)	Sowers
Dakshanamurthy and Raman (52)	D&R
Anderson and Thomson (53)	A&T
Ranganatham and Satyanarayana (54)	R&S
Saito and Miki (55)	S&M
U. S. Bureau of Reclamation (37)	USBR
Altmeyer (56)	Altmeyer
Seed, Woodward, and Lundgren (38)	SWL
Chen (57)	Chen

(Continued)

Vijayvergiya and Ghazzaly (58)	V&G
Vijayvergiya and Sullivan (59)	V&S
Sorochan (60)	Sorochan
Nayak and Christensen (61)	N&C
Komornik and David (62)	K&D

Field sampling
and laboratory testing

123. The field sampling and laboratory testing programs were described in the previous section on the verification of microscale mechanisms. The only exception to the results of the testing occurred during the overburden swell testing program where the overburden pressure was supposed to reflect in situ conditions of wet density and depth to test specimen, thus yielding a measured deformation corresponding to the potential swell as defined earlier. However, because of an error in testing assignment, a constant overburden pressure of 0.28 tsf was used for all specimens. For test specimens whose actual and tested overburden pressures did not agree, a second specimen was tested. The results of the reruns and those tests not requiring additional testing are given in Reference 4. With the reruns completed, all of the overburden swell test data conform with the definition of potential swell. Briefly, the overburden swell test procedure begins by applying a very small seating load followed by application of the overburden pressure and inundation of the specimen. The specimen is allowed to swell to equilibrium (end of swell condition), then the specimen is consolidated in increments to the void ratio corresponding to overburden conditions and rebounded in decrements to the seating load (end of test condition).

Analysis of the data

124. The evaluation of expedient methodology for identification/classification of expansive soils involved a two-fold effort. First, the data collected during the laboratory testing program were used in conjunction with the published identification/classification techniques to compare the indicated qualitative magnitude of swell with the measured potential swell as previously defined and measured in the overburden swell test. Second, the measured potential swell and laboratory data were analyzed using the statistical analysis program to obtain the best correlations between the two groups of data.

Establishment of
potential swell categories

125. It was stated earlier that the evaluation of the published techniques is basically an evaluation of the techniques as they are applied rather than how they were developed since there was a large variation in the definition of potential swell used. Evaluation of the accuracy of the published techniques requires a basis or standard for comparison, or, more precisely, a categorization of the potential swell as previously defined and measured in the laboratory testing program.

126. For the 20 samples tested, the swell varied from 0.01 to 12.7 percent, with the majority of the measured values between 0.14 and 3.65 percent. During previous discussions with representatives of the State Highway Agencies it was indicated that a two- or three-category classification of potential swell would be preferable to the four- to six-category classifications published in the literature. With this in mind, the measured potential swell was divided into three categories. The three categories and their corresponding potential swell limits are as follows:

Potential Swell %	Classification
<0.5	Low
0.5-1.5	Marginal
>1.5	High

For discussion purposes, this classification will be denoted as the U. S. Army Engineer Waterways Experiment Station (WES) classification throughout the remainder of this section. These ranges of potential swell seem low as compared with some of the published techniques; however, they are consistent with identification/classification techniques that used comparable magnitudes of applied load in the potential swell testing program. In addition, this range of catagories divides the 20 samples tested into three groups of approximately the same number. The high category indicates that the material has the maximum potential for volume change and that the characteristic properties (i.e., moisture content, density, fabric, structure, etc.) are conducive to the occurrence of volume change when moisture is made available. The low category indicates that the material has minimal potential for volume change and that the characteristic properties inhibit volume change. The marginal category indicates the presence of moderate to maximum potential for volume change, but the characteristic properties are not consistent with the high category, and the resulting volume change is generally less than anticipated depending on the sensitivity of the material with respect to variations in the characteristic properties.

Discussion of results

127. Careful examination of the results showed that no published identification/classification technique provides a generally applicable methodology for identifying and qualitatively classifying potential swell. However, some of the techniques provide reasonably accurate and consistent indication of problem conditions. For example, the LDOT procedure agrees with the WES categorization for classification of potential swell of 10 of the 20 sampling sites and is conservative in its classification for nine of the remaining 10 samples. Ideally, the best technique should result in a majority of responses in the "agreement" and "conservative" columns and as few as possible in the "nonconservative" column when compared with the WES categorization. The higher the percentage of "agreements" versus the percentage of "conservative" responses, the better the technique.

128. Of the 17 techniques evaluated, four balance accuracy and conservatism best, namely, LDOT, Sowers, Altmeyer, and V&G. All four of these techniques involve Atterberg limits in some form, either individually or in combination. Within this group of techniques, the relative accuracy and conservatism is best for the LDOT procedure and decreases in the order of V&G, Sowers, and Altmeyer. In other words, the most consistent indicators of potential swell are first, the liquid limit (LL) and plasticity index (PI); second, the LL and natural water content (w_i) combined; third, shrinkage limit (SL) and PI; and finally, the SL and linear shrinkage (BLS). This is consistent with the analysis of the WES categorization and laboratory data presented in Reference 4.

Analysis of WES categorization and laboratory data

129. The second portion of the evaluation of expedient identification/classification methodology involved a comparison by statistical analysis of the WES categorization of potential swell with the laboratory data, the purpose being to verify the evaluation of published techniques and determine if more sensitive indicators of potential swell are available. Based on the results of the statistical comparisons between measured swell and the other physical and physicochemical laboratory data, five properties yielded significant correlation coefficients, namely, LL, PI, shrinkage index (SI = LL - SL), BLS, and natural soil suction (τ_{nat}). In other words, these five variables provide the best indicators of potential swell for the samples tested.

Establishment of criteria for identifying potential swell

130. Although four of the published identification/classification techniques were shown to be reasonably accurate in achieving their purpose of identifying problem soils, it was decided to modify the categories and the relative ranges of the properties included in the categories to conform better with the definition of potential swell presented in this report. Two alternatives were available for establishing the criteria; one was to develop a general criterion based on all the data collected and analyzed as a combined group and the other was to develop individual criteria for either the physiographic or climatic groups. The former alternative (i.e., one general criterion) was selected because the latter alternative posed several problems.[4]

131. In establishing the general criteria, the previously defined classifications of potential swell (low, marginal, high) were combined with the measured potential swell and physical properties to more accurately or at least conservatively identify the problem soils. Various combinations of ranges of the indicator properties were tried in an effort to optimize the accuracy and conservatism of the identification/classification criteria. The following ranges were found to provide the most accurate classification:

LL, %	PI, %	τ_{nat}, tsf	Potential Swell, %	Potential Swell Classification
>60	>35	>4	>1.5	High
50-60	25-35	1.5-4	0.5-1.5	Marginal
<50	<25	<1.5	<0.5	Low

The SI was included in the trial criteria with the same ranges as the PI and did not reduce or increase the accuracy of the classification; therefore, it was not included in the final criteria.

Summary

132. This part of the report has summarized the state of the art with respect to expedient methodology for identifying and qualitatively classifying expansive soils. It presented a more comprehensive definition of potential swell and, through the use of laboratory data and statistical analysis procedures, the techniques that constitute the state of the art are evaluated and slightly modified to provide a more accurate procedure for identification/classification of expansive soils. Some of the more important points concerning the evaluation of published identification/classification techniques and analysis of laboratory data are summarized in the following paragraphs.

133. The definition of potential swell that satisfies the largest portion of the field simulation requirements is:

> Potential swell is the equilibrium vertical volume change or deformation for an odometer-type test (i.e., total lateral confinement), expressed as a percent of original height, of an undisturbed specimen from its natural water content and density to a state of saturation under an applied load equivalent to the in situ overburden pressure.

This definition yields the volume change potential of a material when the initial specimen and stress conditions are identical to the in situ conditions. For predicting anticipated volume change, as will be discussed in a subsequent part, the aforementioned definition should be amended to reflect the anticipated final stress conditions such as applied load from the pavement or structure.

134. The categorization and classification of potential swell denoted as the WES classification and given in the following tabulation is a better classification of potential swell with regard to in situ conditions.

Potential Swell %	Classification
<0.5	Low
0.5-1.5	Marginal
>1.5	High

The ranges of potential swell are low as compared with many of the published techniques; however, they are consistent with identification/classification techniques that used comparable magnitudes of applied load in the potential swell testing programs, i.e., Altmeyer.[56]

135. Evaluation of the 17 published techniques selected for consideration, using the previously described definition and classification of potential swell, indicated that four of the techniques balance both accuracy and conservatism. These techniques, in order of decreasing accuracy and conservatism are LDOT, V&G, Sowers, and Altmeyer. In other words, the evaluation shows that the most consistent indicators of potential swell are first, the LL and PI; second, the LL and w_i combined; third, SL and PI; and finally, the SL and linear shrinkage (BLS).

136. Analysis of the laboratory data using correlation studies of potential swell versus 31 independent variables showed that the most consistent indicators of potential swell are LL, PI, SI, BLS, and τ_{nat}. This is consistent with the results of the evaluation of published identification/classification techniques.

137. Using the WES classification of potential swell, the categories of indicator properties that maximize the accuracy and conservatism of the criteria for identifying and qualitatively classifying expansive soils are as follows:

LL, %	PI, %	τ_{nat}, tsf	Potential Swell, %	Potential Swell Classification
>60	>35	>4	>1.5	High
50-60	25-35	1.5-4	0.5-1.5	Marginal
<50	<25	<1.5	<0.5	Low

PART V: EVALUATION OF TESTING AND PREDICTION METHODOLOGY

138. Once a potentially expansive soil has been identified and classified in the marginal or high categories of potential swell, quantitative characterization of the expansive material becomes a necessity in order to accurately estimate the amount and rate of anticipated volume change. Accurate estimates of volume change are a requisite for the selection of efficient treatment alternatives or preparation of adequate designs. Techniques available for quantitative characterization of expansive soils fall into three categories, namely, odometer tests, soil suction tests, and empirical methodology. In precious discussions,[1] the odometer and soil suction tests were considered together under the direct laboratory techniques, and the empirical methodology was considered under the combination of identification and testing techniques. However, since this part of the report deals with quantitatively estimating anticipated volume change, the odometer, soil suction, and empirical catagorization provides a better basis for discussion.

Odometer Tests and Prediction Procedures

139. 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 simplest 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.

140. 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., multistory building), 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.

Review of odometer
test and prediction procedures

141. The swell and swell pressure test procedures previously described are very general in nature and give an indication of the concept behind odometer testing. The fact is, no standard procedure currently exists for conducting odometer swell tests. The major points of disagreement concerning odometer tests begin with the controversy over whether to run the test for identification purposes (i.e., standard surcharge pressure for all tests) or for design purposes (i.e., surcharge appropriate to design situation) and continue with loading sequence and definition and interpretation of test data. To get a better understanding of the differences in published odometer test procedures, the following paragraphs present a summary of several of the published techniques.

142. Navy Method.[63] This test procedure involves the collection of swell data from several specimens tested under different surcharge pressures. The resulting data are used to prepare a depth (i.e., overburden pressure) versus percent swell curve. From this curve and the soil profile, a depth versus total swell curve (area under depth versus percent swell curve) can be calculated, and the amount of swell for a given depth of active zone or depth of undercut to minimize swell can be determined. An example of the two curves and the procedure for interpreting them is given in Figure 11.

143. Noble Method.[64] This is an odometer test procedure developed on a statically compacted Canadian soil. The results are presented as an empirical relationship which has the form

$$V.C. = \left(2.92 \frac{1}{w_i} - 1\right) \log \frac{0.00385}{p(w_i)^{5.3}} \qquad (7)$$

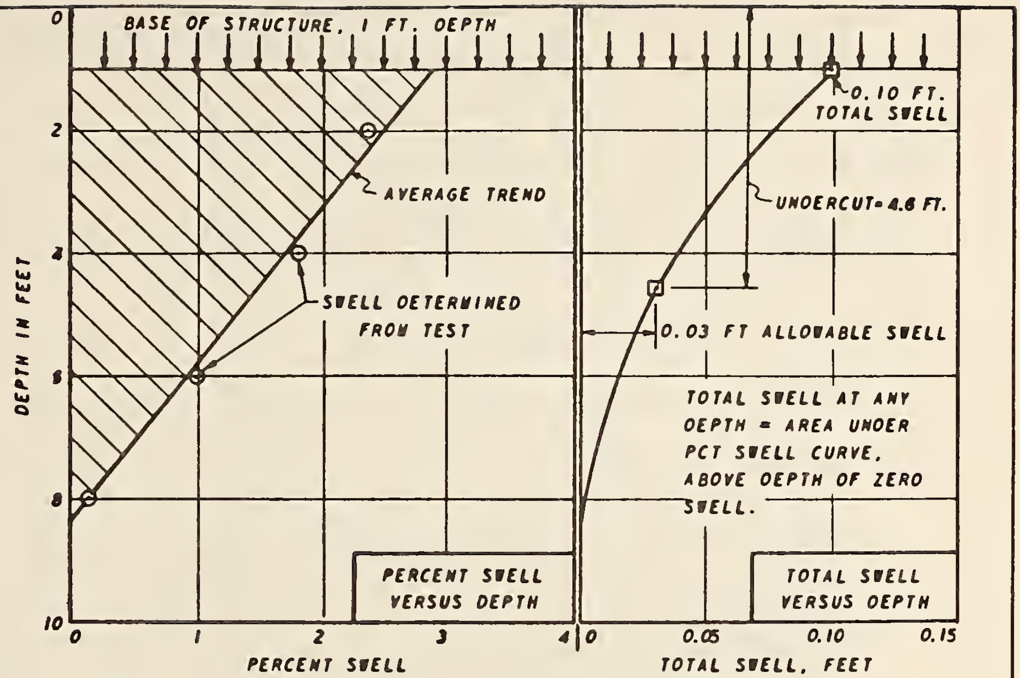
where

V.C. = volume change, percent

w_i = initial moisture content, percent

p = surcharge pressure, kg/cm^2

when plotted, this equation yields a family of curves relating volume change to surcharge pressure for various initial moisture contents, Figure 12. To apply the testing and prediction procedure, four samples must be tested (two initial moisture contents under two surcharge pressures), which help define the numerical constants in the equation for



MATERIALS INVESTIGATED ARE CLAYS, HIGHLY OVERCONSOLIDATED BY CAPILLARY STRESSES THAT ARE EFFECTIVE PRIOR TO THE CONSTRUCTION OF THE STRUCTURE UPON THEM.

PROCEDURE FOR ESTIMATING TOTAL SWELL UNDER STRUCTURE LOAD.

1. OBTAIN REPRESENTATIVE UNDISTURBED SAMPLES OF THE SHALLOW CLAY STRATUM AT A TIME WHEN CAPILLARY STRESSES ARE EFFECTIVE; I.E., WHEN NOT FLOODED OR SUBJECTED TO HEAVY RAIN.
2. LOAD SPECIMENS (AT NATURAL WATER CONTENT) IN CONSOLIDOMETER UNDER A PRESSURE EQUAL TO THE ULTIMATE VALUE OF OVERBURDEN FOR HIGH GROUND WATER, PLUS WEIGHT OF STRUCTURE. ADD WATER TO SATURATE AND MEASURE SWELL.
3. COMPUTE FINAL SWELL IN TERMS OF PERCENT OF ORIGINAL SAMPLE HEIGHT AND PLOT SWELL VERSUS DEPTH, AS IN THE LEFT PANEL.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO THE AREA UNDER THE PERCENT SWELL VERSUS DEPTH CURVE. FOR THE ABOVE EXAMPLE:

   TOTAL SWELL = $1/2\,(8.2 - 1.0) \times 2.8/100 = 0.10$ FT.

PROCEDURE FOR ESTIMATING UNDERCUT NECESSARY TO REDUCE SWELL TO AN ALLOWABLE VALUE.

1. FROM PERCENT SWELL VERSUS DEPTH CURVE PLOT RELATIONSHIP OF TOTAL SWELL VERSUS DEPTH AT THE RIGHT. TOTAL SWELL AT ANY DEPTH EQUALS AREA UNDER THE CURVE AT LEFT, INTEGRATED UPWARD FROM THE DEPTH OF ZERO SWELL.
2. FOR A GIVEN ALLOWABLE VALUE OF SWELL, READ THE AMOUNT OF UNDERCUT NECESSARY FROM THE TOTAL SWELL VERSUS DEPTH CURVE. FOR EXAMPLE, FOR AN ALLOWABLE SWELL OF 0.03 FT, UNDERCUT REQUIRED = 4.6 FT. UNDERCUT CLAY IS REPLACED BY AN EQUAL THICKNESS OF NONSWELLING COMPACTED FILL.

Figure 11. Example computation of swell of desiccated clays (from Reference 63)

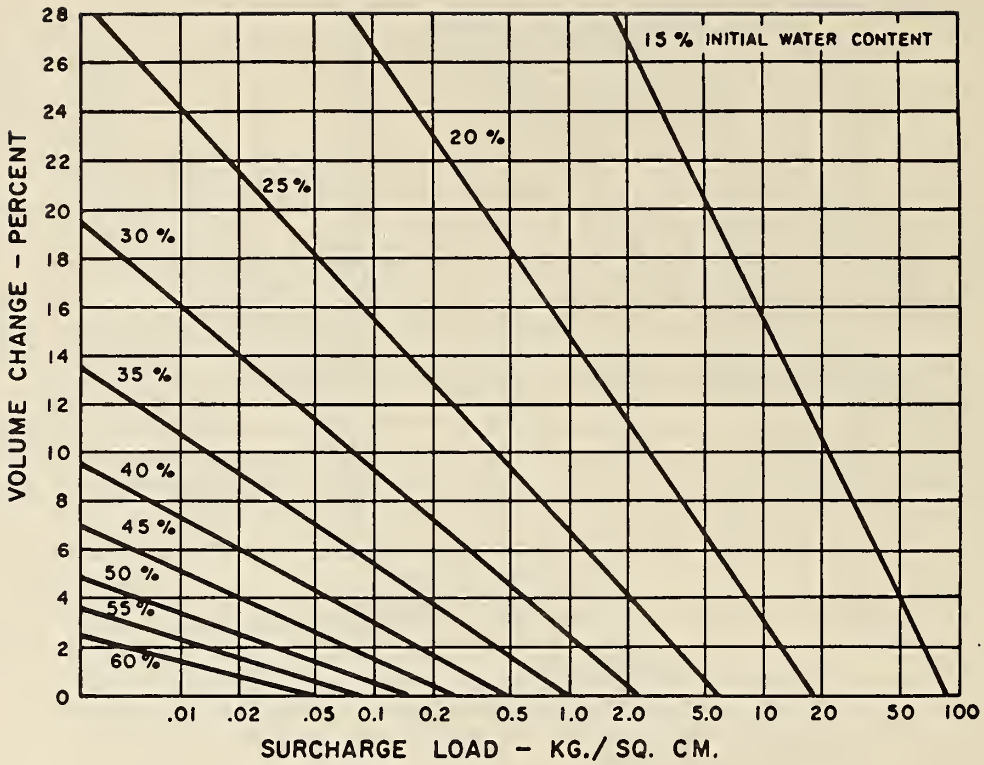
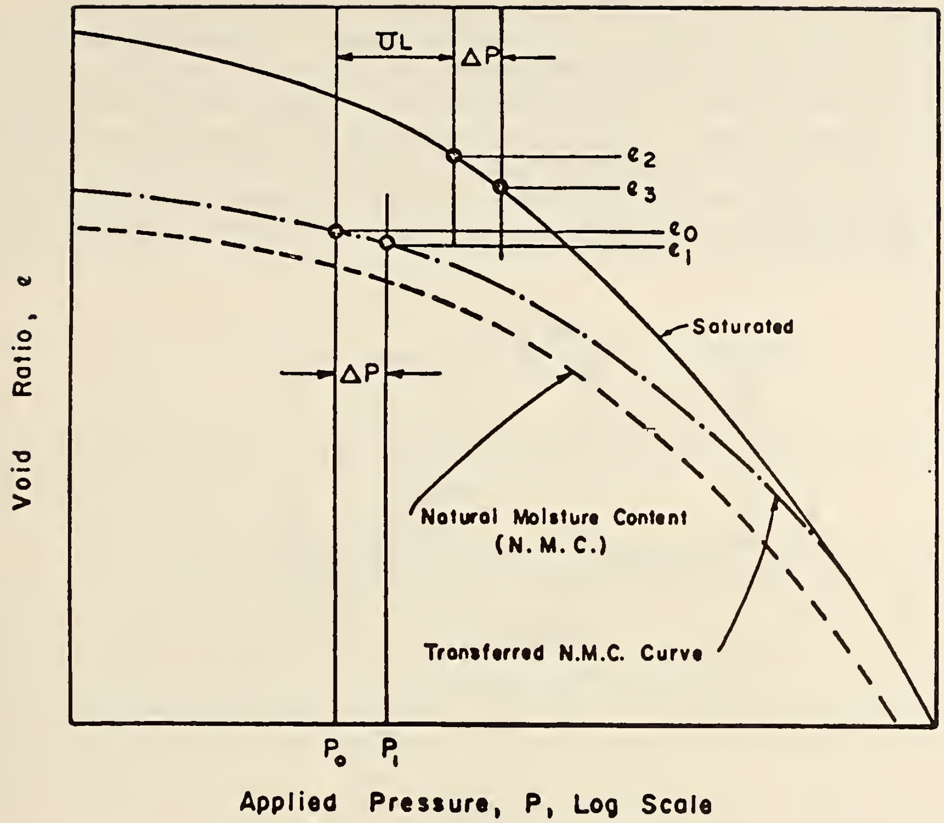


Figure 12. Volume change versus surcharge load from empirical (from Reference 64)

the given soil. With the constants determined, a family of curves similar to Figure 12 can be plotted, and various initial conditions (i.e., moisture content and load) can be evaluated.

144. Double odometer.[65-68] An odometer test in which two adjacent undisturbed samples are subjected to different loading conditions. One speciment is lightly loaded (0.01 tsf) for approximately 30 min, inundated, allowed to swell to equilibrium, and then consolidated using routine consolidation test procedures. The second specimen is treated similarly, except it is not inundated and the routine consolidation test is conducted on the specimen at its natural moisture content. The resulting e versus log p curves are adjusted to bring the virgin compression portions of the curves coincident, Figure 13a. An example of the heave calculations using the double odometer data is shown in Figure 13b.

145. Improved simple odometer.[69] As a result of criticism of the double odometer test, some modifications were made to simplify the testing procedure. The result was an odometer test on a single undisturbed



(a)

1	2	3	4	5	6	Void Ratio				11	12	13
Layer No.	Depths at top and bottom of layer feet.	Depth of double Oedom. test feet.	P_o at mean depth ton/sq. ft.	ΔP at mean depth ton/sq. ft.	σ_L at mean depth ton/sq. ft.	7 e_0	8 e_1	9 e_2	10 e_3	Heave of unloaded surface $\frac{H}{1+e_0}(e_0-e_2)$ in.	Settlement under buildings, ins. $\frac{H}{1+e_0}(e_0-e_1)$ in.	Nett heave under building $\frac{H}{1+e_0}(e_0-e_3)$ in.
1	4.00-6.56	5.25	0.30	0.10	0.97	0.419	0.418	0.492	0.481	1.585	0.021	1.342
2	6.56-8.87	7.87	0.45	0.07	0.89	0.428	0.427	0.460	0.455	0.623	0.019	0.525
3	8.87-10.80	9.87	0.59	0.06	0.84	0.418	0.417	0.449	0.447	0.506	0.016	0.474
4	10.80-13.35	11.74	0.73	0.04	0.78	0.544	0.543	0.563	0.562	0.362	0.020	0.343
5	13.35-18.81	14.85	0.97	0.04	0.70	0.626	0.625	0.640	0.639	0.565	0.040	0.524
6	18.81-	18.87	1.21	0.04	0.59	0.523	0.522	0.524	0.523	0.017	0.017	0
									TOTALS:	3.658	0.133	3.208

(b)

Figure 13. Example of e versus log P curves showing adjustment to make the straight line portions coincident, (a), and typical heave calculations, (b) (from Reference 65)

specimen loaded to its in situ overburden pressure, unloaded to a prescribed seating load, inundated, allowed to swell to equilibrium, and then consolidated using routine procedures, Figure 14a. A further simplification was made based on experience when it was noted that the slope of the curve between $(e_o)_n$ and $e_o p_o$ was so small that minimal error was introduced by assuming $(e_o)_n$ to be equal to e_o at p_o, therefore that portion of the test was eliminated, Figure 14b. The analytical procedures for calculating heave are identical to those for the double odometer test.

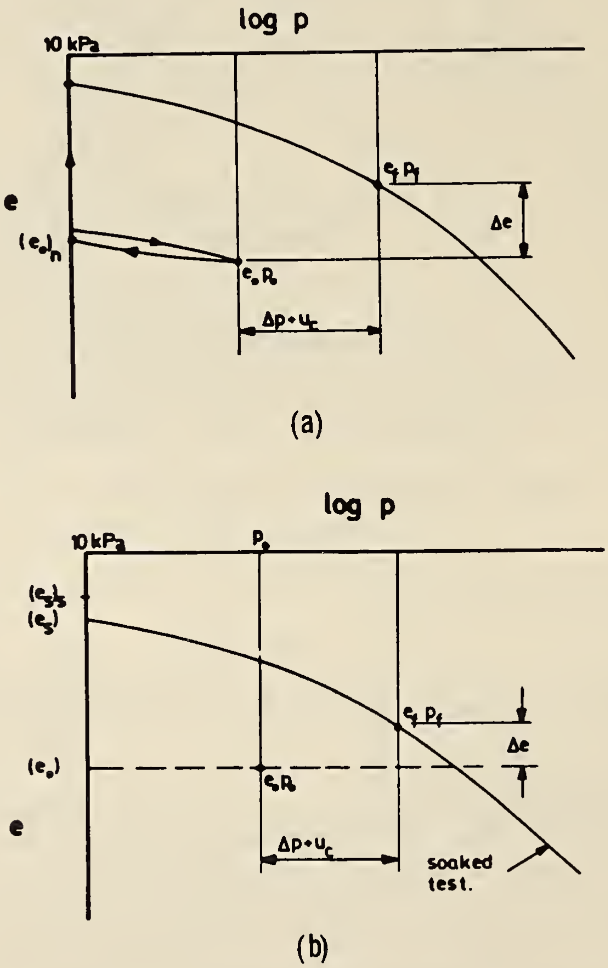


Figure 14. Typical e versus log p curves for improved simple odometer test (from Reference 69)

146. Sampson, Schuster and Budge.[70] This odometer testing and prediction procedure is similar in principle to the double odometer test; however, the actual testing procedures are slightly different. One specimen is loaded to a pressure sufficient to inhibit any volume change when inundated. After equilibrium is established, the specimen

is inundated, unloaded to a minimal seating load, and allowed to swell to equilibrium, curve 1 in Figure 15. The second of the companion specimens is loaded to its in situ overburden condition, inundated, and allowed to swell to equilibrium, curve 2 in Figure 15. The purpose of two curves is to obtain the volume change due to overburden removal and water access, Δe_o , and the volume change due to effective stress reduction as field pore pressure increases after paving, Δe_s . The total volume change, Δe , that will occur during and subsequent to construction is

$$\Delta e = \Delta e_o + \Delta e_s \tag{8}$$

An estimate of anticipated volume change is then made using the reverse consolidation theory.

147. Sullivan and McClelland.[71] This odometer test procedure is, in principle, a constant volume swell pressure test. The test procedure involves a single specimen loaded to its in situ overburden pressure, inundated, and incremental load applied to maintain constant volume. The swell pressure is noted, and the specimen is unloaded in decrements to some small seating load, Figure 16. The variation of this procedure from the generally accepted procedure involves an effective stress interpretation of the e versus log pressure data, as shown in Figure 16. The swell pressure is equivalent to the soil suction and the initial and final effective stress conditions are determined by applying a modified version of the classical effective stress equation:

$$\sigma' = \sigma - \chi p'' \tag{9}$$

where

σ' = effective stress

σ = total stress

χ = parameter representing the portion of the soil suction that contributes to the effective stress[72,73]

p'' = negative pore water pressure or soil suction

An example of the application of the laboratory testing procedure is shown in Figure 17. Calculation of anticipated heave for a building being monitored in central Texas is given in the following tabulation:

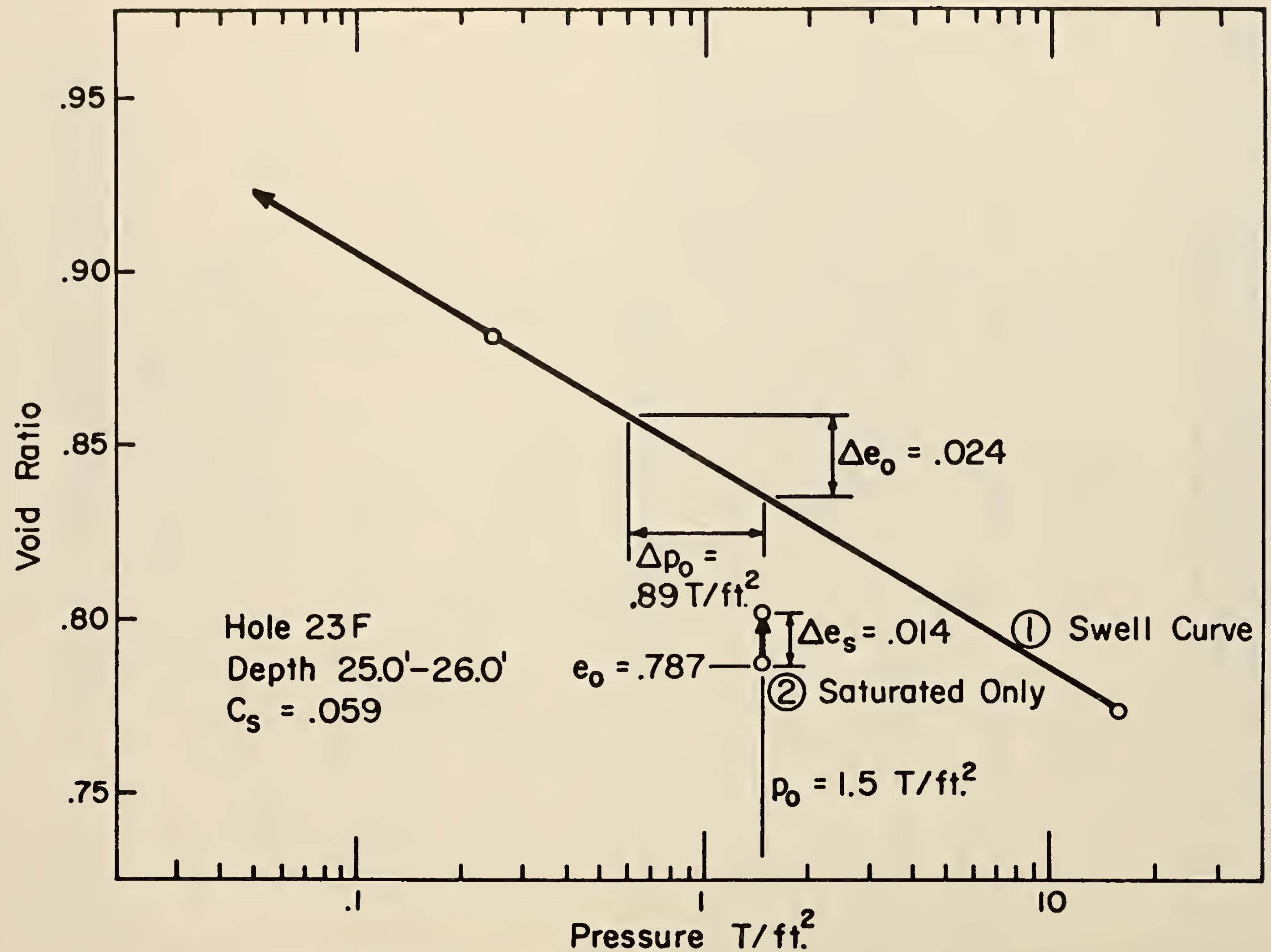


Figure 15. Examples of e versus log p curves using Sampson, et al. testing procedure (from Reference 70)

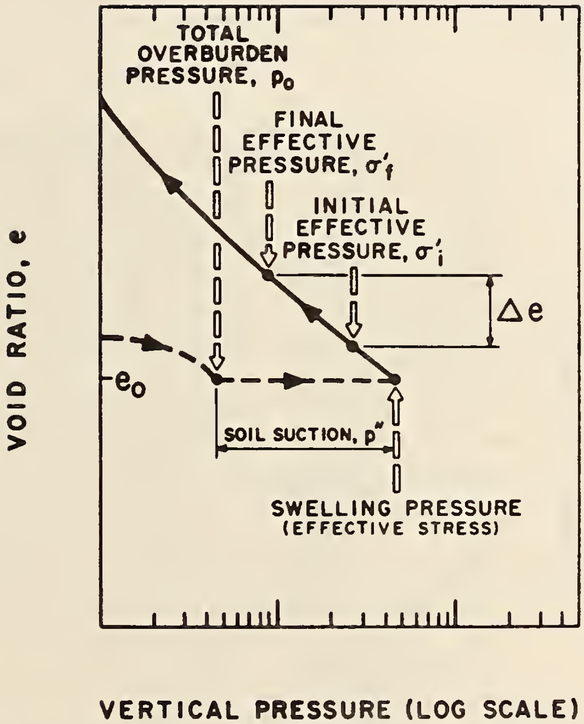
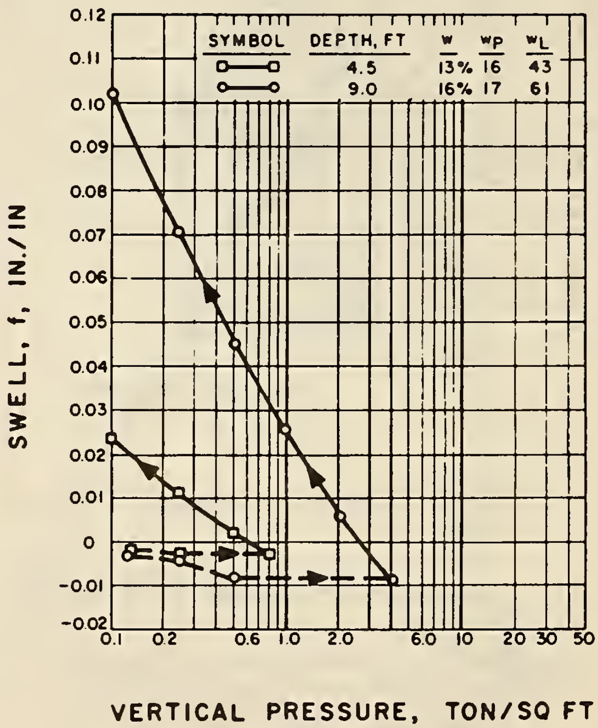


Figure 16. Laboratory relationship between void ratio and effective pressures (from Reference 71)

Sample depth, ft	4.5	9.0
Material	Sandy Clay	Clay
Initial moisture content, %	13	16
Unit dry weight, pcf	117	115
Plastic limit	16	17
Liquid limit	43	60
Initial void ratio	0.447	0.469
Initial saturation, %	79	92
Total overburden pressure, tsf	0.25	0.54
Swelling pressure, tsf	0.8	4.1
Expansion under 0.1 tsf pressure,%	2.7	11.0

(a)



(b)

Figure 17. Laboratory results of constant volume swell tests; (a) tabular summary of soil properties and (b) swell versus log pressure relationship (from Reference 71)

	Sandy Clay Stratum	Clay Stratum
Thickness of stratum, H, ft	5	6
Sample depth, ft	4.5	9.0
Overburden pressure at sample depth, tsf	0.25	0.54
Initial suction, p_i'', tsf	0.55	3.60
Effective stress parameter, χ	0.82	0.42
Initial stress, $\sigma_i' = p_o + \chi p_i''$	0.70	2.05
Final suction (assuming static equilibrium), p_f'', tsf	0.39	0.24
Effective stress parameter, χ	0.86	0.94
<u>Neglecting floor loads</u>:		
Final stress, $\sigma_f' = p_o + \chi p_f''$	0.59	0.77
Swelling increment, Δf, in./in. for stress change $\Delta\sigma' = \sigma_i' - \sigma_f'$	0.003	0.026
Heave, $\Delta H = H \dfrac{\Delta f}{1 - f_o}$, in.	0.18	1.86
<u>With floor load, $\Delta p = 0.1$ tsf</u>:		
Final stress, $\sigma_f' = p_o + \Delta p + \chi p''_f$	0.69	0.87
Swell increment, Δf, in./in. for stress change $\Delta\sigma' = \sigma_i' - \sigma_f'$	0.001	0.024
Heave, $\Delta H = H \dfrac{\Delta f}{1 - f_o}$, in.	0.006	1.73

148. <u>Komornik, Wiseman, and Ben-Yaacob.</u>[74] This odometer test procedure involves the measurement of swell data on several specimens representing various depths in the active zone loaded to their in situ overburden pressure plus an additional surcharge equal to the anticipated equilibrium suction. The procedure requires estimates of the depth of the active zone and the equilibrium suction profile in the active zone. The authors suggest that the equilibrium suction be estimated by establishing the swell pressure minus overburden pressure versus water content/plastic limit ratio relationship. The difference between swell pressure and overburden pressure represents the soil suction. Equilibrium values of the water content/plastic limit ratio are generally between 1.1 and 1.3. The percent swell from the odometer tests is plotted versus depth. The area under this curve is the surface heave.

149. <u>Orange County Expansion Index.</u>[42] This is a odometer test

developed and specified for remolded soils. In actuality, it is nothing more than an identification/classification technique. A specimen is dynamically compacted to conditions resulting in a degree of saturation of 50 percent ($\pm$ 1 percent). The specimen is trimmed and placed in an odometer and a 1-psi surcharge is placed on the specimen. The specimen is inundated and allowed to swell to equilibrium. The expansion index (EI) is calculated using the following formula:

$$EI = 1000\, \Delta h\, F \qquad (10)$$

where

Δh = change in height of sample

F = fraction of sample passing U. S. No. 4 sieve

Quantitatively, the results could be used to estimate heave; however, it is generally used to classify potential swell.

150. Holtz.[37,75] This odometer test procedure involves the testing of two sepcimens to provide limits between which the volume change of additional samples can be estimated. One specimen (undisturbed or remolded) is inundated, and the load increased to maintain constant volume until an equilibrium is reached; then the load is decreased in decrements to a nominal seating load, Curve B, Figure 18. The second specimen is lightly loaded, inundated, and allowed to swell to equilibrium. The specimen is then consolidated in increments to the maximum pressure measured on specimen number one, Curve A, Figure 18. Curve C, Figure 18, is an estimate of the behavior of a specimen loaded to an intermediate load, inundated, and allowed to expand. Calculations for estimating anticipated heave are based on the reverse consolidation theory.

151. Mississippi State Highway Department.[76-79] This odometer test involves collecting swell data on undisturbed or remolded samples. A specimen is trimmed and prepared for testing, loaded to its field overburden pressure, inundated, and allowed to swell to equilibrium (point 1 to point 2), Figure 19. The specimen is unloaded (point 2 to point 3), then a normal consolidation test is run on the specimen. The no volume change load is determined by extending line 2-3 until it crosses the field void ratio (e_f). Calculation of heave is accomplished through the reverse consolidation theory. An alternate method is often used that is based on experience and reduces testing time. If, after testing several samples, the two rebound curves are parallel as they have been for many of the samples tested, the procedure is modified and point 3 is eliminated. In other words, the normal consolidation curve is run after swell equilibrium is obtained. The no volume change load is then determined by drawing a line parallel to line 5-6 through point 2 until it crosses the field overburden pressure.

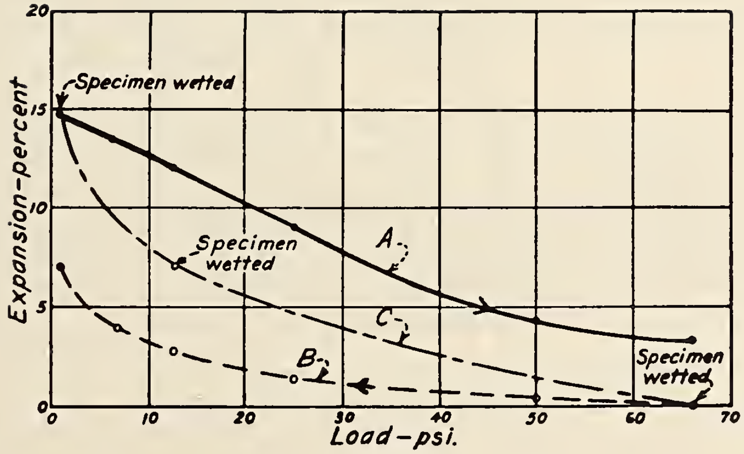
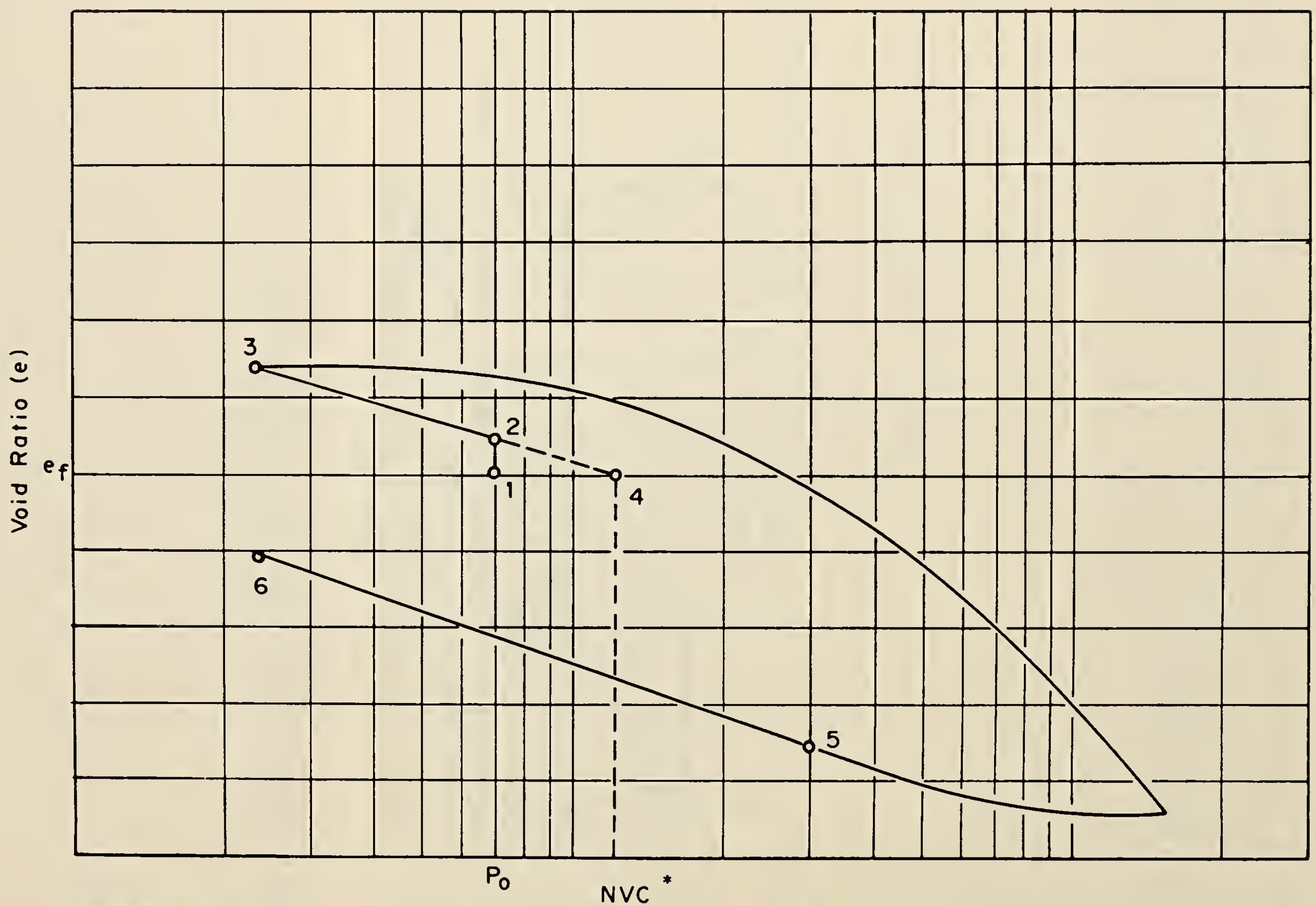


Figure 18. Load-expansion curves for two undisturbed expansive soil specimens (from Reference 75)



*NO VOLUME CHANGE

Figure 19. Typical e versus log P relationship using Mississippi State Highway Department test and interpretation procedure (from Reference 76)

152. <u>Third cycle expansion pressure test.</u>[80] This is a test procedure similar in nature to an odometer test; however, it is conducted in conjunction with the R-value test, which is a parameter used to indicate the stability (strength) of a material for highway design purposes. The procedure involves molding a series of samples identical to those used for the R-value test. The specimens are tested in their molds by placing a deflection bar and dial gauge over the specimen and inundating. The specimen is allowed to swell (develop swell pressure) and the pressure is relieved twice. At the end of the third standing period, the dial reading and associated expansion pressure are recorded. The data are then used in conjunction with the R-value test to determine cover requirements (thickness) and construction controls to reduce pavement distortion from expansive soils.

153. <u>Volumenometer.</u>[81] This is a test procedure using a device that is a cross between an odometer and triaxial cell. The test procedure involves air drying undisturbed samples, placing them in the volumenometer and applying a confining pressure equal to the in situ overburden pressure. The specimens are then wetted incrementally (i.e., 5 cm^3 of water per increment) until equilibrium is obtained. Calculation of vertical heave is based on the area under the volumetric strain versus depth relationship determined from test data.

154. <u>Salas and Serratosa.</u>[82] This is an odometer test procedure that collects swell and swell pressure test data. Swell data are collected on specimens lightly loaded (i.e., 1 psi), inundated, and allowed to swell to equilibrium. Swell pressure data are collected on specimens using constant volume swell pressure test procedures. After establishing the swell pressure, the specimen is unloaded in decrements to a nominal seating load. Calculation of anticipated heave is based on the reverse consolidation theory.

155. <u>Lambe and Whitman.</u>[83] This is a prediction procedure that uses change in effective stress to establish the change in void ratio necessary to apply the reverse consolidation theory. Conventional consolidation tests are conducted on undisturbed (or remolded) specimens taken from various depths and the rebound portion of the e-log p curve established and presented in terms of effective stress. Initial and final pore water pressure distributions (tension) are then established with the final values based on depth to the groundwater table (negative hydrostatic head). The effective stress at each specimen is computed for before- and after-loading conditions, and the associated void ratio changes are taken from the rebound curve and used to estimate heave.

Factors influencing odometer testing procedures and results

156. The testing and prediction procedures described in the previous paragraphs 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 affecting the testing procedure directly that, in turn, influences the results of the prediction equation. Fredlund[84] describes these procedural factors that should be of concern when testing expansive soils as follows:

a. Loading procedure. Specifically involved are 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[39] 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.

157. 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 solution to the procedural problems are simple but often overlooked because of either minimal laboratory inspection and calibration or lack of a standard odometer test for expansive soils.

158. Two 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 category 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 for as odometer testing is concerned, these two factors must be compromised to obtain a practical measurement and estimate of anticipated volume change.

Soil Suction Test and Prediction Procedures

159. The swell and swell pressure of an expansive soil may be determined using the soil suction testing and prediction procedure. Contrary to the odometer test procedures, soil suction can be measured using a variety of different equipment and procedures;[31] however, equipment normally used includes pressure plate and pressure membrane apparatus, tensiometers, and psychrometers.

Review of soil suction tests and prediction procedures

160. The concept and definition of soil suction were introduced earlier, along with a procedure for using the data to estimate volume change. The following paragraphs provide a summary of currently used published prediction techniques that use soil suction data.

161. Richards method.[85,86] In this procedure, Richards measures the soil suction of undisturbed samples using psychrometers. The matrix suction versus moisture content relationship is used to predict eqilibrium moisture content values. The associated equilibrium suction is determined by using correlations between equilibrium soil suction and a climatic rating. With estimates of the change in moisture content made and assuming the volume change of the soil is equal to the volume of water taken up, the volume change ($\Delta V/V$) is

$$\frac{\Delta V}{V} = \frac{(w_1 - w_2)\ G_s}{100 + w_1 G_s} \tag{11}$$

where

w_1 = initial moisture content, percent

w_2 = final moisture content, percent

G_s = specific gravity

Then, assuming equal volume changes in the vertical and horizontal directions, the vertical heave ($\Delta H/H$) is:

$$\frac{\Delta H}{H} = \frac{1}{3}\,\frac{\Delta V}{V} = \frac{1}{3}\,\frac{(w_i - w_2)G_s}{100 + w_1 G_s} \tag{12}$$

An example of calculations using the Richards method is given in Table 7.

162. Australian method.[87,88] This method involves collection of data from a modified odometer in which both applied load and soil suction can be controlled during the test. The data from these tests are plotted to provide linear strain ($\Delta H/H$) versus soil suction relationships for various load levels. With the slope of this relationship and the initial suction known, and an estimate of the final suction made, sufficent data are available for estimating in situ heave. An example of the calculations is given in Table 8.

163. McKeen.[89,90] This procedure is similar in nature to the Australian procedure previously discussed; however, the soil suction (initial) data are determined using the filter paper procedure. Final moisture conditions are estimated using guidelines presented by Russam.[91] Load changes and response factors (i.e., $\Delta H/H$ versus swell) are determined from existing relationships. An example of the calculations used in the procedure is given in Table 9.

Table 7

Sample Calculations of Soil Movement (from Reference 85)

L, cm	Initial Suction (h_o) cm H_2O	Initial Effective Stress ($\bar{\sigma}_o$) cm H_2O	Final Suction (h_f) cm H_2O	Final Effective Stress ($\bar{\sigma}_f$) cm H_2O	w_o, %	Δw, %	ΔL, cm
			From Driest Condition to Equilibrium Profile				
0-10	90,000	90,000	1,400	1,400	11.5	15.7	1.08
10-20	45,000	45,000	1,400	1,400	14.6	12.6	0.82
20-30	10,000	10,000	1,300	1,300	21.0	6.4	0.37
30-40	5,000	5,000	1,300	1,300	23.3	4.1	0.23
40-60	3,200	3,200	1,300	1,300	24.6	2.8	0.30
60-80	2,500	2,500	1,300	1,300	25.1	2.3	0.25
80-100	1,800	1,800	1,300	1,300	26.4	1.0	0.11
100-120	1,500	1,500	1,300	1,300	27.1	0.3	0.03
120-140	1,400	1,400	1,300	1,300	27.2	0.2	0.02
					Surface movement		3.21
			From Driest to Wettest Condition (i.e., Seasonal Movement)				
0-10	90,000	90,000	800	800	11.5	16.7	1.15
10-20	45,000	45,000	700	700	14.6	13.8	0.90
20-30	10,000	10,000	600	600	21.0	7.8	0.45
30-40	5,000	5,000	520	520	23.3	5.8	0.33
40-60	3,200	3,200	580	580	24.6	4.3	0.46
60-80	2,500	2,500	870	870	25.1	2.9	0.32
80-100	1,800	1,800	1,120	1,120	26.4	1.1	0.13
100-120	1,500	1,500	1,350	1,350	27.1	--	--
120-140	1,400	1,400	1,300	1,300	27.2	--	--
					Surface movement		3.74

Table 8

Sample Calculation of Total Swell (from Reference 88)

Depth ft	Vertical Increment (H), in.	Suction, pF* Initial	Suction, pF* Final	Overburden and Surcharge Pressure lb/ft^2	Slope $(\Delta H/H)$ 0.1 pF	$\Delta H/H$	Lateral Restraint Factor, f	$f(\Delta H/H)$	Average $f(\Delta H/H)$ for the Increment	Incremental Vertical Movement in.	Total Vertical Movement in.
0		4.0	3.2	0	0.0034	0.0270	1.00	0.0270			1.00
1	12	3.9	3.2	112	0.0031	0.0215	1.00	0.0215	0.0242	0.29	0.71
2	12	3.8	3.2	224	0.0028	0.0170	1.00	0.0170	0.0190	0.23	0.48
3	12	3.7	3.2	336	0.0026	0.0130	1.00	0.0130	0.0150	0.18	0.30
6	36	3.3	3.2	672	0.0017	0.0017	1.00	0.0017	0.0075	0.27	0.03
9	36	3.2	3.2	1008	0.0009	0	1.00	0	0.0008	0.03	0
12	36	3.2	3.2	1344	0	0	1.00	0	0	0	0

* pF = log of suction in centimetres of water, i.e.

pF = 1 = 10 cm of H_2O = 0.98 kN/m^2 = 0.142 psi

pF = 2 = 100 cm of H_2O = 9.81 kN/m^2 = 1.42 psi

pF = 3 = 1000 cm of H_2O = 98.1 kN/m^2 = 14.2 psi.

Table 9

Sample Calculations for Surface Heave (from Reference 89)

Layer No.	Layer Thickness ft	$(\Delta\tau)_{avg}$ pF	$(P_i)_{avg}$ psi	$(P_f)_{avg}$ psi	Δ % per pF	$\Delta L/L$ %	ΔL ft	ΔL in.	$\Sigma\Delta L$ in.
1	0-3	1.70	1.1	2.7	-1.97	+1.1	0.04	0.50	0.50
2	3-8	1.28	3.8	5.1	-2.57	+1.1	0.06	0.66	1.16
3	8-12	0.72	4.5	8.0	-2.32	+0.2	0.01	0.10	1.26
4	12-15	0.30	6.6	10.1	-1.52	0	0	0	1.26
5	15-20	0.05	8.9	12.4	-2.37	-0.3	-0.02	-0.18	1.08

164. Johnson.[29,29,33,34,92-99] The soil suction data are obtained using thermocouple psychometers and used to prepare the soil suction versus water content relationship, which is linear on a semilog scale for the range of moisture contents of concern to engineers. The soil suction versus water content relationship can be represented by the linear equation

$$\log \tau_m^{\,o} = A - Bw \tag{13}$$

where

$\tau_m^{\,o}$ = matrix soil suction without surcharge pressure, tsf

A,B = constants (y-intercept and slope, respectively)

w = water content

With the expansive soil characterized with respect to soil suction, the volume change of an expansive clay stratum may be estimated using the following equation:

$$\frac{\Delta H}{H} \approx \frac{C_\tau}{1 + e_o}\left[(A - Bw_o) - \log(\tau_{mf} + \alpha\sigma_f)\right] \tag{14}$$

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.

165. The final matrix suction, τ_{mf} , is dependent on the assumed equilibrium moisture profile,[94] i.e., saturated or negative hydrostatic head. For a saturated profile, the value of τ_{mf} is assumed equal to

zero. For negative hydrostatic head and a shallow water table, τ_{mf} at a specific depth (X) is simply the difference between the depth to the water table and the depth (X) converted to pressure terms. For negative hydrostatic head and a deep water table (can also be applied to shallow-water table case), the following equation may be used:

$$\tau_{mf} = \tau_{ma} + (X_a - X)\gamma_w \tag{15}$$

where

τ_{ma} = in situ matrix suction at depth of active zone, X_a, tsf

γ_w = unit weight of water, $\mathrm{ton/ft^3}$

The depth of the active zone (X_a) is defined as the depth below which the soil suction does not change.

166. The compressibility factor, α, is the fraction of the applied pressure that is effective in changing the pore water pressure.[94]

$$\tau_{mf} = \tau^{o}_{mf} - \alpha\sigma_f \tag{16}$$

where

τ^{o}_{mf} = final matrix suction with no external pressure, tsf

It is obtained by multiplying the unit weight of water (in grams per cubic centimetres) by the slope of a curve relating the reciprocal of the dry density (in cubic centimetres per gram, specific total volume)[100] to water content (percent of dry weight). This factor will be zero for incompressible soils, such as clean sands at low degrees of saturation, but it will be equal to one for all fully saturated or quasi-saturated soils. 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 plasticity index (PI) by[101]

$$PI < 5 \qquad \alpha = 0 \tag{17a}$$

$$PI > 40 \qquad \alpha = 1 \tag{17b}$$

$$5 < PI < 40 \qquad \alpha = 0.0275\ PI - 0.125 \tag{17c}$$

167. 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.

Empirical Prediction Methods

168. In an effort to reduce the amount of time required to conduct odometer testing procedures to obtain data for estimating anticipated volume change, researchers and consultants began collecting odometer test data and correlating it with physical and index properties. The results of the correlation studies were equations relating swell or swell pressure to such index properties as liquid limit, plasticity index, natural moisture content, etc. In many cases, the equations were developed on specimens prepared in the laboratory using commercial grade clay minerals and sand mixtures and give very little indication of true field behavior. Even in cases where natural undisturbed samples were used, the relationships were limited in application to materials outside the geographical area of consideration. Despite these limitations, empirical procedures are used to estimate anticipated volume change. The following paragraphs provide a summary of typical published empirical methods.

169. Potential Vertical Rise (PVR).[102-105] This is a procedure based on correlations between odometer swell test results on undisturbed and remolded samples and index properties. Field heave (PVR) is estimated using a family of curves which relate plasticity index, volumetric change, surcharge pressure (overburden), and PVR of the expansive soil stratum. The initial water content is compared with an estimated maximum (0.47 LL + 2) and minimum (0.2 LL + 9) water content. An example of the calculations necessary to determine the PVR is shown in Table 10.

170. Nayak and Christensen.[61] This method involves a statistical relationship between measured swell under 1-psi surcharge and swell pressure and index properties. Correlation data were collected on compacted (AASHTO T-99 optimum moisture content and maximum dry density), laboratory mixed samples. The resulting empirical equations for predicting swell and swell pressure are

$$S_p = (2.29 \times 10^{-2})(PI^{1.45})(C/w_i) + 6.38 \tag{18a}$$

$$P_s = (3.58 \times 10^{-2})(PI^{1.12})(C^2/w_i^2) + 3.79 \tag{18b}$$

Table 10

Example of Potential Vertical Rise Calculation (from Reference 102)

Depth ft	Avg Load psi	LL	Dry 0.2 LL + 9	Wet 0.47 LL + 2	% Moisture	Dry Avg Wet	% - No. 40	PI	% Volume Swell (Figure 1)	% Free Swell	PVR, in. Top of Layer	PVR, in. Bottom of Layer	Difference	Modi-fied -40 Factor	Modi-fied Density Factor*	PVR in Layer in.
0-2	1	21	--	--	3.1	Dry	100	4	0.0	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 in.			
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 pcf wet density assumed for all layers. When greater accuracy is desired use $\frac{125}{\text{Actual wet density of soil, pcf}}$ as the modifier.

** See example on Figure 2, Reference 102.

† Since the 12-ft layer from 20 to 32 ft 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 in., respectively, or a difference of 0.46 in., will be obtained which is a summation of increments (difference) as shown above for the bottom 12 ft. When layers of expansive clays of less than 2 ft exist (Example 4-4.6 ft) it is preferable to enter the abscissa on the proper swell curve at 4 and 4.6 ft, respectively, and use the difference in the respective ordinate readings as the unmodified swell in the 0.6-ft-thick layer.

where

S_p = percent swell

P_s = swell pressure, psi

C = clay content = percent minus 2 micrometres

171. Komornik and David.[62] This method involves a statistical relationship between measured swell pressure and physical and index properties. Correlation data were collected on undisturbed natural soil samples. The empirical equation for predicting swell pressure is

$$\log P_s = -2.132 + 0.0208(LL) + 0.000665(\gamma_d) - 0.0269(w_i) \qquad (19)$$

where

P_s = swell pressure, kg/cm^2

γ_d = natural dry density, kg/m^3

172. Van Der Merwe.[106] This method involves relationships between potential expansiveness and physical and index properties. Anticipated volume change is estimated using the following equation:

$$S_p = \frac{100}{n} \sum_{D=1}^{D=n} F_D \, (PE) \qquad (20)$$

where

S_p = percent swell

n = depth of soil layer, ft

F_D = reduction factor, Figure 20

PE = potential expansiveness or unit heave, Figure 20

For the various categories of potential damage the PE values are:

Low	PE = 0 in. per foot of depth
Medium	PE = 1/4 in. per foot of depth
High	PE = 1/2 in. per foot of depth
Very High	PE = 1 in. per foot of depth

173. Seed, Woodward, and Lundgren.[38] This method involves a statistical relationship between swell and physical (clay content) and

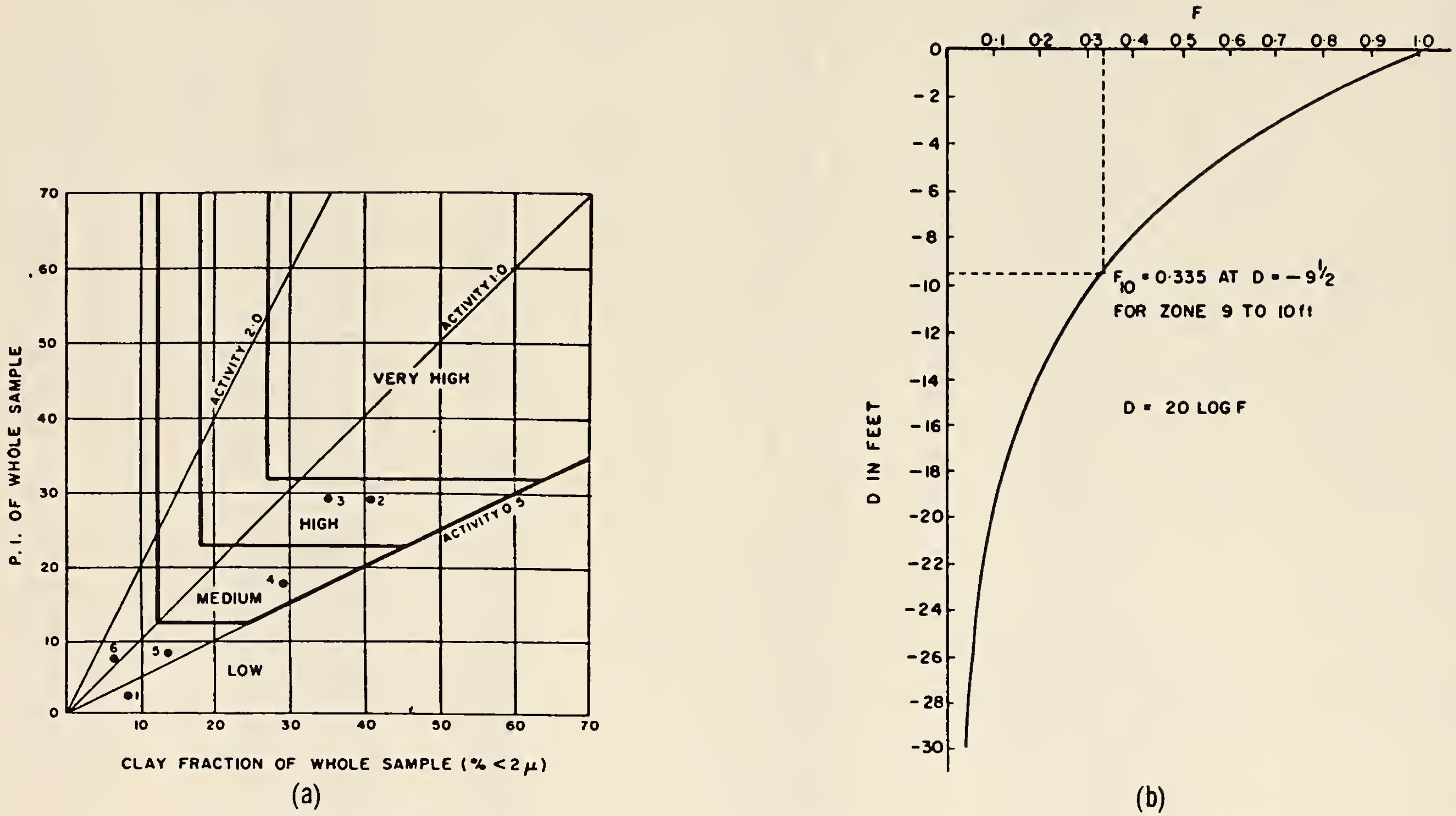


Figure 20. Relationships used to determine: (a) potential expansiveness and (b) reduction factor for Van Der Merwe's empirical method (from Reference 106)

index (PI) properties. Correlation data were collected on compacted (AASHTO T-99 optimum moisture content and maximum dry density), laboratory mixed samples under 1-psi surcharge. The resulting empirical equation is

$$S_p = (3.6 \times 10^{-5})(A^{2.44})(C^{3.44}) \tag{21}$$

where

S_p = percent swell

A = activity = PI/C

The equation was simplified for application to natural soils using documented limits for activity:

$$S_p = 0.00216 \ PI^{2.44} \tag{22}$$

174. Ranganatham and Satyanarayana.[54] This method is based on statistical relationship between measured swell and index properties. Correlation data were collected on compacted, laboratory mixed and natural soils under 1-psi surcharge. The empirical equation is

$$S_p = 41.13 \times 10^{-5} \ (SI)^{2.67} \tag{23}$$

where

S_p = percent swell

SI = shrinkage index = LL - SL

175. Vijayvergiya and Ghazzaly.[58,59] This method involves statistical relaionships between measured swell and swell pressure and physical (w, γ_d) and index (LL) properties. Correlation data were collected on undisturbed natural soil samples under 0.1-tsf surcharge pressure. The results were a series of empirical equations:

$$\log S_p = \frac{1}{12} (0.4 \ LL - w + 5.5) \tag{24a}$$

$$\log S_p = \frac{1}{19.5} (\gamma_d + 0.65 \ LL - 130.5) \tag{24b}$$

$$\log P_s = \frac{1}{12} (0.4\ LL - w - 0.4) \qquad (24c)$$

$$\log P_s = \frac{1}{19.5} (\gamma_d + 0.65\ LL - 139.5) \qquad (24d)$$

where

S_p = percent swell

P_s = swell pressure, tsf

176. Schneider and Poor.[107] This method involves statistical relationships between measured swell under varying overburden loads and index (PI and w) properties. A series of equations is available for various surcharges (expressed in feet of overburden), as shown in the following tabulation:

Surcharge, ft	Log S_p
0	0.90 (PI/w) -1.19
3	0.65 (PI/w) -0.93
5	0.51 (PI/w) -0.76
10	0.41 (PI/w) -0.69
20	0.33 (PI/w) -0.62

where

S_p = percent swell

Geotechnical Properties and Environmental Conditions that Influence Volume Change

177. The geotechnical properties and environmental conditions that influence volume change include the properties which determine whether a material possesses a potential for swell and the extent (magnitude and rate) the potential volume change will be realized. Numerous authors[42,90,92,108,110] have presented their views on the various factors. In Reference 1 the geotechnical properties (referred to as intrinsic properties) and environmental conditions that influence volume change include:

Geotechnical Properties	Environmental Conditions
Soil Composition	Soil Profile
Density and Moisture Content	Depth of Desiccation

(Continued)

Geotechnical Properties	Environmental Conditions
Soil Fabric	Depth of Seasonal Moisture Variation
Soil structure	Vegetative Cover
Pore Fluid Properties	Topography and Surface Drainage
Confinement (i.e., load)	Groundwater Table
Permeability	Climate
Time	
Temperature	

Detailed descriptions of the influence of each of these factors are presented in Reference 1. All of these factors relate to the in situ behavior of expansive soils; therefore, to accurately characterize an expansive soil and estimate the magnitude and rate of anticipated volume change, these factors must be included, simulated, or otherwise accounted for in the laboratory test and/or associated prediction procedure. As far as the geotechnical properties are concerned, all except confinement, time, and temperature, which are not actially geotechnical properties but rather ambient factors that affect the geotechnical properties and their influence on volume change, are accounted for when a sample is selected for odometer testing. In other words, the selection of an odometer test specimen that is representative of the material under consideration establishes the datum for soil composition, dry density, moisture content, fabric, structure, and permeability. Pore fluid properties are accounted for by compromise, i.e., use of distilled water. Of the environmental conditions, the influence of soil profile is the only condition that can be accounted for in test procedures, i.e., varying load (overburden) representing varying depth. In some prediction procedures, the influence of the groundwater table may be included when estimating final moisture conditions.

Requirements for estimates of volume change

178. Accurate calculation of anticipated volume change requires definition of initial conditions, such as moisture content, density or void ratio, and stress (overburden) conditions, and an estimate of final conditions, such as moisture content, pore water pressure, and stress (overburden plus applied load), likely to develop considering the material, site, and environment. All test and prediction procedures have these basic requirements in common. The problems with and differences between the various procedures develop when considering how these requirements are met and how well the procedures simulate field behavior. Selection of test and prediction procedures should provide the most realistic and practical simulation of field behavior as well as simplicity and economy of operation for data collection and analysis.

179. Volume change occurs in the field because of changing moisture and pressure conditions due to construction of a pavement. The initial conditions are easily obtained through accepted laboratory

testing procedures. Certain of the final conditions, such as load applied and duration of application, are available as a result of design calculations; however, the most important factor, final moisture content profile, is the most difficult to obtain. Generally, the moisture content profile is estimated by assuming a saturated profile, which is the basis for application of the reverse consolidation theory and the odometer swell tests used to obtain the data for the theory. The major problem with this approach is that few of the heavily overconsolidated materials that are representative of most expansive soils ever become saturated during odometer testing or in the field. The other option for estimating final moisture content profile (pore water pressure) involves a triangular distribution of head decreasing with depth.

Evaluation of Test and Prediction Procedures

180. Evaluation on a comparative basis of all of the testing and prediction procedures discussed in previous paragraphs would be virtually impossible because of the specific requirements of the different testing procedures alone. To simply collect the data based on the required test procedures would be totally impractical and of questionable use, particularly for the odometer test procedures, since the basic concept of the odometer test procedure is the same. In other words, all odometer test procedures inundate a specimen in distilled water and measure deformation or pressure. Therefore, evaluation of odometer tests will be limited to test procedures that balance practicality and simplicity with maximum simulation of environmental conditions. Evaluation of the soil suction test and prediction procedure will concentrate on procedures that utilize the thermocouple psychrometer testing procedure. This procedure has been shown[3,29,33,34,35,85,86,95,98,99] to be a simple, accurate, and reliable method for characterizing the soil suction relationships for expansive soils. All of the empirical techniques will be evaluated because the data needed to apply them are available and a decision needs to be made concerning the usefulness of any or all of the techniques.

Basis for comparison

181. Evaluation of test and prediction procedures requires that standards be established for rating the various procedures. The actual evaluation is the result of direct comparisons between predicted and measured values of swell; however, this alone should not be the sole justification for rating one procedure over another. As a minimum basis for comparison, the following factors should be considered:

a. Practicality and simplicity of test and prediction procedure.

b. Reliability and reproducibility of test equipment.

c. Simulation of geotechnical and environmental conditions.

182. In order for a test and prediction procedure to be accepted, it has to be demonstrated that the procedure is accurate, reliable, and practical. Implementation of a procedure, such as the soil suction technique (thermocouple psychrometer), also requires consideration of availablilty of equipment and adaptability of test method within routine procedures of State Highway Agencies.

Laboratory testing program

183. Details of the field sampling program have been presented in previous sections of the report.[3,4] In addition to the previously described sampling sites (20), undisturbed samples were taken at one additional site (Cameron, Ariz.) late in the research program because the Arizona Department of Transportation made available field monitoring data being collected at the site. Sampling was conducted at a asphalt-rubber membrane test section and at the control section for the field study. The laboratory testing program supplemented the first phase program presented in Reference 3 and 4. The same type of data, such as specific gravity, grain-size distribution, Atterberg limits, natural moisture content, dry density, and natural soil suction, were collected on 120 samples in addition to the first-phase data. This provided characterization of the materials at the various sites with depth. Results of the laboratory testing program on undisturbed specimens are given in Appendix A, Volume II. The results on an individual site basis include: a tabular summary of the physical properties, Atterberg limits, and soil suction parameters; graphical representation of these results including a boring log; a plasticity chart showing classification of the materials by the USCS; and a tabular and graphic (e-log p) summary of overburden swell (OS) tests on selected specimens. A summary of the overburden swell test data for the undisturbed specimens is shown in Table 11. In order to have data available on remolded specimens (AASHTO T-99 optimum moisture content, w_{opt}, and maximum dry density, γ_{max}) representing the 20 sampling sites (Cameron, Ariz. not included because of limited supply of samples), soil suction and OS tests were conducted; the results are given in Appendix B, Volume II and summarized in tabular form in Tables 12 and 13. Since the most widely used chemical treatment for expansive soils involves lime, a laboratory testing program was conducted on lime-treated specimens molded at AASHTO T-99 w_{opt} and γ_{max} Lime percentages were determined using the pH test and Atterberg limits. pH test, Atterberg limits, soil suction, and OS test data for the 20 sampling sites are given in Appendix C, Volume II and summarized in tabular form in Tables 14 and 15.

Field data

184. In conjunction with the field sampling and laboratory testing programs, selected State Highway Agencies were requested to provide field data from monitored pavement sections on which various treatment

Table 11

Summary of Data from Overburden Swell Tests

Site No.	Initial Conditions*				End of Swell Conditions				End of Test Conditions				Δe**	Swell† %
	e_o	w_o	S_o	γ_o	e_f	w_f	S_f	γ_f	e	w	S	γ		
1	1.138	40.3	97.4	80.3	1.409	51.3	100	71.2	1.485	54.9	100+	69.1	0.271	12.7
2	0.7826	25.6	87.7	93.8	0.8061	30.1		92.6	0.815	28.2	92.9	92.2	0.0235	1.32
3	1.105	39.5	98.6	81.8	1.125	40.8		81.0	1.179	42.6	99.8	79.0	0.020	0.96
4	0.6079	20.6	92.2	105.6	0.6210	22.8		104.7	0.628	22.2	96.2	104.3	0.0131	0.82
5	0.7851	24.0	83.5	95.4	0.7966	29.2		94.8	0.823	31.6	100+	93.5	0.0115	0.64
6	0.4750	13.8	82.2	119.7	0.4753	16.8		119.7	0.485	22.7	100+	118.9	0.0003	0.02
7	0.4546	15.4	93.2	117.9	0.4608	16.8		117.5	0.472	16.6	96.7	116.6	0.0062	0.43
8	0.4082	12.5	86.4	124.9	0.4084	14.5		124.9	0.409	13.3	91.6	124.9	0.0002	0.01
9	0.5387	9.4	48.0	111.5	0.5588	20.3		110.1	0.590	19.0	88.7	108.0	0.0201	1.31
10	0.728	19.3	74.5	101.5	0.791	28.1		97.9	0.794	27.5	97.2	97.8	0.063	3.65
11	0.5858	9.4	44.0	107.8	0.5910	21.6		107.5	0.598	24.0	100+	107.1	0.0052	0.33
12	0.765	26.1	93.8	97.2	0.783	28.5		96.2	0.810	27.7	94.1	94.8	0.018	1.02
13	0.4554	16.7	96.4	112.8	0.4605	17.5		112.4	0.463	16.6	94.5	112.2	0.0051	0.35
14	0.7791	26.4	94.9	98.2	0.7944	28.4		97.4	0.826	28.7	97.1	95.7	0.0153	0.86
15	1.185	38.2	89.0	78.8	1.188	43.0		78.7	1.203	41.8	95.9	78.2	0.003	0.14
16	0.633	16.0	68.8	103.9	0.685	25.2		100.7	0.729	25.2	93.9	98.2	0.052	3.18
17	0.734	26.4	97.5	97.5	0.748	27.6		96.7	0.776	28.3	98.6	95.2	0.014	0.81
18	0.779	15.7	56.0	97.5	0.827	29.7		94.9	0.824	25.0	84.2	95.1	0.048	2.70
19	0.5542	17.6	87.7	110.8	0.5598	20.3		110.4	0.569	22.4	92.2	110.3	0.0056	0.36
20	1.035	34.5	90.7	83.4	1.073	39.5		81.9	1.108	38.7	95.2	80.6	0.038	1.87
21a††	0.4099	10.1	68.5	123.0	0.4297	15.6		121.3	0.476	14.8	86.8	117.6	0.0198	1.40
21b††	0.2805	8.2	80.7	134.5	0.4081	14.8		122.3	0.532	20.2	100+	112.4	0.1276	9.97

Note: e = void ratio, w = moisture content (%), S = degree of saturation (%), γ = dry density (pcf).
* Corresponds to overburden conditions on e-log p plot.
** $\Delta e = e_f - e_o$.
† % swell = $\Delta e/(1 + e_o)$.
†† 21a is test section and 21b is control section.

Table 12

Summary of Molding Conditions and Soil Suction Parameters for Remolded Samples

		Molding Conditions*			Soil Suction Parameters			
Site	G_s	w %	γ pcf	e	τ tsf	A	B	α
1. Jackson, MS	2.78	26.18	81.4	1.1298	40.96	4.555	0.1124	1.00
2. Hattiesburg, MS	2.72	20.33	105.7	0.6054	2.17	16.600	0.8000	1.00
3. Monroe, LA	2.74	28.12	84.0	1.0343	6.05	5.140	0.1550	0.57
4. Lake Charles, LA	2.72	15.01	111.7	0.5193	6.16	4.110	0.2212	1.00
5. San Antonio, TX	2.69	19.48	102.0	0.6449	1.47	11.494	0.5814	0.66
6. Vernon, TX	2.82	21.04	105.7	0.6654	8.70	4.247	0.1572	0.90
7. Durant, OK	2.75	17.21	108.3	0.5850	5.80	7.539	0.3937	1.00
8. Hennessey, OK	2.79	18.80	110.2	0.5797	1.10	15.080	0.8000	1.00
9. Holbrook, AZ, #1	2.75	22.79	101.2	0.6954	2.37	2.326	0.0858	1.00
10. Holbrook, AZ, #2	2.81	20.52	97.2	0.8031	20.11	5.904	0.2242	1.00
11. Price, UT	2.75	18.79	109.2	0.5710	6.33	4.300	0.1862	0.85
12. Hayes, KS	2.72	26.47	95.7	0.7733	5.04	5.488	0.1808	1.00
13. Ellsworth, KS	2.54	27.34	84.3	0.8795	4.67	10.000	0.3413	1.00
14. Limon, CO, #1	2.65	20.94	101.3	0.6319	3.73	5.204	0.2212	1.00
15. Limon, CO, #2	2.68	31.09	86.7	0.9297	4.81	12.721	0.3876	0.23
16. Denver, CO	2.77	23.83	95.6	0.8084	2.80	15.434	0.6289	0.90
17. Newcastle, WY, #1	2.79	19.95	91.5	0.9022	34.04	3.517	0.0995	1.00
18. Newcastle, WY, #2	2.78	19.96	100.9	0.7199	11.09	5.460	0.2212	1.00
19. Billings, MT	2.79	19.10	104.2	0.6701	4.08	12.623	0.6289	0.57
20. Reliance, SD	2.73	31.92	74.6	1.2835	11.66	12.111	0.3460	1.00

* AASHTO T-99 compactive effort, sleeve rammer.

Table 13

Summary of Data from Overburden Swell Tests - Remolded Specimens

Site No. and Location	G_s	Initial Conditions* e_o	w_o	γ_o	S_o	End of Swell Conditions e_f	w_f	γ_f	S_f	End of Test Conditions e	w	γ	S	Δe**	Swell† %	P_o tsf
1. Jackson, MS	2.78	1.141	27.1	81.0	66.0	1.691	60.8	64.5	100	1.634	59.2	65.9	100+	0.55	25.7	0.100
2. Hattiesburg, MS	2.72	0.652	21.1	102.7	88.0	0.706	26.0	99.5		0.729	25.3	98.2	94.5	0.054	3.27	0.123
3. Monroe, LA	2.74	1.031	30.5	84.2	81.1	1.191	43.5	78.0		1.185	41.3	78.3	95.4	0.16	7.88	0.111
4. Lake Charles, LA	2.72	0.568	15.7	108.2	75.2	0.640	23.5	103.5		0.630	21.9	104.2	94.6	0.072	4.59	0.128
5. San Antonio, TX	2.69	0.650	19.5	101.7	80.7	0.669	24.9	100.6		0.665	20.5	100.9	82.8	0.19	1.15	0.121
6. Vernon, TX	2.82	0.679	17.0	104.8	70.6	0.728	25.8	101.8		0.723	22.7	102.2	88.3	0.049	2.92	0.128
7. Durant, OK	2.75	0.630	18.7	105.3	81.6	0.747	27.2	98.2		0.802	27.5	95.3	94.2	0.117	7.18	0.125
8. Hennessey, OK	2.79	0.593	19.6	109.3	92.2	0.595	21.3	109.2		0.597	20.3	109.1	94.6	0.002	0.13	0.131
9. Holbrook, AZ, #1	2.75	0.634	21.7	105.0	94.1	0.636	23.1	104.9		0.644	22.0	104.4	94.1	0.002	0.12	0.119
10. Holbrook, AZ, #2	2.81	0.852	21.4	94.7	70.6	0.990	35.2	88.1		1.022	34.2	86.8	94.2	0.138	7.45	0.119
11. Price, UT	2.75	0.588	19.0	108.1	88.9	0.590	21.5	107.9		0.599	19.1	107.4	87.9	0.002	0.13	0.127
12. Hayes, KS	2.72	0.809	25.9	93.8	87.1	0.983	36.1	85.6		1.012	36.8	84.4	98.8	0.174	9.62	0.120
13. Ellsworth, KS	2.54	0.957	28.5	81.0	75.6	1.030	40.6	78.1		1.046	37.6	77.5	91.4	0.073	3.73	0.110
14. Limon, CO, #1	2.65	0.708	22.0	96.8	82.3	0.745	28.1	94.8		0.738	25.1	95.2	90.3	0.037	2.17	0.119
15. Limon, CO, #2	2.68	1.075	31.4	80.6	78.3	1.195	44.6	76.2		1.214	42.3	75.6	93.4	0.12	5.78	0.110
16. Denver, CO	2.77	0.843	25.0	93.8	82.1	0.912	32.9	90.4		0.915	31.0	90.3	93.7	0.069	3.74	0.120
17. Newcastle, WY, #1	2.79	0.830	26.5	95.1	89.1	0.917	32.9	90.8		0.948	31.7	89.4	93.4	0.087	4.75	0.120
18. Newcastle, WY, #2	2.78	0.753	22.0	98.9	81.2	0.855	30.8	93.5		0.876	29.6	92.5	93.9	0.102	5.82	0.120
19. Billings, MT	2.79	0.721	20.2	101.2	78.2	0.804	28.8	96.5		0.837	27.7	94.8	92.3	0.083	4.82	0.124
20. Reliance, SD	2.73	1.298	38.8	74.1	81.6	1.458	53.4	69.3		1.475	52.3	68.9	96.9	0.16	6.96	0.109

Note: e = void ratio, w = moisture content (%), γ = dry density (pct), S = degree of saturation (%).

* Corresponds to overburden conditions on $e - \log P$ plot.

** $\Delta e = e_f - e_o$.

† % swell = $\Delta e/(1 + e_o)$.

Table 14

Summary of Molding Conditions and Soil Sunction

Parameters for Lime-Treated Samples

Site	Lime Content %	Molding Conditions*			Soil Suction Parameters			
		w %	γ pcf	e	τ tsf	A	B	α
1. Jackson, MS	4	26.56	81.0	1.1420	24.69	13.804	0.4673	1.00
2. Hattiesburg, MS	4	21.27	93.5	0.8149	6.38	30.347	1.3889	0.65
3. Monroe, LA	5	29.89	81.9	1.0879	1.53	8.606	0.2817	1.00
4. Lake Charles, LA	3	15.78	98.3	0.7269	2.94	7.513	0.4464	1.00
5. San Antonio, TX	3	20.46	96.9	0.7317	1.81	6.922	0.3257	1.00
6. Vernon, TX	3	22.04	102.7	0.7142	6.92	4.686	0.1745	0.95
7. Durant, OK	3	18.94	100.7	0.7050	8.20	7.800	0.3636	0.88
8. Hennessey, OK	3	19.55	106.5	0.6284	4.44	12.147	0.5882	0.93
9. Holbrook, AZ, #1	4.5	18.76	108.3	0.5847	2.51	17.455	0.9091	0.58
10. Holbrook, AZ, #2	3	21.79	97.0	0.8073	5.35	7.758	0.3226	0.94
11. Price, UT	2	19.34	107.6	0.5945	6.15	2.013	0.0633	0.62
12. Hayes, KS	4	27.39	91.7	0.8517	3.27	4.919	0.1608	0.65
13. Ellsworth, KS	3	28.82	77.2	1.0567	6.31	27.000	0.9091	1.00
14. Limon, CO, #1	4	21.89	92.3	0.7914	3.84	12.105	0.5263	0.55
15. Limon, CO, #2	4	32.36	76.9	1.1743	6.82	12.600	0.3636	1.00
16. Denver, CO	5	24.62	90.4	0.9134	4.82	41.717	1.6667	1.00
17. Newcastle, WY, #1	4	22.84	89.5	0.9451	8.96	5.431	0.1961	1.00
18. Newcastle, WY, #2	3	21.34	95.6	0.8146	7.08	7.109	0.2933	1.00
19. Billings, MT	4	19.64	96.9	0.7962	3.09	3.028	0.1292	1.00
20. Reliance, SD	4	33.39	77.2	1.2076	6.87	5.134	0.1287	1.00

* AASHTO T-99 compactive effort, sleeve rammer.

Table 15

Summary of Data from Overburden Swell Tests - Lime-Treated Specimens

Site No. and Location	Lime %	Initial Conditions* e_o	w_o	γ_o	S_o	End of Swell Conditions e_f	w_f	γ_f	S_f	End of Test Conditions e	w	γ	S	Δe**	Swell† %	P_o tsf
1. Jackson, MS	4	1.234	26.4	77.7	59.5	1.527	54.9	68.6	100	1.292	44.2	75.7	95.1	0.293	13.12	0.100
2. Hattiesburg, MS	4	0.779	21.7	95.4	75.8	0.783	28.8	95.2		0.781	25.3	95.3	88.2	0.004	0.22	0.123
3. Monroe, LA	5	1.207	30.9	77.5	70.1	1.224	44.7	76.9		1.212	38.9	77.3	87.9	0.017	0.77	0.111
4. Lake Charles, LA	3	0.723	16.0	98.5	60.2	0.727	26.7	98.3		0.725	22.5	98.4	84.2	0.004	0.23	0.128
5. San Antonio, TX	3	0.863	19.7	90.1	61.4	0.861	32.0	90.2		0.862	28.4	90.2	88.6	-0.002††	-0.11	0.121
6. Vernon, TX	3	0.716	17.5	102.5	68.9	0.891	31.6	93.1		0.763	24.6	101.3	94.3	0.175	10.2	0.128
7. Durant, OK	3	0.745	18.8	98.3	69.4	0.759	27.6	97.6		0.749	24.4	98.1	89.5	0.014	0.80	0.125
8. Hennessey, OK	3	0.664	19.2	104.6	80.7	0.664	23.8	104.6		0.665	20.9	104.6	87.8	0.00	0.00	0.131
9. Holbrook, AZ, #1	4.5	0.620	21.5	105.9	95.4	0.619	22.5	106.0		0.619	20.2	106.0	89.8	0.001	0.06	0.119
10. Holbrook, AZ, #2	3	0.846	21.3	95.0	70.7	0.856	30.5	94.5		0.849	26.4	94.8	87.2	0.01	0.54	0.119
11. Price, UT	2	0.582	19.4	108.5	91.7	0.582	21.2	108.5		0.585	21.0	108.3	98.9	0.00	0.00	0.127
12. Hayes, KS	4	0.971	26.6	86.1	74.5	0.986	36.3	85.5		0.975	32.0	85.9	89.2	0.015	0.76	0.120
13. Ellsworth, KS	3	1.098	28.8	75.5	66.6	1.258	49.5	70.2		1.156	43.7	73.5	95.9	0.160	7.63	0.110
14. Limon, CO, #1	4	0.812	22.5	91.3	73.4	0.814	30.7	91.1		0.813	26.0	91.2	84.7	0.002	0.25	0.119
15. Limon, CO, #2	4	1.242	31.7	74.6	68.4	1.253	46.7	74.2		1.244	40.6	74.5	87.5	0.011	0.49	0.110
16. Denver, CO	5	0.985	25.3	87.1	71.1	0.987	35.6	87.0		0.987	30.4	87.0	85.3	0.002	0.10	0.120
17. Newcastle, WY, #1	4	1.041	25.0	85.3	67.0	1.154	41.4	80.8		1.035	35.9	85.6	96.8	0.113	5.54	0.120
18. Newcastle, WY, #2	3	0.897	22.4	91.4	69.4	0.908	32.7	90.9		0.901	27.6	92.3	86.2	0.011	0.58	0.120
19. Billings, MT	4	0.836	21.2	94.8	70.7	0.954	34.2	89.1		0.851	28.0	94.0	91.8	0.118	6.43	0.124
20. Reliance, SD	4	1.418	38.5	70.5	74.1	1.426	52.2	70.2		1.423	45.7	70.3	87.6	0.008	0.33	0.109

Note: e = void ration, w = moisture content (%), γ = dry density (pcf), S = degree of saturation (%).

* Corresponds to overburden conditions on e - log p plot.

** $\Delta e = e_f - e_o$.

† % swell = $\Delta e/(1 + e_o)$

†† Minus sign indicates consolidation.

alternatives had been used. Ideally, the data should represent both control and test sections. The purpose of the data was to provide quantitative verification of the effect of various treatment alternatives on volume change, and the primary use of the data was in conjunction with subsequent tasks on evaluation of treatment alternatives. Data were received from a total of eight sections. At six of the monitored sections surface deflection or swell versus time data were collected that are applicable to the portion of this task involving comparison between measured and predicted swell. Those monitoring sections and the corresponding sampling site number at which the swell versus time data were collected include:

Monitoring Section Location	Sampling Site Number
San Antonio, Tex.	5
Holbrook, Ariz., Site No. 1	9
Holbrook, Ariz., Site No. 2	10
Limon, Colo.	14
Morrison, Colo.	16
Hysham, Mont.	19

The field monitoring data for these and the remaining two sites (Cameron, Ariz. and Reliance, S. Dak.) are presented in Appendixes D through K, Volume II. More details concerning the field monitoring sites will be presented in subsequent sections of the report.

Prediction of swell-
odometer swell test data

185. As briefly indicated earlier, characterization of expansive soils using odometer swell test procedures involves preparing a test specimen, applying a specified load, inundating, and allowing swell to occur to equilibrium. This is an oversimplification of the procedure and the problems associated with the procedure. In previous sections of this report, 13 different odometer test and prediction procedures were briefly reviewed. With so many possibilities available, the selection of an odometer test and prediction procedure for further evaluation became a significant problem, particularly with respect to loading sequence. The problem was somewhat simplified as a result of an extensive laboratory testing procedure evaluation conducted by the U. S. Army Engineer Waterways Experiment Station.[94,95,97] Although not absolutely conclusive on the subject of odometer tests, the best comparisons were obtained using data collected from OS test and the constant volume swell pressure test. Details of the testing procedures are presented in the following paragraphs.

186. Testing procedures. The OS test selected for evaluation is a deformation test consistent with the definition of potential swell

presented elsewhere.[3] The loading sequence used for the OS test is shown in Figure 21a. After careful preparation of the test specimen, it was placed in the odometer and a light seating load (0.02 tsf) was applied. The load remained at this value for approximately 5 min. The odometer ring and water container were covered with cellophane wrap to minimized moisture change by evaporation. After 5 min, the applied load was increased to the in situ overburden pressure. The specimen was inundated 30 min after application of the overburden pressure unless significant deformation occurred. In which case, inundation was delayed until essentially all of the deformation occurred. As swelling occurred, a deformation versus log of time curve was maintained. When the deformation versus log time leveled off, the consolidation portion of the test was begun. The specimen was consolidated until the void ratio at the overburden pressure was reached, thus providing an alternative definition of swell pressure; then the specimen was rebounded in decrements to the seating load. At each load in the consolidation and rebound portion of the curves, the deformation versus log time plots were maintained to assure that equilibrium was reached as close as possible under a given load. The specimens were then removed for water content determinations.

187. In order to provide data for comparison of measured swell pressure data with the swell pressure prediction techniques, constant volume swell pressure (CVSP) tests were conducted on specimens trimmed from the same samples used for the OS tests. The loading sequence for the CVSP test is shown in Figure 21b and is identical to the OS test up to and including inundation of the specimen. At this point, when the specimen began to expand, small increments of load were applied to maintain constant void ratio. When no further deformation was noted and the specimen was in equilibrium with the applied load, the swell pressure was defined. The specimen was 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 was then removed for water content determinations.

188. <u>Details of heave predictions</u>. In its simpliest form, prediction of heave using OS test data involves the relationship:

$$\frac{\Delta H}{H} = \frac{e_f - e_o}{1 + e_o} \tag{25}$$

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

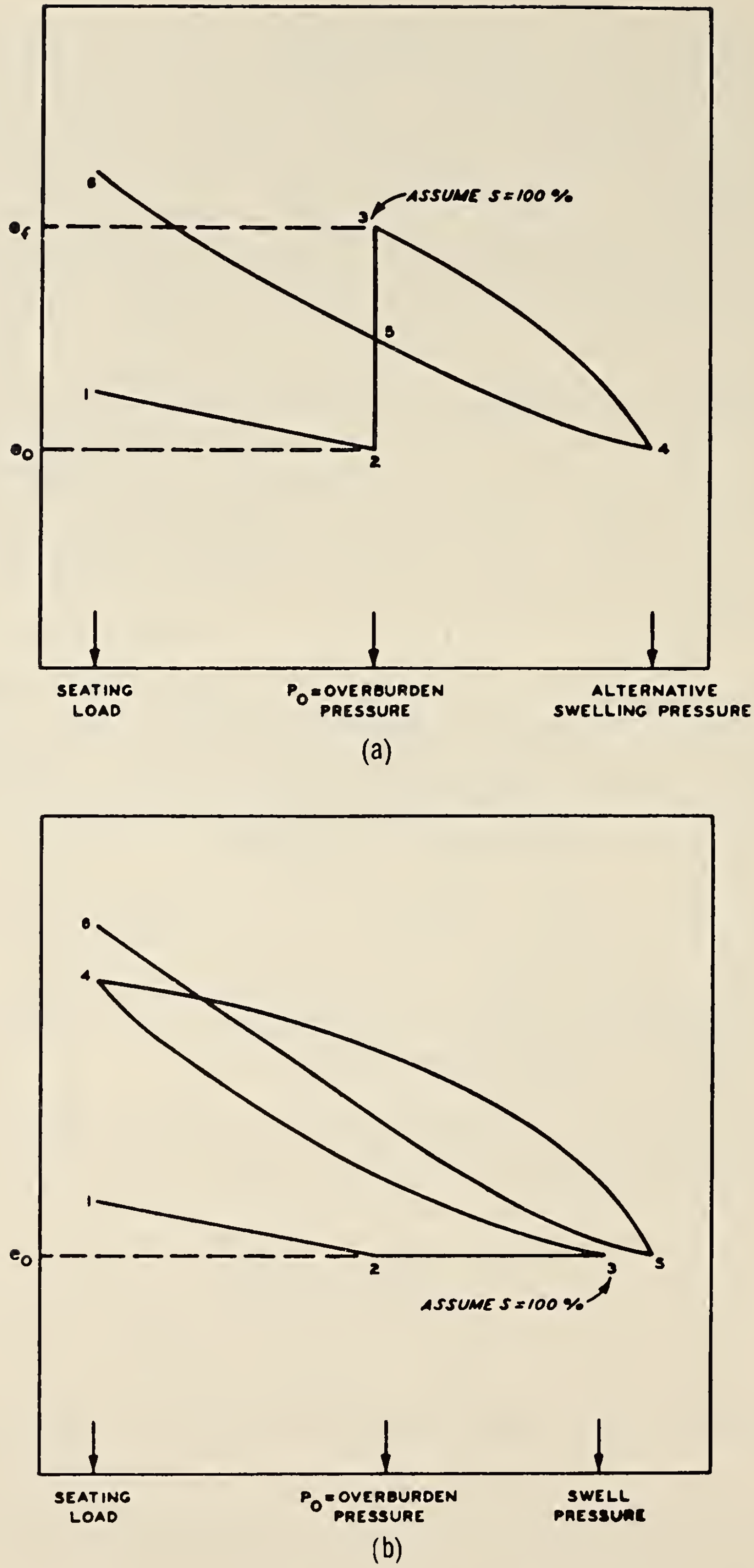


Figure 21. Loading sequence for (a) overburden swell test and (b) constant volume swell pressure test

The predicted heave is the product of the percent swell and layer thickness represented by the soil test specimens. 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 the 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 achieve 100 percent saturation is very long, if it can be attained at all.

189. For the 21 sampling sites in this study, 1 OS test and 1 CVSP test were run on selected samples from each of the sites. Although this provides minimal data for characterizing the profiles at each of the sites involved, to obtain data for each layer within each profile would be far too expensive and time-consuming based on the time required to conduct the tests (i.e., 3 to 4 weeks per specimen). The data collected from the OS tests were applied to the profiles at each of the sites and assumed to represent the entire profile within the estimated depth of active zone. The loading conditions for the laboratory tests and prediction calculation corresponded to overburden loads. The depth of active zone was estimated using the soil suction versus depth profiles in Appendix A, Volume II. Details of the estimation of the depth of active zone will be presented in subsequent sections.

190. Predicted heave. Predicted heave values using OS test data are summarized along with values from the soil suction and empirical techniques for the 21 sampling sites in Table 16. The predicted heaves vary from less than 0.1 in. for sites 6 and 8 to 12.2 and 13.2 in. for sites 1 and 21b, respectively. The majority of the predicted heaves fall into the 0.4- to 1.9-in. range.

Prediction of swell-soil suction test data

191. In Part III of this report, the soil suction concept was introduced and the basis for its measurement using thermocouple psychrometers was described. To briefly review, soil suction is the force exerted by a soil mass responsible for soil water retention, or more simply the pulling force exerted on soil water by the soil mass. Measurement of soil suction can be achieved by a number of procedures; however, the use of thermocouple psychrometers is by far one of the simplest procedures. In simple terms, the psychrometer measures the relative humidity of a specimen in a sealed container. Relative humidity can be used to calculate soil suction directly.

192. Test procedures. The laboratory test procedures used for measuring soil suction using thermocouple psychrometers were the same as those described by Johnson.[94-97] The procedure is simple, accurate

Table 16

Summary of Predicted Surface Heaves for Field Sampling Sites

Site No.	Estimated Depth of Active Zone, ft	Potential Vertical Rise (PVR) in.	Empirical Techniques: Nayak and Christensen Method in.	Van Der Merwe Method in.	Seed, Woodward and Lundgren Method in.	Ranganathan and Satyanarayana Method in.	Vijayvergiya and Ghazzaly: Eq. = f(LL, w) in.	Vijayvergiya and Ghazzaly: Eq. = f(u, γ) in.	Schneider and Poor Method in.	Overburden Swell Test Data, in.	Soil Suction Technique, Assumptions for Final Soil Suction Profile: #1 in.	#2 in.	#3 in.	#4 in.	Measured Surface Heave in.
1	8.0 (4)*	4.1	2.5	5.2	84.2	70.1	2.8	6.6	1.3	12.2	11.4	2.0	0.9	--	-0.8
2	9.0 (4)	1.8	13.9	4.7	10.1	9.8	1.9	1.7	1.2	1.4	8.7	3.0	1.2	--	--
3	10.0 (4)	2.3	22.0	5.9	44.6	38.4	1.3	1.8	1.0	1.2	14.6	-3.6	2.8	--	--
4	6.0 (2)	1.4	8.0	3.0	4.1	4.0	1.2	1.1	0.8	0.6	4.0	2.4	0.9	1.3	--
5	9.0 (4)	3.1	16.6	4.7	12.9	21.5	24.1	9.9	20.3	0.7	15.6	0.2	6.2	--	6.5
6	8.5 (4)	1.3	12.7	0.6	2.9	2.2	4.6	9.5	1.1	<0.1	5.3	-2.2	2.3	--	--
7	8.0 (4)	2.2	16.2	2.7	9.7	7.3	11.8	20.0	2.5	0.4	2.7	-1.1	0.6	--	--
8	7.0 (3)	1.5	13.3	3.1	5.2	1.8	5.3	7.3	2.4	<0.1	3.5	-2.0	1.9	--	--
9	9.5 (4)	1.2	12.8	1.8	1.7	2.8	11.4	23.6	3.5	1.5	12.2	0.0	5.0	--	0.4
10	10.0 (4)	1.6	11.6	1.2	2.4	8.5	9.7	13.7	1.3	4.4	23.6	1.0	8.5	--	0.6
11	10.5 (4)	1.9	28.6	3.1	4.9	2.9	32.0	170.8	45.7	0.4	17.2	2.0	1.8	--	--
12	8.0 (4)	4.0	25.2	5.2	35.4	24.2	15.6	19.1	5.8	1.0	8.1	0.3	0.3	--	--
13	9.5 (4)	2.7	15.1	3.0	10.6	27.0	42.6	23.2	2.4	0.4	6.2	-0.2	2.9	--	--
14	7.0 (3)	1.3	11.2	3.5	6.7	6.0	1.4	3.0	0.8	0.7	12.1	-1.1	1.2	--	2.0
15	8.0 (3)	1.4	12.6	5.2	13.7	12.5	0.6	0.7	0.5	0.1	14.1	1.2	3.4	--	--
16	8.0 (4)	1.9	13.6	3.7	8.3	7.1	5.9	2.5	2.6	3.1	5.1	-1.4	2.8	--	1.3
17	7.0 (3)	2.5	10.7	4.0	8.4	11.3	5.0	4.7	2.5	0.7	5.0	1.6	1.8	--	--
18	6.0 (3)	2.2	14.4	4.3	8.8	6.8	13.2	13.0	4.4	1.9	8.6	0.4	5.6	--	--
19	10.0 (3)	3.6	22.1	5.5	16.4	16.4	17.8	22.3	7.9	0.4	7.4	-2.3	1.5	--	1.2
20	8.0 (5)	2.1	15.4	5.2	24.6	24.5	1.3	1.6	0.8	1.8	4.5	1.7	1.8	0.5	--
21a	9.0 (4)	2.9	24.1	2.6	15.2	13.1	102.8	256.0	100.3	1.5	9.0	-1.7	3.4	--	2.2
21b	11.0 (7)	1.9	20.8	2.6	4.7	6.1	23.8	80.0	8.6	13.2	15.7	8.9	2.8	4.4	--

* Values in () are number of layers analyzed for heave prediction.

and practical, and the equipment, Figure 22a, is easy to obtain and relatively inexpensive. Specifically, the procedure involved cutting a section of an undisturbed sample into twelve 1- by 1-in. (approximate dimensions) cubes. The cubes were placed in individual sample containers (1 pt metal cans with interiors coated with wax to prevent corrosion), which acted as environmental chambers when sealed. Of the 12 specimens, two were tested at their natural water contents, and the remainder, depending on their natural water content, were dried (at room temperature) for varying lengths of times and wetted by adding distilled water. Generally, five were dried and five were wetted. The purpose of the wetting and drying process was to establish a range of moisture contents over which the soil suction could be measured. The natural water content and wetted specimens were sealed in the sample containers immediately after cutting and adding water. The dried specimens were sealed after their respective drying time. The sample containers were sealed with rubber stoppers (No. 13-1/2) through which a thermocouple psychrometer had been placed, Figure 22b. When sealed the psychrometer extends approximately 1-in. beyond the bottom of the stopper into the sample container, thus allowing it to measure the relatively humidity within the sample container. All of the sample containers were then placed in a polystyrene temperature chest, Figure 22c. which minimizes temperature variations around the sample containers. The specimens were allowed to equilibrate for approximately 48 hr, after which the soil suction and temperature readings (voltage output) were 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 was determined using the volume displacement method. The water content of the specimens was then determined. The thermocouple voltage output (millivolts) values were converted to temperature (°C) using the following conversion:

$$\text{Temperature, } ^\circ\text{C} = \frac{\text{output in millivolts}}{0.0395 \text{ millivolts}/^\circ\text{C}} \tag{26}$$

The psychrometer voltage output (microvolts) was converted to soil suction in tons per square foot using Equation 6. The microvolt output at the measured temperature was converted to microvolt output at 25°C by

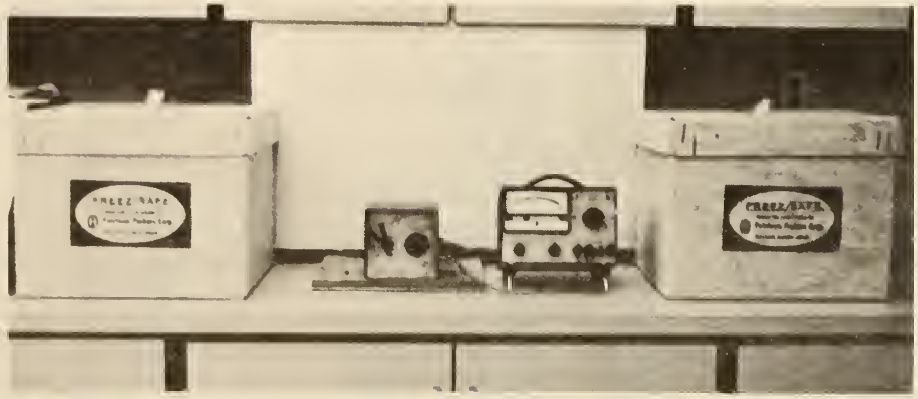
$$E_{25} = \frac{E_T}{0.325 + 0.027\ T} \tag{27}$$

where

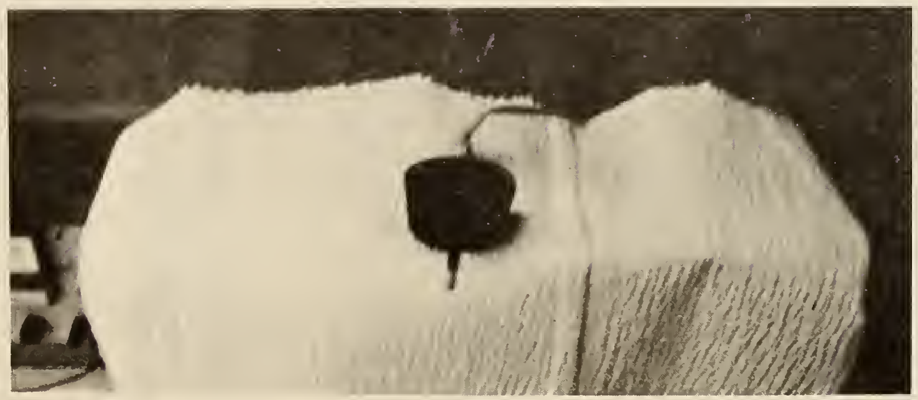
E_T = voltage output at T°C, microvolts

T = measured temperature, °C

193. Soil suction versus water content. Following reduction of



a.



b.

c.

Figure 22. Thermocouple psychrometer equipment for measurement of soil suction; (a) output measurement equipment and temperature chests, (b) thermocouple psychrometer placed through rubber stopper, and (c) sealed sample containers in polystyrene temperature chest

the soil suction data and determination of the corresponding water contents, the data were plotted on a semilog plot with soil suction on the log scale. Generally, three-cycle semilog paper was sufficient to accomodate all of the data points. Keeping track of the points representing natural conditions, all of the data points were 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 were used to establish the line. Examples of soil suction versus water content relationship for Jackson, Miss (sampling site 1), are shown in Figure 23 and may be described using Equation 13. The slope, B, of the line was determined by calculating the inverse of the change in water content over one cycle (i.e., 1-10 tsf) of soil suction. The intercept, A, was calculated by applying Equation 13 at soil suction equal to 1 tsf.

194. Contrary to data obtained using such equipment and procedures as the pressure plate or pressure membrane devices, no hysteresis was indicated in any of the soil suction versus water content relationships. Although no experimental verification was made concerning the 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 then wetting or completely saturating 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.

195. As described in the explanation of Equation 13, 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, 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. Therefore, for all practical purposes the soil suction versus water content relationship represents the total soil suction.

196. Specific volume versus water content. In Equation 14 for the prediction of volume change using soil suction data, a volumetric compressibility factor, α , was included that relates the change in volume to the corresponding change in water content. The value of α was 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 Jackson, Miss. (sampling site 1) are shown in Figure 24.

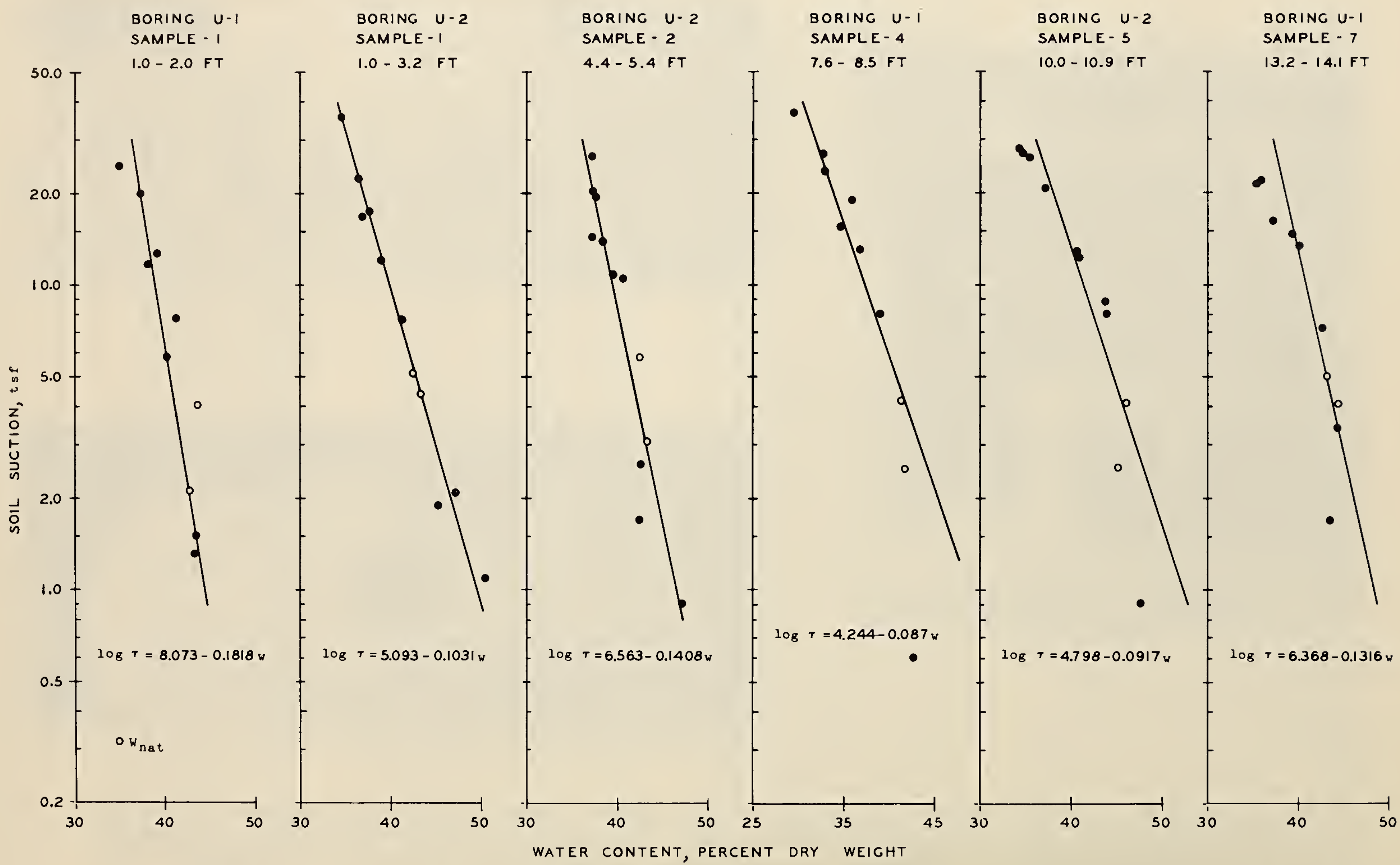


Figure 23. Soil suction versus water content relationships for the Yazoo clay, Jackson, Miss.

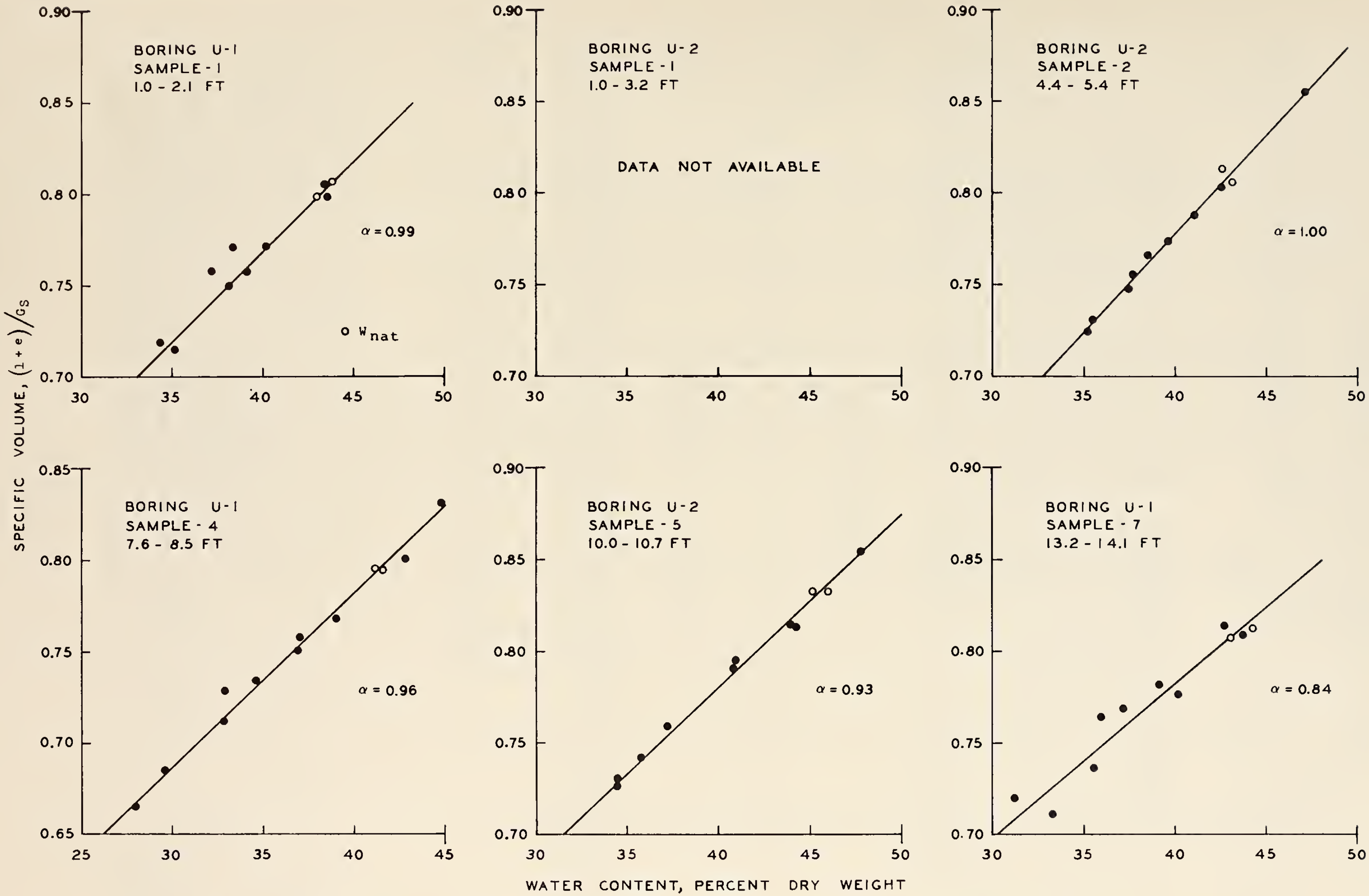


Figure 24. Specific volume versus water content relationships for the Yazoo clay, Jackson, Miss.

197. Lytton[100,111] also used a compressibility factor defined as the fraction of applied pressure that is effective in altering pore pressure. This factor may be related to the α used in Equation 14, but they are not necessarily equal. In view of the fact that additional research beyond the scope of this research program is needed to more fully understand the relationship between the two factors, they are assumed to be equal.

198. Suction swell pressure. In the CVSP test, the specimen was loaded and inundated with distilled water; then the swell pressure measured. If the specimen had been inundated with a fluid (water) identical in ionic composition to the in situ pore fluid, the osmotic influence would have been negated and the swell pressure would have corresponded to the mechanisms that comprise the matrix component of soil suction. In other words, the matrix suction at zero confining pressure may be used to estimate the swell pressure of an expansive soil. In the CVSP test, the degree of saturation, S, is equal to one at equilibrium after sorption of water at a constant void ratio, e. The equilibrium water content in terms of void ratio is

$$\bar{w} = \frac{100\ Se}{G_s} = \frac{100\ e}{G_s} \tag{28}$$

Then the suction swell pressure, SP_s, in terms of void ratio is

$$\log SP_s = A - \frac{100\ Be}{G_s} \tag{29}$$

The suction swell pressure in terms of initial matrix suction with zero confining pressure is obtained by substituting Equations 28 and 29 into Equation 13

$$\log SP_s = \log \tau_{mf} - [A(1 - S)] \tag{30}$$

where S is the initial degree of saturation.

199. The suction swell pressures may be equivalent to the CVSP test value provided the CVSP test is conducted at constant volume until the specimen is fully saturated, which is a virtual impossibility. Therefore, the likelihood of an exact comparison between swell pressures of specimens of the sample tested by the different procedures is remote. Results of the CVSP tests for 20 of the 21 sampling sites are shown in Table 17. Suction swell pressures calculated using Equations 29 and 30 and data summarized in the Summary of Physical and Engineering Properties tables in Appendix A, Volume II for corresponding samples are shown in Table 18. Also shown in Table 18 are predicted swell pressures from the empirical

Table 17

Summary of Data from Constant-Volume Swell Pressure Tests

Site No.	Initial Conditions*				End of Test Conditions				Swell Pressure tsf
	e_o	w_o	S_o	γ_o	e	w	S	γ	
1	1.155	43.6	100+	77.9	1.477	55.0	100+	67.8	3.82
2	0.716	26.1	97.7	97.5	0.765	29.2	100+	94.8	0.96
3	1.265	45.1	97.0	79.4	1.282	46.1	97.8	74.4	0.43
4	0.686	24.4	96.8	100.7	0.697	25.1	98.1	100.1	0.35
5	0.840	27.3	88.8	92.6	0.884	28.5	88.0	90.4	0.90
6	0.421	13.6	91.5	124.3	0.425	14.1	94.1	124.0	0.66
7	0.544	18.0	91.0	111.1	0.559	19.2	94.5	110.1	0.65
8**	0.603	16.0	74.9	109.8	0.603	20.2	94.3	109.8	0.06
9	0.548	9.2	46.2	110.9	0.594	18.9	87.6	107.7	1.17
10	0.727	17.4	67.3	101.5	0.771	27.1	98.9	99.0	0.93
11**	0.306	8.6	77.0	130.9	0.310	10.5	92.0	129.4	0.17
12	0.736	25.8	96.4	98.8	0.784	28.0	98.2	96.2	0.95
13	0.432	14.1	85.9	114.6	0.439	14.9	86.2	114.1	1.16
14	0.765	24.1	88.2	110.1	0.816	27.0	92.5	96.3	0.80
15	1.125	36.3	89.1	81.0	1.145	39.9	96.2	80.3	0.38
16	0.750	13.3	48.2	97.0	0.768	25.5	90.5	96.8	1.00
17	0.725	25.4	94.9	98.0	0.793	28.4	96.9	94.3	1.07
18	0.677	15.8	64.8	103.4	0.751	21.8	80.6	99.2	2.37
19	0.589	17.7	83.0	108.4	0.586	20.8	95.1	108.0	0.59
20	0.990	32.3	88.7	85.3	1.132	41.5	99.6	79.6	2.54

Note: e = void ratio, w = moisture content (%), S = degree of saturation (%), γ = dry density (pcf).

* Corresponds to overburden conditions on e-log p plot.

** No swell pressure developed under applied overburden pressure, indicated swell pressure developed when applied pressure was reduced to seating load.

Table 18

Summary of Predicted Swell Pressures for Constant Volume

Swell Pressure Test Specimens

Site No.	Empirical Techniques: Nayak and Christensen tsf	Komornik and David tsf	Vijayvergiya and Ghazzaly: Eq. = f(LL, w) tsf	Eq. = f(LL, γ) tsf	Soil Suction Swell Pressures: Eq. No. 29 tsf	Eq. No. 30 tsf	Constant Volume Swell Pressure tsf
1	1.35	0.52	0.74	1.94	4.25	4.17	3.82
2	0.61	0.28	0.60	0.58	1.24	1.21	0.96
3	0.60	0.20	0.11	0.50	0.25	0.25	0.43
4	0.60	0.24	0.49	0.46	0.38	0.44	0.35
5	0.58	0.31	1.02	0.47	0.18	0.26	0.90
6	0.63	0.32	0.94	1.31	<0.01	<0.01	0.66
7	1.03	0.51	1.74	3.11	0.27	0.30	0.65
8	0.61	0.22	0.64	0.28	<0.01	0.01	0.06
9	0.54	0.31	1.62	0.50	0.02	0.10	1.17
10	0.33	0.43	2.07	0.91	1.09	1.07	0.93
11	1.22	0.63	3.00	4.22	10.35	7.18	0.17
12	1.33	0.57	1.59	2.47	1.86	1.80	0.95
13	0.54	0.43	1.47	1.84	2.78	2.78	1.16
14	0.92	0.41	1.19	1.35	3.50	3.50	0.80
15	0.71	0.10	0.08	0.10	0.18	0.21	0.38
16	0.64	0.23	1.06	0.20	0.01	0.06	1.00
17	0.37	0.22	0.38	0.48	2.06	2.08	1.07
18	1.67	0.62	3.35	2.12	1.10	1.07	2.37
19	1.32	1.00	5.41	6.73	0.55	0.56	0.59
20	0.88	0.33	0.64	0.63	0.46	0.51	2.54
21a	0.46	0.31	1.50	0.60	0.01	0.07	--
21b	3.65	1.49	9.62	33.54	9.15	6.59	--

techniques and a summary of the CVSP test results for comparison purposes. Suction swell pressure values do not compare very well with CVSP data probably because of the nature of the test, i.e., small increments of deformation are balanced by increments of load that return the specimen to its original void ratio, and the nature of the materials, i.e., many of the materials are hard, indurated shales with low permeabilities and initial degrees of saturation and probably never become totally saturated. No trends with respect to over- or underestimation of the swell pressure are prevalent since in 9 cases $SP_s > CVSP$ and in 11 cases $CVSP > SP_s$. One interesting note, which does not fit the accepted trend, involves sampling sites 11 and 14 for which the SP_s values are considerably larger than the CVSP values. No other samples exhibited this large a difference between SP_s and with $SP_s > CVSP$, and sites 11 and 14 were the only two sites showing osmotic components.[3]

200. Details of heave predictions. Volume change predictions using soil suction data were made using Equation 14. The laboratory data necessary to apply Equation 14 for each of the layers in the sampling site profiles are contained in the respective Summary of Physical and Engineering Properties tables in Appendix A, Volume II. Specifically these data are G_s, e_o, A, B, w_o, and α. The remaining two variables, τ_{mf} and σ_f^o, 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 number of layers analyzed at each of the sampling sites was dependent on the field boring logs and the undisturbed samples retrieved and tested. In most cases, the number of layers corresponded to the number of samples tested in the active zone. However some sites were uniform enough that only selected samples were used, thus the number of layers were reduced. The depth of active zone and the number of layers analyzed are shown in Table 16. The final applied pressure, σ_f, corresponded to the overburden pressure at the midpoint of the respective layer.

201. 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 this research task, the soil suction versus depth profiles for the 21 sampling sites were used to estimate the value (see Appendix A, Volume II). 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, provided a reasonable estimate of the depth of active zone. The "rules of thumb" used were as follows:

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

Example: Sampling site 1, Jackson, Miss., Figure 1 (Appendix A, Volume II): the soil suction became constant below a depth of 7.7 ft and the depth of active zone was set at 8.0 ft.

Example: Sampling site 4, Lake Charles, La., Figure 10 (Appendix A, Volume II): the soil suction became constant below a depth of 5.0 ft and the depth of active zone was set at 6.0 ft.

b. 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.

Example: Sampling site 3, Monroe, La., Figure 7 (Appendix A, Volume II): the soil suction increased to 10.2 tsf at 8.6 ft, then decreased to 5.8 tsf at 11.4 ft, which constitutes the major change in magnitude described above and the depth of active zone was set at 10.0 ft (approximate average of two depths involved was used because of the small difference in magnitude of the soil suction).

Example: Sampling site 17, Newcastle, Wyo., No. 1, Figure 49 (Appendix A, Volume II): 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 magnitude 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 very consistent with reported experience.

202. 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,[91,101] Richards,[86] and Johnson[94,95,97] have all prepared guidelines for estimating final suction profiles and thus the final moisture conditions. In their simplest terms, all of the recommended guidelines may be summarized

as three assumptions, namely, saturated profile, negative hydrostatic, and constant at some equilibrium value. For the purpose of this research, each of these assumptions plus one additional (i.e., $\tau_{mf} = 0$) was used in the comparison of prediction techniques. Figure 25 shows the soil suction versus depth profile at sampling site 1 and three of the four assumptions 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 linear 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 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 (Equation 28) be used in equation 13 to estimate the final suction value. This is probably the most realistic value for estimating potential heave since it involves measured physical properties of the soils rather than assumed relationships. Assumption 4 (not shown in Figure 25) 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, La., Figure 10 (Appendix A, Volume II), the average soil suction beneath the depth of active zone was approximately 0.25 tsf. 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 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 13 to estimate the final soil suction profile.

203. Predicted heave. The predicted heaves using soil suction techniques are summarized in Table 16. The range of predicted values varied with the final soil suction profile assumption as expected. For assumption 1, the predicted heaves varied between 3.5 and 23.6 in. with the remaining values fairly well distributed within this range. For assumption 2, the predicted heaves varied between -2.4 and 2.4 in. For assumption 3, the predicted heaves varied between 0.2 and 8.5 in. with the remaining values fairly well distributed within the range. For assumption 4, only three sites (4, 20, 21b) had soil suction profiles to which the assumption could be applied. The predicted heaves were 1.3 in. for site 4, 0.5 in. for site 20, and 4.4 in. for site 21b.

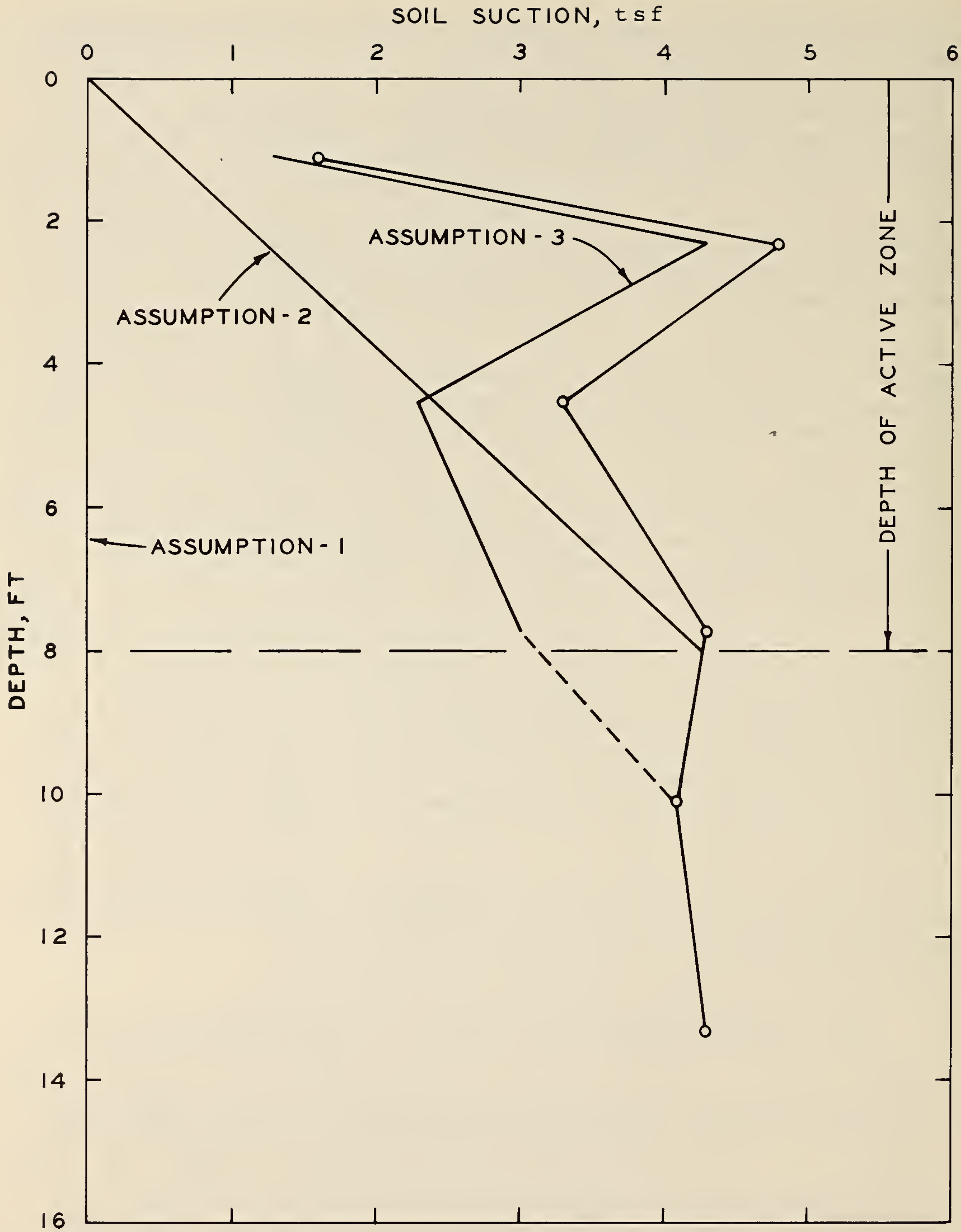


Figure 25. Soil suction versus depth at sampling site 1, Jackson, Miss., with various assumptions for final soil suction profile

Prediction of
swell-empirical techniques

204. In previous sections of this report, eight different empirical techniques were introduced (seven for swell and one for swell pressure), and procedures for their application described. The empirical techniques represent efforts to simplify the testing and prediction portion of the analysis and design sequence by correlating measured laboratory swell with various physical or index properties. The techniques vary from simple one-variable equations, such as the Ranganathan and Satyanarayana method, to several three-variable equations, such as the Vijayvergiya and Ghazzaly method. In some cases reported, local experience has been very good; however, the general applicability of these types of procedures was found to be somewhat limited.

205. Details of heave prediction. The seven empirical techniques involving heave calculations were used to calculate predicted heave at each of test sites using the same depth of active zone and number of layers in the profile. Data for application of these empirical techniques are presented in the corresponding Summary of Physical and Engineering Properties tables in Appendix A, Volume II. Actual calculations were according to the equations reported in the literature and reviewed in previous sections of the report.

206. Predicted heave. Predicted heaves for the seven empirical techniques are summarized in Table 16. Simply noting the relative magnitudes of the predicted heaves essentially eliminates three of the empirical techniques (i.e., Nayak and Christensen; Seed, Woodward, and Lundgren; and Ranganathan and Satyanarayana). Of the remaining methods, only two (PVR and Van Der Merwe method) show close agreement with the measured values for more than one site. These two methods are closest to the measured values (i.e., slightly below or above the measured heaves) at seven of the eight field sites. More discussion on the relative accuracy of all of the methods will be included in subsequent paragraphs.

Comparison with
measured field data

207. Evaluation of testing and prediction procedures is complicated by the large number of geotechnical properties and environmental condition that influence the behavior of expansive soils. To minimuze the influence of these variables, the option selected for evaluation of test and prediction techniques under this research task was direct comparison between predicted and measured volume change. A group of eight field monitoring sites, which will be described in more detail later, were monitored for a minimum of two years to provide data for comparisons with predicted heaves using various prediction procedures. The direct comparison procedure does not eliminate all of the variable influences; for example, climate, drainage, and vegetation are factors that influence field behavior but cannot be simulated or accounted for in the laboratory.

Therefore, to determine the accuracy and reliability of testing and prediction procedures, these and other variables not properly accounted for should be considered before making recommendations for further use of any techniques.

208. For the evaluation by comparison, the predicted and measured heaves are considered to be ultimate values, and the closer the predicted and measured heaves are the more accurate the procedure is in estimating volume change. For the purpose of this evaluation, predicted values within 25 percent (plus or minus) were considered to be within reason. In lieu of the 25 percent accuracy figure, conservatism was considered as an alternative evaluation indicator. In other words, conservatism (predicted > measured) was given high comparative ratings. On the other hand, procedures that were excessively conservative (predicted >>> measured) were considered less favorable since they resulted in overdesign of the pavement or overtreatment of the expansive soil.

209. Odometer swell test data. Predicted versus measured heave at the eight field monitoring sites using OS test data is shown in Figure 26. None of the data fall within the $\pm$25 percent criteria, and the split between conservative and nonconservative is even (i.e., four in each category). In addition, one site in each of the conservative (site 1) and nonconservative (site 5) categories is excessively conservative or nonconservative. The point of the matter is that the data make an evaluation decision difficult since there is a nearly even split with regard to predicted versus measured behavior. Therefore, it is difficult to determine where the problem with odometer-type tests is the result of the test procedure or the result of the limited simulation of field conditions. It is evident from the data in Figure 26 that the OS test provides reasonable data in view of the general cluster of six data points around the line of equality. One general source of error that may have adversely influenced the predicted values involves the use of a single OS test to represent the entire profile. In addition samples from site 1 were taken in the early spring following three years of above average rainfall. Since sampling and during the monitoring period (approximately 1 year), the rainfall has been at or below average thus modifying (i.e., decreasing) the moisture regime and resulting in the volume decrease. At site 5, the samples were taken approximately six years after the beginning of observations, and the predicted heave is based on moisture conditions after ponding and approximately five years of adjustment to environment conditions.

210. 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.

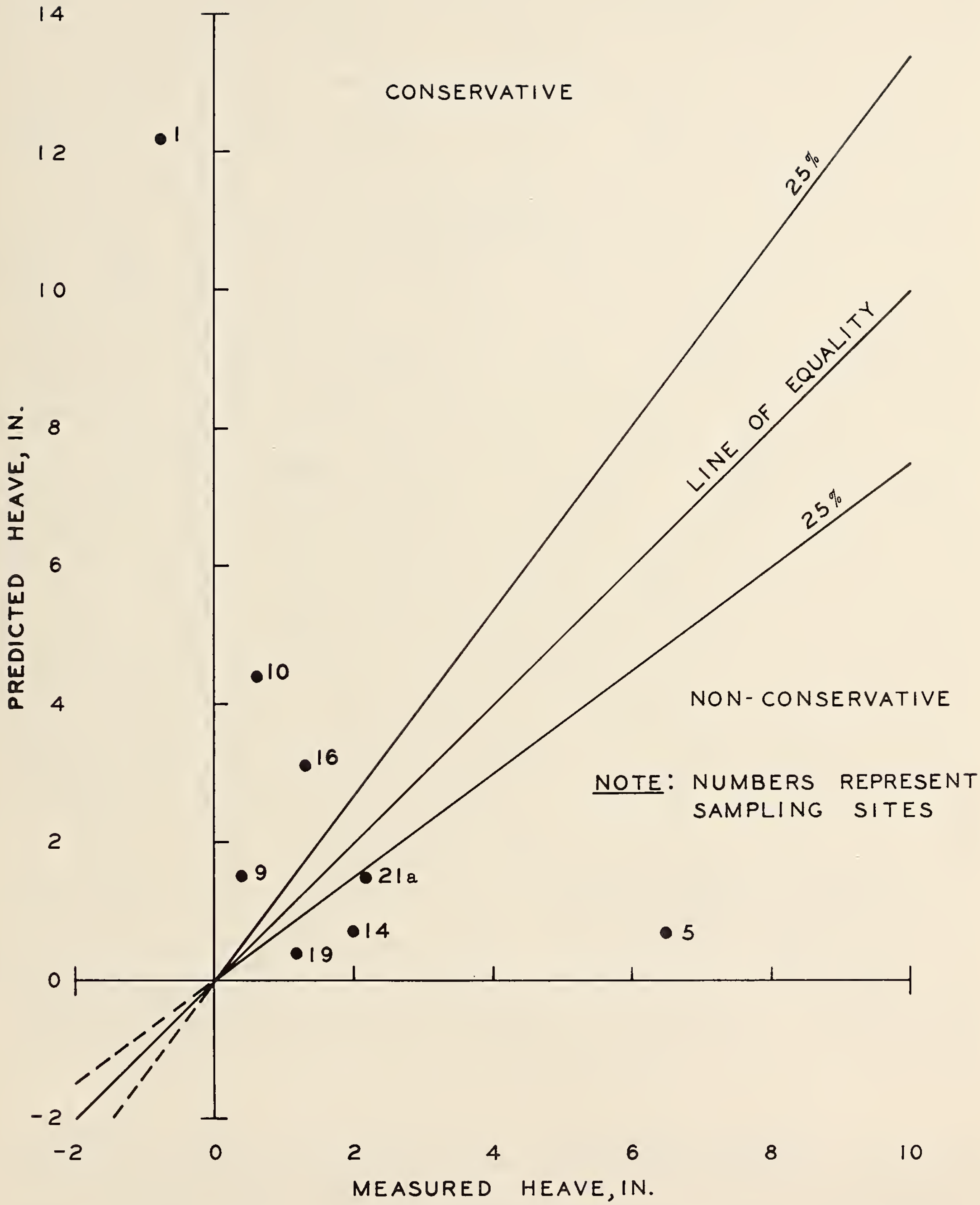


Figure 26. Comparison of predicted versus measured heave at field test sites (predicted data based on odometer (OS test) technique)

211. Soil suction test data. Predicted versus measured heave for the eight field monitoring sites are shown in Figures 27, 28, and 29 for final soil suction profile assumptions 1, 2, and 3, respectively. None of the field monitoring sites exhibited initial soil suction profiles that allowed application of assumption 4. Assumption 1 requires that the final soil suction (τ_{mf}) in Equation 14 be equal to zero through the depth of active zone. It is obvious from Figure 27 and a basic understanding of the soil suction concept, that this assumption is far too conservative for practical applications. In order for soil suction to be effectively reduced to zero, the entire profile would have to be submerged, and even then the permeability of these types of materials may never allow for total saturation. Assumption 2 requires that the final soil suction vary linearly from zero at the ground surface to the initial measured profile value at the depth of active zone. In Figure 28, six of the eight predicted heaves are nonconservative and five of those values are excessively nonconservative. In fact, predicted values for four of the sites indicated shrinkage (consolidation). This is not unlikely because of the S-shaped soil suction profiles, provided the change in soil suction is comparable to assumption 2. Again it should be noted that there is no way to effectively and accurately predict moisture regime changes, so to compensate for this void in technology, assumptions must be made to balance practicality, reality, and accuracy. Assumption 3 requires that the final soil suction profile be determined by estimating the saturated water content (Equation 28) for samples representing the soil profile, then calculating the final soil suction for saturated conditions (Equation 13). In Figure 29 at two of the eight sites, predicted values fall within the 25 percent limit (sites 5 and 19), one value just outside the 25 percent limit on the nonconservative side (site 14), and the remainder of the predicted values on the conservative side. Predicted values at sites 9 and 10 are overconservative based on assumption 3. One of the major reasons for the better comparisons using assumption 3 is that the change in soil suction profile (initial to final) more closely approximates the reported actual changes in the moisture regime with time. For site 9, subgrade moisture variations were monitored along with surface deformations (see Appendix D, Volume II). The maximum changes in water content at site 9 were: depth = 2 ft, Δw = +2.5 percent; depth = 4 ft, Δw = +0.7 percent; depth = 6 ft, Δw = -1.4 percent; and depth = 8 ft, Δw = +0.3 percent. These values were used to calculate the final soil suction profile which was, in turn, used to calculate the predicted heave. The resulting predicted heave was 1.1 in. This significantly emphasizes the need to develop a better understanding of the influence of climate and seasonal change on the soil suction profile. Where experience and field data similar to that collected at site 9 are available, then the final soil suction profile assumptions should be modified to incorporate the information.

212. Empirical techniques. As indicated earlier, two of the seven empirical techniques that predict heave provided the closest estimates of field heave. These were the PVR and Van Der Marve (VDM) methods. Predicted versus measured heaves for the PVR and VDM methods are shown in Figures 30 and 31, respectively. Both techniques have one predicted

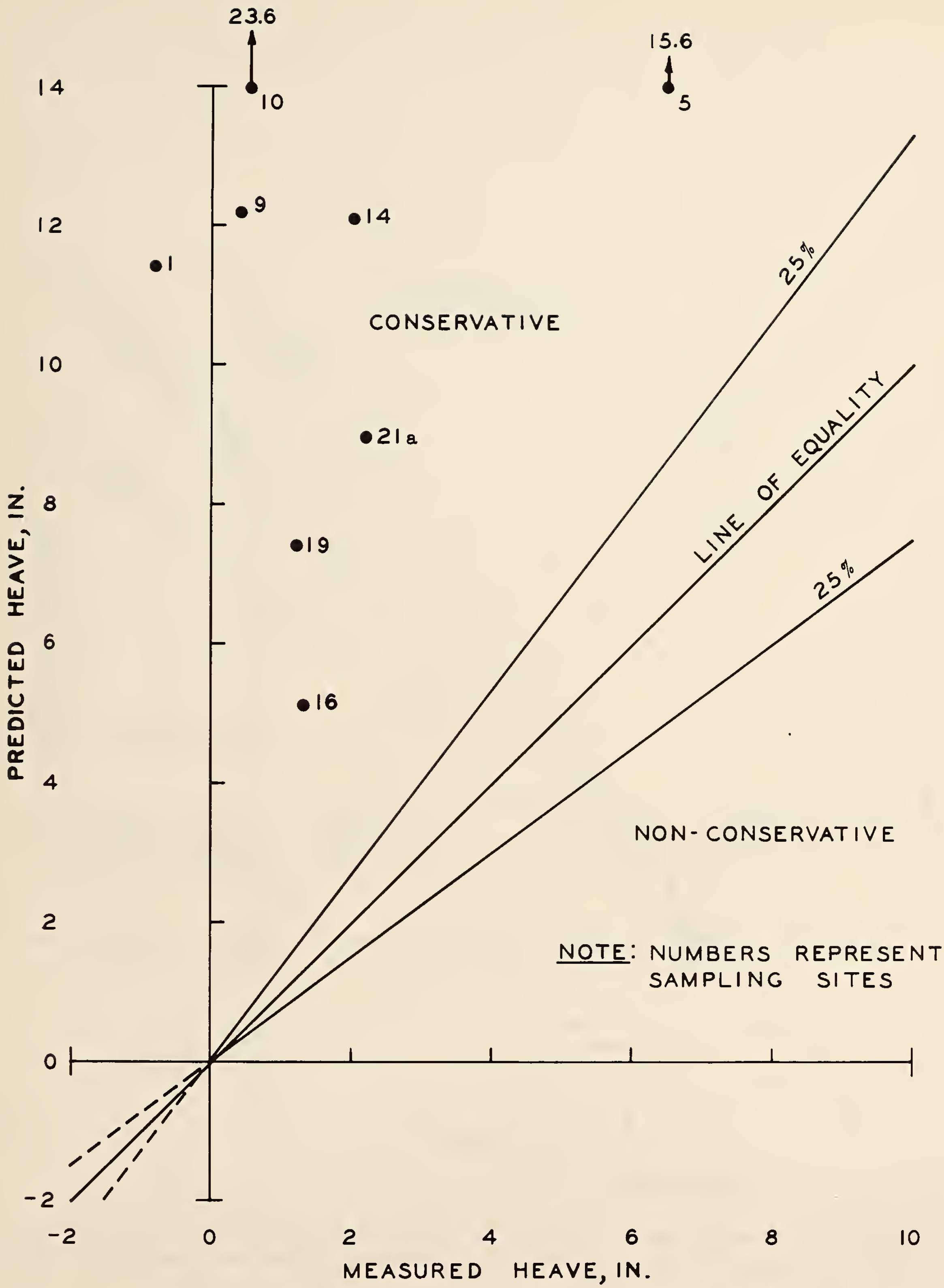


Figure 27. Comparison of predicted versus measured heave at field test sites (predicted data based on soil suction technique, assumption 1)

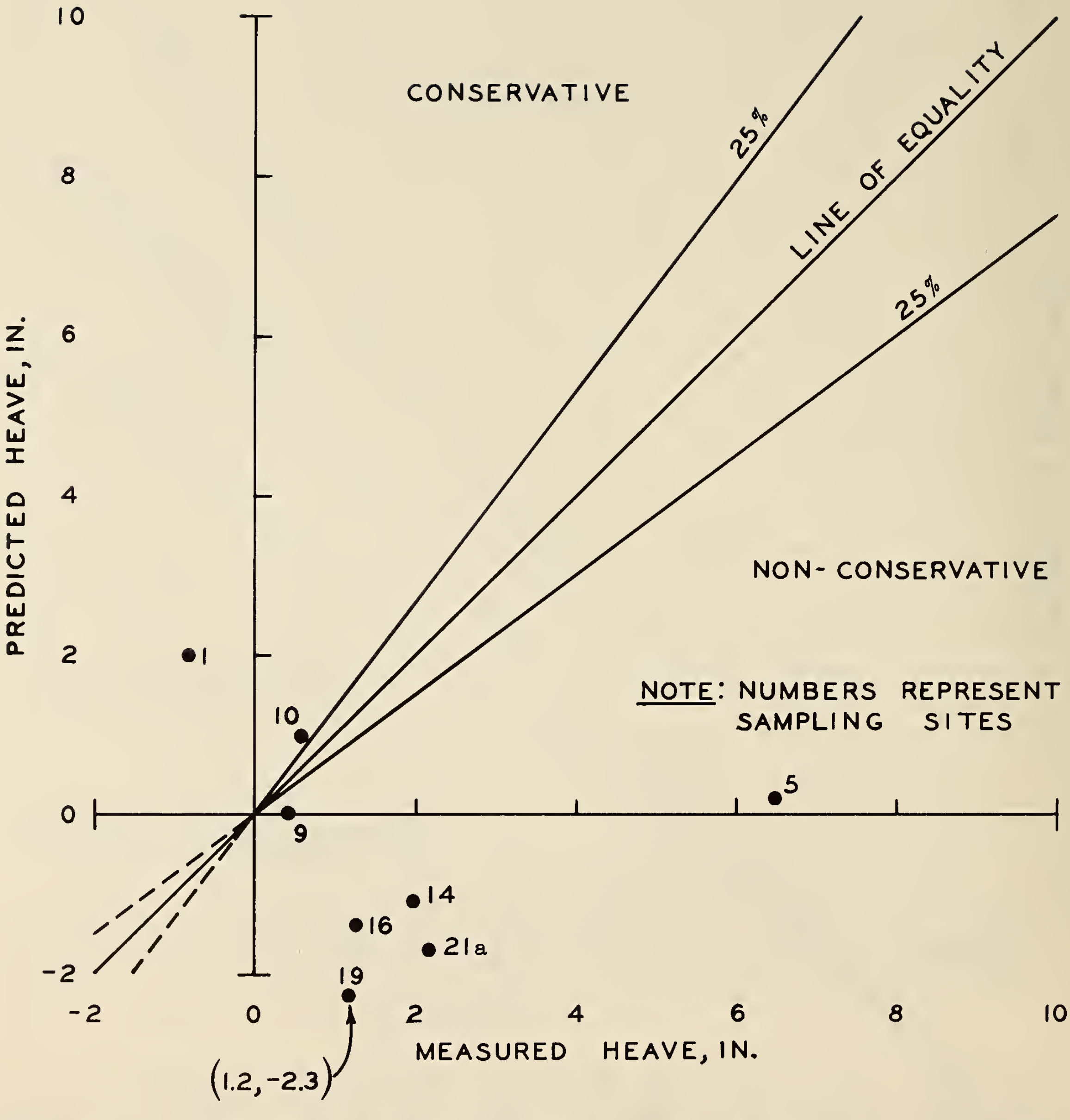


Figure 28. Comparison of predicted versus measured heave at field test sites (predicted data based on soil suction technique, assumption 2)

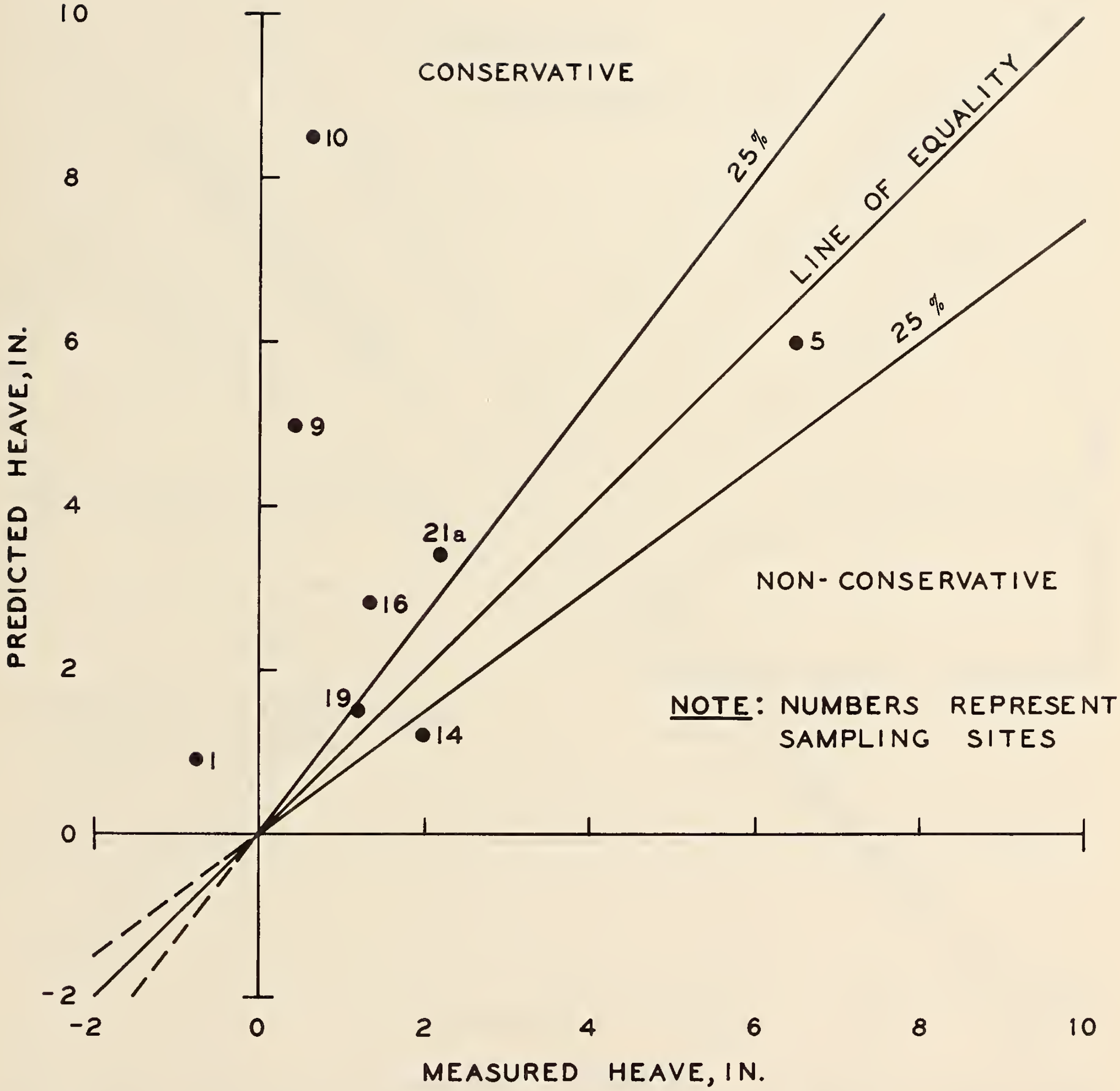


Figure 29. Comparison of predicted versus measured heave at field test sites (predicted data based on soil suction technique, assumption 3)

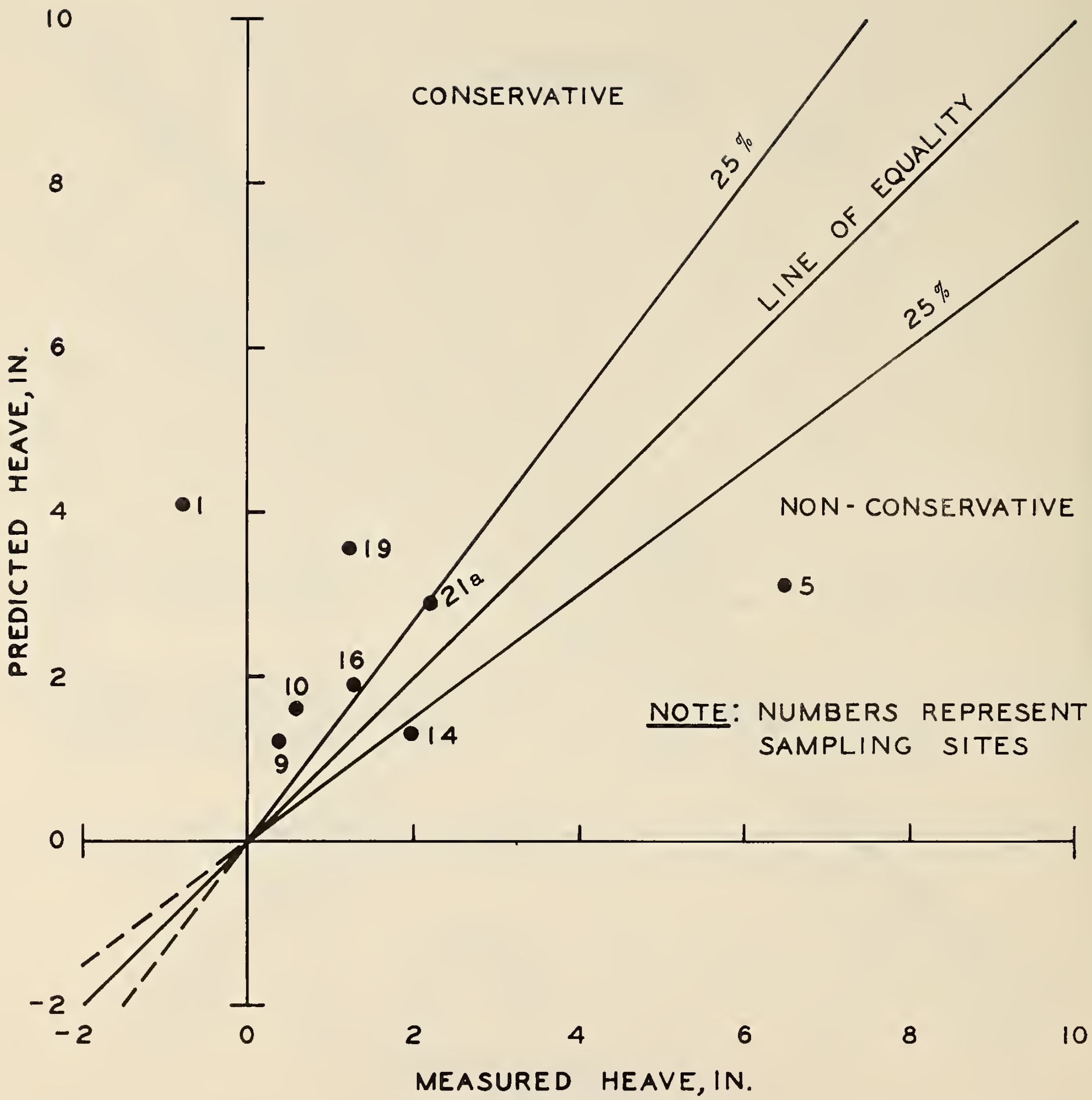


Figure 30. Comparison of predicted versus measured heave at field test sites (predicted data based on Potential Vertical Rise method)

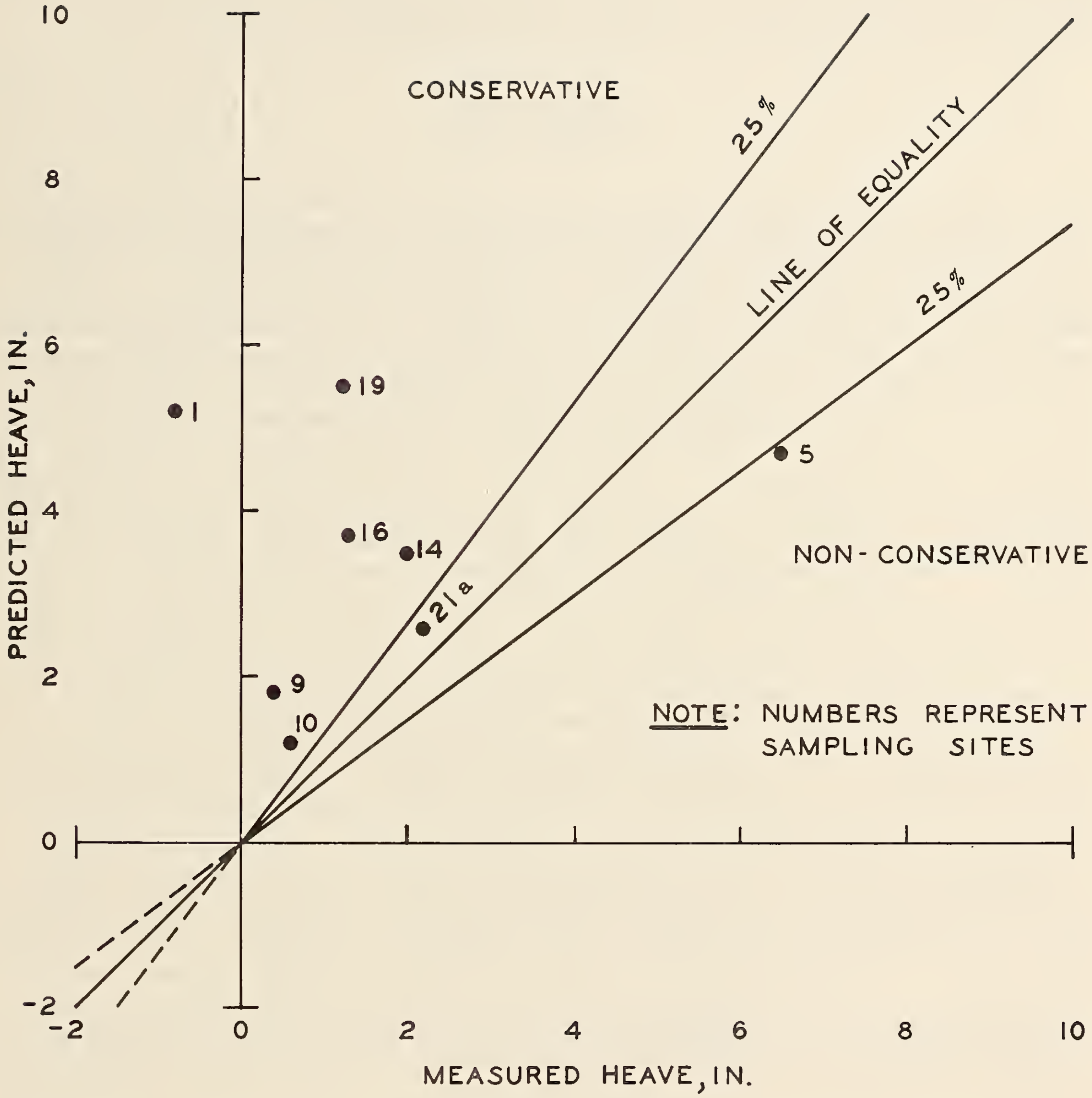


Figure 31. Comparison of predicted versus measured heave at field test sites (predicted data based on Van Der Merwe method)

value within the 25 percent accuracy limit, and the majority of the predicted data are on the conservative side of the equality line. For the PVR method, six of the predicted heaves are conservative while seven predicted heaves are conservative for the VDM method; however, two of the predicted heaves (sites 1 and 19) are overly conservative for the VDM method. The probable reason for the better accuracy of these two empirical techniques is that both require adequate profile definition and consider the influence of overburden on the predicted heave.

Rate of Heave

213. 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 then the magnitude. 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 calculation of rate of heave has been studied by investigators with marginal success. 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 not only is 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 the consolidation theory applied in reverse. More recently, efforts have been made to use diffusion theory[97,100,112] to predict rate of heave. Johnson[97] has developed a finite difference computer solution to the diffusion equation that has been reasonably successful in estimating the heave at selected times. The major problem with use of the computer program 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 and recommending that future research concentrate on more accurate and practical rate prediction methods.

Recommendations for Usage

214. In this part of the report, procedures for testing expansive soils and predicting the magnitude of potential volume change have been reviewed and evaluated. The problems associated with the use of the various testing and prediction techniques have been defined and, where

possible, alternatives for minimizing the effects of the problems have been discussed. The results of the investigation are not totally conclusive; however, enough definite trends are available to formulate recommendations for use of the testing and prediction techniques. These recommendations are summarized in the following paragraphs.

215. Based on the published experience of others[86,89,90,94-98,100] and the results of this investigation, the soil suction concept should be used on a routine basis to characterize the volume change behavior of potentially expansive soils. The measurement of soil suction by thermocouple psychrometers is a simple, inexpensive, accurate, and reliable procedure, which is readily implementable. From a practical point of view, the soil suction procedure is much less time-consuming than odometer procedures, and the measured data are applicable to a wide range of moisture conditions. The testing procedure and equipment discussed in previous sections should be used to measure the soil suction parameters and estimate the anticipated volume change. Procedures outlined for estimating the depth of active zone and final soil suction profile should be used in conjunction with soil suction testing and prediction technique. In an abbreviated step-by-step description, the testing of expansive soils and prediction of anticipated volume change should include:

- 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 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 parameters A, B, τ_{mo}, and α defined.

- c. Selection of the depth of active zone and final soil suction profile. Then the anticipated volume change should be calculated for each layer in the profile and summed to obtain the total surface movement.

216. If soil suction measurement equipment is not available, then two options are possible for testing and prediction of expansive soils. One is the OS test that, if used, should be conducted using the procedure outlined in this report. The OS tests should be conducted on a comparable frequency to the soil suction test; however, the time to obtain the data and the limited range of application of the data may preclude this frequency. The other option involves the use of empirical techniques, specifically the PVR, since most State Highway Agencies are familiar with the method and it is well documented in published literature. The decision to select the alternative options will hinge on available equipment, time available to obtain the data, and extent of the detail

of the analysis required. From a practical point of view and when additional detail is desired for a given profile, the PVR would be preferable. The procedure for calculating the PVR is outlined in detail in Reference 102. Further details concerning the recommendations for testing and prediction procedures will be included in the technical guidelines being prepared in conjunction with the study.

PART VI: EVALUATION OF PRE- AND POST-CONSTRUCTION TREATMENT ALTERNATIVES

217. 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 which 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 of more extensive 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 phase of the research strictly was to develop the guidelines for systematic and logical selection of treatment alternatives or combinations of treatments that effectively minimize volume change and the associated damage to pavements.

218. 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, physically, 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.

Avoid the Expansive Soil

219. 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 pressures 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[2] show subjective categorizations of potential swell; natural hazards maps currently being prepared by the National Science Foundation and U. S. Geological Survey include expansive soils as a topic; recent Soil Conservation Service county soil surveys include tables relating many engineering properties such as shrink-swell potential; and State Geological Survey publications, such as the one prepared by Hart[113] in Colorado, map potentially swelling soils on detailed scales. This abundance of information combined with State Highway Agency experience is making the option of avoiding the expansive soil problem a more viable alternative in more situations.

Mechanically, Physically, or Chemically Alter the Expansive Soil

Mechanical alteration

220. 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 w-γ 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.

221. 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).

222. Performance experience from highways constructed on expansive soil subgrades where ripping or scarifying was used is limited at best.

In Mississippi,[114] experimental construction on I-220 near Jackson involved subexcavation; however, cut sections outside the experimental construction area were ripped to a depth of approximately 1.5 ft below final grade and then recompacted. The purpose of the ripping was twofold: first, to disturb the natural fissure and cracking pattern of the Yazoo clay; and second, to improve the uniformity at cut/fill interfaces. The ripped sections will provide the standard sections for comparison with the experimental construction. At present, the project is being paved; therefore, some time will be required to verify the influence of ripping as well as the experimental construction. The other reported experience with ripping of the subgrade involves the field monitoring section at Morrison, Colo.,[115] see Appendix H, Volume II. Standard construction for the cut sections in the relocation of State Highway 8 involved ripping the subgrade to a depth of approximately 2 ft below final grade, wetting, then recompacting with the top 6 in. compacted to 95 percent AASHTO T-99 density and optimum minus 2 percent. The purpose of the standard section treatment was to increase the moisture content and decrease the density of the natural material, as well as to disturb the oriented fabric of the clay particles. In addition, the treatment provided more uniformity at the cut/fill interface. The test feature involves the use of a fabric membrane extending from beneath the edge of the shoulder, down the verge slope, and up the backslope, and will be discussed in a later section of the report. At present, field monitoring data are somewhat limited concerning the performance of the test versus standard section; however, the general trend gives a slight edge to the test section treatment. In other words, the standard section treatment is performing nearly as well as the test feature (i.e., membrane).

223. Experience with subexcavation has been better documented than the ripping or scarifying; however, the success of the treatment has depended primarily on the type of material replacing the subexcavated material. The more granular the backfill, the less successful the application of the treatment alternative as explained in paragraph 225.

224. In South Dakota, experience[116] has shown that limited undercutting and recompaction of the subgrade (6-18 in.) did not solve their pavement warping problems. Although placement of lime-treated select base course gravel did improve the overall serviceability of the treated roadways, it did not provide the desired long-term results. Therefore during construction of I-90 through the Pierre shale in central South Dakota, more extensive subexcavation and recompaction treatments were applied. The maximum depth of application[117] was 6 ft with isolated fault zones excavated to a maximum depth of 12 ft. A summary of the performance of I-90 between Cactus Flats and Chamberlin is shown in the following tabulation.

Roughness Index (RI) Values for Subexcavated Sections, I-90, MP 134 to MP 265

Milepost (MP)	Date of Construction	RI Values At Date of Construction	1975	1976	1977
134*-149	1968	4.70	4.38	4.40	4.26
149-160	1969	4.70	4.26	4.26	3.84
160-174	1969	4.70	4.29	4.16	4.16
174-183	1970	4.70	4.17	4.15	3.86
183-198	1970	4.70	4.32	4.29	3.99
198-213	1971	4.70	4.27	4.17	3.59
213-226	1971	4.70	4.38	4.25	4.15
226-236	1972	4.70	4.57	4.49	4.38
236-251	1973	4.70	4.63	4.60	4.40
251-265**	1972	4.70	4.18	3.96	3.72

* MP 134 Cactus Flats, S. Dak.
** MP 265 Missouri River at Chamberlin, S. Dak.

Compaction specifications required the 6-ft zone to be a minimum of 92 percent of AASHTO T-99 with a target density of 95 percent; the minimum water content was AASHTO T-99 optimum with a target value of 3 percent above optimum. As indicated in the tabulation, the overall serviceability of the entire section of roadway is very good, particularly for some of the older sections.

225. The Colorado Department of Transportation has reported both successful and unsuccessful experiences with subexcavation. The unsuccessful experiences[118-121] involved subexcavation to a depth of 2-ft and backfill consisting of various gradations and types of granular materials. The pervious granular material permitted entrance of moisture from surface runoff and hydrogenesis that resulted in swelling and pavement distortion. Open-graded gravels were found to be the worst offenders of the gradations. As in South Dakota, it was determined that minimal subexcavation (i.e., less than 2 ft) was not sufficient to significantly reduce pavement distortion. During construction of selected sections of primary and interstate highways, particularly around Denver, more extensive subexcavations were used. Based on this experience and that in other areas,[122] standards were adopted for suggested depth of moisture-density control below grade for cuts and tops of fills. For interstate and primary highways, the depths of moisture-density control[123] are as follows:

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

226. In an effort to obtain the benefits of subexcavation for postconstruction application in some of the hard indurated shales of Western Colorado, the Colorado Department of Transportation has used controlled blasting of the subgrade materials.[124] Borings were driven through the pavement and into the subgrade shale (depth and spacing based on geologic material and experience), and the holes were loaded with explosives, backfilled, and detonated. The result was a fractured and reoriented material that, if moisture was kept out, had a lower overall density and a lower tendency for volume change. Except for the initial experience in which the entire pavement was "fractured," the alternative has been successful in reducing swell in areas that have a history of high volume change potential.

227. Experiences in Wyoming[125,126] with the use of untreated gravel bases placed on expansive subgrades, whether subexcavated or not, are similar to Colorado's experience. The thicker and cleaner the gravel base layers, the greater the heaves. Experience with subexcavation and replacement using moisture and density control indicates that where interbedded layers intersect the subgrade, a more uniform subgrade is obtained and many of the short, choppy heaves are eliminated. However, the general conclusion from the use of moisture and density control in hard, dry shales is that it places moisture in areas where it ordinarily would not reach; thus, the better approach is to prevent moisture intrusion (i.e., membranes).

228. Although minimal experience is currently available, the Mississippi Department of Transportation is currently using subexcavation and replacement with density control for new construction and reconstruction of an extensively damaged section of highway. The new construction involves the experimental section of I-220 previously described. In the experimental section, two of the test features involve excavation of the material above the horizontal line between the ditch inverts (i.e.,

approximately 3.7 ft at the pavement center line). The material is being replaced at 90 percent of AASHTO T-99 maximum dry density with no moisture control. Sections of I-20 west of Jackson, Miss. badly damaged by the Yazoo clay, have been removed, the subgrade material has been subexcavated to a depth of approximately 3.5 ft, and the natural material placed at 90 percent of AASHTO T-99 maximum dry density with no moisture control. The major reason for subexcavation in this area is to destroy the fractured and fissured nature of the natural material that provides pathways for the ingression of moisture.

229. Surcharge loading of expansive soils using additional pavement layers or fill have been reported in California[127] and Utah.[128] Conclusive field data on the application and performance of this treatment alternative are limited; however, at best, the procedure is limited in application to materials with low potential expansive pressures since the moderate and high potentially expansive materials would require excessively high fills to balance the swell pressure.

Physical alteration

230. In this context, physical alteration refers to attempts to minimize swelling potential by mixing granular (sand) or nonswelling cohesive (silt) materials with expansive soils. Minimal experience has been reported with this treatment alternative in the United States. What little work done considering this alternative has been done in South Africa or India. The obvious advantage of the alternative is the "dilution" of the expansive clay minerals with the nonexpansive materials. The major disadvantage of the alternative includes the fact that nothing is done to modify or alter the expansive nature of the clay minerals. In other words, the soil is still expansive, only to a lesser extent than before. A second disadvantage involves the influence of the nonexpansive material on the permeability of the combined soil. Sand or silt mixed with an expansive clay reduces the potential for swell but increases the permeability of the soil mass so that more water gets in faster. For these reasons, no further consideration was given to this alternative in the evaluation.

Chemical Alteration

231. Chemical alteration refers to the addition of chemical compounds to alter the characteristics of the clay mineral or clay-water combination that, in turn, reduces the potential expansiveness. Literally hundreds of chemicals have been tried. Cementation by lime, lime-fly ash, 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.[129] Lime continues to be the most widely used and effective additive for modification of expansive clays.[129-133]

232. In an extensive experimental construction section, the South Dakota Department of Transportation evaluated the field performance of various chemical stabilizing agents.[131-132] A test road composed of various cross sections (Figure 322, Appendix K, Volume II) included untreated sections so that test versus control comparisons could be made. Figures 323 through 344 (Appendix K, Volume II) represent subgrade moisture content variations (depth $\approx$2 ft), roughness index, California Bearing Ratio (CBR), and plate-load results for selected sections of the experimental construction. The data cover an average period of approximately 12 years (1964 to 1976). Using the roughness index as a basis for comparison, the figures show that the lime-treated sections have resulted in better ride quality throughout the monitoring period. The fact that a relatively small difference between the treated and untreated sections exists was explained by the deep-seated nature of the expansive soil problem versus the relatively shallow treatment depths; thus, the decision was made to use subexcavation to greater depths, as described in previous paragraphs.

233. 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.[134-136]

234. Conventional and deep-plow lime application procedures are applicable to preconstruction conditions. Two procedures have been used to apply lime in postconstruction situations, namely, drill-hole lime and lime slurry pressure injection (LSPI). Both procedures have been the center of controversies concerning their mechanisms and successful performance. Although conclusive data to clear up the controversies were not obtained during this study, the following experiences describe the procedures and some of the more successful applications.

235. <u>Drill-hole</u>. This 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[137] 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 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. The question then arises, why use the

lime? Why not drill holes and backfill with the same material wetted and lightly compacted? Unfortunately, information on this type of application is nonexistant. Nonetheless, the drill-hole lime has a relatively good success record, as noted by the following experiences.

236. The Oklahoma Department of Transportation[138] has reported several successful instances of drill-hole lime application. Typically 9-in.-diam, 30-in.-deep holes on 5-ft centers were backfilled with lime slurry, and the upper portion of the holes was sealed with natural material and asphalt cold-mix.

237. Colorado Department of Transportation experiences with the drill-hole lime techniques have proven quite successful.[139] Generally, 12-in.-diam holes with depth ranging from 6 to 20 ft, depending upon the extent of treatment desired, on a 5- by 6-ft grid or 5-ft centers were used. Experience showed that slurry concentrations greater than 1 lb of lime per gallon of water result in less lime and water migration. Holes at least 12 in. in diameter were routinely used and recommended since smaller diameter holes provide less surface area for contact with the slurry. The mechanism of stabilization observed indicated that lime did not migrate over 2-3 in. from the periphery of the hole and mostly at the bottom of the hole (lime is not completely soluble in water and thus settles out). Therefore, the swelling potential is reduced, as indicated above, due to moisture increases and stress relief.

238. As a remedial measure, the South Dakota Department of Transportation[140] placed a lime slurry composed of 1-part lime, 1-part water, and 1-part sand into 4-ft-deep holes placed on 5-ft centers (no hole size given) into an expansive subgrade of Pierre shale. Results showed some reduction in the frequency and sharpness of the bumps. With time, a definite improvement in the roughness index (serviceability) was noted for these sections over companion untreated sections.

239. Lime slurry pressure injection. 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 are: first, 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).

240. At present, no experience has been reported in which 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.[141-144] Primarily, the LSPI has been used in railroad track stabilization to increase the strength of the foundation soil. However, the Mowry formation of southwest South Dakota, generally recognized to be moderately expansive, was effectively stabilized with regard to volume change.[145]

241. Wright[146,147] also observed that when lime slurry is injected into heavy clay, the slurry migrates through available fractures and fissures in the soil, creating a network of lime seams. The added moisture may cause a noticeable swell of 2-8 in. at the time of injection, depending upon the original moisture content of the soil. This preswell is beneficial as the lime seams and the upper 4- to 6-in. stabilized layer create moisture barriers that assist in maintaining a constant moisture content and thus eliminate subsequent cracking and swelling. Because of the lime seam effect, the quality of the LSPI cannot be evaluated by conventional tests, i.e., Atterberg limits, pH, swell, or strength tests, on recovered samples.

242. Ingles and Neil[148] evaluated lime and cement grouting at seven sites in Australia. Two limewater grouts, 1:1 and 1:2 by weight, and a comparison cement grout, 1:1, were injected under pressure into the soil via sealed 4-in. auger holes ranging from 3 to 8 ft deep. Visual inspection of recovered cores showed that the grout penetrated fissures and not pores. In this context, dry-season grouting, when desiccation cracks are most prevalent, enhances grout penetration. Postgrouting results indicated that surface movements occurred shortly after grouting due to the moisture being added, but that surface level fluctuations in montmorillonitic soils and total swell were reduced by 50 percent over untreated areas. By comparison, cement grouting was less satisfactory with surface movements in the montmorillonitic soil being reduced by 10 percent.

243. In a recent publication (1975), Thompson and Robnett[149] summarized that although there are conflicting reports concerning the effectiveness of LSPI, it seems logical to conclude that LSPI may be an effective swell control procedure under certain circumstances. The condition most favorable to the achievement of success with the LSPI treatment of expansive soils is the presence of an extensive fissure and crack network into which the lime slurry can be successfully injected. The treatment mechanisms explaining LSPI effectiveness, i.e., prewetting, development of soil-lime moisture barriers, and effective swell restraint with the formation of limited quantities of soil-lime reaction products, all have validity.

244. 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 design 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. A more detailed description of procedures for doing this is the subject of subsequent paragraphs. 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.[142] 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.

Lime content
for minimizing swell

245. Whether lime is being used in preconstruction (mix-in-place) or postconstruction (drill-hole or LSPI) application, the basic question that must be answered is whether the lime will react with the soil to a sufficient degree to produce positive results. If only mix-in-place applications are considered, then two additional questions arise: namely, How much lime is required to produce the required results? How effective is the lime in achieving the required results?

246. As the percent lime added to a responsive soil is increased, the plasticity will be sharply reduced and the pH will increase rapidly. The lime content at which the plasticity approaches a near constant value and the pH approaches 12.4 has been termed the "lime fixation point."[150] At this lime percentage, maximum compression of the ionic atmosphere and all cation exchange have occurred as indicated by the constant pH. The soil has become saturated with calcium ions. Since this lime percentage represents maximum modification conditions, it is denoted as the "modification optimum." Although small strength gains are obtained with lime percentages below or at modification optimum, these gains are probably related to aggregation of clay particles and increased angle of internal friction of the cohesive soils rather than extensive cementation by pozzolanic reactions. As the lime content is increased above the modification optimum, additional strength increases are obtained until a maximum is achieved, and strength reductions result with further addition of lime. This maximum strength lime content is generally referred to as the "stabilization optimum" and represents optimum conditions in the soil for pozzolanic reactions forming cementatious minerals. Numerous mix design procedures[151] exist for selection of lime stabilization optimum; however, limited data are available on determination of lime modification optimum, particularly with respect to reduction of volume change in expansive soils. The general criteria for selection of lime modification optimum involved running Atterberg limits (LL and PL) on soil-lime mixtures with the lime percentage varied. The minimum lime percentage producing the desired reduction in plasticity

was taken as the lime modification optimum. The problem with this approach was that often more tests were run than actually needed to determine the lime modification optimum.

247. Eades and Grim[152] published a quick test method for estimating lime requirements for stabilization. Basically, the test consists of monitoring the pH of various soil-lime mixtures with increasing lime contents until a constant pH of approximately 12.4 was achieved. Their results showed that the lime percentage resulting in a pH of 12.3 to 12.4 was sufficient to minimize plasticity and, with curing, provided some strength increase, but that additional lime usually resulted in longer reaction and increased strength. Thompson and Eades[153] presented results showing that the pH test accurately predicts the stabilization optimum while Hardy[154] and Marks[155] presented results showing that the pH test underestimates the stabilization optimum. From a volume change reduction point of view, several investigations[138,156-159] have shown that lime percentages in the range of 0.5 to 2.0 percent were sufficient to reduce swell to tolerable limits (i.e., <2 percent under 1 psi surcharge). With this in mind and the fact that the pH test accurately estimates the lime modification optimum, it is obvious that the pH test conservatively estimates the lime percentage necessary to minimize volume change.

Determination of lime content

248. The lime- pH test provides an estimate of the percent lime required to effectively reduce the plasticity of a given soil. A standard procedure for preconstruction determination of the lime content is not available, the only standard procedure that is available is for determining the lime content of stabilized soils.[160] The procedure, suggested by Eades and Grim[152] and summarized in Table 19, is much simplier and can be completed in less than 2 hr. Basically, the procedure involves mixing the dry soil and lime, adding water and thoroughly mixing the combination, and determining the pH of the mixture. The results can be plotted on arithmetic scales to help determine the lime percentage based on pH of 12.4. pH versus lime content plots for the 20 sampling sites are shown in Appendix C, Volume II. The other plot accompanying the pH versus lime content plot shows the effect lime has on the Atterberg limits. As lime content increases, the liquid limit decreases and the plastic limit increases with the net result being reduced plasticity index. The use of the Atterberg limit test in conjunction with the pH test is recommended since it provides verification of the lime percentage and a measure of the effect lime has on the given soil. Atterberg limits need not be run at all lime percentages used in the pH test. It is generally sufficient to run Atterberg limits at the "lime modification optimum" (LMO), i.e., lime percentage at pH = 12.4, and LMO plus and minus two percent.

249. The Atterberg limits, specifically the plasticity index, provide a method of establishing the effectiveness of lime on reducing swell as well as plasticity. In Reference 4, the plasticity index was shown

Table 19

Suggested pH Test Procedure For
Soil-Lime Mixtures (from Reference 152)

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.

to be one of the best indicators of potential swell for natural soils. From a lime treatment point of view, this is also true. If a 50 percent reduction in the plasticity index is not obtained at LMO then the practicality of using lime to minimize swell is essentially nonexistent. For low and marginal potential swell soils, i.e., plasticity index of 35 or less, the reduction should be to 15 or less. For the 20 sampling sites, the 50 percent reduction in plasticity index was met on all but four of the sites, and for those four sites the 15 or less requirement was met. In fact, the 15 or less requirement was met on 16 of the 20 sites with the remaining four sites having plasticity indexes at LMO's of 16, 16, 17, and 25. Therefore, based on the plasticity requirements, lime was shown to be an effective chemical for reducing potential volume change.

250. Quantification of the reduction in volume change caused by the addition of lime should be considered providing the "initial" and "final" conditions of the material are correctly simulated in the volume change test. For example, if the grade line is through a cut section and an odometer test is to be used, then the loading and material placement conditions should simulate those that will actually exist following construction. The more the test loading and material placement conditions vary from the actual conditions, the greater the discrepancy in estimating the influence of the lime on volume change. Comparison of odometer swell test results summarized in Tables 11 and 15 for undisturbed and lime treated specimens, respectively, reveals that the percent swell remained essentially the same or decreased for lime-treated specimens for 15 of the 20 sampling sites. For the remaining five sampling sites (1, 6, 13, 17, and 19) the percent swell significantly increased with the addition of lime. At site 1 the discrepancy can be explained by the large difference between initial water contents (i.e., w for undisturbed specimen = 40.3 while w for lime-treated specimen = 26.4). At the other four sites (6, 13, 17, and 19) there is no reasonable explanation for the increased swell. Laboratory procedure was ruled out when duplicate specimens yielded the same results. These four sites also showed an increase in swell for the lime-treated as compared with the remolded specimens, Table 13. No indication was available from any of the data to specifically explain the behavior of these four samples; however, it is possible that the variation in the general trend can be related to the soil or clay particle arrangements. The undisturbed specimens showed no apparent particle orientation,[3] while the remolded specimens (molded at AASHTO T-99 optimum water content and compaction energy) could have obtained some particle orientation (i.e., more face-to-face interaction) and thus a greater potential for swell. In the soil suction test on undisturbed and lime-treated specimens, the indicated trend was toward increased A and B parameters for the lime-treated specimens. For 16 of the 20 sites, A increased while B increased for 13 of the 20 sites, and A and B both increased for the same 13 sites. Most of the variations in behavior were related to the material molding conditions. To make valid comparisons, the moisture contents and, ideally, the densities should be the same.

Control Subgrade Moisture Conditions in the Expansive Soil

Maintaining in situ moisture conditions

251. Since volume change of expansive soils is the result of variations in subgrade moisture content, it is obvious that if the expansive soil could be isolated from any moisture changes, then volume change could be reduced or minimized. In this context, waterproofing membranes have been used successfully to minimize moisture variations and volume change. This section of the report describes in detail the experiences of several State Highway Agencies with the use of various types of waterproofing membranes.

Colorado's experience[118-121,123,126,161-174]

252. By far, the most experience with waterproofing membranes has been accumulated and published by the Colorado Department of Transportation. Primarily, the membrane applications in Colorado have been preconstruction oriented, consisting of sprayed asphalt with an occasional use of synthetic fabric products. In most cases, the construction was experimental in nature, so monitoring data were collected to compare the performance of the various treatment alternatives. Descriptions of the major experimental projects and their results or conclusions are summarized in the following paragraphs.

253. Clifton-Highline Canal project.[121,123,126,164-166] This experimental project involved construction of 19 test sections in cuts and fills in the Mancos Shale (I-70) of Western Colorado. Two of the test sections (Nos. 16 and 18) involved the use of sprayed asphalt membranes placed in 2-ft-deep subexcavated sections and backfilled with sand, A-3 (No. 16), and silt, A-4 (No. 18). Comparison of field data from 1965 through 1973 showed that the rate of loss of serviceability for the asphalt membrane sections (results combined for comparison with the other test section types) was less and that for the period 1971-1973, had the best serviceability rating of the various test section types. In addition, the moisture content at the top of subgrade increased less than any of the other sections. The swell (surface movement) was next to the least for all of the section types monitored with the maximum swell over the eight year period of 1.6 in.

254. Elk Springs project.[123,126,162,167,168] This project involved construction of 15 test features in which the major variable was pavement cross section. Specifically, the major variables were surface and base course thickness. Most sections had sprayed asphalt membranes extending from beneath the edge of the pavement, down the verge slope, and up the backslope. In one section, the sprayed asphalt was placed over a synthetic fabric material. In all sections except two, the

pavement layers were placed directly on the expansive subgrade. In those two sections, the upper 1 ft of the subgrade was encapsulated with sprayed asphalt membranes, above and below. In this project, the asphalt membranes did not perform as well as expected. It was concluded that even with full-depth asphalt pavements the asphalt membrane should extend over the entire roadway, i.e., backslope to backslope. As far as the variable thickness portion of the project was concerned, it was concluded that a minimum thickness of 9 in. (2-in. surface and 7-in. asphalt-treated base) was sufficient to provide reasonable serviceability for a major portion of the design life. The encapsulated layer sections did not perform as well as the sections without encapsulated layers.

255. Agate project.[123,169,170] This project consisted of three test features, one of which consisted of a sprayed asphalt membrane applied directly on the subgrade. The membrane was covered with 2 in. of sand, 4 in. of untreated base, and 9 in. of Portland Cement Concrete (PCC) pavement. The other test features were subgrade subexcavation to depths of 2 and 4 ft. The 4-ft subexcavated and membrane sections both performed very well; however, the extent of the swelling problem that was anticipated prior to construction did not develop. The main reason for the minimal expansion problem was the occurrence of several rainstorms just prior to applying the membrane and placing the pavement. The rainstorms helped to increase the moisture contents of the upper portion of the subgrade from 4 to 5 percent above AASHTO T-99 optimum. This alone was enough to significantly minimize subgrade volume change.

256. Whitewater project.[123,171,172] This project consisted of two well-instrumented sections of pavement. Both sections were identical with the exception that one section had a plastic membrane covering the subgrade. The purpose of the membrane was to study hydrogenesis, or the condensation of moisture from the air caused by temperature differentials. The membrane was used to collect the hydrogenesis water for the experimental study. The project pointed out the effectiveness of membranes in denying access of water to the subgrade by hydrogenesis.

257. Limon project.[173] This project involves monitoring of a section of I-70 near Limon, Colo. The monitoring section is in a cut section in the Pierre shale. The project was monitored for approximately two years in response to a request from the WES Research Team for the purpose of obtaining field data on various treatment alternatives. The pavement was constructed in 1973 with the entire subgrade covered with a sprayed asphalt membrane from median to backslope. Figures 211 through 239 (Appendix G, Volume II) show the field data monitoring locations, pavement cross section, and field data collected during the two-year monitoring period (May 1976 through March 1978). Although movement records are not available for the entire period since construction, available data show movement occurring with a general increase through the October 1977 monitoring cycle and a decrease for the subsequent cycle. The transverse hump appearing at sta 152+00 (approximate) in Figures 213-218 is likely the result of deformation from overburden removal (elastic release) since that station corresponds roughly to the maximum depth of cut (≈22 ft) relative to the original ground surface. With this

exception, the maximum movement of the pavement surface at the center line has been uniform and less than 0.1 ft. The surface deformation versus time plots, Figures 219-225, show the general increase with time through the October 1977 monitoring cycle followed by a decrease. This behavior matches the climate in the area for the monitoring period. Specifically, 1975 and 1976 were very dry years (actual drought) followed by 1977 with near normal precipitation. The general trend is now returning to an equilibrium, thus the reduction in deformation. From a serviceability standpoint, Figure 226, the overall performance of the roadway is very good. Benkelman Beam deflections, which reflect the strength of the subgrade, measured along the roadway and shown in Figures 227-229 are relatively uniform and constant up until the October 1977 monitoring cycle, after which they begin to decrease. This follows the indicated trend of establishment of a new equilibrium set by the surface deformation measurements. Benkelman Beam deflections versus time, Figures 230-236, also verify this trend. Subgrade moisture variations, Figures 237-239, were monitored by direct sampling at the edge of the shoulder at sta 149+00 and 155+00. Some variations occurred at the 1- and 3-ft levels likely as the result of successive borings that penetrated the membrane and were not repaired afterwards and water migrated beneath the surface. Even with these variations the moisture content profiles are of the same general shape, i.e., increasing with depth to approximately 4-ft, then relatively constant with depth, and show a moderate increase with time. In summary, the membrane has not totally eliminated moisture intrusion, but it has definitely minimized the influence of surface moisture conditions and has helped to maintain more uniform subgrade moisture conditions.

258. Morrison project.[174] This project also involves monitoring of a section of roadway in response to the WES Research Team request. The monitoring section is located in a cut section on State Highway 8, east of Morrison, Colo., which was relocated in 1975 in conjunction with the construction of Bear Creek Reservoir. The highway is four lanes with paved median and shoulders of full-depth asphalt pavement. The standard subgrade treatment for all cut sections was to scarify to a depth of approximately 2 ft, add water, and recompact with the top 6 in. compacted to 95 percent maximum dry density and optimum minus 2 percent minimum (AASHTO T-99). The test feature in the project involved the use of a synthetic fabric membrane extending from beneath the pavements edge down the verge slope and up the backslope. Because of additional borrow material required in another area, one side of the roadway in the monitoring section was cut so that the ditch was so far away from the pavement that the membrane extended approximately 20 ft down the verge slope. In this project, both the test feature and a control section, which were identical except for the membrane and adjacent, were monitored. The location of field data monitoring points, pavement cross section, and field data are shown in Figures 240-264, (Appendix H, Volume II). The synthetic fabric membrane has not been as effective as anticipated in reducing subgrade moisture variations and volume change. In fact, it is difficult to tell whether the membrane had any positive effect on the pavement performance in lieu of the subgrade treatment and the use of

full-depth asphalt pavement. The lack of positive influence is related more to construction than actual membrane performance since gusty winds that were blowing when the fabric membrane was laid down precluded a good, level placement of the fabric. In addition, the placement of the longitudinal joints overlap or shingle-style without any bonding may have had an effect, particularly in the left (south) ditch. Although the difference is small, the surface deformation along the roadway, Figures 242-250, does appear to be less for the test section than the standard section, particularly in the middle portion of the pavement (i.e., 8 and 20 ft, respectively, right and left of center line). In the vicinity of the south shoulder and ditch (i.e., 31 and 41 ft left), there is very little difference in the surface deformation plot. Deformation versus time plots indicate a similar pattern to those at the Limon site, namely, an increase in deformation through the April 1977 monitoring cycle and a decrease with following cycles. The same rationale concerning climate influence is applicable to this site as well as the Limon site. From a performance standpoint, using the Chloe present serviceability index (PSI), there is essentially no difference in serviceability between the test, Figure 255, and standard, Figure 256, sections. However, since the Benkelman Beam deflection data for the eastbound standard section lanes are much more erratic than any of the other data, it would indicate somewhat better performance from a strength point of view for the test section. These trends are also noticeable in the Benkelman Beam deflection versus longitudinal distance, Figure 257, and versus time, Figures 258-261, plots. As noted earlier, the subgrade moisture variations, Figures 262-264, were relatively erratic, thus indicating poor performance of the membrane. This is not as conclusive as it seems since the moisture content samples were taken at sta 76+00, which is between the test and standard sections. One very interesting fact about the maintenance patch at sta 76+00 (extending both directions) is that experience has shown that deformation often occur at the end of membrane sections. In other words, the occurrence of swells at transitions from membrane to nonmembrane sections points to the positive influence of waterproofing membranes.

Wyoming's experience[125,126,175,176]

259. In Wyoming, fewer experimental sections (one) were constructed to study the effect of membranes; however, more routine construction was monitored to quantify the performance of membranes. Descriptions of some of the more important projects and the results obtained during the studies are summarized in the following paragraphs.

260. Kaycee-South project.[175,176] This project was the only major experimental construction involving waterproofing membranes. The roadway is on I-25 south of Kaycee in gently rolling terrain in the Cody formation. The pavement was constructed in 1965 and consists of a 2-in. asphalt concrete surface course, an 8-in. hot mix asphalt-stabilized base, and a variable section (minimum of 3 ft) soft sandstone subbase. Three test features were included in the project: (a) a cut section in which a plastic membrane was placed approximately middepth of the sandstone

subbase and extending from beneath the pavements edge (not tied into or sealed by the pavement) to beneath the backslope, (b) a cut section with no special treatment, and (c) a shallow fill with no treatment. Nuclear depth moisture and density gages were used to monitor subgrade moisture through aluminum access tubes. The results show that in the nonmembrane sections significant increases in moisture content (i.e., 3-8 percent) occurred at the sandstone-clay interface below the membrane. In the membrane section, there has been a 2-8 percent increase in moisture content above the membrane while the increase was limited to approximately 1 percent below the membrane.

261. Upton to Newcastle project.[125,126] This project involved monitoring routine construction. The project is located on U. S. Highway 16 in Northeastern Wyoming in rolling terrain in a shale formation similar to the Pierre formation. The pavement was constructed in 1969 and consisted of a 7-in. plant-mixed, asphalt-stabilized base, which also served as a temporary wearing surface. In most of the cut sections, a sprayed asphalt membrane extending from the pavement edge down through the ditch and up the backslope was used to reduce moisture infiltration. Three of these cut sections, one with a membrane and two without, were selected for postconstruction moisture studies. The results of the moisture studies show that the upper portion of the subgrade (i.e., 0-2 ft) actually has a moisture content less than the compaction moisture content, so the membranes were quite successful in eliminating subgrade moisture variations.

262. Worland to Ten Sleep.[125,126] This project also involved postconstruction monitoring of routine construction. The project roadway is on U. S. Highway 16 in north central Wyoming in rolling terrain in the Cody formation. The pavement was constructed in 1968 and consists of a 2-in. asphalt concrete surface course, a 2-in. hot mix asphalt base, and a 4-in. crushed aggregate subbase. All cuts in potentially expansive soils have sprayed asphalt membranes over the subgrade from backslope to backslope. Four of these cuts were selected for postconstruction moisture studies. The results of the moisture studies show as of late 1970, no change in the subgrade moisture content from the construction value and the roadway virtually free of surface heaves.

Mississippi's experience[76-78]

263. In Mississippi, one experimental project involved the use of ponding (to be discussed later) and membranes to minimize the volume change of the highly expansive Yazoo clay. The roadway used in the study was part of State Highway 475 near the Jackson Airport. The entire roadway section involved in the study (ponded and membrane sections alike) was undercut to a depth of 3 ft. In the two membrane test areas (one in a cut and one in a fill), the sprayed asphalt membrane was placed on the undercut surface; then approximately 3 ft of fill was placed over the membrane. The top 6 in. of the fill material was lime treated (7 percent) and compacted. Moisture content and surface heave were monitored in both the

membrane-treated cut and fill sections. The results of the field monitoring showed that essentially no heave occurred in the asphalt membrane sections after nearly three years of data collection.

South Dakota's experience[177]

264. In South Dakota, the reported[177] use of membranes involved an experimental project on U. S. Highway 12. The pavement was constructed in 1965 and consisted of a 6-in. asphalt concrete pavement over a 6-in. layer of lime and asphalt (RC-1)-stabilized subgrade, which formed the waterproof cover. A plastic membrane was placed in a vertical ditch to a depth of 4 ft adjacent to the edge of the pavement to form a vertical cutoff. Moisture contents inside and outside the vertical membrane and surface roughness (rideability) were monitored during the period 1965 through 1971. The results indicated that there were no significant differences in the moisture contents of sections with moisture barriers and those without. Apparently, the plastic membrane cutoff was not placed deep enough, and the fractured nature of the shale permitted moisture to move underneath the wall. There were more moisture fluctuations in the areas with the moisture barrier, and the riding surface was better in areas without the moisture barrier. The moisture seemed to be higher and fluctuated more in the area close to the barrier itself, indicating thermal changes may be causing condensation near the plastic cutoff.

Montana's experience[178]

265. In Montana many of the alternatives discussed thus far have been used with varying degrees of success; however, published information on the performance of these alternatives is somewhat limited. In response to a request by the WES Research Team, the Montana Department of Highways agreed to expand a monitoring program on a membrane treated section of I-94 and provide the data to the WES team. The monitored section is located in the eastbound lanes of I-94 near the town of Hysham, Mont. The pavement consists of a 2-1/2-in. asphalt concrete surface course, a 4-in. plant-mixed asphalt base, approximately 1.5 ft of select base core materials, and 3-1.2-in. of sand to protect the sprayed asphalt membrane. The asphalt membrane covers the entire subgrade and extends from median to backslope. The location of field data monitoring points, pavement cross sections, and field data are shown in Figures 265-284 (Appendix I, Volume II). The monitoring project consisted of a test section, as previously described, and a control section that is identical with the exception of the sprayed asphalt membrane. The only data collected for a sufficient period for presentation was surface deformation, which at selected points were collected for over 10 years. For the term of this monitoring project (1976-1977), the surface deformation along the roadway shows some variable behavior with time for the control sections, Figures 267-269, while showing a general consolidation trend in the test section, Figures 270-272. The trend toward larger amounts of consolidation is limited to the western end of the monitoring section; otherwise, the deformation is relatively uniform and of low magnitude. The western end

of the test section (approximate sta 1100+00 to 1104+00) appears more susceptible to consolidation because of several factors: (a) seepage collected from the north cut moves under the westbound lanes through a culvert, which exists at sta 1104+00 and flows westward in the median, thus keeping the moisture content high in that area; (b) the profile sheets show a shallow sidehill fill in the area from sta 1100+00 to 1104+00; and (c) the eastbound lanes carried all of the I-94 traffic during the design and construction of the westbound lanes, which were under construction at the time of the monitoring program. This combination of factors has lead to the consolidation of the pavement subgrade (i.e., fill) in this area. Surface deformations for transverse sections for the control section, Figures 273-275 and the test section, Figures 276-279, reflect the same trends; however, comparison of the early (pre-1976 data) shows a more stable trend in the membrane section as compared with the nonmembrane section. Benkelman Beam deflection data are shown in Figures 279-284, and although quite limited, the data reinforces the concept of a more stable moisture regime beneath the membrane-treated section.

Arizona's experience[179-185]

266. In Arizona, sprayed asphalt membranes were first applied as a preconstruction treatment technique in 1972;[183,184] however, earlier experience with paved cut ditches indicated that the concept of waterproofing membranes could be used successfully to minimize pavement damage caused by volume change of expansive soils. By 1973, construction of a majority of Arizona's Interstate Highway System was nearly complete and the Arizona Department of Transportation (ADOT) was faced with the problem of pavement damaged by expansive soils. The concept of using sprayed asphalt-rubber membranes as a postconstruction treatment alternative to minimize reflection cracking was already in use in the city of Phoenix. The ADOT first applied the concept to damaged interstate highways in 1973 on the premise that in addition to providing a stress absorbing interlayer the asphalt-rubber membrane would provide a barrier against surface moisture infiltration. The concept was applied by placing a level-up course over the damaged pavement, spraying the asphalt-rubber membrane, then overlaying the roadway with a new surface course. The membrane was applied over the entire pavement, verge slopes, and up the backslopes a specified distance in cut sections. Overall experience with the use of asphalt-rubber membranes has been excellent. Several sections have been in place for over five years and are still performing very well. The ADOT have very diligently monitored the performance of the asphalt-rubber membrane sections. In response to a request from the WES Research Team the ADOT provided field monitoring data from three of these sections to the research team. The following paragraphs provide descriptions of the monitoring sections and summaries of the data collected.

267. I-40, Holbrook, Ariz. (Pinto to McCarroll project).[183,184,186] This project involved the overlay of a damaged section of I-40, 30 miles east of Holbrook, Ariz., near the Painted Desert National Park. The

remedial work involved level-up, drainage improvement, application of an asphalt-rubber membrane, and overlay (1.25 in.) with asphalt concrete. A section of roadway 11.8 miles long was treated in this manner. The adjacent (west) 11.6-mile stretch of pavement was treated in the same manner without the asphalt-rubber membrane to provide a basis for comparison of performance. Data were not provided to the WES Research Team on the nonmembrane section. Field data monitoring locations, pavement cross sections, and plots of the field data for the membrane test section are shown in Figures 127-159 (Appendix D, Volume II). Examination of the surface deformation along the roadway at various times, Figures 129-135, shows that the deformation is generally upward, very uniform, and less than 0.05 ft except in some isolated locations. One profile, 6 ft right of the center line of the monitoring points (note that the center line of pavement does not coincide with the center line of monitoring points, Figure 127), shown in Figure 133 indicates consolidation to be about 0.03 ft. This is due to the fact that the profile coincides with the right wheel path of the travel lane that has rutted under traffic loads slightly in comparison with the datum profile taken after the overlay was placed. Surface deformation for transverse sections, Figure 136, and deformation versus time plots at various stations, Figures 137-143, emphasize the same uniform, low-magnitude trend. Actual comparisons between data from the membrane section and the adjacent nonmembrane section were not made because the nonmembrane section data were not requested. However, Forstie et al.[183,184] have made those comparisons for a number of factors (i.e., surface deformation, subgrade moisture variations, and roughness index). Table 20 shows the results of a statistical analysis of the elevation differences between the October 1975 (after overlay construction) and April 1978 (2.5 years later) data sets. The results show that neither section is showing significant distress; however, the section without the membrane shows considerably larger average deformations. Maysmeter roughness data versus time, Figure 144, have been extrapolated linearly back to the original construction data and show that the general rate of increase of roughness is less following the overlay than the initial construction. This was expected since following the initial construction the subgrade moisture content was moving toward a higher equilibrium value whereas following the overlay only minimal redistribution of moisture occurred. Time and continued monitoring of the section will provide important data on quantifying the influence expansive soils have on pavement performance. Figures 145 and 146 show the maximum Dynaflect deflection versus location with time and versus time for various locations, respectively. Very little information concerning performance is obvious in these figures; however, it may be noted that since the maximum deformation appears to be decreasing with time, the moisture redistribution mentioned earlier may be indicated. Subgrade moisture contents were monitored using nuclear depth gages and aluminum access tubes placed across the roadway. Subgrade moisture variations with time for various depths are shown in Figures 147-154. Analyzing the figures as a group, it becomes obvious that the membrane is quite effective in minimizing subgrade moisture variations beneath the pavement. The access tubes near each edge of the membrane (tubes 1 and 7 outside the edge and tubes 2 and 6 inside the edge) show more variations

Table 20

Statistical Analysis of Elevation Differences Between October 1975 and April 1978 Monitoring Data Sets (after Reference 183)

Project/Section	Maximum Heave (ins)	*Average Difference (ins)	Statistical Variance of Elevation
I-40-5 (38) No Membrane			
Fill	2.2	-0.036	0.00107
Cut to Fill	2.0	0.414	0.00114
Cut	0.5	0.042	0.00027
I-40-5 (44) Asphalt Rubber Membrane			
Fill	0.6	0.026	0.00010
Cut to Fill	0.4	-0.026	0.00034
Cut	1.4	-0.020	0.00045

$$* \text{ Average} = \frac{\text{Heave } (+) + \text{Settlement } (-)}{\text{Number of Observations}}$$

NOTE: Values were determined from a survey grid of 217 survey points per location. Each grid represented a 300 foot length of highway, 38 feet wide. Elevations were taken at 18, 12, and 6 feet right and left of centerline as well as on the centerline. A survey across the road was taken at 10-foot increments.

throughout the profile than the remaining access tubes under the pavement. In comparing subgrade moisture variations beneath the membrane and nonmembrane sections, Forstie et al.[183-184] show significant differences in the statistical variance of the moisture content between the travel lanes (access tube 4) for the two sections. For example, at the 6-ft depth in the cut section with a membrane, the variance was 0.5 percent while the same value for the nonmembrane section was 1.8 percent or over three times larger. For the fill section with the membrane, the variance was 0.1 percent while the comparable nonmembrane value was 1.0 percent at the 6-ft level or 10 times larger. The larger moisture variations at the edge of the membrane are more obvious in Figure 155, which shows the subgrade moisture content for the transverse section through the access tubes. Moisture content profiles at various times for access tubes 1 through 4 are shown in Figures 156-159. The important point to note in these figures is the moisture content in the upper 4 ft of the subgrade. Moving from outside the membrane (tube 1) to beneath the travel lane (tube 4), the magnitude of the moisture content and the moisture variation are both smaller.

268. <u>State Highway 180, Holbrook, Ariz. (near Petrified Forest).</u>[187] This project involved new construction in which the waterproofing membrane concept was achieved by paving the shoulder (verge) slope and backslope (to a point 9 in. above the finished pavement) with 2 in. of asphalt concrete. The project is located 20 miles east of Holbrook, Ariz., on State Highway 180 near the entrance to the Petrified Forest National Park. The pavement was constructed in 1973 and consists of 9 in. of asphalt concrete placed directly on the subgrade. Field data monitoring point locations, pavement cross sections, and plots of field data are shown in Figures 160-178 (Appendix E, Volume II). Surface deformation along the roadway for various times, Figures 162-166, show that the deformations for the outside lines (20 ft left and right of center line) of monitoring points are much more erratic and of a higher magnitude than the interior lines, thus indicating the minimization of moisture variations beneath the pavement. In other words the deformation occurring at the edge of the distress lane would be more widespread over the travel lane if the membrane was not there to minimize surface infiltration from the verge slope. The same trend of edge movement is evident in Figure 167. Surface deformation versus time plots, Figures 168-175, also exhibit the more erratic behavior at the edge of pavement. Maysmeter roughness data with time, Figure 176, indicates that the rate of loss of ride quality is very low, and after five years of service, this section of the roadway is still providing a comfortable ride. Reports from ADOT personnel indicate that other sections of the roadway without paved ditches are not performing nearly as well as the monitoring section. Dynaflect data show a relatively uniform behavior along the roadway, Figure 177, but an erratic behavior with time at sta 857+00, Figure 178. This erratic trend occurs at all stations and is probably the result of the Dynaflect data being collected near the edge of the pavement and the fact that the data are not corrected to a standard temperature. This last factor would not be as significant as one would expect because of the range of temperature difference between the dates of the monitoring cycles.

269. U. S. Highway 89, Cameron, Ariz.[188] This project involved the reconstruction of a section of U. S. Highway 89 approximately 12 miles south of Cameron, Ariz. The pavement was originally constructed in 1960 and consists of a 3-in. asphalt concrete surface course, a 6-in. cement-treated base, and approximately 18 in. of select material. In 1976, drainage improvements were made along the roadway, a level-up course was applied, an asphalt-rubber membrane was placed, and the roadway section was overlaid with 2 in. of asphalt concrete. As part of another ADOT research project, this section of roadway (at two locations) and another section (same treatment without a membrane) were being monitored to determine the influence of climate and other environmental variables on subgrade moisture conditions. The ADOT provided the field monitoring data to the WES Research Team to help strengthen the data base on performance of membranes. Field data monitoring point locations, typical sections before and after the overlay, and field data plots for the two test and one control section are shown in Figures 179-210 (Appendix F, Volume II). Since surface deformation was not being routinely monitored in conjunction with the other ADOT project and was added sometime after the original WES request was agreed to, sufficient surface deformation data records are not available to provide an insight into the membrane performance. However, very detailed subgrade variation data are available, as well as Maysmeter roughness and Dynaflect data. Maysmeter roughness data versus time for the two test and one control section, Figure 182, show that the rate of roughness increase is, as expected, less after treatment than before. No explanation is available for the high roughness value for the control section following the overlay. The figure also shows that test section No. 1 was more seriously damaged than either of the other sections prior to the overlay. Dynaflect data, Figures 183-186, are not very conclusive; however, the continuing decrease of Dynaflect deflection with time in the two test sections supports the concept of subgrade moisture redistribution following membrane placement and the elimination surface moisture infiltration. Subgrade moisture data, Figures 187-210, support the previously discussed concept that the membranes minimize the moisture variations beneath the pavement by maintaining the infiltration wetting surface some distance away from the pavement itself. In other words, it increases the distance that water must move to cause volume change beneath the pavement as well as eliminate the ingression of water through the damaged pavement.

270. In both pre- and postconstruction applications the waterproofing concept, i.e., asphalt-rubber membranes or paved ditches, has proved successful in Arizona. The dual role of the asphalt-rubber membrane (stress absorbing interlayer and water barrier) makes it one of the more viable postconstruction alternatives currently available.

Increasing in situ moisture conditions

271. The objective of prewetting or ponding is to provide a continuous source of water so that a desiccated expansive soil can obtain a new and hopefully more stable equilibrium moisture condition prior to

placing the pavement. The higher initial moisture content and resulting preswell of the subgrade minimizes the likelihood of volume change after construction. Major unknowns in the use of ponding include determination of what depth the water must penetrate to effectively reduce volume change and how long the section must be ponded to achieve the depth of penetration. At present, experience and judgment are the only ways to cope with these unknowns. Other problems, lesser in extent but still important, involve scheduling of construction and special initial (dikes, sources of water, etc.) and follow-up (drainage, stabilization of soaked surface, etc.) procedures required to apply the ponding technique.

272. One of the earliest, more notable highway ponding projects was on U. S. Highway 81 north of Waco, Tex., over Wilson clay loam, which is developed from the Taylor marl.[189] In 1948, two areas were ponded; one site had 4-in.-diam holes drilled to a depth of 8 ft on 5-ft centers; at the second site, 4-in.-diam holes were drilled to a depth of 7.5 ft on 6-ft centers. The holes were backfilled with sand or gravel to minimize sloughing of the walls and filled with water daily for four months. Most of the water entered the upper 3.5 ft of soil, and the quantity added was so small compared with the volume of soil being wetted that some parts of the soil were still below the shrinkage limit two months after filling of the holes began. To expedite the swelling process, two areas were ponded for approximately three months. In the 40 days prior to ponding, there was no evidence of surface heave resulting from the daily filling of the holes with water. However, after three days of ponding, the surface rose 1 in. Several experiments to accelerate water movement from the holes were tried. In one experiment, pressures of 25-90 psi were applied in sealed holes. Two comparable experimental sections, one with 4-in.-diam holes 8 ft deep on 5-ft centers and one without any holes, were both ponded. All these experiments concluded that the holes were of little value in wetting the soil and that ponding was more effective. The apparent reason for this conclusion was that the blocky-structured natural clay afforded easy penetration of the water. Hence, it was recommended that ponding be completed prior to any grading that might alter the natural fissures.

273. In 1958, sections of I-35 north of Waco, again crossing the lower member of the Taylor marl,[103,105,190,191] were ponded for 22-41 days. Results of the project showed that the water did not penetrate more than 4 ft downward during a ponding period of 24 days. Nevertheless, after several days of ponding, the moisture contents at the 20-ft depth level increased. Results of this ponding show that after seven years of service, only 2 of the 15 ponded sections have become rough, while several unponded sections in the same area have heaved and been overlaid or replaced.[191]

274. In 1970, the Texas Department of Highways and Public Transportation (TDHPT) ponded a 27-ft-deep cut on U. S. Highway 90 west of San Antonio, Tex., in the Taylor formation.[192-195] An area encompassing the main lanes, median, and shoulders between sta 242+00 and 275+00 was divided up and ponded at various times for a maximum length of time of

45 days (minimum approximately 30 days). Following ponding, the ponded subgrade surface was lime stabilized and compacted to provide a working surface. The pavement is asphalt concrete with varying thicknesses of select material base course. Instrumentation to monitor the behavior of the ponded section was placed at various stations within the ponded section along the highway center line and approximately 120 ft left of center line. Additional instrumentation outside the ponded area was placed to provide comparison data. In 1975, the TDHPT agreed to continue monitoring the instrumentation for a period of approximately two years in response to a request from the WES Research Team. Pertinent information concerning the project, such as location and layout of the ponding site, highway cross section, location of instrumentation, and field data plots, is shown in Figures 285-321 (Appendix J, Volume II). Surface deformation versus time plots, Figures 289-296, substantiate the positive influence of ponding relative to the preswelling of the subgrade soils. In Figures 289 and 290, which respresent instrumentation located outside the ponded area, the major surface deformation does not begin until nearly 1.5 years after construction of the pavement. In the remaining deformation versus time plots for instrumentation within the ponded areas, it is obvious that a significant portion of the deformation occurred during or immediately following the ponding and prior to pavement construction, thus minimizing the actual damage to the pavement. Subgrade moisture content profiles are shown in Figures 297-304. It should be noted that moisture content data above the depth of approximately 2.5 to 3.0 ft are of little consequence since the pavement components (surface, base, subbase, and lime-treated subgrade) extend to that depth. Figures 297 and 298 (sta 173+00 and 242+00, respectively) show significant moisture variations to a depth of approximately 10 ft, while at the remaining stations, Figures 299 to 304, the moisture contents below approximately 5 ft exhibit minimal variations. This trend is also obvious in the subgrade moisture content versus time plots for various depths, Figures 305-312. The variations in the upper 3 ft shown in these figures are more related to the technique used to monitor subgrade moisture content (i.e., nuclear moisture gage) and associated problems with measuring moisture in the base and subbase materials. The serviceability index (SI) data, collected using a high-speed profilometer, are shown in Figures 313-317. Comparisons between specific sections are somewhat difficult because of the pavement configurations; however, average SI values for all ponded and nonponded sections, Figure 317, give a slight edge in performance to the ponded section for the eastbound lanes with the nonponded section performing slightly better in the westbound lanes. The differences are not large; therefore, significance of ponding in minimizing swell is difficult to discern without comparison of other factors such as surface deformation and subgrade moisture variations. Figures 318 and 319 show the maximum Dynaflect deflection at selected stations with time. These data are likewise inconclusive when used alone; however, the variations with time appear to be more uniform for the ponded section as compared with the nonponded sections. Figure 320 provides a very interesting record of surface cracking and maintenance work for the ponded and nonponded sections. It is obvious from this figure that much more cracking, plus subsequent maintenance work, has been done

in the nonponded areas. Figure 321 shows a comparison between calculated and observed vertical rise along the roadway. This provides further substantiation of the reliability of the PVR, which was evaluated in Part III of this report.

275. The Mississippi State Highway Department conducted an experimental cut-and-fill ponding section on State Highway 475 overlying the Yazoo clay.[76-78] The entire section was undercut 3 ft below finished subgrade elevation, a grid of 6-in.-diam sand drains 20 ft deep (laboratory tests showed swelling to 20 ft could be anticipated) on 5-ft centers was constructed, and the section was ponded for 140 days. After drainage, the section was brought to grade, and the upper 6 in. was stabilized with 7 percent lime. The lime stabilization extended from ditch to ditch to prevent future desiccation. Moisture content determination at various depths under the section proved to be inconclusive; however, it was concluded that the permeability of the remolded fill clay was so low that ponding was not effective in reducing the swell potential. Ponding was effective in the cut area. However, performance data indicate that the experimental sections have required no maintenance, while the companion control sections have experienced considerable distortion.

Miscellaneous Postconstruction Treatment Alternatives

276. Thus far, discussions have centered on preconstruction treatment alternatives with subsequent discussions on postconstruction application of those alternatives that can be applied to reduce further damage to existing highways. From an experience point of view, those postconstruction alternatives already discussed make up the majority of the available selection. However, one relatively "exotic" postconstruction treatment alternative that has been used with success in treating expansive soil subgrades is electro-osmosis. The other item which merits mentioning as a postconstruction option is remedial maintenance. Both of these categories will be discussed in more detail in the following paragraphs.

Electro-osmosis and base exchange

277. The concept of electro-osmosis 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. The most complete research study evaluating the use of electrochemical stabilization for minimizing volume change of expansive soil subgrades was conducted for the ADOT by O'Bannon.[195-206] Both laboratory and field studies were conducted on the highly expansive,

montmorillonitic, Chinle clay of eastern Arizona. In laboratory studies evaluating aluminum versus steel electrodes, calcium chloride, calcium chloride plus magnesium chloride, aluminum chloride, calcium chloride plus magnesium chloride plus aluminum chloride, potassium chloride, and sodium chloride solutions, it was found that potassium chloride and steel electrodes were consistently the most effective electrode-chemical combination for treatment of the Chinle. Further testing established that 4-5 percent by weight of commercial grade potassium chloride was the optimum percentage. In an attempt to increase the rate of penetration of the potassium chloride solution into the clay, several wetting agents were evaluated. These included Aerosol OT (sodium dioctyl sulfosuccinate), Aerosol AY (sodium diamyl solfosuccinate), C-61 (ethanolated alkylqucinidineamine complex), propanol alcohol, and Ultra Wet. The results of laboratory and field tests showed that C-61 and Aerosol AY were promising, with C-61 being the most effective and recommended for usage. From these considerations, field test sections using various electrode arrangements and methods of adding potassium chloride to the clay were evaluated. Site 1 used horizontal electrodes and solution wells (6-in.-diam, 18 in. deep in subgrade, on 8-ft centers); site 2 used horizontal electrodes and the base course was flooded with the chemical solution; and site 3 used horizontal electrodes and a central trench cut 18 in. deep in the subgrade and filled with potassium chloride solution. Evaluation of these sites showed that the solution wells provided the greatest uniformity, depth, and economy of treatment. Considering the entire project, the conclusions of O'Bannon and Mancini's[205] can be summarized as follows:

a. Electrochemical soil treatment can be successfully completed by ADOT maintenance personnel using the field procedure established in the study.

b. Based on qualitative scientific data, it is evident that electrochemical stabilization with potassium chloride solution is effective in altering the physical characteristics of montmorillonite and thereby reducing the expansive pressure of the soil. The scientific evidence indicates that the environment of the Chinle clay's particle interlayer space has been altered, thus affecting the engineering properties of the clay.

c. The lower the initial moisture content of the soil, the more effective the electrochemical treatment.

d. The higher the percentage of montmorillonite, the more effective the electrochemical treatment.

e. The most effective electrode configuration for field installation is one in which both the anode and cathode are placed in a horizontal position.

f. Potassium chloride is a water-soluble metallic salt that will effectively reduce the swelling of montmorillonite.

g. The recommended average voltage gradient (potential/distance between electrodes) for field projects is between 0.2 and 0.4 volt/cm. Below 0.2 volt/cm, the time requirement is prohibitive. Above 0.4 volt/cm, a noticeable heating of the soil occurs, indicating a high energy loss.

h. Because of the inherently random nature of the electrochemical process when used in a heterogeneous material like the Chinle clay, a very fine network of solution wells is required to inundate the soil with the stabilizing chemical. The network used on the ADOT project (6-in.-diam holes on 8-ft centers) was not fine enough. The fineness of the solution well network will be a function of the soil characteristics and must be determined by laboratory testing.

i. Electrochemical soil stabilization is most effective on a highly localized clay mass with a high swell potential.

The results show an effective but expensive treatment alternative, which can and should be considered for localized expansive soil problems such as highways in urban areas where right-of-way is limited and in roadways beneath overpasses where maximum clearance is important.

Remedial maintenance

278. Remedial maintenance refers to maintenance techniques such as isolated overlays (patchs) to level up distortions, removal and replacement of sections of the pavement, and portions of the subgrade to remove the distortions, or mud jacking 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 Highway Agency's maintenance program and will continue to do so. No set guidelines are available for application of remedial maintenance techniques; instead, they are generally left to the district maintenance engineer's discretion. 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: DESIGN, CONSTRUCTION, AND MAINTENANCE RECOMMENDATIONS

279. The pre- and postconstruction treatment alternatives discussed in the previous part of this report constitute the major options for modifying or altering (physically, mechanically or chemically) the characteristics of an expansive soil or controlling the subgrade moisture conditions (membrane 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 Highway Agency experience as gathered primarily through personal contact and secondarily through State Highway Agency publications.

Design and Construction Recommendations

Drainage

280. 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 the 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 32 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 Highway Agencies; however, in many states these details are overlooked especially when expansive soils are present. Figure 32a 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 Highway 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 32b shows a sloping grade in a cut. AASHTO guidelines[207] for maximum grade vary with type of highway, design speed, and type of topography. If the situation shown in Figure 32b occurs in rolling or hilly terrain and the highway grade (slope 1) is greater than 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 32b should be greater than slope 1 by approximately 0.1 to 0.3 percent depending on the relative 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 32c 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.

281. 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., goundwater 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. Steinberg[208,209] has reported success in reducing localized swells in a roadway adjacent to a sewage lagoon. Whether a subsurface drain is needed, how deep and long the drain must

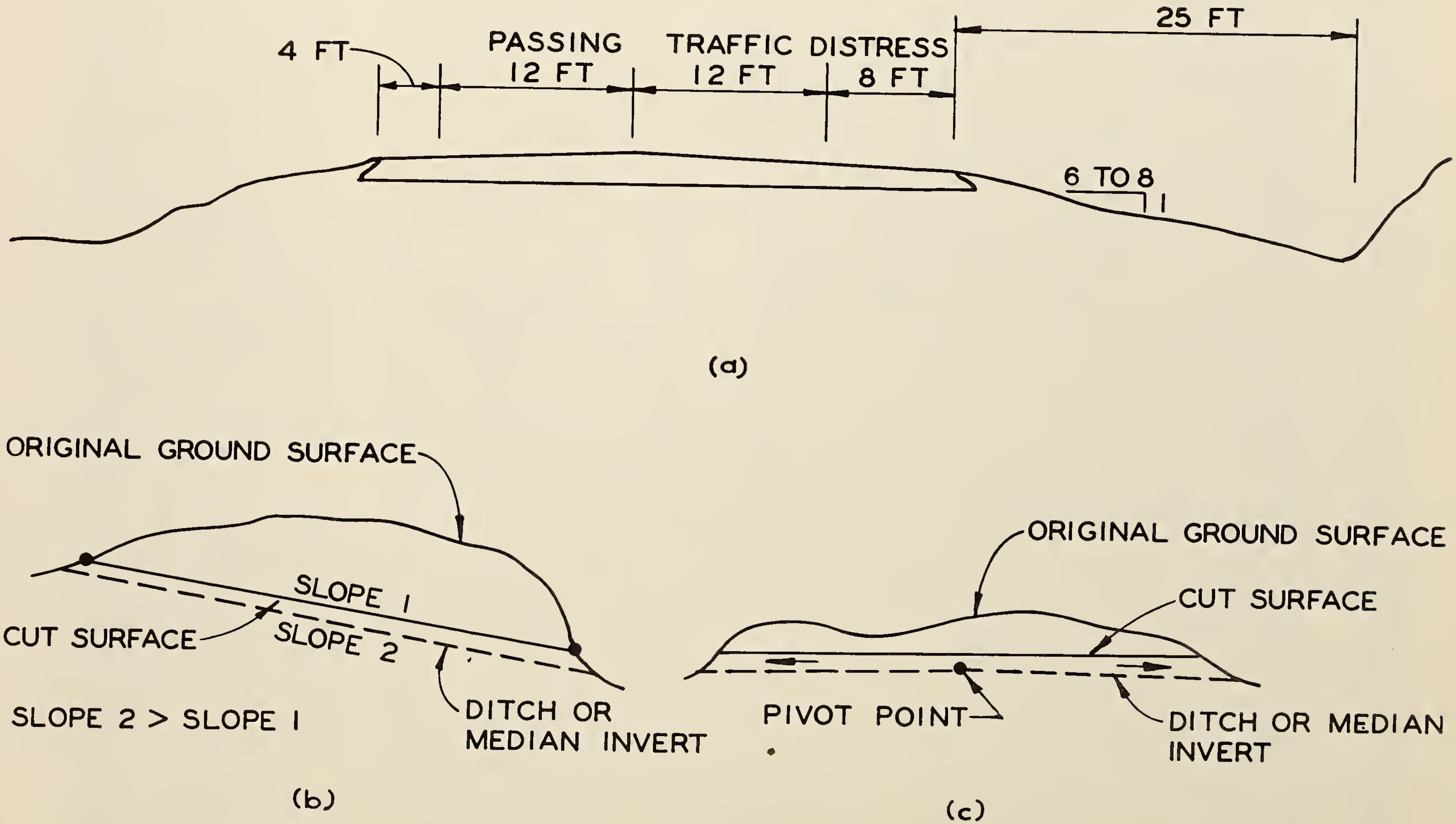


Figure 32. Diagram of surface drainage design and construction recommendations for (a) transverse and (b) and (c) longitudinal highway sections

be, and the drain component design are all factors dependent on specific situations. Guidance on these factors are provided in numerous publications[210-215] 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

282. 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.

283. 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 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 section; instead, elimination or reduction in depth of those cuts in high potential swell soils.

284. 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 a 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 "over-compacted" since high initial densities generally result in larger volume changes.

285. 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, (1) the pavement provides a "membrane" that fullfills the requirement of the waterproofing membrane concept and thus minimizes surface moisture infiltration; (2) if volume change occurs, the "flexible" nature of the pavement allows it to accomodate more distortion before significant pavement failure; and (3) 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-planers. 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, 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.

286. Another pavement cross-section feature that can be helpful in reducing moisture infiltration is the paved shoulder. AASHTO guidelines[207] suggest a 10-ft right shoulder (distress lane) and a 4-ft left (median) shoulder. As discussed in the previous part of this report, the further the infiltration wetting surface can be maintained from the travel and passing lanes, the less the likelihood of damage to the pavement. 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 additionsl likelihood of severe damage to the pavement should be seriously considered if the width is reduced to less than 4 ft.

287. Special construction for membrane. 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 minimized leaks through the membrane.

Maintenance Recommendations

Drainage

288. The comments in paragraphs 280 and 281 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 32 for surface drainage should likewise be applied as part of the reconstruction program. Guidelines[210-215] for subsurface drainage are equally applicable to postconstruction situations. The important recommendation within this category is maintenance of proper drainage. In cut sections, surface soil sloughs that impede surface drainage should be removed as soon as maintenance forces become aware of them. During wet seasons, 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 philosphy should be adopted for subsurface drainage systems: for example, maintenance of outlet markers, clearing of obstructions at the outlet, and providing surface drainage away from the outlet.

Routine maintenance and repair

289. 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 Highway 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.

290. 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 guardrail, traffic sign, or reflector posts are routinely moved or destroyed by traffic.

PART VIII: CONCLUSIONS

291. The purpose of this study was to evaluate and make recommendations concerning the major aspects of the expansive soil in highway subgrade problems: namely, describe (on the basis of physiographic areas) the occurrence and distribution of expansive soils, define and verify the roles of the microscale mechanisms which cause volume change, evaluate expedient methodology for identification and classification of potentially expansive soils, evaluate methodology for testing and prediction of anticipated volume change, evaluate and recommend appropriate treatment alternatives for new and existing highways, and recommend practical procedures for the design and construction of new pavements and maintenance of existing pavements. As a result of the research efforts within each of these major study tasks, a better understanding of the problems with expansive soils in highway subgrades has been developed. The results of the research study have been presented in four interim reports[1-4] and the preceding parts of this report. Based on these results and associated discussions, numerous conclusions have been drawn with regard to the purpose of the study. The conclusions are summarized in the following paragraphs.

292. The occurrence and distribution maps[1-2] discussed in Part II of this report, along with the discussions and tabular summary of geologic information, provide a simple yet relatively reliable method for establishing the location of potential expansive soil problem areas. The three phase categorization, although subjective, has been shown to be quite accurate in relation to the severity of the expansive soil problem (i.e., low, moderate, or high) and reported experience from various state and local agencies. The maps provide a useful qualitative tool for identifying potential problem areas during corridor planning stages and assistence in planning field exploration programs. The scale of the maps preclude their use for detailed analyses. Information complimentary to the occurrence and distribution maps include Soil Conservation Service county soil surveys, geologic hazard maps developed or being developed by several Federal and State agencies, and experience with highways or other structures within the general area of a proposed right-of-way.

293. Analysis of the results of the first phase laboratory testing program[3] indicate that for the materials and range of moisture contents tested, the microscale mechanisms which play the major role in causing volume change in expansive clays include clay particle attraction and the closely associated ion hydration. At higher moisture contents and higher cation concentration environments, the osmotic repulsion mechanism provides a secondary influence on volume change behavior. As a result of the research on microscale mechanisms, it was concluded that the total soil suction, which is comprised of a matrix and an osmotic component, best defines the influence of the mechanisms on volume change behavior. The matrix component represents the clay particle attraction and ion hydration mechanisms while the osmotic component represents the

osmotic repulsion mechanism. Confirmation of the secondary nature of role of the osmotic repulsion mechanism was made using the soil suction data when the testing results indicated that no significant osmotic component of soil suction was present in 18 of the 20 samples tested.

294. The occurrence and distribution maps, previously discussed, provide the first step in the decision process for dealing with expansive soils. They provide a qualitative assessment of potential problem areas. The next step in the process is evaluation of potential swell for samples taken from the proposed route. In this step, the various materials being traversed by the roadway are evaluated and assigned a semiquantitative rating. In other words, the identification/classification methodology is applied to assess the potential swell of specific materials along the selected route. Evaluation of 17 published identification/classification procedures showed that the most consistent indicators of potential swell were LL, PI, SL, and BLS. Analysis of the laboratory testing using simple linear regression correlation studies comparing measured swell and 31 independent variables showed that the most consistent indicators of potential swell were LL, PL, SI, BLS, and τ_{nat} . With the results of these two evaluations in mind, the categories of indicator properties that provide the best criteria for identifying and qualitatively (semiquantitatively) classifying potential swell are as follows:

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

295. The definition of potential swell that satisfies the largest portion of the field simulation requirements is:

> Potential swell is the equilibrium vertical volume change or deformation from an odometer-type test (i.e., total lateral confinement), expressed as a percent of original height, of an undisturbed specimen from its natural water content and density to a state of saturation under an applied load equivalent to the in situ overburden pressure.

This definition yields the volume change potential of a material when the initial specimen and stress conditions are identical to the in situ conditions. For predicting anticipated volume change, as will be discussed in a subsequent report, the aforementioned definition should be amended to reflect the anticipated final stress conditions such as applied load from the pavement or structure.

296. 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 phychrometers is a simple, inexpensive, accurate, and reliable testing procedure which could be easily implemented. From a practical point of view, the soil suction testing procedure was shown to be much less time-consuming than odometer procedures, and the measured data were applicable to a wider range of moisture conditions. The procedure suggested for estimating the depth of active zone based on the soil suction versus depth profile provides a reasonable estimate of the depth of active zone that is consistent with reported experience. The depth of active zone is useful in establishing the testing programs and provides limits for applying the prediction technique. A major requirement of any prediction technique involves an estimation 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 thermocouples 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 parameters A, B, τ_{mo}, and α defined.

c. Selection of the depth of active zone and final soil suction profile. Then the anticipated volume change should be calculated for each layer in the profile and summed to obtain the total surface movement.

297. In the absence of soil suction testing equipment, the odometer (OS) test procedure provides reasonable and generally conservative estimates of volume change based on comparisons with measured field behavior. The 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.

298. The PVR and VDM methods, empirical prediction techniques, provide estimates of anticipated volume change that are actually more accurate that the OS test. In the absence of soil suction data, the PVR or VDM methods are quicker and simpler than odometer test procedures providing classification data is available. The major reason for the improved accuracy of these techniques is that they require adequate

profile definition and consider the influence of overburden on the predicted heave.

299. The rate of heave is a function of the permeability and the amount of structural discontinuities present in a soil mass; therefore, the calculation of the rate of heave is solely dependent on the characterization of the soil's permeability. Permeability is a difficult property to determine, particularly for overconsolidated clays and shales. With all 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 expansive soils for predicting the magnitude of volume change.

300. Mechanical alteration of expansive soil subgrades, which includes ripping or scarifying and recompacting, and subexcavation and replacement, is a preconstruction treatment alternative that successfully reduces volume change by disturbing the natural structure of the soil, increasing the moisture content, and decreasing the dry density of expansive soils. Experience has shown that mechanical alteration, specifically subexcavation, performs best on materials for which the natural water content is below optimum and and the natural dry density is high (i.e., greater than 100 to 110 pcf). In this situation, the advantages stem from the increased moisture content and decreased density as well as remolding the structure. For materials in which numerous structural discontinuities (fissures, fractures, slickensides, etc.) are present, remolding the soil tends to reduce the avenues for water entry. In most cases where mechanical alteration has been used, the placement conditions are placement moisture content between optimum and optimum plus 5 percent and placement dry density between 92 and 95 percent of maximum (AASHTO-99). The required depth of subexcavation treatment is difficult to determine; however, the guidelines used by the Colorado Department of Transportation provide a practical solutions to the problem.

301. Lime continues to be the most practical and reliable chemical for treating expansive soils to reduce volume change. Quantities of lime as small as 2 percent by weight can significantly reduce volume change. The use of lime, as with any other treatment, should be selectively applied to those situations in which it performs best. The lime pH test and Atterberg limits adequately determine the LMO and verify the influence of lime on reducing plasticity, respectively. From a mix-in-place lime treatment point of view, if a 50 percent reduction in the PI 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 is equal to or less than 35, the reduction should be to PI of 15 or less.

302. Postconstruction lime application procedures, such as the drill-hole technique, have been used with extremely variable success and

considerable controversy. The LSPI technique has not been used as a postconstruction treatment for expansive soils primarily because of its controversial nature and problems with getting the material into the expansive subgrade soil. Before the LSPI is considered as a viable alternative, it must be determined whether the soil is reactive with lime and whether sufficient fractures and fissures are present to allow injection of the slurry.

303. Waterproofing membranes, particularly sprayed asphalt (catalitically blown, emulsified, or asphalt-rubber) membranes, have been used successfully in many western states. Asphalt membranes performed best when they were applied over the entire subgrade section, down the verge slopes, and up the backslope a specified distance (i.e., equivalent vertical distance of approximately 1.5 ft or 6-12 in. above the finished pavement grade elevation). Membranes have been applied on a number of different soil types and profiles as well as in various climatic zones. Although membranes have been used successfully in Mississippi (humid climate), 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 climates are dry-subhumid or drier (semiarid or arid). This stands to reason since these conditions generally describe a situation in which the major influence on subgrade moisture is from surface infiltration; 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, did not perform as well as the continuous sprayed membranes.

304. Postconstruction application of asphalt-rubber membranes have been very successful in Arizona. The membranes provide a stress absorbing interlayer and waterproofing membrane. The concept has performed best when applied to a damaged pavement following a level-up course to reduce surface distortions. Following placement of the membrane (backslope to backslope), the roadway section is overlayed with a new wearing course and sealed. Field monitoring data on three test sites in Arizona have shown measurable differences in the performance of rehabilitated roadways with and without the membranes.

305. Prewetting or ponding of an expansive soil to establish a higher and, hopefully, more stable moisture condition has been tried in two states (Texas and Mississippi). In both states, reported performance indicates that the ponded sections have better ride quality and fewer distortions than the nonponded sections. Ponding, as a preconstruction treatment alternative, appears to perform best in materials 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 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, provide the major obstacles to the wider use of the technique.

306. Electrochemical soil treatment was shown to be an effective but somewhat expensive postconstruction treatment alternative, which is most successful when applied on a highly localized soil mass with high swell potential. The electrode configuration and chemical compounds that provide the best performance, along with a suggested procedure for determining other variables associated with the technique, have been detailed by O'Bannon and Mancini.[205]

307. A number of practical design, construction, and maintenance recommendations for new and existing highways were discussed in Part VIII. Considering design and construction recommendations for new highways, the discussions evolved around two major categories, namely, drainage (surface and subsurface) and pavement cross-section features. For maintenance recommendations for existing highways, the discussions also centered on two categories, namely, drainage (surface and subsurface) and routine maintenance and repairs. Within each of these categories, practical recommendations that help to minimize subgrade moisture variations were discussed in detail, and criteria were presented in detail to assist in their implementation.

PART IX: RECOMMENDATIONS FOR FUTURE RESEARCH

308. Throughout the research study, the major aspects of the expansive soils in highway subgrade problem were carefully studied. In most cases, adequate solutions to the problems were obtained; however, in certain situations, insufficient time and data were available to fully investigate the details of a possible solution. Therefore, the following recommendations for future research and field experimentation should be considered:

- a. Verification of the soil suction equipment and procedures in several State Highway Agency laboratories under operational conditions and comparisons between predicted and measured volume change.

- b. Evaluation of the filter paper concept for measuring soil suction.

- c. Field and laboratory studies to establish correlations between soil suction with depth profiles and climate, specifically the seasonal variation of soil suction with depth and correlation of the variations with climatic classifications such as the Thornthwaite moisture index.

- d. Development and verification of more practical rate of heave prediction methods.

- e. Experimental demonstration of the use of lime slurry pressure injection as a postconstruction treatment alternative, including appropriate monitoring of treated and untreated section so that the effectiveness of the procedure can be fairly judged.

- f. Development and experimental demonstration of equipment that would more effectively distribute and mix lime to greater depths in overconsolidated clays and some softer shales.

- g. Development and experimental demonstration of equipment to install vertical membrane cutoffs (i.e., synthetic fabric or sprayed asphalt), and the monitoring of appropriate test and control sections.

- h. An operation-mode study of electrochemical soil treatment including appropriate test and control sections, as well as a major effort to reduce the initial cost of applying the technique.

- i. More research funds available to State Highway Agencies for monitoring the performance of suggested treatment alternatives used in experimental or routine construction.

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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.

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* 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.