SCHOOL OF CIVIL ENGINEERING

JOINT HIGHWAY RESEARCH PROJECT

JHRP-77-24

STRESS-DEFORMATION AND
STRENGTH CHARACTERISTICS OF
A COMPACTED SHALE

R. Aubrey Abeyesekera

PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

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Interim Report

STRESS-DEFORMATION AND STRENGTH CHARACTERISTICS
OF A COMPACTED SHALE

TO: J. F. McLaughlin, Director
Joint Highway Research Project

December 27, 1977

Project: C-36-51

FROM: H. L. Michael, Associate Director
Joint Highway Research Project

File: 6-6-12

The Interim Report titled "Stress-Deformation and Strength Characteristics of a Compacted Shale" is submitted for acceptance as partial fulfillment of the objectives of Phase II of the HPR Project C-36-51 titled "Design and Construction Guidelines for Shale Embankments". The research and report were performed by Mr. R. Aubrey Abeyesekera, Graduate Instructor in Research on our staff under the direction of Professors C. W. Lovell and L. E. Wood on our staff and W. J. Sisiliano, Soils Engineer of the ISHC.

The report includes the results of laboratory testing of a mechanically hard but non-durable shale from the New Providence Formation in Indiana. The compaction, consolidation and shearing characteristics of the shale are presented, as well as the testing procedures used. The report should be an aid to ISHC engineers in evaluating the stress-deformation characteristics and effective stress strength parameters of compacted shales.

The report is submitted as partial fulfillment of the overall objectives of the research project. Copies of the report will also be submitted to the ISHC and the FHWA for their review, comment and similar acceptance.

Respectfully submitted,

Harold L. Michael
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Interim Report
STRESS-DEFORMATION AND STRENGTH CHARACTERISTICS
OF A COMPACTED SHALE

by

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Project No.: C-36-51

File No.: 6-6-12

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project
Engineering Experiment Station
Purdue University

in cooperation with the

Indiana State Highway Commission

and the

U. S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Purdue University
West Lafayette, Indiana
December 27, 1977

1. Report No. JHRP-77-24	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle STRESS-DEFORMATION AND STRENGTH CHARACTERISTICS OF A COMPACTED SHALE		5. Report Date December 27, 1977	
		6. Performing Organization Code	
7. Author(s) R. Aubrey Abeyesekera		8. Performing Organization Report No. JHRP-77-24	
9. Performing Organization Name and Address Joint Highway Research Project Civil Engineering Building Purdue University West Lafayette, Indiana 47907		10. Work Unit No.	
		11. Contract or Grant No. HPR-1(14) Part II	
		13. Type of Report and Period Covered Interim Report	
12. Sponsoring Agency Name and Address Indiana State Highway Commission State Office Building 100 North Senate Avenue Indianapolis, Indiana 46204		14. Sponsoring Agency Code	
15. Supplementary Notes Prepared in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Research Study titled "Design and Construction Guidelines for Shale Embankments".			
16. Abstract When shales are encountered in road cuts, economic and environmental considerations usually dictate that they be used in adjoining embankments. However, unless special precautions are taken, the stability of a shale embankment can deteriorate with time on account of the non-durable nature of some shales in the presence of water. Various tests have been developed to classify shales. This report presents the results of a laboratory investigation to determine the influence of compaction and confining pressure on the stress-deformation and strength characteristics of a mechanically hard but non-durable shale from the New Providence Formation. Some construction guidelines are also recommended. Cylindrical specimens formed by kneading compaction were saturated under a low effective confining pressure, consolidated to the desired effective stress and sheared at a constant rate of strain. The compaction characteristics, the volume changes after saturation, the consolidation characteristics, and the undrained shearing response including pore water pressure changes are reported for triaxial tests. The initial gradation of crushed shale aggregate, the molding water content, the compaction pressure, and the pre-shear consolidation pressure were adopted as the test variables. The effective stress strength parameters were found to be essentially independent of the initial conditions when they were evaluated at maximum deviator stress, except for loose uncompacted aggregate. The volume change characteristics, the induced pore water pressures, and the consolidated undrained strength were found to be greatly dependent on the initial conditions.			
17. Key Words Shale, Argillaceous Rocks, Embankments, Compaction, Stress-Deformation, Pore Water Pressures, Strength, Effective Stresses, Strength Parameters, Triaxial Tests, Construction Guidelines		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 417	22. Price

ACKNOWLEDGEMENTS

The author wishes to express his deep appreciation to his major professors, Dr. C. W. Lovell and Dr. L. E. Wood, for their advice, guidance and encouragement throughout the course of the research, and for their patient reviewing of the manuscript.

Thanks are also due to Dr. W. R. Judd, Dr. G. A. Leonards, Dr. A. G. Altschaeffl and Dr. R. D. Holtz for their valuable comments and suggestions.

Thanks are extended to the Federal Highway Administration and the Indiana State Highway Commission for financial support of the research, and for valuable discussion at several Advisory Committee meetings. Special thanks are extended to Mr. W. J. Sisiliano, Soils Engineer, Division of Materials and Tests, ISHC, who not only served on the said committee but also made arrangements through his staff for the collection of shale samples, visits to construction sites, etc.

Willard DeGroff and Dr. John Scully helped in ordering and setting up the triaxial testing equipment. James Lambrechts helped in developing the computer program used for analysing the triaxial test data. Janet Lovell assisted in the X-ray diffraction analyses. Carol Latowski and Peter Massa helped in the preparation of shale aggregate. John Mundell, John Titrington and Peggy McFarren prepared the figures, and the draft was typed by Janice Bollinger and Edith Vanderwerp. Janice Bollinger typed the final manuscript. Thanks are extended to all of them.

Special thanks are due to Dr. G. A. Leonards who helped the author gain re-admission to Purdue University. The author's trip to the United States was made possible by a U. S. Fullbright Travel Grant and by the award of no-pay study leave by the Government of Sri-Lanka.

Finally, the love, prayers, patience and devotion of the author's wife, Manel, are gratefully acknowledged.

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LIST OF NOTATIONS

A	Skempton's pore pressure parameter
A_f	A parameter at failure
$A_{\max}$	maximum value of A parameter
A_o	area of cross-section of specimen as-compacted
B	Skempton's pore pressure parameter
c'	effective stress strength parameter
a	hyperbolic stress-strain parameter
b	hyperbolic stress-strain parameter
d	diameter of particle
D	diameter of top size of aggregate
E_i	initial tangent modulus
e	void ratio
e_o	as-compacted void ratio
e_c	void ratio after consolidation
e_s	void ratio after saturation
e_f	void ratio at failure
$(\Delta e)_c$	change in void ratio after consolidation
$(\Delta e)_s$	change in void ratio after saturation
$\overline{CIU}$	isotropically consolidated undrained triaxial shear test with pore pressure measurements
G_s	specific gravity of soil solids
H_o	height of specimen as-compacted
$\log$	common logarithm, $\log_{10}$
n	integer
P	percent finer than, by weight

LIST OF NOTATIONS (continued)

p'	mean effective normal stress = $(\sigma'_1 + \sigma'_3)/2$
p'_f	mean effective normal stress at failure
p'_{ult}	mean effective normal stress at end of shear
p_c	kneading compaction pressure
p_p	compaction prestress
q	shear stress = $(\sigma_1 - \sigma_3)/2$
q_f	shear stress at failure
R_f	failure ratio
S_r	degree of saturation expressed as a percentage
S'_r	degree of saturation expressed as a ratio
t	elapsed time during consolidation
u	pore water pressure
Δu	pore water pressure change
u_b	back pressure applied to pore water
V_o	volume of specimen as-compacted
V_a	volume of air in specimen as-compacted
$(\Delta V)_c$	change in volume of specimen during consolidation
$(\Delta V)_f$	change in volume of specimen after saturation and consolidation
$(\Delta V)_s$	change in volume of specimen after saturation
W	weight of aggregate
W_a	weight of molding water added to aggregate
W_p	weight of water in aggregate
W_o	weight of specimen as-compacted
W_f	weight of specimen after saturation and consolidation
$(\Delta W)_f$	change in weight of specimen after saturation and consolidation

LIST OF NOTATIONS (continued)

w	water content expressed as a percentage
w'	water content expressed as a ratio
w_i	initial moisture content of shale aggregate
w_c	as-compacted moisture content of specimen
w_m	molding water content of specimen
α	intercept, q versus p' ($= c' \cos \phi'$)
β	slope, q versus p' ($= \sin \phi'$)
ϵ_a	axial compressive strain
γ_a	weight of aggregate per unit volume of specimen
γ_b	bulk density of specimen
γ_d	dry density of specimen
γ_w	unit weight of water
σ'	effective normal stress (σ = total normal stress)
σ'_{oct}	effective octahedral normal stress $= (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$
σ'_1	effective major principal stress
σ'_2	effective intermediate principal stress ($= \sigma'_3$)
σ'_3	effective minor principal stress
σ'_{ff}	effective normal stress on failure plane at failure
σ'_c	effective consolidation pressure prior to shearing (isotropic)
σ'_s	effective confining pressure after saturation phase (isotropic)
τ	shear stress
τ_{ff}	shear stress on failure plane at failure
τ_{oct}	octahedral shear stress $= \sqrt{2}(\sigma'_1 - \sigma'_3)/3$, for $\sigma'_2 = \sigma'_3$
ϕ'	effective stress strength parameter (effective angle of friction)

Special Notations

Notations such as (2-25-200) are used to indicate the compaction history of the test specimens. The first number denotes the initial gradation used, the second number denotes the amount of molding water added in gm per 500 gm of shale aggregate, and the third number denotes the kneading compaction pressure, in psi.

Notations such as (2-25-200-40) are used to indicate the compaction and consolidation history of the test specimen. The fourth member denotes the pre-shear consolidation pressure, in psi.

Miscellaneous Notations

In addition to the above list of notations, other notations have been used mainly in the Review of Literature. Such notations are defined wherever they appear in the text.

HIGHLIGHT SUMMARY

This thesis reports the results of a laboratory investigation to determine the influence of compaction on the stress-deformation and strength characteristics of a mechanically hard but non-durable shale from the New Providence formation.

Kneading compacted specimens were saturated under a low effective confining pressure, consolidated to the desired effective stress, and sheared undrained at a constant rate of strain. The volume changes during isotropic saturation and consolidation, and the axial load and the pore pressure changes during axial compression were observed.

Six gradations were used, with a top size of 3/4 in. The molding water content ranged from 5% to 8%, and the applied compaction pressures were 50, 100 and 200 psi. The pre-shear consolidation pressures were 0, 1, 5, 10, 20 and 40 psi.

The principal conclusions for the shale studied follow.

1. The shale slakes partially when mixed with water and undergoes some mechanical breakdown when compacted.
2. The influence of molding water on the compacted dry density is not as well defined as in fine grained cohesive soils.

3. For the experimental range of molding water contents, the mean dry density increased linearly with the logarithm of the compaction pressure.
4. The as-compacted dry density ranged from 95 pcf to 132 pcf, the void ratio ranged from 0.829 to 0.296, and the degree of saturation ranged from 5.5% to 95%.
5. For a given initial condition, there is a critical effective confining pressure below which the specimen swells, and above which it compresses, when saturated.
6. The saturated consolidated undrained strength increases with increasing compaction pressure, consolidation pressure, and molding water content.
7. The stress-strain curves of the loose specimens are of the strain softening type, whereas those of the denser specimens are of strain hardening type. The latter type can be represented very well by an hyperbolic stress-strain relationship.
8. The main features of the pore pressure response are summarized below:
 - (a) The peak positive excess pore water pressure increases linearly with increasing consolidation.
 - (b) The peak pore pressure strain increases with increasing consolidation pressure and decreasing compaction pressure.
 - (c) The incremental change in pore water pressure beyond a certain strain is more or less independent of the consolidation pressure, for specimens having the same compaction history.

- (d) As the ratio of the compaction pressure to the consolidation pressure increases, the pore water pressures at failure change from positive to negative values.
9. The pore pressure parameter (A) varies with the axial strain, and its value at peak deviator stress (A_f) ranged from 2.2 to -0.4, depending on the compaction pressure, the molding water content and the consolidation pressure.
 10. Effective stress paths similar to those of loose sands and sensitive clays, highly overconsolidated clays and dense sands, and normally consolidated soils were observed.
 11. The peak principal effective stress ratio is reached before the peak deviator stress for dense specimens, and after it for loose specimens.
 12. When the peak deviator stress failure criterion was used for the compacted and saturated specimens, $c' = 1$ to 2 psi and $\phi' = 28$ to 30 degrees, whereas for the loose specimens, $c' = 0$ psi and $\phi' = 25$ degrees.

CHAPTER 1

INTRODUCTION

In December 1971 and January 1972, a major slope failure occurred within an embankment on I-74 near St. Leon in Dearborn County, Indiana (Wood, Lovell and Deo, 1973). The embankment was constructed in 1961 and the fill material was obtained from intermittent beds of limestone and shale, and weathered soil principally from the shale. The embankment was built as a rockfill in lifts up to 3 foot thick. Large settlements preceded the actual slide. The harder limestone and the weaker shale were present in about equal amounts and their distribution within the embankment was random.

Shale embankments located elsewhere have also settled excessively and some have experienced slope failures (Waterways Experiment Station, 1976). In the state of Kentucky several small embankments along I-75 south of Covington experienced slope failures about six to eight years after construction. These embankments were also constructed using fill from interbedded shale and limestone in 1959-1960. A series of five embankments on U.S. 460 in West Virginia have settled 2 to 3 feet since construction in 1970. A bridge approach embankment in Ohio on I-70, compacted in lifts of 8 inch up to a height of 55 feet, settled only 3 inches. However, lateral movements of the shale foundation have been enough to cause distress to the abutments.

The fact that all these embankments failed many years after the end of construction suggests that either the mass properties deteriorated with time or the effective normal pressures which contributed to the shearing resistance of the fill material decreased with time, for example, due to the build up of pore water pressures. Both factors could, however, have taken place concurrently to varying degrees.

It has been known for a long time that the interlocking in an granular mass contributes significantly to its shearing resistance (Taylor, 1948). It has also been known that the degree of interlocking progressively decreases when the mass is subject to shear stresses and strains (Taylor, 1948). The reverse process could, however, also take place if the mass is initially in a loose state. Apart from shear stresses and shear strains, interlocking could also be destroyed if the material itself degrades in the service environment. The degree of degradation in turn would tend to increase with increasing normal and shear stresses. Thus an initially well interlocked shale embankment, could with time, progressively deteriorate into one with little or no interlocking on the macro-scale. The maximum shearing resistance of such an embankment would then be determined mainly by the cohesion and frictional characteristics of the minerals present in the shale. The mineralogical composition of the shale is an important factor not only in regard to its deterioration, but also to its ultimate shearing resistance.

Some mechanisms which seem to explain the shearing resistance of an irregular surface are examined in detail in Appendix G. The degree of interlocking is shown to decrease with increasing normal stress and

shear strain and decreasing material strength.

When a saturated soil is unloaded, there is a tendency for the external stresses to be transferred to the pore water as an isotropic negative stress, commonly referred to as soil suction, pore water tension, etc. (Hirschfeld, 1963). This negative pore water pressure dissipates very fast in highly permeable soils and very slowly in rocks of low permeability such as shale. When beds of shale are unloaded, the induced negative pore water pressures are retained for many years, and the magnitude of the negative pore water pressure is proportional to the external stress release.

Very large negative pore water pressures in samples of shale have been measured and they tend to prevent the discrete particles in shale from disintegrating (Chenevert, 1970a). These negative pore water pressures are reduced when the shale comes into contact with and absorbs free water. The discrete particles could separate from the bulk materials if the cementation and other bonds are not strong enough to resist the gravitational forces acting on the particles. This and other mechanisms of strength reduction in shales have been proposed and these are examined in more detail in Chapter 2.

When shales containing expansive clay minerals are used in embankments, the associated problems are of a different type. Here, an initially dense embankment tends to swell in the presence of moisture available either in the atmosphere or in the ground water. The amount of swelling that takes place increases with decreasing confining pressure and increasing compaction pressure (Leonards, 1952). The initial structure of the compacted mass and the moisture

content, as well as the method of compaction, also have an influence on the amount of swell.

From the foregoing remarks it would be seen that inadequate compaction could lead to settlements, whereas overcompaction could lead to swelling, depending on the consolidation characteristics of the compacted shale.

Whereas clays and sands can be tested in the laboratory in their original size and shape, this is not the case with shale. In the field, shale appears in beds which are often separated by beds of other sedimentary rocks such as limestone and sandstone. Large chunks of shale, along with finer sizes which may be as small as the finest colloidal size particles, are present in a shale embankment. It is this wide range of material sizes which distinguishes a shale embankment from soil and rockfill embankments.

In the technique used in the study of rockfill materials, a laboratory grain size distribution is chosen to parallel the field gradation (Marachi, Chang and Seed, 1972). The maximum size that can be used in the laboratory is at least an order of magnitude less than the field maximum size, when conventional triaxial cells of 4 to 6 inches are used. Large triaxial cells, some as large as 3 feet in diameter and 7 feet in height, are currently being used for testing rockfill materials to study the variation of frictional and dilatancy characteristics with maximum size of the material (Reese, 1970; Marsal, 1973).

Since the contact forces, for a given average pressure, are dependent on the size of the particles, some consideration should be

given to the question of relating the relative levels of laboratory and field confining and shear stresses, when the particle size used in the laboratory differs from that used in the field. Similitude laws have been applied in the study of the shear and dilatancy characteristics of jointed rock by Einstein et al. (1969). Their relationships are presented in Appendix G, and it will be seen that it is difficult to satisfy all the similitude requirements.

Very little work appears to have been done on the engineering characteristics of compacted shale pieces. On the other hand, if work has in fact been done, the results have not been published in adequate detail in the technical literature familiar to geotechnical engineers. This is a very unfortunate situation, not only because shale is a material which must be used in large quantities when encountered in road cuts, but also because its behavior is very complicated and often unpredictable.

Steps have recently been taken to rectify this situation. As a first step, various classification systems have been proposed to identify mechanically hard but non-durable shales, which present most of the field problems. After shales are identified and classified, those that are durable can be placed as a rockfill, whereas those that are non-durable can be placed as a soil fill.

Most of these classification systems are based on tests performed on shale aggregate with hardly any load acting on the specimen. In the field the shale is under load and this load could affect the slaking and degradation characteristics of the shale in the presence of moisture. A classification system based on strength reduction due

to soaking has been proposed for distinguishing argillaceous soils and rocks by Morgenstern and Eigenbrod (1974).

The U. S. Army Corps of Engineers has recently completed Phases I and II of a three phase project on compacted shale embankments. A Survey of Problem Areas and Current Practices (Phase I) is covered in Shamburger, Patrick and Lutten (1975), and the Evaluation and Remedial Treatment of Shale Embankments (Phase II) is covered in Bragg and Zeigler (1975). Phase III of this study which is related to the "Development of Methodology for Design and Construction of Compacted Shale Embankments" is expected to be completed in June 1978. During the current Phase III work, 5 inch diameter undisturbed field sampling and pressuremeter tests, together with K_o consolidated triaxial tests, have been completed (Waterways Experiment Station, 1976).

The embankment problems along I-74 in Indiana motivated research and development activities both directly by the Indiana State Highway Commission and through the Joint Highway Research Project at Purdue University, as reported by Wood, Sisiliano and Lovell (1976) at the Seventh Ohio River Valley Soils Seminar. These efforts to improve the state-of-the-art of compacted shale fall into four principal categories; (a) storage and retrieval of existing data on Indiana shales, (b) shale classification, (c) study of compaction and degradation characteristics of shales, and (d) definition of shear strength parameters for compacted shales.

The results of studies on categories (b) and (c) have been published by the Joint Highway Research Project, Purdue University, in Deo (1972), Chapman (1975), and Bailey (1976). Procedures and test results relating to category (d) are reported in this study.

Consolidated undrained triaxial compression tests with pore pressure measurements were used throughout the study to evaluate the effective stress strength parameters of soaked specimens. These parameters are presumed to control the long-term stability of an embankment. The initial gradation of crushed shale aggregate, the molding water content, the compaction pressure, and the pre-shear consolidation pressure were adopted as the test variables.

The effective stress strength parameters were found to be essentially independent of the initial conditions when they were evaluated at maximum deviator stress, except for very loose uncompacted aggregate. The volume changes during saturation and consolidation, the induced pore water pressures during shearing, the consolidated undrained strength, and the stress-strain response of soaked samples were found to be greatly dependent on the initial conditions.

CHAPTER 2

LITERATURE REVIEW

The Literature Review is presented in two parts. The first part, which deals exclusively with shale, is presented in this chapter under the headings (1) Occurrence of Shale, (2) Formation and Composition of Shale, (3) Weathering of Shale, (4) Engineering Classification of Shale, and (5) Compacted Shale Strength.

The second part of the Literature Review is presented in Appendix G, and the studies referred to therein were mainly made on sands, silts and clays. This literature is of general interest and is presented under the headings (1) Compaction and Prestress, (2) Compressibility, Collapse and Swelling, (3) Shear Strength, (4) Stress-Strain Models, (5) Pore Water Pressure Changes, (6) Progressive Failure, (7) Back Pressure Saturation, (8) Analysis of Triaxial Test Data, and (9) Strain Rate Effects.

2.1 Occurrence of Shale

Shale is the most common sedimentary rock on the earth and it comprises about 50 percent of the exposed bedrock. In the state of Indiana the extent of exposed shale is much less because it is overlain by glacial deposits. It is, however, frequently encountered in the southern part of the state of Indiana in road cuts. The occurrence and

extent of shale in Indiana is recorded in Harrison and Murray (1964) and in Shaver et al. (1970).

2.2 Formation and Composition of Shale

Shale is a fine grained, argillaceous, highly compacted, fissile sedimentary rock which formed in the depths of oceans and lakes. The processes of compaction that convert clays, silts and fine-textured materials into shales are well documented in Rieke and Chilingarian (1974) and Attewell and Farmer (1976). The processes of diagenesis in argillaceous sediments are discussed in Muller (1967). They include: (1) formation of new minerals, (2) redistribution and recrystallization of substances in sediments, and (3) lithification. These changes are enhanced by time, depth of burial and high temperature. Tectonic activity (usually accompanied by elevated temperatures) may also play a role in the process of diagenesis, which is associated with strength increase.

The most important allogenic components of argillaceous sediments include (Rieke and Chilingarian, 1974): (1) various clay minerals including gibbsite, (2) quartz, (3) feldspars, (4) carbonates, (5) amorphous silica and alumina, (6) pyroclastic material, and (7) organic matter. Biogenic materials which form in the basin itself include carbonates, amorphous silica and organic matter (Muller, 1967).

The most common clay minerals are illite, montmorillonite, chlorite, kaolinite and to a lesser extent halloysite. The frequency of these minerals and their relative proportions vary over the earth's surface.

The Indiana shales were formed between the Ordovician and Pennsylvanian periods which covered a time span of approximately 210 million years (Indiana Department of Conservation, Geological Survey, Circular No. 5). The New Providence shale used for shear testing in this study is described in detail in Chapter 4. This formation was formed about 261 million years ago, and it was subsequently overlain by other sedimentary rocks which have since been eroded away. The formation is thus heavily overconsolidated and the compaction pressures during its diagenesis are likely to have been at least 2600 psi (author's estimate based on thickness of overlying strata eroded away).

Shale is composed of discrete particles (clastic) which have been brought together by pressure. A soil-like shale is one which lacks intergranular cementing, and a rock-like shale, in contrast, is cemented or recrystallized (Underwood, 1967). A soil-like shale derives its shearing resistance from the interatomic or molecular bonds and from negative pore water pressures. The number of bonds per unit area is proportional to the normal pressure, and the strength of the bonds is dependent on the mineralogical composition of the grains (Mitchell, 1976). In a normally consolidated soil-like shale the number of bonds is directly proportional to the effective consolidation pressure. Overconsolidation leads to more bonds than for a normally consolidated shale at the same effective consolidation pressure. A rock-like shale derives its strength from the strength of the cementing materials and from the intermolecular bonds.

The magnitude of the consolidation pressure induced in a shale deposit is dependent on a number of factors. These include the depth

of burial, the rate of sedimentation, the permeability of the underlying and overlying sediments and the duration of loading (Rieke and Chilingarian, 1974). During the initial stages of sedimentation the fluid pressure is equal to the hydrostatic head. As compaction proceeds, the escape of interstitial water is inhibited by the decrease in permeability, and fluid pressures greater than the hydrostatic pressure may result. When this happens, the increase in the effective consolidation pressure is less than the increase in the total stress by the magnitude of the excess fluid pressure.

The inter-relationship among pressure, porosity reduction and fluid release in shale formations is influenced by several factors. According to Mead (1968) they are, (1) particle size, (2) type of clay minerals, (3) adsorbed cations, (4) interstitial electrolyte solution, (5) acidity, and (6) temperature.

In addition to the above, there are many other secondary factors which modify the pressure - pore volume relationship (Rieke and Chilingarian, 1974). They are, (1) deformation and granulation of mineral particles, (2) cementation, (3) solution, (4) recrystallization, (5) squeezing together of the grains, (6) mechanical rearrangement, (7) expulsion of adsorbed water, (8) chemical change and cementation between grains, (9) occurrence of carbonates and sands, (10) abnormally overpressured zones, (11) crushing of grains, (12) mineralogical transformation, (13) dehydration, and (14) NaCl filtration.

In Figure 11 of Muller (1967) an attempt has been made to correlate the main mechanical and chemical-mineralogical changes

occurring during diagenesis with the depth of burial, increase in temperature and pressure, and the duration of burial. The changes in initial composition of argillaceous sediments are distinguished from changes in composition due to the formation of new minerals. This figure is so instructive that it is reproduced here as Figure 1.

The transformation of montmorillonite to illite takes place during the deep-burial stage of diagenesis. This process is accompanied by the desorption of large amounts of bound water which then becomes free water. Some of the new minerals are formed at shallow depths, whereas others are formed under conditions of extremely high pressure and elevated temperatures. New minerals formed under the latter conditions may become unstable at lower pressures.

When the confining stresses acting on a formation of shale are released, for example, by erosion of the overlying deposits, there is a tendency for the formation to swell. This is inhibited by the molecular and cementation bonds previously established, and also by the pore water tension induced in the shale during unloading. The magnitude of the pore water tension would depend on the initial pore pressure, the depth of material eroded, and the permeability of the shale. Some elastic rebound may also take place during unloading.

2.3 Weathering of Shale

As progressive failure of a shale embankment is very often caused by progressive failure of the shale chunks, a review of the causes for the latter seems appropriate.

The progressive deterioration of shale usually takes place in the presence of moisture. The mechanisms most often used to explain the

FIGURE 1 RELATIONSHIP OF MECHANICAL AND MINERALOGICAL-CHEMICAL CHANGES OCCURRING IN ARGILLACEOUS SEDIMENTS DURING DIAGENESIS WITH DEPTH OF BURIAL, PRESSURE, TEMPERATURE, AND DURATION OF BURIAL. (AFTER MULLAR (1967); IN LARSEN & CHILINGAR, 1967).

reduction of rock strength in "wet" conditions have been reviewed by Van Eeckhout (1976). They are, (1) fracture energy reduction, (2) capillary tension decrease, (3) pore pressure increase, (4) frictional reduction, and (5) chemical and corrosive deterioration. The paper, however, does not deal with the time dependency of these phenomena. How long it will take a chunk of shale to deteriorate in a "wet" environment while under load is not easy to predict.

Murayama (1966) has presented experimental data to support other mechanisms, namely, (6) shear stresses generated due to anisotropic expansion parallel and perpendicular to bedding planes, (7) unequal expansion of the minerals, and (8) non-uniform swelling due to unequal distribution of sorbed water.

Reidenouer, Geiger and Howe (1974, 1976) have evaluated the detrimental effects of two expansive clay minerals (illite and montmorillonite) plus sericite on the suitability of shale. A Clay Factor (CF) is defined as $CF = \text{montmorillonite factor} + \text{illite factor} + \text{sericite factor}$, where the factors are weighted according to the percentage of the two clays and sericite present. They also define a Suitability Factor (SF) as follows

$$SF = (\text{gyratory factor})(\text{Washington degradation factor}) \\ + \text{wet/dry factor}$$

where the factors were determined from factor analysis of gyratory compaction, Washington degradation and wet/dry tests. The dependency of these factors on time effects is not dealt with.

The weathering of shales is time dependent and usually takes place in the presence of water (Chenevert, 1970b). Alternate cycles of wetting and drying enhance the weathering process (Deo, 1972). The presence of expansive clay minerals in shales is potentially dangerous because they tend to expand in the presence of water (Mitchell, 1973). Thus, poorly cemented argillaceous shales composed mainly of expansive clay minerals will, if adequate moisture is available, be capable of swelling. The actual amount of swell that takes place will be dependent to a great extent on the confining pressure (Kassiff, Baker and Ovadia, 1973).

Badger, Cummings and Whitmore (1956) investigated the disintegration of shales in water and found that it occurs by two processes. Characteristic of all shales is the softening and dispersion of the colloidal matter which binds the constituent particles of the dry shale together. Mechanically weak shales disintegrate through the air-pressure developed in the pores after water has been drawn into them by capillary action. The amount of breakdown is largely governed by weaknesses in the structure of the dry shale and not by the amount of colloid present. The degree of dispersion depends on the type of exchangeable cations attached to the colloids and the properties of the liquid causing disintegration, i.e., its ionic characteristics and dielectric properties. The ease of ionic dispersion of clay minerals follows the order: sodium clay, potassium clay, magnesium clay, and barium clay. The higher the dielectric constant of the liquid the greater is the degree of dissociation of ions from the shale colloid into the liquid, and consequently the degree of dispersion and

disintegration is increased.

The clay minerals in the shale may also undergo weathering. The result is usually an increase in the plasticity and a decrease in the shearing resistance of the shales (Shamburger, Patrick and Lutten, 1975).

Cement weathering depends on the type of cement present in the shale and on the nature of the weathering environment which is characterized by its oxidizing potential and its acidity (Shamburger, Patrick and Lutten, 1975). Cements composed of quartz, silica and the iron and aluminum oxides and hydroxides are stable, whereas the calcite, dolomite and pyrite cements are unstable.

Weathering of a clay shale may also be caused by certain physical factors. These include, (1) unloading of confining stresses (2) inducement of shear stresses and strains, (3) release of stored strain energy (Bjerrum, 1967).

Reidenover, Geiger and Howe (1974, 1976) found that shales that are thinly laminated (lamellae thinner than 0.5 mm) are almost always of low durability. Shales that have more coarse sized particles were found to be more durable.

2.4 Engineering Classification of Shale

Shales which are difficult to break down during compaction and which subsequently degrade in the service environment present particular engineering problems. Deo (1972) developed index tests for the engineering classification of shales based on their durability. Shales

were classified as "rock like" shales, "intermediate 1 and 2" shales and "soil like" shales.

Chapman (1975) made a comparative study of shale classification tests and systems proposed by Gamble (1971), Deo (1972), Morgenstern and Eigenbrod (1974), Saltzman (1975), and Reidenouer, Geiger and Howe (1974). Based upon the testing of six shales, all from Indiana, he reached the following conclusions which apply specifically to the use of shale in compacted embankments.

1. Tests which show little promise in adequately characterising shale variabilities are the:
Washington Degradation, Ultrasonic Cavitation,
Schmidt Hammer, Ethylene Glycol, Modified
Soundness and Los Angeles Abrasion.
2. Tests which are particularly useful in classifying shales are the: Slake Index, Slake Durability, and Rate of Slaking.
3. Deo's classification system, seems to have some limitations, as "intermediate 1 and 2" shales have not as yet been identified either in his study or by extensive testing by the Indiana State Highway Commission.
4. Predictions of how thoroughly to degrade and compact shales can be accomplished from classification test values, but only after considerable monitoring of actual embankment performance.

Bailey (1976) measured shale degradation and its relation to compaction effort and unit weight. The static compression test was favored in comparison to kneading, gyratory and impact compaction. Additionally, the Scleroscope hardness and, particularly, the point load strength test were found to show some promise as indices of the engineering properties of shales.

The residual friction angle ϕ'_r is sometimes considered as being an index property as well as an engineering property (Heley and MacIver, 1971). It is considered to be uniquely related to the mineralogic composition of a material and independent of the type of specimen (intact, pre-cut or remolded), normal stress, and rate of strain, as long as there is sufficient displacement to produce a minimum shear strength.

In a subsequent report of the Corps of Engineers, Townsend and Gilbert (1974) found that correlations of the form

$$\tan \phi'_r = \frac{1}{\alpha + \beta/c}$$

best describe the relationship between the air dried index property c (LL, PI, or percent of minus 2 μ size) and ϕ'_r . α and β are parameters depending upon the index property and pretreatment procedures used to prepare the shales for testing. The parameters α and β are unfortunately also dependent on the type of shale. The relationship is useful for predicting variations in ϕ'_r within a formation. For the numerous shales tested, the correlated values of α and β were found to range as shown in Table 1.

Table 1
Coefficients of Empirical Equations Correlating Index
Properties and $\tan \phi'_r$

Physical Property c	Range of Coefficients	
	α	β
Liquid Limit	7.29 to 16.50	- 13.4 to - 423.4
Plasticity Index	7.29 to 11.81	- 2.3 to - 110.1
% < 2 μ	7.25 to 11.21	- 0.6 to - 154.6

(From Townsend and Gilbert, 1974).

2.5 Compacted Shale Strength

The state-of-the-art of compacted shale strength is to be found in Shamburger, Patrick and Lutten (1975) and Bragg and Zeigler (1975).

The report by Shamburger, Patrick and Lutten (1975) includes strength data for several compacted shales extracted from Corps of Engineers design memoranda. The effective stress shear strength parameters range from $c' = 0$ to 0.86 tsf (11.94 psi) and $\phi' = 25$ to 45 degrees. The as-compacted dry densities ranged from 65 to 142 pcf. Consolidation pressures up to 20 tsf (278 psi) were used in the tests. The total stress shear strength parameters determined from tests on samples saturated by back pressuring generally gave lower values than samples tested at compacted water contents. Strength parameters for a majority of the tests on saturated samples indicated values of $\phi = 14$ to 20 degrees and values of c as large as 0.5 tsf (6.9 psi), on a total stress basis. A number of the total stress and effective stress envelopes were found to be curved, and the high values of the strength intercept were attributed to the curvature of these envelopes.

Hall and Smith (1971) have reported total stress friction angles ranging from $\phi = 14$ to 34 degrees, depending upon the density and moisture content of the compacted shale. The angles of effective Mohr failure envelopes for these same tests were found to range from $\phi = 30$ to 38 degrees. Smith and Kleiman (1971) found that the failure envelope obtained from multi-stage triaxial tests on a single specimen was comparable to that obtained by testing several specimens at different confining pressures. They have quite correctly pointed out that this duplication of test results might not necessarily be true

for all materials. Values of strength intercept and friction angle based on both total stress and effective stress are reported.

The properties of some compacted shales were presented at the Seventh Ohio River Valley Soils Seminar (1976). In the paper by Drnevich, Ebelhar and Williams (1976), the principal stress ratio was used as the failure criterion. The use of the maximum shear strength criterion, however, would give a lower value for the effective stress strength parameters. The peak strength parameters were found to be related to the plasticity index and to the as-compacted condition of the shale.

In the paper by Bishop and Rose (1976), effective angles of shearing resistance ranging from $\phi' = 29$ to 34 degrees, with values near 30 degrees being more common, are reported.

In the paper by Fetzner (1976), effective angles of shearing resistance range from $\phi' = 36$ to 41 degrees and effective cohesion range from $c' = 0$ to 0.7 tsf (9.7 psi). The maximum shear strength failure criterion was used.

Compared to the voluminous literature on the shearing resistance and stress-strain behavior of sands and clays, the literature on compacted shale is very meager. Information on consolidation characteristics and pore water pressure response is also lacking. Much of the work done on shale has been on intact specimens.

CHAPTER 3

PURPOSE AND SCOPE

The primary purpose of this research is to develop laboratory testing techniques for the evaluation of the long term strength parameters of compacted shales. The effective stress strength parameters are presumed to be the most appropriate for the analysis of the long term stability of an embankment. These parameters can be determined by performing either drained tests or undrained tests with pore water pressure measurements, and they can only be used in situations where the pore water pressures in the embankment are known.

The effective stress strength parameters were evaluated by performing consolidated undrained triaxial compression tests on soaked specimens. The initial gradation of crushed shale aggregate, the molding water content, the compaction pressure, and the pre-shear consolidation pressure were adopted as the independent test variables.

The secondary objectives include a study of the factors which influence the compaction characteristics, consolidation characteristics, and the stress-pore pressure-strain behavior during undrained shear.

A limited number of one-dimensional compression tests were performed to evaluate the prestress induced in specimens of compacted shale (Appendix D). Two one-dimensional compression tests followed by wetting were performed to demonstrate the non-durable nature of the shale aggregate used in the study (Appendix E).

New Providence shale was selected for testing in preference to other shales as it combines a relatively high resistance to mechanical degradation (breakdown) with a relatively low durability. The test shale was sampled from a road excavation on I-265 in Floyd County, south central Indiana.

3.1 Test Procedure

All the triaxial test specimens were compacted, using a California type kneading compactor, to a nominal height of 8 1/2 inches and a nominal diameter of 4 inches. The specimens were saturated using deaired distilled water under a back pressure exceeding 50 psi. All the specimens were isotropically consolidated under elevated pore water and cell pressures. They were then sheared undrained at a constant rate of strain up to an axial compressive strain of 20%, maintaining a constant cell pressure for each test.

3.2 Rationale for Selection of Test Variables

Regardless of the specifications, the as-compacted condition of shale in most embankments is often not uniform. The density and moisture content could vary from place to place depending on such factors as the lift thickness, the method used for adding water and mixing, and the maximum size and gradation of the shale prior to compaction. There is also the possibility that some parts of an embankment may contain poorly compacted material with large voids and low density.

The laboratory testing program adopted in this research is directed towards the simulation of the field conditions referred to above as far as possible. The gradation of the shale aggregate prior

to compaction was varied using a top size of 3/4 inch. The aggregates were used at their natural moisture content. The molding water and the compaction pressures were varied to obtain a field range of specimen densities and moisture contents. Consolidation pressures varying from 0 to 40 psi were used. Larger aggregate sizes and higher consolidation pressures could not be used due to the limited capacity of the testing equipment.

3.3 Summary Listing of Independent Variables and Levels

A. Materials & Processing

1. Shale: 1 level, New Providence formation.
2. Shale Mineralogy: 1 level, determined by X-ray diffraction.
3. Processing: 1 technique; crushed and scalped to 3/4 inch top size.
4. Gradation: 6 levels, blended to approximate exponential curves with 3/4 in. and 1/2 in. top size (see Figure 3 and Table 7).

B. Compaction

1. Compactive Technique: 1 level, kneading; single sample size 8 1/2 inch x 4 inch; each lift of 500 gm shale aggregates plus molding water.
2. Nominal Compactive Effort: 3 levels; achieved by varying foot pressure to 50 psi; 100 psi and 200 psi; 30 tamps per lift of 500 gm shale aggregate.
3. Molding Water Added: 4 levels; 25 gm, 30 gm, 35 gm and 40 gm per 500 gm of shale aggregates (5% to 8%).

C. Saturation and Consolidation:

1. Technique for Saturation: 2 levels; vacuum saturation and back pressure; the former plus higher back pressure.
2. Technique for Consolidation: 1 level; saturation at $\sigma'_c = 1$ psi, followed by consolidation at as high a back pressure as system will allow.
3. Consolidation Pressure: 6 levels; 0, 1, 5, 10, 20, 40 psi, all isotropic.
4. Volume Change Measurements: 1 technique (see Section 5.4).

D. Shear Testing

1. Testing Technique: 1 level ($\overline{\text{CIU}}$); axial compression at constant rate of strain, constant cell pressure, axisymmetric stress system ($\sigma_2 = \sigma_3$).
2. Pore Pressure Measurements: 1 technique; pore pressure transducer.
3. Axial Load Measurement: 1 technique; load cell.
4. Limiting Test Condition: Ultimate ($\sigma_1 - \sigma_3$) or 20% strain.

CHAPTER 4

DESCRIPTION OF TEST SHALE

4.1 General

As pointed out in Chapter 2, the shales most likely to produce difficulties are those which combine a relatively high resistance to mechanical degradation with a relatively low durability. A shale believed to meet these criteria was sampled in May 1975 at an elevation of 575 feet along a road excavation on I-265 in Floyd County, south central Indiana. Figure 2 shows the sampling location with respect to the bedrock geology of Indiana.

4.2 Geology

The geologic description of the shale as reported by the Indiana State Highway Commission (1975) is as follows:

Era	-	Paleozoic
Period	-	Carboniferous
Epoch	-	Mississippian
Series	-	Valmeyeran (Osage)
Group	-	Borden
Formation	-	New Providence
Age	-	241 to 261 million years.

Mississippian rocks in Indiana lie in a band that trends north-westward-southeastward across approximately the middle of the state (Harrison and Murray, 1964). Mississippian epoch rocks are assigned to four series in Indiana: Kinderhook, Osage, Meramac, and Chester

FIGURE 2 BEDROCK GEOLOGY OF INDIANA AND SAMPLING LOCATION.

(Table 2). The oldest (Kinderhook) rocks are at the east edge of the band, and the youngest (Chester) rocks are at the west edge.

The New Providence shale lies at the base of the Borden group, which lies in a narrow band about 12 to 15 miles wide, reaching from New Albany on the Ohio river to just south of Lafayette. The lower Borden shales (including the New Providence formation) are similar mineralogically, and contain illite, chlorite, kaolinite, quartz and feldspar (Harrison and Murray, 1964). They are commonly sandy and silty, and range in color from blue gray to brown. In most places they are massive to blocky on fresh surfaces, but on weathered surfaces they display definite partings and break out in small pieces. They vary from relatively soft to very hard. In general, the finer grained shales are softer and the coarser grained ones are harder.

4.3 Engineering Tests on the Sample

A battery of engineering tests was run by the Indiana State Highway Commission (ISHC) on the particular New Providence shale used in this study. The results of these tests are summarized in Table 3. This shale is of a light gray color, is medium hard, and has a "massive" category of fissility. It is classified as "soil-like" according to the classification procedure developed by Deo (1972).

In terms of soil classification it is a silty clay, and falls within the AASHTO classification as an A-4 (10) material, with a plastic limit of 20.8%, a liquid limit of 30.8%, and hence a plasticity index of 10.0%.

The particle sizes are comprised of: sand (2%), silt (67%), clay (21%) and colloids (10%). The specific gravity of solids is 2.78,

Table 2
Mississippian Rock Units in Indiana

Series	Group	Formation
Chester		Kinkaid Limestone Degonia Sandstone Close Limestone Palestine Sandstone Menard Limestone Waltersburg Sandstone Vienna Limestone Tar Springs Formation
	Stephensport	Glen Dean Limestone Hardinsburg Formation Golcenda Limestone Big Clifty Formation Beech Creek Limestone
	West Baden	Elwren Formation Reelsville Limestone Sample Formation Beaver Bend Limestone Bethel Formation
	Blue River	Paoli Limestone St. Genevieve Limestone St. Louis Limestone
Meramec		Salem Limestone Harrodsburg Limestone
Osage	Borden	Edwardsville Formation Floyds Knob Formation Carwood Formation Locus Point Formation NEW PROVIDENCE SHALE
Kinderhook		Rockford Limestone New Albany Shale (upper part)

From Harrison and Murray (1964)

Table 3

Summary of Indiana State Highway Commission Test Results on
New Providence Shale (Test Shale)

Laboratory No. (ISHC) 75-55731

General Physical Description

Color - Light Gray Hardness - Medium Fissility - Massive

Shale Classification		Soil Classification	
Degradability-	Soil-like	Textural	Silty-Clay
Slaking Index-1 cycle	3%	AASHTO	A-4 (10)
-5 cycles	48%	Plastic Limit	20.8%
Slake Durability Index		Liquid Limit	30.8%
Dry -200 revolutions	91%	Plasticity Index	10.0%
-500 revolutions	76%	Sand Size	2%
Soaked-200 revolutions	71%	Silt Size	67%
-500 revolutions	41%	Clay Size	21%
Fissility Number	26	Colloid Size	10%

Physical Properties		Moisture-Density Relations	
Natural Wet Density	154.8 pcf	Compactive Effort - Standard AASHTO	
Natural Dry Density	145.0 pcf	Maximum Size	No. 4
Natural Moisture	6.1%	Maximum Wet Density	135.2 pcf
Specific Gravity	2.78	Maximum Dry Density	120.1 pcf
pH	6.5	Optimum Moisture Content	12.1%
Shrinkage Limit	21.2%		
Lineal Shrinkage	1.9%	California Bearing Ratio	
Loss on Ignition	4.0%	As-compacted CBR	10.2%
		After soaking CBR	1.1%
		Average Swell	2.8%

the natural moisture content is 6.1%, and the shrinkage limit is 21.2%. The percentage swell measured in the CBR mold, for minus No. 4 material compacted at optimum moisture content (12.1%) and using standard AASHTO effort, is 2.8%.

4.4 Mineralogy

The X-ray diffraction test was conducted on powdered shale samples to identify the principal clay and non clay minerals present. The tests were performed according to the procedure outlined by Kinter and Diamond (1956). Five specimens were prepared and treated as follows:

1. Air dried
2. Oven dried at 550°C
3. Treated with KCl and oven dried at 105°C
4. Treated with MgCl₂ and oven dried at 105°C
5. Treated with Glycerol

The diffractometer was adjusted as indicated below:

Voltage	50	kilovolts
Current	39	milliamperes
Goniometer Speed	0.2	degrees/min
Beam Slit	1	degree
Detector Slit	0.1	degree
Time Constant	2	seconds
Chart Speed	60	inches/hour
Linear Scale	5000	counts/second
E	5	volts
ΔE	6	volts
Radiation	Mo	(Molybdenum)
Filter	Zr	(Zirconium)

The diffraction patterns obtained with the five treated samples are shown in Appendix F. After determining the positions of the peaks (2θ), the interplanar spacings ($d\text{\AA}$) were calculated using Bragg's Law:

$$d = \frac{n\lambda}{2 \sin \theta}$$

where d = Interplanar spacings in Å

λ = Wave length of radiation (for molybdenum radiation
 $K\alpha_1 = 0.70926 \text{ Å}$)

$n = 1$ for basal (001) peak position

2θ = Diffractometer angle in degrees for position of peaks.

The interplanar spacings so obtained were then compared with a "Simplified X-ray Analysis Guide for Clay Minerals" (Diamond, 1975), shown in Table 4 for " d " and Table 5 for " 2θ ".

The characteristic features of kaolinite and illite were clearly displayed in all five specimens. The oven dry specimen heated to 550°C showed a peak corresponding to vermiculite and/or chlorite, but these corresponding peaks were not observed in the air dry and glycerol treated specimens. The presence of halloysite, in either the hydrated or the dehydrated form, could not be readily established, as its diffraction pattern is very close to kaolinite and illite in many respects.

The shale was then studied under a scanning electron microscope (SEM). The tubular-like structure of halloysite was not observed.

The non-clay material in the shale was found by X-ray diffraction analysis to be predominantly quartz.

The new Providence shale is therefore comprised of quartz, kaolinite and illite. Traces of chlorite and vermiculite may also be present. Montmorillonite which is identified with more recent rocks, volcanic activity, and weathering in arid regions (Wood, Lovell, and Deo, 1973) is not present.

Table 4

Simplified X-Ray Analysis Guide for Clay Minerals

"Basal" (001) peak position*

(After Diamond, 1975)

Mineral	Air Dry	Glycerol treated	K ⁺ -sat. & oven dried	MgCl ₂	Heated to 550°C
Kaolinite	7.2 Å	7.2 Å	7.2 Å	7.2 Å	disappears
Halloysite (dehyd. form)	7.3 - 7.4 Å (broad)	7.4 Å (broad) or 10.6 Å	7.3 Å (broad)	7.3 - 7.4 Å	disappears
Halloysite (hyd. form)	10 Å	10.6 Å	7.3 Å (broad)	10.0 Å	disappears
Montmorillonite	11-15 Å (variable)	17.7 Å	10-12 Å	14-16 Å	about 9.7 Å
Illite	10.2 Å	10.2 Å	10.2 Å	10.2 Å	about 10 Å
Vermiculite	12.5 Å or 14 Å	14.5 Å	10-11 Å	14 Å	10 Å (sometimes re-expands to 14 Å in air after heating)
Chlorite	14 Å	14 Å	14 Å	14 Å	13.6 - 14 Å (sometimes intensity reduced)

*Subject to some uncertainty depending on substitution in the lattice, cation saturation, etc.

Table 5

Values of 2θ for Mo Radiation (Basal 001 - Peak Position)

Mineral	Air Dry	Glycerol	KCl	MgCl ₂	550°C
Kaolinite	5.646	5.646	5.646	5.646	-
Halloysite	broad 5.569 to 5.494	(broad) 5.494 or 3.834	5.569 (broad)	5.569 to 5.494	-
Halloysite	4.065	3.834	5.569 (broad)	4.065	-
Montmorillonite	3.695 to 2.709 (variable)	2.296	4.065 to 3.387	2.903 to 2.540	4.190 about
Illite	3.985	3.985	3.985	3.985	4.065
Vermiculite	3.251 or 2.903	2.803	4.065 to 3.695	2.903	4.065 or 2.903*
Chlorite	2.903	2.903	2.903	2.903	2.988 to ** 2.903

*Sometimes on account of re-expansion in air after heating.

**Sometimes intensity reduced.

Partial dehydration of hydrated halloysite takes place at temperatures of 60° to 70°C (Grim, 1953). It has also been shown by electron microscopic studies that drying of hydrated halloysite may result in a splitting or unrolling of the tubes (Mitchell, 1976). However, complete dehydration only takes place at temperatures of the order of 400°C . Hydrated halloysite ($4\text{H}_2\text{O}$) can dehydrate irreversibly to metahalloysite ($2\text{H}_2\text{O}$).

4.5 Scleroscope Hardness

Bailey (1976) performed scleroscope hardness tests on the New Providence shale at various moisture contents. The values ranged from 21 to 40.5 with a mean value of 33. The hardness value H was found to be a function of the particle moisture content w_i % according to the linear regression equation

$$H = 32.1 - 2.4 w_i$$

with a Pearson's R^2 correlation coefficient of 0.43. The hardness was measured perpendicular to the bedding plane and the moisture content of the particles ranged from 3.6% to 0.0%. At zero moisture content (oven dried at 105°C), the hardness values ranged from 27.5 to 40.5. At the highest moisture content of 3.6% the range of values was very much less, being from 23.5 to 24.5. Bailey attributed the scatter in hardness values at zero moisture content to the deterioration of the shale as evidenced by the development of small hair line cracks, chipping and flaking. The scatter was very much reduced at higher moisture contents.

4.6 Point Load Strength

The point load strength of the New Providence shale ranges from 490 to 1980 psi at zero moisture content (Bailey, 1976). The scatter was attributed to natural inhomogeneities in the test specimens and deterioration on oven drying to 105°C. This deterioration appeared as flaking, chipping and the formation of hair line cracks. The relationship between point load strength and water content for the New Providence shale showed too much scatter within the limited moisture content range of 0.0% to 2.46% for any trends to be recognized. However, for four of the other shales tested, it was found that the point load strength decreased with increase in moisture content in a more or less linear manner.

4.7 Soaking Degradation and Absorption

Bailey (1976) studied the degradation of New Providence shale. Aggregates of shale in a loose condition were kept immersed in water for variable lengths of time ranging from 7 days to 226 days. The initial gradation of the aggregate was kept more or less constant and thirteen one-pound samples were used. Each sample was soaked for a different length of time and the gradation after soaking was determined by wet sieving. He found that either the degradation resulting from slaking was not a time-dependent phenomenon, or that the test procedure was insensitive to the degradation that occurred. This may partly be due to the fact that separate samples were used for each time period. It could also be inferred that the degradation varied with time up to 7 days (or maybe less time), and thereafter no further degradation took place. It is also possible that the maximum

time allowed for slaking degradation was not adequate.

As regards increase in moisture content by absorption, Bailey (1976) found that the percentage absorption of water by the particles increased as the particle size decreased. After seven days soaking, the water content of particles ranged from 10.2% (3/4 in. to 1/2 in. fraction) to 62.3% (No. 30 to No. 50 fraction). He also found that the increase in water content with soaking time (7 to 226 days) was negligible for the smaller particle sizes and was very slight for the larger (3/4 in. to 1/2 in.) particles.

Bailey's research on slaking degradation and absorption seem to indicate that the two phenomena are directly linked for all periods of time. Degradation and water absorption appear to take place simultaneously and the following mechanisms are offered for the slaking process.

A shale aggregate is a three phase system of solids, water and air. If water is absorbed without a change in volume, the water entering the aggregate would compress the air and its pressure would increase. The increase in the pore air pressure could cause the aggregate to split and slake (Badger, Cummings and Whitmore, 1956).

If the aggregate absorbs water without a change in the pore air pressure, then it will probably expand or swell. When the shale swells, the void ratio increases with a corresponding increase in the distance between the primary clay size particles. The bonds holding the clay particles together are then broken and the shale aggregate slakes.

The two mechanisms probably originate at the surface of the shale aggregate and progressively propagate towards its interior.

4.8 Comparison Among New Providence Shale Samples

The ISHC has reported tests on a total of four samples of New Providence shale from Floyd County, Indiana. Two of the samples were obtained from a road excavation for I-265 at an elevation of 575 ft. The other two samples were from two rock core borings offset from an existing road (SR 111) at an elevation of around 394 to 395 feet. The shale used in this study is described under Laboratory No. 75-55731, in Table 6.

The slaking index (SI) refers to the degradation of pieces of shale when subjected to cycles of drying and wetting. It measures the weight loss on slaking expressed as a percentage of the original weight. In the left hand column of Table 6, the numbers within parentheses refer to the number of cycles. The variation in this index for New Providence shale is very large, and all the shales slake to varying degrees.

The fissility of the New Providence shales varies from massive to flaky. The variation in the Slake Durability Indices is large only for the soaked, 500-cycle test. The plastic limits are more or less the same and this is an indication that the mineralogical constituents are similar. The size distribution of the individual grains of which the shale is composed is also essentially similar with 0 to 8% sand sizes, 60 to 70% silt sizes, and the balance in clay sizes and colloids.

The shrinkage limit varies from 16.5 to 28.7% and the CBR swell is around 2.1% under a surcharge of 0.35 psi. The linear shrinkage, which is a measure of cracking potential, lies between 1.9 and 4.8%. Loss on ignition amounts to around 3.6 to 4.0%.

Table 6

Comparison of Properties of Test Shale with Other Samples of
New Providence Shale

	<u>Test Shale</u>			
Laboratory No.	75-55505	75-55731	76-55529	76-55530
Highway	I-265	I-265	SR111	SR111
Date Sampled	2-21-75	5-27-75	3-16-76	3-16-76
Date Received	2-24-75	5-28-75	3-25-76	3-25-76
Station	116 + 00	116 + 00	1096+12	1097+12
Offset ft.	na	na	19	20
Depth ft.	na	na	13	37
Elevation ft.	575	575	395	394
Source of Sample	Cut	Roadway	Rock Core	
	Slope	Excavation	Boring	
<u>Gen. Phy. Description</u>				
Color	Gray	Lt. Gray	Gray	Gray
Hardness	Medium	Medium	Medium	Medium
Fissility	Flaky	Massive	Flaggy	Flaggy
Shale Classification	Soil like			
SI (1)	7.9	3.0	1.8	99.0
(5)	46.3	47.7	30.2	99.2
(Id) _d (200)	90.7	91.0	na	na
(500)	82.9	76.0	86.5	88.0
(Id) _s (200)	66.9	71.0	na	na
(500)	45.1	41.0	71.4	73.6
Fissility	15	26.0	100	47
Modified Soundness (Is)%	na	na	na	na
Soil Classification	Silty	Silty	Silty	Silty
Textural	Clay	Clay	Clay Loam	Clay Loam
AASHTO	A-6(12)	A-4(10)	A-4(9)	A-4(10)
w _p %	22.6	20.8	20.6	22.5
w _l %	33.6	30.8	30.9	32.7
I _p %	11.0	10.0	10.3	10.2
Sand Size %	0.4	2.0	7.9	7.2
Silt Size %	60.6	67.0	67.9	69.3
Clay Size %	23.2	21.0	17.0	16.8
Colloid Size %	15.8	10.0	7.2	6.7
<u>Physical Properties</u>				
Nat. Wet Density pcf	150.9	154.8	161.1	162.0
Nat. Dry Density pcf	139.4	145.0	157.6	158.8
Nat. Moisture %	8.3	6.1	2.2	2.0
Specific Gravity	2.78	2.78	2.773	2.776
pH	7.4	6.5	6.8	6.8
Shrinkage Limit %	28.7	21.2	16.5	17.6
Linear Shrinkage %	2.1	1.9	4.8	4.6
Loss on Ignition %	3.8	4.0	3.7	3.6

Table 6 (cont.)

<u>Moisture Density</u>	Std	Std	na	na
<u>Relations</u>	AASHTO	AASHTO	na	na
Maximum Size	#4	#4	na	na
Max. Wet Density pcf	133.4	135.2	na	na
Max. Dry Density pcf	120.6	120.1	na	na
Optimum Moisture %	10.3	12.1	na	na
<u>California</u>				
<u>Bearing Ratio</u>				
As-compacted	11.0	10.2	na	na
After soaking	0.7	1.1	na	na
Swell %	2.1	2.8	na	na
Surcharge (psi)	0.35	0.35	na	na

na: not available

Three of the samples were acidic (pH = 6.5 to 6.8) and one sample was basic (pH = 7.6). The natural moisture content varies from 2.0 to 8.3%, the specific gravity varies slightly from 2.773 to 2.780. The data on moisture-density and CBR values are not sufficiently complete to justify comment.

CHAPTER 5

APPARATUS AND PROCEDURE

5.1 Preparation of Size Fractions

The maximum particle size that can be accommodated is governed by the size of the triaxial test specimen proposed to be used. In the case of soils, the necessity for breaking down the particles does not arise, and small cylinders up to 2 1/2 inches in diameter are tested.

In the case of rock like material such as shales, the size of the particles in the test specimen usually has to be scaled down. This is performed by mechanical degradation of the chunks of shale which have been excavated. For a 4-inch diameter test specimen, the maximum particle size is taken as 3/4 inch. The gradation of the crushed material can be selected as desired. In cases where the limits of the field gradation are known, the laboratory gradation is selected to parallel the field one (Lowe, 1964). The field gradation on the other hand is dependent on the specifications laid down for the construction of the embankment. The size of the chunks excavated from the beds, the height of the lifts, the type and weight of the compaction equipment, the number of passes and the amount of water used are the major factors which influence the field maximum size and gradation for any particular shale.

Figure 3 and Table 7 show the initial gradations used in this study. Chunks of shale up to 18 inches in size were broken down with

FIGURE 3 INITIAL GRADATIONS USED

Table 7
Initial Gradations Used

Size Fraction	Weight of Size Fractions in Grams					
	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
P 3/4" R 1/2"	95	50	50	50	-	-
P 1/2" R 3/8"	52	150	150	150	250	200
P 3/8" R No. 4	103	100	100	100	250	100
P No. 4 R No. 8	73	100	100	200	-	100
P No. 8 R No. 16	52	-	100	-	-	100
P No. 16 R No. 30	37	100	-	-	-	-
P No. 30 R No. 50	26	-	-	-	-	-
P No. 50 R No. 100	19	-	-	-	-	-
P No. 100 R No. 200	30	-	-	-	-	-
P No. 200	13	-	-	-	-	-
Total Weight (gms)	500	500	500	500	500	500

a hammer to approximately a 6 inch maximum size. These were fed into a jaw crusher which produced shale pieces, the bulk of which passed through a one inch sieve. Pieces larger than 3/4 inch were broken down with a small hammer.

The broken shale was then sorted into different sizes by a series of sieves (3/4", 1/2", 3/8", and Nos. 4, 8, 16, 30, 50, 100 and 200). The sorted pieces of shale were then stored in sealed polythene bags at room temperature and relative humidity. The bulk sample of shale, prior to crushing, was stored in a sealed metal container inside the laboratory. The crushing and sorting of the test shale was spread out during the testing program time.

The weight of sample corresponding to each lift in the compaction mold was held constant at 500 gm at its natural moisture content. Once the gradation for a test specimen was selected, the different sizes were weighed accordingly and placed in a small polythene bag. Nine bags containing 500 gms of shale particles at the natural moisture content were prepared for each test.

One 500 gm sample was used to determine the average moisture content of the entire gradation. This moisture content was found to decrease from a maximum of 4.0% to a minimum of 1.1% over a period of thirteen months (from June 1975 to July 1976). The remaining eight 500 gm samples were used for forming compacted specimens.

Gradation No. 1 was obtained from the Talbot and Richart (1923) equation used earlier by (Hennes, 1952)

$$P = 100 \left(\frac{d}{D} \right)^n$$

where P = percentage by weight finer than size d

d = any diameter,

D = maximum grain diameter

n = an abstract number.

A uniform grain size corresponds to a n -value of infinity. Gradation No. 1 corresponds to a n -value of 0.5. The variation of P with n for a maximum particle size $D = 3/4$ inch is shown in Figure 4.

Comparing Figures 3 and 4 it is seen that the initial gradations used to form compacted triaxial specimens fall within the exponential gradations, with n ranging from 0.5 to 4. The minimum grain size used varied from No. 4 to less than No. 200. The maximum size used varied from $3/4$ inch to $1/2$ inch. The latter size was used when the $3/4$ inch to $1/2$ inch size fraction had been exhausted. The gradation of $1/2$ inch to No. 4 made use of the large amount of pieces remaining in this size range after crushing.

5.2 Formation of Compacted Specimens

5.2.1 Molding Water

The addition of water to pieces of shale usually causes slaking, and mixing of the two constituents leads to further degradation. In the field, large chunks of shale are broken down by the compaction equipment as much as possible, and water is added from a water truck which spreads the water as it moves forward. The distribution of water at first is far from uniform on account of the varying porosity of the material and the tendency for water to accumulate along the tracks of the water truck and earth moving equipment. In addition,

FIGURE 4 EXPONENTIAL GRADATIONS

non-uniform discharge from the water spreader sometimes takes place due to blocked and worn out nozzles. The material is then disked to achieve a more even distribution of water. However, this phase is often omitted unless specified. The material is then compacted by rollers. The elapsed time between placing the material, breaking it down, adding water, diskings and rolling is relatively short.

A procedure simulating field practice was adopted in the laboratory for adding water to the shale pieces and mixing the constituents prior to placing the mixture in the mold for compaction. A known amount of water was added to 500 gm of previously prepared (Section 5.1) shale pieces contained in a small polythene bag. The water was distributed by rotating and shaking the bag, great care being taken to prevent loss of moisture. A molding action was imparted to the bag to distribute the water in cases where the pieces adhered to each other. The contents of the bag were then placed into an assembled steel mold set up on the rotating base of a kneading compactor, spread with a large steel spatula, and compacted (Section 5.2.2).

The amount of molding water used was varied from 25 gm to 40 gm per 500 gm of shale pieces. This amounts to a variation in moisture content from 5% to 8%.

5.2.2 Kneading Compaction

The compaction of laboratory specimens can be accomplished by several methods, the more common being vibration, static pressure, dynamic or impact energy, gyratory, and kneading compaction. In the field, compaction of embankments has been achieved through the use of vibratory compactors, tampers, tamping-type rollers, smooth wheel

power rollers, pneumatic tired rollers, track type tractors, sheepsfoot rollers, disc-shaped rollers, etc.

The kneading-type compaction of laboratory test specimens was originated in 1937 in an attempt to devise a method that would more closely simulate the field compaction produced by conventional sheepsfoot and pneumatic tired rollers. The characteristics of kneading-type compaction are summarized below (Wahls, Fisher and Langfelder, 1966).

1. The compaction pressure is gradually built up, allowed to dwell on the sample for a specified length of time, and then gradually released.
2. Lateral shearing stresses and strains are developed in the soil as the compaction pressure is applied over a limited area.

In this study the California type mechanical kneading compactor was used to compact 4 in. diameter x 8.5 in. high test specimens. Each lift, containing 500 gms of shale particles at the natural moisture content plus added water (Section 5.2.1), was subjected to 30 tamps from a 3.2 sq. in. steel tamper shoe over a period of one minute at a constant setting of the compaction pressure. The number of lifts required to achieve a compacted height of approximately 8.5 to 8.8 inches depended on the compaction pressure used, and was found to vary from 7 to 8. After the required height was obtained, the top of the sample was trimmed to form a right circular cylinder. A steel blade approximately 3 inches wide and a spare kneading compactor foot were used to break up and level the uneven compacted surface, while the specimen was still in the mold. The top and bottom of the steel mold were then sealed and left to cure for a period of 24 hours.

Compaction pressures of 50, 100 and 200 psi and water contents ranging from 5% to 8% were used.

5.2.3 Extrusion of Specimen

A simple extrusion machine which could either be driven manually or by an electric motor was used to recover the sample from the steel mold. The bolts holding the two halves of the mold together were completely unscrewed in a uniform manner. A half inch thick steel disc having a diameter of 4 inches was then placed against the bottom of the sample. The mold containing the sample was transferred to the extruding machine. The adhesion between the compacted specimen and the mold was broken by applying a pressure through a screw-jack to the bottom of the sample, which was free to move inside the stationary mold. This part of the operation was carried out by manually turning the pulley which drove the vertical screw-jack. The specimen was thereafter completely extruded from the split mold by driving the screw-jack at a uniform speed using the electric motor. The sample was then slid laterally onto a smooth steel plate to facilitate moving the sample without subjecting it to handling stresses.

5.2.4 Compacted Condition

The overall condition of the compacted specimen is described in terms of its bulk density, dry density, moisture content, void ratio and degree of saturation. These properties are determined by measuring the bulk volume and weight of a sample, its water content, and the specific gravity of solids.

A balance accurate to 0.01 lb. was used for the compacted specimens, which weighed around 9 lb. The height and diameter of the test specimens were determined using a vernier caliper accurate to 0.001 inches. Four measurements spaced at 90° were taken of the height and were averaged. Six measurements were taken of the diameter, viz., at the top, center and bottom of the specimen, across two mutually perpendicular diameters and were averaged. The bulk volume and density were then computed.

The moisture content was calculated from the measured water content of the pieces, the known weight of a lift (500 gm), and the known weight of water added to the 500 gm of materials as follows:

a) Initial Moisture Content of Particles (w_i)

$$w_i = \frac{W_p}{W - W_p} \times 100\%$$

where W = weight of particles in gms.

W_p = weight of moisture lost from W gm of particles in gms.

w_i = initial water content of particles, percent.

b) As-compacted Moisture Content (w_c)

$$w_c = \frac{W_p + W_a}{W - W_p} \times 100\%$$

where W_a = weight of moisture added to W gm of particles, in gms.

w_c = as-compacted moisture content, in percent.

The dry density was computed from the bulk density, determined earlier, and the as-compacted moisture content using the equation

$$\gamma_d = \frac{\gamma_b}{1 + w'_c}$$

where γ_d = dry density, in pcf

γ_b = bulk density, in pcf

w'_c = moisture content expressed as a ratio = $w_c/100$.

The void ratio e was calculated using the equation

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1$$

where G_s = specific gravity of solids = 2.78

γ_w = unit weight of water = 62.4 pcf

and the degree of saturation S_r was calculated using the equation

$$S_r = \frac{G_s w_c}{e}$$

A typical set of calculations for evaluating the dry density, the moisture content, the void ratio and the degree of saturation of the test specimens at various stages is given in Appendix A.

5.3 Saturation and Consolidation

5.3.1 Setting Up Sample

The saturation and consolidation of each partially saturated test specimen was carried out after setting it up in the triaxial cell. The specimen was enclosed in a 0.01 inch thick rubber membrane. Top and bottom drainage was provided by porous stones. Filter paper discs were placed between the porous stones and the specimens to facilitate

the removal of the specimens after the test, and to prevent the porous stones from getting clogged by fine clay particles. Stretched O-rings were placed to provide a watertight seal between the membrane and the loading caps, the cylindrical surfaces of which were smeared with a silicone vacuum grease. The triaxial chamber was then positioned and clamped. Deaired distilled water was introduced into the chamber until it was completely filled.

5.3.2 Back Pressure Saturation

The cell pressure was increased to 1 psi, and deaired distilled water was allowed to enter the specimen through the bottom drainage line while a vacuum was applied to the top drainage line. In the case of samples having a high void ratio, the flow of water into the voids was relatively rapid. In the case of dense samples, the flow of water was relatively slow and a period of approximately 4 to 6 hours was required for the rising column of water to reach the top drainage line. The flow of water into and out of the sample was measured, and when the flow of bubbles from the sample appeared to be very small in comparison to the flow of water, the top drainage valve was closed. The vacuum line was then disconnected and replaced by a previously saturated drainage line branching off the bottom drainage line. Thereafter, the sample was progressively back pressure saturated by increasing the cell pressure and the pore water pressure in equal increments following the procedure described by Lowe and Johnson (1960). The specimen was left to saturate under a back pressure in excess of 60 psi and an effective stress of 1 psi until the inflow of water ceased. The saturation of samples, first by partial evacuation of trapped air, then by compression and solution of any remaining air, was achieved within a period of approximately 24 hours.

Three indirect methods are available to check whether a specimen is fully saturated or not. In the B parameter method, an increment of confining pressure is applied to the sample with the drainage line closed, and the pore pressure response is measured. The ratio of the pore pressure response to the applied increment in confining pressure, B, for a fully saturated soil is given by Skempton (1954).

$$B = \frac{1}{1 + n \frac{C_w}{C_s}}$$

where n = porosity of the sample

C_s = compressibility of soil skeleton

C_w = compressibility of water.

When the factor $n C_w / C_s$ is negligible in comparison to unity, the value of B is one. However, when the compressibility of the soil skeleton decreases, the factor $n C_w / C_s$ increases and a fully saturated soil may give a B value less than one. For the samples tested in this study, the B value ranged from 0.9 to 0.96, depending on the compaction pressure used to compact the specimens.

In the second check for saturation, the sample was prevented from expanding by keeping the cell water line closed and the pore water pressure was increased by 30 psi. The instantaneous entry of water into the sample is equal to the compression of the air in the sample, after correcting for system flexibility. With passage of time, the air goes into solution under the higher pressure in the pore water.

The third method is to determine the weight, volume and degree of saturation of the specimen after the shear test. Determination of the weight and the water content presents no problems. The determination of the volume of a distorted specimen at the end of a triaxial test is, however, a very tedious process. Removal of the external stresses acting on the sample and the drop of the pore water pressure results in an expansion of pore air and a corresponding expansion of the sample. These volume changes which take place during removal of the sample from the triaxial cell cannot be accounted for satisfactorily. Hence, the measurement of the sample volume at the end of the test, to check whether it was fully saturated, was not carried out.

5.3.3 Consolidation

After the specimen had reached saturation, generally within a period of 24 hours, using the procedures described earlier, the cell pressure was increased, keeping the back pressure constant, to give the desired consolidation pressure. {A back pressure of only 50 psi and a cell pressure of 90 psi (maximum air line pressure) was used to consolidate samples at 40 psi.} The cell pressure and the back pressure were kept as high as possible and their difference adjusted to give the required consolidation pressure which took values of 0, 1, 5, 10, 20 and 40 psi.

The expulsion of water from the specimen during consolidation was monitored with time up to and slightly beyond the condition of primary consolidation. Hardly any air was found to come out of the sample while it drained. After consolidation, the drainage lines were closed and the B parameter determined once more to check the saturation of the

specimen prior to undrained shear. A B parameter of around 0.96 was considered satisfactory for the measurement of pore water pressures during undrained shear.

5.4 Undrained Shearing

After the saturated specimen was fully consolidated under the hydrostatic confining pressure, the drainage lines were closed and the specimen axially compressed at a constant rate of strain keeping the cell pressure constant by means of an air pressure regulator.

The axial load was measured by means of a calibrated load cell and the change in pore pressure was measured at the base of the specimen using a calibrated pore pressure transducer. The data were continuously recorded on a dual channel automatic recorder. A deformation rate of 0.011 inches per minute was used and the test terminated at 20% axial strain. For the size of samples used, the rate of strain amounted to around 0.113% per minute and the total strain was reached in 2 1/2 hours.

Two replicate specimens were axially loaded to 20% strain over a period of 10 hours to study the effect, if any, of the strain rate on the development of pore water pressures and the axial load.

Some tests were terminated at an axial strain less than 20%, when it appeared that the deformed specimen was tending to touch the sides of the triaxial chamber. This problem was mostly encountered at the commencement of the testing program when a triaxial chamber having an internal diameter of 5 inches was used. In the bulk of the tests a triaxial chamber of 6 inch diameter was used and the deformation of specimens upto 20% axial strain presented no problems.

As the axial load was measured at the top of the loading piston, the load on the piston due to the cell pressure was deducted from the measured load to give the corrected load on the test specimen.

On completion of shearing, the cell water was completely drained, the sample was removed from the cell, placed on a steel plate with the membrane still on and weighed, great care being taken to prevent loss of moisture. The weight of the sample was then determined by deducting the weight of the steel plate and the membrane. The change in weight of the sample from its as-compacted condition to its condition after saturation, consolidation and shearing is the change in the weight of water in the specimen. The change in volume of the specimen was then computed following the procedure detailed below.

The volume of air (V_a) in the specimen in its as-compacted state is given by

$$\begin{aligned} V_a &= V_o (1 - S'_r)(1 - \gamma_d/G_s \gamma_w) \\ &= V_o (1 - S'_r)\{e_o/(1 + e_o)\} \end{aligned}$$

where V_o = sample volume

e_o = void ratio

S'_r = degree of saturation expressed as a ratio = $S_r/100$

γ_d = dry density

G_s = specific gravity of solids

γ_w = unit weight of water

During saturation and consolidation the sample will either swell, compress or remain at constant volume. If the increase in

weight of the sample from its initial as-compacted condition is greater than $V_a \gamma_w$, then the sample has undergone swelling during saturation and consolidation. On the other hand, if the increase in weight is less than $V_a \gamma_w$, then the sample has compressed. When the increase in weight is exactly equal to $V_a \gamma_w$, no volume change has taken place, the water merely replacing the air in the partially saturated as-compacted specimen.

The change in volume $(\Delta V)_f$ of the specimen on account of saturation and consolidation is obtained from the equation for the increase in weight

$$(\Delta W)_f = V_a \gamma_w + (\Delta V)_f \gamma_w$$

Thus

$$(\Delta V)_f = \{(\Delta W)_f - V_a \gamma_w\} / \gamma_w$$

The corresponding change in void ratio is given by

$$(\Delta e)_f = (1 + e_o) (\Delta V)_f / V_o$$

where e_o is the as-compacted void ratio and V_o is the as-compacted volume of the sample. The percent increase in volume is given by $100 (\Delta V)_f / V_o$.

Since no volume changes can take place during undrained shear of a fully saturated specimen, the void ratio of the sample during shear is equal to the void ratio after saturation and consolidation and is given by

$$e_f = e_o + (\Delta e)_f$$

where e_f is the average void ratio of the specimen at failure.

CHAPTER 6

RESULTS

The test results are presented in the form of tables and figures. Some of them are placed in the main body of the text whereas others are placed in the appendices. All the tables and figures are listed in the preliminary pages under the List of Tables and the List of Figures.

The reader is advised to first get acquainted with the location of these tables and figures, the kind of information presented therein, and particularly the notations used as given in the List of Notations. This first should greatly facilitate the subsequent reading and dissemination of the text.

Appendix A contains tabulated test results for each of the triaxial specimens. The data are arranged in the order in which the tests were carried out. Table A1 gives the compaction, saturation and consolidation characteristics, and Tables A2 through A53 give the undrained shearing response. Tables A54 through A57 give the compaction characteristics of specimens compacted at 200, 100, 50, and 0 psi (grouped data), respectively. Table A58 gives the saturation and consolidation characteristics expressed as a percentage volume change, whereas Table A1 gives the corresponding void ratios.

Appendix B gives the hyperbolic stress strain parameters for each of the triaxial test specimens, listed under Table B1.

Appendix C contains figures excluded from the body of the text to make it more readable. Figures C1 and C2 relate to compaction, Figures C3 through C6 relate to volume changes from the as-compacted state to the saturated and consolidated state and Figures C7 through C16 relate to the undrained shearing response.

6.1 Compaction Characteristics

The compaction characteristics of the test shale are summarized in Tables A54 through A57. Included in these tables are the aggregate gradation and moisture content, the molding water added per 500 gm of shale aggregate, the compaction pressure and the as-compacted condition of the specimen represented by its dry density, water content, void ratio and degree of saturation, and also the aggregate density and molding water content.

The bulk aggregate density is obtained from the dry density using the equation

$$\gamma_a = \gamma_d (1 + w_i/100)$$

where γ_a is the bulk aggregate density, γ_d is the dry density, and w_i is the aggregate moisture content. The percent molding water content w_m is equal to

$$w_m = (W_a/500)100\% = (W_a/5)\%$$

where W_a is the weight of water added per 500 gm of shale aggregate. This relationship is included to study the compaction characteristics as a function of the total weight of the aggregate including the

particle moisture, as a function of the water added from an external source.

Table 8 gives the average dry densities for specimens having the same initial gradation, molding water content and compaction pressure. The average dry densities for a given compaction pressure are also included. The compaction pressure is denoted by p_c and $(\gamma_d - 94.9)$ represents the gain in dry density resulting from the kneading compaction pressure used.

Figures C1(a) to C1(d) show plots of dry density vs water content for samples compacted at 200, 100, 50 and 0 psi, respectively. Specimens having the same initial gradation, amount of added water (molding water), and compaction pressure are linked by lines. The differences in the as-compacted moisture content is due to variations in the aggregate moisture content. Figures C2(a) to C2(c) show plots of aggregate density in the compacted specimens vs molding water content for samples compacted at 200, 100 and 50 psi, respectively. The aggregate moisture before adding the molding water is shown within brackets.

Figures 5, 6 and 7 show plots of dry density vs aggregate moisture content for samples compacted at 200, 100, 50 psi, respectively. Linked data points refer to samples having the same initial gradation, molding water content and compaction pressure. Data points which show an increase in dry density with increase in aggregate moisture content, all other conditions being equal, are indicated by the shaded symbols.

Table 8
Average Dry Densities

Gradation	W_a gm	p_c psi	No. of Tests	Average γ_d pcf	$(\gamma_d - 94.9)$ pcf	$\frac{p_c}{(\gamma_d - 94.9)}$ psi/pcf
2	25	200	5	131.9		
"	35	"	2	131.3		
"	40	"	3	132.7		
4	25	"	1	130.4		
"	40	"	1	132.1		
5	40	"	1	134.5		
6	40	"	3	132.4		
Average		200	16	132.2	37.3	5.362
1	25	100	3	130.6		
"	40	"	4	128.0		
3	35	"	1	131.3		
5	25	"	2	125.3		
"	30	"	1	130.3		
"	35	"	1	128.6		
"	40	"	2	131.0		
"	45	"	1	125.1		
6	25	"	4	127.9		
"	35	"	3	129.8		
Average		100	22	128.7	33.8	2.958
5	25	50	2	119.0		
"	30	"	5	121.6		
"	35	"	2	121.7		
5	40	"	3	121.1		
Average		50	12	121.1	26.2	1.908
5	0	0	2	94.9	0	-

FIGURE 5 DRY DENSITY-AGGREGATE MOISTURE
CONTENT AT 200 PSI COMPACTION
PRESSURE

FIGURE 6 DRY DENSITY - AGGREGATE MOISTURE CONTENT AT 100 PSI COMPACTION PRESSURE

FIGURE 7 DRY DENSITY - AGGREGATE MOISTURE
CONTENT AT 50 PSI COMPACTION
PRESSURE

Figures 8, 9 and 10 show various relationships between the average dry density and the compaction pressure, using the data given in Table 8. The upper and lower limits of the dry density for a given compaction pressure are shown in Figure 8 only. A nominal compaction pressure of 1 psi was taken for samples not subject to kneading compaction but packed with the aid of a standard 5.5 lb Proctor hammer, in the plot of dry density vs logarithm of compaction pressure (Figure 9).

6.2 Volume Change Characteristics

The volume change characteristics after saturation and consolidation are shown in Table A58 in Appendix A. Included in this table are the test number, the as-compacted void ratio and degree of saturation, and the percent volume changes (with respect to the initial as-compacted volume) after saturation and consolidation, under effective confining pressures σ'_s and σ'_c , respectively. The numbers in the last column give the initial gradation, the amount of water added W_a in gm/500 gm of shale aggregate, and the kneading compaction pressure p_c in psi.

Some of the specimens were consolidated to the desired effective confining pressure during the saturation phase. For these specimens $\sigma'_s = \sigma'_c$, and $(\Delta V)_s = (\Delta V)_c$. The remaining specimens were saturated under a nominal effective confining pressure and the consolidated under a higher pressure. In these tests $\sigma'_s < \sigma'_c$, and $(\Delta V)_s > (\Delta V)_c$. A negative sign is used to denote decreases in void ratio and volume. The absence of any sign denotes an increase.

Figures C3 through C6 show plots of the percent change in volume from the as-compacted condition to the saturated consolidated condition

FIGURE 8 AVERAGE DRY DENSITY VS
COMPACTION PRESSURE

FIGURE 9 AVERAGE DRY DENSITY VS LOG COMPACTION PRESSURE

FIGURE 10 TRANSFORMED HYPERBOLIC
RELATIONSHIP OF DRY
DENSITY AND COMPACTION
PRESSURE

as a function of the initial as-compacted void ratio. Each plot relates to a constant consolidation pressure (40, 20, 10 psi) but the data points themselves relate to samples compacted with different initial gradations, added water and compaction pressure. In Figure C6, however, the data relating to consolidation pressures of 5, 1, and 0 psi are grouped. Sufficient tests were not conducted at these pressures to observe any trends in volume change over a wide range of as-compacted void ratios, for a given value of the consolidation pressure. Also, the range in consolidation pressure (5 to 0 psi) is too small for any major trends to be observed.

Figure 11 shows contours of percent volume change as a function of the initial void ratio and the consolidation pressure. The critical as-compacted void ratios corresponding to the contour of zero volume change are given in Table 9 (p 101) as a function of the isotropic consolidation pressure. The data points in Figure 11 and Table 9 were obtained by interpolation of the data in Figures C3 through C6.

Some typical consolidation curves are given in Figure 12. The coordinates used are the void ratio and the log elapsed time in minutes.

6.3 Undrained-Shearing Response

The stress-strain responses during undrained shear are given in the Tables A2 to A53 in Appendix A. The results for each test specimen appear on a separate page. The measured and computed variables are shown as a function of the axial strain (ϵ_a). The tables give the change in pore water pressure (Δu), the deviatoric stress ($\sigma_1 - \sigma_3$), the major and minor principal effective stress

- $\triangle$ Contraction (-)
- $\odot$ Zero Volume Change
- $\diamond$ Expansion (+)

FIGURE 11 CONTOURS OF PERCENT VOLUME CHANGE

FIGURE 12 TYPICAL ISOTROPIC CONSOLIDATION CURVES

(σ'_1 and σ'_3) and their ratio (σ'_1/σ'_3), Skempton's pore pressure parameter (A), the inverse of the secant shear modulus ($\epsilon/(\sigma_1 - \sigma_3)$), the shear stress $q = 1/2 (\sigma_1 - \sigma_3)$ and the mean effective normal stress $p' = 1/2 (\sigma'_1 + \sigma'_3)$, and their ratio q/p' . Also included are the octahedral shear stress (τ_{oct}) and effective octahedral normal stress σ'_{oct} defined as

$$\tau_{oct} = \frac{\sqrt{2}}{3} (\sigma_1 - \sigma_3)$$

$$\sigma'_{oct} = \frac{1}{3} (\sigma'_1 + 2 \sigma'_3)$$

assuming that $\sigma'_2 = \sigma'_3$ in an axisymmetric compression test. The constant cell pressure σ_3 , the initial back pressure u_b , the preshear consolidation pressure $\sigma'_c = \sigma_3 - u_b$, and the strain rate or more correctly the axial deformation in inches per minute are given below each table.

Figures 13 through 16 and C7 through C16 show plots of the test results. Each figure relates to specimens having the same compaction history (constant initial gradation, molding water content, and compaction pressure). The consolidation pressure, however, is a controlled variable for each series of tests. The compaction history is included in the title for each figure. For example, Figure 13 relates to samples having an initial gradation No. 2, added water 25 gm/500 gm of aggregate, kneading compacted under a pressure of 200 psi. This set of samples is denoted by Series (2-25-200).

For each series of tests the corresponding figure is comprised of five separate plots (a) to (e), as follows

- (a) Deviatoric Stress vs Axial Strain
- (b) Pore Pressure Change vs Axial Strain
- (c) A Parameter vs Axial Strain
- (d) Effective Stress Paths
- (e) Void Ratio vs Log Stresses

Included in plot (d) is the failure envelope in q - p' space and the computed effective stress strength parameters c' and ϕ' evaluated at $(\sigma_1 - \sigma_3)_{\max}$. The data points shown in plot (e) are the void ratio vs the log compaction pressure (p_c) which is a total stress, the log consolidation pressure (σ'_c), the log shear stress at failure (q_f), and the log mean effective normal stress at failure (p'_f).

The effective stress strength parameters c' and ϕ' were obtained from the q - p' effective stress path diagram using the linear equation

$$\begin{aligned}
 q_f &= c' \cos \phi' + p'_f \sin \phi' \\
 &= \alpha + \beta p'_f
 \end{aligned}$$

where $q_f = 1/2 (\sigma_1 - \sigma_3)_f$

$$p'_f = 1/2 (\sigma'_1 + \sigma'_3)_f$$

The slope (β) of the straight line is equal to $\sin \phi'$, and its intercept (α) with the q -axis is equal to $c' \cos \phi'$.

The failure condition is more commonly represented as a limiting shear stress - effective normal stress plot in which the failure envelope is a line tangential to the Mohr circles representing σ'_1 and σ'_3 at failure. In this case the general equation of the failure

envelope is referred to as the Mohr-Coulomb failure criterion and may be written as

$$\tau_{ff} = c' + \sigma'_{ff} \tan \phi'$$

where τ_{ff} is the shear stress on the failure plane at failure, σ'_{ff} is the effective normal stress on the failure plane at failure, and c' and ϕ' are the effective stress strength parameters.

Figures 17 through 20 show typical hyperbolic stress-strain plots for the consolidated undrained triaxial shear tests, along with the corresponding plots of deviator stress vs axial strain. The hyperbolic equation is expressed as (Kondner, 1963)

$$\frac{\varepsilon}{(\sigma_1 - \sigma_3)} = a + b \varepsilon$$

where a represents the inverse of the initial tangent modulus, and b represents the inverse of the deviator stress as the strain increases to very large values.

Table B1 in Appendix B gives the values of the hyperbolic stress-strain parameters a and b for each test specimen, along with their reciprocals

$$1/a = E_i = \text{Initial Tangent Modulus}$$

$$1/b = (\sigma_1 - \sigma_3)_{ult} = \text{Ultimate Deviator Stress}$$

The failure ratio R_f which is defined as

$$R_f = \frac{(\sigma_1 - \sigma_3)_{\max}}{(\sigma_1 - \sigma_3)_{\text{ult}}}$$

is also included, along with the preshear consolidation pressure, and the average void ratio of the specimen during shear.

For some specimens the hyperbolic stress-strain relationship is not suitable, particularly for those that display strain softening behavior after a relatively small strain, e.g., Figure 17. In such cases the fitted straight line tends to have a zero intercept ($a = 0$), and the initial tangent modulus tends to infinity. As this result is clearly due to the adoption of an inappropriate stress-strain relationship, the values of a and its inverse E_1 are not shown in Table B1.

FIGURE 13 $\overline{\text{CIU}}$ RESULTS, SERIES 2-25-200

(c) A Parameter vs. Axial Strain

FIGURE 13, CONT'D

(d) Effective Stress Paths

FIGURE 13, CONT'D

(e) Void Ratio vs Log Stresses

FIGURE 13, CONT'D

(a) Deviator Stress vs. Axial Strain

(b) Pore Pressure Change vs. Axial Strain

FIGURE 14 $\overline{CIU}$ RESULTS, SERIES 6-25-100

(c) A Parameter vs. Axial Strain

FIGURE 14, CONT'D

(d) Effective Stress Paths

FIGURE 14 , CONT'D

(e) Void Ratio vs. Log Stresses

FIGURE 14, CONT'D

(a) Deviator Stress vs Axial Strain

(b) Pore Pressure Change vs Axial Strain

(c) A Parameter vs Axial Strain

FIGURE 15 $\overline{\text{CIU}}$ RESULTS, SERIES 5-30-50

(d) Effective Stress Paths

FIGURE 15, CONT'D

(e) Void Ratio vs Log Stresses

FIGURE 15, CONT'D

FIGURE 16 $\overline{\text{CIU}}$ RESULTS, SERIES 5-0-0

FIGURE 16, CONT'D

FIGURE 17 HYPERBOLIC STRESS-STRAIN RELATIONSHIP (SPECIMEN 32)

FIGURE 18 HYPERBOLIC STRESS-STRAIN
RELATIONSHIP (SPECIMEN 26)

FIGURE 19 HYPERBOLIC STRESS-STRAIN
RELATIONSHIP (SPECIMEN 28)

FIGURE 20 HYPERBOLIC STRESS-STRAIN RELATIONSHIP (SPECIMEN 29)

CHAPTER 7

DISCUSSION OF RESULTS

7.1 Compaction Characteristics

The shale aggregates used in this study had initial moisture contents ranging from 4.00% to 1.13% as indicated in Table A1. This variation may be attributed to partial dehydration during storage. Thus even when the same amount of moisture is added from an external source to compact shale aggregates having the same weight, the as-compacted moisture content of the specimen varies.

Plots of dry density vs. moisture content for specimens compacted at 200 psi are shown in Figure C1(a). The dry densities range from 130.4 to 134.5 pcf over the moisture content range of 6.28% to 10.4%. Plots having the same initial gradation and added moisture are linked by straight lines.

Specimens compacted at 100 psi have dry densities ranging from 121.8 to 131.9 pcf over the moisture content range of 6.28% to 12.74% {(Figure C1(b))}. Points joined by straight lines relate to the same gradation and added moisture.

Specimens compacted at 50 psi have dry densities ranging from 118.7 to 123.4 pcf, over the moisture content range of 6.07% to 9.53% {(Figure C1(c))}.

Two specimens (33 and 32) were prepared by placing shale particles in a mold and subjecting them to very light tamps with a 5.5 lb. Proctor hammer without addition of any water. The dry density was found to be 94.9 pcf at 1.65% moisture content {(Figure C1(d))}.

Several theories have been presented for the mechanics of compaction of fine grained soils. These theories have been summarized in Wahls et al. (1966) and Hilf (1975). In chronological order, they are:

1. Proctor's (1933) capillarity and lubrication theory,
2. Hogentogler's (1936) viscous water theory,
3. Lambe's (1958) structural theory, and
4. Olson's (1963) effective stress theory.

These theories are by no means mutually exclusive and each theory is an attempt by each writer to rationally explain the characteristic peak of the dry density-moisture content curve (Proctor curve).

The first three theories directly involve the interaction of fine grained soils and water. Olson's effective stress theory is based on the principle that the shearing strength of the interparticle contact surfaces must be exceeded before the density of a soil can be increased by movement of particles. None of these theories consider the effects of strength of aggregates, fracture of aggregates, slaking of aggregates, etc. and hence they cannot fully explain the compaction characteristics of shale aggregates.

According to Hilf (1975), the peaked curve relationship between dry density and water content (Proctor curve) that is characteristic of all cohesive fine grained soils is ill defined or nonexistent for cohesionless soils such as clean sands and gravels. In these soils,

high densities are obtained when the soil is either in a completely dry or in a completely saturated condition. At intermediate moisture contents, the capillary stresses tend to resist the compactive effort and lower densities are obtained. For soils where the Proctor curve concept is not applicable, the normally used compaction criterion is relative density, introduced by Terzaghi (1925).

Bailey (1976) found that the point load strength of shale aggregates decreased with increasing aggregate moisture content. Thus aggregate breakage during compaction would tend to be higher with increasing aggregate moisture content. However, he found that the sample moisture content was not significantly related to either the wet or dry density or to any of the indices of aggregate degradation. He attributed this apparent anomaly to the rather limited range in aggregate moisture contents.

Some of the results obtained in this study show a tendency for the dry density to increase with increasing aggregate moisture content for constant conditions of initial aggregate gradation, added moisture, and compaction pressure. The data points which show this tendency are shown shaded in the plots of dry density vs. aggregate moisture content (Figures 5 to 7).

Consider the data for gradations 2 and 6 compacted at 200 psi (Table A54 and Figure 5a). For added water, $W_a = 25$ gm/500 gm, the dry density increases somewhat linearly with aggregate moisture content. The same tendency is also observed though not so convincingly, for $W_a = 35$ and 40 gm/500 gm. Comparing points at more or less the same aggregate moisture content, it is seen that the dry density which

increases with the amount of added water W_a , is probably due to increased slaking.

Similar trends for samples compacted at 100 psi are shown in Figure 6 using the tabulated data in Table A55. For example, for gradation No. 6, the dry density increases with aggregate moisture content for $W_a = 25$ and 35 gm/500 gm of aggregate. The same can be said for gradation No. 5 compacted with $W_a = 25$ and 40 gm per 500 gm of shale aggregates. Comparing aggregates at essentially the same moisture content it is seen that the dry density increases with increasing amounts of added water. This is probably due to increased slaking.

In the case of samples compacted at 50 psi, the influence of particle moisture is less evident (Figure 7).

When the amount of added water is increased, increased slaking of the shale aggregates takes place. The additional amount of water increases the degree of saturation and the air paths in the specimen become occluded. When the air voids become completely discontinuous, the air permeability of the soil drops to zero and no further densification is possible, as explained earlier by Olson (1963), Barden and Sides (1970) and Hilf (1975).

The compaction characteristics of shales are controlled by: (1) the strength of the aggregates, which decreases with increasing moisture content; (2) the amount of added water, which controls the slaking that takes place during mixing, softening of contacts, and the occlusion of air; (3) the size and gradation of the aggregates, which influence the aggregate contact forces; and (4) the compaction pressure, which causes aggregate degradation.

The influence of compaction pressure on the as-compacted dry density is illustrated in Figure 8 for the experimental range of gradations, aggregate moisture contents and added water. The dry unit weight varies more or less linearly with the logarithm of the compaction pressure as shown in Figure 9.

The relationship between the average dry density γ_d and the compaction pressure p_c may also be expressed in hyperbolic form as

$$(\gamma_d - 94.9) = \frac{p_c}{a + b p_c}$$

where a and b are constants and 94.9 is the dry density at zero compaction pressure. The constants a and b are determined by fitting a straight line to the data points (Figure 10).

$$\frac{p_c}{(\gamma_d - 94.9)} = a + b p_c$$

The inverse of a is the initial rate of change of dry density with compaction pressure, and the inverse of b is the asymptotic value of the increase in dry density as the compaction pressure assumes very large values.

From the plot of $p_c / (\gamma_d - 94.9)$ vs. p_c , the constants a , b and their inverse values are (Figure 10):

$$a = 0.75 \text{ psi/pcf}$$

$$b = 0.023 \text{ pcf}^{-1}$$

$$1/a = \lim_{p_c \rightarrow 0} (\gamma_d - 94.9)/p_c = 1.33 \text{ pcf/psi}$$

$$p_c \rightarrow 0$$

$$1/b = \lim_{p_c \rightarrow \infty} (\gamma_d - 94.9) = 43.48 \text{ pcf}$$

$$p_c \rightarrow \infty$$

Thus the asymptotic value of the dry density for large values of the compaction pressure is equal to

$$\begin{aligned}(\gamma_d)_{\max} &= 43.5 + 94.9 \\ &= 138.4 \text{ pcf.}\end{aligned}$$

which may be compared with the insitu dry unit weight of 145.0 pcf shown in Table 3.

7.2 Consolidation Characteristics

The performance of a highway embankment will depend not only upon the shear strength of the compacted materials but also upon its volume change characteristics.

Depending upon the construction specification, the embankment material is compacted to some initial void ratio and degree of saturation. With increasing height of the embankment, during the construction phase, the previously compacted material is subject to increasing confining pressures. Under the action of these confining pressures, the compacted soil could undergo changes in void ratio and degree of saturation, and a redistribution of the water content. Some loss of water may take place due to evaporation, and an increase of water content could take place by percolation of rain water.

At a later stage, after completion of the embankment, saturation may occur with the confining pressure acting on the materials. This saturation may occur over a period of several years, depending upon the availability of water from an external source, the permeability of the embankment material, its affinity for water, and the drainage provided.

The volume changes that take place in compacted New Providence shale are examined hereafter. It will be shown that the volume change on saturation depends to a large extent on the initial void ratio and the confining pressure. Collapse of the shale aggregates occurs in specimens having high initial void ratios, and swelling occurs in specimens having low void ratios.

The relationships between the as-compacted void ratio and the percent volume change that takes place on saturation under a given consolidation pressure is shown in Figures C3 to C6. For a given consolidation pressure, the percent increase in volume increases with decreasing initial void ratio and the percent decrease in volume increases with increasing initial void ratio.

For a consolidation pressure of 40 psi, the critical as-compacted void ratio is approximately equal to 0.36 (Figure C3), i.e., when the void ratio is greater than this value the sample decreases in volume, and when it is less than this value the sample increases in volume.

Similar critical void ratios can be established by estimating, from Figures C4 to C6, the void ratios corresponding to zero volume change for each consolidation pressure. These conditions are shown in Table 9.

Table 9

Critical Conditions for Zero Volume Change After
Saturation and Consolidation

Consolidation Pressure psi	As-Compacted Void Ratio
40	0.36
20	0.37
10	0.40
5	0.44

Critical void ratios for consolidation pressures less than 5 psi are not included due to insufficient data.

The effect of molding water content on the volume change characteristics is not clearly defined.

The current practice in embankment construction is to compact the soil to the same void ratio irrespective of its final depth in the embankment. For any given as-compacted void ratio, there is a critical depth for which saturation under the confining pressure associated with this depth will produce no volume changes. Samples of soil above this critical depth will swell, and samples below the critical depth will compress. The net effect could result in a change in the finished grade of the embankment with progress of time.

From Figure C6 it is seen that an as-compacted void ratio of 0.30 to 0.35 can result in swelling from 4 to 9%, for consolidation pressures less than 5 psi. Taking 1 psi as equivalent to 1 foot of soil, the swelling could be a problem at shallow depths in the case of saturation of densely compacted New Providence shale.

In Figure 11 are shown contours of equal volume change. This diagram is especially useful for estimating the volume change that would take place on saturation, as a function of the initial ratio and the consolidation pressure.

7.3 Stress-Strain Response

The variation of the deviator stress with axial strain displays many features which are dependent on the compaction pressure, the consolidated void ratio and the consolidation pressure.

7.3.1 High Density Specimens

Specimens compacted at 200 and 100 psi and consolidated up to 40 psi, have stress-strain curves which do not show a peaked deviator stress within the axial strain range of 0 to 20%. The curves have a hyperbolic shape and increase with increasing strain. The initial tangent modulus increases with increasing consolidation pressure and hence confining pressure. The maximum deviator stress also varies in a similar fashion. A very noticeable feature is that the maximum deviator stress is not linearly proportional to the consolidation pressure. The response of the specimens is typical of overconsolidated behavior.

For example, consider the series of tests in which the specimens were compacted at 200 psi, with 25 gm of added water per 500 gm of shale aggregate having gradation No. 2 {(Figure 13(a))}. At a consolidation pressure of 1 psi, specimen No. 19 attained a deviator stress of 19 psi at an axial strain of 20%. Specimen No. 46 sheared after consolidation to 40 psi reached a deviator stress of 56 psi at

20% strain. Thus an increase in the consolidation pressure by a factor of 40 resulted in an increase of the deviator stress by a factor of a little over 2.9. When the maximum deviator stress is plotted versus consolidation pressure, as in Figure 38 on page 156, the data points are distributed about a curved line.

The rate of development of shear strength with axial strain increases with increasing consolidation pressure. This is due to the increase in the initial tangent modulus with increasing consolidation pressure. However, as the axial strain increases, the further increase in strength decreases with increasing consolidation pressure.

For example, and again referring to Figure 13, at 2% axial strain, specimen No. 19 consolidated to 1 psi reached 58% of its maximum strength, whereas specimen No. 46 consolidated to 40 psi attained 77% of its maximum strength.

As an example of the behavior of specimens compacted at 100 psi, consider the series of tests in which the specimens were compacted with 25 gm of added water per 500 gm of shale aggregates having a gradation No. 6 {Figure 14(a)}. In this series, specimen No. 8 was sheared after consolidation at zero psi and it attained a maximum deviator stress of 9 psi. At 40 psi consolidation pressure, specimen No. 45 attained a maximum deviator stress of 40 psi. The relationship among maximum deviator stress and consolidation pressure for this series of tests is shown in Figure 39. It is seen that the deviator stress varies non-linearly with consolidation pressure, as was the case for specimens compacted at 200 psi.

The mobilization of shear strength with axial strain follows a similar pattern. The shear strength increases continuously with axial strain. The percent strength reached at any strain increases with increasing confining pressure. The form of the stress-strain curves is very similar to those obtained with specimens compacted to 200 psi.

7.3.2 Medium Density Specimens

The stress strain curves of specimens compacted at 50 psi, are markedly different from those compacted at 200 and 100 psi. The deviator stress increases up to approximately its maximum value at about 2% axial strain and the increase in stress beyond this strain up to 20% strain is nominal, as seen for example in Figure 15(a).

7.3.3 Low Density Specimens

The stress-strain curves of specimens No. 32 and No. 33 {Figure 16(a)} are particularly interesting. These specimens were formed in a mold by light tamping of shale aggregate (gradation No. 5) without addition of any water. They were subsequently saturated, consolidated and sheared undrained. The maximum deviator stress was reached at 1 to 2% axial strain. Thereafter, the deviator stress decreased appreciably with further straining. In the case of specimen No. 32, consolidated to 20 psi, the deviator stress reached a constant value at 20% strain. Specimen No. 33 consolidated to 10 psi, on the other hand, had a stress-strain curve which was still decreasing at 20% axial strain.

7.3.4 Controlling Factors

The factors which determine the stress-strain behavior of soils have been discussed by many researchers. The textbook by Lambe and Whitman (1969) is particularly helpful in explaining the test results obtained in this study.

The stress strain curves of even normally consolidated specimens of shale aggregate do not have a unique normalized relationship, see Figure 16. These specimens (Nos. 32 and 33) had an initial void ratio of 0.829. After saturation and consolidation to 20 and 10 psi, the final void ratios were found to be 0.583 and 0.595, respectively. The peak undrained strengths were found to be 10.8 and 8.7 psi, respectively. If these curves are normalized with regard to the confining stress, the peak normalized stress decreases as the confining pressure increases, accompanied by a slight increase in the strain at which this peak occurs. This is typical of weak aggregates, with a tendency for crushing and breaking during shear under sufficiently high stress levels.

In the case of normally consolidated remolded sand composed of quartz particles, and fine grained soils, the normalized stress-strain curves are almost identical for different confining pressures within the range of pressures less than those required to cause crushing and breaking of particles. In the case of sands composed of quartz these pressures are around 500 psi (Lambe and Whitman, 1969) and exceedingly higher pressures would be required to cause fracturing of fine grained soils.

Interlocking also contributes to the overall shearing resistance, and the degree of interlocking usually increases with decreasing void ratio. The interlocking, however, can decrease as the confining pressure increases, particularly when particles become flattened at contact points, sharp corners are sheared off, and particles break as in the case of shale aggregates. Lambe and Whitman (1969) have pointed out that even though these actions result in a denser specimen, they make it easier for shear deformations to occur.

The effect of interlocking and material crushing on the shear strength of rock discontinuities has been discussed by Hoek and Bray (1974), and the concepts presented by them are also applicable to particulate media. Figure 21 shows the simplified relationships between shear strength and normal stress for rough surfaces.

The rock sample is considered to contain a set of teeth as shown in Figure 21. If this specimen is subjected to shear and normal loads as in conventional shear testing, shear movement can only take place if the projections ride over one another or if they are sheared through. When the projections ride over one another, initial movement is no longer parallel to the shear stress τ , but it takes place along a line inclined at an angle i to the direction of τ , where i is the angle of incidence of the projections with the direction of shear displacement. During this phase the overall volume of the specimen increases, and this phenomenon is usually termed dilatation.

The shear stress along the inclined discontinuity is given by

$$\tau_m = \tau \cos i - \sigma \sin i \quad 7.3(1)$$

FIGURE 21 SIMPLIFIED RELATIONSHIPS BETWEEN SHEAR STRENGTH AND NORMAL STRESS FOR ROUGH SURFACES. (AFTER HOEK AND BRAY, 1974)

Similarly, the normal stress perpendicular to the line of initial movement is given by

$$\sigma_m = \sigma \cos i + \tau \sin i \quad 7.3(2)$$

If ϕ is the basic friction angle for the inclined surface of sliding, and it is assumed that the surfaces have no cohesive strength, the relationship between the shear stress τ_m required to initiate movement and the normal stress σ_m is

$$\tau_m = \sigma_m \tan \phi \quad 7.3(3)$$

Substituting for τ_m and σ_m and expressing $\tan \phi$ as $\sin \phi / \cos \phi$ gives

$$\begin{aligned} \cos \phi (\tau \cos i - \sigma \sin i) &= \sin \phi (\sigma \cos i + \tau \sin i) \\ \tau (\cos \phi \cos i - \sin \phi \sin i) &= \sigma (\sin \phi \cos i + \cos \phi \sin i) \\ \tau \cos (\phi + i) &= \sigma \sin (\phi + i) \\ \tau &= \sigma \tan (\phi + i). \end{aligned} \quad 7.3(4)$$

Patton (1966) used this relationship in stability analyses of unstable limestone slopes. The average value of the angle of discontinuities i was determined by measurements from photographs of bedding planes, and values of the friction angle ϕ came from laboratory tests on smooth limestone surfaces.

Barton (1971) suggested that the relationship presented by equation 7.3(4) is too simple because it assumes that the average angle i remains constant throughout the range of normal stresses under which dilatation can take place. He suggests that the effective value of i depends upon the magnitude of the normal stress σ . At very low

normal stresses, smaller and steeper sided projections control movement. As the normal stress increases, these small projections are broken off and the gentler undulations of the first order irregularities control the behavior. Barton's (1971) empirical relationship for i is presented by Hoek and Bray (1974) as follows

$$i = 20 \log (\sigma_c / \sigma) \quad 7.3(5)$$

where σ_c is the uniaxial compressive strength of the wall material of the discontinuity and σ is the normal stress in a direction perpendicular to the direction of applied shear stress. According to this equation, $i = 0$, when $\sigma = \sigma_c$. Barton (1971) derived this equation by performing experiments on model material which had been fractured in tension, and from shear test data on tension joints in granite. The assumption used in deriving this relationship was that the basic friction angle of the wall material $\phi = 30^\circ$.

Barton's equation 7.3(5) cannot be directly applied to all materials, as the constant was determined for granite. Its form however is particularly interesting as it offers an explanation of the curvature in the Mohr Coulomb failure envelope at increasing stresses in the case of materials which break down during shear.

The magnitude of the back pressure applied to the pore water will influence the stress-strain response of specimens which tend to dilate during shear. This phenomenon may be explained as follows.

In an undrained test the pore water pressures vary, and if the boundary confining stress (cell pressure) is kept constant the effective confining stress varies. When the change in pore water

pressure is positive, the effective confining stress is decreased, and when it is negative, the effective confining stress is increased, according to the Terzaghi's equation for effective stress

$$\sigma'_3 = \sigma_3 - u \quad 7.3(6)$$

where σ_3 is the total stress, u is the pore water pressure and σ'_3 is the effective stress.

Positive pore water pressures are developed when the sample tends to compress, and the increase is not dependent on the magnitude of the back pressure applied to the pore water. On the other hand, negative pore water pressures are developed when the sample tends to expand, and the net decrease in pore water pressure is dependent on the magnitude of the back pressure used to saturate the samples.

Release of dissolved air begins to take place when the pressure in the pore water approaches - 1 atmospheres (- 14.5 psi). When tests are conducted at high back pressures, as done in this study, release of air is artificially prevented, and there is no lower bound for the negative change in pore water pressure. Consequently, there is no upper bound for the effective confining pressure, which keeps on increasing, to counteract the dilatation tendency of dense specimens. This increase in effective confining stress is accompanied by an increase in shear stress which continues to rise to very large strains.

7.3.5 Drained vs Undrained Behavior

The stress-strain curves would have been totally different if the specimens were subjected to consolidated drained shear. In a drained

test the effective confining pressure remains constant and volume changes take place. Consolidated drained tests were not carried out in this study and it is not appropriate to compare the results of the consolidated undrained tests with consolidated drained behavior of soils in general.

The drained and undrained behavior of the same soils are related through the principle of effective stress. Volume changes in a drained test are associated with pore water pressure changes in an undrained test. Drained and undrained strengths usually differ and so do the strains at failure (maximum deviator stress). The Mohr Coulomb failure envelope, however, is not affected by the type of test. For a fuller discussion of the relationship between drained and undrained behavior of saturated soils see Lambe and Whitman (1969), Ladd (1964), Lee and Seed (1967), Seed and Lee (1967), and Henkel (1956), Bjerrum and Simons (1960).

7.4 Pore Water Pressure Response

In an undrained test with constant cell pressure, the pore water pressure changes during shear. The variation of pore water pressure with axial strain is a function of the compaction history and the consolidation pressure.

7.4.1 Variation with Axial Strain

Dense specimens, Figures 13(a) and 14(a), sheared under low confining pressures display excess pore water pressures which are initially positive and then decrease or become increasingly negative at axial strains exceeding 2%. The positive increase in excess pore

water pressure increases with increasing consolidation pressure and the development of negative pore pressure changes increases with decreasing consolidation pressure. These pore pressure changes are typical of overconsolidated specimens which tend to compress at low strains and then tend to dilate at larger strains (Henkel 1956).

Medium density specimens, Figure 15(b), such as those compacted at 50 psi, display excess pore water pressures which are always positive up to 20% strain. The maximum excess pore water pressure is reached at low strains of around 2% and thereafter no significant changes in excess pore water pressures take place. These specimens have therefore reached their critical states (Schofield and Wroth, 1968).

In the case of low density specimens No. 33 and No. 32, Figure 16(b), which were not subjected to kneading compaction but were formed in a mould by light tamping, the positive excess in pore water pressure is equal to the applied deviator stress at axial strains equal to 2% and 6% for consolidation pressures of 20 and 10 psi, respectively. In the case of specimen No. 32, the excess pore water pressure and the maximum deviator stress are equal at 2% axial strain. The behavior of this specimen at peak deviator stress is thus similar to that of a normally consolidated remolded clay wherein the Skempton (1954) parameter A_f at peak deviator stress is very nearly equal to unity (Henkel, 1956).

When these specimens are strained further, the excess pore water pressure exceeds the deviator stress and a pore pressure parameter $A = 2.2$ at $\sigma'_c = 20$ psi was observed at 20% strain. An A-parameter

greater than unity could be associated with the degradation of the shale aggregate during shear. The degradation that takes place increases with increasing strain and there is a corresponding increase in the pore pressure parameter A with strain.

Specimen No. 32 was normally consolidated to 20 psi and had a pore pressure parameter A which was approximately twice that for specimen No. 32 normally consolidated to 10 psi when compared at equal axial strains as shown in Figure 16(c). This behavior indicates that the degradation of shale depends not only on the level of strain but also on the level of normal stress.

The shear strains induced during isotropic consolidation are negligible and in fact equal to zero for an isotropic and homogeneous material. The overall arrangement of the shale aggregate, namely its macroscopic structure, is not likely to be very different at different consolidation pressures. Any difference in structure would be restricted to the number of contacts, and the area of such contacts would increase with increasing pressure and decreasing compressive strength of the material (Yong and Warkentin, 1975).

When the specimen is subjected to shear strains after isotropic consolidation, the macroscopic structure changes appreciably as particles rotate and slide over each other. Further, in the case of a mechanically degradable material such as shale, particle breakdown by shearing takes place. The degree of particle breakage increases with increasing levels of stress.

Thus when two isotropically consolidated specimens having the same structure are in equilibrium under different consolidation

pressures, the specimen under the higher consolidation pressure is potentially more unstable. As a result, the potential for transferring some of the effective consolidation pressure already acting on the specimen to the pore water (during undrained shear) increases with increasing consolidation pressure. A pore pressure parameter A greater than 1 unity is associated with a loose structure which collapses upon load application (Lambe & Whitman 1969).

7.4.2 Variation with Compaction History and Consolidation Pressure

The effect of compaction history on the pore pressure response may be compared for samples having the same consolidation pressure. Constant compaction history means constant gradation, molding water and compaction pressure but with slight variations in the aggregate moisture contents. The test results of such samples are shown by a single symbol and different symbols are used to distinguish specimens having different compaction histories in Figures 22 through 27.

The following discussion relates to the maximum and failure pore pressure changes.

1. The maximum excess pore pressure $(\Delta u)_{\max}$ increases linearly with consolidation pressure (Figures 22 through 25) and is only slightly influenced by the compaction history. The following equations, which are applicable for $0 < \sigma'_c < 40$ psi,

$$(\Delta u)_{\max} = 0.875 \sigma'_c \quad (p_c = 0 \text{ psi})$$

$$(\Delta u)_{\max} = 0.625 \sigma'_c \quad (50 < p_c < 200 \text{ psi})$$

represent the average variation of $(\Delta u)_{\max}$ with σ'_c for specimens having different compaction histories. The compaction pressure in terms of total stress is denoted by p_c .

2. The pore pressure changes at maximum deviator stress, $(\Delta u)_f$ are both positive and negative. The magnitude of the positive pore pressures increases with increasing consolidation pressure. The absolute value of the negative pore pressures decreases with increasing consolidation pressure. These pore pressure changes are designated as $(\Delta u)_f$. Their variation with σ'_c may be represented by an equation of the form.

$$(\Delta u)_f = -a + b \sigma'_c \quad (5 < \sigma'_c < 40 \text{ psi})$$

where a and b are positive numbers. The values of a and b are greatly influenced by the compaction history of the specimens.

3. The magnitudes of the pore pressure change $\{(\Delta u)_{\max} - (\Delta u)_f\}$ for specimens compacted at 200 and 100 psi are shown in Figures 26 and 27 as a function of the consolidation pressure. For a given compaction history (represented by a different symbol) the variation of $\{(\Delta u)_{\max} - (\Delta u)_f\}$ with consolidation pressure is relatively small. For a given consolidation pressure the variation of $\{(\Delta u)_{\max} - (\Delta u)_f\}$ with compaction history is very significant. The magnitude of $\{(\Delta u)_{\max} - (\Delta u)_f\}$ increases as the amount of molding water (W_a) is increased.

FIGURE 22 MAXIMUM AND FAILURE PORE PRESSURE CHANGES FOR SAMPLES COMPACTED AT 200 PSI

FIGURE 23 MAXIMUM AND FAILURE PORE PRESSURE CHANGES FOR SAMPLES COMPACTED AT 100 PSI

FIGURE 24 MAXIMUM AND FAILURE PORE PRESSURE CHANGES FOR SAMPLES COMPACTED AT 50 PSI

FIGURE 25 PORE PRESSURE RESPONSE AT $(\sigma_1 - \sigma_3)_{\max}$ AND AT 20% STRAIN VS CONSOLIDATION PRESSURE FOR NORMALLY CONSOLIDATED SPECIMENS NOS. 32 AND 33

Grad - $W_a - p_c$

□ 6-40-200

◇ 2-40-200

△ 2-35-200

○ 2-25-200

FIGURE 26 CHANGE IN PORE PRESSURE
DUE TO DILATATION TENDENCY
FOR $p_c = 200$ PSI

FIGURE 27 CHANGE IN PORE PRESSURE
DUE TO DILATATION TENDENCY
FOR $p_c = 100$ psi

The effects of the compaction history may be examined in terms of its independent components, namely, gradation, molding water and compaction pressure. The effect of these three variables on the parameters a and b in the expression for $(\Delta u)_f$ may be compared by considering one variable at a time holding the other two variables constant. This comparison may be made over the range of consolidation pressures ($0 < \sigma'_c < 40$ psi).

The parameter " a " increases with increasing amounts of molding water. The comparison is made between samples having constant gradation and constant compaction pressure at various consolidation pressures. This indicates that samples compacted with more water tend to dilate more than samples compacted with less water. The reason for this is that degradation during compaction increases with increasing amounts of added water. The degraded particles form a dense matrix which tends to dilate when sheared. The effect of the amount of molding water on the parameter " a " increases with the compaction pressure. This is so because degradation increases with increasing compaction pressure.

When the samples are compacted at 50 psi, the structure is still loose and hence there is no tendency for volume expansion during shear.

7.4.3 Variation of Pore Pressure Parameter ($A_{\max}$) with Axial Strain and Consolidation Pressure

The pore pressure parameter A varies with the axial strain as shown in Figures 13(c) to 16(c). The maximum value of this parameter (Skempton and Bjerrum, 1957) designated $A_{\max}$, is reached at relatively low strains of 0 to 4% during which the consolidated shale

specimen tends to compress. The strain at which $A_{\max}$ occurs increases with increasing consolidation pressure. The increase in $A_{\max}$ with increasing consolidation pressure as shown in Figures 28 to 30 is due to increased degradation.

7.4.4 Variation of Pore Pressure Parameter at Failure

Henkel (1956) showed that for failure conditions the pore pressure parameter

$$A_f = \left(\frac{\Delta u}{\Delta \sigma_1 - \Delta \sigma_3} \right)_f = \left(\frac{\Delta u}{\sigma_1 - \sigma_3} \right)_f$$

is uniquely related to the overconsolidation ratio and that its value depends on the type of soil. He came to this conclusion by testing remolded London and Weald clays with different combinations of maximum past pressures σ'_m and consolidation pressure σ'_c . The overconsolidation ratio OCR is defined as

$$\text{OCR} = \sigma'_m / \sigma'_c$$

When a sample is normally consolidated, $\text{OCR} = 1$, and the A_f values are close to unity, as found earlier by Casagrande and Wilson (1953) and Bjerrum (1954). The relationship between A_f and OCR has been established for London and Weald clays (Henkel, 1956). The values of A_f show a rapid initial drop with increasing overconsolidation ratio and the A_f -value for both clays is close to zero at an $\text{OCR} = 4$. At overconsolidation ratios greater than 4, the A_f -values are negative and the rate of change of A_f at an overconsolidation ratio of 40 is very small.

FIGURE 28 MAXIMUM POSITIVE VALUE OF PORE PRESSURE PARAMETER (A_{max}) VS CONSOLIDATION PRESSURE FOR 200 PSI COMPACTION PRESSURE

FIGURE 29 MAXIMUM POSITIVE VALUE OF PORE PRESSURE PARAMETER (A_{max}) VS CONSOLIDATION PRESSURE FOR 100 PSI COMPACTION PRESSURE

FIGURE 30 MAXIMUM POSITIVE VALUE OF PORE PRESSURE PARAMETER (A_{max}) VS CONSOLIDATION PRESSURE FOR 50 PSI COMPACTION PRESSURE

Whereas in Henkel's tests, the effective past pressure σ'_m was known, the effective past pressures of the compacted samples of shale is not known. Hence it is not possible to plot A_f against the over-consolidation ratio in terms of effective stresses.

We may however study the variation of A_f with the ratio p_c/σ'_c where p_c is the total compaction pressure. The plots of A_f against p_c/σ'_c are shown in Figure 31, and it is seen that a unique relationship does not exist. This shows that the effective prestress induced in the sample during compaction is not uniquely defined by the applied compaction pressure. It also depends on the gradation and moreover on the amount of water added during compaction.

Comparing specimens having the same compaction history, it is seen that the A_f -value increases from negative to positive values with increasing consolidation pressure, Figures 32 to 34. This variation is to be expected because the OCR varies inversely with the consolidation pressure.

Comparing specimens having the same consolidation pressure, it is seen that the A_f value varies with the amount of molding water. Specimens having the same initial gradation and compaction pressure but having different amounts of molding water during compaction, may be compared at the same consolidation pressure. It is seen that the A_f value decreases with increasing amounts of water added during compaction. This tendency is illustrated in Table 10 for samples having gradation No. 2, compacted at 200 psi with added water varying from 25 to 40 gm for 500 gm of shale aggregate, at natural moisture content, and for $0 < \sigma'_c < 40$ psi.

FIGURE 31 VARIATION IN A_f WITH $\log(p_c / \sigma'_c)$

FIGURE 32 PORE PRESSURE PARAMETER AT FAILURE (A_f) VS CONSOLIDATION PRESSURE (σ'_c) FOR 200 PSI COMPACTION PRESSURE

FIGURE 33 PORE PRESSURE PARAMETER AT FAILURE (A_f) VS CONSOLIDATION PRESSURE (σ'_c) FOR 100 PSI COMPACTION PRESSURE

FIGURE 34 PORE PRESSURE PARAMETER AT FAILURE (A_f) VS CONSOLIDATION PRESSURE (σ'_c) FOR 50 PSI COMPACTION PRESSURE

Table 10
Variation of A_f with Molding Water and
Consolidation Pressure

Consolidation Pressure σ'_c psi	Molding Water W_a gm/500 gm			
	25	30	35	40
0	na	na	na	-.300
1	-.298	na	na	na
5	-.289	na	na	na
10	-.162	na	na	na
20	.050	na	-.131	-.263
40	.314	na	-.041	-.152

na - not available

The results show that the prestress induced in the sample during compaction increases with increasing amounts of added water. It also follows that the tendency for dilating during shear increases with increasing amounts of added water used for compaction. This is so because degradation during compaction increases with the amount of water present. The degree of packing increases with increasing degradation and a denser structure results.

When the shale aggregate is thoroughly broken down during compaction, the subsequent degradation that can take place during shear is very much reduced. Table 11 shows the effect of compaction pressure and molding water on the A_f values for samples having a constant gradation (No. 5) and consolidation pressure (20 psi).

Table 11
Variation of A_f with Molding Water and
Compaction Pressure

Moisture (W_a) gm/500 gm ^a	Compaction Pressure p_c (psi)				σ'_c psi
	200	100	50	0	
40	-.288	-.223	.640	na	20
35	na	na	.530	na	20
30	na	0	.450	na	20
25	.300	.300	.730	na	20
0	na	na	na	2.20	20

na - not available

It is to be noted that the influence of added water is not so pronounced at 50 psi compaction pressure. Further, the pore pressure parameter at failure is always positive and hence there is no tendency for volume to increase during shear.

7.5 Effective Stress Paths

The isotropically consolidated undrained effective stress paths in p' , q space shown in Figures 13(d) to 16(d) are extremely diverse. They show, in essence, the relative influences of compaction history and consolidation pressure on the mobilized shear strength and mean effective normal stress.

The interpretation of shear strength parameters via the shape of the stress paths is most promising. Some stress paths show much lower friction angles at maximum stress difference than at maximum principal stress ratio. In other cases, higher friction angles are obtained at

maximum principal stress ratio than at maximum principal stress difference.

7.5.1 High Density Specimens

The stress paths shown in Figures 13(d) are typical of highly overconsolidated clays. These samples were compacted at 200 psi and sheared undrained after consolidation up to 40 psi. A noticeable feature is that the mean effective normal stress increases during the course of the test, and its ultimate value at large strains is much larger than the initial isotropic consolidation pressure acting on the sample at the start of shearing. The ratio of the ultimate mean stress to the consolidation pressure decreases with increasing consolidation pressure, and this relationship is dependent on the compaction history as shown in Figure 35.

Another noticeable feature of these highly overconsolidated stress paths is that the hypothetical drained stress path intersects the undrained stress path at a point which is practically the same as the point of maximum shear strength for samples consolidated to 40 psi. The ratio of the peak deviator stress to the deviator stress corresponding to the point of intersection of the undrained stress path with the hypothetical drained stress path decreases with decreasing consolidation pressure.

The incremental rate of development of shear strength with effective normal stress increases at first and then decreases as the level of normal stress increases. The peak stress failure envelope for samples having a constant compaction history may be determined from

FIGURE 35 p'_{ult} / σ'_c vs σ'_c

a series of tests conducted under different consolidation pressures. In order to do so, we may draw a series of Mohr circles corresponding to points on the effective stress paths as shown in Figure C9(d).

The peak failure envelope is seen to have a distinct curvature even at the relatively low stress levels used in this study. It is to be noted that the Mohr circles which define the curved peak envelope correspond more or less to conditions of maximum stress obliquity $(\sigma'_1/\sigma'_3)_{\max}$ or $(q/p')_{\max}$. If, however, we choose $(\sigma_1 - \sigma_3)_{\max}$ as the failure criterion, it is seen that the failure envelope is practically straight over the experimental range of mean effective normal pressure ($p' = 0$ to 100 psi). Also note that the $(\sigma_1 - \sigma_3)_{\max}$ failure envelope lies below the $(\sigma'_1/\sigma'_3)_{\max}$ failure envelope.

The principal stress ratio (σ'_1/σ'_3) reaches its maximum value at axial strains ranging from 4 to 8% and decreases thereafter. The deviator stress on the other hand approaches its maximum value at much larger strains, usually around 20%. The $(\sigma'_1/\sigma'_3)_{\max}$ failure envelope is therefore applicable for low strain levels and the $(\sigma_1 - \sigma_3)_{\max}$ failure envelope for high strain levels up to 20%.

The residual failure envelope would probably be lower than the $(\sigma_1 - \sigma_3)_{\max}$ failure envelope. It was not possible to determine this envelope for the dense specimens compacted at 200 psi (under discussion), as the triaxial test is not suitable for straining cylindrical specimens beyond 20%. Non uniform conditions of stress and strain become increasingly important at large strains. The area correction for a constant volume (undrained) test

$$A = A_0 / (1 - \epsilon)$$

underestimates the area at any strain when bulging takes place, and overestimates the area when a slip plane develops.

The stress paths for samples compacted at 100 psi (Figure 14(d)) are similar to those obtained for the samples compacted at 200 psi. However, there are some differences in regard to magnitude. For instance, the ratio of ultimate mean effective stress to consolidation pressure (p'_{ult}/σ'_c) decreases with decreasing compaction pressure, when compared at equal consolidation pressures, as shown in Figure 35.

The test results of sample No. 49 shown in Figure C13 are not similar to other samples compacted at 100 psi. The compacted void ratio of this sample was 0.424 whereas the average void ratio of the other samples compacted at 100 psi was around 0.34. The kneading compactor was probably not functioning at the correct pressure during the compaction of this specimen or the compaction pressure used was incorrectly recorded. A void ratio of 0.424 is more typical of samples compacted at 50 psi. Its behavior is also similar to samples compacted under a pressure of 50 psi.

7.5.2 Medium Density Specimens

The undrained effective stress paths of samples compacted at 50 psi are practically vertical, Figure 15(d). The variation in the mean effective stress is very small. As a result, the ratio of the ultimate mean effective stress to the consolidation pressure is very close to unity over the range of consolidation pressure of 10 to 40 psi, as shown in Figure 35.

7.5.3 Low Density Specimens

The stress path of samples not subject to any compaction gives a pretty good indication as to what could happen when degradable shale aggregate is subject to shear stresses. The stress path of sample No. 32 (void ratio 0.583, consolidation pressure 20 psi) shown in Figure 16(d) is very similar to the stress path of loose saturated Ham river sand with initial porosity 45.3% (void ratio 0.818) consolidated to 990 psi and then sheared undrained (Bishop, Webb and Skinner, 1965). It is also similar to the stress path of the consolidated undrained test on saturated Banding sand with initial porosity of 45.5% (void ratio 0.833) consolidated to 57.7 psi (Castro, 1969). These stress paths are presented by Bishop (1971). The normalized plots are compared in Figure 36. The dependence of shear strength not only on the consolidation pressure but also on the void ratio is well illustrated by the shape of these stress paths.

The stress paths for the uncompacted specimens, as in Figure 16(d), show that the maximum stress ratio is reached at large strains. The maximum stress difference is reached at lower strains. The residual friction angle is therefore greater than the peak stress friction angle whereas the reverse was observed for the dense samples.

A feature of many stress paths is that in some regions in p' , q space the shear stress increases with a corresponding increase in the normal pressure. In other regions the shear stress increases despite a decrease in the normal pressure. A mechanistic interpretation of such behavior in undrained shear has been made by Yong and Vey (1962). They define the friction parameter ϕ' as that property in the shear

<u>Sym</u>	<u>Material</u>	ϕ' at $(\sigma_1 - \sigma_3)_{\max}$	<u>Source</u>
$\triangle$	Loose Sand	16.1°	Castro (1969)
$\square$	Loose Sand	21.3°	Bishop etal (1935)
$\diamond$	Loose Shale	18.2°	Sample No. 32

FIGURE 36 COMPARISON OF STRESS PATHS OF
LOOSE SHALE AND LOOSE SAND

stress versus normal stress region where the shear stress increases with effective normal stress. The cohesion parameter c' is defined as the parameter in the same region where the shear stress increases despite a decrease in effective normal stress.

Varying shapes of stress loci are a feature of undrained tests and they are quite different from the stress path in a drained test, where the shear stress q increases linearly with increase in the normal stress p' . The mechanism of cohesive resistance and of frictional resistance during undrained and drained shear would therefore seem to be somewhat different.

7.6 Choice of Failure Criterion

The criterion that should be used to define strength parameters for use in design is one of the questions put to discussion by Leonards (1976) at the Seventh Ohio River Valley Soils Seminar on Shales and Mine Wastes. He listed the following possibilities which have been used over the years.

1. $(\sigma_1 - \sigma_3)_{\max}$ - peak deviator stress
2. $(\sigma'_1/\sigma'_3)_{\max}$ - maximum obliquity
3. Specified limiting strain (say 15%)
4. Proportional limit or yield point, as defined by the stress-strain curve.

The above criteria are only applicable for conditions leading up to and including failure. There are, however, some situations where the post failure behavior is of concern, particularly in regard to progressive failure, e.g., Skempton (1964) and Bjerrum (1967). In such situations the ultimate or residual strength and the corresponding strength parameters also need to be determined. The shape of the

stress-strain curve will ultimately determine the factor of safety against such failure. We may therefore add two other possibilities for selection of a failure criterion

5. $(\sigma_1 - \sigma_3)_{ult}$ - ultimate or residual deviator stress

6. $(\sigma'_1/\sigma'_3)_{ult}$ - ultimate obliquity

The development of a crack in a critical zone of an earth embankment could lead to a dangerous situation (Leonards and Narain, 1963). These cracks usually appear on the exposed surfaces and are caused by shrinkage which takes place when moisture in the compacted mass is lost to the atmosphere by evaporation or by differential settlements. Quite often open cracks get filled with water which tends to soften and weaken the soil below. Accumulation of water in vertical cracks could also increase the pore pressures within the embankment. Loss of strength due to a decrease in the effective normal stress, accompanied by a loss of shearing resistance together with high swelling pressures may lead to failure conditions. Hence, in addition to strength parameters, there seems to be a need to fix some criterion for local failure based on the tensile resistance (Krishnayya, Eisenstein and Morgenstern, 1974).

The different failure criteria may now be examined with respect to drained and undrained shear on normally and highly overconsolidated (or prestressed by compaction) soils.

7.6.1 $(\sigma_1 - \sigma_3)_{max}$ vs $(\sigma'_1/\sigma'_3)_{max}$

The $(\sigma_1 - \sigma_3)_{max}$ and $(\sigma'_1/\sigma'_3)_{max}$ failure criteria were discussed by several authors at the ASCE (1960) Research Conference on the

Shear Strength of Cohesive Soils. In the paper by Hvorslev (1960), the Mohr-Coulomb failure criterion is expressed as

$$\frac{\sigma'_1}{\sigma'_3} = \frac{(\sigma'_1 - \sigma'_3) - 2 c' \cos \phi'}{\sigma'_3 \sin \phi'}$$

which shows that the maximum values of (σ'_1/σ'_3) and $(\sigma'_1 - \sigma'_3)$ occur simultaneously or at the same strain when σ'_3 is held constant, as in a drained test with increasing axial load. Hence the choice of either failure criterion would give the same effective stress strength parameters in a σ'_3 constant test. The $(\sigma'_1/\sigma'_3)_{\max}$ and the $(\sigma'_1 - \sigma'_3)_{\max}$ criterion would also give the same result for increasing values of $(\sigma'_1 - \sigma'_3)$ if the cohesion parameter c' is zero. The $(\sigma'_1/\sigma'_3)_{\max}$ criterion will lead to inconsistencies when the soil possesses cohesion.

In an undrained test on a saturated normally consolidated soil with constant cell pressure, σ'_3 decreases with axial strain and the maximum value of (σ'_1/σ'_3) may then occur after the maximum value of $(\sigma'_1 - \sigma'_3)$ is attained. Such behavior was observed in this study in two uncompacted shale aggregate specimens sheared after isotropic consolidation, as shown in Figure 16(d). In this case the choice of $(\sigma'_1 - \sigma'_3)_{\max}$ as the failure criterion is proposed as it gives a lower angle of friction than that obtained by the $(\sigma'_1/\sigma'_3)_{\max}$ criterion.

In an undrained test on a saturated heavily overconsolidated (or prestressed) soil with constant cell pressure, σ'_3 increases with axial strain and the maximum value of (σ'_1/σ'_3) may then occur before the maximum value of $(\sigma'_1 - \sigma'_3)$ is reached. Such behavior is observed

in this study with several compacted specimens sheared after isotropic consolidation, as shown in Figures 13(d) and 14(d). In this case also the choice of $(\sigma_1 - \sigma_3)_{\max}$ as the failure criterion is preferred, as it gives a lower angle of friction than that obtained by the $(\sigma'_1/\sigma'_3)_{\max}$ criterion.

In general, the definition of the state of failure by the peak value of (σ'_1/σ'_3) tends to produce greater values of ϕ' , and smaller values of c' than those obtained when failure is defined by the maximum value of $(\sigma_1 - \sigma_3)$.

In some of the tests carried out on compacted shale specimens, it is observed that $(\sigma_1 - \sigma_3)$ continues to increase whereas (σ'_1/σ'_3) remains fairly constant. The test results plotted in Figure C7(a) show this type of behavior. In such a situation, the changing stress conditions during the last part of the test are very close to those of the failure envelope. The strength parameters may then be determined by drawing a straight line parallel to the stress path and passing through the point on the stress path where (σ'_1/σ'_3) is a maximum. This procedure was used by Schultze (1966) to obtain the strength parameters from the results of a single test.

7.6.2 Specified Limiting Strain

Schmertmann and Osterberg (1960) showed that the friction and cohesion parameters (ϕ', c') vary with axial strain. In a more recent paper Schmertmann (1976) introduced the concept of a Constant Structure Mohr Envelope (CSME) whose shape is given by the equation

$$\tau = I_o + \alpha \sigma'_t - \beta \sigma'_t \ln \sigma'_t$$

where τ is the shear strength, σ'_t is the effective normal stress, I_o represents the real mobilized bond shear resistance of the soil structure at zero normal stress and α and β are dimensionless parameters. The α parameter seems to depend on the soil friction, dilatancy, and dispersion aspects of the soil structure. The β parameter seems to depend primarily on grain size and shape and probably reflects the nonlinear behavior occurring at particle contact points.

The terms I_o , α and β in Schmertmann's equation depend only on the shear strain.

The achievement of a more or less constant soil structure in an earth embankment is one of the main objectives in compaction control. If this objective is met in the field, then changes in structure of the soil would be influenced mainly by the shear strain. The ambient effective stresses will produce only negligible changes in particle orientation, packing, etc., and therefore negligible changes in structure. If a limiting strain is specified as the failure criterion, the structure of the soil at failure would be independent of the level of ambient stresses, and would depend only on the level of shear strain as pointed out by Schmertmann.

The parameters I_o , α and β may be determined by using data from three tests or by using his curve hopping technique on a single sample. Three values of τ and σ'_t at the same strain are obtained, and used to solve three simultaneous equations with I_o , α and β as the unknowns.

7.6.3 $(\sigma_1 - \sigma_3)_{\max}$ vs $(\sigma_1 - \sigma_3)_{ult}$

The shearing resistance of a fully developed failure surface is equal to the residual or ultimate values, which may be much less than

the maximum values (Skempton, 1964). The residual strength parameters would in general be less than the peak values.

Failure in shale embankments may take place many years after construction. Settlements accompanied by progressive sliding usually occur prior to failure under conditions of approximately constant total stress. Loss of shearing resistance, therefore, may be attributed to either a reduction in the strength parameters or a reduction in the effective normal stresses. In some situations both phenomena may take place concurrently and these changing conditions would probably be strain dependent.

Schofield and Wroth (1968) suggest that the critical state (or residual) strength rather than the peak strength should be used in effective stress stability analysis of brittle clays. Compacted shale is a highly prestressed material and its behavior under confining pressures which are much lower than the induced prestress is similar to that of a brittle clay or a dense sand.

Bishop (1967, 1971) defined a brittleness index I_B where

$$I_B = \frac{\tau_f - \tau_r}{\tau_f}$$

to represent the post peak loss in strength. The subscripts f and r stand for peak (or failure) and residual (or critical state) values, respectively, of the shear strength τ .

Residual state conditions are often not reached in a triaxial test on highly prestressed samples. Hence indirect methods have to be used to obtain residual parameters. Such a method is available

for a drained test, but no method has yet been developed for an undrained test.

One convenient method of determining the residual or critical state strength of a cohesionless soil in a drained test is to plot the friction angle at failure ϕ'_f against the rate of volume change at failure for a series of samples at different initial densities tested in triaxial compression. The friction angle ϕ'_{cv} of the sample failing at zero rate of volume change at failure is found by extrapolation (Bishop, 1971).

The true friction angle ϕ'_u of the mineral surfaces of the particles is less than ϕ'_f and ϕ'_{cv} . The following equations have been proposed relating ϕ'_{cv} and ϕ'_u :

$$\text{Caquot (1934)} \quad \tan \phi'_{cv} = \frac{\pi}{2} \tan \phi'_u$$

$$\text{Bishop (1954)} \quad \sin \phi'_{cv} = \frac{15 \tan \phi'_u}{10 + 3 \tan \phi'_u}$$

Sadasivan and Raju (1977) have presented a new theory for the relationship between the peak angle of friction at constant volume ϕ'_{cv} and the interparticle friction angle ϕ'_u . Their theory, unlike those presented by previous investigators, accounts for the influence of void ratio on the limiting or peak value of ϕ'_{cv} . For a given void ratio the peak ϕ'_{cv} increases as ϕ'_u increases, and for the same material (ϕ'_u remaining constant) ϕ'_{cv} increases with decreasing void ratio. The

equation relating ϕ'_{cv} , ϕ'_u and the void ratio e is given as

$$\begin{aligned}
 (\sigma'_1/\sigma'_3)_{crit} &= \tan^2(45 + \phi'_{cv}/2) \\
 &= \left(\frac{\tan \phi'_u}{(1 + \tan^2 \phi'_u)^{1/2}} \ln \frac{(1 + \tan^2 \phi'_u)^{1/2} + \tan \phi'_u + \tan (\beta/2)}{(1 + \tan^2 \phi'_u)^{1/2} - \tan \phi'_u - \tan (\beta/2)} \right. \\
 &\quad \left. - \frac{2}{1 - \tan^2 (\beta/2)} \right) \Bigg|_{\beta=0}^{\beta=\beta_s} = (1 - \sin \beta_s) \tan (\phi'_u + \beta_s)
 \end{aligned}$$

where $\beta_s = \sin^{-1} (1 - \phi'_u/45)(1 + e)/12$

They found that the computed value of ϕ'_u is independent of the void ratio. This result was considered as indirect evidence in support of their theory, since interparticle friction is a surface property of the material and is independent of the void ratio, grain size, their distribution and arrangement, etc.

The above equations relating ϕ'_{cv} and ϕ'_u have practical significance. The value of ϕ'_{cv} obtained from consolidated undrained tests may be used to obtain the value of ϕ'_u (Caquot, 1934; Bishop, 1954). If the equation developed by Sadasivan and Raju (1977) is used, both ϕ'_{cv} and the consolidated void ratio e must be substituted into their equation and the value of ϕ'_u obtained. It is to be noted that all three equations are strictly applicable to granular materials possessing no cohesion.

7.7 Effective Stress Strength Parameters

7.7.1 Test Results

The effective stress parameters c' , ϕ' may be determined for any desired level of shear strain as done by Schmertmann and Osterberg

(1960). They may also be determined for any level of normal pressure when the Mohr failure envelope is curved as done for example by Skempton (1961), who designated the parameters as intrinsic cohesion and friction.

The results of this study show (Table 12) that the effective stress strength parameters evaluated at $(\sigma_1 - \sigma_3)_{\max}$ vary only slightly for compacted specimens. Typical values of c' range from 0.59 to 2.3 psi and ϕ' ranges from 28.2° to 31.3° . The specimens which were not compacted have a much lower friction angle of 24.6° at $(\sigma_1 - \sigma_3)_{\max}$, which is reached at a low strain. The friction angle increases to 35.7° at 20% strain (see Figure 16(d)).

When the shear strength parameters are compared at selected strains ranging from 0 to 20%, the influence of compaction history and consolidation pressure are very marked. Dense specimens have higher friction angles than less dense specimens at low strains.

7.7.2 Mechanism of Shearing Resistance

If we assume that the unit cohesion for a material is a constant, then the total cohesion would be proportional to the total area of contacts. As the void ratio decreases, the number of contacts would increase, and there would be a corresponding increase in the total cohesion as shown earlier by Hvorslev (1936).

The area of contacts increases with increasing normal pressure and there is also a tendency for new contacts to be formed. The net result of increased pressure, therefore, is to increase the total contact area and hence the cohesion.

Table 12

Effective Stress Strength Parameters At
 $(\sigma_1 - \sigma_3)_{\max}$ From CIU Triaxial Tests

Compaction History			Strength Parameters		No. of Tests
Gradation No.	Added Water W_a gm/500gm	Kneading Compaction Pressure P_c psi	c' psi	ϕ' degrees	
2	25	200	1.75	30.5	5
2	35	200	2.29	29.1	2
2	40	200	1.16	30.6	3
6	40	200	1.15	30.0	3
1	25	100	1.74	30.6	3
1	40	100	1.74	30.3	4
5	25	100	1.73	30.0	2
6	25	100	1.15	30.2	4
6	35	100	1.70	28.2	3
5	25	50	1.15	30.0	2
5	30	50	0.59	33.4	5
5	35	50	1.16	30.8	2
5	40	50	1.75	31.3	3
5	0	0	0	24.6	2

When a contact is subject to a shear strain under combined normal and shear stresses, there is a tendency for the area of the contact to increase, and this is accompanied by a tendency for the material to compress (Calladine, 1971). When the ratio of the shear force to the normal force at the contacts approaches the coefficient of sliding friction, the contacts would begin to slide with respect to one another.

When sliding takes place, there is a tendency for particles to ride over one another. The result would be a volume change tendency which would depend to a large degree on the density of the material. Loose samples would tend to compress, and dense samples would tend to expand or dilate (Reynolds 1886). When volume reduction takes place, there would be an increase in the area of contacts and when volume expansion takes place, there would be a slight decrease in the area of contacts. The same may be said of the total number of contacts.

When the area of contacts decreases during dilation, there is a tendency for particle contacts to break (Hoek and Bray, 1974), and this breakage increases with increasing pressure. When particles break, the contact area increases and this results in an increase in the total cohesion.

From the foregoing discussion, it is readily seen that cohesion, friction and particle interlocking combine to give the total shearing resistance of the material. Further, the relative amounts mobilized at any strain depends on the level of strain and the level of normal stress.

7.7.3 Correction for Dilatation Tendency

The peak friction angle increases with decreasing void ratio, but the ultimate friction angle attains a constant value irrespective of the initial density. However, if a correction is made for dilatancy, as done by Rowe (1962, 1971), the true physical angle of friction ϕ'_f for the material is given by the equation

$$\frac{\sigma'_1}{1 + \frac{d \epsilon_v}{d \epsilon_1}} = \sigma'_3 \tan^2(45 + \phi'_f/2) + 2c'_u \tan(45 + \phi'_f/2)$$

where $d \epsilon_v / d \epsilon_1$ is the positive rate of volume expansion with axial strain during shear. This equation was derived for drained tests wherein volume changes take place. In an undrained test $d \epsilon_v / d \epsilon_1$ is zero, but the pore pressure varies to counteract the volume change tendency. An expression analogous to that developed by Rowe for a drained test has not been developed as yet for an undrained test.

The rate of change of the A parameter with axial strain is dependent to a large extent on the constant void ratio of the sample. For small differences in void ratio between samples, the variation in $dA/d\epsilon$, during the initial stages of loading, depends mainly on the consolidation pressure. At larger strains, $dA/d\epsilon$ attains a more or less constant value which is only slightly influenced by the consolidation pressure, but greatly by the void ratio as seen in Figures 13(c) through 16(c).

An equation of the form

$$\frac{\sigma'_1}{\left(1 - \frac{dA}{d\epsilon_1}\right)} = \sigma'_3 \tan^2 \left(45 + \frac{\phi'_f}{2}\right) + 2 c'_u \tan \left(45 + \frac{\phi'_f}{2}\right)$$

would seem to be worthy of consideration for an undrained test. The negative sign is used in the term $(1 - dA/d\epsilon)$ because positive volume changes in a drained test are associated with negative pore pressure changes in a drained test. The validity of this expression could be checked by plotting $\sigma'_1/(1 - dA/d\epsilon)$ against σ'_3 for each test. If a straight line relationship is obtained, the expression may be considered to be applicable for an undrained shear test. This intuitive relationship remains to be fully investigated.

7.8 Critical States in p' , q , e Space

A soil is considered to be in its critical state when the applied strain no longer causes a change in its state (Schofield and Wroth, 1968). In an undrained test at constant cell pressure, the critical state is defined when the deviator stress and the change in pore pressure reach a constant value. In a drained test the deviator stress and the void ratio must reach a constant value. For both types of test there is a unique relationship between the shear stress and mean normal pressure (q , p') and between the void ratio and the mean normal pressure (e , p').

Out of the many tests carried out in this study, only a few of them could be considered to have reached their critical states. These are the tests where the deviator stress and the pore pressure remain constant beyond a certain level of strain. Samples formed at zero and 50 psi reached their critical states, as seen in Figure 15 and 16.

The other samples were in the process of reaching their critical states when the tests were terminated at 20% axial strain. In these tests the deviator stress and the pore pressure were still changing at the end of the test, as shown in Figures 13 and 14.

The relationship between p' and q at 20% axial strain for all the tests conducted is linear, as shown in Figure 37(a). However, there is considerable scatter in the corresponding plot of void ratio e against normal pressure p' shown in Figure 37(b). In the p' , q relationship it is possible for p' and q to change so that their locus follows the failure envelope. In an undrained test the void ratio remains constant and the mean effective normal stress changes over a wide range. Hence, it is almost impossible to define the e , p' relationship when the critical state has not yet been reached.

The overall void ratio of the test specimen will in most cases be different from the void ratio in the failure zone. This is due to non uniform deformation of the cylindrical specimen. Hence, even in cases where the critical state stresses have been reached, the determination of the critical state void ratio in the failure zone presents difficulties.

The main problem is that it is not possible to determine the void ratio of the failure zone while the critical state stresses are still acting on the sample. When the effective stresses on the sample are released, its volume will change. When the back pressure on the pore water is reduced to atmospheric pressure the air which was compressed under the high back pressure will expand. Some of the air dissolved in the pore water will also be released.

FIGURE 37 EFFECTIVE MEAN STRESS, SHEAR STRESS AND VOID RATIO RELATIONSHIPS AT FAILURE

On account of these experimental difficulties, the determination of the void ratio of the failure zone at failure is not possible. The very best we could do is to speak in terms of the overall void ratio, which in the case of an undrained test is the consolidated void ratio.

7.9 Consolidated Undrained Strength

In Figures 38 to 40 are shown plots of the maximum principal stress difference $(\sigma_1 - \sigma_3)_{\max}$ against the preshear consolidation pressure σ'_c for specimens compacted at 200, 100 and 50 psi, respectively. Specimens having the same initial gradation and the same amount of added water during compaction are identified by a separate symbol.

For the range of consolidation pressures used ($0 \leq \sigma'_c \leq 40$ psi), the relationship between $(\sigma_1 - \sigma_3)_{\max}$ and σ'_c is non-linear, for specimens compacted at 200 and 100 psi (Figures 49 and 50). The relationship is similar to that displayed by overconsolidated clays, the ratio of the undrained strength to the consolidation pressure decreasing with increasing consolidation pressure. In the case of specimens compacted at 50 psi (Figure 40), the relationship is essentially linear, and this behavior is similar to that displayed by normally consolidated clays.

7.9.1 Effect of Molding Water

Comparing specimens having the same initial gradation, compaction pressure and consolidation pressure, e.g., as in Figures 38 and 39, it is seen that the saturated undrained strength increases as the amount of water added during compaction (molding water) is increased

FIGURE 38 STRENGTH VS CONSOLIDATION PRESSURE FOR SPECIMENS COMPACTED AT 200 psi

FIGURE 39 STRENGTH VS CONSOLIDATION PRESSURE FOR SPECIMENS COMPACTED AT 100 psi

FIGURE 40 STRENGTH VS CONSOLIDATION PRESSURE FOR SPECIMENS COMPACTED AT 50 psi

(within the range of molding water contents used, 5% to 8%). In Figure 40, the variation in undrained strength due to variations in the molding water content is less pronounced.

The increase in undrained strength with increasing molding water content, all other conditions being equal, is due to the development of higher negative pore water pressure changes during shear. The magnitude of the negative pore water pressure generated increases, in turn, with increasing tendency to dilate. Thus the effect of an increase in the molding water content is to produce a more dilative specimen during shear.

In the case of sands, the tendency to dilate increases with increasing relative density (decreasing void ratio), and decreases with increasing consolidation pressure. In the case of remolded clays, dilation increases with increasing overconsolidation ratio. At very high stress levels, there is a tendency for particle contacts to crush and for particles to break. When this phenomenon takes place during shear, the result is a decrease in the dilatancy during drained shear and a corresponding decrease in the magnitude of the developed negative pore water pressure during undrained shear.

Shale slakes (partially) in the presence of added water, undergoes mechanical breakdown during compaction, and very likely undergoes further breakdown when sheared. The more breakdown that takes place during compaction, the less likely will breakdown occur during shear. An increase in the molding water content tends to produce a specimen whose particles are more resistant to subsequent breakdown during shear, due to increased degradation during compaction, especially at

high pressures. More stable particles in a specimen give rise to an increased tendency to dilate during drained shear and to a corresponding increased tendency for the development of negative pore water pressures during undrained shear.

7.9.2 Effect of Initial Gradation

The effect of initial gradation will be illustrated by referring to the results shown in Figure 39. Consider the set of specimens (1 - 25 - 100) and (6 - 25 - 100), both sets having the same amount of molding water (25 gm/500 gm) and compaction pressure (100 psi), but with different initial gradations (No. 1 and No. 6). Initial gradation No. 1 has a wider range of particle sizes than initial gradation No. 6, as can be seen from Figure 3. The data in Figure 39 indicate that the undrained strength for a given consolidation pressure is greater for the set of specimens (1 - 25 - 100) than for the set of specimens (6 - 25 - 100). The same trend is evident from the results shown in Figure 38 for sets of specimens (2 - 40 - 200) and (6 - 40 - 200), but to a lesser degree.

We may therefore conclude that an initial gradation composed of a wider range of particle sizes tends to produce specimens with increased dilative characteristics and decreased particle breakdown during shear. The increase in the undrained strength is explained by an increase in the magnitude of the negative pore water pressure with increased tendency to dilate.

7.9.3 Undrained Strength Principle

The usefulness of the effective stress strength parameters rests upon the presumption that the effective normal stresses in a failure plane can be determined or estimated in any problem, and that the effective stress strength parameters are only slightly dependent on the initial condition.

In situations where the effective normal pressure can neither be determined nor estimated, the undrained strength principle is made use of in stability analyses. This principle has been discussed by Whitman (1960). He has stated that the undrained strength depends entirely upon the conditions which exist at the start of the undrained shear process. This is also evident from the results shown in Figures 38 to 40 for compacted New Providence shale. He has also pointed out that there still remains the problem of deciding which of all the shear stresses in a test specimen at failure is to be called the shear strength. Four possibilities have been suggested:

- τ_{tf} shear stress in plane of tangency to Mohr envelope at failure
- τ_{ff} shear stress on failure plane at failure
- $\tau_{\beta f}$ shear stress on plane of maximum obliquity at failure
- τ_{mf} maximum shear stress at failure.

The differences among τ_{tf} , τ_{ff} , $\tau_{\beta f}$, and τ_{mf} are seldom great and it has been observed that whereas τ_{mf} predicts the factor of safety against failure fairly well, it incorrectly predicts the surface along which slip will take place.

CHAPTER 8

DESIGN AND CONSTRUCTION GUIDELINES

8.1 State-of-the Art

Current practices in the design and construction of compacted shale embankments vary from place to place depending upon the mechanical and mineralogical properties of the shale and the accumulated experience of the designer.

The report by Shamburger, Patrick and Lutten (1975) presents the state-of-the art as far as the United States is concerned. The contributions made by Purdue University are to be found in Deo (1972), Deo, Wood and Lovell (1973, 1975), Wood, Lovell and Deo (1973), Chapman and Wood (1974), Chapman (1975), Bailey (1976), Wood, Sisiliano and Lovell (1976), Chapman, Wood, Lovell and Sisiliano (1976).

8.2 Comments on Past Practice

The first aspect that needs to be examined is the question of site or foundation preparation for the shale embankment. Some of the side-hill embankments that failed were built on colluvial soil overlying the unweathered shale. Building upon such soils is not recommended unless strength and compressibility tests performed on soaked samples of them give satisfactory results. If the results are poor, such material should be excavated and recompactd, or even totally discarded, depending on the strength and compressibility test results on compacted soaked samples.

The second aspect relates to the provision of benches excavated into the hillside for sidehill embankments. The provision of such benches would obviously add to the initial cost of construction, but there are very good reasons why this should be done. The interface between unweathered bedrock and compacted shale is likely to be "smooth" with little or no interlocking. Hence, it is preferable that these planes be made horizontal rather than left inclined downslope. The provision of benches would also insure that the interface between the compacted lifts would be horizontal. Whereas shale chunks would tend to interlock with each other within a particular lift, such interlocking is less likely to be present between the lifts.

The third aspect is the provision of an adequate drainage system either during or soon after placing the fill to control the build up of excess pore water pressures with time. This subject has been dealt with in great detail by Shamburger, Patrick and Lutten (1975). It is emphasized that any drainage pipes provided for flow of water from the uphill side, through the embankment, and on to the downhill side should be of the flexible type to withstand differential settlements.

The fourth aspect relates to situations where intermittent beds of limestone and shale are encountered. As pointed out by Wood, Lovell and Deo (1973), there is a tendency for the harder limestone to protect the softer shale from reduction in size by the compaction process. Hence, it is desirable that the limestone and the shale be excavated and placed in separate layers rather than as a random mix.

When shale aggregate is compacted, the moisture-dry density relationship is not as well defined as in fine grained cohesive soils. The terms "dry of optimum", "at optimum", and "wet of optimum", have limited meaning. The standard effort laboratory compaction tests are usually performed on the minus 3/4 in. fraction material. In the field, shale chunks as large as 6 to 12 inches may be present even after compaction. Hence, the adoption of standard laboratory compaction test results using minus 3/4 in. material is clearly not feasible for controlling field compaction of shale chunks, unless a correction is made for over sizes.

The natural dry density and water content of the in situ shale was used by U. S. Bureau of Reclamation (1959) as a reference for compaction control. This unusual criterion was used when it was found that standard effort laboratory compaction tests gave erratic results because of continuous breakdown of siltstone particles during compaction (Shamburger, Patrick and Lutten, 1975). Water was added to maintain a near saturated condition in the absorptive siltstone chunks (short of creating a spongy fill).

As the degree of saturation of the as-compacted shale is increased, by addition of increasing amounts of water, the pore water pressures developed during the placement of subsequent lifts will tend to increase. When the end of construction pore water pressures happen to be higher than the long-term steady state values, then the likelihood of a long-term failure taking place is minimized. If on the other, the end of construction pore water pressures happen to be much less than the long-term values, then the likelihood of a long-term failure is enhanced.

When a fixed compaction effort and moisture content is used while placing the fill, irrespective of the final location of the layers with respect to the completed embankment, differential swelling and compression are bound to take place as the fill imbibes water. If these changes could be made to take place during construction of the embankment, but some means, before the highway pavement is constructed, the long-term performance of the entire structure would be much improved.

8.3 Construction Technique for Hard Non-Durable Shales

Mechanically hard but non-durable shales pose special problems. First, as it is difficult to mechanically breakdown the material during compaction, the compacted fill is likely to have large voids. Secondly, the process of degradation in the presence of water is a time dependent phenomenon. Thirdly, the rate of degradation is dependent on the mineralogical composition of the shale, the amount of water present, and the loads acting on the particles.

The presence of water during compaction will tend to assist mechanical breakdown, but this does not mean that further degradation would not take place with continued exposure to water. In other words, there is a limit to the amount of degradation that can take place during compaction no matter how much water is present.

Some means must therefore be found to speed up the degradation process and thereby the settlements, so that the highway pavement could be laid on an embankment which has already reached an equilibrium condition, comparable to the long-term case.

To the writer's knowledge no attempt has yet been made to accelerate the rate of degradation by artificial means. The current practice is to lay the highway pavement on the as-compacted embankment within a year after construction. Various drainage systems are installed to prevent ground and surface water from seeping into and saturating the fill. In spite of these precautions, water eventually enters the fill in some undefined and non-uniform manner. This results in differential movements within the embankment and along the highway pavement. The changes that will take place cannot be controlled and corrected, other than by overlaying the pavement, a solution which is not only very costly but also obstructs the free flow of traffic. The provision of berms, flattening of the slopes, etc., to prevent a slope failure from occurring may have to be resorted to at great cost, but there is always a great deal of uncertainty as to when this should be done and where, and to what extent.

The following technique might be considered for controlling the post-construction settlements, of relatively hard but non-durable shales which present most of the long-term problems. It is assumed that an adequate supply of water is available for use at a reasonable cost, during the construction period.

1. Compact the shale in thin lifts with the addition of water to assist the degradation of the larger sizes.
2. Install a network of flexible perforated PVC pipes within the fill as illustrated schematically in Figure 41. These pipes should preferably be wrapped by filter cloth, and they should be placed as construction proceeds.

FIGURE 4I SCHEMATIC ARRANGEMENT OF PERFORATED
PIPES FOR SATURATING AND DRAINING FILL

3. Introduce water into the network of pipes after they are buried in the fill. A low pressure just sufficient to cause an adequate flow should be used.

4. Saturation of the fill will cause the non-durable shale to degrade and the embankment will settle. The settlements will cause excess pore water pressures to develop but they would dissipate when flow takes place through the network of perforated pipes kept exposed to atmospheric pressure.

5. After the full height of the embankment has been built, monitor the settlements of the surface and the side slopes until such time as the movements approach equilibrium values.

6. Construct the pavement when the previously saturated embankment has reached an equilibrium condition.

7. The perforated pipes will act as permanent underdrains and the likelihood of excess pore water pressures developing with time will be minimized.

The installation of the perforated pipes in a rock-like fill will present some problems. There is the danger that they would tend to get crushed under the weight of the construction equipment. To overcome this problem, it is recommended that they be laid in a trench and backfilled with gravel. The trenches may be excavated with the blade of a ripper or other equipment. For the schematic arrangement shown in Figure 41, a linear foot of perforated pipe drain is associated with 300 cu. ft. of fill. This ratio may be adjusted as required depending on the time available for saturation, the permeability of the fill, and the cost of installation.

As an alternative to pumping water into the pipes to accelerate degradation during construction, a stabilizing agent in liquid form may be pumped into the compacted fill to control post construction degradation. Another possibility is to mix the shale with a solid stabilizing agent such as lime and use the pipes to furnish water for the chemical reactions to take place.

8.4 Evaluation of Effective Stress Strength Parameters

For the New Providence shale, the effective stress strength parameters c' and ϕ' of soaked specimens evaluated at maximum principal stress difference from consolidated undrained triaxial tests, were found to be essentially independent of the initial gradation, the molding water content, the compaction pressure and the pre-shear consolidation pressure (Table 12). This experimental result permits the evaluation of the strength parameters by performing only a few tests such that the mean normal stress at failure (maximum principal stress difference) is representative of the range likely to be present in the embankment.

The data in Figure 35 show that the mean normal pressure at failure is mainly a function of the consolidation pressure, the compaction pressure and, to a lesser extent, the initial gradation and the molding water content. The ratio of the mean normal pressure at failure to the consolidation pressure increases as the ratio of the compaction pressure to the consolidation pressure increases. This figure is useful for predicting the mean normal pressure at failure for a given initial condition prior to shearing.

The procedure outlined hereafter is recommended for evaluating the effective stress strength parameters of compacted shales similar to New Providence shale.

8.4.1 Type of Test

It is recommended that the effective stress strength parameters be determined from both consolidated undrained and consolidated drained triaxial tests on 4 inch diameter by 8 1/2 inch high specimens. The parameters may differ for the two types of test, depending on the failure criterion used. When the failure criterion used is the maximum principal stress ratio $(\sigma'_1/\sigma'_3)_{\max}$, the two types of test will likely give very similar strength parameters. When the maximum principal stress difference $(\sigma'_1 - \sigma'_3)_{\max}$ is used as the failure criterion, the consolidated drained test will probably give higher strength parameters. When the strength parameters are evaluated at 20% strain, the two types of tests will likely give very similar values. Both types of tests are recommended because there is a great need for the generation of and comparison between strength parameters obtained from both types of tests.

8.4.2 Initial Gradation

It is recommended that the initial gradation be the same as that obtained by crushing the shale chunks in a jaw crusher and rejecting pieces larger than 3/4 inch nominal diameter. The initial gradation may, however, be artificially prepared to parallel the field gradation, whenever the latter is known, e.g., from test pads.

8.4.3 Molding Water

The amount of molding water to be used during compaction of triaxial specimens may be set at three levels. The lowest level is that which will produce a specimen that can be removed from the mold

without loose pieces of shale sloughing from the surfaces. The highest level is that which will not cause the shale to become unduly spongy during compaction. The intermediate level of molding water content may be taken as the mean of the two extreme values.

8.4.4 Curing Time

The time allowed for curing, i.e., the time elapsed between mixing the shale aggregate with water, and its subsequent compaction will most likely have an influence on the moisture-dry density relationship. As the curing time is increased the shale aggregate will tend to absorb some or all of the molding water, and the strength of the shale pieces will tend to decrease. The decrease in strength of the shale pieces will make the aggregate more susceptible to breakdown during compaction. Further, there will be less water present between the shale pieces and the expulsion of air during compaction will be easier than when the pores contain the molding water.

When the curing time is relatively short, the strength of the aggregate will be altered to only a slight degree. Further, the molding water will remain on the surfaces of the shale aggregate and the likelihood of pore water and pore air pressures building up during compaction will increase.

A short curing time in the laboratory would appear to be more logical because such is the case in field practice.

8.4.5 Compaction

It is recommended that the test specimens be compacted using a kneading compactor to simulate field compaction as described in Section 5.2.

The following discussion relates the compaction pressure to be used in kneading compaction with the results of impact compaction. The compaction data indicate that the average dry density increases linearly with the logarithm of the kneading compaction pressure (see Figure 9). Attewell and Farmer (1976) have presented data which show that the dry bulk density increases linearly with the logarithm of the compaction energy per unit volume.

A comparison of the dry densities of New Providence shale is given below for the purpose of fixing a realistic compaction energy or pressure for molding triaxial test specimens.

Table 13

Dry Densities of New Providence Shale

In-situ bedrock	145.0 pcf
Impact compaction of aggregate	
at Std. AASHTO	120.6 pcf
at Mod. AASHTO	133.0 pcf
Kneading compaction of aggregate	
at 200 psi	132.2 pcf
at 100 psi	128.7 pcf
at 50 psi	121.7 pcf
Light tamping of aggregate	
with 5.5 lb Proctor hammer	94.9 pcf

Comparing the dry densities given above, it is to be noted that the Standard AASHTO compaction test gives a dry density which is

similar to that obtained by kneading compaction with a foot pressure of 50 psi. Wahls, Fisher and Langfelder (1966) have reported that a foot pressure of at least 200 psi is specified for compacting soils with sheepsfoot rollers. The state-of-the-art report on shale embankments by Shamburger, Patrick and Lutten (1975) indicates that higher foot pressures are sometimes specified. Hence, the Standard AASHTO compaction test would appear to be inappropriate for controlling the field density of shales using sheepsfoot rollers.

The dry density of 133 pcf in Table 13 obtained by Witsman (1977) using the Modified AASHTO compaction test is similar to that resulting from kneading compaction with a foot pressure of 200 psi. The Modified AASHTO compaction test would therefore seem to be more appropriate for controlling the field density of shales when sheepsfoot rollers are used. If on the other hand, a California type kneading compactor is available, it should be used in preference to the impact type of compaction associated with the AASHTO tests, to simulate field compaction.

8.4.6 Saturation

The procedure outlined in Section 5.3 is recommended for saturating the specimens. During the initial stage of saturation, water is allowed to enter the pores of the specimen under a vacuum and thereafter a high back pressure is applied to the pore water with a corresponding increase in the cell pressure so that any remaining air is compressed and goes into solution. The triaxial cell used in the research had a working pressure of 140 psi, but the compressed air

line and regulators could only provide 90 psi pressure. Hence, when a back pressure of 50 psi was used it was possible to consolidate the specimens to only 40 psi. Whenever triaxial cells are ordered for testing specimens which need to be fully saturated during shearing, they should preferably be of the high pressure type, so that higher back pressures and consolidation pressures could be applied. Compressed nitrogen cylinders and special low release regulators, along with high pressure copper tubing and other fittings, may be used to provide the higher pressures.

8.4.7 Consolidation

The procedure outlined in Section 5.4 may be used. However, instead of applying the consolidation pressure in a single increment, the required consolidation pressure may be built up in several increments. The latter procedure will necessarily take more time, but it has the advantage of producing the isotropic e - $\log p'$ relationship for each specimen.

8.4.8 Shearing

There are two main concerns when triaxial test specimens are subject to axial compression. They are non-uniform conditions of deformation and non-uniform conditions of pore water pressure dissipation during a drained test or pore water pressure equalization in an undrained test.

Non-uniform deformation can be minimized by using lubricated and oversized end platens, along with specially designed porous stones as described for example by Rowe and Barden (1964), Lee and Seed

(1964), and Bishop and Green (1965). It is recommended that this technique be used when it is desired to obtain relationships involving the void ratio at failure. When non-uniform deformation takes place, the average void ratio of the specimen will not give a correct value for the void ratio of the failure zone. Also the area correction based on uniform deformation would be subject to error. Uniform deformation also results in more uniform distribution of pore pressures and the pore pressures measured at the base of the specimen would also be those in the failure zone irrespective of the constant rate of strain. Elimination of end-friction dead zones allows the use of shorter test specimens, reduces the tendency to bulge at large strains, and permit appreciable reductions in testing time.

During drained shear there will always be a gradient in the pore water pressure, whether the deformations in the cylindrical specimen are uniform or not. This is so because the flow of water can only take place under a gradient. The magnitude of the change in pore water pressure is zero at the ends, but its value at the center of the specimen may differ from zero depending on the rate of shearing and the state of stress and strain in the specimen. No pore pressure measurements are ordinarily taken in a drained test and a sufficiently slow rate of strain is adopted so that the pore water pressure changes are dissipated as fast as they are induced. For these conditions the effective stress is equal to the total stress.

During undrained shearing there is no flow of water across the boundaries of the specimen. However, when non-lubricated end platens

are used the specimen deforms non-uniformly and migration of the pore water takes place within the specimen itself. The pore water pressure changes are also non-uniform but when adequate time is allowed for migration the degree of non-uniformity is reduced and equalization eventually takes place. When the rate of strain is sufficiently slow, the pore pressure ordinarily measured at the base of the specimen will be equal to that in the middle of the specimen where failure usually takes place.

The choice of a suitable test duration (t_f) for the evaluation of the effective stress strength parameters for conventional drained or undrained triaxial tests may be based on theoretical and experimental data. By "conventional" is meant the following test conditions:

- (a) non-lubricated end platens
- (b) pore water pressure measurements taken at the base of the specimen
- (c) constant rate of deformation

The following equations for the test duration are taken from Blight (1963). For tests without vertical drains

$$t_f = 1.6 H^2 / c_v$$

where 1.6 is the time factor corresponding to 95 percent equalization of pore pressure in tests without drains, H is half the specimen height or the drainage path, and c_v is the coefficient of consolidation of the soil. Similarly for tests with all-round drains

$$t_f = 0.07 H^2 / c_v$$

If the object of the test is to measure the shear strength parameters of the soil only, then the test duration is equal to the time to failure. If complete information on the stress path is required, the test duration will be the period between the start of shear and the first significant stress measurement. It is also the period between successive stress measurements.

Rather than relying on an equation for fixing the test duration (Blight, 1963), the procedure described by Rowe (1963) may be used. The specimen is loaded in a series of steps, either stress controlled or strain controlled. After each increment of either the axial load or the axial strain, time is allowed for steady state conditions to be reached and measurements are taken.

Strain controlled tests enable the study of the stress-strain response upto and beyond failure, whereas stress controlled tests are only suitable for the study of the stress-strain response leading upto failure. However, during a strain controlled test, the axial load, the pore water pressure and the deformations are constantly changing, and measurements have necessarily to be taken whilst these changes are taking place. The step strain technique described by Rowe (1963), though not ordinarily used in routine testing, deserves consideration, as it combines the advantages of constant rate of strain and stress controlled tests.

Special precautions must be taken in the running of drained and undrained triaxial tests of long duration. Temperature variations, diffusion of water through membranes, and piston friction are the main sources of errors that can affect the tests results.

To eliminate temperature effects, the test should be run in a constant temperature room. Studies conducted by Parry (1976) have shown that temperature rises cause substantial pore pressure increases in clay shale. As the coefficient of thermal expansion for water is about ten times the value for the soil skeleton, a rise in temperature in a sealed saturated sample will cause a rise in pore pressure. The increase in pore pressure increases linearly with the effective stress and at $\sigma' = 100$ psi a response of $\Delta u = 6$ psi/ $^{\circ}$ C was observed.

To eliminate diffusion effects it is recommended that butyl rubber membranes be used in tests of long duration. Butyl rubber has a permeability thirty times less than the latex rubber which is ordinarily used.

Errors due to piston friction may be eliminated by using cells having ball bushings or rotating bushings, or by providing an internal load cell to measure the axial load. The triaxial cell used by the author had a ball bushing piston guide and a "U" cup type packing which expands under pressure and positively prevents leakage. Water was used as the confining fluid.

8.4.9 Failure Criterion

It is recommended that the maximum principal stress difference be adopted as the failure criterion for the purpose of evaluating the shear strength parameters in terms of effective stresses, for reasons given in Section 7.6, for consolidated undrained triaxial tests.

CHAPTER 9

SUMMARY AND CONCLUSIONS

Although it is likely that these comments apply to many Indiana shales, complete experimental evidence is available for only the New Providence formation samples.

1. New Providence shale is non-durable and slakes when mixed with water. The molding water tends to be adsorbed by the smaller sized particles and this phenomenon is likely to take place in the field as well.

2. When a mixture of shale aggregate and water is compacted, further breakdown of the shale aggregate takes place and the crushing of the larger particles can be distinctly heard. However, the determination of the gradation after compaction presents several problems. This is mainly due to the fact that the compacted aggregates are bonded to each other, and it is not possible to separate them without causing further breakdown. The technique of wet sieving often used to separate bonded units, cannot be applied to New Providence and similar non-durable shales on account of the slaking phenomenon.

3. The initial moisture content of the shale aggregate used for compacting the test specimens was found to decrease from about 4% for Test No. 1 to 1.1% for Test No. 57. This loss of moisture took place while the shale chunks were stored in a bulk container, during crushing and sieving to the desired top size of $3/4$ inch, and during subsequent storage, weighing and handling at room temperatures and relative humidities.

4. The influence of molding water on the compaction characteristics of the test shale aggregate is not as well defined as in fine grained cohesive soils. This is probably due to the influence of particle crushing, slaking, etc., peculiar to shale. The variation in dry density, however, for constant conditions of gradation, molding water and compaction effort is small and lies within 1% of the mean value. The higher dry density correlates somewhat with increasing aggregate moisture content. This tendency may be explained by the decrease in aggregate strength and consequently the increase in aggregate breakdown with increasing particle moisture content.

5. The dry density increases linearly with the logarithm of the compaction pressure. Differences in the initial gradation, the molding water content and the aggregate moisture content have only a slight influence on this logarithmic relationship.

6. When compacted specimens of shale are saturated under a nominal effective confining pressure (0 to 2 psi), swelling takes place. Subsequent consolidation under a higher effective confining pressure results in a reduction of the void ratio. The net percentage change in volume from the as-compacted condition to the consolidated condition is a function of the initial void ratio (or compaction pressure) and the effective confining pressure.

7. The percentage change in volume on saturation and consolidation decreases with increasing as-compacted void ratio and increasing effective confining pressure. Both positive and negative net volume changes can take place, and there is a particular combination of initial dry density and effective confining pressure which results in

zero volume change on saturation and consolidation. This combination may be obtained by interpolation of the experimental data.

8. The current practice of compacting shale to a constant dry density and moisture content irrespective of its location in the embankment is from a practical standpoint the most feasible. Varying the as-compacted condition to achieve zero volume change on saturation under a given confining pressure is theoretically possible but would in general be impractical under field conditions.

9. The deviator stress-axial strain curves are greatly influenced by the consolidation pressure and the void ratio (or prestress). The principal features of these curves are summarized below:

- (a) The stress-strain curves of uncompacted shale specimens having a dry density of around 95 pcf are of the strain softening type. The peak deviator stress is reached at axial strains of around 2% and the stress drops off rapidly to a reduced residual value at axial strains of around 2%.
- (b) The stress-strain curves of specimens having a dry density of around 120 pcf reach their peak stress around 2% to 4% axial strain, and this stress remains practically constant up to 20% axial strain.
- (c) The stress-strain curves of specimens compacted to a dry density of 127 to 131 pcf reach their peak stress around 20% axial strain.

- (d) The stress-strain curves of the strain hardening type, (b) and (c), can be represented very well by Kondner's (1963) hyperbolic stress-strain relationship.

10. The influence of compaction pressure, molding water, and consolidation pressure on the undrained strength is summarized below:

- (a) The undrained strength increases with increasing compaction pressure and consolidation pressure.
- (b) For samples compacted at 200 psi and 100 psi, the undrained strength increases non-linearly and at a decreasing rate with consolidation pressure. For samples compacted at 50 psi, the increase in undrained strength is linearly related to consolidation pressure.
- (c) The influence of molding water on the undrained strength is dependent on the compaction pressure. For samples compacted at 200 psi and 100 psi, the undrained strength increases with increasing molding water. The influence of molding water on the undrained strength is negligible for samples compacted at 50 psi.

11. The change in pore water pressure during undrained shearing reflects the volume change tendency of the specimen. The main features of the pore pressure response are summarized below:

- (a) A typical pore pressure-axial strain curve increases non-linearly at a decreasing rate, reaches a peak value, and then decreases almost linearly with increase in strain, depending upon the relative magnitudes of the compaction and consolidation pressures. However, when the ratio of the compaction pressure to the consolidation pressure is low, the sample exhibits normally consolidated behavior and the pore pressure remains constant after the peak value is reached.
- (b) The peak positive excess pore water pressure increases linearly with increasing consolidation pressure, and the slope of the line increases with decreasing compaction pressure (or increasing void ratio).
- (c) The strain corresponding to the peak positive excess pore water pressure increases with increasing consolidation pressure and decreasing compaction pressure (or increasing void ratio).
- (d) The incremental decrease of the pore water pressure with increasing strain is more or less independent of the consolidation pressure and seems to depend only on the compaction history of the sample.

- (e) The strain at which the excess pore water pressure becomes negative increases with increasing consolidation pressure and decreasing compaction pressure (or increasing void ratio).

12. The pore pressure parameter 'A' varies appreciably with the axial strain and is dependent on the consolidation pressure, and the compaction history of the sample, as follows:

- (a) The initial value of 'A', i.e., on commencement of shearing, is not well defined, on account of errors in the measurement of small quantities.
- (b) The value of 'A' increases with increasing consolidation pressure and decreasing compaction pressure (or increasing void ratio).
- (c) The pore pressure parameter 'A' is initially positive and then becomes negative according to the change in pore water pressure.
- (d) The pore pressure parameter ' A_f ' at peak deviator stress (failure) decreases with increasing compaction pressure and decreasing consolidation pressure. The values range from 2.2 to -0.4 depending on the compaction pressure, the molding water and the consolidation pressure.

13. The isotropically consolidated effective stress paths are greatly influenced by the void ratio or compaction pressure and the consolidation pressure. The stress paths of the uncompacted loose specimens are similar to those of loose sands and highly sensitive

clays. The stress paths of the dense specimens are similar to those of highly overconsolidated natural and remolded clays or dense sands. Some of the stress paths are similar to normally consolidated sands and clays.

14. The peak principal stress ratio is reached before the peak deviator stress for dense specimens and after the peak deviator stress for loose specimens.

15. The effective stress strength parameters evaluated at peak deviator stress were found to range as follows: $c' = 1$ to 2 psi, and $\phi' = 28$ to 30 degrees, for the mean effective normal stress range of 0 to 100 psi, and specimen void ratio range of 0.321 to 0.536.

16. For the loose specimens, having an initial void ratio of 0.829 and consolidated void ratios of 0.58 and 0.59, the effective stress strength parameters take on values of $c' = 0$ psi, and $\phi' = 25$ degrees, at peak deviator stress for the mean effective normal stress range of 0 to 20 psi.

17. While the response of the partially saturated shale samples during compaction and saturation seems both unique and complex, the response of the soaked material in consolidation and shear has substantial similarity to that of simple saturated clays.

18. The testing procedure outlined in Section 8.4 should be implemented and improved by the Indiana State Highway Commission.

19. The maximum principal stress difference should be adopted as the failure criterion for the purpose of evaluating the effective stress strength parameters when consolidated undrained triaxial tests are used.

CHAPTER 10

SUGGESTIONS FOR FUTURE RESEARCH

The research just completed suggest the following studies as fruitful avenues for a better understanding of the behavior of compacted shale.

1. Evaluation of the prestress induced in compacted shale as a function of the compaction method, compaction pressure (or energy), the molding water content, the initial gradation and top size.
2. Evaluation of the degree of anisotropy in the structure, prestress and properties of compacted shale.
3. Study of the long-term deformation of compacted shale on exposure to water while under various total stress systems and levels.
4. Separate evaluation of the pore air and pore water pressures in compacted shale for the establishment of the as-compacted effective stress.
5. Study of the effect of aggregate size on the stress-deformation and strength characteristics of compacted shale.

6. Study of the consolidated drained shearing behavior of compacted shale, and the drained strength parameters.
7. Comparison of undrained and drained strength parameters in terms of effective stresses.
8. As much mineralogical analysis as is practical should be accomplished on future shale samples. This should aid in predicting the differences in shear parameters among various shales.
9. Generation of undrained strength data for a wide spectrum of Indiana shales and evaluation of residual friction angle values from triaxial and direct shear tests.

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APPENDICES

NOTICE

The following Appendices (or portions of Appendices) listed in the Table of Contents of this Report have not been included in this copy of the Report.

<u>Appendix</u>	<u>Title</u>	<u>Pages</u>
A (portion not included)	Sample Calculations and Specimen Data	213-270
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West Lafayette, Indiana 47907

APPENDIX A
SAMPLE CALCULATIONS AND SPECIMEN DATA

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SAMPLE CALCULATIONS AND SPECIMEN DATA

Sample Calculations

Typical calculations are presented herein for Specimen No. 57.

1. Initial moisture content of shale aggregate: (w_i)

$$\text{Initial weight of aggregate (W)} = 499.8 \text{ gm}$$

$$\text{Final weight of aggregate after oven drying to } 110^{\circ}\text{C (W - W}_p) = 494.2 \text{ gm}$$

$$\text{Weight of water in aggregate (W}_p) = 5.6 \text{ gm}$$

$$\begin{aligned} (w_i) &= 100 W_p / (W - W_p) \\ &= 100 \times 5.6 / 494.2 = \underline{1.133\%} \end{aligned}$$

2. As-compacted moisture content of specimen: (w_c)

$$\text{Weight of water added to aggregate (W}_a) = 35.0 \text{ gm}$$

$$\text{Weight of water in aggregate (W}_p) = 5.6 \text{ gm}$$

$$\text{Total weight of water (W}_p + \text{W}_a) = 40.6 \text{ gm}$$

$$\text{Dry weight of aggregate (W - W}_p) = 494.2 \text{ gm}$$

$$\begin{aligned} (w_c) &= 100 (W_p + W_a) / (W - W_p) \\ &= 100 \times 40.6 / 494.2 = \underline{8.215\%} \end{aligned}$$

3. As-compacted bulk density of specimen: (γ_b)

$$\text{Weight of specimen (W}_o) = 8.970 \text{ lb}$$

$$\text{Height of specimen (H}_o) = 8.760 \text{ in}$$

$$\text{Area of cross-section of specimen (A}_o) = 12.718 \text{ sq in}$$

$$\text{Volume of specimen (V}_o) = 111.406 \text{ cu in}$$

$$\begin{aligned} \gamma_b &= W_o / V_o \\ &= (8.970 / 111.406) 12^3 = \underline{139.132 \text{ pcf}} \end{aligned}$$

4. As-compacted dry density of specimen: (γ_d)

$$\begin{aligned}\gamma_d &= \gamma_b / (1 + w_c/100) \\ &= 139.132 / 1.08215 = \underline{128.569 \text{ pcf}}\end{aligned}$$

5. As-compacted void ratio of specimen: (e_o)

$$\begin{aligned}e_o &= (G_s \gamma_w / \gamma_d) - 1 \\ &= (2.78 \times 62.4 / 128.569) - 1 = \underline{0.349}\end{aligned}$$

6. As-compacted degree of saturation of specimen: (S_r)

$$\begin{aligned}S_r &= G_s w_c / e_o \\ &= 2.78 \times 8.215 / 0.349 = \underline{65.437\%}\end{aligned}$$

7. As-compacted volume of air in specimen: (V_a)

$$\begin{aligned}V_a &= V_o (e_o / (1 + e_o)) (1 - S_r/100) \\ &= 111.406 (0.349 / 1.349) (1 - .65437) = \underline{9.960 \text{ cu in}}\end{aligned}$$

8. Change in weight of specimen after saturation and consolidation: (ΔW)_f

Weight of specimen after consolidated undrained test (W_f) = 9.320 lb

Initial weight of specimen after compaction (W_o) = 8.970 lb

$$(\Delta W)_f = (W_f - W_o) = \underline{0.350 \text{ lb}}$$

9. Change in volume of specimen after saturation and consolidation: (ΔV)_f

$$\begin{aligned}(\Delta V)_f &= (\Delta W)_f / \gamma_w - V_a \\ &= 0.350 \times 12^3 / 62.4 - 9.960 \\ &= 9.692 - 9.960 = \underline{-0.268 \text{ cu in}}\end{aligned}$$

The negative sign denotes a decrease in volume of the specimen from its as-compacted state to its saturated and consolidated state under the effective isotropic consolidation pressure $\sigma'_c = 40 \text{ psi}$.

10. Change in void ratio of specimen after saturation and consolidation: (Δe)_f

$$\begin{aligned}(\Delta e)_f &= (1 + e_o) (\Delta V)_f / V_o \\ &= (1 + 0.349) (-0.268) / 111.406 = \underline{-0.0032}\end{aligned}$$

11. Final void ratio after saturation and consolidation: (e_f)

$$\begin{aligned} e_f &= e_o + (\Delta e)_f \\ &= 0.349 - 0.003 = \underline{0.346} \end{aligned}$$

12. Volume of water expelled from previously saturated specimen during isotropic consolidation as measured by a graduated burette: $(\Delta V)_c$

$$(\Delta V)_c = \underline{7.444 \text{ cu in}}$$

13. Change in volume of specimen from the as-compacted state to the saturated state: $(\Delta V)_s$

$$\begin{aligned} (\Delta V)_s &= (\Delta V)_f + (\Delta V)_c \\ &= -0.268 + 7.444 = \underline{7.176 \text{ cu in}} \end{aligned}$$

14. Change in void ratio of specimen from the as-compacted state to the saturated state: $(\Delta e)_s$

$$\begin{aligned} (\Delta e)_s &= (1 + e_o)(\Delta V)_s / V_o \\ &= (1 + 0.349) 7.176 / 111.406 = \underline{0.087} \end{aligned}$$

15. Void ratio of specimen after saturation under an effective confining pressure of 0.5 psi (back pressure 90 psi, cell pressure 90.5 psi): (e_s)

$$\begin{aligned} e_s &= e_o + (\Delta e)_s \\ &= 0.349 + 0.087 = \underline{0.436} \end{aligned}$$

Table A1 Summary of Specimen Data- Compaction, Saturation, and Consolidation

Table A1 Summary													
TEST	AGGREGATE GRADE	HOLDING WATER PERCENT	SPECIMEN AGE, DAYS	DENSITY IN PLACE	AS-COMPACTED		SATURATED		SELECT RATIO				
					UNIT WEIGHT	VOID RATIO	UNIT WEIGHT	VOID RATIO					
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				
HD	4.00	4.0	100	1.00	1.00	1.00	1.00	1.00	1.00				

Table A1 (continued)

TEST NO	AGGREGATE GRAD (INIT)	MOIS CONT	MOLDING		KNEAD		SPECIMEN			AS-COMPACTED			CONDITION		SATURATED		CONSOLIDATED	
			WATER GMS/500	FOOT PRESS	AVG HEIGHT INCH	AVG AREA SQIN	WET PCF	DENSITY PCF	DRY PCF	WATER CONTENT PERCENT	VOIDS RATIO	SATU- RATION PERCENT	EFFECT PRES PSI	VOIDS RATIO	EFFECT PRES PSI	VOIDS RATIO		
30	5	1.65	30	50	8.606	12.646	129.3	120.0	7.75	.446	48.3	10.0	.381	10	.381			
31	5	1.64	30	50	8.506	12.650	130.9	121.5	7.73	.428	50.2	5.0	.432	5	.432			
32	5	1.65	0	1	8.812	12.566	110.7	98.2	1.65	.829	5.5	20.0	.583	20	.583			
33	5	1.65	0	1	8.875	12.566	115.2	94.9	1.65	.829	5.5	10.0	.595	10	.595			
34	2	1.61	40	200	8.609	12.638	146.1	133.2	9.73	.303	99.3	.5	.368	20	.354			
35	2	1.62	25	200	8.704	12.647	139.3	131.1	6.60	.323	56.7	.5	.432	20	.370			
36	3	1.33	35	200	8.615	12.687	142.3	121.2	8.48	.322	73.1	.5	.448	20	.396			
37	5	1.19	40	50	8.518	12.683	132.2	120.9	9.28	.434	59.4	.5	.503	20	.420			
38	5	1.01	25	50	8.645	12.653	126.5	119.3	6.07	.454	37.2	1.0	.477	20	.419			
39	5	1.28	35	50	8.337	12.711	132.9	122.6	8.36	.414	56.2	.5	.452	10	.398			
40	5	1.33	40	50	8.592	12.652	131.3	119.9	9.53	.447	59.3	10.0	.427	10	.427			
41	5	1.42	25	50	8.629	12.577	126.4	118.7	6.49	.461	39.1	1.0	.531	10	.489			
42	5	1.46	35	50	8.534	12.642	131.2	120.9	8.56	.435	54.7	1.0	.459	10	.424			
43	5	1.34	30	50	8.486	12.684	130.3	121.3	7.42	.429	48.0	1.0	.469	40	.379			
44	5	1.26	40	50	8.390	12.745	133.9	122.4	9.36	.417	62.4	1.0	.452	40	.374			
45	6	1.22	25	100	8.304	12.578	132.0	124.2	6.28	.336	44.1	1.0	.476	40	.375			
46	2	1.22	25	200	8.684	12.627	138.6	130.4	6.23	.330	52.9	1.0	.435	40	.359			
47	4	1.13	40	200	8.579	12.572	144.7	132.5	9.24	.309	83.1	2.0	.382	40	.351			
48	3	1.21	35	200	8.613	12.570	142.3	131.4	8.30	.321	72.0	2.0	.417	40	.364			
49	5	1.22	25	100	8.450	12.665	129.5	121.3	6.23	.424	41.2	2.0	.455	40	.372			
50	1	1.26	40	200	8.725	12.623	142.8	130.6	9.35	.328	79.3	0	.396	40	.347			
51	6	1.21	40	200	8.896	12.648	143.4	131.2	9.30	.323	80.1	1.0	.411	40	.371			
52	6	1.22	35	200	8.669	12.638	138.7	128.1	8.31	.354	65.2	1.0	.426	40	.358			
53	6	1.21	40	200	8.583	12.641	145.1	132.7	9.31	.307	84.3	1.0	.367	80	.307			
54	2	1.07	25	200	8.651	12.637	136.1	123.2	6.12	.353	49.3	1.0	.374	80	.282			
55	2	1.17	25	200	8.826	12.667	137.3	129.2	6.23	.343	50.5	0	.409	80	.318			
56	3	1.15	40	200	8.603	12.635	145.1	132.3	6.23	.306	84.0	0	.399	80	.318			
57	5	1.13	35	100	8.760	12.718	139.1	123.6	8.22	.349	65.4	.5	.436	40	.346			

NOTICE

The rest of Appendix A (pages 213-270) and Appendices B (pages 271-272), C (pages 273-316) have not been included in this copy of this Report. See page 207A for further information.

APPENDIX D
EVALUATION OF PRESTRESS

APPENDIX D

EVALUATION OF PRESTRESS

A few one-dimensional compression tests were performed to evaluate the prestress induced in kneading compacted specimens of shale. Table D1 shows the specimen characteristics prior to loading, together with the estimated prestress (p_p) using the Casagrande construction. The ratio of the prestress to the compaction pressure (p_p/p_c) varies from 0.49 to 1.0, depending on the initial condition of the specimen. The e-log p curves are shown in Figures D1 to D7. This ratio is based on total stresses.

Specimens C1 and C3 to C6 were loaded and unloaded incrementally. The elapsed times under successive loads are given in Table D2. All the specimens had a diameter of 4 inches and an initial height of 1.5 inches.

Specimens CR2 and CR3 were loaded at a constant rate of deformation of 0.0025 inches per minute. These specimens had a diameter of 4 inches and an initial height of 1.675 inches.

Due to the gradual change in the curvature of the e-log p relationship, the evaluation of the prestress by the Casagrande method presents some difficulties. Further, the Casagrande method is strictly applicable for determining the preconsolidation pressure of saturated clays. The laboratory tests simulated the process by which the void ratio of the clay is changed in the field; namely, the

TABLE D1
Summary of Sample Characteristics and Compaction Prestress

Sample No.	Aggregate		Compaction*		As-Compacted Condition				Prestress	Ratio
	Grad	w _l (%)	P _c (psi)	W _a (gm)	γ _d (pcf)	w _c (%)	S _r (%)	e _o	P _p (psi)	$\frac{P_p}{P_c}$
C1	7	2.23	200	40	128.0	10.39	81.3	.355	126	0.63
C3	5	2.69	100	30	127.6	8.85	68.5	.359	74	0.74
C4	5	2.69	100	30	128.0	8.85	69.2	.355	74	0.74
C5	5	2.69	200	30	133.0	8.85	81.0	.304	122	0.61
C6	5	2.22	50	30	123.3	8.32	56.8	.407	50	1.00
CR2	5	2.22	100	40	130.3	10.36	86.9	.332	68	0.68
CR3	5	2.14	200	40	131.5	10.30	89.6	.320	98	0.49

*Two layers each 500 gm; 30 tamps/layer; 30 tamps/min.

TABLE D2
Elapsed Times In Minutes Under Successive Loads In Oedometer Tests

Sample No	Load on Specimen (kg)													Remarks
	10	20	30	40	80	120	160	320	480	640	1280	1920	2560	
C1	38	956	-	18	79	-	203	75	-	77	1243	5890	-	Loading
	-	-	-	-	-	-	-	884	5	15	12	0	-	Unloading
	-	-	-	-	-	-	-	0	240	1045	360	1015	-	Reloading
C3	10	10	-	10	10	-	10	10	-	10	10	10	10	Loading
C4	10	10	-	10	10	-	10	10	-	10	10	10	-	Loading
	1	1	1	3	1	1	1	1	1	1	1	0	-	Unloading
C5	10	10	-	10	10	-	10	10	-	10	10	5	10	Loading
	1	1	1	1	1	1	1	1	1	1	1	1	0	Unloading
C6	10	10	-	10	10	-	10	10	-	10	10	-	-	Loading
	1	1	1	1	1	1	1	1	1	1	1560	-	-	Unloading

expulsion of water accompanied by an increase in effective stress. It is also presumed that field consolidation takes place under K_0 conditions; i.e., no lateral deformation.

The one-dimensional compression tests reported in this Appendix were conducted on the as-compacted specimens. In Appendix E the results of two tests on shale aggregate are reported to demonstrate the collapse-on-wetting phenomenon.

Figure D8 shows the time-deformation curves for sample C4 for the time interval 0.1 to 10 minutes. The immediate deformations (0 to 0.1 minutes) under the load increments are shown in Figures D9 to D12, together with the additional deformations (0.1 to 10 minutes) under constant load. With each load increment, the void ratio of the specimen decreases. If it is assumed that the water content of the specimen remains constant, then the degree of saturation would increase and the air permeability of the specimen would tend to decrease. With progressive reduction in the air permeability, the rate of change of the immediate and the additional deformation with respect to the elapsed time decreases.

When a partially saturated soil is loaded, the reduction in void ratio is at first mainly accompanied by compression and expulsion of air. The pore water virtually does not escape because it is initially at a pressure less than atmospheric (Yoshimi and Osterberg, 1963). The pore water pressure tends to increase with increasing degree of saturation and when it exceeds atmospheric pressure, the reduction in void ratio is accompanied by the expulsion of pore water and pore air. The permeability of the specimen with respect to water is much

less than with respect to air and this explains the shapes of the time-deformation curves shown in Figure D8 for successive load increments. For the initial load increments the rate of change of deformation with respect to time was practically zero at the end of 10 minutes. For the latter load increments it is to be noted that the specimen was still deforming at the end of 10 minutes. In other words, more time is required for the specimen to come to equilibrium under a particular load increment as its void ratio decreases and its degree of saturation increases.

Yoshimi and Osterberg (1963) also found that the time rate of deformation is dependent on the load increment ratio ($\Delta\sigma/\sigma_0$). A reduction of this ratio causes a reduction in the time rate of deformation. This phenomenon is displayed in Figure D8. For a given elapsed time the deformation increments increase so long as the load increment ratio is equal to unity, but when the load increment ratio is changed to one half, there is a noticeable reduction in the deformation increments.

During the kneading compaction of shale aggregate, mechanical breakdown of the pieces takes place and the induced strains in the vicinity of the tamper shoe have both vertical and lateral components. The densification of the broken down aggregate is primarily accompanied by the expulsion of air, and the water content of the sample during compaction remains practically constant. The as-compacted sample is a three phase system comprised of solids, water and air. A fourth phase called a contractile skin is also considered to exist by Fredlund and Morgenstern (1977).

Although techniques are available for separately measuring the pore water pressure and the pore air pressure (Bishop and Henkel, 1962), such measurements will be extremely difficult to make while the sample is being compacted. Further, even if these measurements are made, either during compaction or on an as-compacted specimen, there still remains the problem of defining the effective stress in terms of the applied total stress and the measured pore water and pore air pressures. Various formulae have been proposed (Table G1) all of which contain one or more experimental parameters that are dependent primarily on the degree of saturation, S_r but they are also influenced by other factors such as soil type and the cycle of wetting or drying or stress change leading to a particular value of S_r (Bishop and Henkel, 1962).

In order to determine the void ratio-effective stress relationship, the soil must therefore be tested in either a dry or a saturated condition. For the dry condition the effective stress is equal to the total stress, and such is also the case for the saturated condition when there is no excess pore water pressure. The question now arises whether the void ratio-effective stress relationship for the dry and the wet conditions would be the same or be influenced by the presence of water. Numerous studies have shown that the relationship is usually different for fine grained soils and in many instances for coarse grained soils and rocks as well.

The evaluation of the effective prestress induced in a compacted, partially saturated soil specimen is a topic that needs to be researched. In the case of non-durable shales slaking of the shale

pieces takes place in the presence of water. Consequently, the void ratio-effective stress relationship will most certainly be different for the dry, the partially saturated, and the saturated states.

FIGURE D1 ONE DIMENSIONAL COMPRESSION (SAMPLE C-1)

FIGURE D2 ONE DIMENSIONAL COMPRESSION (SAMPLE C-3)

FIGURE D3 ONE DIMENSIONAL COMPRESSION (SAMPLE C-4)

FIGURE D4 ONE DIMENSIONAL COMPRESSION (SAMPLE C-5)

FIGURE D5 ONE DIMENSIONAL COMPRESSION (SAMPLE C-6)

FIGURE D6 ONE DIMENSIONAL COMPRESSION (SAMPLE CR-2)

FIGURE D7 ONE DIMENSIONAL COMPRESSION (SAMPLE CR-3)

FIGURE D 8 TIME-DEFORMATION RESPONSE
IN ONE DIMENSIONAL COMPRESSION
(SAMPLE C-4)

FIGURE D 9 IMMEDIATE AND ADDITIONAL DEFORMATION UNDER LOAD INCREMENTS (SAMPLE C-6)

FIGURE D 10 IMMEDIATE AND ADDITIONAL DEFORMATION UNDER LOAD INCREMENTS (SAMPLE C-3)

FIGURE D II IMMEDIATE AND ADDITIONAL DEFORMATION UNDER LOAD INCREMENTS (SAMPLE C-4)

FIGURE D 12 IMMEDIATE AND ADDITIONAL
 DEFORMATION UNDER LOAD
 INCREMENTS (SAMPLE C-5)

APPENDIX E

ONE-DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING

APPENDIX E

ONE-DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING

When water permeates into the voids of a rock-like fill there is a tendency for the fill to collapse. In the case of a fill composed of durable rock, the collapse that takes place is mainly due to the reduction of the frictional characteristics of the material when it is wetted. Further, the long term settlement and the initial settlement are more or less equal. However when a non-durable rockfill is wetted, the ratio of the long term settlement to the initial settlement will be greater than unity due to the breakdown of the rock into smaller and smaller sizes. The phenomenon may be studied by carrying out one-dimensional compression tests followed by wetting.

Two such tests were performed on New Providence shale aggregate (P 1/2" R No. 4). Figure E1 shows the stress-deformation and collapse on wetting of a specimen having a diameter of 2.485 inches, an initial height of 2.393 inches, and an initial bulk density of 83.9 pcf. The specimen was loaded in 10 kg increments up to 80 kg. The resulting load-deformation relationship tended to become linear, and the vertical compression amounted to 1% of the initial of the initial height under 80 kg load. When water was introduced into the voids, through the base of the specimen, a very rapid collapse took place. The axial strain increased from 1% to greater than 7.5% on account of the wetting.

In Figure E2 is shown the results of a similar test but on a specimen having a diameter of 4.43 inches, an initial height of 1.5 inches and an initial bulk density of 86.1 pcf, under an initial surcharge of 10 kg. The specimen was loaded in increments of 10 kg up to a load of 140 kg. The resulting load deformation relationship was again very nearly linear, and the vertical strain amounted to 1.09% under the maximum applied load of 140 kg. When water was allowed to enter the specimen through the base, collapse took place. After a period of 7 minutes the axial strain increased from 1.09% to 12.82% on account of the wetting.

The time deformation response of this specimen for elapsed times greater than 7 minutes after wetting is shown in Figure E3. Note that the deformation continues to increase with time, rapidly at first and more slowly thereafter. The total strain increased from 12.82% after 7 minutes to 17.48% after 1559 minutes.

The one-dimensional compression test on shale aggregate followed by wetting may be used to classify shales. The modulus of compression of the shale aggregate at natural moisture content may be adopted as an indicator of the hardness of the shale. The ratio of the immediate collapse on wetting to the deformation prior to wetting may be used as an index for the change in the frictional characteristics of the shale and its susceptibility to rapid slaking on wetting. The ratio of long term collapse to the immediate collapse, may be used as an indicator of the time required for complete degradation. The ratio of total collapse on wetting to the deformation prior to wetting may be used as an index for the combined effect of water on the frictional and degradation characteristics of the shale.

FIGURE E1 ONE DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING

FIGURE E2 ONE DIMENSIONAL COMPRESSION AND COLLAPSE ON WETTING

FIGURE E3 TIME-DEFORMATION RESPONSE AFTER WETTING

APPENDIX F

X-RAY DIFFRACTION ANALYSIS OF CLAY MINERALS

FIGURE F1 UNTREATED AIR DRIED SPECIMENS

FIGURE F2 UNTREATED OVEN DRIED (550°C) SPECIMENS

FIGURE F3 SPECIMENS TREATED WITH KCL AND OVEN
DRIED AT 105°C

FIGURE F4 SPECIMENS TREATED WITH $\text{Mg}(\text{Cl})_2$ AND OVEN DRIED AT 105°C

FIGURE F5 SPECIMENS TREATED WITH GLYCEROL AND AIR DRIED

NOTICE

Appendix G (pages 346-417) has not been included in this copy of this Report. See page 207A for further information.

COVER DESIGN BY ALDO GIORGINI