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CLASSIFICATION AND OTHER STANDARD TESTS FOR SHALE EMBANKMENTS

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C. W. Lovell

PURDUE UNIVERSITY

JOINT HIGHWAY RESEARCH PROJECT

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M. W. Oakland
C. W. Lovell

Final Report

CLASSIFICATION AND OTHER STANDARD
TESTS FOR SHALE EMBANKMENTS

To: H. L. Michael, Director
Joint Highway Research Project

February 2, 1982

Project: C-36-5L

From: C. W. Lovell, Research Engineer
Joint Highway Research Project

File: 6-6-12

Attached is the Final Report on the HPR Part II Study titled "Design and Construction Guidelines for Shale Embankments". The report is entitled "Classification and Other Standard Tests for Shale Embankments", and is authored by M. W. Oakland and C. W. Lovell of our staff.

The report recommends the adoption of Franklin's classification system, which is based upon 3 simple tests. In addition it proposes standards for 7 shale tests ... sufficient for the immediate needs of the Indiana Department of Highways in the design and construction of shale embankments.

The report is submitted for review, comment and acceptance in fulfillment of the referenced HPR study.

Respectfully submitted,

CW Lovell

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Research Engineer

CWL/nw

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Final Report

CLASSIFICATION AND OTHER STANDARD TESTS FOR SHALE EMBANKMENTS

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Project No.: C-36-5L

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Prepared for an Investigation

Conducted by the

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16. Abstract
Previous research has resulted in eight interim reports. This document summarizes the earlier work and constitutes a final report of the project.

It is concluded that the Franklin classification system should henceforth be used for Indiana shales; this system uses the slake durability test, in combination with the Atterberg plasticity index for the softer shales, and in combination with the point load strength for the harder ones. Standard test procedures are written for the above tests, as well as for the five-cycle slaking test, which can be used in place of the slake durability one.

The other standard test procedures contained herein are: compacted moisture-density relations; compaction-degradation; compressibility, for prediction of settlement; and saturated consolidated undrained shear tests for estimation of stability. Each test is accompanied by sample data, calculations and plots.

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LIST OF ABBREVIATIONS AND SYMBOLS

A:	weight of container + wet shale; also, Skempton's A pore pressure parameter
AASHTO:	American Association of State Highway and Transportation Officials
ASTM:	American Society for Testing and Materials
B:	weight of container + oven-dry shale; also, Skempton's B pore pressure parameter
C:	weight of container; also, degree Centigrade
C_c:	compression index
C_r:	recompression index
C_s:	compressibility of soil skeleton
C_w:	compressibility of water
C_u:	coefficient of uniformity
$\overline{CIU}$:	isotropically consolidated undrained triaxial test, with pore pressure measurement
CBR:	California Bearing Ratio
c_v:	coefficient of consolidation
c:	total stress strength intercept
c':	effective stress strength intercept
cm:	centimeter
cu.:	cubic
D:	maximum aggregate size
D_{10}:	size of 10% passing
D_{60}:	size of 60% passing
d:	initial distance between platens; also, sieve size opening
d_f:	deformation to failure

$\overline{d}$:	mean grain size retained on sieve size d
E_f:	secant modulus to failure
e:	void ratio
F:	compressive load at failure; also, degree Fahrenheit
ft:	foot
G_s:	specific gravity of solids
g:	gram
H:	maximum drainage distance
$I_d(2)$: also $(I_d)_d$	slake durability index, second cycle, dry sample
$(I_d)_s$:	slake durability index, second cycle, soaked sample
I_p:	Atterberg plasticity index, also, point load strength index
$I_p(50)$:	corrected point load strength
I_s:	modified soundness index; also, slaking resistance index, percentage
IC:	index of crushing
in.:	inch
k:	coefficient of permeability
kg:	kilogram
kN	kiloNewton
LIR:	load increment ratio
lb:	pound
m:	meter
ml:	milliliter
mm:	millimeter
MPa:	MegaPascal

N:	Newton
No.:	number
n:	porosity, also exponent
P:	percentage, by weight, finer or passing a sieve size
P_2:	percentage, by weight, finer than the sieve size above size d
P_{RI}:	percentage, by weight, retained on sieve size d before compaction
P_{RF}:	percentage, by weight, retained on sieve size d after compaction
p:	total stress level = $\frac{1}{2}(\sigma_1' + \sigma_3')$
p_f:	p at failure
p':	effective stress level = $\frac{1}{2}(\sigma_1' + \sigma_3')$
p'_f:	p' at failure
psi:	pound (force) per square inch
q:	stress level = $\frac{1}{2}(\sigma_1 - \sigma_3) = \frac{1}{2}(\sigma_1' - \sigma_3')$
q_f:	q at failure
R:	Franklin rating
rpm:	revolution per minute
S_f:	final degree of saturation
S_o:	initial degree of saturation
s:	second
T:	time factor
t:	ton (long or short?)
t_f:	time to failure
u:	pore pressure
u_o:	required back pressure
V:	volume

V_F:	final volume after saturation
V_I:	initial volume before saturation
V_V:	volume of voids
V_{WI}:	volume of water in
V_{WO}:	volume of water out
W_F:	weight of container + oven-dry material retained on No. 10 sieve
W_T:	weight of container + sample at natural water content
W_s:	weight of solids
W_w:	weight of water
W_1:	weight of drum + oven-dry sample before 1st cycle of slake durability test
W_I:	weight of drum + oven-dry sample
W_d:	weight retained on sieve size d
W_t:	total weight of the compacted material
w:	water content, percentage
Δ:	change in a quantity
ε_a:	axial strain, percent
ϕ':	effective stress strength angle
ρ_d:	dry density
ρ_m:	wet density
ρ_w:	density of water
Σ:	summation
σ_c:	applied total vertical stress
σ_1:	major principal total stress
σ_3:	minor principal total stress

σ_1': major principal effective stress

σ_3': minor principal effective stress

HIGHLIGHT SUMMARY

Economic considerations often dictate the use of shales in embankments. Due to the nature of some shales, however, the embankment may deteriorate with time. Typically, these shales are compacted in thin lifts as if they were soil. This not only reduces the deterioration of the embankment, but improves its stability and settlement characteristics. The research described in this report defines a series of laboratory tests and a numerical classification system to be used to predict the performance of shales as embankment materials.

The testing procedures are of two types. First are those which are used to classify the shales as to their hardness and durability. Shales which are soft and/or non-durable are termed soil like, while those which are hard and durable are called rock like. The recommended tests for classification are the Atterberg limits, five-cycle slake resistance, slake durability, and point load strength.

The second type of testing is to determine the properties of the compacted, soft and/or non-durable shales. Two tests are used to evaluate the compaction properties of the shale. These are the compaction-degradation test and the moisture-density relations test. Settlement is modeled by a one-dimensional compression test, and an isotropically consolidated undrained triaxial test is selected to determine the shear strength of compacted shales.

A standard procedure and example is presented for each of these tests. A classification system based on the first group of tests is also recommended.

INTRODUCTION

During the building of the modern interstate system many of the embankments were constructed of shale. This was unavoidable because of the common occurrence of shale near the earth's surface. In cases where the shale was hard it was often placed as a rock fill for economic reasons. This means that large fragments of rock were placed in thick lifts by being dumped from trucks and compacted minimally. A problem developed in that these shales were sometimes non-durable. Many cases can be cited in the literature of excessive settlements and sometimes slope failures of shale embankments which had degraded over time. The particular case which brought this problem to light in Indiana was a major slope failure on I-74 near St. Leon in Dearborn Country (14).

Slaking seems to be the principal mechanism responsible for the shale's degradation. Basically, the problem is one of the large fragments breaking apart and falling into the large voids which exist in a rock fill. This results in large volume changes. The mechanisms of slaking are not completely understood, although several extensive studies have been made. A recent study by McClure (29) suggests that osmotic swelling and/or hydration of ions and surfaces appear to be the major forces in the slaking of natural shales.

To aid in the repair of existing embankments and the construction of new ones, several agencies sponsored studies to develop design and construction criteria for shale embankments. One of the largest of these was conducted at Purdue University, administered by the Joint Highway Research Project and funded by the Indiana Department of Highways and the Federal Highway Administration. This final report summarizes the previous interim reports, and recommends standard procedures for conducting tests required for the design and construction of shale embankments.

All tests are based on those proposed by the previous researchers. Each meets the criteria of consistency, reproducibility, simplicity, sensitivity, and correlation with other tests. The tests provide parameters for the purposes of classification, compaction and degradation control, settlement estimation, and slope stability calculation. The tests selected are: Atterberg limits, five-cycle slaking, slake durability, point load strength, compaction-degradation, moisture-density, one-dimensional compression, and isotropically consolidated undrained triaxial shear.

BACKGROUND

Research on shale has continued for more than a decade at Purdue University, resulting in eight reports and numerous papers. These works have contributed greatly to the understanding of shales which are to be used in compacted embankments, and are the major source of information for this final report. The areas of study have been data collection and classification, degradation and compaction, compressibility, shear strength and slope stability, and stabilization. A brief review of the previous work is given below.

Data Collection and Classification

As was known at the onset of this project, non-durable shales must be compacted. The identification of such shales is the first step in designing an economical embankment. Because of the difficulty in sampling the shales and because the properties are often quite different between strata (or even laterally within the same stratum), it is often necessary to test the shale as it is encountered in the cut. To avoid delays in construction, the long-term durability of the shale must be determined quickly. There are two methods of prediction. The first is to correlate the performance of the shale to its response in simple tests. This was attempted by Deo (14) and later reviewed by Chapman .12). The second method is to establish a data bank and correlate durability to easily identifiable factors such as physiographic unit or geologic member. This was done in part by van Zyl (42).

Deo (14):

Deo's primary objective was to develop a shale classification system for highway embankments. His research consisted of a literature search, collection of shale samples, and a testing program.

Deo considered at least 24 potential sampling sites; of these, 14 were actually sampled. The sample size ranged from 150 to 1500 pounds depending on the ease of sampling. All samples were obtained from open cuts. At attempt was made to retain the natural water content of the shale during transportation and storage.

Next, Deo examined a battery of tests on the shales to identify those useful in contrasting hardness and durability. The types of tests considered were: weathering/degradation, identification, compaction, load-deformation, and miscellaneous.

The weathering/degradation tests are a measure of the durability of the shale in the service environment. These consisted of simple slaking tests in air and water, mechanical abrasion tests (such as the slake durability), modified soundness, and modified abrasion.

Identification tests were conducted on finely disaggregated shale. The tests conducted were the Atterberg limits, grain size distribution, and X-ray diffraction. A third test set measured certain engineering properties directly. These consisted of moisture-density relations, CBR values for both as-compacted and soaked samples, and determination of swell on wetting. The miscellaneous tests included absorption-time characteristics, in situ bulk unit weight, and certain breaking characteristics of the shale.

Deo attempted to correlate the results of various tests using linear and quadratic regression models. A summary of his conclusions is given in Table 1, and his recommended classification system is contained in Figure 1, where:

$(I_d)_d$ = Slake durability index (second cycle) for dried samples;

$(I_d)_s$ = Slake durability and index (second cycle) for soaked samples; and

I_s = Modified soundness index.

Chapman(12):

Chapman made a comparative study of several classification systems and other shale related tests. The classification systems compared were by Deo, Gamble, Morgenstern and Eigenbrod, and Saltzman. The tests which were reviewed were the Washington degradation test, ethylene glycol soaking test, Atterberg limits, and tests concerning the mineralogy of the shale.

Deo's classification flow diagram is shown in Figure 1. This system classifies shales in one of four categories: rock like, intermediate 1 or 2, or soil like.

Gamble's classification system is a result of testing 120 shale samples from many areas in the United States. The factors used to classify shales for engineering purposes are slake durability and

TABLE 1: Usefulness of Various Tests, After Deo (14)

USEFUL TESTS

Slaking in Water (One Cycle)	-
Slaking in Water (Five Cycle)	
Slake Durability (Dry Sample)	
Slake Durability (Soaked Sample)	More Severe Than Simple Slaking Tests
Modified Soundness	
Compaction	Can be Correlated to Strength
California Bearing Ratio	
Bulk Unit Wt. of Chunks	
Fissility Number (Measure of Fragment Shape)	
Swell	Can be Correlated to Durability

LIMITED USEFULNESS

Atterberg Limits	Classified Highly Plastic Clay Shales
Grain Size	-
X-ray Diffraction	Identifies Montmorillonitic (Swelling) Shales

NOT USEFUL

Abrasion (Dry Sample)	-
Abrasion (Wet Sample)	Could be Related to Slake Durability (Soaked)
Absorption	-

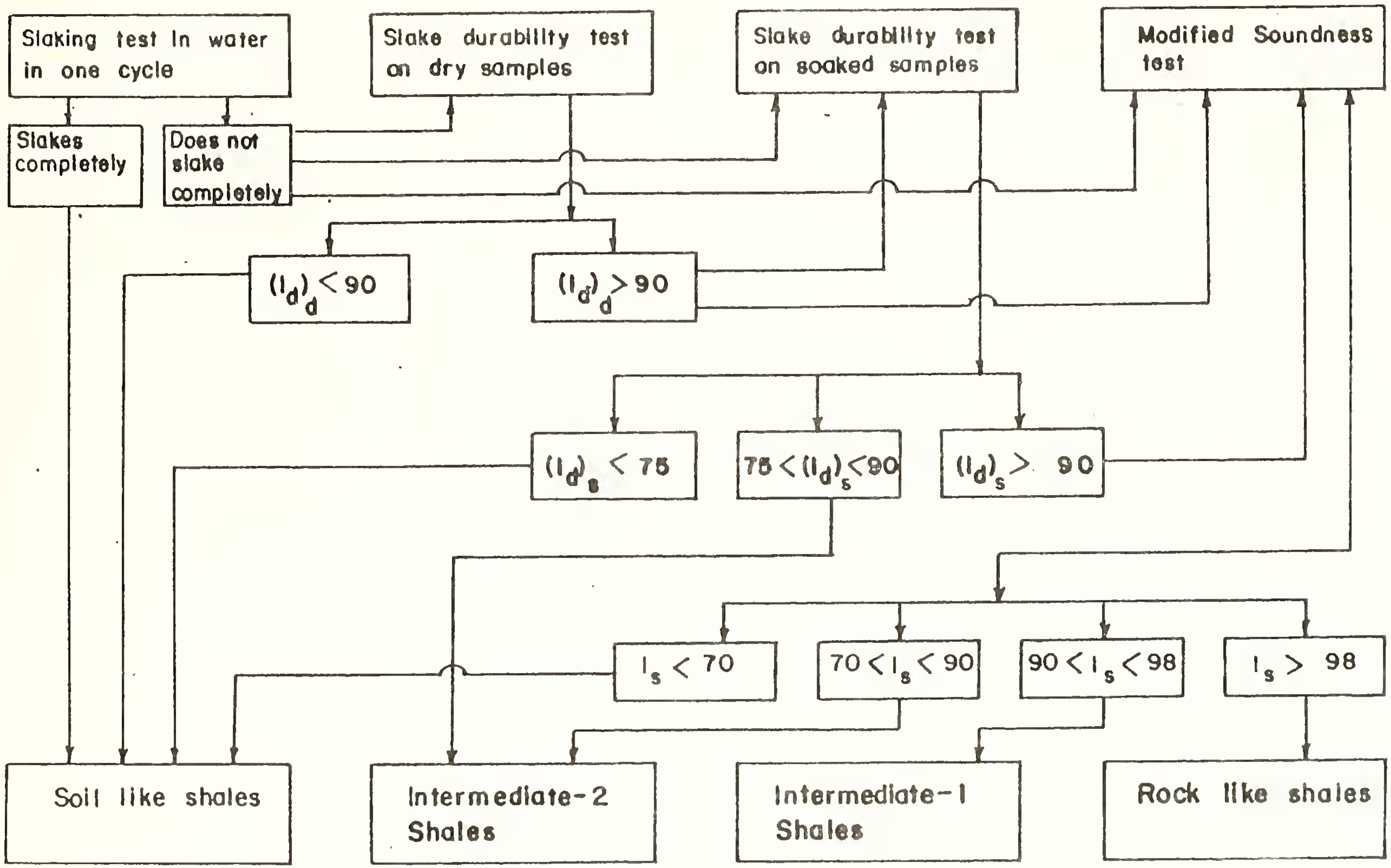

FIGURE 1. PROPOSED CLASSIFICATION OF SHALES FOR EMBANKMENT CONSTRUCTION. FROM DEO (14)

Atterberg limits. A simple grid is used to plot the combination of slake durability index and plasticity. The position on this grid determines the constructional properties of the shale. The more durable, lower plastic shales are preferable as fill material.

A similar classification system was developed by Morgenstern and Eigenbrod which was based on the Atterberg limits and their own rate of slaking test. Again, a simple grid is used to plot the shale and determine its classification. As with the Gamble system, the more desirable shales slake less and have a low plasticity.

Finally, Saltzman at Purdue University developed a classification system for rock which is to be used as rip-rap or other similar purpose. The tests which analyze the rock are the Los Angeles abrasion, ultrasonic cavitation, and Schmidt rebound hammer. This system was not specifically designed for shale and proves to be too severe in most cases.

Chapman made no direct comparison of the different classifications. However, he stated that no Indiana shales had even been classified as "intermediate 1 or 2" by the Indiana Department of Highways.

van Zyl (42):

The Indiana Department of Highways and Purdue University have, for a variety of reasons, collected a large amount of data during the testing of Indiana shales. A statistical study of this data was made by van Zyl and a computerized data bank was organized.

van Zyl reviewed both the geology of Indiana shales and the physiography of Indiana. Great benefits are possible if correlations can be made between the shales of a certain age or area and their engineering properties.

In conducting the statistical analysis of the data, van Zyl used frequency analysis, bivariate correlation analysis, and multiple regression analysis. The results indicated that good correlations may be possible, but more data are needed. van Zyl has indicated a need for a battery of standardized tests so that the data can be more easily compared.

Degradation and Compaction Tests

Shales can vary from being moderately hard to quite soft. It is not surprising that the latter group should act like lumps of clay soil. The moisture-density relation for soft shale is similar to that of a clay. The effect of moisture becomes smaller as the harder shales are tested. A second factor, i.e., degradation, controls the degree of compaction in these cases. Little work is needed to adapt the standard moisture-density tests to soft shales; however, degradation of harder shales under compaction had been little studied. This topic became the subject of reports by Bailey (5) and Hale (18).

Bailey (5):

Bailey studied the degradation which occurs during the laboratory compaction process and methods which can be used to predict it. Degradation during embankment compaction is desired, because it reduces the settlement produced by slaking of the embankment shale.

In order to determine the amount of degradation which occurs, a suitable measure of the gradation is needed. Bailey reviewed commonly used gradation measures and selected two: the aggregate gradation modulus and the index of crushing.

Four types of compaction were tested: kneading, gyratory, static and impact. Bailey found that the static compaction yielded the most consistent results and gave the best correlations between effort, aggregate degradation, and compacted unit weight. Bailey also found that it was simple, inexpensive, rapid and required no special equipment. For these reasons he recommended that degradation tests be performed using static compaction.

Since it is apparent that the degradation which occurs is related to the hardness or the strength of the shale, its evaluation would be simplified if a correlation could be found with a simple hardness or strength measure. Bailey investigated the scleroscope hardness and the point load strength tests for this purpose. A large amount of data scatter was prevalent in both tests, but both showed promise of producing satisfactory correlations, with further testing.

Hale (18):

Hale extended the work of Bailey (5) and developed a standard compaction-degradation test for shales. His logic was much the same as that of Bailey.

Hale used three hard but nondurable shales. As possible compaction methods, Hale chose impact and static. For each type of compaction four levels of effort were tested. Hale found the advantages of the static method to be simplicity and the ability to measure the compactive work by product of force and residual deformation. The advantage of the impact method is its wide acceptance as a standard laboratory compaction test. Hale's recommendation was to use the impact method. The effort which was found to be best suited for general testing was 861 kN-m/m^3 (18,000 ft-lb/ft^3). This is achieved by compacting 1/13.33 ft^3 of shale in three layers using 25 blows per layer from a 4.54 kg (10.0 lb) hammer free falling 45.7 cm (18.0 in).

Also studied were the effects of the initial gradation and maximum aggregate size on degradation and dry density. Based on data from three initial gradations and two maximum sizes, it was found that a good range of results could be obtained from using 38.1 mm (1.5 in) as the maximum size and an initial gradation of:

$$P = (d/D)^1$$

where

P = percent, by weight, passing any sieve size,

d = sieve mesh opening, and

D = maximum aggregate size.

Hale continued the research to study the effect of moisture on degradation and dry density. This is useful because the application of water to aid compaction is a common construction practice.

Typical soil-like behavior was observed for the relationship between moisture content and compacted dry density. There is an increase in dry density with increasing moisture content up to a specific point, then the inverse becomes the case. The relationship was found to be shale specific. The degradation also increased with increasing moisture content, but seemed to approach some limiting value.

Surendra (40):

The hypothesis for Surendra's work was that it is possible to control the slaking of hard non-durable shales, which are difficult to mechanically degrade, through the use of additives. These additives would be mixed with the compaction water and their purpose would be either: (1) to slow the slaking process or (2) to accelerate slaking. In the former case, the shale might be placed as a rock fill. In the latter, the shale could be more easily compacted into a soil fill.

The additives tested were selected inorganic salts and lime. The tests used to evaluate the effect of these additives were the slaking index, slake durability, point load strength, one-dimensional collapse, pore size distribution, compaction, and unconfined compression.

The results were found to be shale specific; for example, 60 days of curing in a 3% lime solution greatly improved the durability of New Providence shale. Effects on the Osgood shale were quite minor.

It was found that sodium chloride, calcium sulfate, and ferrous sulfate improved the durability of New Providence shale, whereas aluminum sulfate reduced it. With Mansfield shale, calcium sulfate, aluminum sulfate, and ferrous sulfate improved the durability, whereas ferric chloride decreased it.

Compressibility and Shear Strength

The service performance of a shale embankment, like any embankment, is primarily controlled by its settlement and slope stability. Modified consolidation and undrained triaxial tests were used to determine the properties of a compacted shale. Abeyesekera (1) and Witsman (44) investigated these properties, for a representative Indiana shale.

The material studied was New Providence shale. It is Mississippian in age and from the Borden series. The shale contains illite, chlorite, kaolinite, quartz, and feldspar, and is commonly sandy and silty in texture.

Abeyesekera (1):

The primary purpose of the Abeyesekera study was to develop laboratory testing techniques to evaluate the strength parameters of a compacted shale. The effective stress parameters were considered to be most appropriate, and so the tests were conducted as consolidated undrained ($\overline{CIU}$) triaxial compression tests on saturated samples. A second objective was a study of the factors which influence the compaction characteristics, consolidation characteristics, and the stress-pore pressure-strain behavior during undrained shear.

Because of the relatively large aggregate sizes which occur in actual compacted shale embankments, it was desirable to use as large a triaxial specimen as possible. A 101.6 mm (4.0 in.) diameter specimen size with a nominal height of 215.9 mm (8.5 in.) was used. All compacted samples were saturated using deaired, distilled water and a back pressure exceeding 50 psi. All samples were consolidated isotropically and sheared undrained at a constant rate of strain to failure or to an axial strain of 20%. The testing variables included the following: six levels of gradation, three levels of compactive effort, four levels of added molding water, two techniques of saturation, and six levels of consolidation pressure.

A summary of the results of the New Providence shale yielded a $c' = 1$ to 2 psi and a $\phi' = 28$ to 30 degrees for all compacted specimens, and a $c' = 0$ and $\phi' = 25$ degrees for loose specimens. In addition, Skempton's "A" parameter at failure varied from 2.2 to -0.4, and decreased with increasing compactive prestress.

Witsman (44):

Abeyesekera (1) conducted a few one-dimensional consolidation tests to determine the prestress which was obtained during the compaction of the shale. Witsman expanded this work to study the effects of compaction variables on compactive prestress. In addition, he also studied the swelling or settling characteristics of compacted shale when saturated under load.

The testing by Witsman used standard 101.6 mm (4.0 in.) diameter oedometer cells. The specimens were compacted using kneading compaction to simulate field conditions. The testing variables were: three levels of compactive effort, three levels of compacted moisture content, as-compacted and saturated moisture content levels during testing, two levels of load increment ratio, and three levels of surcharge at the time of saturation.

Witsman's study showed that the prestress obtained during compaction was equal to the nominal compactive pressure for shales compacted with low to intermediate effort at or dry of optimum moisture content. For all other cases the prestress obtained was less than the compactive pressure. The tests involving saturation of the compaced shale showed that the shale's tendency to heave or settle is controlled by the initial as-compacted conditions and the confining stress at the time of saturation. Swelling was found to be more likely with increasing dry density and/or decreasing confining stress.

A statistical analysis revealed that the factors which have the greatest effect on the value of the prestress are the nominal compactive pressure and the compaction moisture content. Models are given to estimate the compactive prestress, and volumetric strain upon saturation, for samples of compacted New Providence shale.

A summary of all research (including this study) is shown in Figure 2.

Deo (14)	Useful tests and a classification system for determining shale behavior as an embankment material.
Chapman (12)	A comparative study of certain tests and classification systems by Deo, Gamble, Morgenstern and Eigenbrod, and Saltzman for determining shale behavior.
Bailey (5)	A study of field and laboratory shale degradation due to compaction and its relationship to point load strength.
van Zyl (42)	A statistical analysis of existing shale data and a storage and retrieval system for these data.
Abeyesekera (1)	A study of the stress-deformation and strength characteristics of a compacted shale using triaxial testing.
Witsman (44)	Determination of the effect of compacted prestress on compacted shale compressibility.
Hale (18)	The development and application of a standard compaction-degradation test for shales. Correlation with point load strength.
Surendra (40)	A study of additives which can be used to control slaking in compacted shale embankments.
Oakland	A battery of tests and a shale classification system which are useful in the design and construction of compacted shale embankments.

FIGURE 2: Summary of Research Conducted at Purdue University Concerning Compacted Shale.

CLASSIFICATION SYSTEMS

Classification systems for rocks are generally derived from one of two major sources. The first are those developed by geologists. For the most part these depend on observed properties such as color, laminations, or fissility. The systems often simply use adjectives to describe the rocks. The second general type of classification system is based on the engineering properties of the rocks. Specifically for this study, systems which predict the performance of shales as embankment materials are desired. These systems are often based on tests which assess the hardness and durability of the shale.

The engineering classification system is built of two parts. The first part is a rating system. The rating system scales a particular aspect; in this study it is the performance of shales as embankment materials. The second part classifies the rating value. More specifically, it separates shales which have been performance-rated in a continuous function into a discrete number of groups. The number of groups depends on the "state of the art" of shale embankments. Until recently, it was feasible to divide the shales into only two groups: those which are to be compacted as a soil fill and those which are to be placed as a rock fill.

The most commonly used geological classification system for sedimentary rocks is shown in Figure 3. The rocks are rated by the

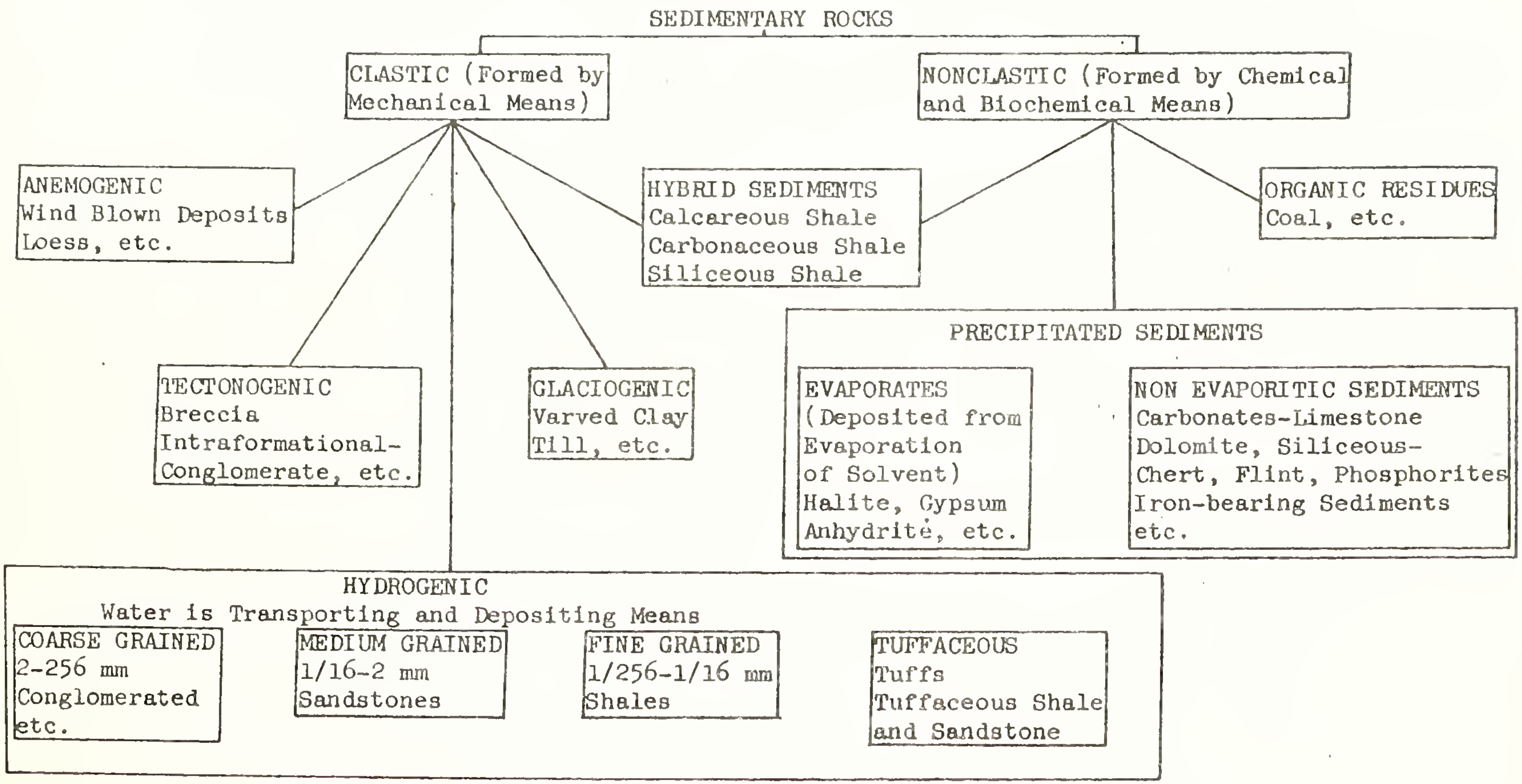

FIGURE 3: Classification of Sedimentary Rocks, After Deo (14)

amount of material deposited mechanically, versus chemically or biochemically and are classified as clastic or nonclastic. Further subdivisions are made based on certain depositional characteristics and grain size. This system is not useful in classifying shales because almost all shales fall into the same category.

A classification system dealing specifically with shales is needed. To define the systems which apply, it is first necessary to define what is a shale. Deo (14) made a summary of popular definitions of shale. These varied widely, often depending on the purposes for which they were being defined. The definition proposed by Deo and adopted for this study is:

> A shale is a sedimentary rock that: 1) is essentially insoluble, 2) is clastic or hybrid, 3) is fissile and/or laminated, 4) consists primarily of clay and/or silt, and 5) contains minerals essentially unaltered since deposition.

In some cases, engineers have been able to classify shales based on past experience and local geologic familiarity. For example, engineers who have worked in the Appalachian Plateau often know that red shales are troublesome (27). There is no substitute for experience, and if the engineer has worked with the particular shale, that work can be used as a model. This is not always possible, and a more quantitative classification system is needed.

The first generation of shale classification systems were generally based on their visual properties. Fissility is an obvious characteristic of most shales, and Alling (3), Ingram (23), and McKee and Weir (30) established scales of fissility.

Other classifications used variations in color, texture, and composition in the alternating laminae. Still another common system classified argillaceous rock by the sedimentary particle size (37). Certain adjectives have long been used to describe shales (21, 36, 41), e.g., bituminous, oil, alumn, arkosic, micaceous, chloritic, and immature.

A second wave of classification systems uses slaking behavior as the primary criterion. Such classifications are more suitable to engineering applications that those previously described.

Mead (31) separated shales into "cemented" and "compacted" categories. Compacted shales lack a significant amount of cementing agent such as calcite. The cemented shales are considered "rock like" while the compacted shales are considered "soil like". A simple slaking test is used to make the distinction. Systems proposed by Chandra (11), Deo (14), and Hudec (22) extended this concept. Chandra and Hudec use slake durability test values to define the various levels of durability. Deo used multiple slaking tests and a soundness test to classify shales. Others have combined a slaking test with some other index. Gamble (17) and Morgenstern and Eigenbrod (33) use a slaking test and the plasticity index. Gamble used the slake durability test while Morgenstern and Eigenbrod developed a rate of slaking test. Such systems are best suited to softer shales, which are easily degraded for the Atterberg limits tests (14).

Deo's system (14) is currently used by the Indiana Department of Highways. The flow chart for this system was given as Figure 1.

Shales classified by this system are soil like, intermediate 1 or 2, or rock like. Chapman (12) found a potential flaw in Deo's system in that no shales were subsequently classified as intermediate 1 or 2. This is true of all the shales tested at both Purdue University and the Indiana Department of Highways.

The most modern systems are by Franklin (16) and Strohm et al. (39). Franklin uses the slake durability test, the plasticity index, and the point load strength test. The point load index is applied to classify the more durable shales, and the plasticity index is used to rate the others.

In choosing a classification system four criteria must be met. First, the tests must be relatively simple, using only readily available and, if possible, easily portable equipment. Second, the test should be relatively rapid to permit classification soon after the shales have been excavated. Third, the system must be able to distinguish clearly among shales in the geologic population. Finally, it is essential that the classification values be quantitative and numerical.

The Franklin system seems to meet all of the above criteria. Most laboratories already have the apparatus to conduct the Atterberg limits and point load strength tests, and the equipment for the slake durability test is relatively inexpensive. As was emphasized earlier, the tests are rapid, especially if a microwave oven is used for drying. The classification scale (R value) is continuous, and is broad enough to cover all but the most exceptionally hard argillites.

The basic procedure is to analyze the shale first for its durability, and then for either its hardness or plasticity. The durability is given by the slake durability index as obtained by the second cycle of the slake durability test. This test involves tumbling a number of shale fragments in a mesh drum which is partially submerged. The index is determined as the percentage of material which is retained in the drum during the second cycle of wetting. The second cycle is used because it is a more accurate descriptor. This test is more fully described later in this report.

If the shales have a slake durability index of greater than 80.0%, they are classified by using the point load index adjusted for a diameter of 50.0 mm (1.97 in.). The index is defined as the force needed to fail a specimen by axial loading between two conical platens divided by the square of the initial distance between the platens. The point load strength test is described later in the thesis.

If the slake durability index is less than 80.0%, the shales are further classified by their plasticity index. The standard methods for calculating the plasticity index, as given by AASHTO T90* or ASTM D424-59*, are satisfactory.

By using combinations of the slake durability index and the point load index or the plasticity index, a rating number is assigned which ranges from 0.0 to 9.0. Figure 4 shows the graph Franklin developed to rate the shales. The R values on the graph can easily be interpreted to the closest 0.1.

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* Dates of latest approval are omitted for AASHTO standards, but are listed for ASTM ones. In any event, the latest standard should be used.

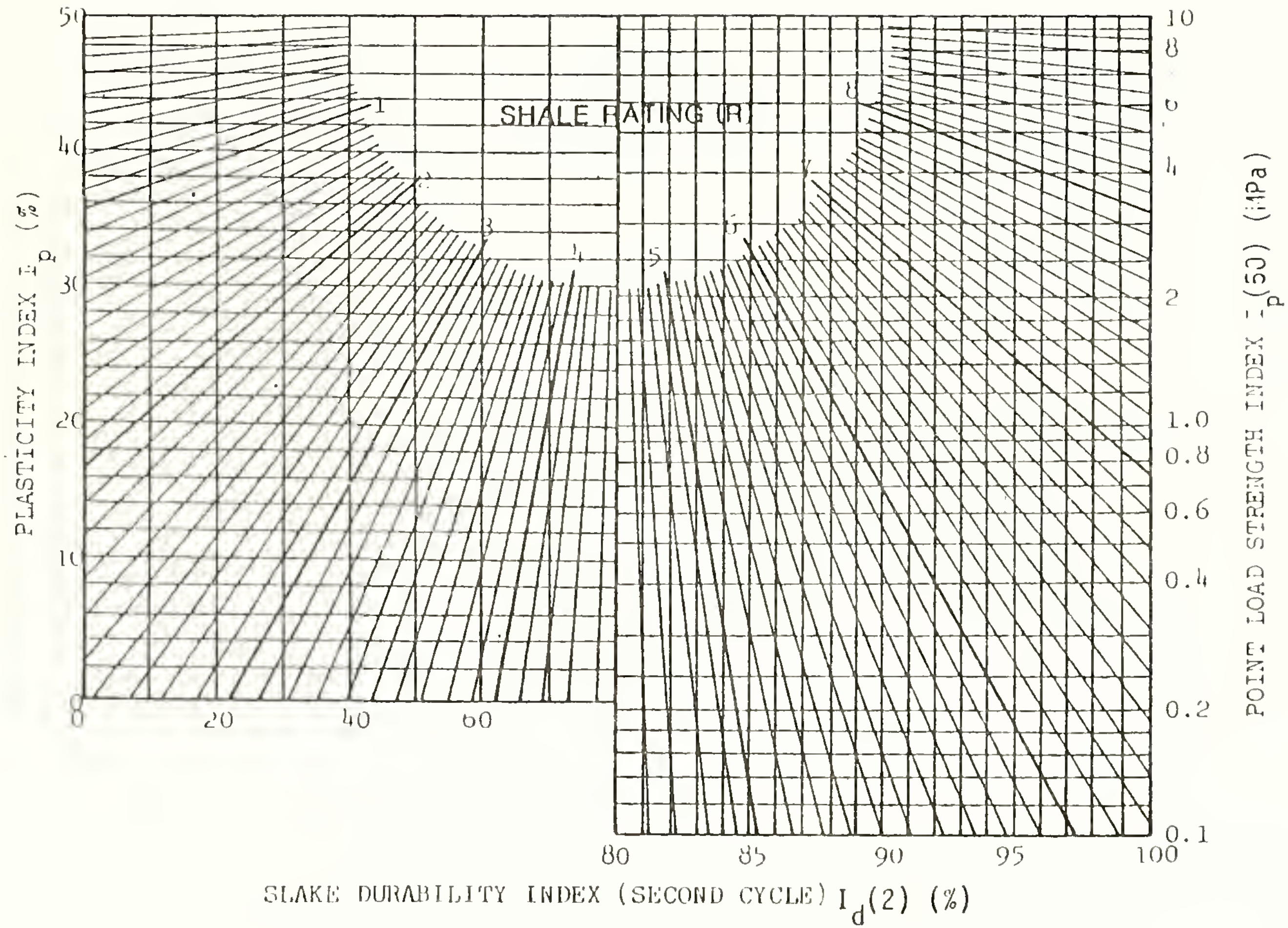


FIGURE 4: Shale Rating Chart

After Franklin (16)

Franklin (16) also gives tentative correlations between his continuous rating system and predicted engineering behavior. The primary concern in constructing a shale embankment is whether to place the shale as a rock fill in thick layers or as a soil fill in thin lifts. As shown in Figure 5, as the shale rating decreases, the thickness of the lift also decreases. The thickness tends to become constant above a rating of five. This would indicate that the Franklin system classifies shales above a rating of five as "rock like". It is interesting to note that shales with a slake durability index of greater than 85 percent will have a rating above five. Deo (14) also used an 85 percent value from the slake durability test as a criteria for "rock like" shales in his classification system.

The lower plot in Figure 5 is a tentative indicator of the dry densities which can be expected when the shale is compacted. The softer, more plastic, shales hold water very much as clays. The very hard shales retain large voids between the fragments due to the resistance to breakage. Shales with medium ratings have the largest compacted densities.

Franklin (16) also relates embankment height and slope angle to the R value (see Figure 6). As would be expected, the slope angle increases with increasing shale quality. In general the slope angle decreases with increasing height, reflecting the greater importance of stability in higher embankments. Franklin also recommends that a lesser slope angle be used in low fills. There is little added expense in widening a shallow embankment, while there is a substantial increase in driver safety and in the ease of maintenance.

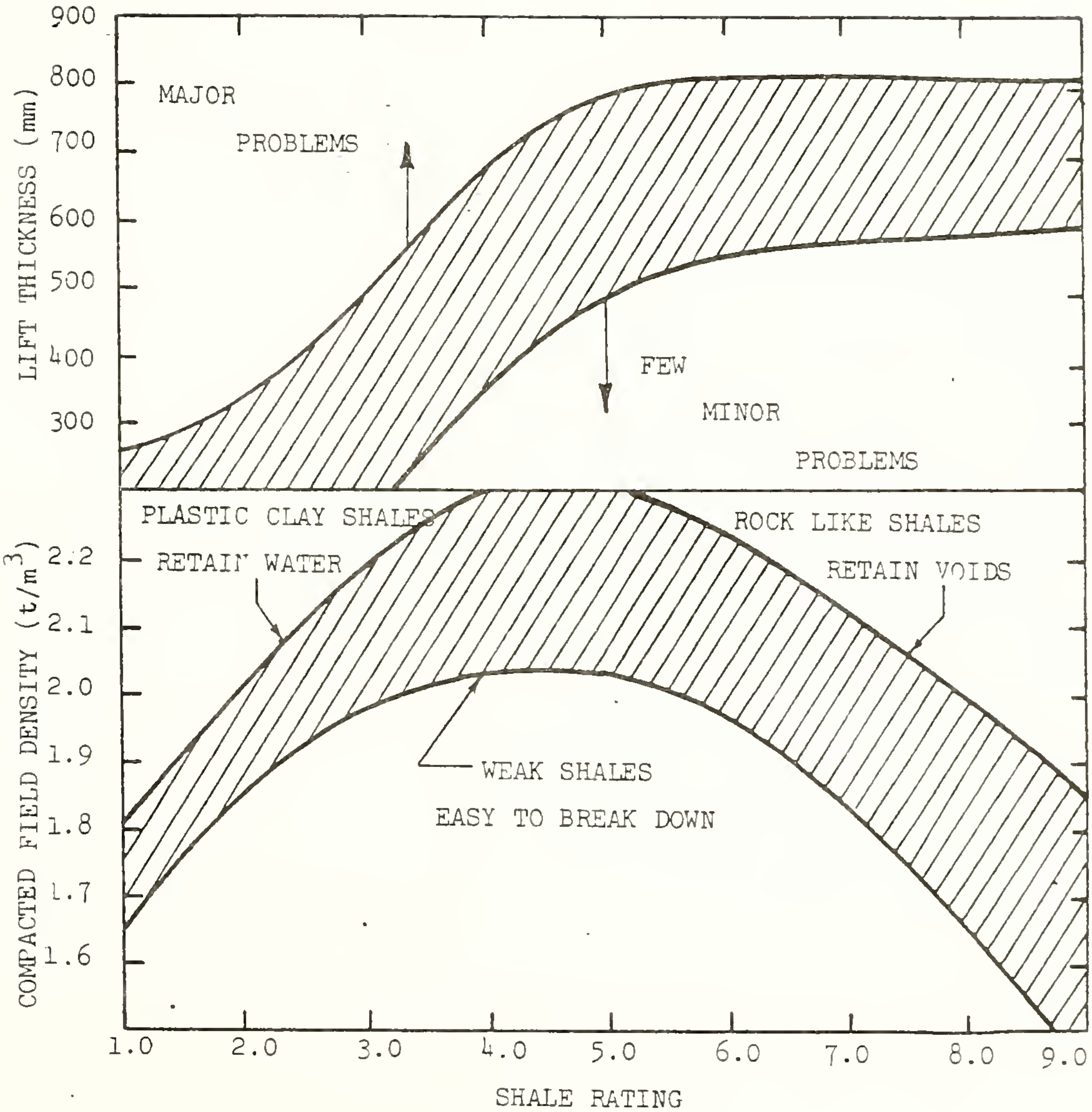


FIGURE 5: Tentative Correlations Between Shale Quality, Lift Thickness and Compacted Densities After Franklin (16)

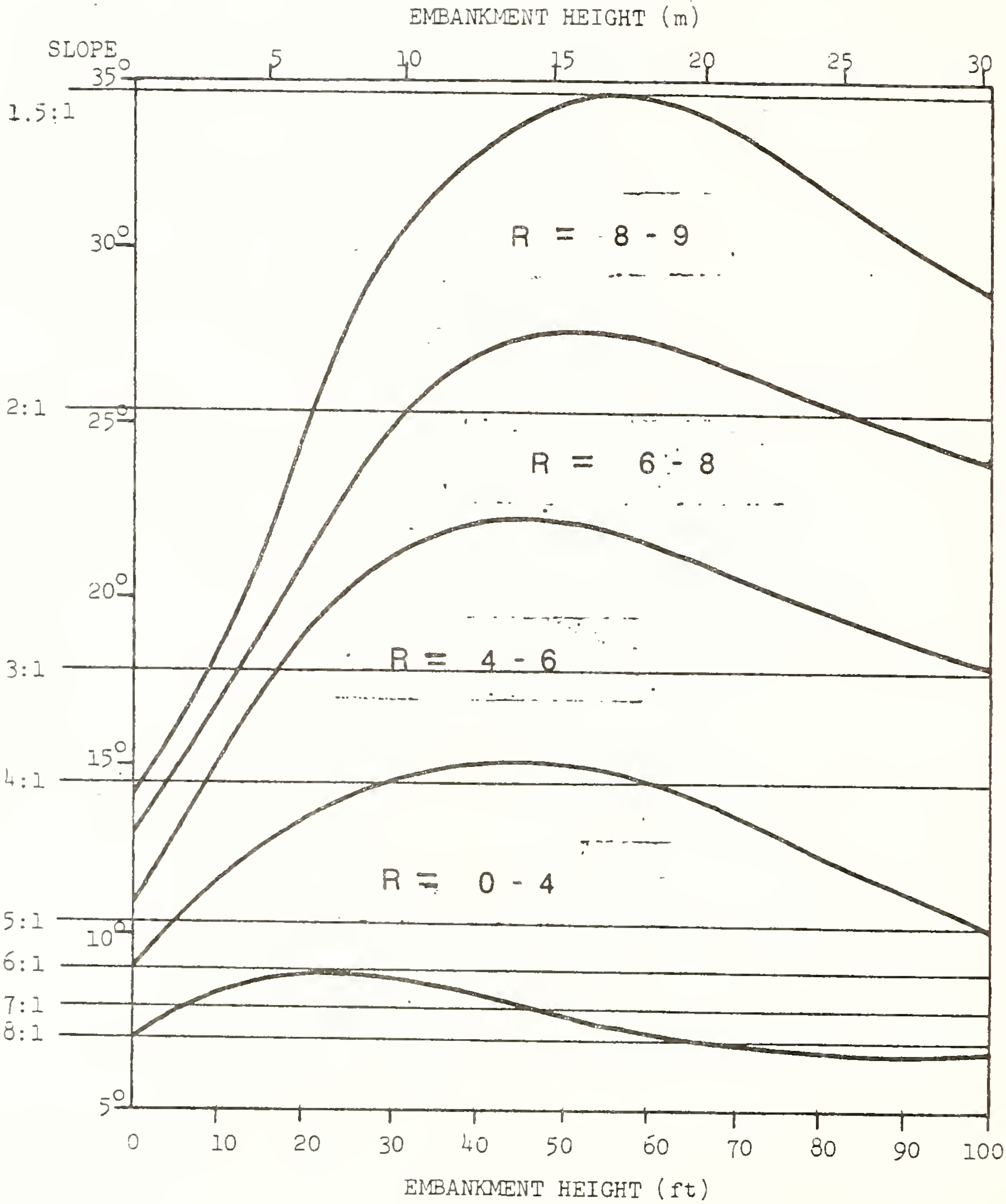


FIGURE 6: Trends in Embankment Slope Angle as a Function of Embankment Height and the Quality of the Shale Fill. After Franklin (16)

Figure 7 shows Franklin's findings of the trends in the shear strength parameters as a function of the shale rating. Typical maximum and minimum cohesion and friction angle values are given by the shaded area. Franklin notes that beyond a shale rating of 8.0, the material acts essentially as granular fill, displaying only nominal cohesion.

Estimates of the permeability of the compacted shale are given in Table 2. The values may vary greatly, especially in the lower ratings, since the amount of compaction will have a large influence. In general, the permeabilities decrease with time as the shale weathers, degrades, and better fills the voids.

Finally, several common shales of Indiana were classified by Franklin's method. These are given in Figure 8. Because the system was derived primarily for Ontario shales, which are harder and more durable than those found in Indiana, it was necessary to check the versatility of the classification. As shown, the Indiana shales are spread over an area which would basically be described as "soil like" and of a non-plastic nature. From Franklin's correlations, these shales would have to be compacted in thin lifts, and embankment slope angles would be limited. From past experience in Indiana, this is known to be the case. Accordingly, the Franklin system is deemed to be suitable in Indiana. The correlations with constructional properties (as given by Franklin) may be helpful, but can not be applied with confidence pending further study.

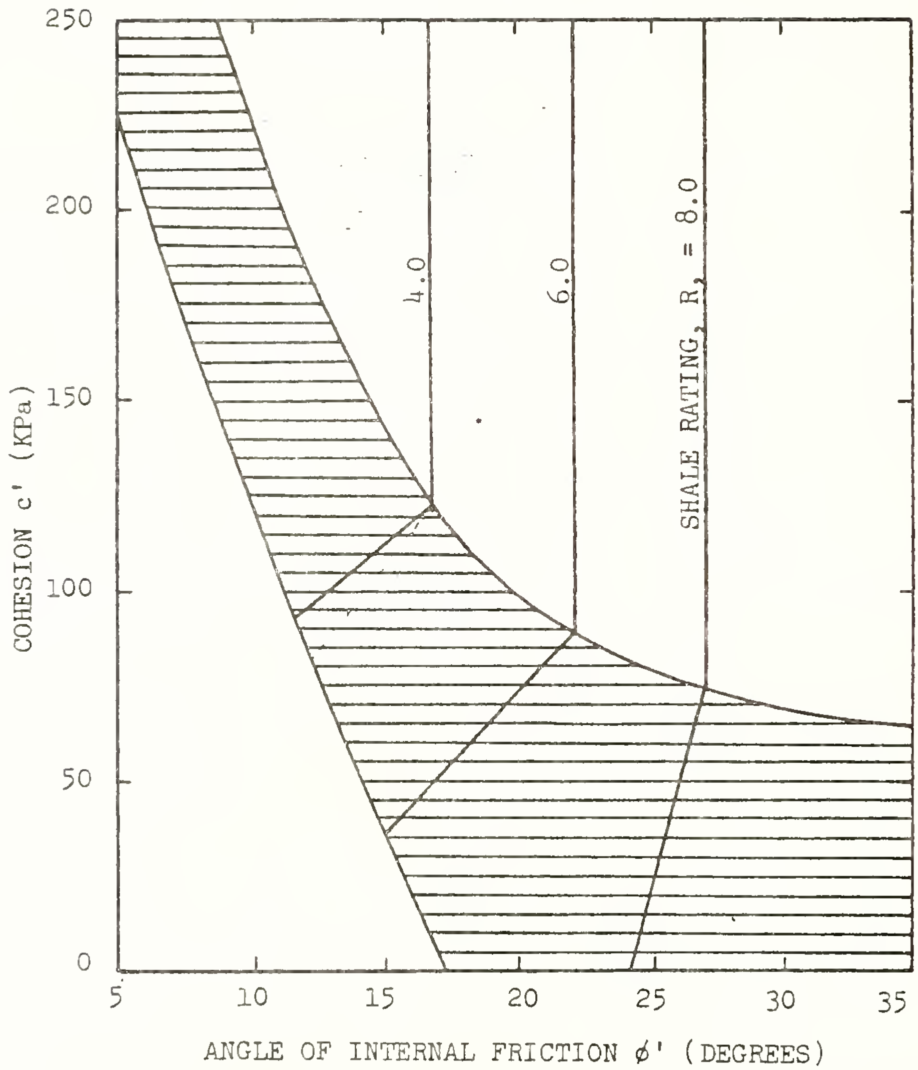


FIGURE 7: Trends in the Shear Strength Parameters of Compacted Shale Fills as a Function of the Shale Quality

After Franklin (16)

TABLE 2: Typical Permeabilities of Compacted Shale Embankments, After Franklin (16)

MATERIAL TYPE	RATING (R)	PERMEABILITY (k)
Shale rockfill	8-9	10^{-5} to 10^{-3} m/s
Durable shale fill	7-8	10^{-6} to 10^{-5} m/s
Moderately durable shale fill	5-7	10^{-7} to 10^{-6} m/s
Moderately durable shale fill	4-5	10^{-8} to 10^{-7} m/s
Well compacted clay shales	0-4	10^{-12} to 10^{-8} m/s

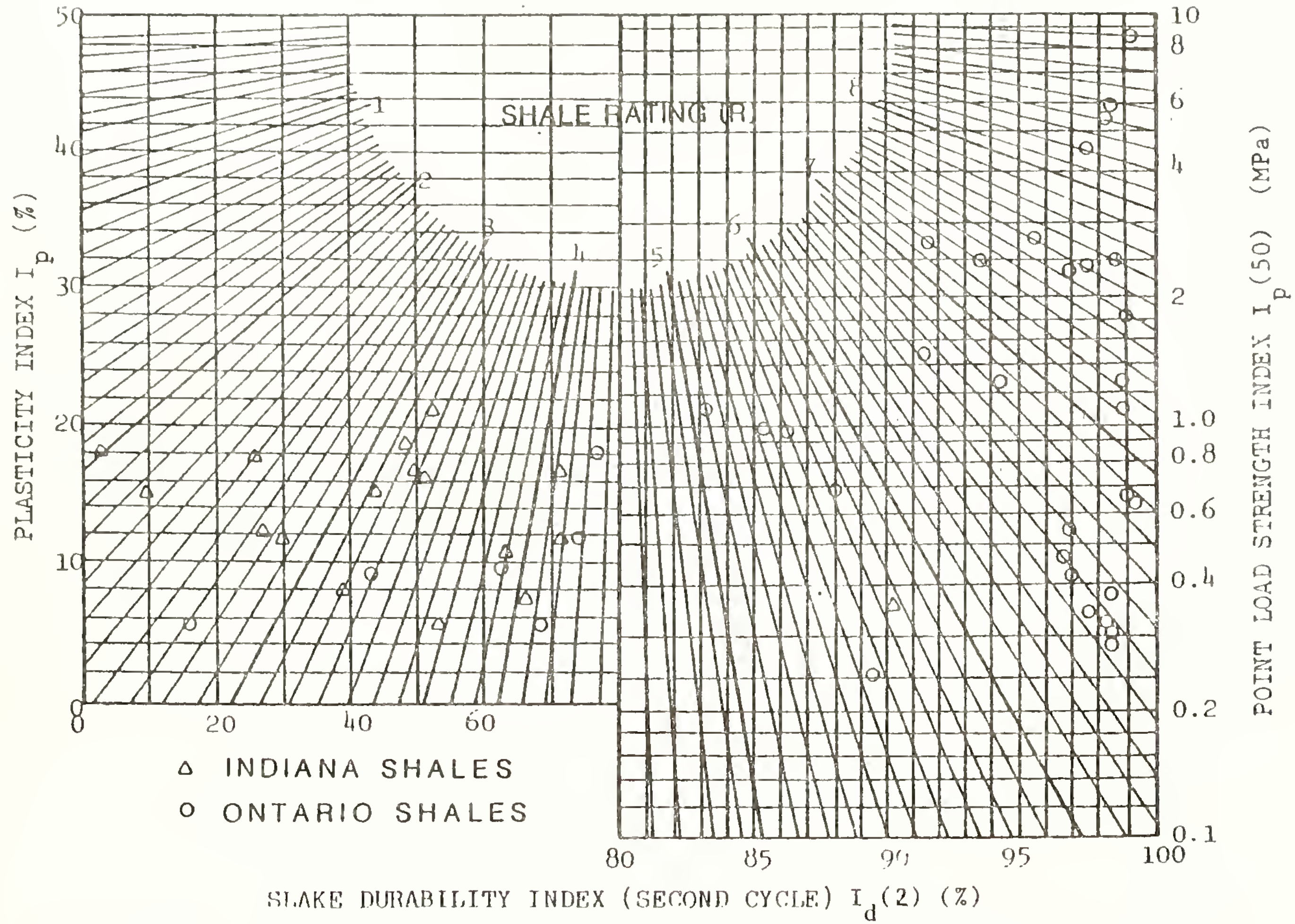


FIGURE 8: Franklin's Rating of Typical Indiana and Ontario Shales

From Abeyesekera and Lovell (2)

EXPLORATION

The basic objective in sampling shale for embankment material is not only to define the shale layers, but to obtain the quantity and quality of shale sample to run the classification and property tests. Table 3 lists the tests generally required, as well as the quantity and the minimum chunk size of the sampled material. Core boring alone may be adequate to classify the material as to hardness and durability, but the layer would have to be both thick and generously sampled to get enough material. The rounded sides of a cored sample may also reduce the abrasion in the slake durability test, and give misleading and unsafe results. In addition, it is often very difficult to obtain much intact material when coring in shale. Bailey (5) cites a case where only 6 meters (20 ft) of material was recovered from 15 meters (50 ft) or boring, and of that there was only one piece more than 8 cm (3 in.) long. Core borings can be used to define the soil and weathered shale depths, the thickness and inclination of the shale strata, and the layers draining into the shale embankment near the cut-fill transition (39). However, for classification purposes a test pit will very often be required. It is also helpful if the stratum can be traced to a nearby existing outcrop, where unweathered material can be sampled in quantity.

TABLE 3: Tests and Sampling Requirements

TEST	PURPOSE	QUANTITY	MINIMUM AGGREGATE SIZE	SAMPLING METHOD
ATTERBERG LIMITS	CLASSIFICATION	500 g	NONE	AUGER
FIVE CYCLE SLAKING	CLASSIFICATION	100 g TO 150 g	INTACT	TEST PIT POSSIBLY CORING
SLAKE DURABILITY	CLASSIFICATION	450g TO 550 g	10 PIECES 40 g TO 60 g each	TEST PIT
POINT LOAD STRENGTH	CLASSIFICATION	20 FRAGMENTS	25 mm DIA.	TEST PIT POSSIBLY CORING
COMPACTION-DEGRADATION	COMPACTION CONTROL	20 lb (9 kg)	1.5 in. (38.1 mm)	TEST PIT
MOISTURE-DENSITY	COMPACTION CONTROL	100 lb (45 kg)	1.5 in. (38.1 mm)	TEST PIT
ONE DIMENSIONAL COMPRESSION	SETTLEMENT ANALYSIS	70 lb (32 kg)	0.75 in. (19.0 mm)	TEST PIT
TRIAXIAL	SLOPE STABILITY ANALYSIS	70 lb (32 kg)	0.75 in. (19.0 mm)	TEST PIT

CLASSIFICATION AND BEHAVIOR CHARACTERIZATION

As a result of the work done at Purdue University a battery of tests has been developed which are useful in the design and construction of shale embankments. These tests are: Atterberg limits, five-cycle slaking, slake durability, point load strength, impact compaction-degradation, moisture-density relations, compressibility, and shear strength. Basically, these tests may be divided into two categories; those which classify the shale by its hardness and durability, and those which give actual design parameters. The Atterberg limits, five-cycle slaking, slake durability, and point load strength tests comprise the former, while the other tests make up the latter.

The hardness and durability of a shale determine if it shall be placed as a rock or soil fill. If the shale is classified as being both hard and durable, it can be placed in thick lifts with relatively little compaction control. If it is not both hard and durable, it must usually be thoroughly degraded and placed as a soil in thin lifts with strict compaction control.

Atterberg Limits

Although there is limited logic in applying a soil plasticity test to shale, the Atterberg limits are used in several classification systems (16, 17). Deo (14) describes the limitation of these tests for shales, while Abeyesekera and Lovell (2) recommend that they be used only for classifying relatively soft shales which are to be degraded and placed as a soil fill. Franklin (16) reinforces this idea by developing a classification system which uses the Atterberg limits only for shales with a slake durability of less than 80. Such shales are usually relatively soft, and standard testing procedures (AASHTO T90 or D424-59) are suitable.

For any hard shale, degradation of the shale to enter the Atterberg limits tests becomes a tedious, time-consuming process. The author recommends this effort be generally avoided. When this is not possible, ultrasonic devices can be used to facilitate degradations. Laguros (26) has developed a suggested procedure for use of this equipment on shales.

Five-Cycle Slaking Test

The five-cycle slaking test is an outgrowth of a slaking test which involves just one cycle of drying and wetting. The one-cycle test was found not to be severe enough to distinguish among the durabilities of various Indiana shale (14). The five-cycle test is useful in separating the "compacted" from the "cemented" shales (36), and can determine the shales which are obviously not durable when subjected to changes in water content. Cemented bonds will usually make the shales more durable (2).

The test, as originally described by Philbrick (36) and used by Deo (14), consisted of five cycles of drying of a 50 to 60 gram shale fragment for eight hours, followed by submerging the fragment in water (or another slaking fluid) for sixteen hours. As Philbrick describes the test it is qualitative. The end condition of the shale is noted as being "fully slaked", "partially slaked", or "not affected". A description of the shale fragments as to their shape and size may also be given.

Surendra (40) and Chapman (12) later used a modified version of the test where the soaking period is twenty-four hours and the drying time is at least sixteen hours. The degree of slaking was also measured quantitatively as the percentage of material able to pass a No. 10 (2.00 mm) sieve after each cycle. They recommend that the appearance of the unslaked fragments be described. However, the procedure used by Surendra and Chapman is too lengthy to be practical. The original periods of eight hours of drying and sixteen hours of soaking are recommended. It is also recommended that the slaking index be defined as the percentage of material retained on a No. 10 (2.00 mm) sieve, in order to parallel the index of the slake durability test, which will be described in detail later. Therefore, a shale which is not affected by water would be given a rating of 100 percent and a shale which totally slaked would be rated as 0.0 percent. The condition of the fragments retained should be recorded since their shape and size can be a help with the durability prediction (2).

An important aspect of the test is the size and shape of the initial shale fragment (2). In order to keep the tests as uniform as possible, the specific surface (surface area divided by the volume) of the shale samples should be as similar as practicable. Therefore, the weight of the samples should be in a constant range of 50 to 60 grams, while the shape is a roughly equi-dimensional with no protruding corners.

The advantage of the test is its simplicity. It directly evaluates susceptibility to slaking, which is the primary evidence of non-durability. The disadvantages are that: it is lengthy, involving a minimum of six days; it may not distinguish among the harder shales; and it does not model the stresses on a shale fragment confined in an embankment.

Although the correlation between this test and the slake durability test used in Franklin's rating system (16) is poor, a general conclusion that it is more severe can be drawn. This is shown by most shales plotting above the line of equality in the slake durability index vs. five-cycle slaking index plot as shown in Figure 9. It is therefore possible to make a conservative estimate of shale durability by using this test in place of the slake durability test, if equipment for the latter is not available.

A suggested procedure and an example data set are given for this test in the Appendix.

Slake Durability Test

To resolve the problems of the five-cycle slaking test, viz, the length of time needed to run the test and the lack of ability to distinguish among the harder shales, an energy input is needed. One solution is the slake durability test developed by Franklin (16). The test adds a tumbling and abrasion action to the normal slaking process. Thus, the slake durability test requires only about two days, and it is more vigorous in evaluating durability. The

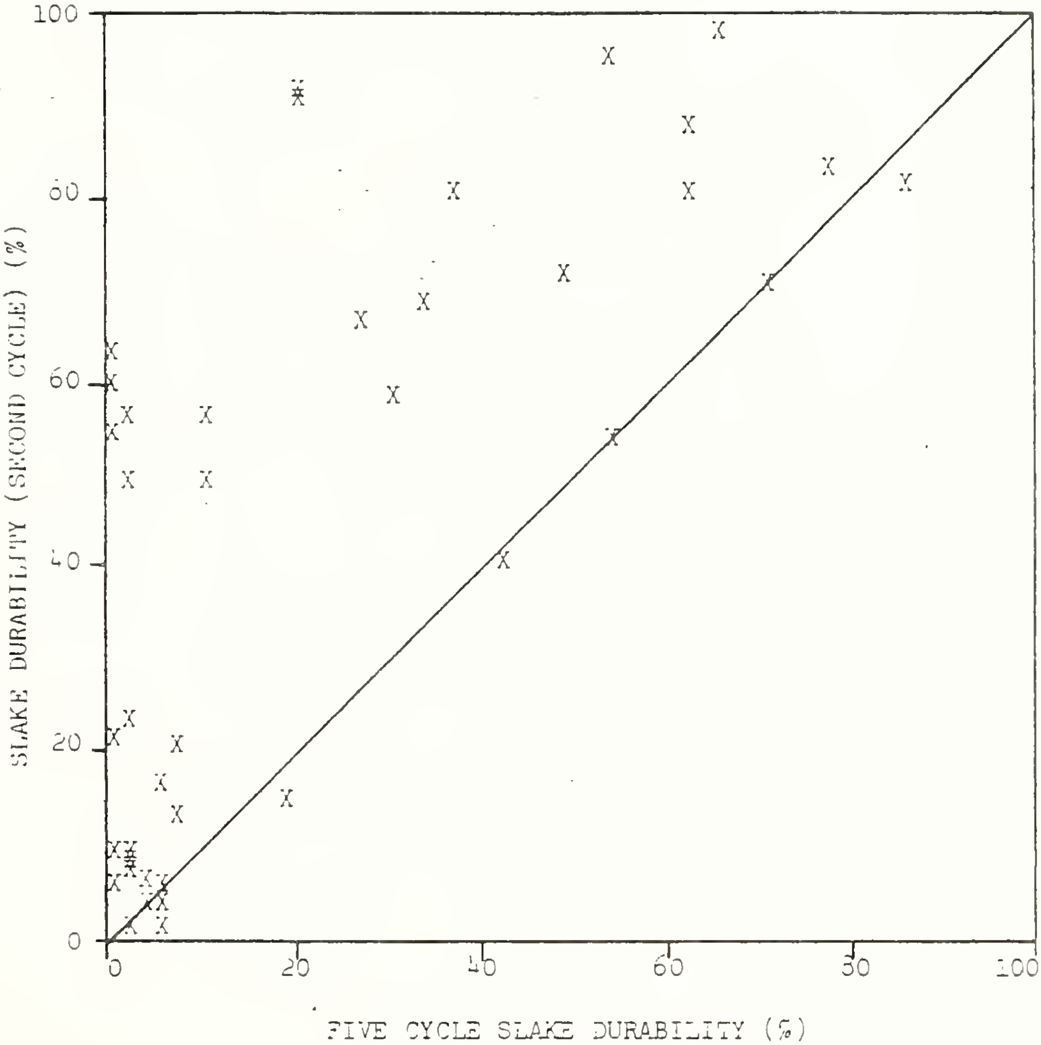


FIGURE 9: Slake Durability (Second Cycle) vs. Five-Cycle Slake Durability

disadvantages are that it requires a special piece of equipment (shown in Figures 10 and 11), and it also does not model embankment confinement.

In this test, ten fragments of shale are tumbled inside a rotating mesh drum which is partially submerged. The drum is made of No. 10 (2.00 mm) wire screen and rotates at 20 revolutions per minute. As with the five-cycle test, the shale charge should be carefully selected. The specific surface of the shale fragments is controlled by using fragments which weigh 40 to 60 grams each, and which are roughly equi-dimensional with no protruding corners. The entire sample shall weigh 450 to 550 grams.

The selection of the number of revolutions used for testing comes from research by Deo (14). He found that the greater the number of revolutions, the greater the contrast among durable and non-durable shales. However, beyond 200 revolutions values were not repeatable. Two hundred revolutions is therefore recommended. The slake durability index is defined as the percentage of material remaining in the drum after being rotated in the slaking fluid. Deo (14) also found that samples allowed to soak in water for six hours before testing degraded more than samples which were tested immediately after being oven dried. In general, the soaked sample testing is too severe to be recommended. However, for the harder shales, specifically those which have an index greater than 85 percent, the soaked test may be used for further durability differentiation (14).

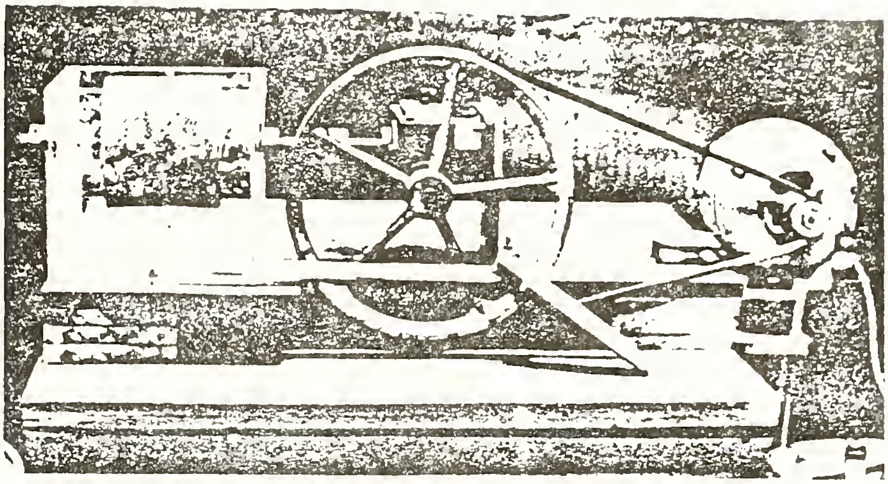


FIGURE 10: Slake Durability Apparatus

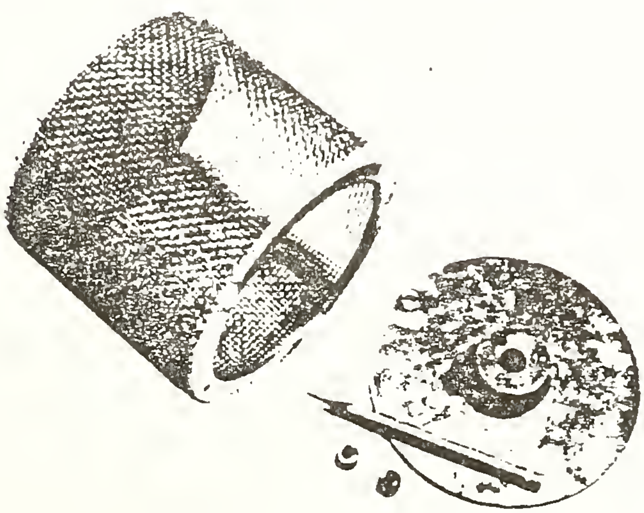


FIGURE 11: Test Drum in Slake Durability Apparatus

Deo (14) and Chandra (11) concluded that the slake durability index determined from the first cycle of the test gave inconsistent and non-representative values. This may be due to loose material initially adhering to the specimen or to easily broken protruding corners. It is therefore recommended that the slake durability index be determined from a second cycle of drying and 200 revolution testing.

Using the second cycle to calculate the slake durability index does not totally eliminate the inconsistencies developed in the first cycle. This is because the calculation is cumulative. Figure 12 shows the correlation between the cumulative amount of slaking and the slaking which occurs in the second cycle only. The relationship is much more simple for the high durability shale than for the others. This may be explained by the presence of durable chunks within predominately soft shales. During the first cycle a large portion of the shale slakes, and the material retained is mostly the harder material. During the second cycle very little additional slaking takes place. Thus, the second cycle alone could not be used, since shales which slaked almost totally on the first cycle would be represented by very high (durable) values.

A recommended procedure for conducting this test and an example set of data are given in the Appendix.

Point Load Strength

The point load strength test normally produces a splitting or tensile failure which can be correlated with the rock hardness and compressive strength. Evaluating the shale chunks in this manner

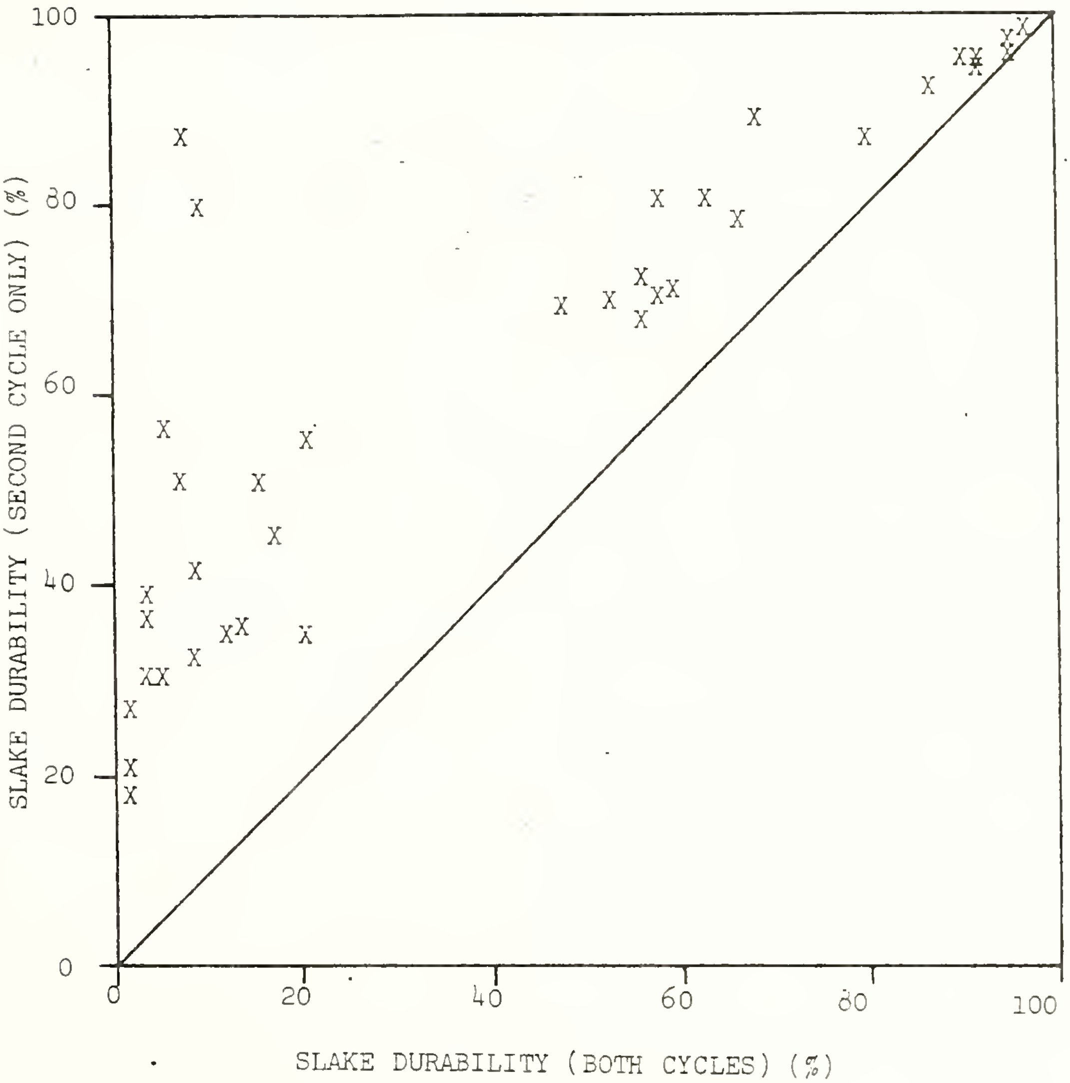


FIGURE 12: Effect of the First Cycle on the Slake Durability Test

is useful, since the failure is like that which may occur under rolling or embankment weight. Again, confinement effects are not simulated in the test.

The point load strength test is primarily advantageous for shales because the preparation of cylindrical samples for uniaxial compression testing is unnecessary. The trimming difficulty is not the only prohibitive factor with shales. The normal rock sampling and preparation techniques of diamond bit coring and grinding requires water as a coolant. This may cause slaking of the shale and may change the natural water content, which is an important factor. A review of point load test history and use is given by Hale (18).

Basically, the procedure is to place a fragment of rock between two axial contact points and load it to failure. The point strength load index, I_p, is defined as the load at failure divided by the square of the initial distance (d) between the platens. The apparatus is shown in Figure 13. The value of I_p has the units of stress and is simply related to the stress on the plane of failure at failure.

The disadvantages of this test are: there is considerable scatter in the results; the index varies with the size of the specimen and a correction factor must be used; and the index varies with the shape of the specimen (18). The advantages include test speed, allowing the difficulty of variation in the results to be overcome by running many tests. Also, the test device is portable and can be conveniently employed in the field. A third advantage accrues from the first two, viz., the test can be conducted with very little change in the natural moisture content.

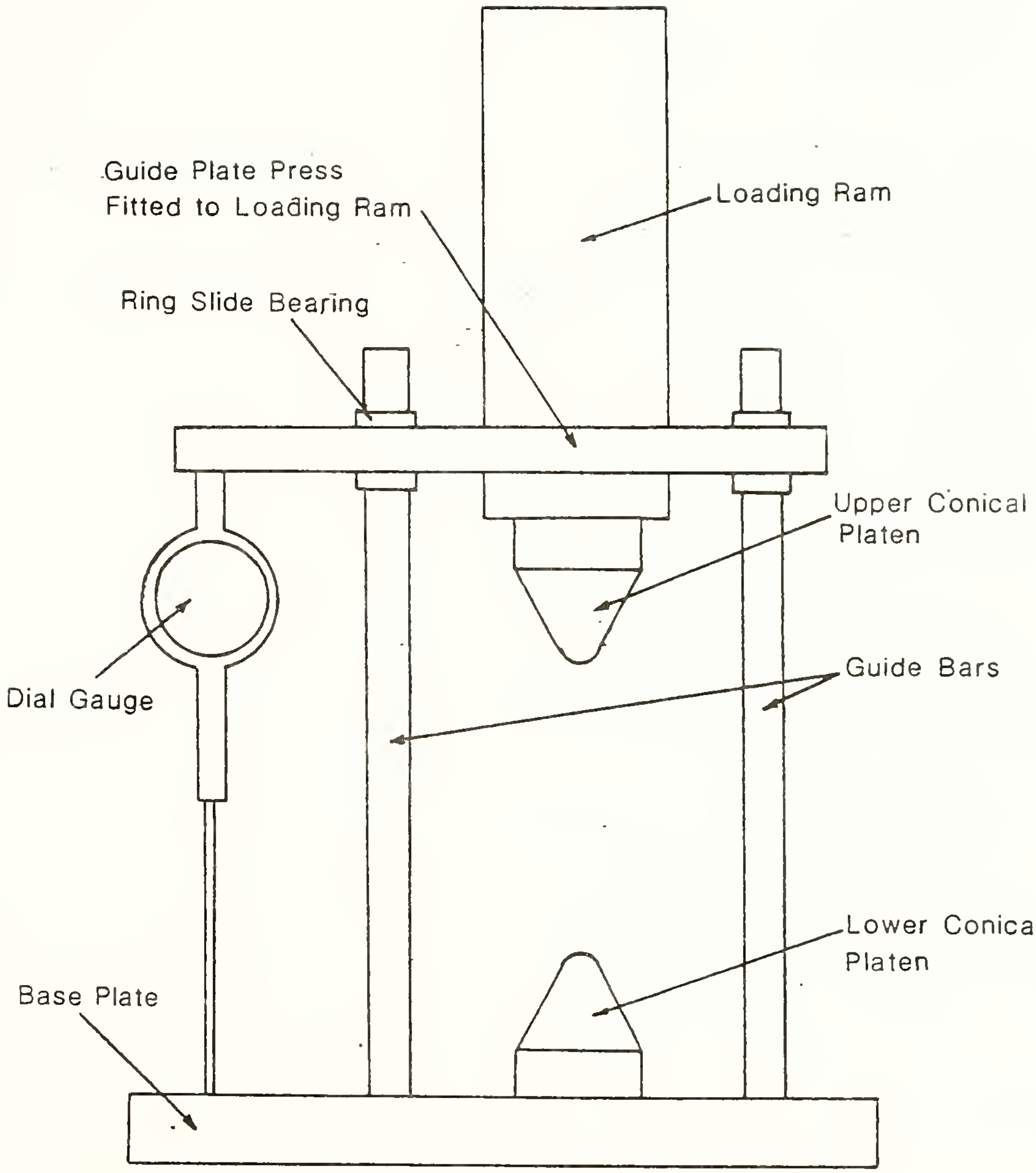


FIGURE 13: Point Load Strength Test Apparatus
After Hale (18)

To account for the dependency of the point load index on the size of the specimen, a correction chart has been developed to adjust the index to that of a standard size specimen. Usually, the standard diameter is 50 mm; however, Hardy (19) suggests that it be 54 mm, which is the diameter of an NX core. The author favors the 50 mm standard size because of its common acceptance, viz., Franklin (16) and the International Society of Rock Mechanics (24). Correction charts have been developed primarily from tests on sandstone, quartzite, and limestone (19). Abeyesekera and Lovell (2) find that these charts are not generally applicable to Indiana shales. The correction factors are dependent not only on the shale type, but also on the sample shape and the orientation of the bedding planes of the specimen.

With sufficient testing, a unique size correction factor can be derived for each major shale member. Test samples should be bulky in shape and larger than a minimum dimension. Hardy (19) recommends that the minimum dimension be 30 mm. Hale (see Figure 14) shows a disproportionately large increase in the point load index as the diameter goes below about 22 mm. This inconsistency may be explained by a certain amount of crushing at the platen contacts, which spreads the load over a finite area. It is believed that the error is a function of the ratio of the finite area of contact and the volume of the specimen. This ratio would be small for larger samples, but increases rapidly in smaller samples. It is therefore recommended that the samples tested be as large as possible, with a minimum least dimension of 25 mm.

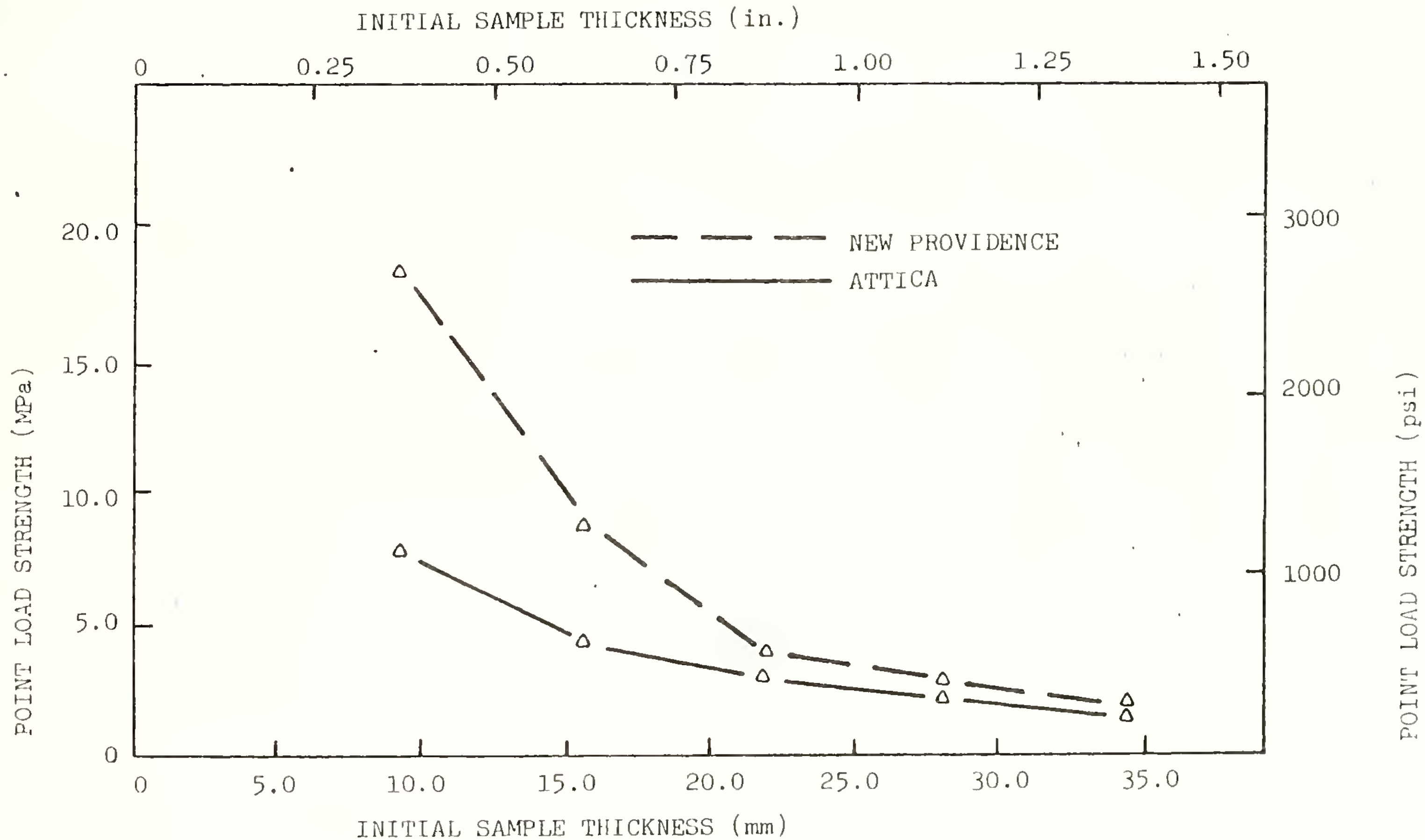


FIGURE 14: Size Effect on Point Load Strength Values From Hale (18)

Investigations of sample shape [Broch and Franklin (10)] led them to recommend that the irregular lump have a diameter (dimension in the axial direction between platens) of 1.0 to 1.4 times the average width. This is an extremely difficult criterion due to the fissility of shale, and especially because the shale is usually loaded perpendicular to the bedding planes. That is, in most cases, the smallest dimension for a shale fragment is perpendicular to the bedding planes, yet the recommendation calls for it to be the largest. All shale fragments should be as close to equi-dimensional as possible.

Since point load strength is dependent on the direction of loading with respect to the bedding planes (2), more accurate and consistent results can be achieved if the samples are always loaded perpendicular to the bedding planes, rather than parallel to them. Usually the bedding planes are a zone of weakness to shear or tension.

The water content of the shale at which the point load test is conducted critically influences the index. Bailey (5) considered the effect of three reference water contents on the point load strengths of Indiana shales: 100 percent saturation, 0 percent saturation, and the natural water content. He found it impractical to use 100 percent saturation, since wetting the shales caused excessive slaking. At zero percent water content there was considerable data scatter caused by micro cracking. On the other hand, Bailey was able to produce reasonable and reliable values at the natural water contents. Accordingly, it is recommended that the point load strength test be conducted at the natural water content of the shale.

A suggested procedure for conducting this test and an example set of data are shown in the Appendix.

COMPACTION AND DEGRADATION TESTS

Since embankments of non-durable shales must be thoroughly degraded and tightly compacted in thin lifts, standard tests which rate the degradability and define the compaction relations are required.

Compaction-Degradation Test

Degradation of the shale will occur in all handling processes from excavation through final compaction. Correlations among rock classifications and methods of excavation are available (43). In most cases these involve blasting factors, or width and depth of ripper passes so that the material can be handled by the available loading and hauling equipment. Because it is important to achieve a particular level of compaction, predictions of degradability under field rolling are needed as well. The laboratory compaction-degradation test is a first step in meeting this need.

The test is basically one in which the change of gradation produced by a standard compaction process is measured. Bailey (5) summarizes the various ways that gradation and change in gradation may be represented. These include gradation coefficients and gradation indices; the change is usually represented as a percentage value. In an extension of Bailey's research, Hale (18) selected the index of crushing as the standard measure of degradation. It was chosen for its simplicity and ability to assign a unique value to each different gradation. Gradation measures based on simple ratios of particular grain sizes can yield the same value for very different gradations. An example of this is shown in Figure 15 where two distinctly different curves give the same coefficient of uniformity (C_u).

The index of crushing is the percentage change in mean aggregate size due to compaction. It is usually approximated by reducing the gradation to a discrete function by a sieve analysis, and summing the product of the mean size and percentage of material retained on each sieve before and after compaction. This percentage change in the mean size is used as a relative measure of the degradation which can be expected in the field. Correlations with field measurements of degradations are highly desirable.

The compaction-degradation test is commonly needed only for the harder shales. Soft shales degrade easily under normal compaction, and are not considered to be a problem if compacted in thin lifts. Indeed, it may be difficult to run the test for soft shales which tend to be cohesive and to have the fragments bonded together when compacted. The task of separating the fragments to determine the final gradation can be quite arbitrary and lead to inconsistent and erroneous results (5).

The initial gradation, that is the gradation before compaction, is one of the variables of the test. Ideally, the gradation should parallel that of the field gradation before compaction, but this is

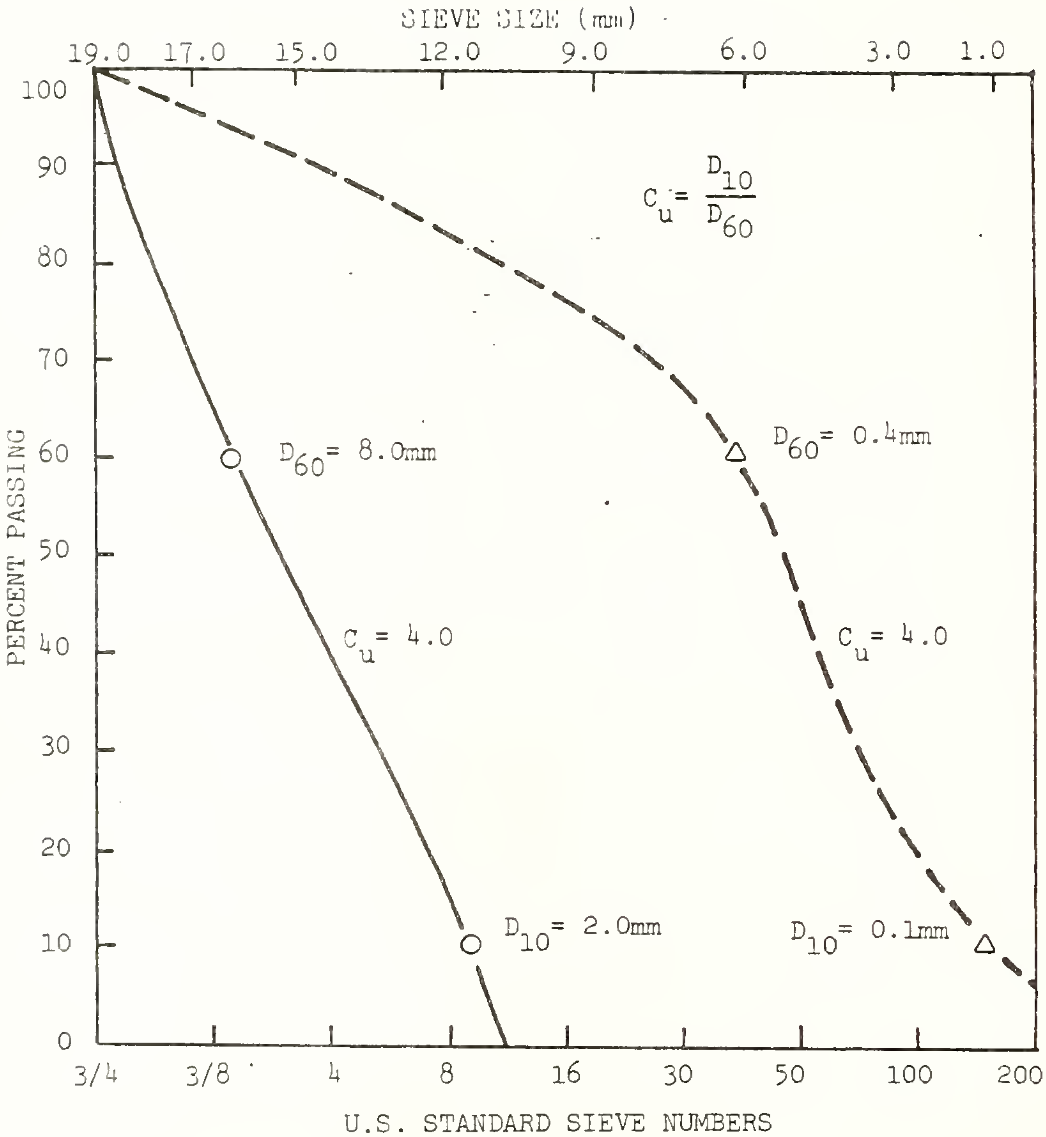


FIGURE 15: Gradation Curves for Two Samples with the Same Coefficient of Uniformity From Bailey (5)

an unknown and varies widely with the project. A convenient gradation afforded by the expression:

$$P = 100(d/D)^n$$

where

P = percentage, by weight, finer than size d,

d = sieve size

D = maximum aggregate diameter, and

n = 1

This is a good approximation of the gradation of the material after crushing in a reciprocating jaw crusher, and wasted material is minimized. In cases where the field gradation is known to be significantly different from this relationship, the gradation for the laboratory test can be modified accordingly.

To produce the desired gradation, the crushed material is separated in a nest of sieves and recombined by size fractions. The sieved product exists in a discrete function and will imperfectly fit the continuous function of the gradation equation. The error is reduced by using more sieves, but this requires additional time to prepare the gradation. Since the test should be conducted at the natural moisture content, significant drying of the material may occur if the sieving process is too long.

In earlier work by Aughenbaugh et al. (4) and Bailey (5), ten sieve groups were used. These ranged from 3/4 inch through the No. 200 sieve sizes. Hale (18) enlarged the average range of each sieve group by increasing the maximum size aggregate, but reducing the number of groups to nine. Hale's sieves ranged from the 1 1/2 inch size through the No. 100. Because of the small percentage of fines in the sample and their even smaller (almost negligible) contribution to the index of crushing, the sieving process can be simplified by eliminating or combining the finer sieve groups. This would greatly reduce the time involved in sieving, and eliminate some error which is due to the drying of the shale. Even though the fines probably contribute very little to the mechanism of degradation (18), their complete elimination is not recommended. The calculations of the initial gradation are more complicated if they are removed, and also field conditions are more closely duplicated if they are retained. However, combining all of the sieve groups smaller than the No. 4 produces and error in the average size of less than 1.0 percent. An error of this magnitude would be caused by all the fines being as coarse as possible, i.e., just passing the No. 4 sieve. This is unlikely, and Figure 16 shows that the fines which are a product of the jaw crusher closely approximate the desired gradation. For both of the shales shown in this figure, the error in mean size is negligible.

The maximum aggregate size also has a major effect on the degradation. Hale (18) found that the larger the maximum size, the larger is the index of crushing. This is logical since the coarser gradation should have fewer aggregate contact points and hence higher contact stresses. The higher values of the index tend to made it easier to distinguish among different shales. It is therefore recommended that the maximum aggregate size be as large as

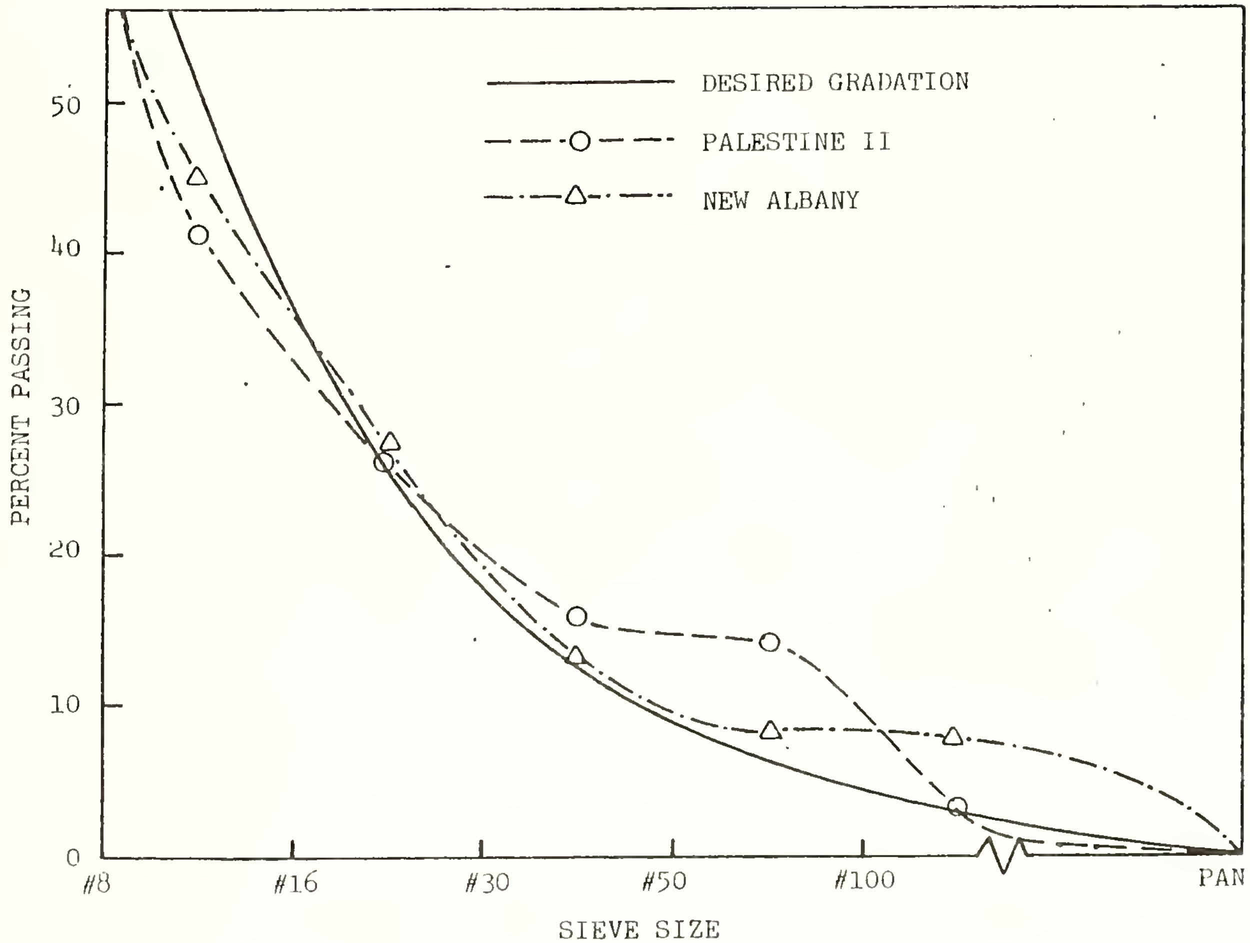


FIGURE 16: Gradation of Material Less Than #8 sieve as Given By Jaw Crusher

is practical. Since a 15.2 cm (6.0 in.) mold will be used, the maximum size which works well is 3.8 cm (1.5 in.).

The type and effort of compaction are other important factors. Bailey (5) studied four types of compaction: kneading, gyratory, static, and impact. He favored the static and impact compaction methods. The static method has the advantages of being consistent, simple, and uses a known compactive force. The impact method displayed almost as much consistency, and had the advantage of being a widely accepted compaction mode. Hale (18) made a further study of the two methods and recommended the impact method for the reason of common acceptance.

Hale (18) also investigated various nominal compaction effort levels. Based on many trials, a nominal effort of 861 $\frac{kN - m}{m^3}$ (18,000 $\frac{ft - lb}{ft^3}$) was selected. This is obtained by compacting 1/13.33 ft^3 in three layers with 25 blows per layer using a 4.54 kg (10.0 lb) modified Proctor hammer having 0.457 m (18.0 in.) of free fall.

Great care must be taken to ensure that the gradation of each layer is representative of the total sample. Without such care, a given layer may have a disproportionately large percent of fines or coarse material. This could cause a great error in the determination of the index of crushing. A sieve analysis on the uncompacted portion of the sample will disclose if a bias exists in the placement technique. If consistent results cannot be obtained, it may be advisable to incorporate specified placement procedures in the standards for the test.

During compaction, fragments tend to fly out of the mold when large pieces are broken. This problem becomes more pronounced as the layers approach the level of the collar. If the fragments are not immediately replaced, both the gradation and the opportunity for such fragments to be further degraded are disrupted. If the problem is excessive, a higher collar or a towel wrapped around the collar may be used.

A third common source of error is degradation which may occur during the final sieving. The error is reduced with reduced sieving time. The amount of time necessary for sieving can be shortened by sieving only small charges of material, and by gently shaking the sieves by hand in a horizontal circular motion before placing them in the mechanical shaker. If the problem is felt to be excessive, a correction calibration (relating additional degradation with sieving time) can be developed for the shale.

A recommended procedure for conducting this test and a numerical example are given in the Appendix.

Compaction Control Test

Whenever possible, the compaction control should be generated in a test pad. It may be stated in terms of an end result, procedure, or combination thereof. The techniques which follow apply to the definition of a laboratory moisture-density curve which can be used in an end result specification.

If the shale is soft and has good absorption characteristics, water content may be used as a variable in the usual way. Raising the water content weakens the shale and increases its degradability (18). Just as with soils, however, there is a point where additional water hampers the compaction process and lowers the compacted dry density. Thus, an optimum water content and maximum density can be defined for a given shale and compactive effort.

The test does not generally apply to hard shales because it is often impractical to change their water contents in the field. Morgenstern and Eigenbrod (33) show that the time for complete softening of shale increases with increasing hardness. This indicates that the absorption of water in harder shales is difficult. Water added to these shales in the field would probably evaporate or run off before it could be absorbed. Therefore, such materials must ordinarily be compacted at or near their natural water contents. Density values for specification are defined by varying the compactive effort at a constant water content either in the laboratory or field.

To achieve consistent and reproducible results, the moisture content throughout the laboratory sample should be as uniform as possible. Two steps are recommended: (1) water is added in a spray and thoroughly mixed; (2) the material is allowed to cure for two days. Bailey (5) found little or no benefit in curing beyond two days.

Because the same basic procedures are used for defining a single point in compaction control and in running the compaction-degradation test, the latter test may be used to produce a single point for the moisture-density curve. (This assumes that the compactive energy is the same for both tests.)

A recommended procedure for conducting this test and an example set of data are given in the Appendix.

COMPRESSIBILITY AND SETTLEMENT TEST

The ordinary one-dimensional consolidation test apparatus may be used to assess the compressibility of compacted shale and to estimate settlements of shale embankments. Example measurements by Witsman (44) are shown in Figure 17. As-compacted compressibility is shown by the initial curve. This curve will change in shape at the compaction prestress. After the shale has compressed under a pressure that approximates an embankment confinement, the sample is saturated and either settles or heaves. The saturation simulates the effect of the environment on the embankment in service, and is represented by the vertical portion of the curve in Figure 17. Further loading would produce settlement according to the compressibility of the shale in a saturated condition.

Such laboratory data may be scaled upward from laboratory model to embankment prototype and used to predict the embankment settlement. Other data by Witsman (44) show that the as-compacted compression (under self weight) will occur as rapidly as the embankment is constructed.

To make this test as consistent as possible with the other shale tests, the sample preparation is much the same as for the moisture-density test. It is recommended that the same gradation function be used for this test as in the compaction-degradation test. The mold

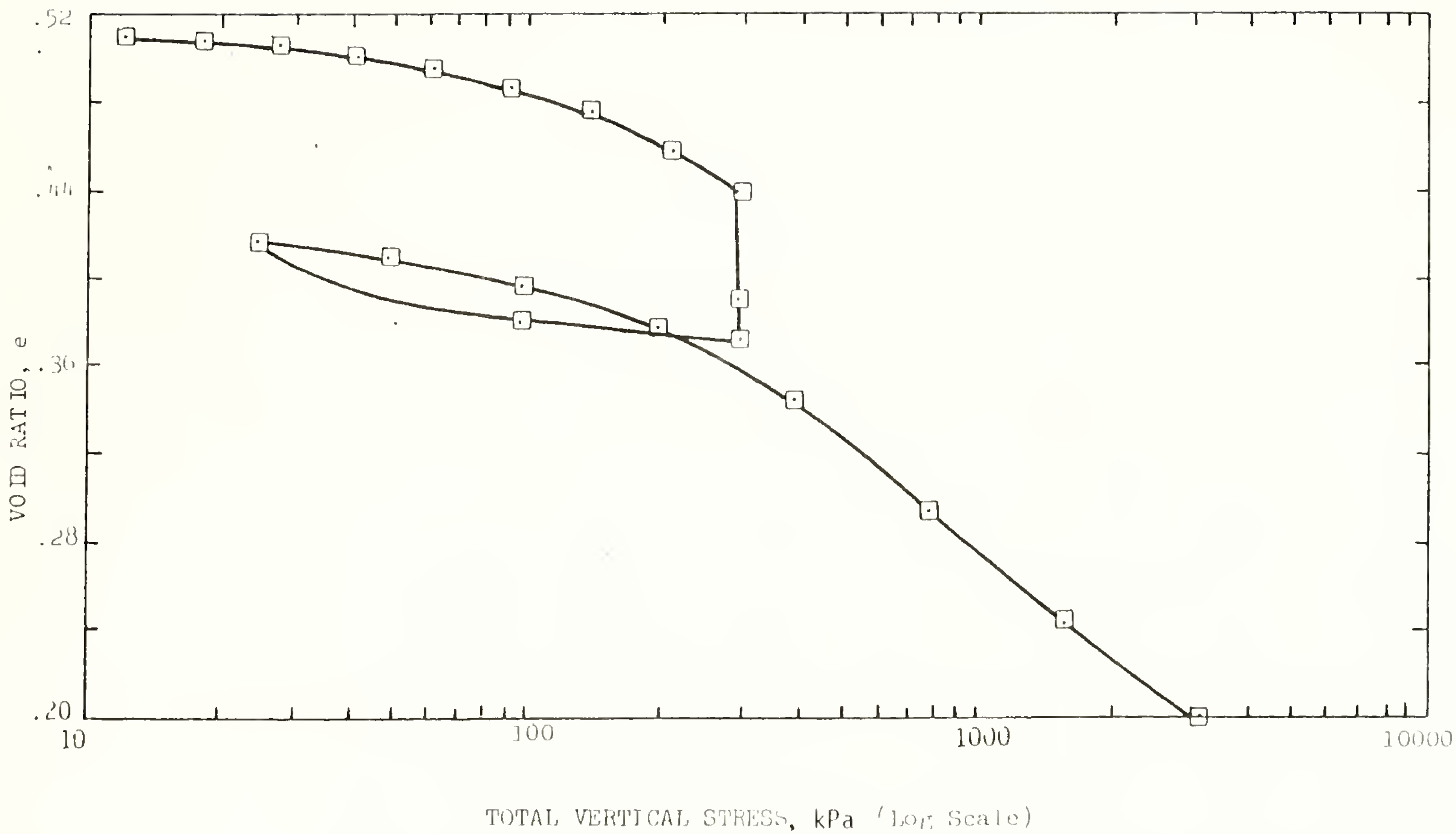


FIGURE 17: One-Dimensional Compression Test on New Providence Shale, After Witsman (44)

for this test is only 10.2 cm (4.0 in.) in diameter and therefore the maximum particle size is reduced to 1.9 cm (0.75 in.).

The water content selected for the test should be that predicted to obtain for the field application. This might be the natural water content or the optimum water content defined in a laboratory moisture-density test. Again, to achieve the most consistent results, the water content must be uniform throughout the sample. It is therefore recommended that the water be added as a spray and the sample be allowed to cure for at least one day. The curing time for these samples is less than for the compaction samples because both the maximum aggregate size and the sample size are smaller.

To approximate the field compaction mode, laboratory kneading compaction is recommended. In this technique, the kneading foot pressures are adjusted to match the density of the laboratory control curve at the desired water content. It is further recommended that these samples be compacted in five equal layers to achieve better sample homogeneity.

The test procedure will start by loading the partially saturated compacted sample in small load-increment ratios (0.5) until the prestress point is defined. The second stage is to saturate the shale while monitoring the volume change. The confining pressure should correspond to a depth in the embankment prototype. It is necessary to conduct several tests to establish the settlement for the entire vertical profile of the embankment. Finally, the sample should be unloaded and reloaded to establish the rebound and loading relations of the saturated shale.

Abeyesekera's collapse test (1) gives an indication of what volume changes to expect if the shale is not well degraded and thoroughly compacted.

A recommended procedure and an example set of data for this test are given in the Appendix.

SHEAR STRENGTH AND SLOPE STABILITY

Triaxial testing is the basis for determining the shear strength of compacted shales. The preferred procedure is to extract intact cylindrical samples from a compacted test pad. Such samples can be confined to approximate various embankment positions. If these are sheared undrained, the as-compacted strength is defined and slope stability analysis can be undertaken by a suitable computer program such as STABL2 (9).

Since test pad construction is expensive, the samples often must be compacted in the laboratory. To be consistent, the sample gradation and preparation are the same as for the compressibility tests. A nominal sample height of 21.6 cm (8.5 in.) with a diameter of 10.2 cm (4.0 in.) are the largest practicable sample dimensions. This nominal height is used so that nine layers (each equal to the layers used during the compaction of the compressibility sample) could be used while still maintaining an approximate 2:1 height to diameter ratio. This ratio is recommended (15) so the effects of the stress concentration at the ends can be neglected. Layer thicknesses are the same as a matter of uniformity among the tests, and to allow the same calibration, relating kneading foot pressure to density at a given water content, to be used.

The situation which is to be modeled is an embankment of partially saturated shale placed as a soil fill. The embankment consolidates under its own weight during or shortly after construction, and then is subject to wetting in service. It is desirable to take all practical steps to drain the embankment to aid in controlling slaking and to improve slope stability. However, due to precipitation, subsurface water movements, and imperfect operation of the drainage system, the material may become saturated in service. To simulate such circumstances, compacted samples should be saturated and sheared undrained. This situation is believed to be the most critical for an embankment (20).

As previously stated, the specimen preparation parallels the consolidation sample preparation as closely as possible. The basic procedure as developed by Abeyesekera (1) is used with the exception that the shale should be allowed to cure for 24 hours after adding water and before compacting. The shale should be compacted at the expected field water content to the dry density predicted by the compaction control test. Kneading compaction is recommended in an attempt to simulate a sheepsfoot roller which is commonly used in the field (1). This procedure was successfully used by Okagbue (35).

Because partially saturated soils compress relatively quickly, and because measures are usually taken which retard early saturation, compression under self weight should be modeled in the partially saturated state. Compression is usually achieved isotropically by increasing the cell pressure in small increments, allowing sufficient time between increments for the pressure within the sample to equalize. Volume changes should be monitored during this phase of testing, as well as all others, for estimating embankment settlements and to define the average sample diameter at failure. The volume change can be measured by monitoring the volume of water in the cell, making corrections for the volume displaced by the piston as it moves. Samples are compressed to a range of values approximating the range of embankment heights.

Back pressuring is necessary to achieve saturation in fine grained materials (25). Back pressuring is a process in which the cell pressure and pore pressure inside the sample are raised by equal amounts, thus keeping the effective stress in the material constant. The volume of air is reduced in three ways: by flow out of the sample, by direct compression according to Boyle's law, and by dissolving into the pore water according to Henry's law of solubility (28).

The saturation process begins by percolating water through the porous stones and sample. It may be necessary to create a pressure gradient by applying pressure at one end and possibly a vacuum at the other. The higher the degree of saturation achieved in this phase, the smaller the pressure required for back pressure saturation (6). An estimate of the pressure needed to achieve saturation (based on the initial degree of saturation) is shown in Figure 18. Because it is necessary to allow the pressure to equalize throughout the sample to avoid overconsolidation (25), and because the back pressure effect is time dependent (1), the back pressure should be increased slowly. A rate of no more than 50 kPa (7.0 psi) per hour should be used. Back

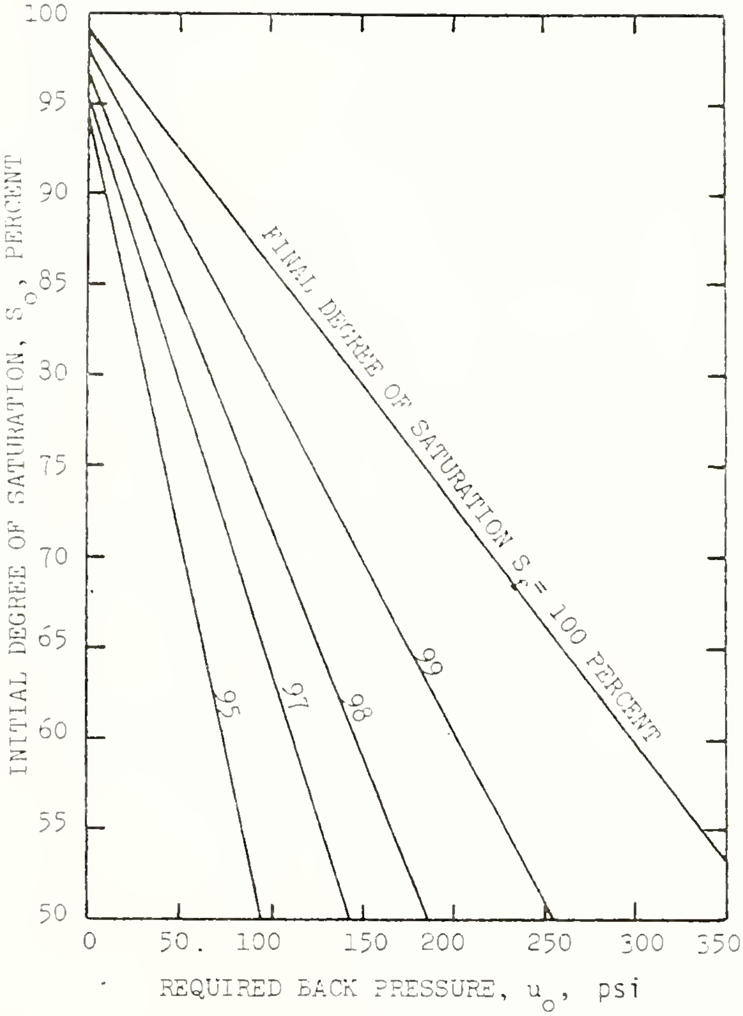


FIGURE 18: Back Pressure Required to Attain Various Degrees of Saturation After Reference (15)

pressuring should continue until saturation is achieved or to the limit of the pressurizing equipment.

There are two useful methods of estimating the degree of saturation during the back pressuring procedure. The first is by measuring Skempton's "B" parameter (1). The parameter is defined as:

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

where

n = porosity

C_s = compressibility of soil skeleton, and

C_w = compressibility of water

For most compacted shales this parameter is very close to unity when fully saturated. It is calculated as the ratio of the change in pore pressure created in an undrained condition (due to an increase in cell pressure) to the change in cell pressure. A second check is afforded by measuring the intake of water for each increase in back pressure. Saturation is reached when no water flows into the specimen as a result of increases in back pressure. Johnson (25) recommends that the first method be used as a quantitative measure, while the second can be used as a qualitative check.

Again, the volume change on saturation should be measured so that the settling or swelling characteristics of the compacted shale in the service environment can be predicted. If the volume change cannot be measured by monitoring the volume of the cell water, it can be estimated by measuring the inflow and outflow of water during saturation, and applying the following equation:

$$\Delta V = V_{WI} - V_{WO} - V_v$$

where,

ΔV = change in sample volume,

V_{WI} = volume of water in,

V_{WO} = volume of water out, and

V_v = volume of voids.

The samples are sheared undrained, preferably with pore pressure measurements. Shearing must be slow enough so that the pore pressures have time to equalize within the sample. Blight (7) recommends that the rate of strain be controlled by:

$$t_f = TH^2/c_v$$

where

t_f = time to failure,

T = time factor,

H = (0.5)(specimen height) for a 2:1 aspect ratio,

c_v = coefficient of consolidation.

After failure or a strain of 20 percent the cell should quickly be dismantled and a final water content sample taken.

It is often advantageous to use filter strips along the sides of the specimen to accelerate the equilization of pore pressures within the sample and allow a higher rate of strain.

Typical stress-strain, pore pressure-strain, A-factor strain, and stress-path relationships for the New Providence shale tested by Abeyesekera (1) are shown in Figure 19. These relationships depend greatly on the confining pressure and its ratio with the compaction prestress. Strength is interpreted in terms of either total or effective normal stress (Mohr-Coulomb); and the long-term undrained stability is assessed.

Values of the effective stress intercept (c') for all reasonable conditions of compaction are expected to be small, while the effective stress strength angle (ϕ') is insensitive to compaction variables. Abeyesekera (1) found a ϕ' for Indiana New Providence shale of 28° to 30°. In contrast, if the same shale were loosely placed, ϕ' decreased to 25°. Excess pore pressures at failure did vary considerably with the details of compaction and the confinement. Accordingly, the factor of safety against an undrained failure could vary significantly with the above compaction factors.

A suggested procedure and example for this test are given in the Appendix.

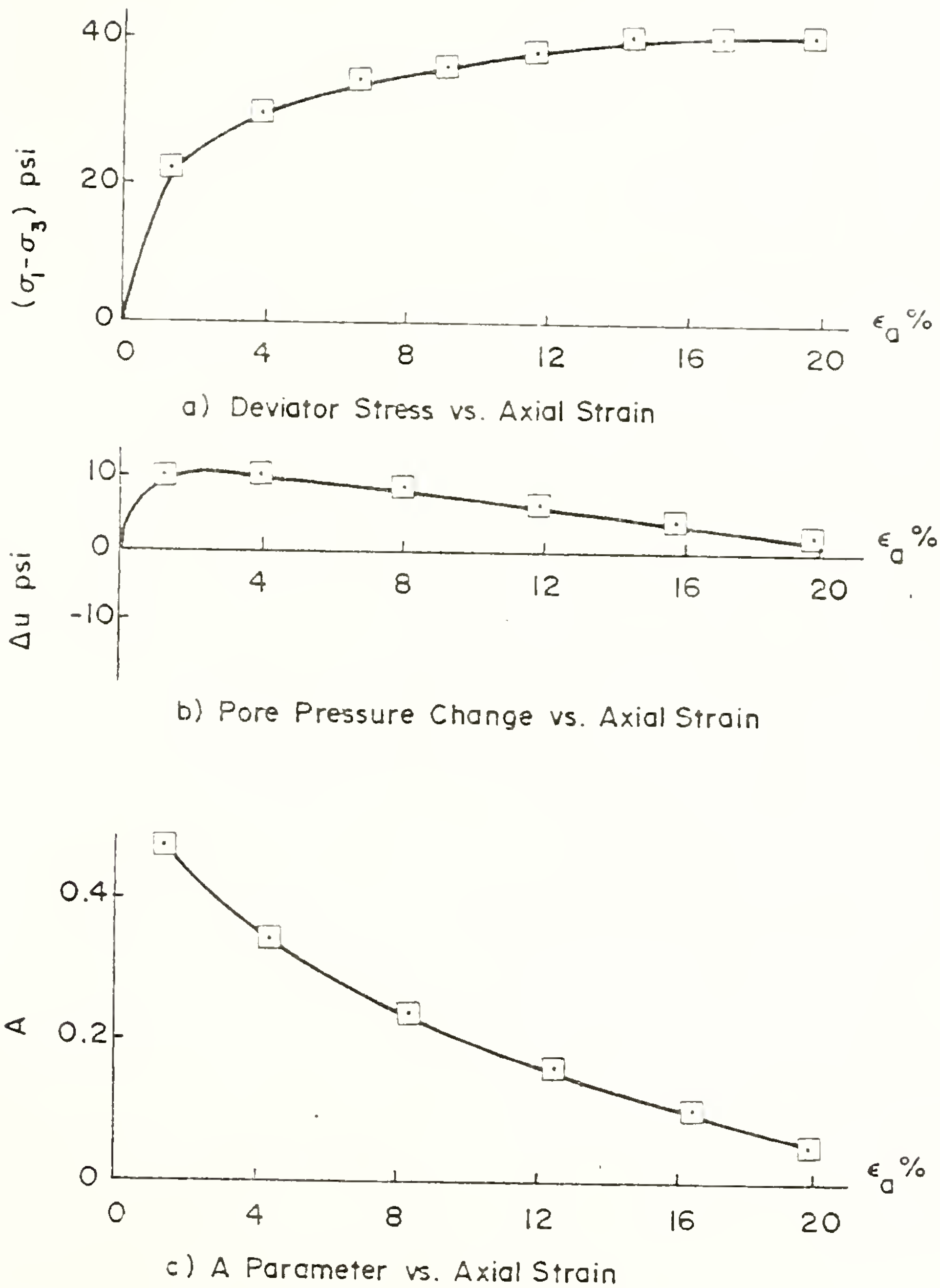
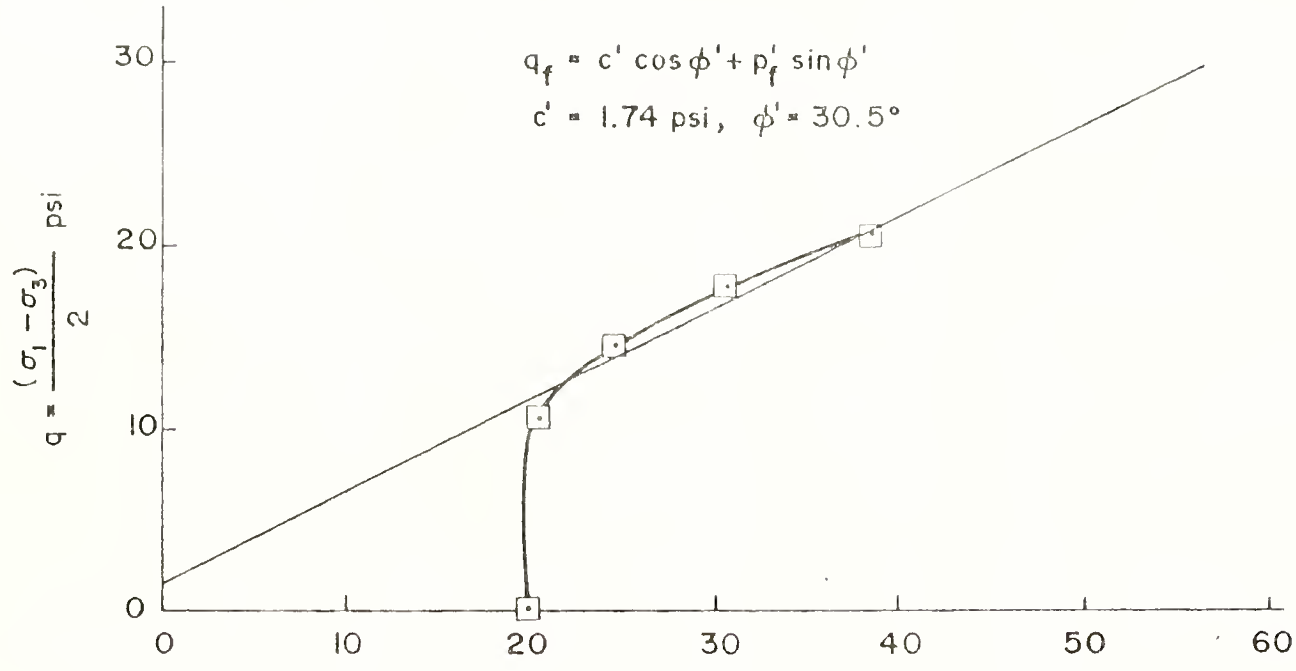


Figure 19 Isotropically Consolidated Undrained Triaxial Shear Test on New Providence Shale (After Reference 1)



(d) Effective Stress Paths

Figure 19, cont'd (After Reference 1)

DESIGN CONSIDERATIONS FOR SHALE PLACED AS ROCK FILL

In cases where the shale fill material is strong enough and durable enough, it is far more economical to place the material as a rock fill than as a soil fill. The savings come about because: far less degradation is necessary, the material can be placed in thick lifts without the use of special spreading equipment, and little compaction is needed. As discussed previously, the Franklin rating system can be used to determine the adequacy of the shale for rock fill. The testing, however, does not truly model long term saturation of the shale. There are special cases of shales which rate very highly according to Franklin's criteria and yet perform poorly in the field.

In one case (34), a very tough, dark gray shale was used to build a 100-foot high rock fill on I-64 near Clifton Forge, Virginia. The shale had a slake durability index of 99 percent, and the need for blasting to excavate it attests to its hardness. This would yield a very high rating. Three years after the end of construction the embankment began to settle. Auger holes through the embankment revealed that most of the shale had become soil like. A study showed that one of the major elements in the shale was a sedimentary chlorite. Also present was pyrite, an iron sulfide. The cause of the deterioration was the dissolving of the chlorite by sulfuric acid which was formed by allowing the embankment to become saturated.

There are four techniques which may be used to prevent or slow this type of reaction to a tolerable level. First, the shale can be degraded and placed as a soil fill. This process and the testing methods associated with it have been discussed earlier. Second, drainage can be used to prevent the shale from becoming fully saturated. Thirdly, an encasing material of low permeability can be used on the outer portions of the embankment. Finally, chemical additives can be used to reduce the potential for slaking.

Good drainage will minimize slaking, and add to the stability of the embankment. Vertical and horizontal drains have been used very effectively in Kansas (13), mostly as a remedial measure. The use of a free drainage rock pad under the embankment has also been found to prolong the life of shale embankments (32). The purpose of the rock pad is twofold. First, it allows the water which has percolated vertically through the embankment to drain away laterally, and second, it prevents ground water from entering the fill through the cut-fill interface. The planting of trees and shrubs on an embankment has been used to keep it drier (13). All of these methods are aided by keeping the shale itself as free draining as possible. The shale should therefore be placed with as few fines as practicable.

Some surface protection is afforded by a pavement, although joints, cracks and edges allow water infiltration into the shale. Other materials which can be used to deter infiltration are encasing layers of clay and vegetative cover. Contouring the surface of the embankment and surrounding areas to channel water away from the fill is also a great help. Burrowing animals make it difficult to maintain a zone of very low permeability around the shale (13).

The use of additives to control slaking was studied by Surendra (40). Surendra's work involved compacted shales in soil lifts; however, it may be possible to extrapolate the results to shale rock fills. The additives which retard slaking vary with the type of shale (40).

Cases where long-term degradation of shale rock fills cannot be predicted by normal testing methods are uncommon (13). Since it is difficult to determine just how a shale will react to long-term saturation, the design and construction techniques described previously are recommended. In general, these methods will not add unduly to the cost, will improve the stability of the embankment, and are a safety measure against serious long-term problems.

SUMMARY

1) Exploration must yield not only the proper amount of material used for testing, but also a minimum aggregate size.

2) Testing should be conducted in two parts:

   a) Classification - These tests are used to determine the hardness and durability of shales. Tests found to be useful for classification are the Atterberg limits, five-cycle slaking, slake durability, and point load strength.

   b) Design Parameters - Parameters which describe compaction, compressibility and shear strength behavior of compacted shales can be evaluated by four tests. Compaction properties are determined by the compaction-degradation test and the moisture-density relations test. Compressibility tests can be used to evaluate settlements in the as-compacted condition, during saturation, and in a saturated condition. Isotropically consolidated, undrained triaxial tests are used to determine the shear strengths for the long term saturated slope stability analyses.

A flow diagram of the recommended testing procedure is shown in Figure 20.

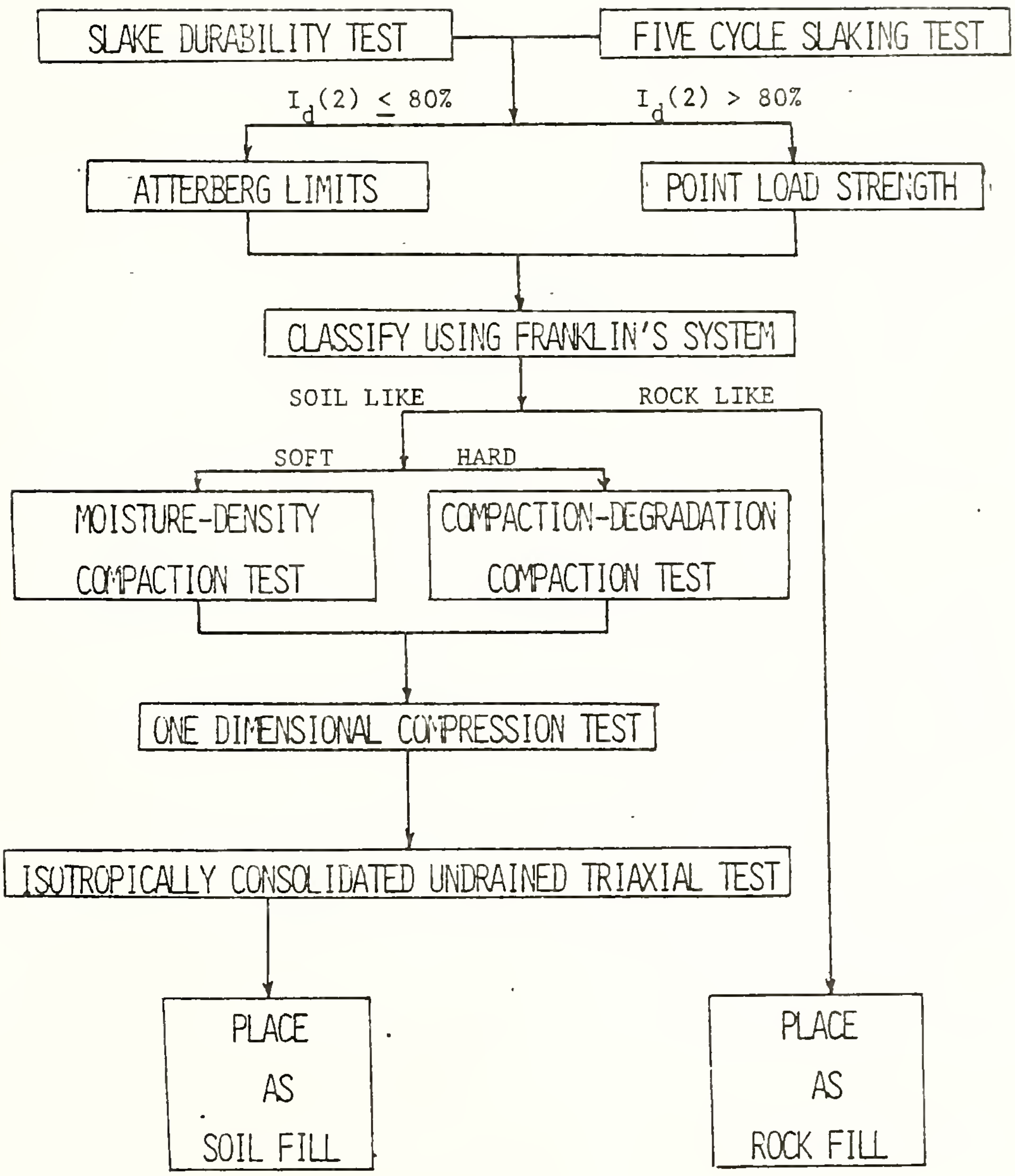


FIGURE 20: Suggested Testing Procedures for Shale to be Used in Embankments.

3) The Franklin rating system, consisting of the slake durability test and either the plasticity index for less durable shales or the point load strength for the more durable ones, is recommended for classification. Current state-of-the-art tends to merely group shales as rock like or soil like. However, the Franklin system contains a continuous scale and can lead to more detailed practical classification with increased correlation with field performance.

4) Shales classified as soil like must be thoroughly degraded and placed in thin lifts with proper compaction.

5) It is important to keep shale embankments properly placed as rock fills from becoming saturated. This will retard the long term degradation which might otherwise occur.

RECOMMENDATIONS FOR FURTHER WORK

1) The development of correlations between Franklin's rating system and the performance of Indiana shales is recommended. The correlations shown earlier in this report are based on Ontario shales, which are believed to be somewhat harder and more durable than Indiana shales. The first step in this process is to review existing embankments. This would involve a study of construction and maintenance records, perhaps a site investigation, and correlation of performance with the rating value.
2) A study of the as-compacted shear strengths should be made to supplement Abeyesekera's work with laboratory compacted saturated samples. It is recommended that testing be conducted with the measurement of both pore air and pore water pressures as described by Blight (8).
3) Correlations between the laboratory compaction-degradation test and field compaction-degradation values are needed in order to properly incorporate this test in the embankment design process.
4) In spite of the techniques employed to prevent them, some cases of excessive shale embankment movement will occur. In such cases, an estimate of the rate of development of distress is needed. Such an estimate can be afforded by laboratory tests which simulate alternate wettings under load with time. These tests should be developed.

5) The results of the decade of research on compacted shales at Purdue University should be the subject of implementation programs at the state and regional levels.

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APPENDIX

Five-Cycle Slake Resistance Test

1. Scope

1.1 This method covers the determination of the slaking resistance index of a shale, resulting from multiple cycles of drying and wetting.

2. Definition

2.1 Slaking resistance index, the percentage, by weight, retained on a No. 10 (2.00 mm) sieve of an intact, dried specimen of shale when subjected to five cycles of drying and soaking for 24 hours in a slaking fluid, usually distilled water.

3. Apparatus

3.1 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 $\pm$ 9 F (110 $\pm$ 5 C).

3.2 Balance, sensitive to 0.1 g and having the capacity to weigh 500 g.

3.3 Miscellaneous apparatus, 600 ml pyrex beakers; distilled water; brush.

4. Test Samples

4.1 Each test sample shall be a representative, intact, roughly equidimensional shale fragment weighing 100 g to 150 g. Any sharp corners shall be broken off and all dust shall be removed by brushing just prior to weighing.

4.2 The samples shall be transported and stored in a manner to retain the natural water content.

5. Procedure

5.1 Each sample shall be placed in an individual beaker, weighed, then dried in the oven for 8 hours or to constant weight. The samples shall be allowed to cool at room temperature for 20 minutes then weighed again. Calculate the natural water content as follows:

$$w = \frac{A - B}{B - C} \times 100$$

where

w = percentage water content,

A = weight of beaker and sample at natural moisture content,

B = weight of beaker and oven-dried sample, and

C = weight of beaker

5.2 Distilled water shall be used to fill each beaker to a height of at least 0.5 in. (10 mm) above the top of the shale. Observations shall be made periodically. If desired, the mechanism of slaking and any variations in slaking rates can be noted.

5.3 At the conclusion of 16 hours of immersion, the material on each beaker shall be gently washed on a No. 10 (2.00 mm) sieve. The material retained shall be washed back into the beaker using distilled water and dried in the oven for 8 hours or to constant weight.

5.4 Repeat steps 5.2 and 5.3 four additional times.

6. Calculations

6.1 The slaking resistance index for each cycle shall be calculated as follows:

$$I_s = \frac{W_F - C}{B - C} \times 100$$

where

I_s = slaking resistance index expressed as a percentage,

W_F = weight of beaker and oven-dried material retained on No. 10 (2.00 mm) sieve after soaking,

B = weight of beaker and oven-dried sample, and

C = weight of beaker.

7. Report

7.1 The report shall include the following:

7.1.1 The mean slaking resistance index for each cycle, and

7.1.2 The mean natural water content of the shale.

7.2 Optional to the report are the following:

7.2.1 Notes on the mechanism and rate of slaking,

7.2.2 Notes on the variability of slaking between samples, and

7.2.3 Notes on the appearance of the unslaked fragments.

8. References

8.1 Chapman, D.R., "Shale Classification Tests and Systems: A Comparative Study", MSCE Thesis and Joint Highway Research Project No. 75-11, Purdue University, West Lafayette, Indiana, June 1975, 90 pp.

8.2 Deo, P., "Shales as Embankment Materials", Ph.D. Thesis and Joint Highway Research Project No. 45, Purdue University, West Lafayette, Indiana, December 1972, 202 pp.

8.3 Indiana State Highway Commission, Division of Material and Tests, "Determining the Slaking Index of Shale", Test Method No. Ind. 503-73, Indianapolis, Indiana.

8.4 Noble, David F., "Accelerated Weathering of Tough Shales", Final Report VHTRC 78-R20, Virginia Highway and Transportation Research Council, Charlottesville, Virginia, October 1977, 38 pp.

8.5 Surendra, M., "Additives to Control Slaking in Compacted Shales", Ph.D Thesis and Joint Highway Research Project No. 80-6, Purdue University, West Lafayette, Indiana, May 1980, 277 pp.

9. Example

9.1 Table A.1 shows the data collection and results of this test for a typical shale.

TABLE A.1: Typical Five-Cycle Slaking Test Data

FIVE-CYCLE SLAKE RESISTANCE TEST

PROJECT Example Test SHALE Osgood SHEET NO. 1
DESCRIPTION Soft, gray shale which breaks into flaggy to massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWG DATE 4-8-80

NATURAL WATER CONTENT

	SAMPLE 1	SAMPLE 2	SAMPLE 3	SAMPLE 4	SAMPLE 5	SAMPLE 6	MEAN
(1) BEAKER NO.	1	2	3	4	5	6	
(2) WT. OF BEAKER, g	277.1	302.5	301.7	275.4	275.4	286.0	
(3) WT. OF WET SAMPLE AND BEAKER, g	399.3	459.2	461.4	392.8	429.1	440.9	
(4) WT. OF DRY SAMPLE AND BEAKER, g	396.9	455.6	458.0	390.6	425.9	437.4	
(5) WT. OF DRY SAMPLE, g	119.8	153.1	156.3	115.2	150.5	151.4	
(6) WT. OF WATER, g	2.4	3.6	3.4	2.2	3.2	3.5	
(7) NATURAL WATER CONTENT, (%)	2.0	2.4	2.2	1.9	2.1	2.3	2.1

SLAKING INDEXES

CYC. 1	(8) WT. OF DRY RETAINED SAMPLE + BEAKER, g	392.3	450.7	452.1	387.3	422.3	429.2	
	(9) WT. OF DRY RETAINED SAMPLE, g	115.2	148.2	150.4	111.9	146.9	143.2	
	(10) SLAKING INDEX (9)/(5), (%)	96.2	96.8	96.2	97.1	97.6	94.6	96.4
CYC. 5	(20) WT. OF DRY RETAINED SAMPLE + BEAKER, g	383.3	441.2	441.4	380.5	412.0	406.0	
	(21) WT. OF DRY RETAINED SAMPLE, g	106.2	138.7	139.7	105.1	136.6	120.0	
	(22) SLAKING INDEX (21)/(5), (%)	88.6	90.6	89.4	91.2	90.8	79.3	88.3

DESCRIPTION OF SHALE FRAGMENTS AND SLAKING FLUID AFTER TESTING: Shale slaked into fine material. Slaking fluid cloudy.

TABLE A.1 (con't)

FIVE-CYCLE SLAKE RESISTANCE TEST

PROJECT Example Test SHALE Osgood SHEET NO. 2

DESCRIPTION Soft, gray shale which breaks into flaggy to massive chunks

LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown

SOURCE Test Pit TESTED BY MWO DATE 4-8-80

	SLAKING INDEXES	SAMPLE 1	SAMPLE 2	SAMPLE 3	SAMPLE 4	SAMPLE 5	SAMPLE 6	MEAN
CYC. 2	(11) WT. OF DRY RETAINED SAMPLE + BEAKER, g							
	(12) WT. OF DRY RETAINED SAMPLE, g							
	(13) SLAKING INDEX (12)/(5), %							
CYC. 3	(14) WT. OF DRY RETAINED SAMPLE + BEAKER, g							
	(15) WT. OF DRY RETAINED SAMPLE, g							
	(16) SLAKING INDEX (15)/(5), %							
CYC. 4	(17) WT. OF DRY RETAINED SAMPLE + BEAKER, g							
	(18) WT. OF DRY RETAINED SAMPLE, g							
	(19) SLAKING INDEX (18)/(5), %							

DESCRIPTION OF SHALE FRAGMENTS AND SLAKING FLUID AFTER TESTING:

Slake Durability Test

1. Scope

   1.1 This method covers the determination of the slake durability index of a shale or similar material after drying and wetting cycles with abrasion.

2. Definition

   2.1 Slake durability index, the percentage, by weight, retained of a collection of shale pieces on a No. 10 (2.00 mm) sieve after 10 minutes of soaking in water with a standard tumbling and abrasion action.

3. Apparatus

   3.1 Slake durability device, the drum (see Figure A.1) shall be made of No. 10 (2.00 mm) square-mesh, woven-wire cloth, conforming to the requirements of AASHTO M92. It shall be cylindrical in shape, with a diameter of 5.5 in. (140 mm) and a length of 3.9 in. (100 mm). The ends shall be rigid plates, with one end being removable. It must be sufficiently strong to retain its shape during use, but neither the exterior of the mesh nor the interior of the drum shall be obstructed by a support. The drum shall be able to withstand a temperature of 230 $\pm$ 9 F (110 $\pm$ 5 C). A trough shall support the drum in a horizontal manner such that the drum is free to rotate about its axis. The trough shall be capable of being filled with

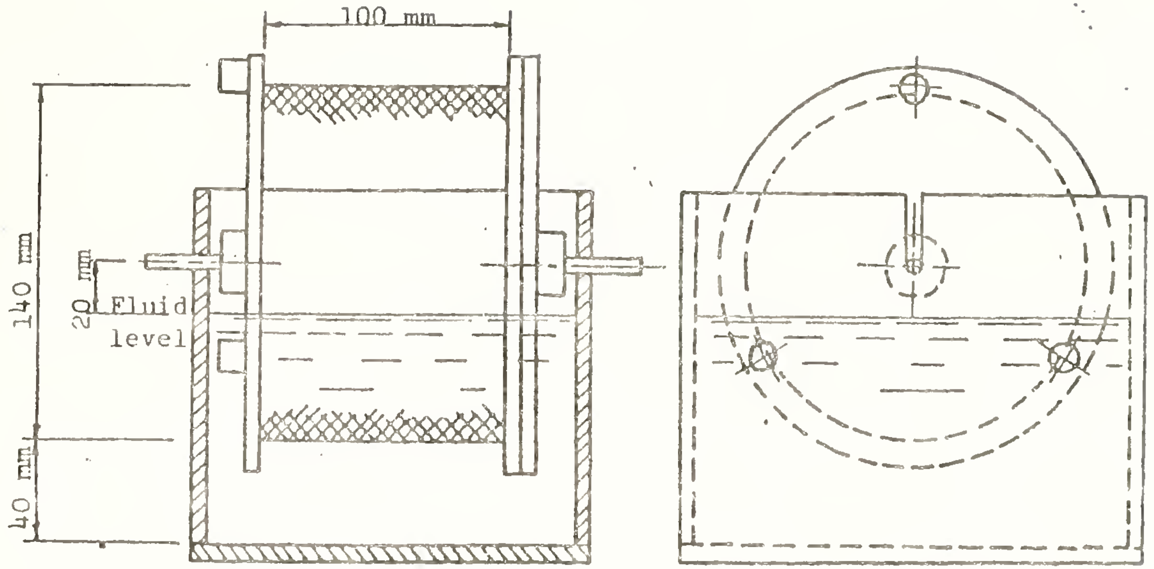


FIGURE A.1: Critical Dimensions of Slake Durability Equipment slaking fluid to 0.8 in. (20 mm) below the drum axis, and shall allow at least 1.6 in. (40 mm) unobstructed clearance between the trough and the bottom of the mesh. The drum shall be rotated by a motor capable of maintaining a speed of 20 rpm, constant to within 5 percent, for a period of 10 minutes.

3.2 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 $\pm$ 9 F (110 $\pm$ 5 C).

3.3 Balance, sensitive to 1 g and having a 1000 g capacity.

3.4 Miscellaneous apparatus, distilled water, brush.

4. Test Samples

4.1 The samples shall consist of 10 representative, intact, roughly equidimensional shale fragments weighing 40 g to 60 g each. Any sharp corners shall be broken off and any dust shall be removed by brushing the sample just prior to weighing. The total sample shall weigh 450 g to 550 g.

4.2 The sample shall be transported and stored in such a manner as to retain the natural water content.

5. Procedure

5.1 The shale fragments shall be placed in the drum, weighed, and dried in the oven for 16 hours or to constant weight. Allow the shale and drum to cool at room temperature for 20 minutes and weigh again. Calculate the natural water content as follows:

$$w = \frac{A - B}{B - C} \times 100$$

where w = percentage water content,

A = weight of drum plus sample at natural moisture content,

B = weight of drum plus oven-dried sample before the first cycle, and

C = weight of drum.

5.2 The drum shall be mourted in the trough and coupled to the motor. The trough shall be filled with distilled water at 68 F (20 C) to 0.8 in. (20 mm) below the drum axis. If specified, another fluid may be used in place of the distilled water. The drum shall be rotated at 20 rpm for a period of 10 minutes.

5.3 The drum shall be removed from the trough immediately after the rotation period is complete and the drum and the sample retained shall be dried in the oven for 16 hours or to constant weight.

5.4 The drum and sample shall be weighed to obtain the oven-dried weight for the second cycle. Steps 5.2 and 5.3 shall be repeated. Again weigh the drum and sample to obtain a final weight.

6. Calculations

6.1 The slake durability index (second cycle) shall be calculated as follows:

$$I_d(2) = \frac{W_F - C}{B - C} \times 100$$

where $I_d(2)$ = slake durability index (second cycle),

B = weight of drum plus oven-dried sample,

W_F = weight of drum plus oven-dried sample retained after the second cycle, and

C = weight of drum.

7. Report

7.1 The report shall include the following:

7.1.1 The slake durability index (second cycle) to the nearest 0.1 percent.

7.1.2 The nature and temperature of the slaking fluid, and

7.1.3 The natural water content.

7.2 Optional to the report are the following:

7.2.1 Notes on the appearance of the fragments retained in the drum, and

7.2.2 Notes on the appearance of the material passing through the drum.

8. References

8.1 Chapman, David R., "Shale Classification Tests and Systems: A Comparative Study", MSCE Thesis and Joint Highway Research Project No. 75-11, Purdue University, West Lafayette, Indiana, June 1975, 90 pp.

8.2 Deo, P., "Shales as Embankment Materials", Ph.D. Thesis and Joint Highway Research Project No. 45, Purdue University, West Lafayette, Indiana, December 1972, 202 pp.

8.3 Franklin Trow Associates, "Field Evaluation of Shales for Construction Projects", Research and Development Project No. 1404, Ministry of Transportation and Communications, Research and Development Branch, Downsview, Ontario, March 1979.

8.4 International Society for Rock Mechanics", Suggested Methods for Determining Slake-Durability Index Properties", Commission on Standardization of Laboratory and Field Tests, November 1972.

8.5 Lutton, Richard J., "Design and Construction of Compacted Shale Embankments, Volume 3, Slaking Indexes for Design", Report No. FHWA-RD-77-1, Federal Highway Administration, Washington, D.C. February, 1977.

8.6 Surendra, M., "Additives to Control Slaking in Compacted Shales", Ph.D Thesis and Joint Highway Research Project No. 80-6, Purdue University, West Lafayette, Indiana, May 1980, 277 pp.

9. Equipment Suppliers

9.1 Engineering Laboratory Equipment, Inc.
2205 Lee St.
Evanston, IL 60202
Phone (312) 869-0420

9.2 Wykeham Farrance, Inc.
Raleigh, North Carolina 27622
Phone (919) 787-0703

10. Example

10.1 Table A.2 shows the data collection and results of this test for a typical Indiana shale.

TABLE A.2: Slake Durability Test Data

SLAKE DURABILITY TEST

PROJECT Example Test SHEET NO. 1
SHALE Palestine II DATE 5-9-81
DESCRIPTION Soft, dark gray shale consisting of flaggy to massive pieces
LOCATION Unknown SAMPLE NO. 1
SOURCE Test Pit DEPTH Unknown
SLAKING FLUID water (room temp) TESTED BY MWO

NATURAL WATER CONTENT

(1) WT. OF DRUM, g	1215.0
(2) WT. OF WET SAMPLE AND DRUM, g	1712.2
(3) WT. OF DRY SAMPLE AND DRUM, g	1687.0
(4) WT. OF WATER, g	25.2
(5) WT. OF DRY SAMPLE, g	472.0
(6) NATURAL WATER CONTENT, %	5.3

SLAKE DURABILITY INDEX (FIRST CYCLE)

(7) WT. OF DRUM, g	1215.0
(8) WT. OF RETAINED DRY SAMPLE AND DRUM, g	1584.6
(9) WT. OF RETAINED DRY SAMPLE, g	369.6
(10) SLAKE DURABILITY INDEX (FIRST CYCLE) ((9)/(5)) x 100%	78.3

SLAKE DURABILITY INDEX (SECOND CYCLE)

(11) WT. OF DRUM, g	1215.0
(12) WT. OF RETAINED DRY SAMPLE AND DRUM, g	1468.9
(13) WT. OF RETAINED DRY SAMPLE, g	253.9
(14) SLAKE DURABILITY INDEX (SECOND CYCLE) ((13)/(5)) x 100%	53.8

DESCRIPTION OF SHALE FRAGMENTS AND SLAKING FLUID AFTER TESTING

The shale fragments were flaky in shape. The slaking fluid was clear.

Point Load Strength Index

1. Scope

   1.1 This method covers the determination of the point load strength index of an irregular lump of shale or similar material.

2. Definition

   2.1 Point load strength index, the ratio of the force required to fail a specimen of shale or similar material between two standard platens to the square of the initial distance between the platens.

   2.2 Secant modulus to failure, the ratio of the point load strength index to the engineering strain at failure.

3. Apparatus

   3.1 Point load strength device - The testing machine shall include the following essential features:

      3.1.1 The loading system shall be adjustable to accept and test rock specimens in the size range of 25 to 100 mm and shall have a load capacity of at least 50 kN. Ram friction shall be low enough as not to impair the accuracy of the load measurement.

      3.1.2 Spherically truncated conical platens are used to transmit load to the specimen. The 60° cone and 5 mm radius spherical truncation shall meet tangentially, and the platens shall be hardened so that they remain undamaged during testing. The platens shall be accurately aligned so that each is coaxial with the other, and the machine shall be rigid to ensure that the platens remain aligned during testing. No spherical seat or other non-rigid component is permitted in the loading system.

3.1.3 A load measuring system shall indicate the failure load to an accuracy of ± 2%, irrespective of the strength of the specimen tested. It shall incorporate a maximum indicating device so that the reading is retained and can be recorded after specimen failure It shall be resistant to hydraulic shock and vibration so that the accuracy of the readings is maintained during testing.

3.1.4 A distance measuring system shall indicate the distance between platen-contact points to an accuracy of ± 0.5 mm. It shall be designed to allow zero check and adjustment.

3.2 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 ± 9 F (110 ± 5 C).

3.3 Balance, sensitive to 0.01 g.

4. Test Samples

4.1 The sample shall consist of at least 20 lumps of rock, each with a diameter greater than 25 mm (1.0 in.) with a ratio of longest to shortest diameter of 1.0 to 1.4. They are trimmed using any convenient technique.

4.2 The sample shall be transported and stored in such a manner as to retain the natural water content.

5. Procedure

5.1 Each lump shall be cleaned of loose material, then placed, approximately centered, between the platens with the bedding planes perpendicular to the axis of the platens. The platens shall be adjusted so the lump is being held in place with minimum force. The distance between the platen-rock contact points shall be measured and recorded using the measuring device mounted on the testing apparatus. Load the platens (at a constant rate of strain of 0.01 inches per minute) until failure of the specimen. Record the force required to fail the specimen and the deformation at failure. Repeat this procedure for the remainder of the sample.

5.2 A water content analysis shall be performed on several representative broken fragments.

6. Calculations

6.1 The point load strength index shall be calculated as follows:

$$I_p = F/d^2$$

where

I_p = point load strength index,

F = compressive load at failure, and

d = initial distance between platens.

The point load strength index shall be corrected to an equivalent index for a sample of 50 mm using the chart given in Figure A.2. This index shall be denoted I_p (50).

6.2 The value of the secant modulus to failure shall be calculated as follows:

$$E_f = \frac{I_p(50)}{(d_f/d)}$$

where

E_f = value of the secant modulus to failure,

d_f = deformation to failure, and

d = initial distance between platens.

6.3 The water content shall be calculated as follows:

$$w = \frac{A - B}{B - C} \times 100$$

where

w = percentage water content,

A = weight of container and material,

B = weight of container and oven-dried material, and

C = weight of container.

7. Report

7.1 The report shall include the following:

7.1.1 The median corrected point load strength index, $I_p(50)$;

7.1.2 The range of the corrected point load strength index, $I_p(50)$;

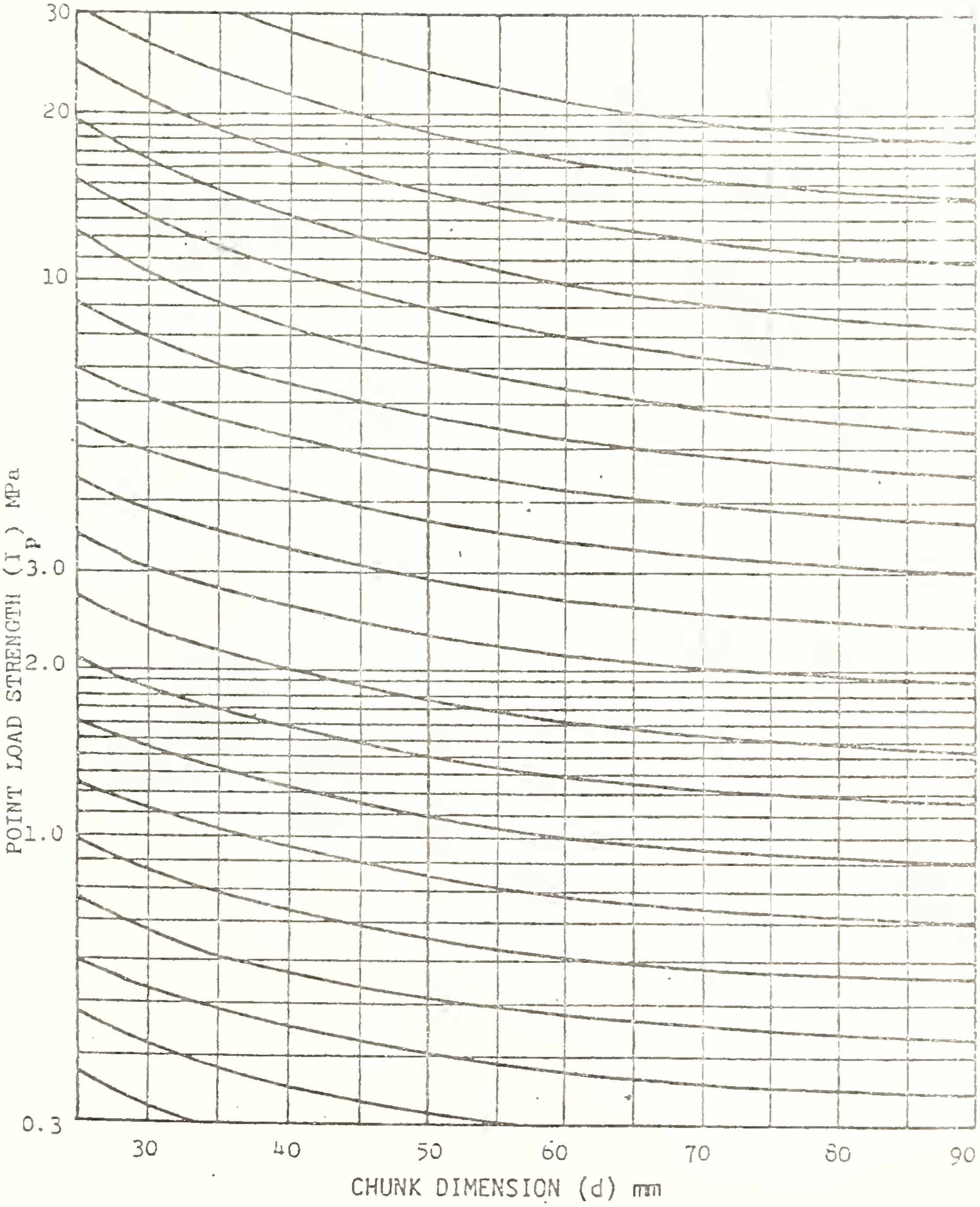


FIGURE A.2: Point Load Strength Test Size Correction Chart
(After reference 2)

7.1.3 The median natural water content of the sample;

7.1.4 The range of the natural water content;

7.1.5 The median secant modulus, E_f, to failure;

7.1.6 The range of the secant modulus, E_f, to failure;

7.1.7 The direction of loading with respect to bedding planes.

8. References

8.1 Bailey, Michael, J., "Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments", MSCE Thesis and Joint Highway Research Project No. 76-23, Purdue University, West Lafayette, Indiana, August 1976, 209 pp.

8.2 Franklin Trow Associates, "Field Evaluation of Shales for Construction Projects", Research and Development Project No. 1404, Ministry of Transportation and Communications, Research and Development Branch, Downsview, Ontario, March 1979.

8.3 Hale, Barney, C., "The Development and Application of a Standard Compaction-Degradation Test for Shales", MSCE Thesis and Joint Highway Research Project No. 79-21, Purdue University, West Lafayette, Indiana, October 1979, 180 pp.

8.4 Hale, B. C., Lovell, C. W. and Wood, L. E., "Development of a Compaction-Degradation Test for Shales", TRR 790, Transportation Research Board, Washington, D.C. July 1981, pp. 45-52.

8.5 Hale, B.C., Lovell, C.W. and Wood, L.E., "Factors Affecting Degradation and Density of Compacted Shales" Proceedings, International Symposium on Weak Rocks, Tokyo, Japan, Vol. I, Sept. 1981, pp. 321-326.

8.6 International Society for Rock, "Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and the Point Load Strength Index", Commission on Standardization of Laboratory and Field Tests, October 1972.

8.7 Surendra, M., "Additives to Control Slaking in Compacted Shales", Ph.D. Thesis and Joint Highway Research Project No. 80-6, Purdue University, West Lafayette, Indiana, May 1980, 277 pp.

9. Equipment Suppliers

9.1 Engineering Laboratory Equipment, Inc.
2205 Lee St.
Evanston, IL 60202
Phone (312) 869-0420

9.2 G.D.I., Inc.
P. O. Box 66310
Chicago, IL 60666
Phone (312) 439-2290

9.3 Soiltest, Inc.
2205 Lee St.
Evanston, IL 60202
Phone (312) 869-5500

9.4 Terrametrics, Inc.
16027 W. 5th Ave.
Golden, CO 80401
Phone (303) 279-7813

9.4 Wykeham Farrance, Inc.
Raleigh, North Carolina 27622
Phone (919) 787-0703

10. Example

10.1 Table A.3 shows the data collection and results of this test for a typical Indiana shale.

TABLE A.3: Typical Point Load Strength Test Data

POINT LOAD STRENGTH TEST

PROJECT Example Test SHALE New Providence SHEET NO. 1
DESCRIPTION Hard, dark gray shale breaking into massive pieces
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 10-8-81

POINT LOAD STRENGTH INDEX

	1	2	3	4	5	6	7*	8	9	10
INITIAL DIAMETER (d) mm	36.7	26.7	30.9	39.8	43.6	26.9	32.1			
LOAD AT FAILURE (F) N	2070	3190	2070	3280	2535	2446	6289			
POINT LOAD STRENGTH (I_p) MPa	1.53	4.44	2.17	2.07	1.33	3.37	6.12			
CORR. POINT LOAD STRENGTH (I_p (50)) MPa	1.42	3.73	1.87	1.88	1.80	2.77	5.17			

SECANT STRESS-STRAIN MODULUS TO FAILURE

DEFORMATION (d_f) mm	4	2	3	4	5	3	3			
SECANT MODULUS (E_f) MPa	13.0	49.8	19.3	19.4	15.7	24.8	55.3			

NATURAL WATER CONTENT

CONTAINER NO.		5			69					
WT. OF CONTAINER, g		36.3			35.9					
WT. OF CONTAINER AND WET SAMPLE, g		356.3			262.1					
WT. OF CONTAINER AND DRY SAMPLE, g		353.4			260.2					
WT. OF WATER, g		2.8			1.9					
WT. OF DRY SAMPLE, g		317.1			304.3					
NATURAL WATER CONTENT (%)		0.88			0.62					

DIRECTION OF LOADING WITH RESPECT TO BEDDING PLANES: Normal

COMMENTS: * Did not break cleanly - results ignored.
Deformations are estimated.

TABLE A.3 (con't)

POINT LOAD STRENGTH TEST

PROJECT Example Test SHALE New Providence SHEET NO. 2
DESCRIPTION Hard, dark gray shale breaking into massive pieces
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 10-8-81

POINT LOAD STRENGTH INDEX

	11	12	13	14	15	16	17	18	19	20
INITIAL DIAMETER (d) mm										
LOAD AT FAILURE (F) N										
POINT LOAD STRENGTH (I_p) MPa										
CORR. POINT LOAD STR. ($I_p(50)$) MPa										

SECANT STRESS-STRAIN MODULUS TO FAILURE

DEFORMATION (d_f) mm										
SECANT MODULUS (E_f) MPa										

NATURAL WATER CONTENT

CONTAINER NO.										
WT. OF CONTAINER, g										
WT. OF CONTAINER + WET SAMPLE, g										
WT. OF CONTAINER + DRY SAMPLE, g										
WT. OF WATER, g										
WT. OF DRY SAMPLE, g										
NATURAL WATER CONTENT (%)										

$I_p(50)$, MPa, Median 1.88 Range 1.42 to 3.73
Nat. Water Content, %, Median 0.75 Range 0.62 to 0.88
E_f, MPa, Median 19.4 Range 13.0 to 49.8

Impact Compaction-Degradation Test

1. Scope

 1.1 This method covers the determination of the index of crushing for shale impact compaction.

2. Definition

 2.1 Index of crushing, the percent change in the mean aggregate size of a standard gradation of shale during impact compaction using a specific amount of energy.

3 Apparatus

 3.1 Mold - The mold shall be made of rigid metal, cylindrical in shape, with an inside diameter of 6.00 in. (152.4 mm) and having a capacity of (1/11.33) $\pm$ 0.0004 ft^3 (2.125 $\pm$ 11 cm^3). It shall be made of two halves capable of being split along the axis and held together by bolts through a flange. The mold shall be provided with a removable collar made of rigid metal with an inside diameter of 6.00 in. (152.4 mm) and a height of at least 2.0 in. (50.8 mm).

 3.2 Rammer, a metal rammer weighing 10.0 lb (4.54 kg), having a 2.0 in. (50.8 mm) diameter circular striking face, and a controlled free fall drop height of 18.0 in. (456 mm) as specified in AASHTO T180. Automatic rammers or sliding weight rammers may be used, provided the compactive effort is the same as that given by the comparable rammers described in AASHTO T180.

3.3 Sieves, a series of sieves of square-mesh, woven-wire cloth, conforming to the requirements of AASHTO M92. The sieves required are as follows:

1 1/2 in.	(38.1 mm)
3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)

3.4 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 $\pm$ 9 F (110 $\pm$ 5 C) for drying moisture content samples.

3.5 Balances - The balance shall have a 30 lb (13.6 kg) capacity and be capable of weighing to the nearest 0.01 lb (5 g).

3.6 Miscellaneous apparatus, filter paper, large mixing pan, brush, spoon, reciprocating jaw crusher, hammer, moisture containers, straight edge.

4. Sample

4.1 The sample shall consist of representative shale specimens small enough to be broken with a hand hammer yet large enough to yield 1 1/2 in. (38.1 mm) aggregates when crushed.

4.2 Approximately 20 lb (9.0 kg) should be sufficient for most intact shales.

4.3 The sample shall be transported and stored in such a manner as to retain the natural water content.

5. Preparation of Test Specimen

5.1 The sample shall be broken with a hammer into fragments small enough to be crushed by a reciprocating jaw crusher adjusted to yield aggregates at least 1 1/2 in. (38.1 mm) in size.

5.2 The product of the jaw crusher shall be sorted by passing the material through a nest of sieves composed of the following:

1 1/2 in.	(38.1 mm)
3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)
Pan	

5.3 Each test specimen shall weigh 11.0 lb (5.0 kg) prepared by combining the material retained on each sieve in the following proportions:

$$P = 100\ (d/D)$$

where

P = percentage, by weight, finer than size d,

d = sieve size, and

D = maximum aggregate diameter.

The standard gradation shall have a maximum aggregate diameter of 1 1/2 in. (38.1 mm). The percent by weight retained on sieve size d shall be calculated as follows:

$$P_{RI} = P_2 - P$$

where

P_{RI} = percentage, by weight, retained on sieve size d before compaction,

P = percentage, by weight, finer than sieve size d, and

P_2 = percentage, by weight, finer than the next larger sieve size above size d.

5.4 At least four test specimens shall be prepared.

6. Procedure

6.1 Compact the shale in the 6.00 in. (152.4 mm) mold (with collar attached) in three equal layers to give a total compacted depth not to exceed 5.0 in. (127 mm), each layer being compacted by 25 uniformly distributed blows from the rammer. During compaction, the mold shall rest on a uniform, rigid foundation, such as is provided by a cylinder or cube of concrete weighing not less than 200 lb (90.7 kg). Following compaction, remove the extension collar and carefully trim the compacted shale even with the top of the mold using the straight edge as a guide. Weigh the mold and compacted shale. Multiply the weight of the mold and shale, minus the weight of the mold, by 13.33 (or divide by 2123.76). Record the result as the wet density, ρ_m, in pounds per cubic foot (kilograms per cubic meter) of the compacted shale.

6.2 The weighed material in the mold shall be recombined with the excess compacted material trimmed from the top. If the material is cohesive and does not pour from the mold freely, the mold shall be removed from the base plate and split by unbolting the two halves. The aggregates shall then be gently separated by hand. The material shall be sorted by passing it through the same nest of sieves as used for the preparation of the test specimen in such a way as to minimize additional degradation to the specimen. The percentage by weight retained on each sieve shall be calculated as follows:

$$P_{RF} = (W_d/W_t)100$$

where

P_{RF} = percentage, by weight, retained on sieve size d after compaction,

W_d = weight retained on sieve size d, and

W_t = total weight of the compacted material.

6.3 A representative sample weighing at least 1.0 lb (0.45 kg) shall be taken from the compacted material for a moisture content sample.

7. Calculations

7.1 Calculate the index of crushing of the shale for each trial, as follows:

$$IC = \frac{\Sigma \bar{d} P_{RI} - \bar{d} P_{RF}}{\Sigma \bar{d} P_{RI}}$$

where

IC = index of crushing,

$\bar{d}$ = mean grain size retained on sieve size d,

P_{RI} and P_{RF} are previously defined.

7.2 Calculate the moisture content of the shale for each trial, as follows:

$$w = \left[(A - B)/(B - C)\right] 100$$

where

w = percentage of moisture in the specimen,

A = weight of container and wet shale,

B = weight of container and dry shale, and

C = weight of container.

8. Report

8.1 The report shall include the following:

8.1.1 The compacted wet density (ρ_m) of the shale,

8.1.2 The index of crushing,

8.1.3 The water content,

8.1.4 The type of rammer face if other than 2 in. (50.8 mm) circular, and

8.1.5 A plot of cumulative percentage, by weight retained, versus sieve size for before and after compaction.

9. References

9.1 Bailey, Michael J., "Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments", MSCE Thesis and Joint Highway Research Project No. 76-23, Purdue University, West Lafayette, Indiana, August 1976, 209 pp.

9.2 Hale, Barney C., "The Development and Application of a Standard Compaction-Degradation Test for Shales", MSCE Thesis and Joint Highway Research Project No. 79-21, Purdue University, West Lafayette, Indiana, October 1979, 180 pp.

7.3 Hale, B. C., Lovell, C. W. and Wood, L. E., "Development of a Compaction-Degradation Test for Shales", TRR 790, Transportation Research Board, Washington, D.C. July 1981, pp. 42-52.

9.4 Hale, B. C., Lovell, C. W. and Wood, L. E., "Factors Affecting Degradation and Density of Compacted Shales", Proceedings, International Symposium on Weak Rocks, Tokyo, Japan. Vol. I, Sept. 1981, pp. 321-326.

10. Example

10.1 Table A.4 and Figure A.3 show the data collection and results of this test for a typical Indiana shale.

TABLE A.4: Typical Impact Compaction-Degradation Test Data

IMPACT COMPACTION-DEGRADATION TEST

PROJECT Example Test SHALE Osgood SHEET NO. 1
DESCRIPTION Soft, gray shale which breaks into flaggy to massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 5-1-81
MOLD NO. 1 VOLUME 1/13.33 cu. ft. WEIGHT ---

		BEFORE IMPACT (CALCULATED)			AFTER IMPACT (MEASURED)		
SIEVE	MEAN SIZE (in.)	% RETAINED	WEIGHT RETAINED (lb)	MEAN X %	WEIGHT RETAINED (lb)	% RETAINED	MEAN X %
1 1/2 in.	-	0	0	0	0	0	0
3/4 in.	1.125	50.0	5.5	56.25	2.85	30.25	34.03
3/8 in.	0.5625	25.0	2.75	14.06	2.68	28.45	16.00
#4	0.2813	12.5	1.375	3.52	1.60	16.99	4.78
#8	0.1406	6.25	0.6875	0.88	0.98	10.40	1.46
PAN	0.0469	6.25	0.6875	0.29	1.31	13.91	0.65
TOTAL		100.0	11.0	(A) 75.00	9.42	100.0	(B) 56.92

WATER CONTENT

(1) WT. OF CONTAINER NO. 1, (g) 538.1
(2) WT. OF WET SAMPLE AND CONTAINER (g) 2835.3
(3) WT. OF DRY SAMPLE AND CONTAINER (g) 2801.4
(4) WATER CONTENT ((2)-(3))/((3)-(1)), % 1.5

INDEX OF CRUSHING

(5) IC = ((A)-(B))/(A), % 24.0
(6) ρ_m (lb/ft^3) 119.9
(7) Type of rammer face std.

IMPACT COMPACTION-DEGRADATION TEST

PROJECT Example Test	SHEET NO. 2
SHALE Osgood	DATE 5-1-81
DESCRIPTION Soft, gray shale which breaks into flaggy to massive chunks	
LOCATION Unknown	SAMPLE NO. 1
SOURCE Test Pit	DEPTH Unknown
TESTED BY MWO	RAMMER FACE Std.

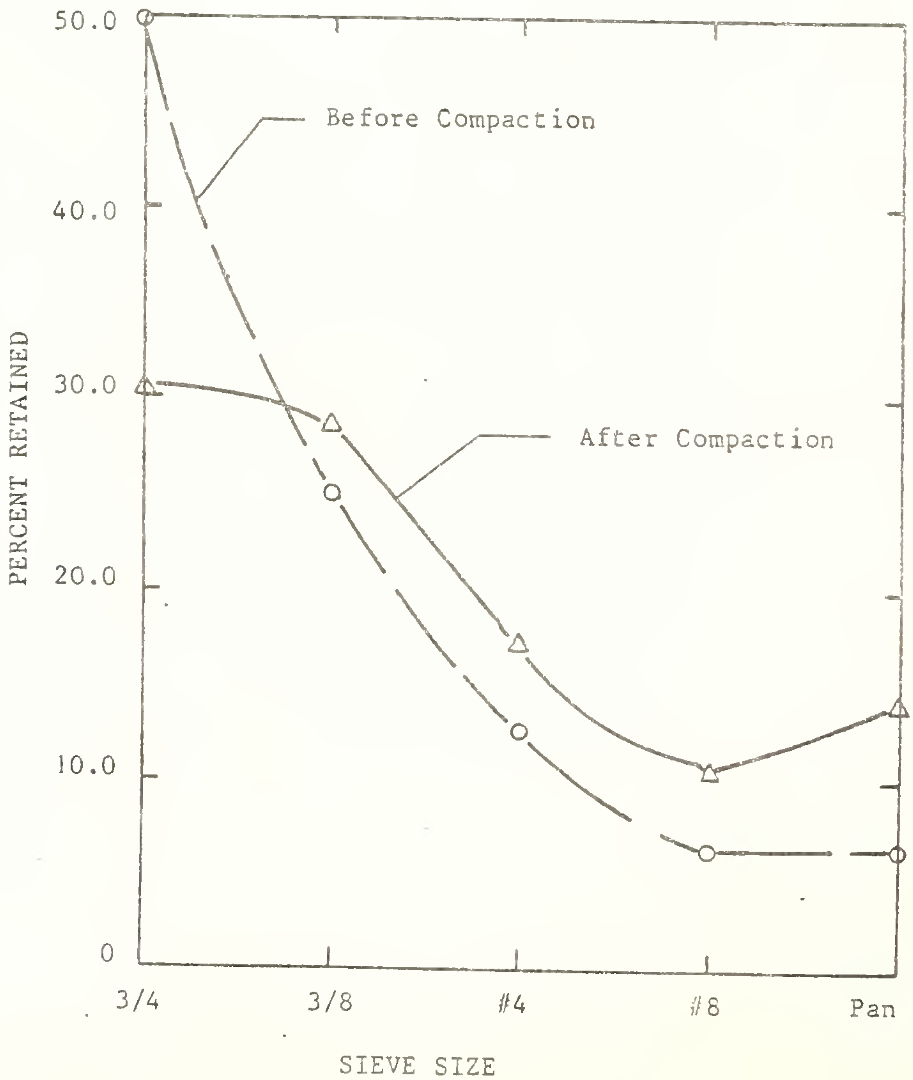


Figure A.3 Gradation Before and After Impact Compaction

Moisture-Density Relations of Shale

1. Scope

   1.1 This method covers the determination of the relationship between the moisture content and density of a shale or similar material of given gradation when compacted in a mold of given size with a 10 lb (4.5 kg) rammer dropped from a height of 18 in. (45.7 cm).

2. Definitions

   2.1 Optimum moisture content - When the dry densities and corresponding moisture contents for the shale are determined, plotted, and connected by a smooth line, a curve is produced. The moisture content corresponding to the peak of the curve shall be termed the optimum moisture content of the shale under the described compaction.

   2.2 Maximum Density - The oven-dry density in pounds per cubic foot or kilograms per cubic meter of the shale at optimum moisture content shall be termed the maximum density under the described compaction.

3. Apparatus

   3.1 Mold - The mold shall be made of rigid metal, cylindrical in shape, with an inside diameter of 6.00 in. (152.4 mm) and having a capacity of (1/11.33) $\pm$ 0.0004 ft^3 (2,125 $\pm$ 11 cm^3).

The mold shall be provided with a removable extension collar made of rigid metal with an inside diameter of 6.00 in. (152.4 mm) and a height of at least 2.0 in. (50.8 mm).

3.2 Rammer, a metal rammer weighing 10.0 lb (4.54 kg), having a 2.0 in. (50.8 mm) diameter circular striking face, and a controlled free fall drop height of 18.0 in. (457 mm) as specified in AASHTO T180. Automatic rammers or sliding weight rammers may be used, provided the compactive effort is the same as that given by the comparable rammers described in AASHTO T180.

3.3 Sieves, a series of sieves of square-mesh, woven-wire cloth, conforming to the requirements of AASHTO M92. The sieves required are as follows:

1 1/2 in.	(38.1 mm)
3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)
Pan	

3.4 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 ± 9 F (110 ± 5 C).

3.5 Balances

3.5.1 The balance used for the water content determination shall have a 1000 g capacity and be sensitive to 0.01 g.

3.5.2 The balance used for all other weighing shall have a 30 lb (13.6 kg) capacity and be sensitive to 0.01 lb (5 g).

3.6 Miscellaneous apparatus, filter paper, large mixing pan, large plastic bags, mixing spoon, spray bottle, distilled water, brush, reciprocating jaw crusher, hammer, moisture content cans, straight edge.

4. Sample

4.1 The sample shall consist of representative rock specimens small enough to be broken with a hand hammer yet large enough to yield 1 1/2 in. (38.1 mm) aggregates when crushed.

4.2 The sample shall be large enough to yield at least four test specimens. Approximately 80 lb (36.3 kg) should be sufficient for most intact shales.

4.3 The sample shall be transported and stored in such a manner as to retain the natural water content.

5. Preparation of Test Specimen

5.1 The sample shall be broken with a hammer into fragments small enough to be crushed by a reciprocating jaw crusher adjusted to yield a maximum aggregate size of at least 1 1/2 in. (38.1 mm).

5.2 The product of the jaw crusher shall be sorted by passing the material through a nest of sieves composed of the following:

1 1/2 in.	(38.1 mm)
3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)
Pan	

5.3 Each test specimen shall weigh 11.0 lb (5.0 kg) prepared by combining the material retained on each sieve in the following proportions:

$$P = 100\ (d/D)$$

where

P = percentage, by weight, finer than size d,

d = sieve size, and

D = maximum aggregate diameter

5.4 A range of moisture contents of 5 percentage points and encompassing the optimum moisture content, shall be obtained by allowing the test specimen to dry or by dampening with distilled water using a spray bottle. Each specimen shall be sealed in a plastic bag and allowed to cure for 48 hours. At least two tests shall be at or above the optimum moisture content and two tests shall be at or below the optimum moisture content. The standard impact-degradation test may be used to define the compacted density at the natural moisture content, assuming the compactive efforts used in the two tests are the same.

6. Procedure

6.1 Compact the shale in the 6.00 in. (152.4 mm) mold (with collar attached) in three equal layers to give a total compacted depth not to exceed 5.0 in. (127 mm), each layer being compacted by 25 uniformly distributed blows from the rammer. During compaction, the mold shall rest on a uniform, rigid foundation, such as is provided by a cylinder or cube of concrete weighing not less than 200 lb (90.7 kg). Following compaction, remove the extension collar and carefully trim the compacted shale even with the top of the mold using the straight edge as a guide. Weigh the mold and compacted shale. Multiply the weight of the mold and shale, minus the weight of the mold, by 13.33 (or divide by 2123.76). Record the results as the wet density, ρ_m, in pounds per cubic foot (kilograms per cubic meter) of the compacted shale.

6.2 A representative sample weighing at least 1.0 lb (0.45 kg), shall be taken from the compacted material for a moisture content sample.

6.3 Repeat for test specimens at other water contents. The results from the standard impact-degradation test may serve as one trial, if the compactive effort is the same.

7. Calculations

7.1 Calculate the moisture content and the dry density of the shale as compacted for each trial as follows:

$$w = \left[(A - B)/(B - C)\right] 100$$

and

$$\rho_d = \frac{\rho_m}{100 + w} \times 100$$

where

w = percentage of moisture in the specimen,

A = weight of container and wet shale,

B = weight of container and dry shale,

C = weight of container,

ρ_d = dry density of compacted shale, and

ρ_m = wet density of compacted shale.

8. Report

8.1 The report shall include the following:

8.1.1 A plot of the dry compacted densities of the shale as the ordinate values and the corresponding moisture contents as abscissa values. Draw a smooth curve connecting the plotted points, and defining an optimum moisture content, and a maximum dry density,

8.1.2 The optimum moisture content,

8.1.3 The maximum dry density,

8.1.4 The natural water content,

8.1.5 The type of compaction face if other than 2 in. (50.8 mm) circular,

8.1.6 The compaction effort,

8.1.7 The specific gravity of solids, G_s, and

8.1.8 A plot of the zero air voids curve.

9. References

9.1 Bailey, Michael J., "Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments", MSCE Thesis and Joint Highway Research Project No. 76-23, Purdue University, West Lafayette, Indiana, August 1976, 209 pp.

9.2 Deo, P., "Shales as Embankment Materials", Ph.D. Thesis and Joint Highway Research Project No. 45, Purdue University, West Lafayette, Indiana, December 1972, 202 pp.

9.3 Surendra, M., "Additives to Control Slaking in Compacted Shales", Ph.D. Thesis and Joint Highway Research Project No. 80-6, Purdue University, West Lafayette, Indiana, May 1980, 277 pp.

10. Example

10.1 Table A.5 and Figure A.4 show the data collection and results of this test for a typical Indiana shale.

TABLE A.5 Typical Moisture-Density Relations Test Data

MOISTURE-DENSITY TEST

PROJECT Example Test SHEET NO. 1
SHALE Osgood DATE 5-81
DESCRIPTION Soft, gray shale which breaks into flaggy to massive chunks
LOCATION Unknown SAMPLE NO. 1
SOURCE Test Pit DEPTH Unknown

COMPACTED DENSITY

Point No.	1	2	3	4
Wt. Mold + Soil (lb)	43.59	44.42	45.07	45.11
Wt. Mold (lb)	34.66	34.66	34.66	34.66
Wt. Soil (lb)	8.93	9.86	10.51	10.55
Wet Density, ρ_m (lb/ft^3)	119.1	131.4	140.0	140.7
Dry Density, ρ_d (lb/ft^3)	117.3	123.9	127.7	125.6
Void Ratio, e	0.4363	0.3598	0.3193	0.3414
Porosity, n	0.3038	0.2646	0.2420	0.2545

COMPACTED WATER CONTENT

Point No.	1	2	3	4
Container No.	1	2	3	4
Wt. Container+Wet Soil, g	2835.3	1777.8	2905.0	2119.8
Wt. Container+Dry Soil, g	2801.4	1685.7	2695.4	1911.3
Wt. Water, W_w, g	33.9	92.1	209.6	208.5
Wt. Container, g	538.1	172.1	526.0	170.5
Wt. Dry Soil, W_s, g	2263.3	1513.6	2169.4	1740.8
Water Content, w (%)	1.50	6.08	9.66	11.98

NATURAL WATER CONTENT

(1) WT. OF CONTAINER NO. 1 (g) 538.1
(2) WT. OF WET SAMPLE+CONTAINER (g) 2835.3
(3) WT. OF DRY SAMPLE+CONTAINER (g) 2801.4
(4) NATURAL WATER CONTENT ((2) - (3))/((3) - (1)) % 1.50

MOISTURE - DENSITY RELATIONS TEST

PROJECT	Example Test	SHALE	Osgood	SHEET NO.	1
DESCRIPTION	Soft, gray shale which breaks into flaggy to massive chunks				
LOCATION	Unknown	SAMPLE NO.	1	DEPTH	Unknown
SOURCE	Test Pit	TESTED BY	MWO	DATE	5-81
COMPACTION ENERGY		RAMMER FACE		SPECIFIC GRAVITY OF SOLIDS	

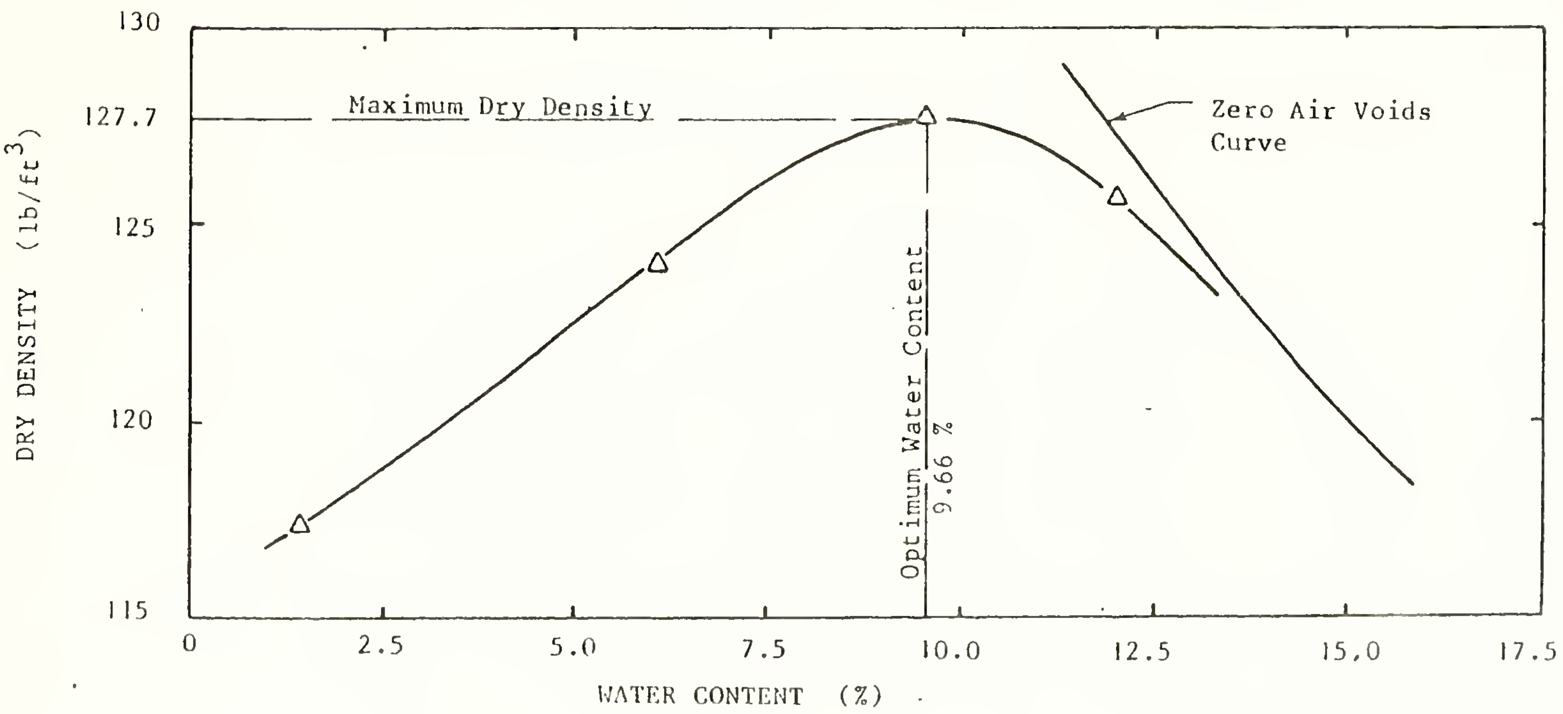


Figure A.4 Typical Moisture-Density Relations Data

One-Dimensional Compression Test

1. Scope

   1.1 This method covers the determination of the one-dimensional, stress-strain relationships for a compacted shale: (a) under loading in the as-compacted condition; (b) with saturation under a series of specified loads; and (c) for unloading and loading in the saturated condition. These relations are needed for predictions of settlement or heave in a compacted shale embankment.

2. Definition

   2.1 Settlement (or heave), the one-dimensional expression of volumetric change occurring when a compacted shale mass is loaded, unloaded, or saturated under load.

   2.2 Compressibility, the one-dimensional volumetric strain as a function of the change in axial load (unload).

   2.3 Prestress, a total stress level below which the compressibility of the compacted shale is relatively low, and above which it is relatively high. The prestress level is established by the roller pressure, and is equal to or less than that pressure.

   2.4 Load increment ratio, the ratio of the load change in a compression test to the previous load; commonly equal to or less than unity.

2.5 Skempton's B parameter, the ratio of undrained pore pressure response to the level of all around total stress; equal to or less than unity.

2.6 Compression index, the ratio of void ratio change to change in the logarithm (base 10) of applied stress in the compression test.

3. Apparatus

3.1 Mold - The mold shall be made of rigid metal, cylindrical in shape, with an inside diameter of 4.00 in. (101.6 mm) and having a capacity of (1/30) $\pm$ 0.0004 ft^3 (944 $\pm$ 11 cm^3). The mold shall be provided with a removable extension collar made of rigid metal with an inside diameter of 4.00 in. (101.6 mm) and a height of at least 2.00 in. (50.8 mm). A base shall be provided which allows the mold to be mounted onto the kneading compactor.

3.2 Kneading compactor, a California kneading compactor or a similar kneading compactor which develops a trace curve of load-time similar to that of the California kneading compactor.

3.3 Compactor foot, a ram having a face shaped as shown in ASTM D1561-76 or AASHTO T247.

3.4 Sieves, a series of sieves of square-mesh, woven-wire cloth, conforming to the requirements of ASTM E11-70 or AASHTO M92. The sieves required are as follows:

3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)

No. 4	(4.75 mm)
No. 8	(2.36 mm)
No. 16	(1.18 mm)
Pan	

3.5 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 ± 9 F (110 ± 5 C).

3.6 Balance - The balance shall have a 5000 g capacity and be sensitive to 0.1 g.

3.7 Load device - The load device shall conform to the standards given for load devices as stated in ASTM D2435-80 or AASHTO T215.

3.8 Consolidometer - The consolidometer shall have an inside diameter of 4.00 in. (101.6 mm) and shall conform to the standards given for consolidometers as stated in ASTM D2435-80 or AASHTO T216.

The consolidometer shall have provisions to monitor the pore water pressure through the use of a pressure transducer at one of the platens.

3.9 Porous stone - The porous stones shall conform to the standards given for porous stones as stated in ASTM D2435-80 or AASHTO T216.

3.10 Miscellaneous apparatus, straight edge, reciprocating jaw crusher, hammer, plastic bags, de-aired water, mixing spoon, spray bottle, moisture containers, compressed air supply, extrusion device, silicone grease, filter paper.

4. Sample

4.1 The sample shall consist of representative rock specimens small enough to be broken with a hand hammer, yet large enough to yield 3/4 in. (19.0 mm) aggregates when crushed.

4.2 The sample shall be transported and stored in such a manner as to retain the natural water content.

5. Preparation of Test Specimen

5.1 The sample shall be broken with a hammer into fragments small enough to be crushed by a reciprocating jaw crusher adjusted to yield aggregates of at least 3/4 in. (19.0 mm) in size.

5.2 The product of the jaw crusher shall be sorted by passing the material through a nest of sieves composed of the following:

3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)
No. 16	(1.18 mm)
Pan	

5.3 The material needed to prepare each test specimen shall weigh 5.0 lb (11.03 kg) and be prepared by combining the material retained on each sieve in the following gradation:

$$P = 100\ (d/D)$$

where

P = percentage, by weight, finer than size d,

d = sieve size, and

D = maximum aggregate size.

The standard gradation shall have a maximum aggregate size of 3/4 in. (19.0 mm). At least four samples shall be needed to determine the appropriate kneading compactor foot pressure to produce the necessary moisture-density combination, plus a sample to determine the compressibility behavior.

5.4 The material shall be wetted to the optimum moisture content as determined by the moisture-density relation test (or to another specified moisture condition) by adding water using a spray bottle, sealing the material in a plastic bag, and allowing it to cure for 24 hours.

5.5 The material shall be compacted in the mold (with collar attached) in five layers to give a total compacted depth not to exceed 5.0 in. (130 mm), each layer being compacted by the kneading compactor for one minute using 30 blows per minute. The foot pressure shall produce a density equal to the density at the optimum moisture content when compacted by impact compaction, as described in the moisture-density relation test (or to another specified moisture-density condition). The foot pressure shall be determined by making a plot of foot pressure versus density consisting of at least four points which straddle the desifed testing density.

5.6 The collar shall be removed from the mold and a straight edge used to trim the specimen flush with the top of the mold. A jack shall be used to extrude the specimen from the mold and push it into the consolidometer ring. Silicone grease shall be used on the walls of the ring to reduce the friction. Trim the sample flush with the consolidometer ring. Immediately use the trimmings for a water content sample, and weigh the ring and shale to determine the actual dry density. Assemble the consolidometer using filter paper between the shale and porous stones. Place the consolidometer in the loading device and apply a small seating load.

6. Procedure

6.1 Each specimen shall be loaded, using a load increment ratio of 0.5 to 0.75, to a predetermined load corresponding to a specific overburden value. Height readings with respect to time shall be recorded. At least one of the specimens shall be loaded sufficiently beyond the point of prestress caused by the compaction process to determine the prestress value. It is suggested that this value be determined by the Casagrande construction [Holtz and Kovacs (1981), page 296].

6.2 The specimen shall be saturated by allowing de-aired water to flow from the bottom to the top platen under a small head. A small vacuum may be applied to aid in de-airing the specimen. Back pressure shall be applied in increments not to exceed 7.0 psi (50 kPa) until saturation is achieved or to the maximum capacity of the pressuring system, allowing at least 60 minutes between each increment for the pressure to equilibrate throughout the sample.

6.3 Height measurements shall be recorded during the entire saturation process and the percent settlement (or heave) shall be calculated. At least 12 hours shall be allowed for the total movement to occur.

6.4 After the movement is complete, drainage from the bottom platen shall be closed, resulting in a singly drained specimen from the top platen, and allowing pore water pressure measurements to be made at the bottom of the specimen by using the pressure transducer.

6.5 A B parameter check shall be conducted for an indication of the degree of saturation. The procedure is as follows:

a) close the drainage from the top platen,

b) apply a known load to the sample,

c) after allowing at least 5 minutes for the pressures inside to reach equilibrium, measure the change in the pore water pressure on the transducer,

d) calculate the B parameter as the ratio of the change in the pore water pressure to the additional load applied,

e) remove the additional load and open the top platen.

6.6 The specimen shall be unloaded using an LIR of 1 to a small seating load, then reloaded using an LIR of 1 to the capacity of the loading frame or some reasonable value. The procedure for unloading and loading and recording measurements shall follow ASTM D2435-80 or AASHTO T216.

6.7 The consolidometer shall be quickly dismantled and the sample pushed from the ring, weighed, and dried in the oven to determine the final moisture content and degree of saturation.

7. Calculations

7.1 Moisture contents shall be calculated from samples taken before and after the test. The moisture content shall be calculated as follows:

$$w = \left[(A - B)/(B - C)\right] 100$$

where

w = percentage of moisture in specimen,

A = weight of container and wet shale,

B = weight of container and dry shale,

C = weight of container.

7.2 The prestress ratio shall be calculated as the value of the prestress divided by the applied foot pressure of the kneading compactor.

7.3 The compression index for both the as-compacted condition and the saturated condition shall be calculated as follows:

$$C_c = -\frac{\Delta e}{\Delta(\log \sigma_c)}$$

The rebound index shall be calculated for the saturated condition as follows:

$$C_R = \frac{\Delta e}{\Delta(\log \sigma_c)}$$

C_R = rebound index for the saturated condition,

C_c = compression index for either the as-compacted condition or the saturated condition,

e = void ratio, and

σ_c = applied vertical stress.

The values of Δe are taken directly from the laboratory curves, over the stress range of primary interest.

8. Report

8.1 The report shall include the following:

8.1.1 Compacted water content,

8.1.2 Compacted dry density,

8.1.3 Applied kneading compactor foot pressure, 8.1.4 The prestress ratio,

8.1.5 The compression index for both the compacted and saturated conditions,

8.1.6 The final degree of saturation,

8.1.7 A plot of the void ratio versus the log of the vertical stress,

8.1.8 The percentage heave or settlement which occurs upon saturation,

8.1.9 Plots of height versus the log of time for each load increment.

9. References

9.1 Abeyesekera, R.A., "Stress-Deformation and Strength Characteristics of a Compacted Shale", Ph.D. Thesis and Joint Highway Research Project No. 77-24, Purdue University, West Lafayette, Indiana, December 1977, 420 pp.

9.2 Abeyesekera, R.A., Lovell, C.W. and Wood, L. E., "Stress-Deformation and Strength Characteristics of a Compacted Shale", Papers of the Conference on Clay Fills, Institution of Civil Engineers, London, England, November 1978, pp. 1-14.

9.3 Lovell, C. W. and Witsman, G. R., "Compactive Prestress in Shales", Bulletin, Association of Engineering Geologists, Vol. 18, No. 3, August 1981, pp. 297-308.

9.4 Witsman, G. E., "The Effects of Compacted Prestress on Compacted Shale Compressibility", MSCE Thesis and Joint Highway Research Project No. 79-16, Purdue University, West Lafayette, Indiana, September 1979, 181 pp.

9.5 Holtz, R. D. and Kovacs, W. D., "An Introduction to Geotechnical Engineering", Prentice-Hall, Inc., Englewood Cliffs, N.J., 1981, p. 296.

10. Example

10.1 Tables A.6 through A.13 and Figures A.5 through A.11 show the data collection and results of this test for a typical Indiana shale

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHALE Osgood SHEET NO. 1
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 9/81
EXPECTED WATER CONTENT 10.0% DESIRED DRY DENSITY 2.05 g/cu. cm

DETERMINATION OF FOOT PRESSURE

FOOT PRESSURE

	1	2	3	4	5
Gage Reading	4.0	5.0	6.0	6.5	
Foot Pressure, kPa	620.5	668.8	779.1	855.0	

DENSITY

Wt. of Compacted Shale and Mold, g	4075.3	4151.0	4197.2	4213.1	
Wt. of Mold, g	2023.0	2023.0	2023.0	2023.0	
Wt. of Compacted Shale, g	2052.3	2128.0	2174.2	2190.1	
Volume of Mold, cu. cm	944.0	944.0	944.0	944.0	
Wet Density of Compacted Shale, g/cu. cm	2.17	2.25	2.30	2.32	
Dry Density of Compacted Shale, g/cu. cm	1.98	2.05	2.09	2.11	

WATER CONTENT CONFORMATION

Container No.	1	2	3	4	
Wt. of Container and Wet Shale, g	2235.5	2362.5	2085.1	2163.4	
Wt. of Container and Dry Shale, g	2050.5	2160.7	1907.2	1980.0	
Wt. of Water, g	185.0	201.8	177.9	183.4	
Wt. of Container, g	162.8	201.7	163.0	164.0	
Wt. of Dry Shale, g	1887.7	1959.0	1744.2	1816.0	
Water Content, %	9.8	10.3	10.2	10.1	

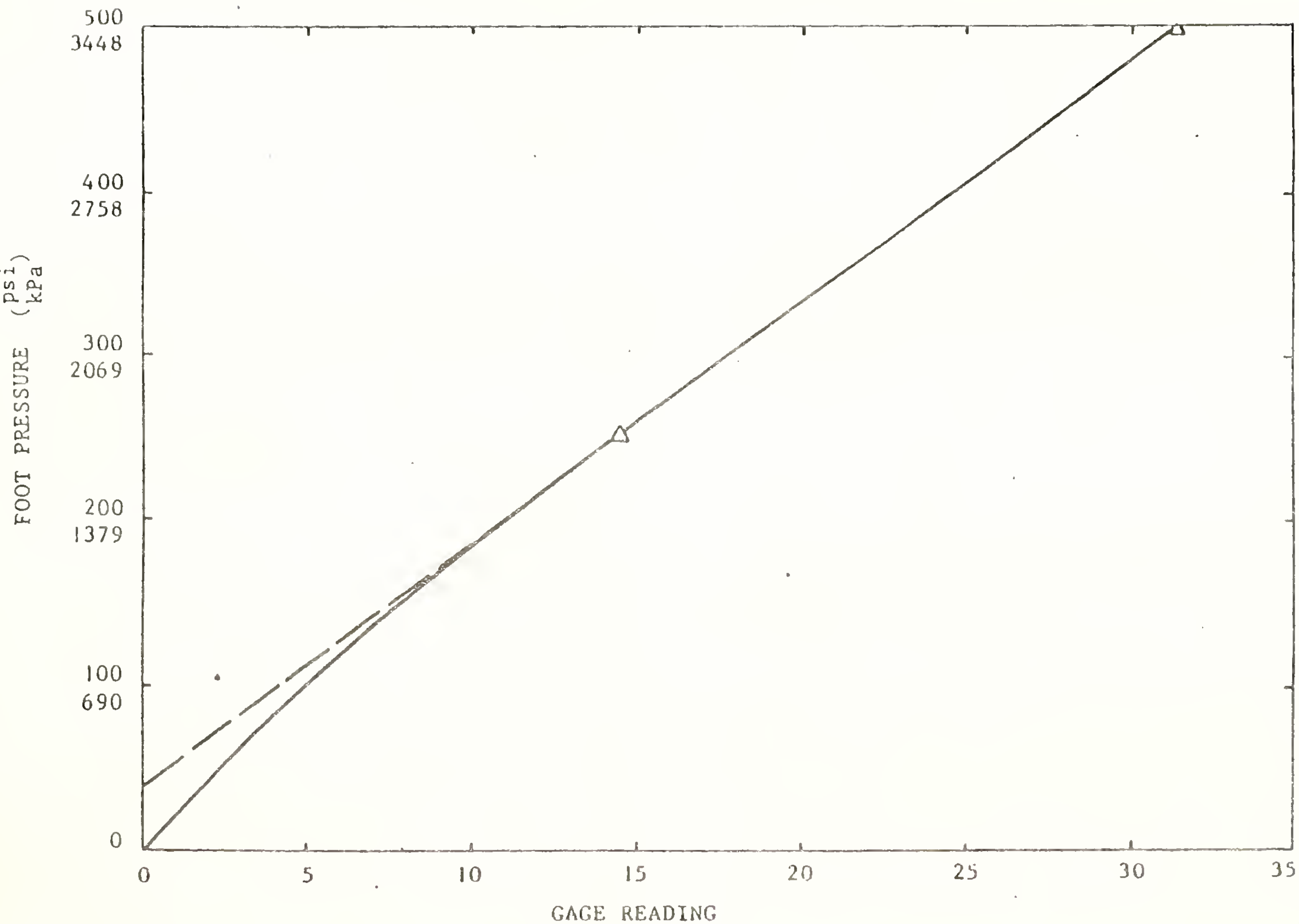


Figure A.5 Kneading Compactor Foot Pressure Calibration Curve

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHALE	Osgood	SHEET NO.	3
DESCRIPTION	Soft, gray shale which breaks into massive chunks				
LOCATION	Unknown	SAMPLE NO.	1	DEPTH	Unknown
SOURCE	Test Pit	TESTED BY	MWO	DATE	9/81
EXPECTED WATER CONTENT	10%	DESIRED DRY DENSITY	2.05 g/cu. cm		

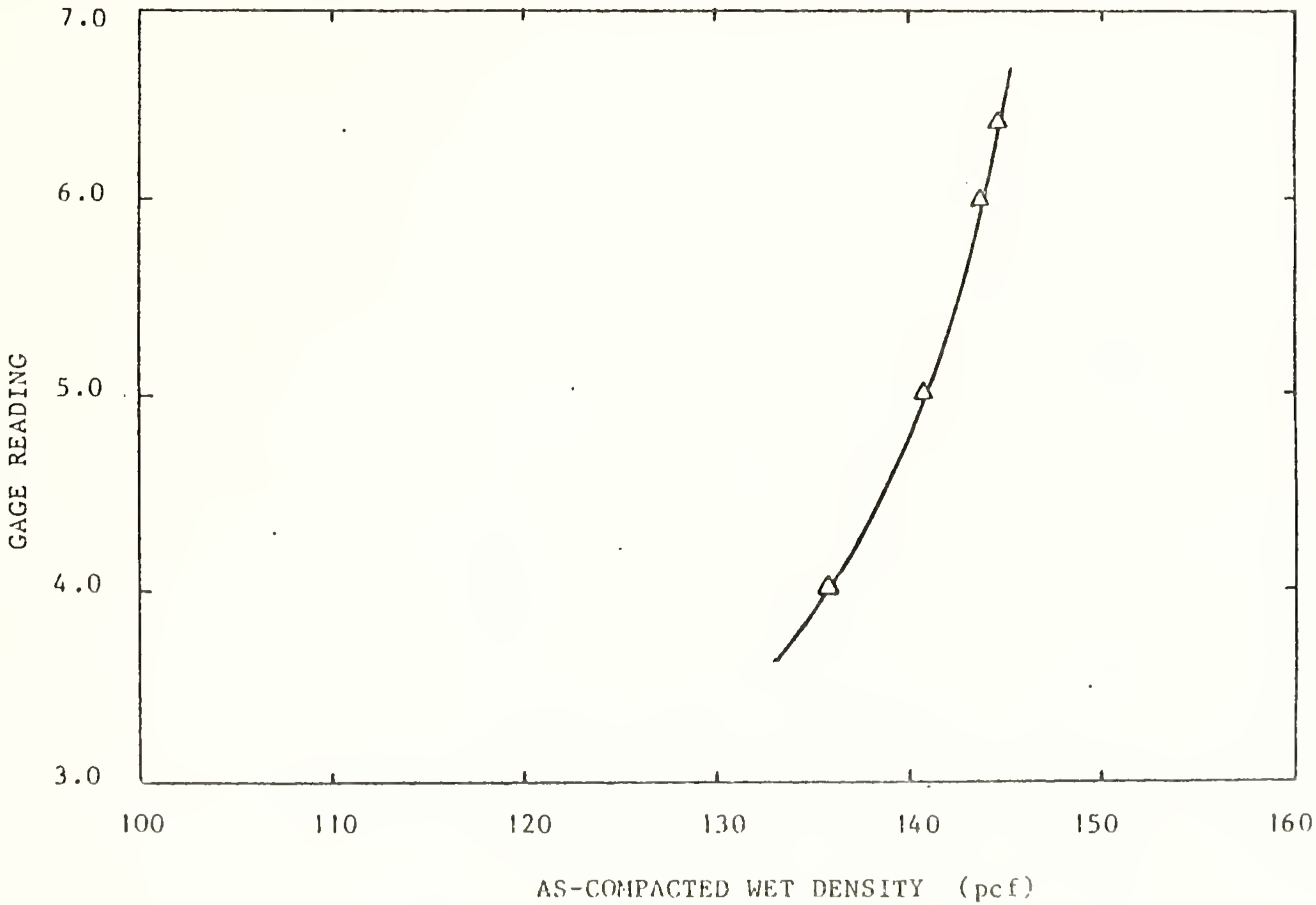


Figure A.6: Gage Reading vs. As-Compacted Wet Density Calibration

TABLE A.7: Typical Shale As-Compacted Initial Conditions for Compressibility Test

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHEET NO.	4
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

AS-COMPACTED INITIAL CONDITIONS

APPLIED KNEADING COMPACTOR FOOT PRESSURE	
Gage Reading	5
Foot Pressure, kPa	675
WATER CONTENT	
Container No.	1
Wt. of Container and Wet Shale, g	163.6
Wt. of Container and Dry Shale, g	150.9
Wt. of Water, g	12.7
Wt. of Container, g	24.5
Wt. of Dry Shale, g	126.4
Water Content, %	10.0
SAMPLE DIMENSIONS	
Diameter, cm	10.13
Height, cm	3.885
Weight, g	704.7
Volume, cu. cm	313.1
Compacted Wet Density, g/cu. cm	2.251
Compacted Dry Density, g/cu. cm	2.046
MISCELLANEOUS	
Specific Gravity of Solids	2.70
As-Compacted Void Ratio	0.320
As-Compacted Degree of Saturation, %	84.4

TABLE A.8: Typical Compressibility Data for One As-Compacted Loading Increment

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHEET NO. 5
SHALE Osgood DATE 9/81
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1
SOURCE Test Pit DEPTH Unknown

AS-COMPACTED LOADING

Increment 36.50 kPa to 60.84 kPa

Load Increment Ratio 0.667

Initial Height, cm 3.8545

Time (min)	Dial Gage Reading (cm)	Height Change (cm)	Height (cm)
0	0.0386	0	3.8545
0.1	0.0533	0.0147	3.8398
0.25	0.0549	0.0163	3.8382
0.5	0.0569	0.0183	3.8362
1.0	0.0582	0.0196	3.8349
2.0	0.0599	0.0213	3.8332
4.0	0.0620	0.0234	3.8311
8.0	0.0643	0.0257	3.8288
16.0	0.0668	0.0282	3.8263

FINAL CONDITIONS

Height, cm 3.8263
Void Ratio 0.2997
Degree of Saturation, % 90.1

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHALE	Osgood	SHEET NO.	6
DESCRIPTION	Soft, gray shale which breaks into massive chunks				
LOCATION	Unknown	SAMPLE NO.	1	DEPTH	Unknown
SOURCE	Test Pit	TESTED BY	MWO	DATE	9/81
INITIAL LOAD	36.50 kPa	FINAL LOAD	60.84 kPa	LOAD INCREMENT RATIO	0.667

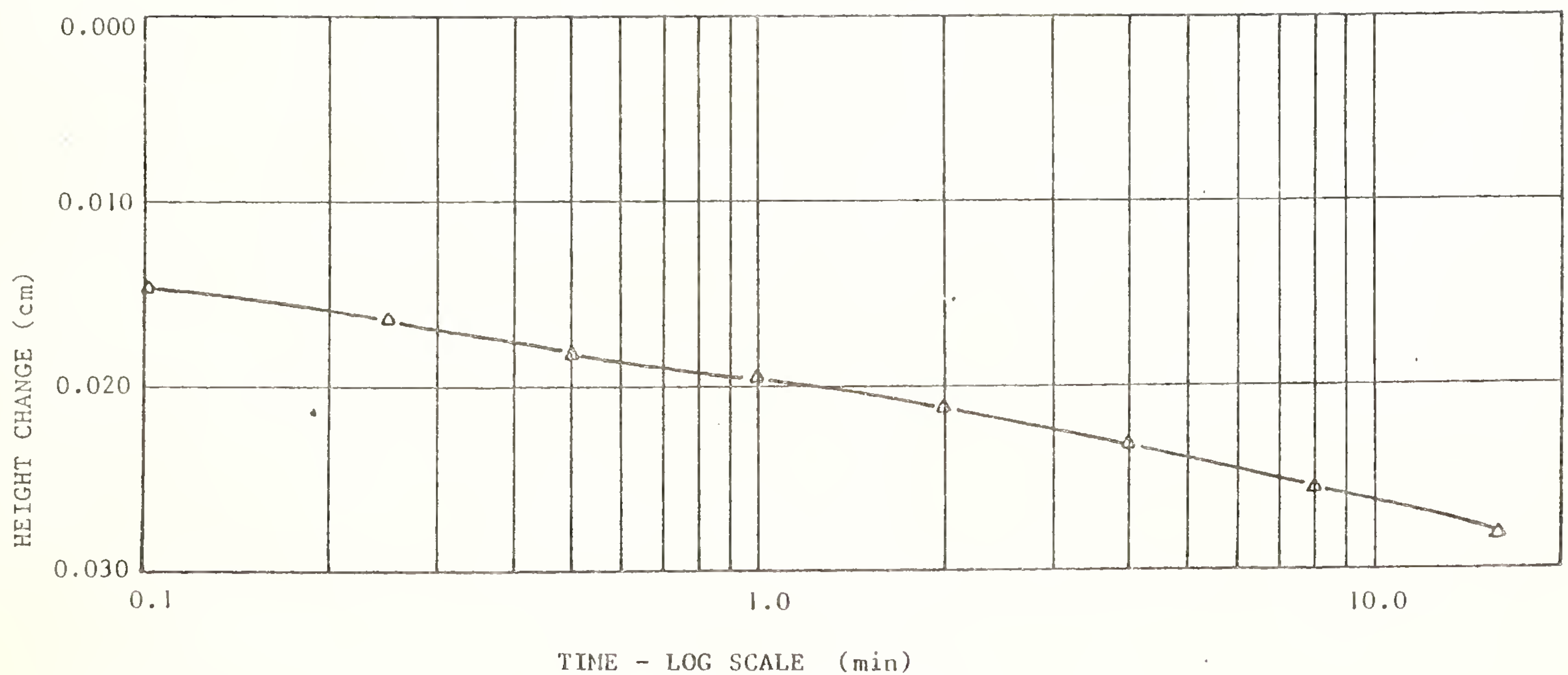


Figure A.7: Typical Compressibility Data for One As-Compacted Loading Increment

TABLE A.9: Typical Final Conditions and Summary of Design Parameters for As-Compacted Loading for Compressibility Test

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHEET NO.	7
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

AS-COMPACTED - FINAL CONDITION AND SUMMARY

FINAL CONDITIONS	
Load, kPa	146.01
Diameter, cm	10.13
Height, cm	3.760
Volume, cu. cm	303.06
Void Ratio	0.277
Degree of Saturation, %	97.38
SUMMARY	
Compression Index (Prestressed Curve)	0.0498
Compression Index (Beyond Prestress)	0.6433
Prestress Value, kPa	50.0
Kneading Compactor Foot Pressure, kPa	675.
Prestress Ratio, %	7.4

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHALE	Osgood	SHEET NO.	8
DESCRIPTION	Soft, gray shale which breaks into massive chunks				
LOCATION	Unknown	SAMPLE NO.	1	DEPTH	Unknown
SOURCE	Test Pit	TESTED BY	MWO	DATE	9/81

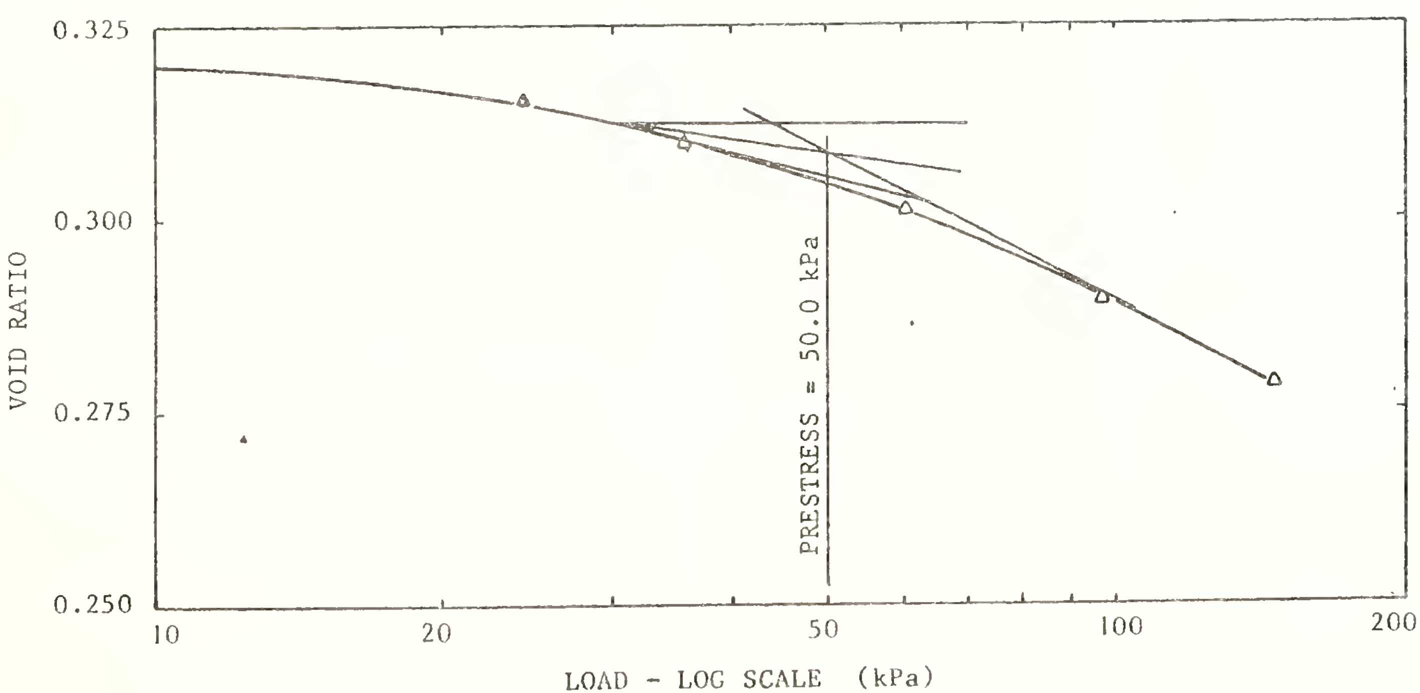


Figure A.8: Determination of Prestress Using Casagrande Construction

TABLE A.10: Typical Data Collection During Saturation Process for Compressibility Test

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHEET NO.	9
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

SATURATION PROCESS

COMPACTION VARIABLES	
Kneading Compactor Foot Pressure, kPa	675.
As-Compacted Water Content, %	10.0
As-Compacted Void Ratio	0.320
INITIAL CONDITIONS	
Load, kPa	146.01
Diameter, cm	10.13
Height, cm	3.7600
Volume, cu. cm	303.06
Void Ratio	0.277
Degree of Saturation, %	97.38
SETTLEMENT (OR HEAVE)	
Initial Dial Gage Reading, cm	0.1250
Final Dial Gage Reading, cm	0.1681
Change in Height, cm	0.0431
Final Height, cm	3.717
Final Volume, cu. cm	299.58
Percent Settlement (Heave negative)	1.15
B PARAMETER CHECK	
Initial Pore Pressure Transducer Reading, volts	0.0426
Initial Pore Pressure, kg/sq. cm	6.00
Load Increase, kg/sq. cm	0.10
Final Pore Pressure Transducer Reading, volts	0.432
Final Pore Pressure, kg/sq. cm	6.10
Pore Pressure Increase, kg/sq. cm	0.10
B Parameter	1.00
FINAL CONDITIONS	
Height, cm	3.760
Volume, cu. cm	303.06
Void Ratio	0.277
Degree of Saturation, %	100.0
Water Content	10.26

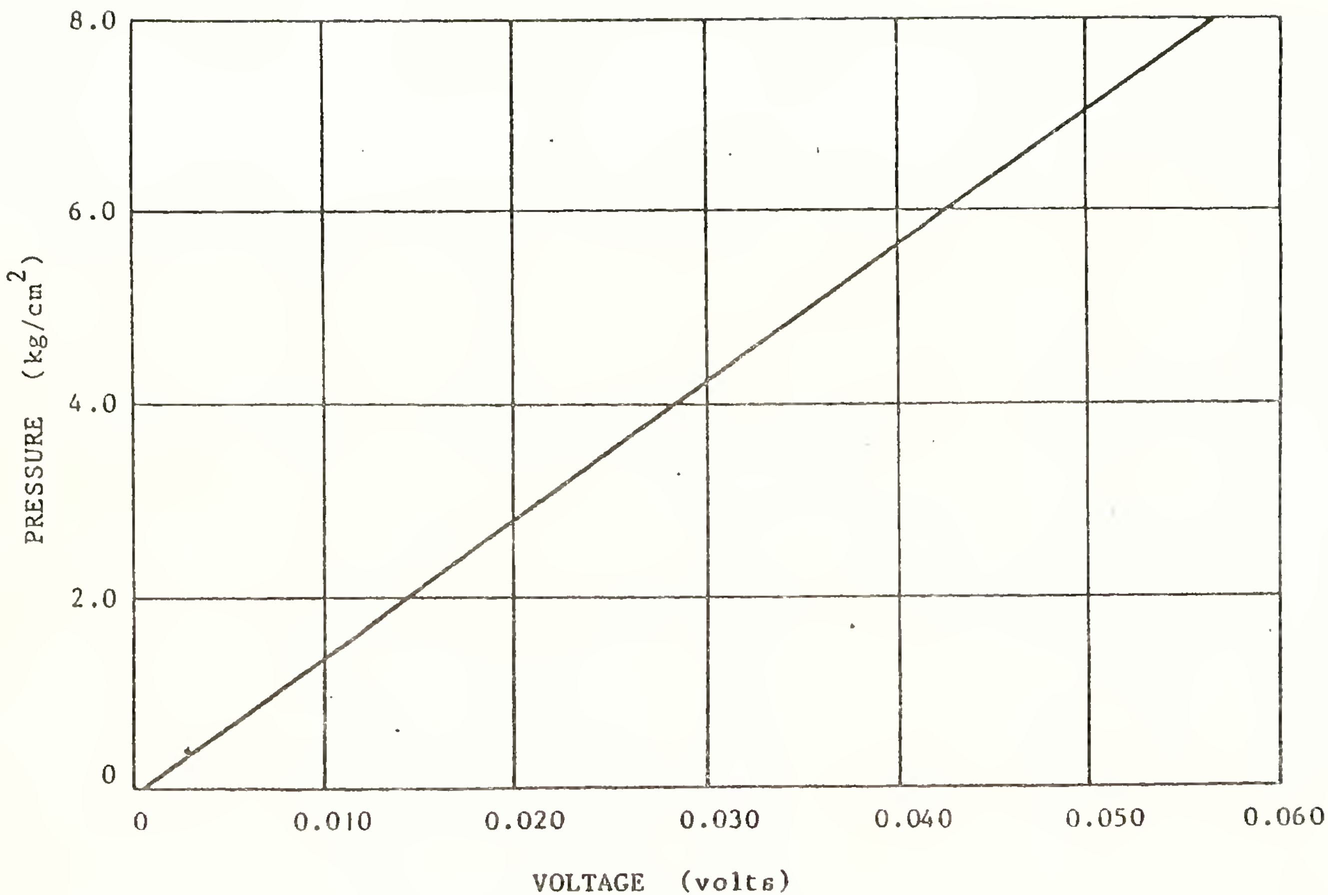


Figure A.9: Pore Pressure Transducer Calibration Curve

TABLE A.11: Typical Data Collection for One Saturated Loading Increment for Compressibility Test

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHEET NO. 11
SHALE Osgood DATE 9/81
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1
SOURCE Test Pit DEPTH Unknown

SATURATED LOADING

Increment 97.3 kPa to 194.5 kPa

Load Increment Ratio 1.0

Initial Height, in. 3.7199

Time (min)	Dial Gage Reading (cm)	Height (cm)	Pore Pressure Transducer (Volts)	Pore Pressure (kg/sq. cm)
0.0	0.1628	3.7199	-	-
0.1	0.1686	3.7140	-	-
0.25	0.1692	3.7135	-	-
0.5	0.1699	3.7128	-	-
1.0	0.1707	3.7120	-	-
2.0	0.1717	3.7110	-	-
4.0	0.1725	3.7102	-	-
8.0	0.1735	3.7092	-	-
16.0	0.1735	3.7092	-	-

FINAL CONDITIONS

Height, in.	3.7092
Volume, cu. cm	298.96
Void Ratio	0.2601
Coefficient of Consolidation, c_v	-

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHALE Osgood SHEET NO. 12
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 9/81
INITIAL LOAD 97.3 kPa FINAL LOAD 194.5 kPa LOAD INCREMENT RATIO 1.00

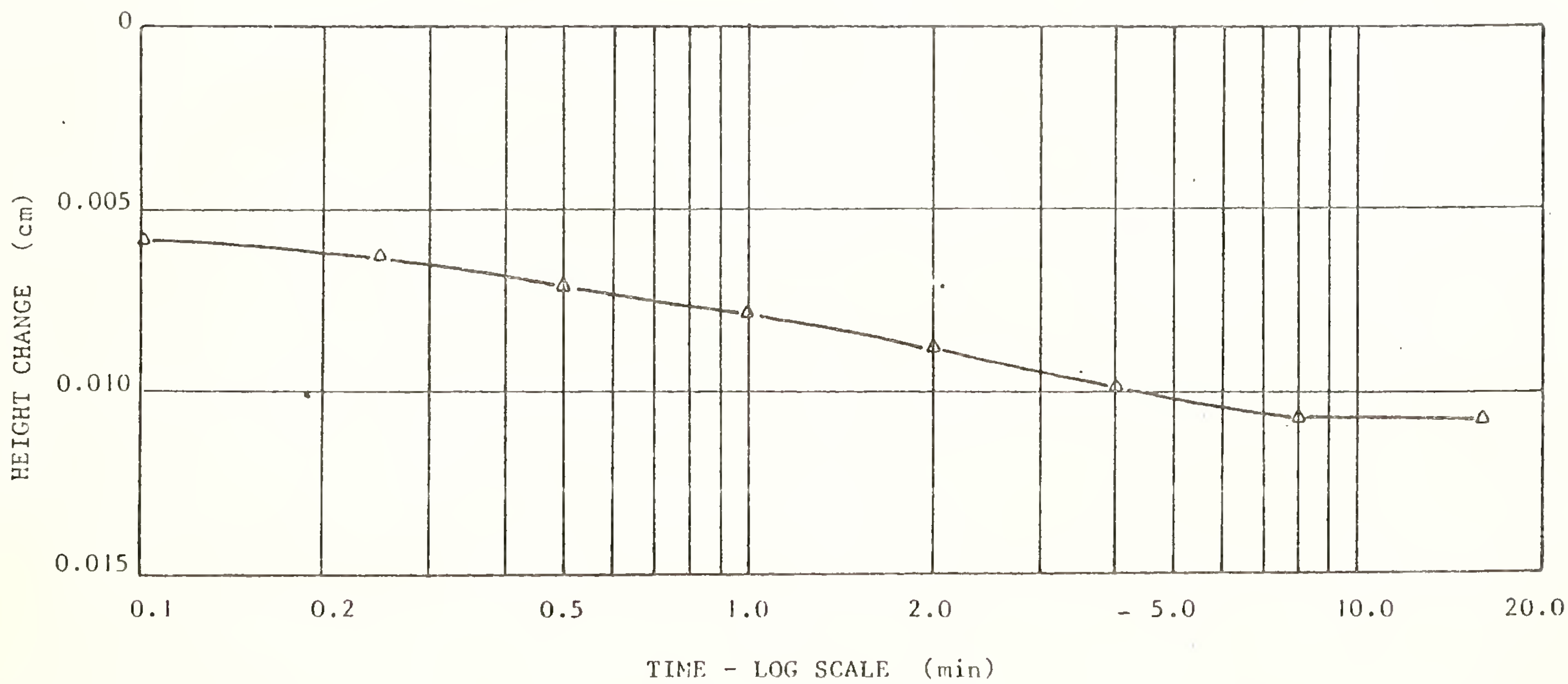


Figure A.10: Typical Data Collection for One Saturated Loading Increment for Compressibility TEst

TABLE A.12: Typical Final Conditioning and Summary of Design Parameters for Saturated Loading for Compressibility Test

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT	Example Test	SHEET NO.	13
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

SATURATED - FINAL CONDITIONS AND SUMMARY

FINAL CONDITIONS	
Load, kPa	1168.1
Diameter, cm	10.13
Height, cm	3.1012
Volume, cu. cm	249.95
Void Ratio	0.0535
WATER CONTNET	
Container No.	-
Wt. of Container and Wet Shale, g	-
Wt. of Container and Dry Shale, g	-
Wt. of Water, g	-
Wt. of Container, g	-
Wt. of Dry Shale, g	-
Water Content	-
SUMMARY	
Degree of Saturation, %	100.0
Compression Index (Prestressed Curve)	0.0551
Compression Index (Beyond Prestress)	0.9059

TABLE A.13: Typical Summary of Loading for Compressibility Test

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHEET NO. 14
SHALE Osgood DATE 9/81
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1
SOURCE Test Pit DEPTH Unknown

SUMMARY OF LOADING

CONDITION	LOAD KPa	VOID RATIO
As-Compacted Loading	0.00	0.3198
	24.33	0.3152
	36.50	0.3094
	60.84	0.3014
	97.34	0.2893
	146.01	0.2773
Saturation	146.01	0.2627
Saturated Unloading	48.67	0.2640
	24.33	0.2647
Saturated Reloading	48.67	0.2645
	97.34	0.2637
	194.68	0.2601
	292.02	0.2137
	1168.08	0.0535

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHALE Osgood SHEET NO: 15
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 9/81

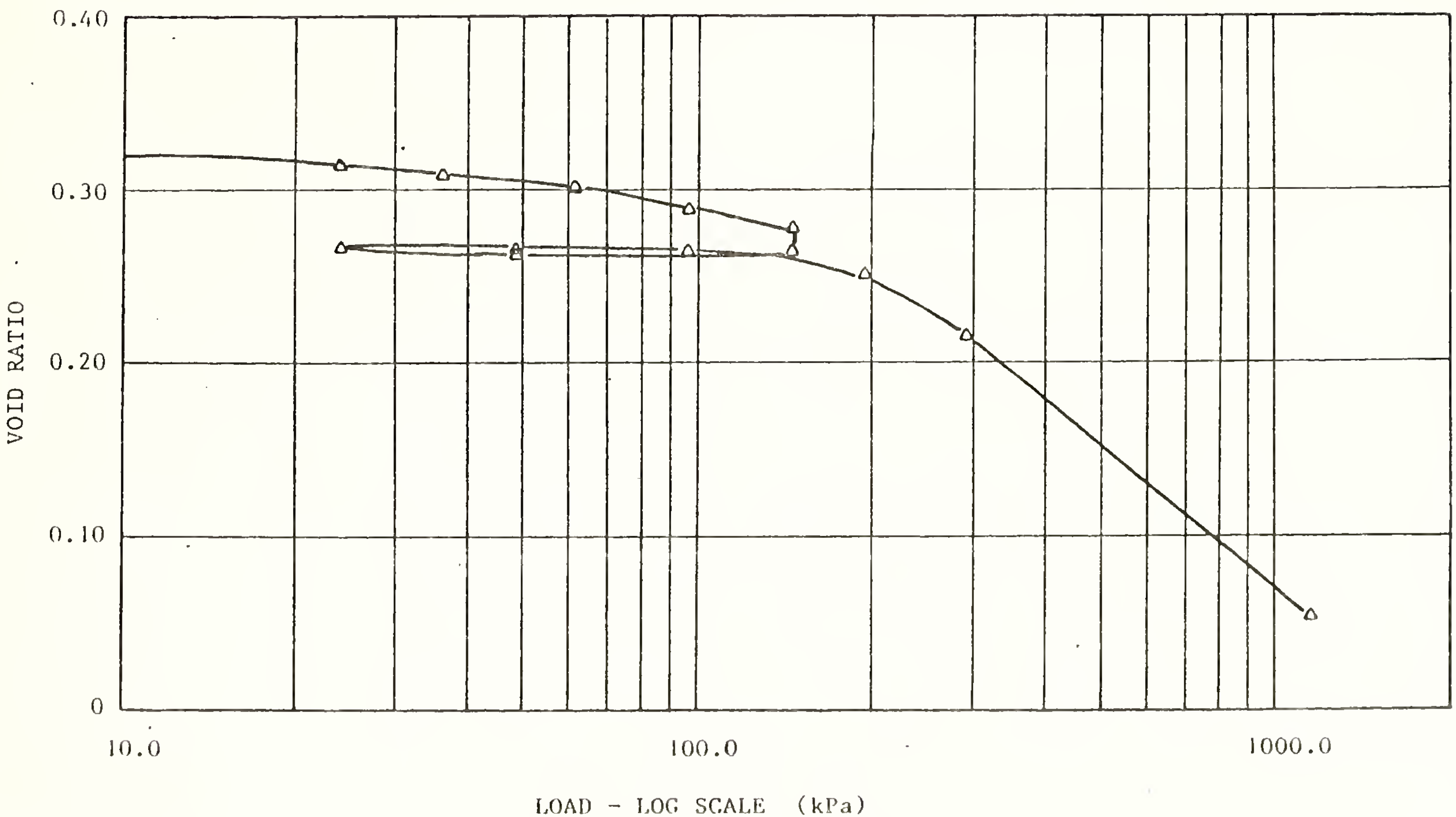


Figure A.11: Typical Void Ratio vs. Load Curve for Compressibility Test

Isotropically Consolidated Undrained Triaxial Compression ($\overline{\text{CIU}}$) Test

1. Scope

1.1 This method covers the determination of the consolidated undrained shear strength and effective stress strength parameters of a compacted and saturated shale.

2. Definition

2.1 Undrained shear strength, the shear stress on the failure plane at failure, as defined by the point of tangency of a straight line Mohr-Coulomb failure envelope.

2.2 Effective stress strength parameters, the intercept (c') and the inclination (ϕ') of a straight line Mohr-Coulomb failure envelope based upon effective stresses.

2.3 Skempton B parameter, the ratio of change in undrained pore pressure response to the change in level of all around total stress; equal to or less than unity.

2.4 Skempton A_f parameter, the ratio of undrained pore pressure at failure to the change in axial stress required to cause failure. (This definition is a simplified one for the saturated sample and constant cell pressure conditions of this test.)

Apparatus

3.1 Mold - The mold shall be made of rigid metal, cylindrical in shape, with an inside diameter of 4.00 in. (101.6 mm) and with a height of 9.0 in. (230 mm). The mold shall be provided with a removable extension collar made of rigid metal with an inside diameter of 4.00 in. (101.6 mm) and a height of at least 2.0 in. (50.8 mm). A base shall be provided which allows the mold to be mounted onto the kneading compactor.

3.2 Kneading compactor, a California kneading compactor or a kneading compactor developing a time-pressure trace curve similar to that of the California kneading compactor.

3.3 Compactor foot, a ram having a foot shaped as shown in ASTM D1561-76 or AASHTO T247.

3.4 Sieves, a series of sieves of square-mesh, woven-wire cloth, conforming to the requirements of ASTM E11-70 or AASHTO M92. The sieves required are as follows:

3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)
No. 16	(1.18 mm)
Pan	

3.5 Drying oven, thermostatically controlled, capable of maintaining a temperature of 230 ± 9 F (110 ± 5 C).

3.6 Testing and measuring apparatus, shall conform to the standards given for apparatus as stated in ASTM D2850-70 (although this is an unconsolidated undrained test, there are similarities in the apparatus) or AASHTO T234.

3.7 Additional miscellaneous apparatus, straight edge, reciprocating jaw crusher, hammer, plastic bags, mixing spoon, spray bottle, extrusion device, rubber membrane cover, filter paper.

4. Sample

4.1 The sample shall consist of representative rock specimens small enough to be broken with a hand hammer, yet large enough to yield 3/4 in. (19.0 mm) aggregates when crushed.

4.2 The sample shall be transported and stored in such a manner as to retain the natural water content.

5. Preparation of Test Specimen

5.1 The sample shall be broken with a hammer into fragments small enough to be crushed by a reciprocating jaw crusher adjusted to yield aggregates of at least 3/4 in. (19.0 mm) in size.

5.2 The product of the jaw crusher shall be sorted by passing the material through a nest of sieves composed of the following:

3/4 in.	(19.0 mm)
3/8 in.	(9.5 mm)
No. 4	(4.75 mm)
No. 8	(2.36 mm)
No. 16	(1.18 mm)
Pan	

5.3 The material needed to construct each test specimen shall weigh 10.0 lb (22.05 kg) and be prepared by combining the material retained on each sieve in the following gradation:

$$P = 100\ (d/D)$$

where

P = percentage, by weight, finer than size d,

d = sieve size, and

D = maximum aggregate size.

The standard gradation shall have a maximum aggregate size of 3/4 in. (19.0 mm).

5.4 The material shall be wetted to the optimum moisture content as determined by the moisture-density relations test (or to another specified moisture content) by adding water using a spray bottle, sealing the material in a plastic bag, and allowing it to cure for 24 hours.

5.5 The material shall be compacted in the mold (with collar attached) in ten equal layers to give a total compacted depth not to exceed 9.5 in. (240 mm), each layer being compacted by the kneading compactor for one minute using 30 blows per minute. The foot pressure shall produce a density equal to the density at the optimum moisture content when compacted by impact compaction (or other selected moisture-density combination) as described in the moisture-density relations test. The foot pressure shall be determined by making a plot of foot pressure versus density consisting of at least four points which straddle the desired testing density.

5.6 The collar shall be removed from the mold and a straight edge used to trim the specimen flush with the top of the mold. The trimmings shall be used for a moisture content sample. A jack shall be used to extrude the specimen from the mold. The specimen shall immediately be weighed, measured, and placed in the triaxial cell with a rubber membrane cover and filter paper between the specimen and the porous stones.

6. Procedure

6.1 The specimen shall be compressed using cell pressure applied in increments not to exceed 7 psi (50 kPa) up to a predetermined load corresponding to a specific overburden value. At least 60 minutes shall be allowed between each increment. The change in specimen volume shall be recorded for each increment.

6.2 The specimen shall be saturated by allowing de-aired water to flow from the bottom platen to the top platen under a small head. A small vacuum may be applied to aid in de-airing the specimen. The net volume of water which is retained in the specimen shall be measured, so volume change in the sample which occurs during saturation can be calculated. Back pressure shall be applied in increments not to exceed 7.0 psi (50 kPa) up to the pressure needed for saturation. At least 60 minutes shall be allowed between each increment of pressure. A B parameter check shall be conducted during the final back pressuring increments as an indication of the degree of saturation. The procedure is as follows:

6.2.1 Close the top and bottom platens,

6.2.2 Increase the cell pressure by some increment, 6.2.3 After allowing the pressure within the specimen to equilize, record the increase in the pore water pressure.

6.2.4 Calculate the B parameter as the ratio of the change in the pore water pressure to the change in the confinement pressure.

6.2.5 Increase the back pressure by the same increment and open the top and bottom platens.

6.3 The specimen shall be sheared undrained as described in ASTM D2850-70 or AASHTO T234 at an appropriate rate of strain.

Calculations

7.1 A moisture content shall be calculated from samples taken before and after the test. The moisture content shall be calculated as follows:

$$w = \left[(A - B)/(B - C)\right] 100$$

where

w = percentage of moisture in specimen,

A = weight of container and wet shale,

B = weight of container and dry shale, and

C = weight of container.

7.2 The change in volume which occurs during saturation shall be calculated as follows:

$$\text{Vol. change} = V_I - V_F$$

where

V_I = the initial volume calculated from the measurements taken before saturation, and

V_F = the final volume calculated as follows:

$$V_F = \frac{V_V}{1 - (\rho_d / G_s \ \rho_w)}$$

where V_V = volume of voids in the saturated specimen, If the specimen is assumed to be completely saturated this may be taken as the volume of water retained during percolation, plus the volume of water present during compaction.

ρ_d = the dry density of the specimen,

G_s = the specific gravity of solids,

ρ_w = the density of water.

7.3 All calculations shall be made as specified by ASTM D2850-70 or AASHTO T234.

8. Report

8.1 The report shall include all items specified by ASTM D2850-70 or AASHTO T234.

8.2 In addition, the report shall include the percent volume change which occurred during saturation.

8.3 The report shall also include the results from similar tests on the same material at different confining pressures in order to determine the effective stress strength parameters, c', ϕ'.

8.4 The report shall include plots of: $(\sigma_1 - \sigma_3)$ vs. ε_a %; Δu vs. ε_a %; A vs. ε_a %; p' vs. q; and p vs. q.

9. References

9.1 Abeyesekera, R. A., "Stress-Deformation and Strength Characteristics of a Compacted Shale", Ph.D. Thesis and Joint Highway Research Project No. 77-24, Purdue University, West Lafayette, Indiana, December 1977, 420 pp.

9.2 Abeyesekera, R. A., Lovell, C. W., and Wood, L. E., "Stress-Deformation and Strength Characteristics of a Compacted Shale", Papers of the Conference on Clay Fills, Institution of Civil Engineers, London, England, November 1978, pp. 1-14.

9.3 Abeyesekera, R. A., Lovell, C. W. and Wood, L. E., "Strength Testing of Compacted Shale", ASTM Geotechnical Testing Journal, Vol. 2, No. 1, March 1979, pp. 11-19.

10. Example

10.1 Tables A.6 and A.14 through A.19 and Figures A.5, A.6, A.9 and A.12 through A.14 show the data collection and results of this test for a typical Indiana shale.

TABLE A.6: Typical Calibration of Kneading Compactor Foot Pressure to Compacted Density Information

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT Example Test SHALE Osgood SHEET NO. 1
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 9/81
EXPECTED WATER CONTENT 10.0% DESIRED DRY DENSITY 2.05 g/cu. cm

DETERMINATION OF FOOT PRESSURE

FOOT PRESSURE

	1	2	3	4	5
Gage Reading	4.0	5.0	6.0	6.5	
Foot Pressure, kPa	620.5	668.8	779.1	855.0	

DENSITY

Wt. of Compacted Shale and Mold, g	4075.3	4151.0	4197.2	4213.1	
Wt. of Mold, g	2023.0	2023.0	2023.0	2023.0	
Wt. of Compacted Shale, g	2052.3	2128.0	2174.2	2190.1	
Volume of Mold, cu. cm	944.0	944.0	944.0	944.0	
Wet Density of Compacted Shale, g/cu. cm	2.17	2.25	2.30	2.32	
Dry Density of Compacted Shale, g/cu. cm	1.98	2.05	2.09	2.11	

WATER CONTENT CONFORMATION

Container No.	1	2	3	4	
Wt. of Container and Wet Shale, g	2235.5	2362.5	2085.1	2163.4	
Wt. of Container and Dry Shale, g	2050.5	2160.7	1907.2	1980.0	
Wt. of Water, g	185.0	201.8	177.9	183.4	
Wt. of Container, g	162.8	201.7	163.0	164.0	
Wt. of Dry Shale, g	1887.7	1959.0	1744.2	1816.0	
Water Content, %	9.8	10.3	10.2	10.1	

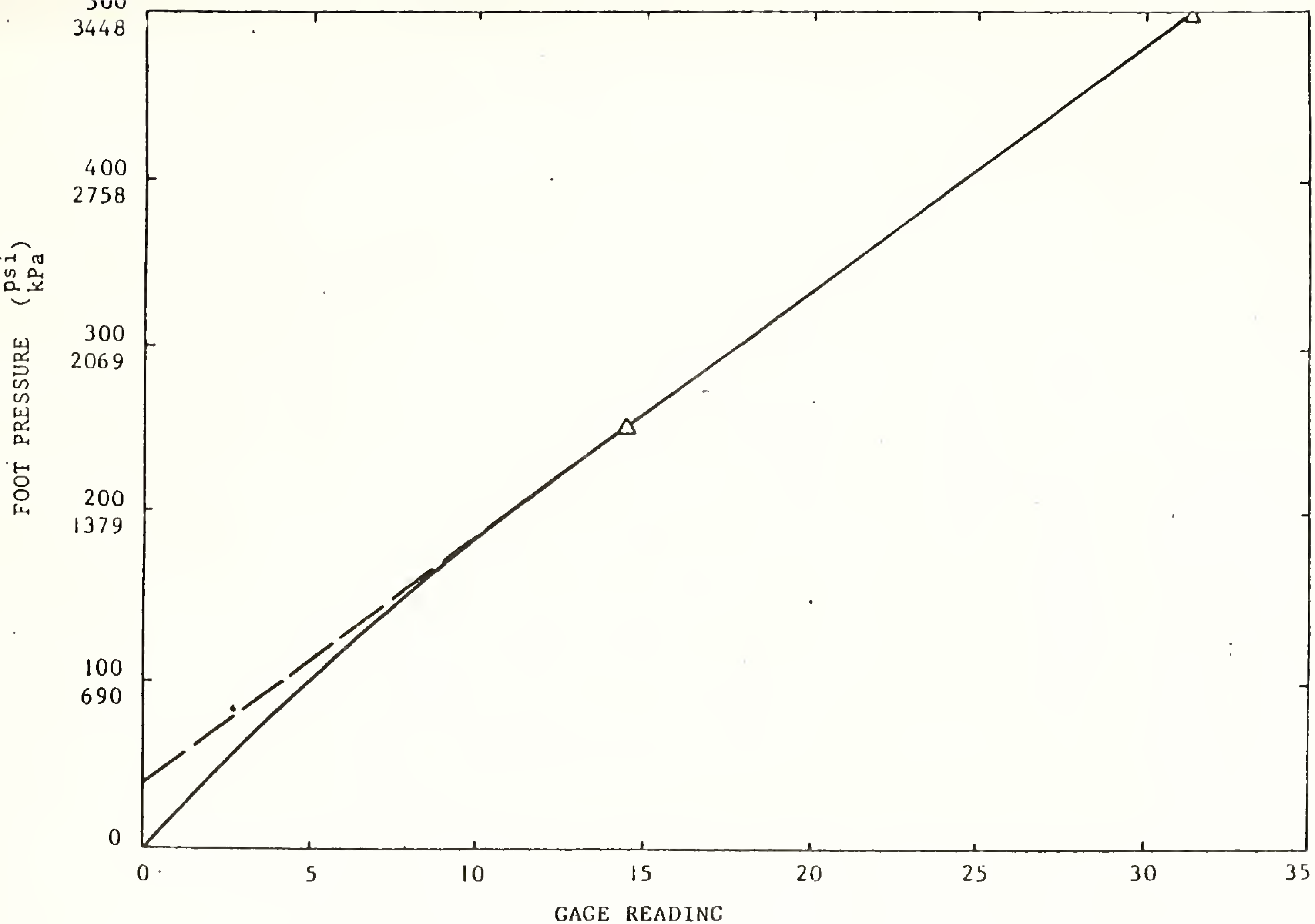


Figure A.5: Kneading Compactor Foot Pressure Calibration Curve

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT Example Test SHALE Osgood SHEET NO. 3
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 9/81
EXPECTED WATER CONTENT 10% DESIRED DRY DENSITY 2.05 g/cu. cm

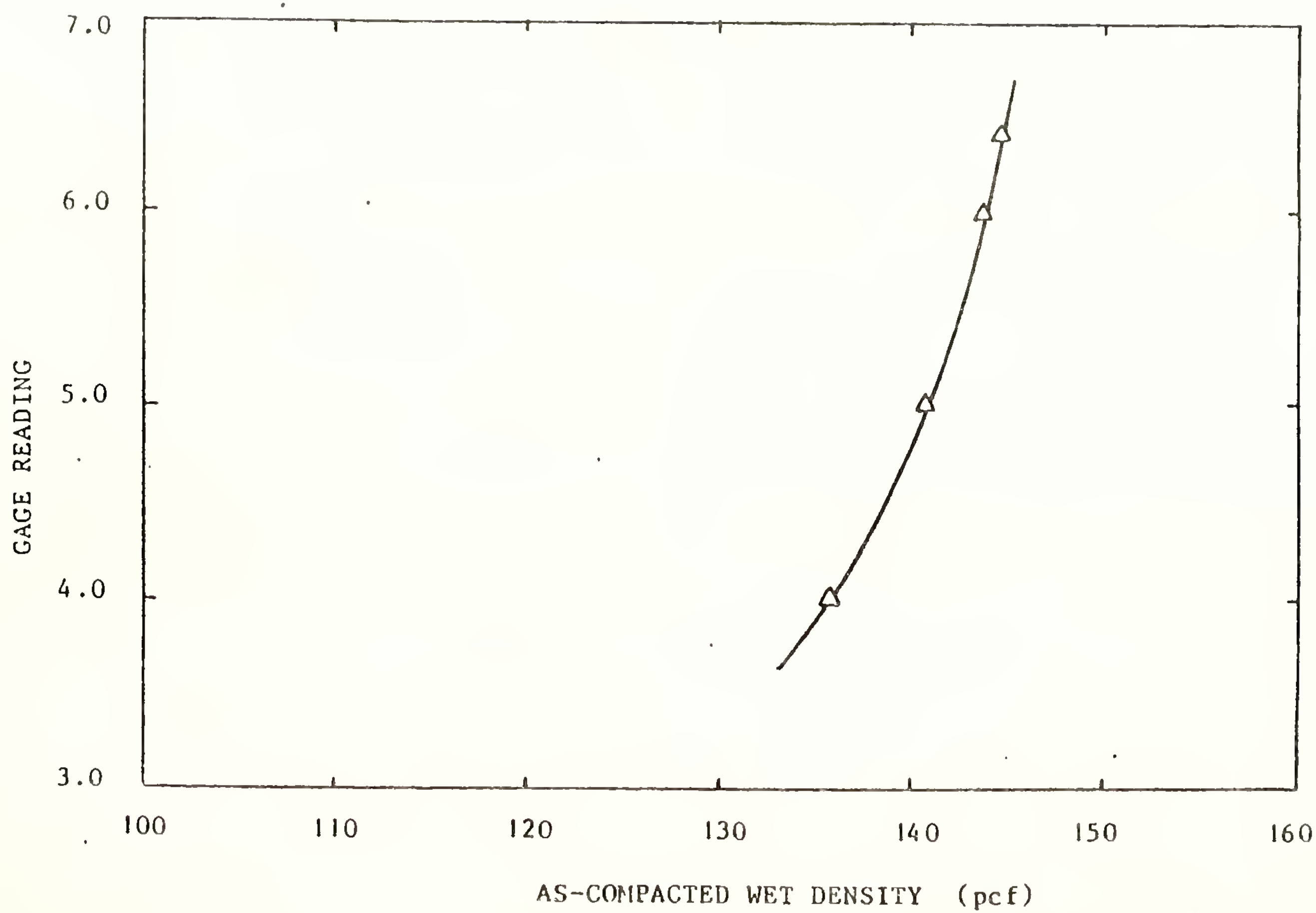


Figure A.6: Gage Reading vs. As-Compacted Wet Density Calibration

TABLE A.14: Typical As-Compacted Conditions for $\overline{\text{CIU}}$ Triaxial Test

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT	Example Test	SHEET NO.	4
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks ino massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

AS-COMPACTED INITIAL CONDITIONS

APPLIED KNEADING COMPACTOR FOOT PRESSURE	
Gage Reading	5
Foot Pressure, kPa	675
WATER CONTENT	
Container No.	1
Wt. of Container and Wet Shale, g	163.6
Wt. of Container and Dry Shale, g	150.9
Wt. of Water, g	12.7
Wt. of Container, g	24.5
Wt. of Dry Shale, g	126.4
Water Content, %	10.0
SAMPLE DIMENSIONS	
Diameter, cm	10.24
Height, cm	19.56
Weight, g	3672.8
Volume, cu. cm	1610.9
Compacted Wet Density, g/cu. cm	2.28
Compacted Dry Density, g/cu. cm	2.07
MISCELLANEOUS	
Specific Gravity of Solids	2.70
As-Compacted Void Ratio	0.3043
As-Compacted Degree of Saturation, %	88.71

TABLE A.15: Typical Consolidation Data for $\overline{\text{CIU}}$ Triaxial Test

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIL TEST

PROJECT	Example Test	SHEET NO.	5
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

CONSOLIDATION

INITIAL CONDITIONS	
Height, cm	19.56
Area, sq. cm	82.555
Volume, cu. cm	1610.9
Void Ratio	0.3043
Degree of Saturation, %	88.71
VOLUME CHANGE	
Consolidation Pressure, kPa	68.95
Initial Cell Burette Reading, cu. cm	-
Final Cell Burette Reading, cu. cm	-
Volume Change, cu. cm	-
Percent Volume Change, %	-
FINAL CONDITIONS	
Height, cm	19.56
Volume, cu. cm	1610.9
Area, sq. cm	82.355
Void Ratio	0.3043
Degree of Saturation, %	88.71

TABLE A.16: Typical Saturation Data for $\overline{CIU}$ Triaxial Test

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT	Example Test	SHEET NO.	6
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

SATURATION

COMPACTION VARIABLES	
Kneading Compactor Foot Pressure, kPa	675.
As-Compacted Water Content, %	10.0
As-Compacted Void Ratio	0.3043
INITIAL CONDITIONS	
Consolidation Pressure, kPa	68.95
Height, cm	19.56
Area, sq. cm	182.355
Volume, cu. cm	1610.9
Void Ratio	0.3043
Degree of Saturation, %	88.71
VOLUME CHANGE	
Initial Cell Burette Reading, cu. cm	-
Final Cell Burette Reading, cu. cm	-
Volume Change, cu. cm	-
Percent Volume Change, %	-
B PARAMETER CHECK	
Initial Pore Pressure Transducer Reading, volts	0.0426
Initial Pore Pressure, kg/sq. cm	6.0
Confining Pressure Increase, kg/sq. cm	0.10
Final Pore Pressure Transducer Reading, volts	0.432
Final Pore Pressure, kg/sq. cm	6.10
Pore Pressure Increase, kg/sq. cm	0.10
B Parameter	1.00
FINAL CONDITIONS	
Height, cm	19.56
Volume, cu. cm	1610.9
Area, sq. cm	82.355
Void Ratio	0.3043
Degree of Saturation, %	100.0

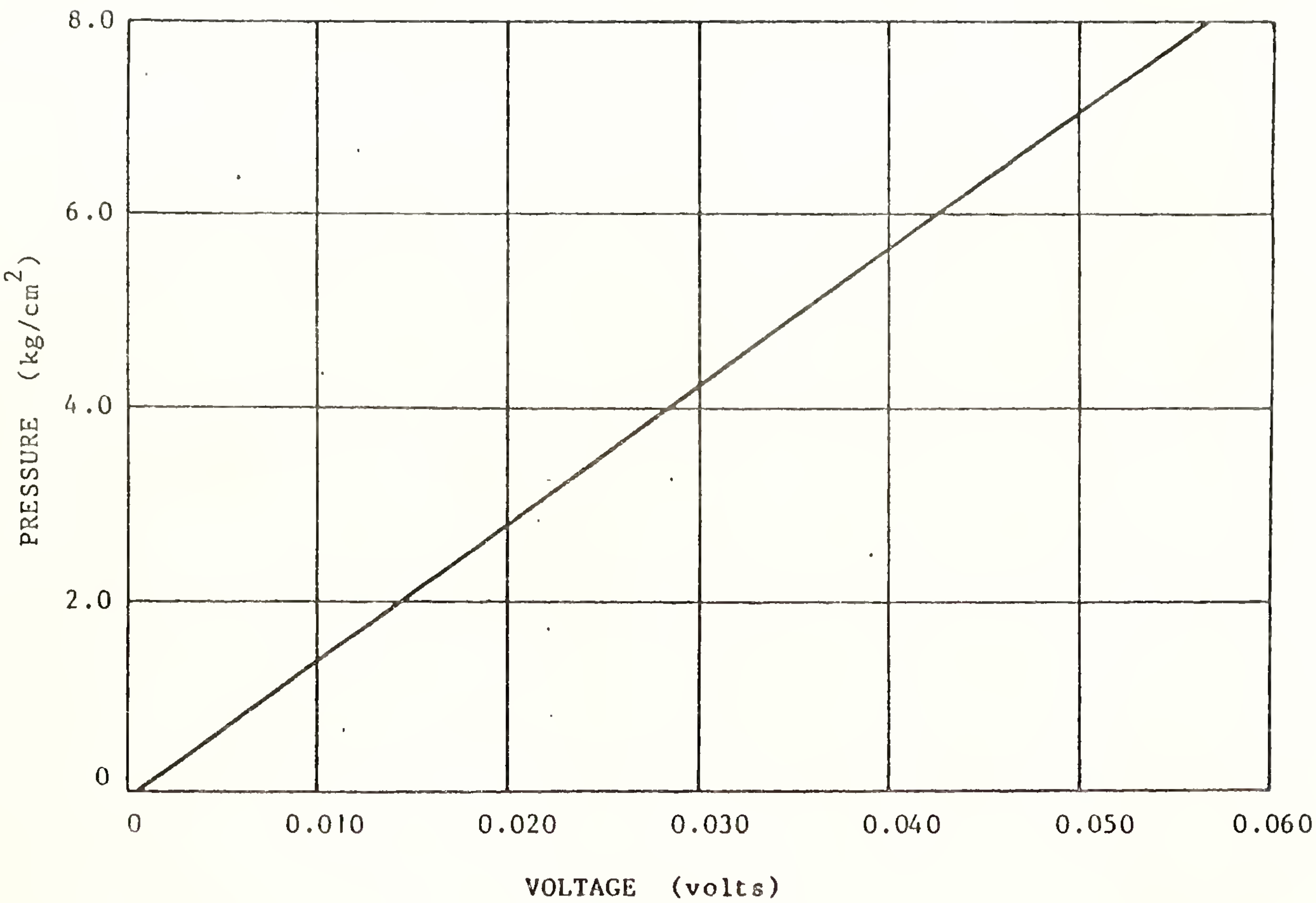


Figure A.9: Pore Pressure Transducer Calibration Curve

PROJECT Example Test SHALE Osgood SHEET NO. 8
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1 DEPTH Unknown
SOURCE Test Pit TESTED BY MWO DATE 9/81

Chamber Pressure		Proving Ring in.	Length Change in.	Strain %	Area in^2	Axial Load lb	Axial Pressure psi	Pore Pressure		σ'_3 psi	σ'_1 psi	$\sigma_1 - \sigma_3$ psi	$\frac{\sigma'_1}{\sigma'_3}$
Volts	psi							Volts	psi				
0.962	93.0	.0000	0.000	0.00	3.162	0	0.0	.0412	83.0	10.0	10.0	0.0	1.0
0.962	93.0	.0041	0.075	0.97	3.194	246	77.0	.0413	83.2	9.8	87.0	77.2	8.9
0.962	93.0	.0063	0.150	1.95	3.225	378	117.2	.0419	84.4	8.6	128.2	118.6	14.9
0.962	93.0	.0080	0.225	2.92	3.258	480	147.3	.0417	84.0	9.0	157.3	148.3	17.5
0.962	93.0	.0093	0.300	3.90	3.291	558	169.6	.0412	83.0	10.0	179.6	169.6	18.0
0.962	93.0	.0105	0.375	4.87	3.324	630	189.5	.0405	81.5	11.5	199.5	188.0	17.3
0.962	93.0	.0114	0.450	5.84	3.359	684	203.6	.0397	79.9	13.1	213.6	200.5	16.6
0.962	93.0	.0122	0.525	6.82	3.394	732	215.7	.0389	78.3	14.7	225.7	211.0	15.4
0.962	93.0	.0135	0.600	7.79	3.430	810	236.2	.0383	77.1	15.9	246.2	230.3	15.5
0.962	93.0	.0138	0.675	8.77	3.466	828	238.9	.0376	75.7	17.3	248.9	231.6	14.4
0.962	93.0	.0143	0.750	9.74	3.504	858	244.9	.0371	74.7	18.3	254.9	236.6	13.9
0.962	93.0	.0147	0.825	10.71	3.542	882	249.0	.0364	73.3	19.7	259.0	239.3	13.1
0.962	93.0	.0152	0.900	11.69	3.581	912	254.7	.0363	73.1	19.9	264.7	244.8	13.3
0.962	93.0	.0156	0.975	12.66	3.621	936	258.5	.0356	71.7	21.3	268.5	247.2	12.6
0.962	93.0	.0159	1.050	13.64	3.662	954	260.5	.0350	70.5	22.5	270.5	248.0	12.0
0.962	93.0	.0159	1.125	14.61	3.704	954	257.6	.0346	69.7	23.3	267.6	244.3	11.5
0.962	93.0	.0160	1.200	15.58	3.746	960	256.3	.0340	68.5	24.5	266.3	241.8	10.9

TABLE A.17: continued

ONE DIMENSIONAL COMPRESSIBILITY TEST

PROJECT Example Test SHEET NO. 9
SHALE Osgood DATE 9/81
DESCRIPTION Soft, gray shale which breaks into massive chunks
LOCATION Unknown SAMPLE NO. 1
SOURCE Test Pit DEPTH Unknown

Strain %	Δu psi	$\Delta\sigma_1$ psi	A	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\sigma_1' + \sigma_3'}{2}$
0.00	0.0	0.0	-	0.0	10.0
0.97	0.2	77.0	.0026	38.6	48.4
1.95	1.4	117.2	.0119	59.3	68.4
2.92	1.0	147.3	.0068	74.1	83.2
3.90	0.0	169.6	.0000	84.8	94.8
4.87	-1.5	189.5	-.0079	94.0	105.5
5.84	-3.1	203.6	-.0152	100.3	113.4
6.82	-4.7	215.7	-.0218	105.5	120.2
7.79	-5.9	236.2	-.0250	115.1	131.0
8.77	-7.3	238.9	-.0306	115.8	133.1
9.74	-8.3	244.9	-.0339	118.3	136.6
10.71	-9.7	249.0	-.0390	119.7	139.4
11.69	-9.9	254.7	-.0389	122.4	142.3
12.66	-11.3	258.5	-.0437	123.6	144.9
13.64	-12.5	260.5	-.0480	124.0	146.5
14.61	-13.3	257.6	-.0513	122.1	145.5
15.58	-14.5	256.3	-.0566	120.9	145.4

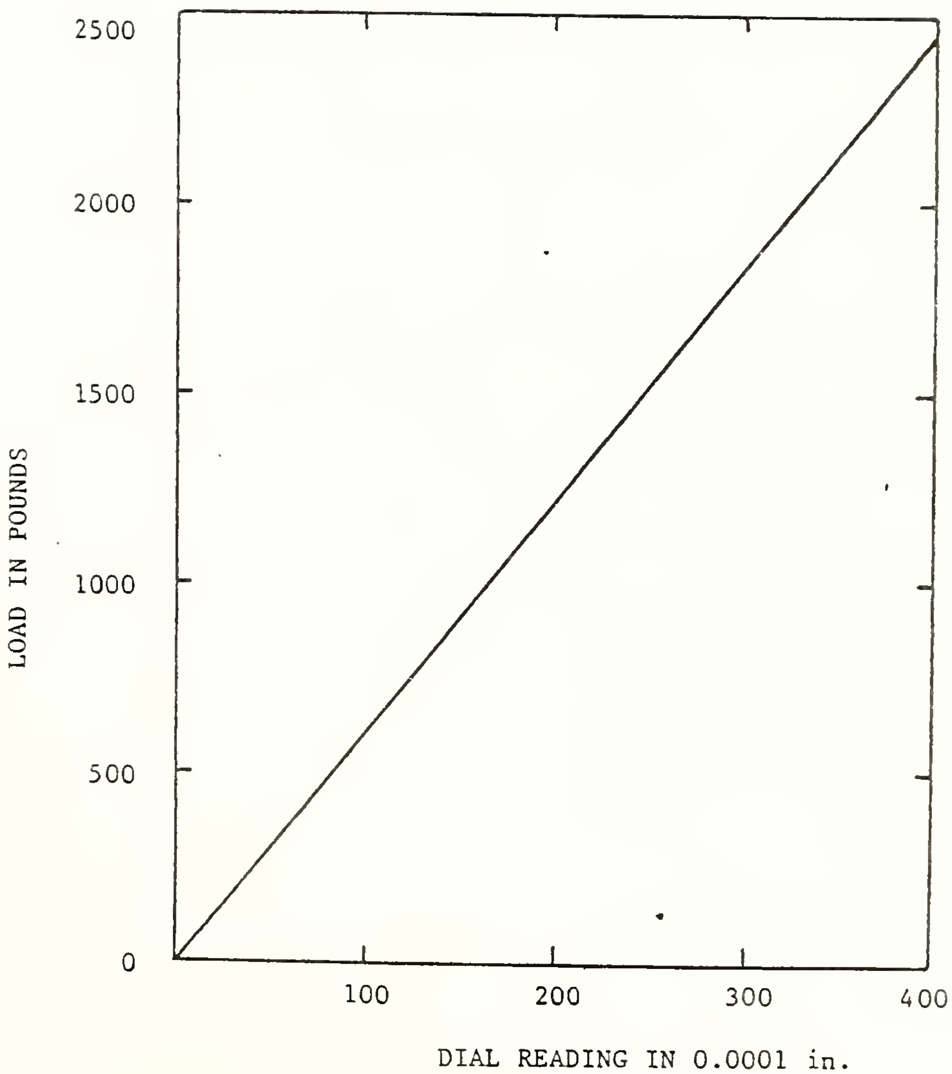


Figure A.12: Proving Ring Calibration Curve

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT	Example Test	SHEET NO.	11
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

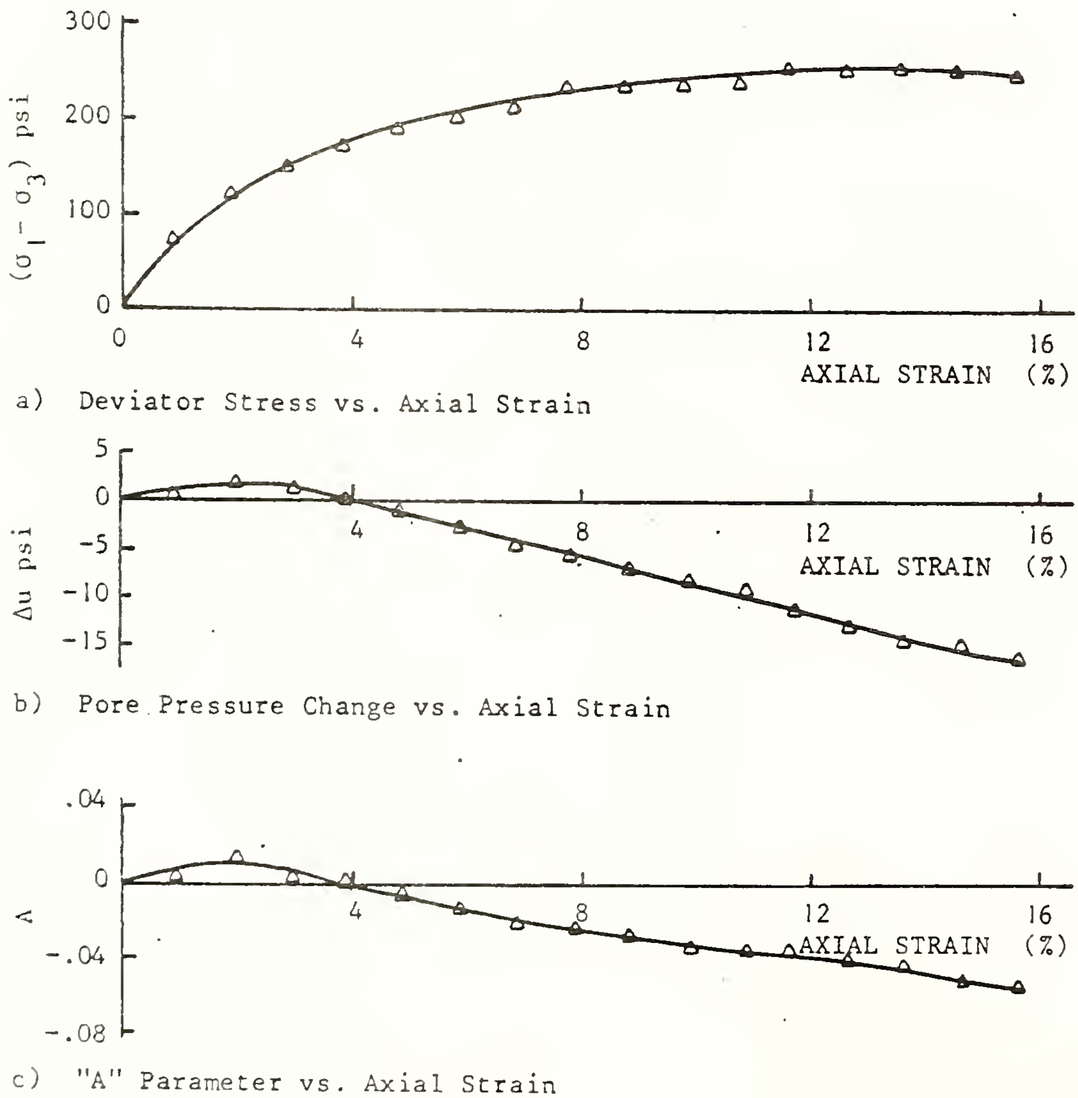
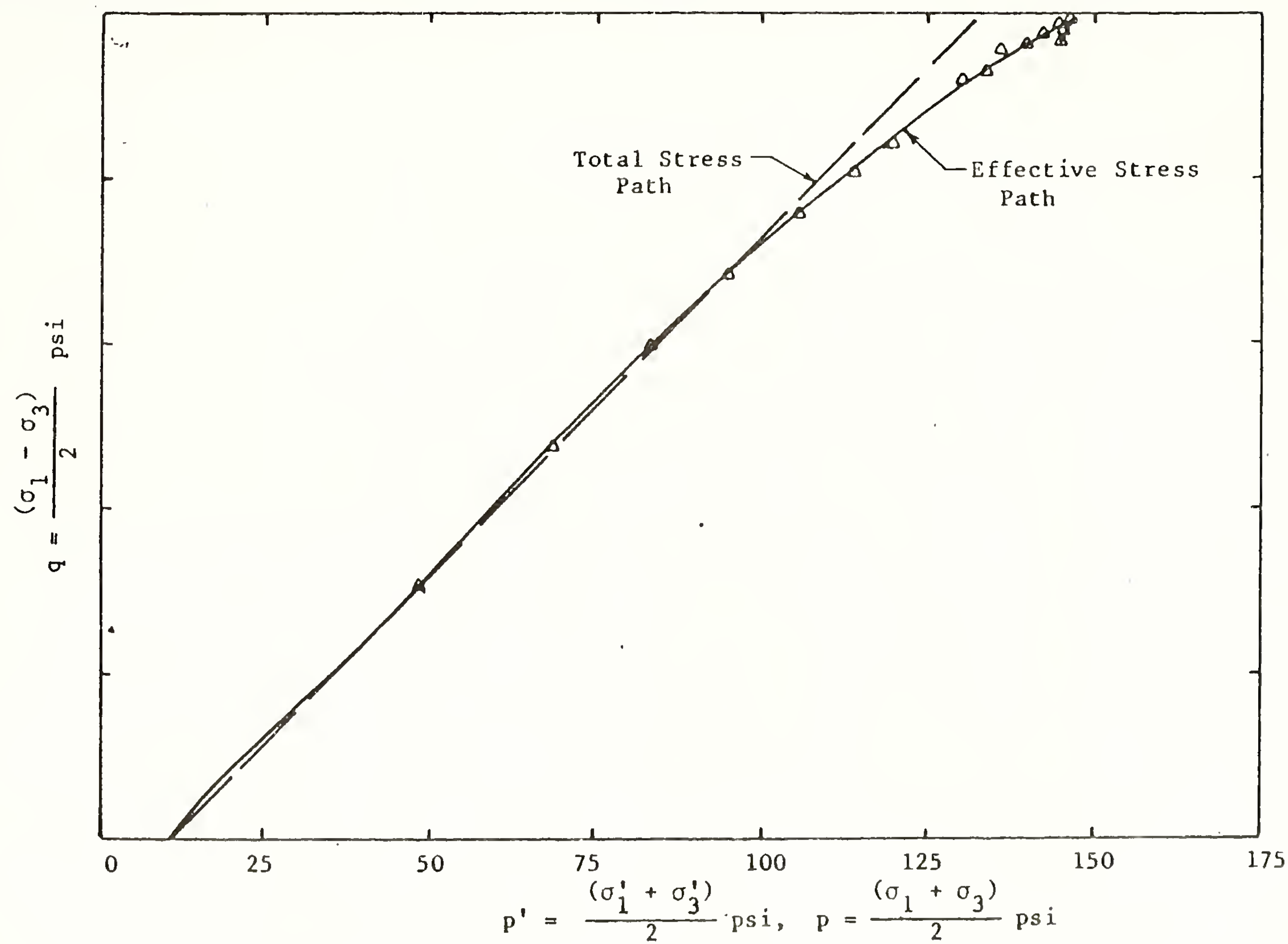


Figure A.13: Typical Deviator Stress, Pore Pressure Change and A Parameter vs. Strain and Stress Path for CIU Triaxial Test

PROJECT	Example Test	SHALE	Osgood	SHEET NO.	12
DESCRIPTION	Soft, gray shale which breaks into massive chunks				
LOCATION	Unknown	SAMPLE NO.	1	DEPTH	Unknown
SOURCE	Test Pit	TESTED BY	MWO	DATE	9/81



d) Stress Path

Figure A.13: Continued

TABLE A.18: Typical Final Conditions and Summary of Design Parameters for CIU Test

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT	Example Test	SHEET NO.	13
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

FINAL CONDITIONS AND SUMMARY

WATER CONTENT	
Container No.	-
Wt. of Container and Wet Shale, g	-
Wt. of Container and Dry Shale, g	-
Wt. of Water, g	-
Wt. of Container, g	-
Wt. of Dry Shale, g	-
Water Content, %	-

SUMMARY	
Kneading Compactor Foot Pressure, kPa	675
As-Compacted Water Content, %	10.0
As-Compacted Void Ratio	0.3043
Consolidation Pressure, kPa	68.95
Percent Volume Change During Consolidation	0
Percent Volume Change During Saturation	0
Percent Strain at Failure	13.6
Deviator Stress at Failure	1710
Change in Pore Pressure at Failure, psi	-12.5
A Parameter at Failure	-0.0480
Final Degree of Saturation, %	100

TABLE A.19: Typical Results for a Series of $\overline{\text{CIU}}$ Triaxial Tests

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT	Example Test	SHEET NO.	14
SHALE	Osgood	DATE	9/81
DESCRIPTION	Soft, gray shale which breaks into massive chunks		
LOCATION	Unknown	SAMPLE NO.	1
SOURCE	Test Pit	DEPTH	Unknown

SUMMARY OF RESULTS OF SERIES OF TESTS

INITIAL CONDITIONS	
Kneading Compactor Foot Pressure, kPa	675
As-Compacted Water Content, %	10.0
As-Compacted Void Ratio	0.3043
Degree of Saturation, %	88.71
STRENGTH PARAMETERS	
c'	-
ϕ'	-

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST

PROJECT	Research Report by Abeyesekera	SHALE	Osgood	SHEET NO.	15
DESCRIPTION	Soft, gray shale which breaks into massive chunks				
LOCATION	Unknown	SAMPLE NO.	Unknown	DEPTH	Unknown
SOURCE	Test Pit	TESTED BY	Abeyesekera	DATE	Unknown

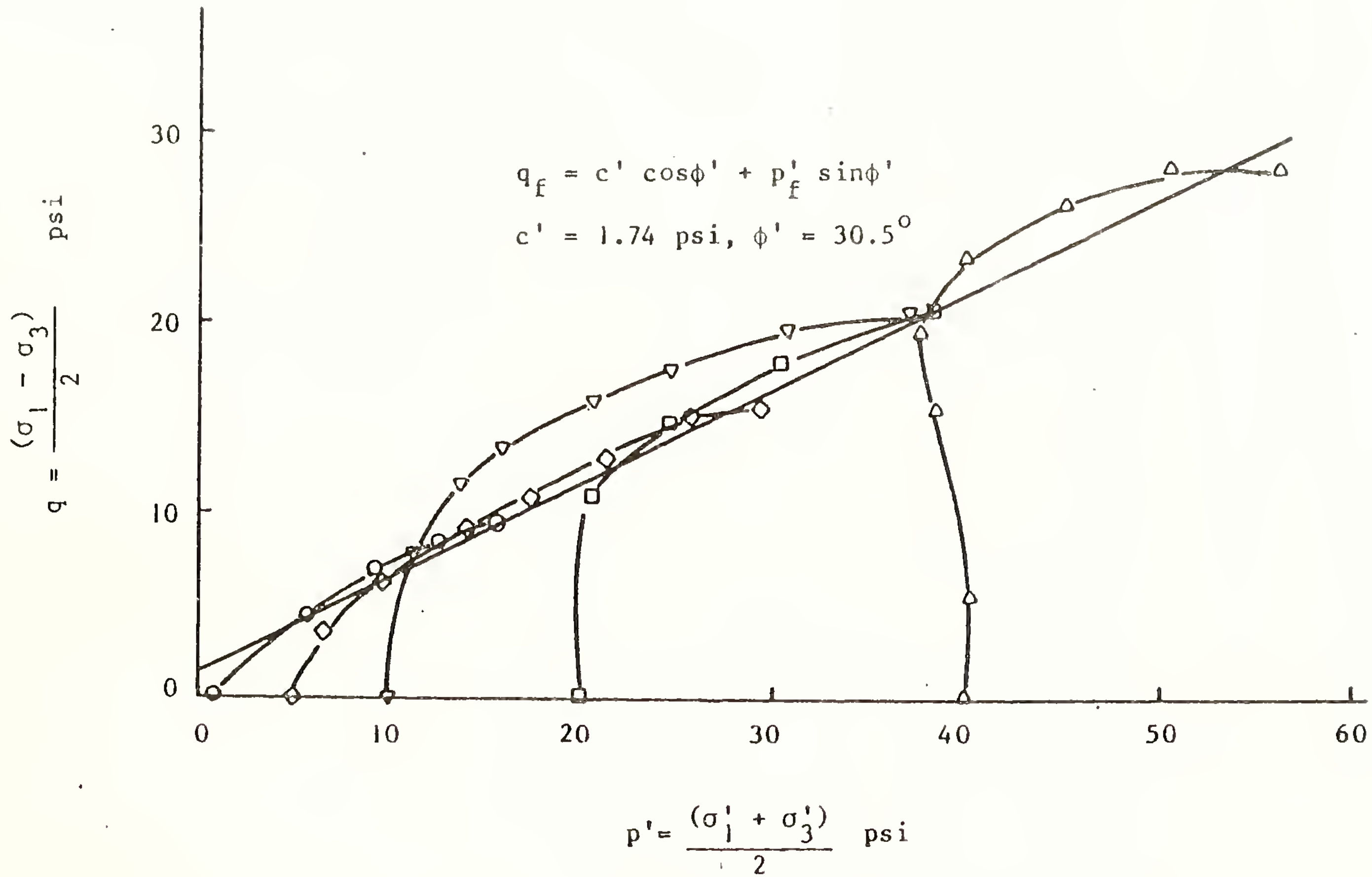


Figure A.14: Typical Design Parameters Obtained from a Series of $\overline{CIU}$ Triaxial Tests

Table A.20 List of Negative Numbers for the Photographs

FIGURE	NEGATIVE NUMBER
10	71650-15
11	71650-16

COVER DESIGN BY ALDO GIORGINI